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EVALUATION OF LONG-LIFE CONCRETE PAVEMENT PRACTICES FOR USE IN FLORIDA

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DISCLAIMER

Prepared in cooperation with the State of Florida Department of Transportation and the U.S. Department of Transportation.

SI (MODERN METRIC) CONVERSION FACTORS (from FHWA)

APPROXIMATE CONVERSIONS TO SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
kip	kilo poundforce	4.45	kilo newtons	kN
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS TO SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
AREA				
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
TEMPERATURE (exact degrees)				
^oC	Celsius	1.8C+32	Fahrenheit	^oF

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
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16. Abstract

The Mechanistic-Empirical Pavement Design Guide (MEPDG) Version 1.1 model was used to analyze the performance of three typical concrete pavement designs in Florida to evaluate their suitability for use as long-life concrete pavements and the effects of various design parameters on their performance. Concrete slab thickness, concrete flexural strength, and the aggregate used in the concrete were found to be the three most significant factors affecting the predicted performance of the pavement evaluated. The type of base material and the stiffness of the base material appear to have no significant effect on the predicted performance.

The Long-Term Pavement Performance (LTPP) database was used to evaluate the effects of various factors on the performance of Jointed Plain Concrete Pavements (JPCP). Critical stress analysis was also performed on the selected LTPP JPCP sections to determine the maximum stress in the concrete slab under a critical load and temperature condition. The computed critical stress-to-strength ratio was found to be the most significant parameter which can be related to the performance of the LTPP pavements. A lower stress-to-strength ratio is related to better observed pavement performance. Results from the critical stress analysis show that the most significant factors affecting the stress-to-strength ratios are the concrete slab thickness and the concrete elastic modulus, modulus of rupture, and coefficient of thermal expansion. Variations in the base and subbase properties were found to have minimal effects on the stress-to-strength ratios for concrete slab thickness of 11 inches or higher.

From the results of this study, the three typical Florida concrete pavement designs are shown to be suitable for use as long-life pavements if the slab thickness was adequate and the concrete had low elastic modulus, low coefficient of thermal expansion, and adequate flexural strength. A concrete slab thickness of 13 or 14 inches is recommended. In addition to meeting the present FDOT specification requirements for these three designs, the concrete mixture must be designed and evaluated according to the procedure recommended in the project report.

17. Key Words

Long-life concrete pavement, MEPDG, critical stress analysis, stress-to-strength ratio, LTPP, concrete slab thickness, modulus of rupture, life cycle cost analysis, elastic modulus, coefficient of thermal expansion.

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EXECUTIVE SUMMARY

Background and Research Needs

The initial design for new construction for both asphalt and concrete pavements in Florida is 20 years. While the rehabilitation period for asphalt pavements varies from 8 to 20 years, that of concrete pavements varies from 20 to 25 years. With the increase in traffic volume on our roadways today and the cost associated with traffic delay due to the needed rehabilitation work, it makes good sense for FDOT to consider the alternative of designing and constructing pavements with a design life of 50 years or more for Florida. Research is needed to determine the best construction practices and design features for long-life concrete pavements with service life of over 50 years, which are most suitable for Florida's climate and conditions.

Evaluation of Concrete Pavement Designs Using MEPDG Model

The MEPDG (Mechanistic-Empirical Pavement Design Guide) model which has been calibrated for the Florida conditions was used to analyze the performance of three typical concrete pavement designs in Florida to evaluate their suitability for use as long-life concrete pavements and the effects of various design parameters on their performance. These three designs are referred to as Types I-A, I-B, and II in this report. Type I-A pavement has a 4-inch treated permeable base over a 2-inch asphalt structural course. Type I-B pavement has a 4-inch asphalt concrete base. Both Type I-A and I-B pavements have a 12-inch Type B stabilized subgrade with a minimum Limerock Bearing Ratio (LBR) of 40. Type II pavement has a 6-inch special stabilized subbase/base over 54-inch A-3 soil. The special stabilized subbase is made up of 3 inches of #57 or #89 coarse aggregate mixed into the top 6 inches of A-3 soil and is used as a working platform during construction.

Concrete slab thickness, concrete flexural strength, and the aggregate used in the concrete were found to be the three most significant factors affecting the predicted performance of the pavement evaluated. The three aggregates used in the analysis included Brooksville limestone, Calera limestone and river gravel. For concrete with the same design flexural strength, Brooksville limestone was shown to give the best predicted performance, followed by Calera limestone and river gravel. The better predicted performance was due to the relatively low elastic modulus and low coefficient of thermal expansion of the concrete made with Brooksville limestone. When the same Brooksville aggregate was used in the concrete, increasing the modulus of rupture of the concrete gave improved predicted performance and increased service life to the pavement.

MEPDG analyses were performed to evaluate the effects of (1) types of base material, (2) stiffness of the base material, (3) erodibility of the base material, and (4) friction between the concrete and base layer on the predicted performance of these three concrete pavement designs used in Florida. The predicted performance of the pavement appeared to have improved slightly with an increase in base thickness. However, the type of base material and the stiffness of the base material appeared to have no significant effect on the predicted performance. Using different erodibility factor and friction factor for the base materials appeared to have no significant effect on the predicted performance according to the results of the MEPDG analyses.

MEPDG analyses were performed to determine the predicted service lives of these three concrete pavement designs. An initial Average Annual Daily Truck Traffic (AADTT) of 17,000, which represents high-volume truck traffic, was used in the analyses. The concrete made with Brooksville aggregate and with a modulus of rupture of 650 psi was used. The predicted service lives for these three designs with pavement thickness varying from 10 to 14 inches are presented in Table 3-44 in the report. When the concrete slab thickness is 13 inches or more, the expected service of all three designs are 50 years or more. Among the three designs, Type II has the best predicted performance, followed by Type I-A and then Type I-B.

Drainage Evaluation

Sensitivity analysis was performed using the DRIP (Drainage Requirements in Pavements) 2.0 program to evaluate the drainage characteristics of Type I-A and Type II concrete pavement designs using the steady flow method and the time-to-drain method. In comparison, the Type II design with a 6-inch permeable base shows better drainage characteristics than the Type I-A design with a 4-inch permeable base. For pavements with 2 to 4 lanes and pavement cross-slope from 2% to 6%, the required base permeability is from 200 to 700 ft/day for Type I-A design, and is from 100 to 500 ft/day for Type II design. If the same base permeability, pavement slope and number of lanes are used in both designs, Type II design has a lower time to drain than the Type I-A design.

Type I-B concrete pavement has a 4-inch asphalt concrete base layer. Since the asphalt concrete layer is a non-permeable layer, the steady-flow analysis and the time-to-drain analysis could not be appropriately performed on this type of pavement. In order for this type of pavement to not have serious issues with drainage, the following conditions must exist throughout the life of the pavement:

(1) The concrete layer remains well bonded to the asphalt concrete layer so that little water would go between these two layers.

(2) The pavement has good surface drainage characteristics, so that most of the water would run off the surface instead of seeping through the concrete into the asphalt concrete layer.

(3) The asphalt concrete is resistant to stripping action, even if some water gets between the concrete and the asphalt layer.

Long-term monitoring of Type I-B pavement is needed to determine whether or not there will be drainage related issue with the use of asphalt concrete base in concrete pavement in Florida.

Analysis of LTPP Data and Critical Stress Analysis

The Long-Term Pavement Performance (LTPP) database was used to evaluate the effects of various factors, which included environmental conditions, drainage types, concrete properties, base types, subgrade types and concrete slab length, on performance of Jointed Plain Concrete Pavements (JPCP) in the U.S. with emphasis on Florida and its neighboring states. Critical stress analysis was also performed, using the FEACONS program, on the selected LTPP JPCP sections to determine the maximum stress in the concrete slab under a critical load and temperature condition. The maximum computed critical stress for each condition was divided by the modulus of rupture of the concrete to determine the stress-to-strength ratio.

The computed critical stress to strength ratio was found to be the most significant parameter which can be related to the performance of the LTPP pavements. A lower stress to strength ratio is related to better observed pavement performance. The better performing pavements were noted to have a computed stress to strength ratio of less than 0.70.

Critical stress analysis was also performed on the three Florida concrete pavement designs. Results from the critical stress analysis show that the most significant factors affecting the stress-to-strength ratios are the concrete slab thickness and the concrete properties, which include the elastic modulus, modulus of rupture, and coefficient of thermal expansion. Variations in the base and subbase properties were found to have minimal effects on the stress-to-strength ratios for concrete slab thickness of 11 inches or higher. This observed results agree well with the findings from the MEPDG analysis that the most significant factors affecting the performance of the concrete pavement are the concrete slab thickness and the concrete properties.

Similar to the results from the MEPDG analysis, when the same aggregate is used in the concrete, increasing the flexural strength of the concrete will result in better predicted pavement performance.

Life Cycle Cost Analysis

The cost estimates for Type I-A, Type I-B, and Type II pavements with concrete slab varying from 10 inches to 14 inches were developed. The predicted service lives of these pavements were based on the results of MEPDG analysis using a concrete made with Brooksville aggregate and modulus of rupture of 650 psi. The estimated total costs, predicted service lives, and annual costs for these three designs are shown in Tables 7-11, 7-12, and 7-13 in the report. Type II design has the lowest cost estimate, which is slightly less than that for Type I-A design, while Type I-B design has the highest cost estimate.

When cost of interest was not considered, the most cost-effective slab thickness for all three designs was 14 inches. With concrete slab thickness of 14 inches, the expected service for Type I-A, I-B, and II designs are 56, 53, and 60 years, respectively. When an interest rate of 3.5% was considered, the most cost-effective slab thickness for all three designs was 13 inches. With concrete slab thickness of 13 inches, the expected service for Type I-A, I-B, and II designs are 51, 50, and 56 years, respectively.

Recommended Long-Life Concrete Pavement Designs for Florida

From the results in this study, it appears that the three typical Florida concrete pavement designs evaluated in this study can be used as long-life pavements if the slab thickness is adequate and the concrete has low elastic modulus, low coefficient of thermal expansion and adequate flexural strength. Among the three designs evaluated, Type II pavement has the best predicted performance from the MEPDG analysis and the best drainage characteristics from the results of the drainage evaluation using the steady flow method and the time-to-drain method. Type II pavement also has the lowest cost estimate.

Type II design is recommended as the preferred design for use as long-life concrete pavements in Florida. However, if the special select A-3 soil is not available, Type I-A and Type I-B can also be used. A concrete slab thickness of 13 or 14 inches is recommended to be used. When 14 inches is used, the top 0.5 to 1 inch can be considered as sacrificial concrete for future grinding during the life of the pavement to restore ride quality, texture and remediate faulting. The present FDOT construction specifications for these three types of design are to be followed. In addition to meeting the present FDOT specification requirements for these three designs, the concrete mixture to be used must be designed and evaluated by the following procedure:

(1) Design the concrete mix to give a flexural strength of at least 650 psi at 28 days. Use an aggregate which has a past history of producing concrete of low elastic modulus and low coefficient of thermal expansion.

(2) Measure the flexural strength, elastic modulus and coefficient of the designed concrete mix at 28 days.

(3) Perform MEPDG analysis to evaluate the predicted performance of the designed pavement for a design life of 50 years, using the measured concrete flexural strength, elastic modulus and coefficient of thermal expansion as input properties for the concrete. If the predicted life of the pavement is at least 50 years, the concrete mix would be acceptable for the project. If the predicted life is less than 50 years, a new concrete mix can be designed by either specifying a higher flexural strength or using a different aggregate. Steps 1 through 3 would be repeated until an acceptable concrete mix for the project is obtained.

It is recommended that FDOT establish a database of concrete mix designs which are acceptable for use in long-life concrete pavements so that optimum concrete mixes can be designed and selected efficiently for this purpose. A research study to evaluate the drainage requirements for these concrete pavement design is also recommended.

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INTRODUCTION

1.1 Background and Research Needs

In Florida, the initial design for new construction for both asphalt and concrete pavements in Florida is 20 years. While the rehabilitation period for asphalt pavements varies from 8 to 20 years, that of concrete pavements varies from 20 to 25 years. With the increase in traffic volume on our roadways today and the cost associated with traffic delay due to the needed rehabilitation work, it makes good sense for FDOT to consider the alternative of designing and constructing pavements with a design life of 50 years or more for Florida.

The concept of designing for long-life pavements is not new. Many concrete pavements in the United States, Canada and many European countries have shown excellent service life of over 50 years, and information about their design features and performance is available in the technical literature. However, as the performance of a pavement is affected not only by its design but also by its local conditions such as weather, soil, topography and available materials, the designs which have worked well in other regions may not be applied directly to Florida without evaluation and suitable adjustments. Research is needed to determine the best construction practices and design features for long-life concrete pavements, with service life of over 50 years, which are most suitable for Florida's climate and conditions.

1.2 Objectives of Study

The main objectives of the research are:

(1) To evaluate the long-life concrete pavement designs which have been used successfully in other states and countries and how they may be applied to Florida conditions.

(2) To develop several designs for long-life concrete pavements, with expected service life of over 50 years, which are suitable for use in Florida.

(3) To recommend courses of action for FDOT in the area of long-life concrete pavement designs.

1.3 Research Approach

The main objectives of the research were achieved through the following tasks:

1. Literature review was conducted on characteristics of long-life concrete pavement.
2. Data on characteristics and performance of jointed plain concrete pavements in the LTPP (Long-Term Performance Pavement) data were analyzed to determine the factors affecting performance of concrete pavements.
3. Three typical concrete pavement designs used in Florida were evaluated using MEPDG (Mechanistic and Empirical Pavement Design Guide) to see if they could be used as long-life concrete pavements.
4. MEPDG results were investigated to determine if it is feasible to use the three designs evaluated for long-life pavement. If it is feasible, what are the required slab thickness and concrete properties?
5. The drainage characteristics of the three Florida designs were evaluated using the DRIP 2.0 software.
6. Recommended long-life concrete pavement designs for Florida were developed.

CHAPTER 2
LITERATURE REVIEW ON LONG-LIFE CONCRETE PAVEMENTS

2.1 Definition of Long-Life Concrete Pavement

The Federal Highway Administration (FHWA) Concrete Pavement Road Map team has proposed the following definitions for Long-Life Concrete Pavement (LLCP) (Ferragut et al., 2005):

- “A ‘no-fix-required’ pavement that would last 50 to 60 years with relatively heavy loads throughout its life”
- “Planned maintenance between 10 and 30 years, followed by heavy joint repair and possibly an overlay to take the total pavement life to 60 years”
- “A mandatory strong foundation with a thinner slab designed for 20 years of service, followed by the construction of a wraparound slab that would provide service for an additional 30 to 40 years” (A wraparound overlay covers both the top and the sides of the existing slab.)

Tayabji and Lim gave a summarization of the definition of long-life concrete pavement in the U.S. at the October 2006 International Conference on Long-Life Concrete Pavements as follows (Tayabji et al., 2006):

- “Original concrete service life is 40+ years”
- “Pavement will not exhibit premature construction and materials-related distress”
- “Pavement will have reduced potential for cracking, faulting, and spalling.”
- “Pavement will maintain desirable ride and surface texture characteristics with minimal intervention activities, if warranted, or ride and texture, joint resealing, and minor repairs”

2.2 Long-Life Concrete Pavement Design Practices by Illinois DOT

Illinois Department of Transportation (IDOT) began researching for longer life concrete pavement in the late 1990's by implementing the following activities (Winkelman, 2006):

1. Accelerated testing to determine optimal structural design features for CRCP (collaborating with the University of Illinois).
2. Refining structure design features, concrete material requirements and construction process pertaining to longer life design.
3. Constructing several projects to determine feasibility of longer life pavements.

As a result, IDOT concrete pavement requirements were changed to include aggregate and construction requirements that achieve LLCP. More specifically, construction specifications were altered to include rigorous concrete material requirements to prevent freeze/thaw in harsh conditions. Summary of requirement changes are highlighted in Table 2-1 (Winkelman, 2006).

Table 2-1. Changes in Illinois Specifications to Achieve Long-life Concrete Pavements. (Winkelman, 2006)

Item	Long-Life Pavement Specification
Thickness design	– Jointed Plain Concrete Pavement (JPCP): IDOT developed mechanistic-empirical design procedure – Continuously Reinforced Concrete Pavement (CRCP): IDOT-modified AASHTO process. – Design Life: 30 to 40 years – Typical pavement thickness: o 250 mm (10 in) for JPCP o 350 mm (14 in) for CRCP
Typical structure	– Up to 350-mm (14 in) CRCP slab. – 100 to 150 mm (4 to 6 in) stabilized base (hot-mix asphalt stabilized base for CRCP) – 300 mm (12 in.) well-graded aggregate subbase (top 75 mm [3 in] maximize size of 40 mm [1.5 in.]; bottom 230 mm [9 in.] maximum size of 200 mm [8 in.] aggregate) – Compacted subgrade
Tie bars	– Use of tie bars at centerline and at lane-to-shoulder longitudinal joints. – Use of 23 mm (1 in.) (#8) steel bars, 750 mm (30 in.) long, spaced at 600 mm (24 in.).
Aggregate Requirements	– Freeze thaw expansion (using IDOT-modified ASTM C666) o 0.0040% for 30 year design and 0.025% for 40 year design (in the past, 0.060%) – Alkali-silica reactivity (ASR) susceptibility (by ASTM C 1260) (applies only for 40-year designs): o If the expansion is greater than 0.15 limit the equivalent alkalis of the cement source to not greater than 0.6%. When fly ash is used, the available alkali as Na_20 shall be a maximum of 1.5% for the fly ash source. o If any blended cement is used, the mortar expansion at 14 days and 8 weeks shall be a maximum of 0.02% and 0.06% respectively.
Construction requirements	– Concrete mixture temperature: 10 to 32° C (50 to 90° F). If the temperature exceeds 32° C (90° F), concrete production will cease until appropriate corrective action is taken. – Slipform paving machine is required to be equipped with internal vibration and vibration monitoring device. – Curing: Type II (white-pigmented) curing compound must be applied to the pavement surface and edge faces within 10 minutes of concrete finishing and tining. – At least 7 days of curing are required before opening the pavement to any construction or regular traffic.
Construction quality	– Surface texture provisions for tining (for safety and low tire-pavement noise): o Use of variable spacing between 17 and 54 mm (0.7 and 2.15 in.). o Use of 10 degree skewed tining (for the sections with speed limit of 90 km/h [55 mi/h] or higher. o Use of perpendicular tining (for the sections with lower speed limits). – Surface profile: Profile Index (PI) using California Profilograph (0-in. blanking band). o Grinding is required if the average PI value is above 760 mm/km (48in/mi) for major highways. – Pavement warranty: covers pavement distress up to 5 years on demonstration projects only. IDOT currently does not use warranties.

2.3 Long-Life Concrete Pavement Design Practices by Minnesota DOT

The Minnesota DOT has adopted Long-Life Concrete Pavement (LLCP) designs for high-volume, urban highways since 2000. The current design features and construction specifications for the LLCP are presented in Table 2-2 (Burnham et al., 2006).

Table 2-2. Minnesota Specifications for Long-Life Concrete Pavements (Burnham et al., 2006)

Item	Present Standard
Design Life	– Design Long-Life concrete pavements (LLCP) for 60 years.
Cross section	– Jointed Plain Concrete Pavement (JPCP) slab thickness: 290 to 340 mm (11.5 to 13.5 in.), depending on truck traffic. – Base: 75 mm to 200 mm (3 to 8in.) dense-graded granular base (MnDOT CL-5 material) or 125 mm (5 in.) open- graded aggregate base on top of 100 mm (4 in.) CL-5 – Subbase: 300 to 1200 mm (12 to 48 in.) select granular (frost-resistant) subbase.
Joint Design	– Joint spacing: 4.6 m (15 ft.). – All transverse joints are doweled.
Dowel bar	– Diameter: 38 to 45 mm (38 mm typical) (1.50 to 1.75 in. [1.50 in. typical]). – Length: 380 to 450 mm (380 mm typical)(15 to 18 in. [15 in. typical]). – Spacing: 300 mm (12 in.). – Bar material: must be corrosion-resistant (stainless steel solid, clad pipe, or tube; plastic-coated steel; zinc-clad steel).
Surface texture	– Astroturf or broom drag. – Requires 1 mm (0.04 in.) average depth in sand patch test (ASTM E 965). Note: Transverse tining is not used due to noise concerns.
Alkali-silica reactivity (ASR)	– Fine aggregates require tests for ASR potential by ASTM C1260. – Expansion to be 0.15% or less. Reject if the expansion is greater than 0.3% – Mitigation is required by using Granulated Ground Blast Furnace Slag (GGBFS) or class C fly ash when the expansion is between 0.15 and 0.30% o 0.15 to 0.25%: GGBFS 35% or fly ash 20% o 0.25 to 0.30%: GGBFS 35% or fly ash 30%
Aggregate gradation	– Combined gradation based on 8-to-18 specification: percentage retained in all specified sleeves should be between 8% and 18%, except finer than no. 30, and the coarsest sieve.
Concrete permeability	– Use of supplementary cementitious materials (GGBFS or class C fly ash) is required to lower the permeability of concrete. – Specification requires rapid chlorine ion permeability test value of 2500 coulomb or less at 28 days, by compounds at later ages.
Air content	– LLCP concrete mixture: 0.7 ± 1.5%. o Increased air content for possible loss of entrained air due to over-vibration or in-filling with secondary compounds at later ages.
Water-to-cementitious materials (w/cm)	– 0.40 or less.
Curing	– A poly-alpha-methylstyrene membrane cure is used under normal weather conditions. – No construction or general public traffic is allowed for 7 days or until the flexural strength of concrete reaches 2.4 MPa (350 lb/in^2).
Construction quality	– Requires monitoring the vibrators during paving. Paver track speed and vibration operating frequencies must be reported daily. – Initial Profile Index values, using 5-mm (0.2 in.) blanking band, greater than 126 mm/km (8 in./mi) require corrective action, generally diamond grinding.

2.4 Long-Life Concrete Pavement Design Practices by Texas DOT

The Texas DOT uses Continuously Reinforced Concrete Pavement (CRCP) as the primary LLCP. The current standards for LLCP used by Texas DOT are presented in Table 2-3 (Won et al. 2006).

Table 2-3. Texas Standards for Long-Life Concrete Pavements (Won et al., 2006)

Items	Present Standards
Design Life	30 years for Continuously Reinforced Concrete Pavements (CRCP)
Thickness	– Use of AASHTO 1989 pavement design guide – Use of reliability value of 95%. – Continuously Reinforced Concrete Pavement design (CRCP): o Minimum slab thickness studied: 203 mm (8 in.).
Stabilized bases	– Two types are used: o 150 mm (6 in.) cement-stabilized base with 25-mm (1 in.) asphalt bond breaker layer on top. o 100 mm (4 in.) asphalt-stabilized base.
Longitudinal steel design	– Use of higher steel content: generally results in more cracks but at shorter spacing and are tight. – Requires staggering splices: to avoid weak spots (less than 1/3 of the splices within a 0.6 m (2 ft.) length of each lane of the pavement).
Percent Steel	– Satisfactory performance between 0.6 and 0.8% steel.
Coefficient of Thermal Expansion (CTE)	– Limits the CTE of concrete to 10.7 microstrain per °C (6.0 microstrain per °F).
Construction Joint	– Past practice for placing additional rebars of same size in a line caused weak spots at the end of the rebars. Revised design details so that the ends of rebars will stagger.
Smoothness	– Smoothness based on IRI o Testing device: High Speed or Lightweight Inertial Profiler o Incentive for IRI < 60 in. /mi o Disincentive for IRI > 65 in./mi o Corrective action for IRI > 95 in./mi

2.5 Long-Life Concrete Pavement Design Practices by Washington State DOT

While the concrete pavements in Washington State were originally designed for 20 years design life, about 38 percent of concrete pavements in the state were over 35 years old with little or no maintenance or rehabilitation as of 2006 (Muench et al. 2006). Based on the experience on concrete pavement performance over the past 40 years, Washington State DOT has modified the design practices for concrete pavements to achieve a design life of 50 years. The modifications of design practices for LLCP in Washington State are summarized in Table 2-4.

Table 2-4. Modifications of Washington State Practices for Long-Life Concrete Pavements (Muench et al., 2006)

Item	Present Standards
Design Life	– Increased to 50 years
Thickness Design	– Typical: 305 mm (12 in.) Portland Cement Concrete (PCC) over 60 to 100 mm (2.4 to 4.0 in.) dense graded Hot-Mix Asphalt (HMA) base over 60 to 100 mm (2.4 to 4.0 in.) crushed stone subbase (Top 25 mm [1 in.] of PCC is considered as sacrificial for future grinding to restore profile and texture) – Minimum slab thickness: 200 mm (8 in.) – Design basis: 1993 AASHTO Guide for the Design of Pavements
Base Materials	– For high-volume truck routes, requires 100 mm (4 in.) dense-graded HMA base on aggregate subbase to limit base deflection, pumping, and joint faulting. – Asphalt-treated base: minimized use due to its potential for stripping – Cement-treated base: not allowed due to increased potential for slab cracking and higher risks of pumping.
Joint Design	– 4.6-m (15 ft.) spacing – Requires dowel bars – Saw cut width: 5 to 8 mm (0.2 to 0.3 in.) single cut. – Joint Sealant: hot-poured sealant. – Tie bars: No. 5 bars, 750 mm (30 in.) long, 900-mm (36 in.) spacing.
Dowel Bars	– Dowel bar types (depending on the risk of corrosion): o Stainless steel: stainless steel clad, stainless steel sleeves with an epoxy coated insert, MMFX2 steel bars. o Zinc-clad steel bars o Epoxy-coated: traditional black steel bar with epoxy coating (ASTM A 943) o Bar dimension: 38 mm (1.5 in.) diameter, 450 mm (18 in.) length, 300 mm (12 in.) spacing o 8 dowels for non-truck and high-occupancy vehicle (HOV) lanes (4 dowels in each wheel path) and 12 dowels for truck lanes.
Outside Shoulder	– 4.3 m wide slab (14 ft.) with tied PCC of HMA shoulder – 3.7 m wide slab (12 ft.) with tied and doweled PCC shoulder
Mix Design	– Use of combined aggregate gradation with maximum size of 20 mm (0.8 in.) – Contractor developed concrete mixtures – Use of Class F fly ash: max 35% by weight of total cementitious materials – Use of Granulated Ground Blast Furnace Slag (GGBFS) and blended cements.
Concrete Quality	– Traffic opening compressive strength: 17 MPa (2500 lb/in^2) by cylinder test or maturity method.
Surface Texture	– Transverse tining: 3.2 to 4.8 mm (0.13 to 0.19 in.) tine depth and width, 12.5 to 32.0 mm (0.50 to 1.25 in.) variable spacing.
Studded tire mitigation	– Research to minimize studded tire wear and mitigate its effect is ongoing. Features include: combined aggregate gradation, higher flexural strength, use of higher cement and slag contents, and use of paste-hardening additives.
Smoothness	– Specification testing device: California Profilograph – Smoothness index: Profilograph Index (PrI) – Blanking band: 0.2 in.

2.6 Other Examples of Long-Life Concrete Pavements

2.6.1 Highway 41, Mölltal, Austria (Pichler, 2006)

Highway 41 in Mölltal, Austria is an example of a LLCP which has been in service since 1956. Its concrete slab structure is shown in Figure 2-1. It can be characterized as a well-constructed pavement. The information on this pavement is summarized as follows:

General Information

Type of pavement	JPCP
Design life of the pavement	30 yrs
Year of construction/ Traffic Opening Date	1956

Present Condition

Pavement is generally in good condition.

Some longitudinal cracking and D cracking have been observed.

Traffic Information

Between 3325 and 6140 average daily vehicles (about 5% heavy vehicles)

Subbase / Base

Type of base material	Stabilized Sand
Base thickness	2 in (5 cm)
Subgrade	Frost resistant

Pavement Properties

Slab layer thickness	7.8 in (20 cm)
Joint spacing	26.3–32.8 ft (8–10 m)
Slab width	N/A
Transverse joint angle	Rectangular
Load Transfer system (dowels / interlock)	dowels
Dowel bar dimensions	1X 20” (2.6 X 50 cm)

Concrete Mixture Properties

Air Content	2%

Additional Information:

Paper membrane was used for curing of concrete.

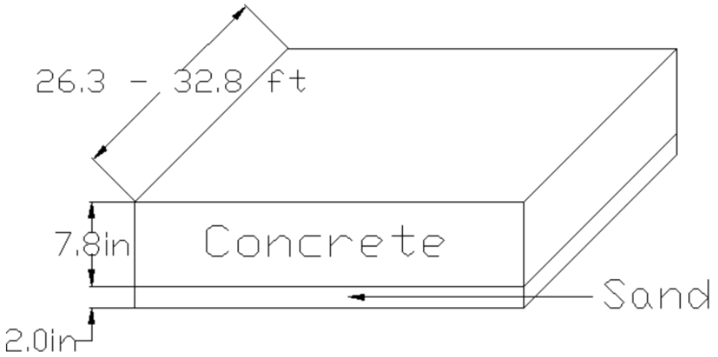


Figure 2-1. Typical slab structure of Highway 41 in Austria.

2.6.2 Superior St – 2nd St, Webster, Iowa (Cable and Ceylan, 2004).

An old concrete pavement on Superior Street, Webster, Iowa is an example of a pavement with a strong base and good concrete quality. Its concrete slab structure is shown in Figure 2-2. The information on this pavement is summarized as follows:

General Information

Type of pavement	JPCP
Design life of the pavement	20 yrs
Year of construction/ Traffic Opening Date	1973

Present Condition

Low percentage of D-cracking was observed.

Traffic Information

Current average daily traffic (ADT): 11700

Subbase / Base

Type of base material	Crushed stone
Base thickness	4 in (10.16 cm)

Pavement Properties

Slab layer thickness	8 in (20.3 cm)
Joint spacing	20 ft (6.1 m)
Slab width	13.25 ft (4.03 m)
Transverse joint angle	Rectangular
Load Transfer system (dowels / interlock)	dowels
Dowel bar dimensions	1.25X 18”
Tie bar spacing	30 in (76.2 cm)
Tie bar dimensions	0.5”X 36”

Concrete Mixture Properties

W/C	0.43 – 0.49
Air Content	6%
Water Content	246 (lb/cy)
Cement	573 (lb/cy)

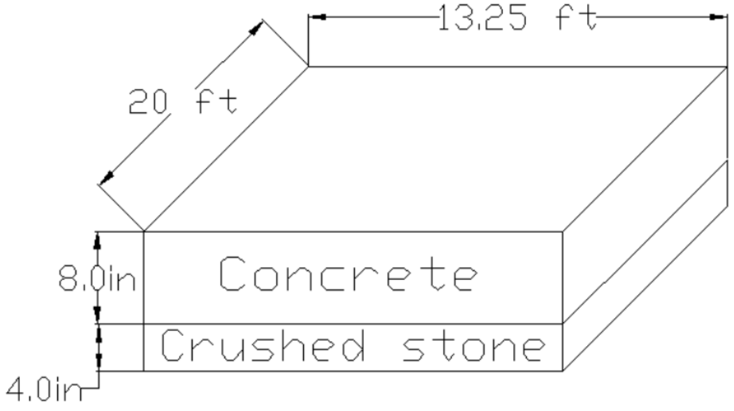


Figure 2-2. Typical slab structure of Superior St. in Iowa.

2.6.3 Highway 427, Toronto, Canada (PIARC, 2009)

Highway 427 in Toronto, Canada is an example of a well-constructed pavement which outperformed its expected design life. Its pavement structure is shown on Figure 2-3. The information on this pavement is summarized as follows:

General Information

Type of pavement	JPCP
Design life of the pavement	30 yrs
Year of construction/ Traffic Opening Date	1968

Present Condition

Experienced some joint stepping, joint failures, joint cracking and distortion. 50% of the pavement section had friction numbers ranging from fair to poor. The remaining 50% of the section was performing in the good to very good range.

Traffic Information

Since the original construction, there were approximately 58 million Equivalent Single Axle Loads (ESALs) in the express lanes before it was rehabilitated. Truck percentage was 12%.

Drainage

Drainage in this section is provided through subdrains and an urban cross-section which includes curb and gutter.

Subbase / Base

Type of base material	Cement treated
Base thickness	6 in (15.2 cm)
Subgrade	Subgrade soil had a Modulus of subgrade reaction 115 pci (31 MPa/m).

Pavement Properties

Slab layer thickness	9 in (22.9 cm)
Pavement joint spacing	20 ft (6.1 m)
Transverse joint angle	Rectangular
Load Transfer system (dowels / interlock)	dowels

Maintenance Performed

After 34 years of operation, maintenance was performed in 2002 to restore the ride quality. Maintenance activities on this highway have included shoulder rehabilitation, diamond grinding to restore pavement friction on certain sections, some areas have machine and manual patching.

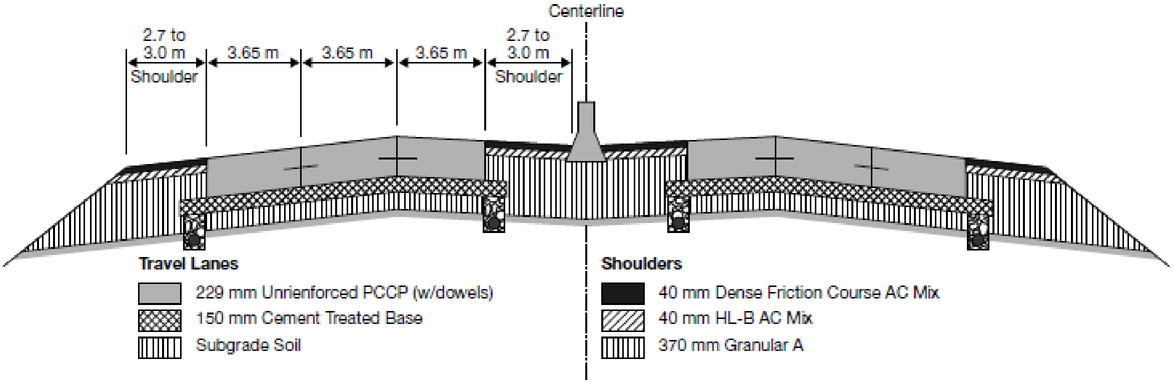


Figure 2-3. Typical slab structure of Highway 427 in Canada.

2.6.4 B47 Highway, Germany (FHWA, 1993)

B47 Highway in Germany is an example of a good performing composite pavement constructed wet-on-wet with good drainage features. Its pavement structure is shown in Figure 2-4. Information on this pavement is summarized as follows:

General Information

Type of pavement	JPCP
Design life of the pavement	30 yrs
Year of construction/ Traffic Opening Date	1965

Present Condition

Small percentage of cracking has been observed

Traffic Information

Greater than 3 million accumulated 10-ton (22-kip) ESAL. Specifically, the average ADT is of 40,000 with 25% heavy trucks. The legal maximum single-axle load was of 10 tons (22,000 lbs) when highway was designed.

Drainage

Consists of a porous concrete layer beneath the shoulder that provides a flow channel to a longitudinal subdrain. This empties at regular intervals into a lateral pipe which goes directly to a longitudinal closed drainage system.

Subbase / Base

Type of base material	Lean Concrete
Base thickness	4 in (10 cm)

Pavement Properties

Slab layer thickness	9 in (22.9 cm)
Slab width	12.3 ft (3.75 m)
Widened Slab into shoulder	1.6 ft (0.48 m)
Shoulder design	Tied concrete
Load Transfer system (dowels / interlock)	Plastic coated dowels
Dowel Spacing	uneven (closer spacing near the wheel paths)

Mixture Properties

Wet-on-wet construction creating good bonding and controlled cracking.

Joints

Notching of the base was immediately performed upon placement—specifically located at the exact position where future transverse and longitudinal joints of the concrete slab are to be sawed (FHWA, 1993). This process was performed to localize the cracks on a specific direction.

Transverse joint angle	Rectangular
Joint spacing	16.4 ft (5 m)
Saw depth	0.25-0.3 of slab thickness
Joint sealant	Compression seal

Additional Information

Surface texture	Light longitudinal brush (Burlap drag)

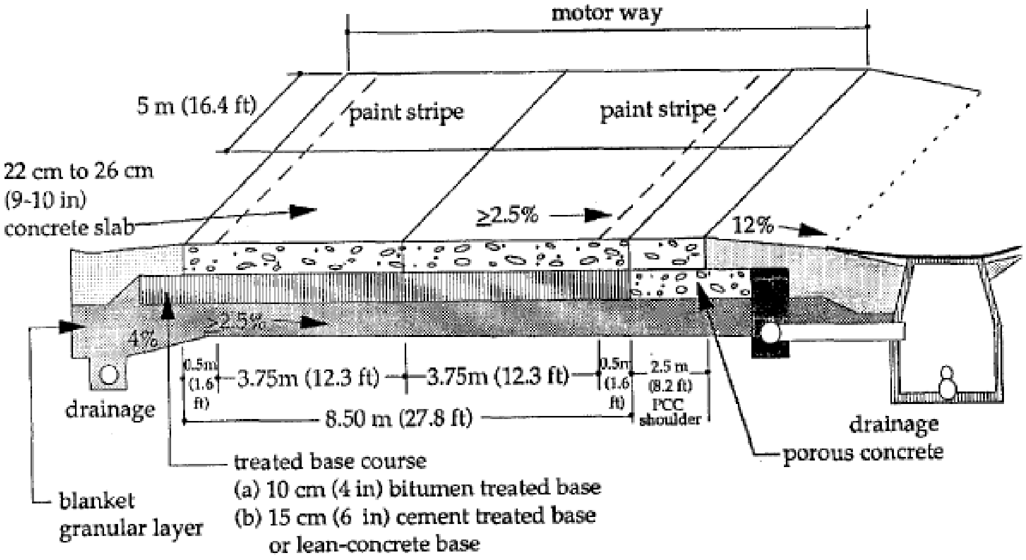


Figure 2-4. **German jointed plain concrete pavement design (FHWA, 1993).**

2.6.5 Avenue de Lorraine, Belgium (Gilles and Jasienski, 2004)

Avenue de Lorraine in Belgium is an example of an old pavement that has performed well due to the additional thickness at the edge, as shown in Figure 2-5. Information on this pavement is summarized as follows:

General Information	
Type of pavement	JPCP
Design life of the pavement	N/A
Year of construction/ Traffic Opening Date	1925
Present Condition	
Pavement was resurfaced in 2003	
Traffic Information	
Information not available	
Drainage	
Information not available	
Subbase / Base	
Type of base material	Crushed stone
Base thickness	N/A
Pavement Properties	
Slab layer thickness	5 in (12 cm) 2 in thicker (5 cm) at edges
Pavement joint spacing	13-16 ft (3.96-4.88m)
Transverse joint angle	Rectangular
Load Transfer system (dowels / interlock)	Aggregate interlock
Maintenance Performed	

Road was rehabilitated using a concrete overlay in 2003 due to rocking of slabs as well as the formation of steps and faulting. Erosion of fine particles was seen and expansion joints were large and uncomfortable.

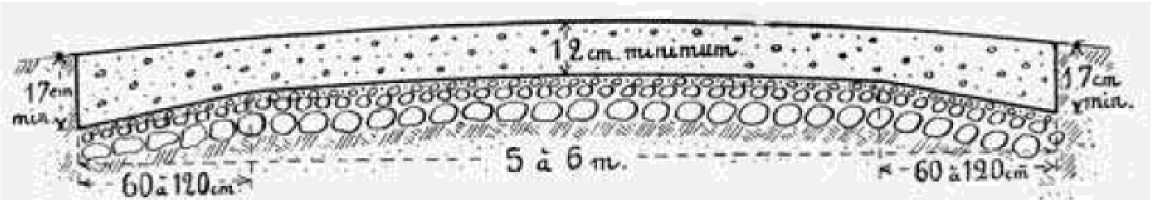


Figure 2-5. Cross-section of the Lorraine Avenue design. (Gilles and Jasienski, 2004)

2.6.6 A1 Highway, Austria (Hall et al., 2007)

A1 Highway in Austria is an example of a good pavement incorporating permeable asphalt base with minimal stud tire damage. The features of this pavement are shown in Figure 2-6. The information on this pavement is summarized as follows:

General Information

Type of pavement	JPCP
Design life of the pavement	30 yrs
Year of construction/ Traffic Opening Date	1970

Present Condition

Studded tire damage was seen due to heavy traffic loading.

Traffic Information

18 to 40 million design axle loads

Drainage

Longitudinal subdrains connected to lateral pipes and a closed drainage system

Subbase / Base

Type of subbase material	Cement stabilized
Subbase thickness	8 in (20 cm)
Type of base material	Permeable Bitumen
Base thickness	2 in (5 cm)

Pavement Properties

Slab layer thickness	10 in (25 cm)
Upper wet-on-wet	1.5 in (4cm)
Lower wet-on-wet	8.5 in (21 cm)
Pavement joint spacing	18-20 ft (5.5 - 6.1 m)
Slab width	≤ 19.7 ft (6 m)
Transverse joint angle	Rectangular
Shoulder design	tied concrete
Load Transfer system (dowels / interlock)	dowels
Dowel bar dimensions	1 X 20” (2.5 X 51 cm)
Dowel Spacing	Spaced closely on wheel paths.

Mixture Properties (wet-on-wet)

- The lower concrete course is 8.3 in (21 cm) thick, made with virgin or recycled—marginal—concrete (1.25 in [32 mm] maximum aggregate size). The upper course is 1.5 in (4 cm) thick and contains smaller aggregate with high wear resistance (FHWA, 1993).
- Compressive strength is specified to be more than 5075 psi (35 MPa) for the lower layer and more than 5800 psi (40 MPa) for the top layer after 28 day curing (FHWA, 1993).
- Flexural strength was more than 708 psi (5.5 MPa) on 4.7- by 4.7- by 14 in beams.

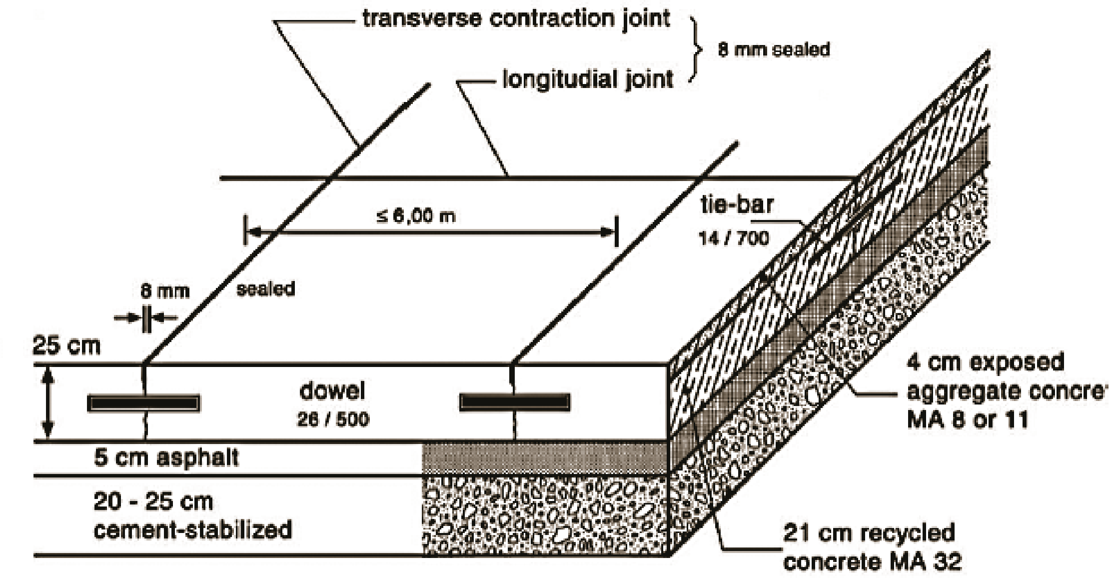


Figure 2-6. **Cross-section of Austrian JPCP construction. (Hall et. al., 2007)**

2.6.7 A6 Freeway, Paris, France (FHWA, 1993)

A6 Freeway in Paris, France is an example of the importance of efficient load transferring systems in long life concrete pavements. The slab design of this pavement is shown in Figure 2-7. The information on this pavement is summarized as follows:

General Information

Type of pavement	JPCP
Design life of the pavement	N/A
Year of construction/ Traffic Opening Date	1978

Present Condition

Slab faulting and stepping has been observed.

Traffic Information

Less than 1500 trucks per day.

Drainage

Geotextile drain placed over subgrade

Subbase / Base

Type of base material	Erosion-resistant lean concrete
Base thickness	6 in (15.2 cm)
Drainage	Longitudinal drainage Along edge joint.

Pavement Properties

Slab layer thickness	9 in (22.9 cm)
Joint spacing	15 (4.5 m)
Slab width	12 ft (3.7 m)
Transverse joint angle	Skewed 1:6 counterclockwise
Load Transfer system (dowels / interlock)	aggregate interlock

Mixture Properties

Air Content	5%
Modulus of Rupture (56 days)	754 psi (5.3 MPa)

Maintenance Performed

- Lack of joint load transfer led to faulting and cracking. Problem was fixed using load transferring devices developed by the LCPC Laboratories with Freyssinet (Seen in Figure 2-8).
- Using the load transfer device showed success in restoring load transfer from less than 50% to over 90% in certain cases (FHWA, 1993).

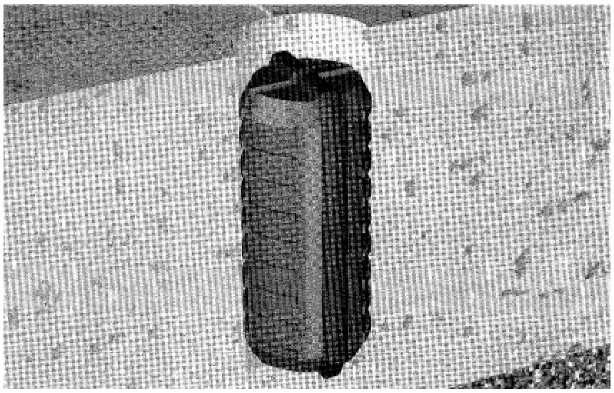


Figure 2-7. LCPC/Freyssinet French load transfer device (FHWA, 1993).

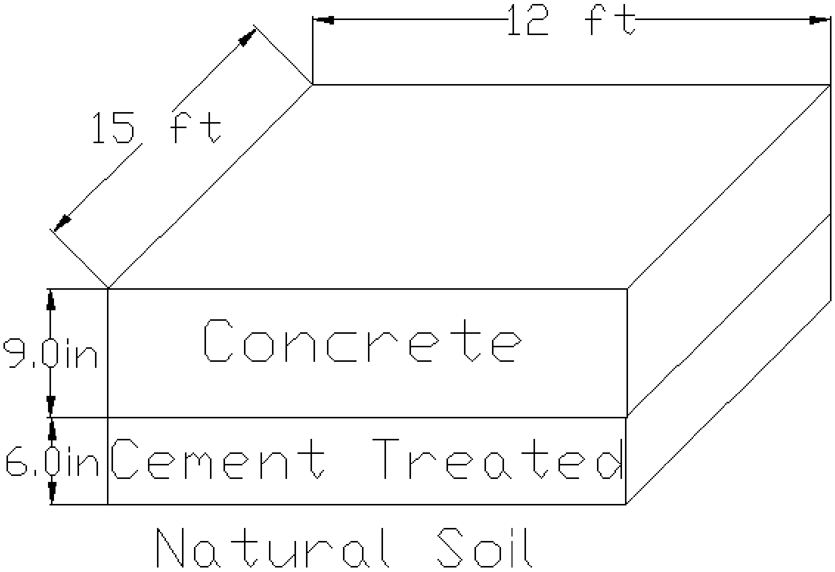


Figure 2-8. Typical slab design in A6 Freeway located in France.

2.6.8 Airport Ring Road, Clay, Iowa (Cable and Ceylan, 2004)

Airport Ring Road in Clay, Iowa, is an example of a well-performing concrete city street, whose slab design is shown in Figure 2-9. The information on this pavement is summarized as follows:

General Information

Type of pavement	JPCP
Design life of the pavement	20 yrs
Year of construction/ Traffic Opening Date	1973

Present Condition

Joints are performing well; good ride quality is observed.

Traffic Information

North 330, East 1550 AADTT, 15% trucks

Subbase / Base

Type of base	Stabilized soil
Base thickness	8 in (20.32 cm)

Pavement Properties

Slab layer thickness	7 in (17.78 cm)
Joint spacing	20 ft (6.1 m)
Slab width	14 ft
Transverse joint angle	Rectangular
Shoulder design	natural soil
Load Transfer system (dowels / interlock)	dowels
Dowel bar dimensions	1.25 X 18”
Dowel Spacing	12 in (30.48c m)

Mixture Properties

W/C	0.53
Air Content	5.5 to 6%
Cement	479 (lb/cy)

Maintenance Performed

Longitudinal cracking at mid-panel and wheel paths. Maintenance has been performed on these sections but not specified.

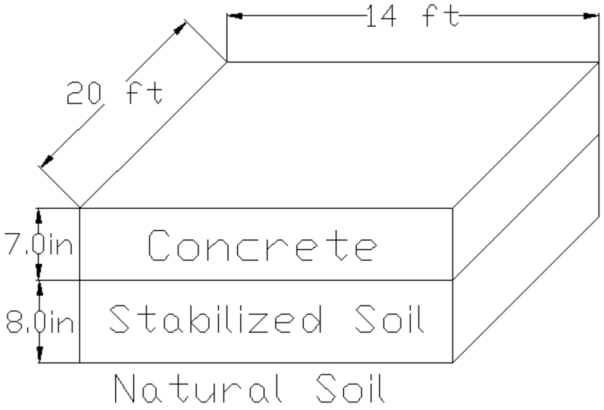


Figure 2-9. Typical slab design located at Airport Ring Rd. in Iowa.

2.7 Examples of Long-Life Concrete Pavements in Florida

2.7.1 US 1 Southbound lanes south of Edgewater, Florida. (Wall and Schmitt, 2007)

Figure 2-10 shows the location of a well-performing whitetopping pavement in Florida. Some slabs have various special dowel configurations including: 3 dowels in each wheel path; some slabs with special dowel spacing of 12, 24, 36, 96,108, and 120 inches from the outside edge of passing lane; no dowels in the last 5 slabs of each test section

The information on this pavement is summarized as follows:

General Information	
Type of pavement	JPCP
Year of construction/ Traffic Opening Date	1988
Present Condition	
Joints are performing well, and pavement has required minimal maintenance.	
Traffic Information	
ADT of 15,920 in 1997.	
Subbase / Base	
Type S asphalt layer	1 in (2.54 cm)
Leveling course	5/8 - 3.5 in (1.6 - 8.9 cm)
Limerock base	8.5 in (21.6 cm)
Pavement Properties	
Slab layer thickness	6-8 in (15.2-20.3 cm)
Joint spacing	12-20 ft (3.7-6.1 m)
Transverse joint angle	Rectangular
Shoulder design	natural soil
Load Transfer system (dowels / interlock)	dowels
Dowel bar dimensions	¾ inch and 1-inch
Dowel bar spacing (for standard sections)	12 inch (30.5 cm)
Special dowel configurations in special sections	
Maintenance Performed	
Minimal maintenance was required.	

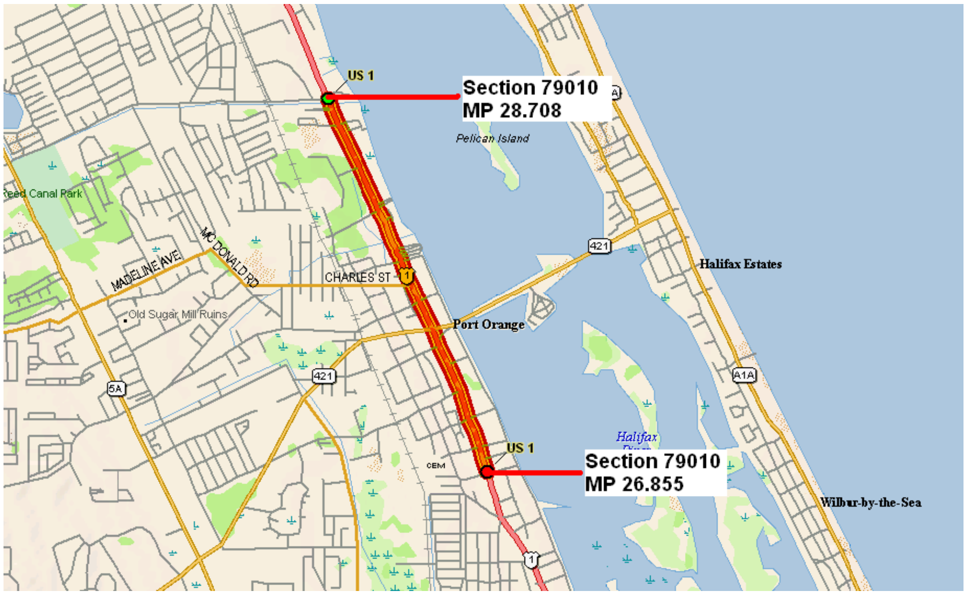


Figure 2-10. US 1 in Daytona location from MP 26.865 to MP 28.708 (Wall & Schmitt, 2007)**.**

2.7.2 US 1 Southbound lanes south of Edgewater, Florida (Wall and Schmitt, 2007)

Figure 2-11 shows the location of a well-performing 4-lane pavement in Florida. The information on this pavement is summarized as follows:

General Information

Type of pavement	JPCP
Year of construction/ Traffic Opening Date	1941

Present Condition

Ride condition has deteriorated somewhat.

Traffic Information

ADT of 51,500 in 2010.

Subbase / Base

Type of base	subgrade
Base thickness	18 in (45.72 cm)

Pavement Properties

Slab layer thickness	11.5 in (15.24-20.32 cm)
Joint spacing	14-16 ft (4.27-4.88 m)
Transverse joint angle	Rectangular
Shoulder design	natural soil

Maintenance Performed

Original 4 lane reinforced PCC roadway constructed in 1941, widened to 6 lanes in 1996. Entire roadway was diamond grinded.

Figure 2-11. US 1 in Daytona location from MP 18.666 to MP 19.648.

2.7.3 US 92 from Daytona to Deland (Wall and Schmitt, 2007).

Figure 2-12 shows the location of a long-life concrete pavement in Florida that has been well maintained. The information on this pavement is summarized as follows:

General Information

Type of pavement	JPCP
Year of construction/ Traffic Opening Date	1935-1945

Present Condition

Ride condition is deteriorated. Cracking is present.

Traffic Information

ADT of 9000 in 1992.

Subbase / Base

Type of base	stabilized subgrade
Base thickness	12 in (30.48 cm)

Pavement Properties

Slab layer thickness	9 in-edge (22.86 cm) 7 in-center (17.78 cm)
Joint spacing	11 ft (3.35 m)
Transverse joint angle	Rectangular
Shoulder design	natural soil

Maintenance Performed

Original 2 lane roadway constructed in segments. Roadway was widened in 1972 to 4 lane. Slab replacements have been performed, yet most of the roadway has not been subject to full rehabilitation.

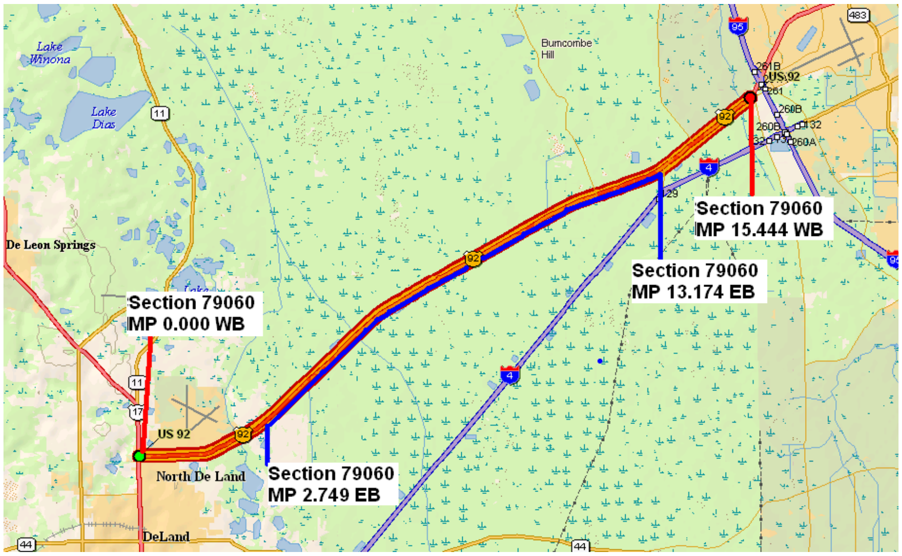


Figure 2-12. US 92 Eastbound and Westbound, Daytona to Deland. MP 0.000 to MP 15.444.

2.7.4 US 17/92 Northbound and Southbound, Deland (Wall and Schmitt, 2007).

Figure 2-13 shows the location of a long-lasting, reinforced PCC pavement in Florida. The information on this pavement is summarized as follows:

General Information

Type of pavement	RCP
Year of construction/ Traffic Opening Date	1939

Present Condition

Ride condition is deficient.

Subbase / Base

Type of base	12 in (30.5 cm) stabilized subgrade

Pavement Properties

Slab layer thickness	7 in; center (17.8 cm) 9 in; edge (22.9 cm)
Joint spacing	10 ft (3.05 m)
Transverse joint angle	Rectangular

Maintenance Performed

Original roadway has had partial slab and asphalt repairs. Decorative concrete intersections have been constructed.

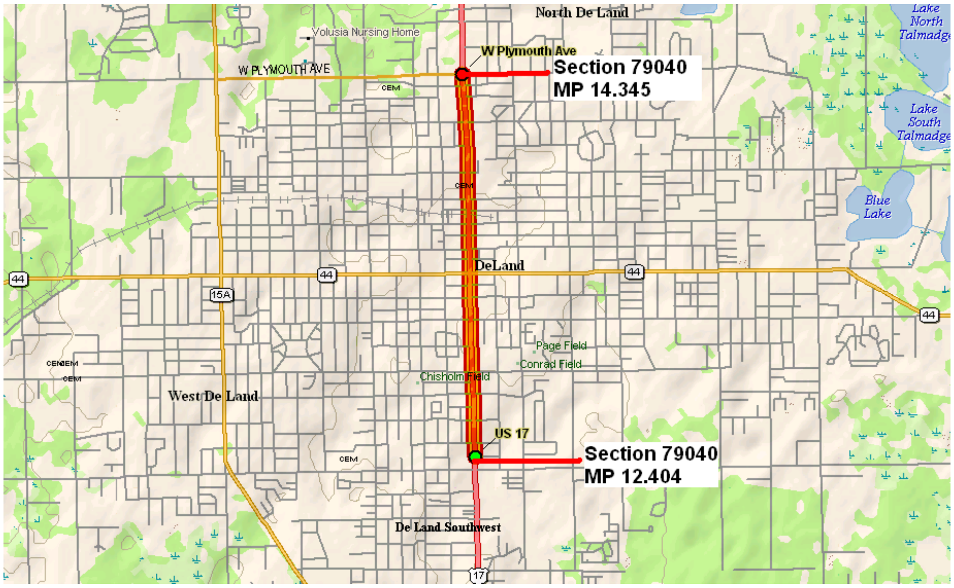


Figure 2-13. US 92 Northbound and Southbound, Deland. MP 12.404 to MP 14.345.

2.7.5 US 17/92 Northbound and Southbound, Winter Park (Wall and Schmitt, 2007).

Figure 2-14 shows the location of another long-lasting, reinforced PCC pavement in Florida. The information on this pavement is summarized as follows:

General Information	
Type of pavement	RCP
Year of construction/ Traffic Opening Date	1936
Present Condition	
Ride condition is deficient.	
Subbase / Base	
Type of base	stabilized subgrade
Pavement Properties	
Slab layer thickness	7 in (17.78 cm)
Joint spacing	10 ft (3.05 m)
Transverse joint angle	Rectangular
Maintenance Performed	
Original roadway has had partial slab and asphalt repairs. Decorative concrete intersections have been constructed.	

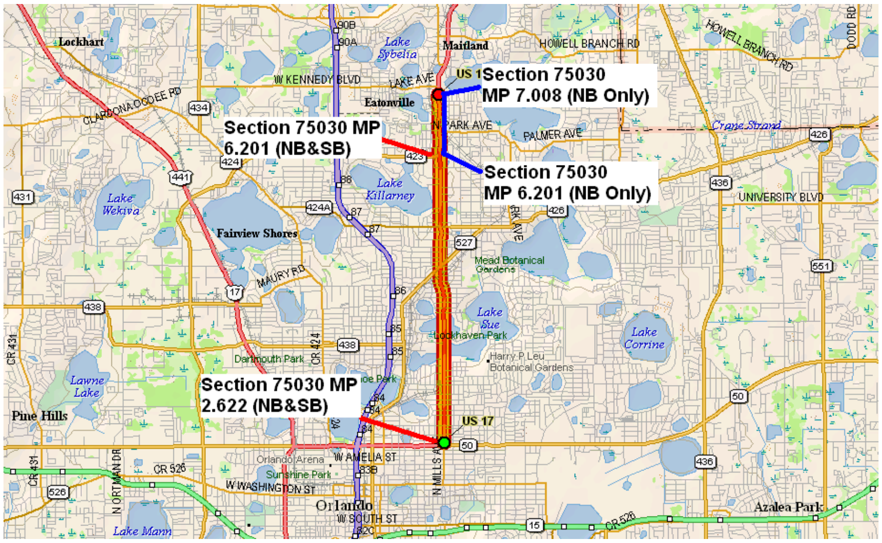


Figure 2-14. US 17/92 Northbound and Southbound, Winter Park. MP 2.622 to MP 7.008.

2.8 Summary

A literature review of some existing long-life concrete pavements is presented in this chapter. While it is not possible to generalize the characteristics of these pavements, some of the notable characteristics of the long-life concrete pavements include (1) good drainage, (2) good-quality concrete, (3) good-quality construction, and (4) adequate concrete slab thickness.

CHAPTER 3

EVALUATION OF FLORIDA CONCRETE PAVEMENT DESIGNS USING MEPDG MODEL

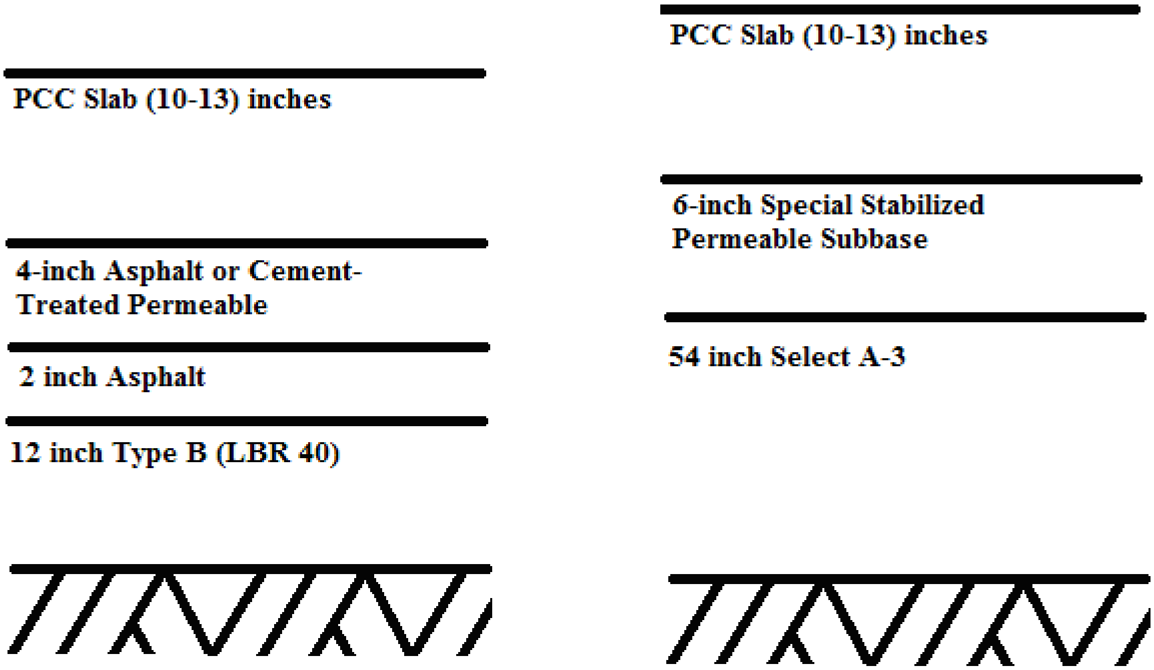
3.1 Typical Florida Concrete Pavement Designs

Three concrete pavement designs which are currently used by FDOT were selected for analysis. Table 3-1 shows the features of these three designs, which include the slab thickness, base and subbase types. These three designs are referred to as Type I-A, Type I-B and Type II in this report. According to FDOT, Type I-A pavement corresponds to a PCC slab of varied thickness with a 4-inch asphalt treated permeable base over a 2-inch asphalt structural course. Type I-B pavement consists of a PCC slab of varied thickness with a 4-inch asphalt concrete base. Both Type I-A and I-B pavements have a 12-inch Type B stabilized subgrade with a minimum Limerock Bearing Ratio (LBR) of 40. Type II pavement has a PCC slab of varied thickness with a 6-inch special stabilized subbase/base over 54-inch A-3 soil. The special stabilized subbase is made up of 3 inches of #57 or #89 coarse aggregate mixed into the top 6 inches of A-3 soil and is used as a working platform during construction. These designs are shown in Figure 3-1.

Table 3-1. Typical Concrete Pavement Designs Used by FDOT (Nazef , 2011).

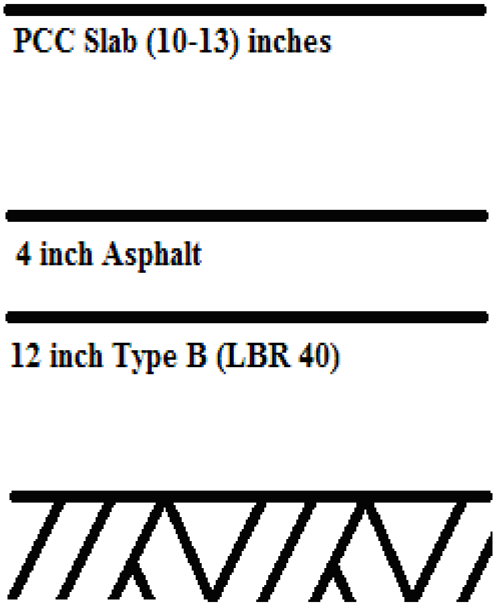
Type*	**Design Slab Thickness**		**Base Type**	**Subgrade Type**
	Minimum (inch)	Maximum (inch)		
Type I-A	8	N/A	4 inch asphalt or cement treated permeable base over 2 inch Type SP asphalt structural course	12 inch Type B stabilized subgrade (LBR 40)
Type I-B			4 inch asphalt concrete base	12 inch Type B stabilized subgrade (LBR 40)
Type II			6 inch special stabilized subbase over 54 inch special select embankment	None

Note: The type designations are used only in this study.



Type I-A

Type II



Type I-B

Figure 3-1. **Three FDOT concrete pavement designs.**

3.2 Inputs to MEPDG Model for Evaluation of Florida Concrete Pavement Designs

The Florida concrete pavement designs, Type I-A, Type I-B and Type II as described in Section A1, were evaluated using the MEPDG (Mechanistic-Empirical Pavement Design Guide) model, which has been calibrated for Florida conditions, to evaluate the effect of traffic volume on their predicted performance. The inputs to the MEPDG software for this analysis are described in the following sections.

The Florida Department of Transportation along with the Texas Transportation Institute (TTI) developed a calibration model to implement the MEPDG in the state of Florida. These calibrations were used to perform the following tests.

3.2.1 Design Life and Terminal Distresses Used

The design life of the concrete pavements to be analyzed by the MEPDG model was set to be 50 years. The outputs of the MEPDG analysis give the predicted performance of the pavements in terms of joint faulting, transverse cracking and International Roughness Index (IRI) over the design period. If one or more of the predicted distresses at the end of the design period exceed the acceptable threshold values, the analyzed pavement would be considered to have failed for the design period. The input threshold values used in the MEPDG are:

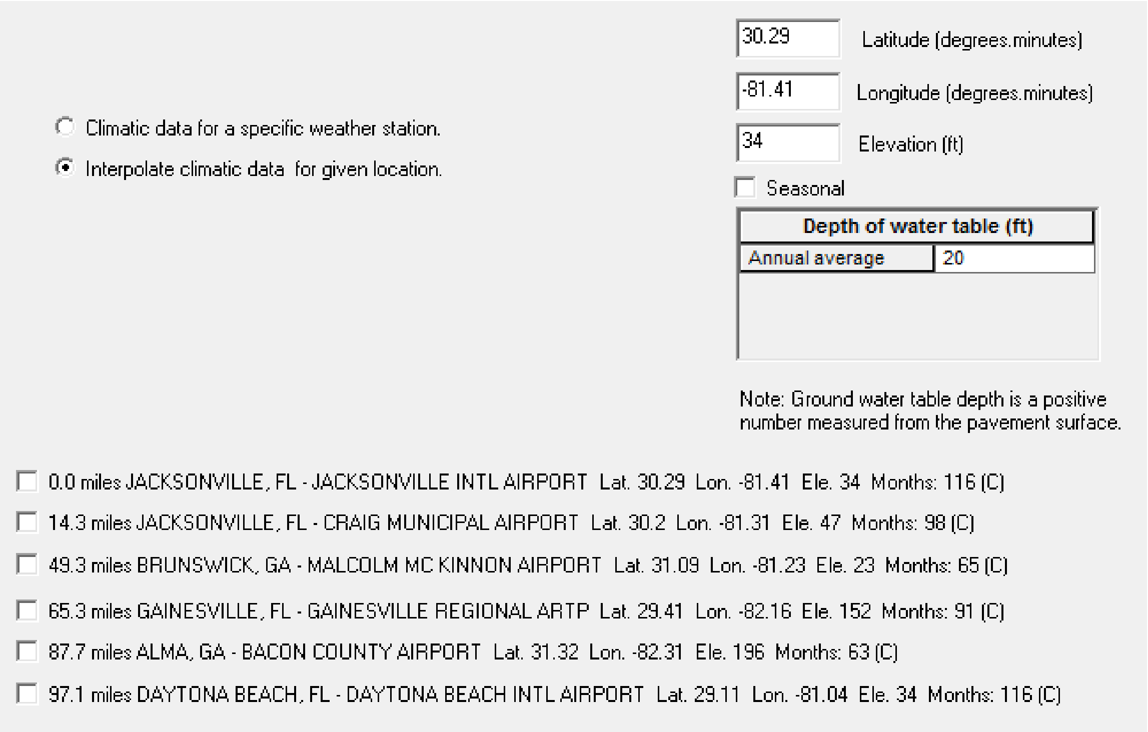
IRI = 180 in/mi
Joint faulting = 0.12 in
Transverse cracking = 10%

A reliability level of 95% was used in the analyses. It is important to note that Florida DOT allows a design reliability of 75% to 95%. With the incorporation of the 95% reliability level, the threshold values are:

IRI = 123 in/mi
Joint faulting = 0.034 in
Transverse cracking = 4.3%.

3.2.2 Climatic Parameters

The climatic condition used for this analysis was that for the North Eastern Florida Region. The MEPDG software contains a climatic database that provides hourly data from 800 weather stations all over the United States. The average data from six weather stations in the North Eastern Florida Region were used for this analysis. The weather stations used are shown in Figure 3-2.



30.29 Latitude (degrees.minutes)

-81.41 Longitude (degrees.minutes)

34 Elevation (ft)

○ Climatic data for a specific weather station.

● Interpolate climatic data for given location.

☐ Seasonal

Depth of water table (ft)	
Annual average	20

Note: Ground water table depth is a positive number measured from the pavement surface.

☐ 0.0 miles JACKSONVILLE, FL - JACKSONVILLE INTL AIRPORT Lat. 30.29 Lon. -81.41 Ele. 34 Months: 116 (C)

☐ 14.3 miles JACKSONVILLE, FL - CRAIG MUNICIPAL AIRPORT Lat. 30.2 Lon. -81.31 Ele. 47 Months: 98 (C)

☐ 49.3 miles BRUNSWICK, GA - MALCOLM MC KINNON AIRPORT Lat. 31.09 Lon. -81.23 Ele. 23 Months: 65 (C)

☐ 65.3 miles GAINESVILLE, FL - GAINESVILLE REGIONAL ARTP Lat. 29.41 Lon. -82.16 Ele. 152 Months: 91 (C)

☐ 87.7 miles ALMA, GA - BACON COUNTY AIRPORT Lat. 31.32 Lon. -82.31 Ele. 196 Months: 63 (C)

☐ 97.1 miles DAYTONA BEACH, FL - DAYTONA BEACH INTL AIRPORT Lat. 29.11 Lon. -81.04 Ele. 34 Months: 116 (C)

Figure 3-2. The six weather stations used to provide climatic inputs. (Fernando et al., 2008)

3.2.3 Portland Cement Concrete (PCC) Thickness

A Jointed Plain Concrete Pavement (JPCP) structure was analyzed. To evaluate the effects of concrete slab thickness on performance, the concrete slab thickness was varied according to the analysis performed.

3.3 MEPDG Rigid Pavement Performance Prediction Equations

3.3.1 Transverse Cracking

MEPDG calculates both bottom-up cracking and top-down cracking of the transverse slab at the same time and presents this data as a percentage value representing predicted amount of cracking. Equation 3-1 shows the equation used by MEPDG for the prediction of transverse cracking in a concrete slab.

$$CPK = \frac{1}{1+(DI_F)^{-1.98}} \qquad (3\text{-}1)$$

Where:

CPK = Predicted amount of bottom-up or top-down cracking (fraction); and,

DI_F = Fatigue Damage calculated using Miner's procedure below.

For fatigue calculations, the Miner's theory is used. This theory accumulates all the damage caused by fatigue in a pavement which causes the transverse cracking and is shown in Equation 3-2.

$$DI_F = \sum \frac{n_{i,j,k,l,m,n,o}}{N_{i,j,k,l,m,n,o}} \qquad (3\text{-}2)$$

Where:

DI_F = Total fatigue damage,

$n_{i,j,k,l,m,n,o}$ = Applied number of load applications at conditions i,j,k,l,m,n,o

$N_{i,j,k,l,m,n,o}$ = Allowable number of load applications at condition i,j,k,l,m,n,o

i = Age (Accounts for change in modulus of rupture and elasticity, slab/base contact friction, deterioration of shoulder),

j = Month (Accounts for change in base elastic modulus and effective dynamic modulus of subgrade reaction),

k = Axle type (single, tandem and tridem for bottom-up cracking; short, medium and long wheelbase for top down cracking),

l = Load level (incremental load for each type),

m = Equivalent temperature difference between top and bottom PCC surfaces.

n = Traffic offset path; and,

o = Hourly truck traffic fraction.

The allowable number of load applications is based on the applied stresses, strength of the slab and is shown in Equation 3-3.

$$\log(N_{i,j,k,l,m,n,o}) = C_1 * (\frac{M_{Ri}}{\sigma_{i,j,k,l,m,n,o}})^{C_2} \qquad (3\text{-}3)$$

Where:

$N_{i,j,k,l,m,n,o}$ = Allowable number of load applications at condition i,j,k,l,m,n,o;

M_{RI} = PCC modulus of rupture at age i, psi,

$\sigma_{i,j,k,l,m,n,o}$ = Applied stress at condition i,j,k,l,m,n,o;

C_1 = Calibration constant 2.0, and;

C_2 = Calibration constant 1.22.

Total cracking is calculated using Equation 3-4 below. It is important to note that, based on the studies performed by FDOT and TTI, the models and equations used for cracking did not have to be calibrated, as these equations have been calibrated for the pavement conditions in Florida.

$$TCR = (CRK_{bp} + CRK_{td} - CRK_{bp} * CRK_{td} * 100\%) \qquad (3\text{-}4)$$

Where:

TCR = Total transverse cracking (percent in all severities),

CRK_{bp} = Predicted amount of bottom-up transverse cracking (fraction); and,

CRK_{td} = Predicted amount of top-down transverse cracking (fraction).

3.3.2 Faulting

MEPDG predicts faulting using an incremental approach. At the beginning of each month, faulting is calculated using data from each of the previous months. These data are then summed up using the Equations 3-5 through 3-10 shown below. Modifications were made in this section to calibrate the MEPDG program to work in Florida conditions. These modifications were made in relation to a long study which compared MEPDG distress results to actual field data (Fernando et al., 2008).

$$Fault_m = \sum_{t=1}^{m} \Delta Fault_i \qquad (3\text{-}5)$$

$$\Delta Fault_i = C_{34} * (FAULTMAX_{i-1} - Fault_{i-1})^2 * DE_i \qquad (3\text{-}6)$$

$$FAULTMAX_i = FAULTMAX_0 + C_7 * \sum_{j=1}^{m} DE_j * Log(1 + C_5 * 5.0^{EROD})^{c_6} \tag{3-7}$$

$$FAULTMAX_0 = C_{12} * \delta_{curling} * \left[Log(1 + C_5 * 5.0^{EROD}) * Log(\frac{P_{200} * WetDays}{P_s}) \right]^{C_v} \tag{3-8}$$

$$C_{12} = C_1 + C_2 * FR^{0.25} \tag{3-9}$$

$$C_{34} = C_3 + C_4 * FR^{0.25} \tag{3-10}$$

Where:

Fault$_m$ = Mean joint faulting at the end of the month, inches,

ΔFault$_i$ = Incremental change (monthly) in mean transverse joint faulting during month i, inches,

FAULTMAX$_i$= Maximum mean transverse joint faulting for month i, inches,

FAULTMAX$_0$= Initial maximum mean transverse joint faulting, inches,

EROD = Base/Subbase erodibility Factor

DE$_i$ = Differential Density of energy of subgrade deformation accumulated during month i,

$\delta_{curling}$ = Maximum mean monthly slab corner upward deflection PCC due to temperature curling and moisture warping, in,

P$_s$ = Overburden on subgrade, lb,

P$_{200}$ = Percent subgrade material passing 200 sieve,

WetDays = Average annual number of wet days (greater than 0.1 rainfall),

$C_{1,2,3\ldots}$ = Global Calibration Constants (Florida Calibration). [C_1=2.0; C_2=1.1; C_3=0.001725; C_4=0.0008; C_5=250; C_6=0.4; C_7=1.2]

FR = Base freezing index defined as a percentage of time, not applicable in Florida testing.

As mentioned above, slab curling and warping is calculated for each month using the Integrated Climatic Model (ICM) weather data that are preloaded into MEPDG. Equation 3-11 shows the procedure to find the temperature differential for each month using the MEPDG program.

$$\Delta T_m = \Delta T_{t,m} - \Delta T_{b,m} + \Delta T_{sh,m} + \Delta T_{PCW} \tag{3-11}$$

Where:

ΔT_m = Effective temperature differential for month m,

$\Delta T_{t,m}$ = Mean PCC top-surface nighttime temperature (from 8:00 pm to 8:00 am) for month m,

$\Delta T_{b,m}$ = Mean PFCC bottom-surface nighttime temperature (from 8:00 pm to 8:00 am) for month m,

$\Delta T_{sh,m}$ = Equivalent temperature differential due to reversible shrinkage for month m; and,

ΔT_{PCW} = Equivalent temperature differential due to permanent curl/warp.

Load Transfer Efficiency (LTE) is calculated in the MEPDG using Equation 3-12. This is used to calculate the load on the transverse joints of a pavement in question.

$$LTE_{joint} = 100\left[1-(1-\frac{LTE_{dowel}}{100})(1-\frac{LTE_{agg}}{100})(1-\frac{LTE_{base}}{100})\right] \tag{3-12}$$

Where:

LTE_{joint} = Total transverse joint LTE, %,

LTE_{dowel} = Joint LTE if dowels are the only mechanism of load transfer, %,

LTE_{base} = Joint LTE if base is the only mechanism of load transfer, %; and,

LTE_{agg} = Joint LTE is aggregate interlock is the only mechanism of load Transfer, %.

Maximum faulting is a calculation based on the differential energy from truck loading, shear stress at slab corner and maximum dowel and joint bearing stress. Calculations can be made using Equation 3-13 through 3-15 shown below.

$$DE = \frac{k}{2}(\delta_L{}^2 - \delta_U{}^2) \tag{3-13}$$

$$\tau = \frac{AGG*(\delta_L - \delta_U)}{h_{PCC}} \tag{3-14}$$

$$\sigma_b = \frac{\zeta_d*(\delta_L - \delta_U)}{d*dsp} \tag{3-15}$$

Where:

DE = Differential energy, lb/in,

δ_L = Loaded corner deflection, in,

δ_U = Unloaded corner deflection, in,

AGG = Aggregate interlock stiffness factor,

k = Coefficient of subgrade reaction, psi/in,

h_{PCC} = PCC slab thickness, in,

δ_d = Dowel stiffness factor =Jd*k*l*dsp,

d = Dowel diameter, in,

dsp = Dowel spacing, in,

J_d = Non-dimensional dowel stiffness at the time of load application; and,

l = Radius of relative stiffness, in.

Load transfer data were gathered from the Portland Cement Association to determine loss of shear capacity (Δs) in designed pavement structures using MEPDG. These losses are created by traffic loading and are shown in Equations 3-16 through 3-18.

$$\Delta s = \begin{array}{ll} 0 & if \quad jw \le 0.001 h_{PCC} \\ \sum_j \dfrac{0.005}{1.0 + (\dfrac{jw}{h_{PCC}})^{-5.7}} (\dfrac{n_j}{10^6})(\dfrac{\tau_j}{\tau_{ref}}) & if \quad jw \le 3.8 h_{PCC} \\ \sum_j \dfrac{0.068}{1.0 + 6.0 * (\dfrac{jw}{h_{PCC} - 3})^{-1.98}} (\dfrac{n_j}{10^6})(\dfrac{\tau_j}{\tau_{ref}}) & if \quad jw \triangleright 3.8 h_{PCC} \end{array} \tag{3-16}$$

$$\tau_j = \frac{AGG * (\delta_L - \delta_U)}{h_{PCC}} \tag{3-17}$$

$$\tau_{ref} = 111.1 * e^{-e^{0.988 * e^{-0.1089 \log Jagg}}} \tag{3-18}$$

Where:

n_j = Number of applied load applications for the current increment by load group, j,

jw = Joint opening, mils (0.001 in),

τ_j = Shear Stress on the transverse crack from the response model for the load group j, psi,

τ_{ref} = Reference shear stress derived from the PCA test results, psi; and,

J_{agg} = Joint Stiffness on the transverse crack computed for the time increment.

Last, the damage at the dowel-concrete interface has to be computed using Equation 3-19.

$$DAM_{dow} = C_8 \sum_j \left(\frac{j_d * (\delta_L - \delta_U) * dsp}{df'c} \right) \tag{3-19}$$

Where:

DAM_{dow} = Damage at dowel-concrete interface,

C_8 = Coefficient equal to 400,

n_j = Number of applied load applications for the current increment by load group, j,

J_d = Non-dimensional dowel stiffness at the time of load application,

δ_L = Loaded corner deflection, in,

δ_U = Unloaded corner deflection, in,

dsp = Space between adjacent dowels in the wheel path, in,

f’c = PCC compressive strength, psi; and,

d = Dowel diameter, in.

3.3.3 International Roughness Index (IRI)

MEPDG combines the initial profile of the pavement and the overall loss of smoothness with age. These values were calibrated using LTPP data which include field data and spalling calculated using Equation 3-20. As mentioned previously, calibrations were made to the equations to adapt results to Florida conditions.

$$IRI = IRI_I + C1*CRK + C2*SPALL + C3*TFAULT + C4*SF \qquad (3\text{-}20)$$

Where:

IRI = Predicted IRI, in/mi,

IRI_I = Initial smoothness measured as IRI, in/mi,

CRK = Percent slabs with transverse cracks (all severities),

SPALL = Percentage of joints with spalling (medium and high severities),

TFAULT = Total joint faulting accumulated per mi, in; and,

C1 = 0.8203

C2 = 0.4417

C3 = 2.5 (Florida calibration)

C4 = 25.24

SF = Site factor.

The site factor equation is an equation based on the pavement age and it is shown below in Equation 3-21. Percentage of joints spalled is found using Equation 3-22.

$$SF = AGE(1 + 0.5556*F1)(1 + P_{200})*10^{-6} \qquad (3\text{-}21)$$

Where:

AGE = Pavement age, yrs,

F1 = Freezing index (Not applicable in Florida); and,

P_{200} = Percent subgrade material passing No. 200 sieve.

$$SPALL = (\frac{AGE}{AGE + 0.01})(\frac{100}{1 + 1.005^{-12*AGE+SCF}}) \qquad (3\text{-}22)$$

Where:

SPALL = Percentage of joints spalled (medium and high severities),

AGE = Pavement age, yrs; and,

SCF = Scaling actor based on site-design (climate related).

Scaling factor is calculated in MEPDG based on properties of PCC slab that are used for the design. Equation 3-23 shows this procedure.

$$SCF = -1400 + 350 * AC_{PCC} * (0.5 + PREFORM) + \\ 3.4 f'c * 0.4 - 0.2(FT_{cycles} * AGE) + 43H_{PCC} - 536WC_{PCC} \qquad (3\text{-}23)$$

Where:

AC_{PCC} = PCC air content, %,

AGE = Time since construction, yrs,

PREFORM = 1 if preformed sealant is present, 0 if not,

f’c = PCC compressive strength, psi,

FT_{cycles} = Average annual number of freeze-thaw cycles (Not applicable in Florida),

H_{PCC} = PCC slab thickness, in; and,

WC_{PCC} = PCC W/C ratio.

3.3.4 Standard Error Calculations

A standard error calculation is performed by MEPDG to adjust the values predicted by the program. The error calculations for cracking, faulting and IRI are shown in Equations 3-24 through 3-26.

$$Se_{CR} = -0.00198 * CRACK^2 + 0.5686 CRACK + 2.7668 \qquad (3\text{-}24)$$

Where:

Se_{CR} = Standard error of the estimate of transverse cracking at the predicted level of mean cracking; and,

CRACK = Predicted transverse cracking based on mean inputs (corresponding to 50 percent reliability), percentage of slabs.

$$Se_F = (0.00761 * Fault(t) + 0.00008099)^{0.445} \qquad (3\text{-}25)$$

Where:

Fault(t) = Predicted mean transverse joint faulting at any given time, in.

Se_F = Standard error of the estimate of faulting at the predicted level of mean faulting.

$$Se_{IRI} = (Var_{IRI_i} + C1^2 * Var_{CRK} + C2^2 * Var_{SPALL} + \qquad (3\text{-}26)$$
$$C3^2 * Var_{FAULT} + S_e{}^2 * 0.5$$

Where:

Se_{IRI} = Standard deviation of IRI, in/mi

Var_{IRI} = Variance of initial IRI = 29.16 $(in/mi)^2$

Var_{CRK} = Variance of cracking, % of slab cracked

Var_{SPALL} = Variance of spalling = 46.24%

Var_{FAULT} = Variance of faulting (in^2); and,

S_e^2 = Variance of overall model error = 745.3 $(in/mi)^2$.

3.4 Traffic Inputs

In the MEPDG model, traffic is input either as AADTT (Average Annual Daily Truck Traffic) or as AADT (Average Annual Daily Traffic) where a percentage of heavy vehicles (Class 4 or higher) are multiplied to find the AADTT. A typical high-volume traffic in Florida was used and is shown in Table 3-2.

Table 3-2. Truck Traffic Inputs to the MEPDG Model

Initial two-way AADTT:	4000
Number of lanes in design direction:	2
Percent of trucks in design direction (%):	50
Percent of trucks in design lane (%):	95
Operational speed (mph):	60

In addition to the AADTT, the user may adjust the distribution of truck traffic on a monthly basis. An adjustment factor of 1 was used; this is MEPDG's current default value for all months and all truck classes. Hourly truck distribution as shown in Table 3-3 was used in this analysis. Table 3-4 shows the vehicle class distribution used in this analysis.

Table 3-3. Hourly Traffic Distribution by Period

Hourly truck traffic distribution

Midnight	2.3%	Noon	5.9%
1:00 am	2.3%	1:00 pm	5.9%
2:00 am	2.3%	2:00 pm	5.9%
3:00 am	2.3%	3:00 pm	5.9%
4:00 am	2.3%	4:00 pm	4.6%
5:00 am	2.3%	5:00 pm	4.6%
6:00 am	5.0%	6:00 pm	4.6%
7:00 am	5.0%	7:00 pm	4.6%
8:00 am	5.0%	8:00 pm	3.1%
9:00 am	5.0%	9:00 pm	3.1%
10:00 am	5.9%	10:00 pm	3.1%
11:00 am	5.9%	11:00 pm	3.1%

Table 3-4. Vehicle Class Distribution by Vehicle Class

	AADTT distribution by vehicle class
Class 4	1.8%
Class 5	24.6%
Class 6	7.6%
Class 7	0.5%
Class 8	5.0%
Class 9	31.3%
Class 10	9.8%
Class 11	0.8%
Class 12	3.3%
Class 13	15.3%

The general design involves two lanes traveling in each direction, 50 percent of trucks in each direction, 95 percent of trucks in each design lane and an operational speed of 60 mph. Along with these design values, monthly volume adjustments, vehicle class distribution, hourly truck traffic distribution, traffic growth factor, number of axles per truck and wheelbase truck tractor factors have been adjusted accordingly. These adjustments can be seen on Table 3-5 through Table 3-8.

Table 3-5. Monthly Traffic Volume Adjustment Factors, Level 3 Default

	Vehicle Class									
Month	**Class 4**	**Class 5**	**Class 6**	**Class 7**	**Class 8**	**Class 9**	**Class 10**	**Class 11**	**Class 12**	**Class 13**
January	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
February	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
March	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
April	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
May	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
June	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
July	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
August	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
September	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
October	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
November	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
December	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Table 3-6. Traffic Growth Factor Model

Vehicle Class	Growth Rate	Growth Function
Class 4	4.0%	Compound
Class 5	4.0%	Compound
Class 6	4.0%	Compound
Class 7	4.0%	Compound
Class 8	4.0%	Compound
Class 9	4.0%	Compound
Class 10	4.0%	Compound
Class 11	4.0%	Compound
Class 12	4.0%	Compound
Class 13	4.0%	Compound

Table 3-7. Number of Axles per Truck

Vehicle Class	Single Axle	Tandem Axle	Tridem Axle	Quad Axle
Class 4	1.62	0.39	0.00	0.00
Class 5	2.00	0.00	0.00	0.00
Class 6	1.02	0.99	0.00	0.00
Class 7	1.00	0.26	0.83	0.00
Class 8	2.38	0.67	0.00	0.00
Class 9	1.13	1.93	0.00	0.00
Class 10	1.19	1.09	0.89	0.00
Class 11	4.29	0.26	0.06	0.00
Class 12	3.52	1.14	0.06	0.00
Class 13	2.15	2.13	0.35	0.00

Table 3-8. Axle Spacing

	Short	Medium	Long
Average Axle Spacing (ft)	12	15	18
Percent of trucks	33%	33%	34%

3.5 Evaluation of the Effects of Slab Thickness and Concrete Aggregate Type on the Performance of Florida Concrete Pavement Designs

3.5.1 Typical Florida Concrete Pavement Designs

Three concrete pavement designs which are currently used by FDOT were selected for analyses. These three designs are referred to as Type I-A, Type I-B and Type II in this study, and have been described in Section 3.1 of this chapter.

3.5.2 Inputs to MEPDG Model for Evaluation of Florida Concrete Pavement Designs

The Florida concrete pavement designs, as described in Section 3.1, were evaluated using the MEPDG (Mechanistic-Empirical Pavement Design Guide) model to determine the feasibility of using them for long-life concrete pavements. The MEPDG model was used to evaluate whether or not these two designs could provide long-life concrete pavements with a design life of 50 years, and to determine the required slab thickness and concrete properties for such application. The inputs to the MEPDG software for this analysis are described in the following sections.

3.5.2.1 Design Life

The design life of the concrete pavements to be analyzed by the MEPDG model was set to be 50 years. Complete information on the design life and failure criteria are described in Section 3.2.1 of this chapter.

3.5.2.2 Climatic Parameters

Climatic conditions used in this analysis are described in Section 3.2.3 of this chapter.

3.5.2.3 Portland Cement Concrete (PCC) Thickness

A Jointed Plain Concrete Pavement (JPCP) structure was analyzed. To evaluate the effects of concrete slab thickness on performance, the concrete slab thickness was varied from 10 to 14 inches in the analysis.

3.5.2.4 Coefficients of Thermal Expansion

Three typical concretes used in Florida, made with three different aggregates were used for this analysis. These three aggregates are (1) Brooksville limestone (a porous limestone from Brooksville, Florida, (2) Calera limestone (a dense limestone from Calera, Alabama), and (3) river gravel (from Alabama). Typical properties of these concretes as obtained from previous studies for FDOT were used as inputs for concrete material properties.

Table 3-9 shows the typical unit weights of these concretes made with these three different aggregates. These values were used as the unit weights for the three different concretes considered in the analysis.

Table 3-9. Typical Unit Weights of Florida Concretes

Condition	Aggregate	Unit weight (pcf)
28-day cure	Brooksville	145
	Calera	152
	River Gravel	150

(Source: "Field and Laboratory Study of Modulus of Rupture and Permeability of Structural Concretes in Florida" by Tia et al, 1990)

Table 3-10 shows the typical coefficients of thermal expansion of these concretes made with these three aggregates. These values were used as the coefficient of thermal expansion for these three concretes in the analysis.

Table 3-10. Typical Coefficient of Thermal Expansion of Florida Concretes

Condition	Aggregate	CTE (x 10^{-6} in/in/F)
28-day	Brooksville	5.68
	Calera	5.99
	River Gravel	7.2

(Source: "Coefficient of Thermal Expansion of Concrete Used in Florida" by Tia et al, 1991)

A flexural strength of concrete of 650 psi was used in the analysis. In a previous study by Tia et al (1990) for the FDOT, the elastic modulus of Florida concrete was related to the flexural strength, unit weight and the type of aggregate used. The developed equations for estimation of elastic modulus were used in estimating the elastic moduli of these three concretes used in the MEPDG analyses. Table 3-11 shows the calculation of the elastic moduli of these three concretes with a flexural strength of 650 psi.

Table 3-11. Elastic Modulus of Concretes Made with Different Aggregates

Equation	Condition	Aggregate	[w] Unit weight	[f_r] Modulus of Rupture	[E] Elasticity
$E = 4.20 (w^{1.5}) f_r$	**28-day**	Brooksville	145 pcf	650 psi	4766665 psi
$E = 4.09 (w^{1.5}) f_r$		Calera	152	650	4981981 psi
$E = 3.69 (w^{1.5}) f_r$		River Gravel	150	650	4406326 psi

(Source: *Field and Laboratory Study of Modulus of Rupture and Permeability of Structural Concretes in Florida* by Tia et al., 1990)

3.5.2.5 Joint Spacing and Dowels

Florida's Rigid Pavement Design Manual specifies a maximum joint spacing of either 15 ft or 24 times the slab thickness, whichever is smaller. For this analysis, a joint spacing of 15 ft was used.

Doweled transverse joints were used in the modeling of the pavement sections. The dowel diameter was set to 1.5 inches with dowel bar spacing of 12 inches c/c. This relates to the standard of practice used for metal dowel bar assembly.

3.5.2.6 Edge Support and Base Properties

Table 3-12 shows the inputs used for the edge support and base properties for the two pavement designs analyzed. A widened slab of 13 feet was used in the analysis. Table 3-13 shows the strength properties of the granular permeable base used. The properties of the structural asphalt base material are shown in Table 3-14. The properties of the A-3 soil base are shown in Table 3-15. The properties of the LBR40 stabilized subgrade are shown in Table 3-16.

Table 3-12. Edge Support and Base Properties Used

Edge Support	Tied PCC shoulder, Widened slab
Long-term LTE(%):	50
Widened Slab (ft):	13
Base Properties	
Base type:	Granular
Erodibility index:	Very Erodible (5)
PCC-Base Interface	Zero friction contact
Loss of full friction (age, in months):	n/a

Table 3-13. Strength Properties of Granular Permeable Base Used

Strength Properties	
Input Level:	Level 3
Analysis Type:	ICM inputs (ICM Calculated Modulus)
Poisson's ratio:	0.35
Coefficient of lateral pressure,Ko:	0.5
Modulus (input) (psi):	20000

Table 3-14. Properties of Structural Asphalt Base Used

General Properties	
General	
Reference temperature (F°):	70
Volumetric Properties as Built	
Effective binder content (%):	11.6
Air voids (%):	7
Total unit weight (pcf):	150
Poisson's ratio:	0.35 (user entered)
Thermal Properties	
Thermal conductivity asphalt (BTU/hr-ft-F°):	0.67
Heat capacity asphalt (BTU/lb-F°):	0.23

Table 3-15. Properties of A-3 Soil Base Material Used

Strength Properties

Input Level:	Level 3
Analysis Type:	ICM inputs (ICM Calculated Modulus)
Poisson’s ratio:	0.35
Coefficient of lateral pressure,Ko:	0.5
Modulus (input) (psi):	16000

Table 3-16. Properties of Stabilized Subgrade (LBR 40)

Strength Properties

Input Level:	Level 2
Analysis Type:	ICM inputs (ICM Calculated Modulus)
Poisson's ratio:	0.35
Coefficient of lateral pressure,Ko:	0.5
California Bearing Ratio (CBR):	32
Modulus (calculated) (psi):	23479

3.5.2.7 Traffic Inputs

Traffic inputs used in this analysis are described in Section 3.4 of this chapter.

3.5.3 Effects of Concrete Aggregate Type on Type I-A Pavement

This section presents the results of MEPDG analysis on the effects of concrete aggregate type on the expected performance of pavement with Type I-A design.

3.5.3.1 Brooksville Aggregate

Table 3-17 shows the predicted terminal distresses of concrete pavements of Type I-A design and using concrete containing Brooksville limestone aggregate. The flexural strength of the concrete used in the analysis was 650 psi. It can be seen that when the concrete slab thickness is 12 inches or less, the pavement is predicted to have failed at the end of 50 years. However, with a slab thickness of 13 inches, the predicted distresses are below the threshold values, and the pavement is considered adequate for 50-year design life.

Table 3-17. Predicted Terminal Distress of Type I-A Concrete Pavement Using Brooksville Aggregate

Brooksville Aggregate

Distress Predicted (E = 4766665 psi)

Pavement Distress		**Slab Thickness (in)**			
Type	**Measurement**	**10**	**11**	**12**	**13**
Terminal IRI	(in/mi)	78.6	100.1	65.4	73
Transverse Cracking	(% slabs cracked)	67.9	39.5	12.6	3.1
Mean Joint Faulting	(in)	0	0.007	0.002	0.013
Pass/Fail		Fail	Fail	Fail	Pass

3.5.3.2 Calera Aggregate

Table 3-18 shows the predicted terminal distresses of similar concrete pavements of Type I-A design and using concrete containing Calera limestone aggregate. It can be seen that when the concrete slab thickness is 14 inches or less, the pavement is predicted to have failed at the end of 50 years. However, with a slab thickness of 15 inches, the predicted distresses are below the threshold values, and the pavement would be considered adequate for 50-year design life.

Table 3-18. Predicted Terminal Distress of Type I-A Concrete Pavement Using Calera Aggregate

Calera Aggregate

Distress Predicted Table (E = 4981981 psi)

Pavement Distress		**Slab Thickness (in)**					
Type	**Measurement**	**10**	**11**	**12**	**13**	**14**	**15**
Terminal IRI	(in/mi)	143	147.6	70.4	90.4	81.3	63.4
Transverse Cracking	(% slabs cracked)	94.5	84.4	38.1	26.2	12.9	4.3
Mean Joint Faulting	(in)	0.003	0.009	0.003	0.013	0.014	0
Pass/Fail		Fail	Fail	Fail	Fail	Fail	Pass

3.5.3.3 River Gravel Aggregate

Table 3-19 shows the predicted terminal distresses of similar concrete pavements of Type I-A design and using concrete containing river gravel. It can be seen that when the concrete slab thickness is 15 inches or less, the pavement is predicted to have failed at the end of 50 years.

However, with a slab thickness of 16 inches, the predicted distresses are below the threshold values, and the pavement would be considered adequate for 50-year design life.

Table 3-19. Predicted Terminal Distress of Type I-A Concrete Pavement Using River Gravel

River Gravel Aggregate

Distress Predicted (E = 4406326)

Pavement Distress		**Slab Thickness (in)**						
Type	**Measurement**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
Terminal IRI	(in/mi)	118.2	100.1	65.5	108.3	63.2	65.3	63.6
Transverse Cracking	(% slabs cracked)	97.2	90.5	69.5	34.5	25.4	8.9	2.4
Mean Joint Faulting	(in)	0.001	0.003	0	0.032	0	0.003	0.001
Pass/Fail		Fail	Fail	Fail	Fail	Fail	Fail	Pass

3.5.4 Effects of Concrete Aggregate Type on Type I-B Pavement

This section presents the results of MEPDG analysis on the effects of concrete aggregate type on the expected performance of pavement with Type I-B design.

3.5.3.1 Brooksville Aggregate

Table 3-20 shows the predicted terminal distresses of concrete pavements of Type I-B design and using concrete containing Brooksville limestone aggregate. The flexural strength of the concrete used in the analysis was 650 psi. It can be seen that when the concrete slab thickness is 12 inches or less, the pavement is predicted to have failed at the end of 50 years. However, with a slab thickness of 13 inches, the predicted distresses are below the threshold values, and the pavement is considered adequate for 50-year design life.

Table 3-20. Predicted Terminal Distress of Type I-B Concrete Pavement Using Brooksville Aggregate

Brooksville Aggregate

Distress Predicted (E = 4766665 psi)

Pavement Distress		**Slab Thickness (in)**			
Type	**Measurement**	**10**	**11**	**12**	**13**
Terminal IRI	(in/mi)	80.6	98.6	64.4	62.7
Transverse Cracking	(% slabs cracked)	70.4	19.5	6.3	4.2
Mean Joint Faulting	(in)	0.008	0.006	0.002	0.001
Pass/Fail		Fail	Fail	Fail	Pass

3.5.3.2 Calera Aggregate

Table 3-21 shows the predicted terminal distresses of similar concrete pavements of Type I-B design and using concrete containing Calera limestone aggregate. It can be seen that when the concrete slab thickness is 14 inches or less, the pavement is predicted to have failed at the end of 50 years. However, with a slab thickness of 15 inches, the predicted distresses are below the threshold values, and the pavement would be considered adequate for 50-year design life.

Table 3-21. Predicted Terminal Distress of Type I-B Concrete Pavement Using Calera Aggregate

Calera Aggregate

Distress Predicted (E = 4981981 psi)

Pavement Distress		**Slab Thickness (in)**					
Type	**Measurement**	**10**	**11**	**12**	**13**	**14**	**15**
Terminal IRI	(in/mi)	145.8	137.2	75.3	80.3	66.7	65.4
Transverse Cracking	(% slabs cracked)	100.6	86.5	42.5	16.4	6.9	4.2
Mean Joint Faulting	(in)	0.009	0.008	0.007	0.005	0.001	0.001
Pass/Fail		Fail	Fail	Fail	Fail	Fail	Pass

3.5.3.3 River Gravel Aggregate

Table 3-22 shows the predicted terminal distresses of similar concrete pavements of Type I-B design and using concrete containing river gravel. It can be seen that when the concrete slab thickness is 15 inches or less, the pavement is predicted to have failed at the end of 50 years. However, with a slab thickness of 16 inches, the predicted distresses are below the threshold values, and the pavement would be considered adequate for 50-year design life.

Table 3-22. Predicted Terminal Distress of Type I-B Concrete Pavement Using River Gravel

River Gravel Aggregate

Distress Predicted (E = 4406326)

Pavement Distress		**Slab Thickness (in)**						
Type	**Measurement**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
Terminal IRI	(in/mi)	110.5	108.3	100.2	98.3	60.1	68.6	62.1
Transverse Cracking	(% slabs cracked)	99.7	97.4	70.4	26.7	15.4	8.9	2.4
Mean Joint Faulting	(in)	0.007	0.006	0.005	0.032	0	0.003	0.001
Pass/Fail		Fail	Fail	Fail	Fail	Fail	Fail	Pass

3.5.4 Effects of Concrete Aggregate Type on Type II Pavement

This section presents the results of MEPDG analysis on the effects of concrete aggregate type on the expected performance of pavement with Type II design.

3.5.4.1 Brooksville Aggregate

Table 3-23 shows the predicted terminal distresses of concrete pavements of Type II design and using concrete containing Brooksville limestone aggregate and with a flexural strength of 700 psi. It can be seen that when the concrete slab thickness is 12 inches or less, the pavement is predicted to have failed at the end of 50 years. However, with a slab thickness of 13 inches, the predicted distresses are below the threshold values, and the pavement is considered adequate for 50-year design life.

Table 3-23. Predicted Terminal Distress of Type II Concrete Pavement Using Brooksville Aggregate

Brooksville Aggregate

Distress Predicted (E = 4766665 psi)

Pavement Distress		**Slab Thickness (in)**			
Type	**Measurement**	**10**	**11**	**12**	**13**
Terminal IRI	(in/mi)	63.7	63.5	63.3	66.6
Transverse Cracking	(% slabs cracked)	68.1	39	13.8	2.3
Mean Joint Faulting	(in)	0	0	0.001	0.004
Pass/Fail		Fail	Fail	Fail	Pass

3.5.4.2 Calera Aggregate

Table 3-24 shows the predicted terminal distresses of similar concrete pavements of Type II design and using concrete containing Calera limestone aggregate. It can be seen that when the concrete slab thickness is 15 inches or less, the pavement is predicted to have failed at the end of 50 years. However, with a slab thickness of 16 inches, the predicted distresses are below the threshold values, and the pavement would be considered adequate for 50-year design life.

Table 3-24. Predicted Terminal Distress of Type II Concrete Pavement Using Calera Aggregate

Calera Aggregate

Distress Predicted (E = 4981981)

Pavement Distress		**Slab Thickness (in)**						
Type	**Measurement**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
Terminal IRI	(in/mi)	102.2	70.9	81.6	64.1	72.1	63.7	63.2
Transverse Cracking	(% slabs cracked)	95.3	81.8	31.7	22.2	12.7	5.2	1.2
Mean Joint Faulting	(in)	0	0	0.008	0	0.009	0.002	0.001
Pass/Fail		Fail	Fail	Fail	Fail	Fail	Fail	Pass

3.5.4.3 River Gravel Aggregate

Table 3-25shows the predicted terminal distresses of similar concrete pavements of Type II design and using concrete containing river gravel. It can be seen that when the concrete slab thickness is 15 inches or less, the pavement is predicted to have failed at the end of 50 years.

However, with a slab thickness of 16 inches, the predicted distresses are below the threshold values, and the pavement would be considered adequate for 50-year design life.

Table 3-25. Predicted Terminal Distress of Type II Concrete Pavement Using River Gravel

River Gravel Aggregate

Distress Predicted (E = 4406326)

Pavement Distress		Slab Thickness (in)						
Type	**Measurement**	**10**	**11**	**12**	**13**	**14**	**15**	**16**
Terminal IRI	(in/mi)	147.7	71.9	64.4	90.2	84.2	76.8	80.8
Transverse Cracking	(% slabs cracked)	97.3	90.8	70.6	30.1	14.8	8.2	2.5
Mean Joint Faulting	(in)	0.006	0	0.003	0.025	0.015	0.02	0.028
Pass/Fail		Fail	Fail	Fail	Fail	Fail	Fail	Pass

3.5.5 Summary of Findings on Effects of Slab Thickness and Aggregate Type

Three typical concrete pavement designs used in Florida were evaluated using the MEPDG model to assess their suitability for use as long-life concrete pavement with a design life of 50 years. The factors evaluated include the concrete slab thickness and the aggregate used in the concrete. The results of the analysis show that the concrete slab thickness and aggregate used in the concrete have significant effects on the predicted performance.

The three aggregates used in the analysis included Brooksville limestone, Calera limestone and river gravel. For concrete with the same design flexural strength, Brooksville limestone was shown to have the best predicted performance, followed by Calera limestone and the river gravel is the worst. The better performance of the Brooksville aggregate is possibly due to the relatively low elastic modulus and low coefficient of thermal expansion of concrete made with Brooksville limestone.

Among the three Florida concrete pavement designs evaluated, there appeared to be only small differences in predicted performance. Table 3-26 shows the predicted terminal distresses of the passing concrete pavements according to MEPDG. When concrete with Brooksville aggregate was used, all three designs required a minimum slab thickness of 13 inches for a 50-

year design life. Type II design shows slightly less predicted cracking than the other two designs. Type I-B designs shows slightly lower predicted faulting and roughness than the other two designs.

Table 3-26. Predicted Terminal Distresses of Passing Concrete Pavements

		Type I-A			Type I-B			Type II		
Aggregate		Brooksville	Calera	River Gravel	Brooksville	Calera	River Gravel	Brooksville	Calera	River Gravel
Slab Thickness (inches)		13	15	16	13	15	16	13	16	16
Modulus of Elasticity (x 10^-6 psi)		4.8	5.0	4.4	4.8	5.0	4.4	4.8	5.0	4.4
Pavement Distress	**Terminal IRI (in/mi)**	74	63.4	63.6	62.7	65.4	62.1	66.6	63.2	80.8
	Transverse Cracking (% slabs cracked)	3.1	4.3	2.4	4.2	4.2	2.4	2.3	1.2	2.5
	Mean Joint Faulting (in)	0.013	0	0.001	0.001	0.001	0.001	0.004	0.001	0.028

Note: Flexural strength of concrete used = 650 psi

3.6 Evaluation of the Effects of Modulus of Rupture of Concrete on the Performance of Florida Concrete Pavement Designs

3.6.1 Analysis to Evaluate the Effects of Modulus of Rupture using the MEPDG Model

The MEPDG model was used to evaluate the effects of modulus of rupture of concrete on the performance of the three Florida concrete pavement designs, Type I-A, Type I-B and Type II. The concrete using Brooksville aggregate, which gave the best predicted performance as presented in Section 3.5, was used in the analysis. The coefficient of thermal expansion of the concrete used was 5.68 X 10^{-6}/ °F. The effects of the modulus of rupture of concrete on the performance of these three concrete pavement designs were evaluated by varying the modulus of rupture from 500 psi to 800 psi in increments of 100 psi.

Except for the inputs for the concrete material properties, the other inputs to the MEPDG model were similar to those used in the analysis as presented in Section 3.5.

3.6.2 Effects of Modulus of Rupture of Concrete on Type I-A and Type I-B Pavements

MEPDG analyses were performed on the concrete pavements using Type I-A and Type I-B design and Brooksville aggregate. Tables 3-27 and 3-28 show the predicted terminal distresses (at the end of 50-year period) of these concrete pavements with modulus of rupture of concrete of 500, 600, 700 and 800 psi. It is noted that the elastic modulus of the concrete changes as the strength of the concrete changes. The estimated elastic moduli (E) of concrete of different moduli of rupture (f_r) were determined by regression using Equation 3-28 (Tia et al., 1989).

$$E = 4.20(w^{1.5}) f_r \qquad (3\text{-}28)$$

Where:

E = Elastic modulus, psi,

w = Unit weight, pcf; and,

f_r = Modulus of rupture, psi.

The values of the elastic moduli used are also shown on Tables 3-27 and 3-28.

For Type I-A and Type I-B designs, it can be seen that when the modulus of rupture is less than or equal to 600 psi, all the concrete pavements with a slab thickness of 13 inches or less are predicted to have failed before the 50-year period. For the concrete pavement with a modulus of rupture of 700 psi, the pavement is predicted to be adequate at 50 years if the concrete slab thickness is 12 inches or higher. For the concrete pavement with a modulus of rupture of 800 psi, the pavement is predicted to be adequate at 50 years if the concrete slab thickness is 11 inches or more.

Table 3-27. Predicted Terminal Distresses of Type I-A Concrete Pavement with Different Modulus of Rupture

Brooksville Aggregate

Modulus of Rupture 500 psi (E=3,667,000 psi)

Pavement Distress		***Slab Thickness (in)***			
Type	***Measurement***	***10***	***11***	***12***	***13***
Terminal IRI	*(in/mi)*	*103.9*	*128.2*	*69.7*	*65.9*
Transverse Cracking	*(% slabs cracked)*	*99.2*	*97.1*	*91.7*	*78.7*
Mean Joint Faulting	*(in)*	*0*	*0*	*0*	*0*
Pass/Fail		*Fail*	*Fail*	*Fail*	*Fail*

Modulus of Rupture 600 psi (E=4,400,000 psi)

Pavement Distress		***Slab Thickness (in)***			
Type	***Measurement***	***10***	***11***	***12***	***13***
Terminal IRI	*(in/mi)*	*75.5*	*123.5*	*65.5*	*63.4*
Transverse Cracking	*(% slabs cracked)*	*87.9*	*68.7*	*39.5*	*6*
Mean Joint Faulting	*(in)*	*0*	*0.006*	*0*	*0*
Pass/Fail		*Fail*	*Fail*	*Fail*	*Fail*

Modulus of Rupture 700 psi (E=5,133,000 psi)

Pavement Distress		***Slab Thickness (in)***			
Type	***Measurement***	***10***	***11***	***12***	***13***
Terminal IRI	*(in/mi)*	*98.1*	*63.1*	*71.8*	*63.2*
Transverse Cracking	*(% slabs cracked)*	*40.2*	*15.7*	*2.3*	*1.6*
Mean Joint Faulting	*(in)*	*0.003*	*0*	*0.012*	*0*
Pass/Fail		*Fail*	*Fail*	*Pass*	*Pass*

Modulus of Rupture 800 psi (E=5,867,000 psi)

Pavement Distress		***Slab Thickness (in)***			
Type	***Measurement***	***10***	***11***	***12***	***13***
Terminal IRI	*(in/mi)*	*64.7*	*63.3*	*64*	*63*
Transverse Cracking	*(% slabs cracked)*	*6.9*	*1.5*	*0.3*	*0.1*
Mean Joint Faulting	*(in)*	*0.001*	*0*	*0.002*	*0*
Pass/Fail		*Fail*	*Pass*	*Pass*	*Pass*

Table 3-28. Predicted Terminal Distresses of Type I-B Concrete Pavement with Different Modulus of Rupture

Brooksville Aggregate

Modulus of Rupture 500 psi (E=3,667,000 psi)

Pavement Distress		Slab Thickness (in)			
Type	**Measurement**	**10**	**11**	**12**	**13**
Terminal IRI	(in/mi)	106.5	134.5	75.4	70.9
Transverse Cracking	(% slabs cracked)	103.3	100.2	99.9	87.7
Mean Joint Faulting	(in)	0.001	0	0.001	0.002
Pass/Fail		Fail	Fail	Fail	Fail

Modulus of Rupture 600 psi (E=4,400,000 psi)

Pavement Distress		Slab Thickness (in)			
Type	**Measurement**	**10**	**11**	**12**	**13**
Terminal IRI	(in/mi)	85.1	112.3	70.3	66.2
Transverse Cracking	(% slabs cracked)	98.9	80.2	69.4	59.8
Mean Joint Faulting	(in)	0.004	0.003	0	0.001
Pass/Fail		Fail	Fail	Fail	Fail

Modulus of Rupture 700 psi (E=5,133,000 psi)

Pavement Distress		Slab Thickness (in)			
Type	**Measurement**	**10**	**11**	**12**	**13**
Terminal IRI	(in/mi)	100.3	90.1	80.5	70.2
Transverse Cracking	(% slabs cracked)	50.3	25.3	24.3	5.6
Mean Joint Faulting	(in)	0.006	0.005	0.012	0.001
Pass/Fail		Fail	Fail	Fail	Pass

Modulus of Rupture 800 psi (E=5,867,000 psi)

Pavement Distress		Slab Thickness (in)			
Type	**Measurement**	**10**	**11**	**12**	**13**
Terminal IRI	(in/mi)	68.7	65.3	58.9	50.3
Transverse Cracking	(% slabs cracked)	10.5	9.6	2.3	1.1
Mean Joint Faulting	(in)	0.001	0.001	0.002	0
Pass/Fail		Fail	Fail	Pass	Pass

3.6.3 Effects of Modulus of Rupture of Concrete on Type II Pavement

Similar MEPDG Analyses were performed on the concrete pavements using Type II design and Brooksville aggregate, with the modulus of rupture of concrete varying from 500 psi to 800 psi in increments of 100 psi. Table 3-29 shows the predicted terminal distresses of these concrete pavements with modulus of rupture of concrete of 500, 600, 700 and 800 psi.

It can be seen that when the modulus of rupture is less than or equal to 600 psi, all the concrete pavements with a slab thickness of 13 inches or less are predicted to have failed before the 50-year period. For the concrete pavement with a modulus of rupture of 700 psi, the pavement is predicted to be adequate at 50 years if the concrete slab thickness is 12 inches or higher. For the concrete pavement with a modulus of rupture of 800 psi, the pavement is predicted to be adequate at 50 years if the concrete slab thickness is 11 inches or more.

3.6.4 Summary of Findings on Effects of Modulus of Concrete

According to the results of MEPDG analyses on the Florida concrete pavement designs, it can be seen that when the same aggregate is used in the concrete, increasing the modulus of rupture of the concrete will give improved performance and increased service life to the pavement.

Table 3-29. Predicted Terminal Distresses of Type II Concrete Pavement with Different Modulus of Rupture

Brooksville Aggregate

Modulus of Rupture 500 psi (E=3,667,000 psi)

Pavement Distress		Slab Thickness (in)			
Type	**Measurement**	**10**	**11**	**12**	**13**
Terminal IRI	(in/mi)	102.7	126.2	119.2	123
Transverse Cracking	(% slabs cracked)	89.2	98.3	93.2	80.2
Mean Joint Faulting	(in)	0.003	0.002	0.005	0.007
Pass/Fail		Fail	Fail	Fail	Fail

Modulus of Rupture 600 psi (E=4,400,000 psi)

Pavement Distress		Slab Thickness (in)			
Type	**Measurement**	**10**	**11**	**12**	**13**
Terminal IRI	(in/mi)	75.2	124.2	80.3	76.3
Transverse Cracking	(% slabs cracked)	89.3	70.3	56.8	6.8
Mean Joint Faulting	(in)	0.006	0.007	0.004	0.013
Pass/Fail		Fail	Fail	Fail	Fail

Modulus of Rupture 700 psi (E=5,133,000 psi)

Pavement Distress		Slab Thickness (in)			
Type	**Measurement**	**10**	**11**	**12**	**13**
Terminal IRI	(in/mi)	56.4	80.5	64.6	65.3
Transverse Cracking	(% slabs cracked)	39.9	14.7	4.7	1.7
Mean Joint Faulting	(in)	0.008	0.009	0.009	0.003
Pass/Fail		Fail	Fail	Pass	Pass

Modulus of Rupture 800 psi (E=5,867,000 psi)

Pavement Distress		Slab Thickness (in)			
Type	**Measurement**	**10**	**11**	**12**	**13**
Terminal IRI	(in/mi)	65.8	63.6	70.9	71.3
Transverse Cracking	(% slabs cracked)	6.7	1.6	0.5	0.4
Mean Joint Faulting	(in)	0.001	0	0.016	0.015
Pass/Fail		Fail	Pass	Pass	Pass

3.7 Evaluation of the Effects of Base Type on the Performance of Florida Concrete Pavement Designs

3.7.1 Analysis to Evaluate the Effects of Base Type using the MEPDG Model

The MEPDG model was used to evaluate the effects of base type and its associated parameters on the predicted performance of Florida concrete pavement designs, Types I-A, I-B and II.

3.7.1.1 Design Criteria

Similar to the previous analyses, the design life of the concrete pavements to be analyzed was set to be 50 years and the same failure criteria as described in Section 3.2.1 of this chapter were used. The same climatic conditions as described in Section 3.2.3 were used. The same traffic inputs as described in Section 3.4 were used in this analysis.

3.7.1.2 Concrete Slab and Joint

The concrete slab thickness was varied from 10 to 14 inches in the analysis. The same joint spacing of 15 ft was used. Doweled transverse joints using dowel diameter of 1.5 inches with dowel bar spacing of 12 inches c/c was used.

3.7.1.3 Concrete Properties

Typical properties of pavement concrete made with Brooksville limestone were used in this set of analyses. The following properties of concrete were used:

Unit weight = 145 pcf
Coefficient of thermal expansion = 5.68 X 10^{-6} /°F
Flexural strength = 650 psi
Elastic modulus = 4.77 X 10^{6} psi

3.7.1.4 Base Types

Three different types of base materials were evaluated in MEPDG analyses. They were (1) crushed stone, (2) permeable gravel, and (3) permeable asphalt(Type I-A)/non-permeable (Type I-B).

For the crushed stone base material, the following elastic modulus values were used:

- 20,000 psi
- 30,000 psi
- 40,000 psi

For the permeable gravel base material, the following elastic modulus values were used:

- 10,000 psi
- 15,000 psi
- 20,000 psi

For the asphalt base material, the following grades of asphalt were used:

- PG 64-22
- PG 70-22
- PG 76-22

Three different base layer thicknesses were used. For the Type I-A design, the three different base thicknesses used were 4, 6 and 8 inches. For the Type II design, the base thicknesses evaluated were 6, 8 and 10 inches.

Two different levels of erodibility of the base materials were used in the analyses. They were:

(1) High erodibility with an Erodibility Factor of 5
(2) Low erodibility with an Erodibility Factor of 1

Two conditions of friction between the concrete and the base layers were used in the analyses. They were:

(1) Zero friction
(2) Full friction

Table 3-12 shows the inputs used for the edge support and base properties for the three pavement designs analyzed. A widened slab of 13 feet was used in the analysis. Table 3-13 shows the strength properties of the granular permeable base used. The properties of the structural asphalt base material are shown in Table 3-14. The properties of the A-3 soil base are shown in Table 3-15. The properties of the LBR40 stabilized subgrade are shown in Table 3-16.

3.7.2 Effects of Base Type on Type I-A Pavement

Table 3-30 shows the predicted terminal distresses of concrete pavements of Type I-A design using concrete containing Brooksville limestone aggregate with a base thickness of 4 inches and varying the stiffness of the base materials (by varying the elastic modulus of the crushed stone and gravel, or by varying the grade of the asphalt in the asphalt base). A high erodibility factor of 5 and zero friction between the base and the concrete slab were used in the analyses. It can be seen that when the concrete slab thickness is 12 inches or less, the pavement is predicted to have failed at the end of 50 years for all base materials considered. However, with a slab thickness of 13 inches, the predicted distresses are below the threshold values, and the pavement would be considered adequate for 50-year design life with either crushed stone, permeable granular aggregate or permeable asphalt. The type of base material and the stiffness of the base material appear to have no significant effect on the predicted performance according to the results of the MEPDG analyses.

Tables 3-31 and 3-32 show the predicted terminal distresses of similar concrete pavements of Type I-A design with base thickness of 6 inches and 8 inches, respectively. It can be seen that the predicted performance of the pavement appears to have improved slightly with an increase in base thickness. For the designs with 6-inch base, two of the designs with a concrete slab thickness of 12 inches have predicted distresses below the threshold values. These two designs are (1) one using crushed stone base with an elastic modulus of 20,000 psi, and (2) one using permeable asphalt base using PG 64-22 asphalt. For the designs with 8-inch base, three of the designs with a concrete slab thickness of 12 inches have predicted distress below the threshold values. These three designs are (1) one using crushed stone base with an elastic modulus of 40,000 psi, (2) one using permeable asphalt base using PG 64-22 asphalt, and (3) one using permeable asphalt base using PG 70-22 asphalt.

Table 3-30. Predicted Distresses of Type I-A Concrete Pavement Using Brooksville Aggregate and a 4 inch Base Layer (with High Erodibility and No Friction)

Brooksville Aggregate PCC

		Asphalt Properties		PCC Slab Thickness (Inches)		
		Superpave Binder Grading				
	Modulus (psi)	Low Temp (*C)	High Temp (*C)	12	13	14
Crushed Stone	20000	N/A	N/A	(1) 63.1	65.1	66.1
				(2) ***12.6***	3.1	2.3
				(3) 0	0	0.004
	30000	N/A	N/A	73.7	63.1	63.2
				5.1	3.2	1.9
				0.011	0	0
	40000	N/A	N/A	68.6	61.1	63.1
				14.5	2.7	1.9
				0.004	0	0
Permeable Gravel	10000	N/A	N/A	80.9	63.4	63.2
				13.4	2.4	0.7
				0.012	0	0
	15000	N/A	N/A	64.1	63.5	63.5
				9	3	1
				0.001	0.001	0
	20000	N/A	N/A	65.4	63.5	64.5
				12.6	3.1	1.4
				0.002	0	0.002
Permeable Asphalt	N/A	-22	64	63.3	63.3	63.1
				6.2	3.4	0.3
				0	0	0
		-22	70	68.8	63.1	63.2
				6.2	3.4	0
				0.001	0	0
		-22	76	63.2	63.1	63.7
				6.2	3.4	0
				0	0.002	0.001

Note: (1) IRI (in/mi)
(2) Transverse Cracking (% Slabs Cracked)
(3) Joint Faulting (in)

Table 3-31. Predicted Distresses of Type I-A Concrete Pavement Using Brooksville Aggregate and a 6 inch Base Layer (with High Erodibility and No Friction)

Brooksville Aggregate PCC

		Asphalt Properties Superpave Binder Grading		PCC Slab Thickness (Inches)			
	Modulus (psi)	Low Temp (*C)	High Temp (*C)	11	12	13	14
Crushed Stone	20000	N/A	N/A	(1) 63.3 (2) ***38.8*** (3)0.014	63.2 2.5 0.002	63.1 3 0	62.1 1.6 0
	30000	N/A	N/A		63.1 ***7.7*** 0	63.1 3.2 0	63.1 1.6 0
	40000	N/A	N/A		63.1 ***5*** 0.029	63.2 2.9 0	63.1 1.8 0
Permeable Gravel	10000	N/A	N/A		63.2 ***9.6*** 0	63.8 2.1 0.001	63.2 1.3 0
	15000	N/A	N/A		63.1 ***10.3*** 0	63.1 2.5 0	63.1 1.5 0
	20000	N/A	N/A		63.1 ***8.9*** 0	63.1 3 0	63.2 1.6 0
Permeable Asphalt	N/A	-22	64	66.2 ***25.8*** 0.004	63.1 2.3 0.002	61.2 3.2 0	60.4 0.2 0
		-22	70		63.3 ***13.3*** 0	63.2 3.1 0	63.1 0.2 0
		-22	76		63.3 ***12.1*** 0	63.1 3.1 0	63.1 0.1 0

Note: (1) IRI (in/mi)
(2) Transverse Cracking (% Slabs Cracked)
(3) Joint Faulting (in)

Table 3-32. Predicted Distresses of Type I-A Concrete Pavement Using Brooksville Aggregate and a 8 inch Base Layer (with High Erodibility and No Friction)

Brooksville Aggregate PCC							
		Asphalt Properties Superpave Binder Grading		PCC Slab Thickness (Inches)			
	Modulus (psi)	Low Temp (*C)	High Temp (*C)	11	12	13	14
Crushed Stone	20000	N/A	N/A	(1)	63.2	62.8	63.1
				(2)	***8.5***	3.2	1.2
				(3)	0	0.013	0
	30000	N/A	N/A		63.2	62.7	61.5
					6.3	3.3	0.5
					0	0.001	0.002
	40000	N/A	N/A	63.2	63.2	63.2	63.6
				4.8	3.8	2.8	0.2
				0	0	0	0
Permeable Gravel	10000	N/A	N/A		64	63.3	71.8
					10.5	2.2	1.2
					0.001	0	0.014
	15000	N/A	N/A		77.4	63.1	64.6
					9.6	2.3	1.3
					0.011	0	0.002
	20000	N/A	N/A		63.1	63.1	68.6
					8.5	3.2	1.2
					0	0	0.009
Permeable Asphalt	N/A	-22	64	74.1	66.5	63.4	62
				12.1	3	2.1	1.8
				0.001	0.001	0.002	0.001
		-22	70	74.8	67.1	63.4	62.3
				13	3.7	2.5	1.2
				0.001	0.001	0.002	0.001
		-22	76		73.9	66.4	63.5
					11.9	2.9	0.2
					0.001	0.001	0

Note: (1) IRI (in/mi)
(2) Transverse Cracking (% Slabs Cracked)
(3) Joint Faulting (in)

3.7.3 Effects of Base Type on Type I-B Pavement

Table 3-33 shows the predicted terminal distresses of concrete pavements of Type I-B design using concrete containing Brooksville limestone aggregate with a base thickness of 4 inches and varying the asphalt concrete grade of the base material. Similarly, a high erodibility factor of 5 and zero friction were used in the analyses. It can be seen that when the concrete slab thickness is 12 inches or less, the pavement is predicted to have failed at the end of 50 years for all base materials considered. However, with a slab thickness of 13 inches, the predicted distresses are below the threshold values, and the pavement would be considered adequate for 50-year design life with either PG 70-22 or PG 76-22.

Table 3-33. Predicted Distresses of Type I-B Concrete Pavement Using a 4 inch Asphalt Concrete Base Layer (with High Erodibility and No Friction)

		Asphalt Properties Superpave Binder Grading		PCC Slab Thickness (Inches)				
	Modulus (psi)	Low Temp (*C)	High Temp (*C)	10	11	12	13	14
Asphalt Concrete	N/A	-22	64		(1)	69.4	66.7	65.4
					(2)	***7.3***	***6.9***	4.2
					(3)	0.002	0.001	0.001
		-22	70			64.4	62.7	60.9
						6.3	4.2	3.5
						0.002	0.001	0.001
		-22	76			61.3	60.5	59.2
						6.2	3.3	2
						0.001	0.001	0.001

Note: (1) IRI (in/mi)
(2) Transverse Cracking (% Slabs Cracked)
(3) Joint Faulting (in)

3.7.4 Effects of Base Type on Type II Pavement

Table 3-34 shows the predicted terminal distresses of concrete pavements of Type II design using concrete containing Brooksville limestone aggregate with a base thickness of 6 inches and varying the stiffness of the base materials. Similarly, a high erodibility factor of 5 and zero friction were used in the analyses. It can be seen that when the concrete slab thickness is 12 inches or less, the pavement is predicted to have failed at the end of 50 years for all base materials considered. However, with a slab thickness of 13 inches, the predicted distresses are below the threshold values, and the pavement would be considered adequate for 50-year design life with either crushed stone, permeable granular aggregate or permeable asphalt. The type of base material and the stiffness of the base material appear to have no significant effect on the predicted performance according to the results of the MEPDG analyses.

Tables 3-35 and 3-36 show the predicted terminal distresses of similar concrete pavements of Type II design with base thickness of 8 inches and 10 inches, respectively. It can be seen that the predicted performance of the pavement appears to have improved slightly with an increase in base thickness. For the designs with 8-inch base, three of the designs with a concrete slab thickness of 12 inches have predicted distresses below the threshold values. These three designs are (1) one using crushed stone base with an elastic modulus of 20,000 psi, (2) one using permeable gravel base with an elastic modulus of 20,000 psi, and (3) one using permeable asphalt base using PG 76-22 asphalt. For the designs with 10-inch base, three of the designs with a concrete slab thickness of 12 inches have predicted distress below the threshold values. These three designs are (1) one using permeable gravel base with an elastic modulus of 20,000 psi, (2) one using permeable asphalt base using PG 70-22 asphalt, and (3) one using permeable asphalt base using PG 76-22 asphalt.

Table 3-34. Predicted Distresses of Type II Concrete Pavement Using Brooksville Aggregate and a 6 inch Base Layer (with High Erodibility and No Friction)

Brooksville Aggregate PCC

		Asphalt Properties Superpave Binder Grading			PCC Slab Thickness (Inches)		
	Modulus (psi)	Low Temp (*C)	High Temp (*C)		12	13	14
Crushed Stone	20000	N/A	N/A	(1)	74.5	72.2	70.7
				(2)	***7***	2.6	2.4
				(3)	0.011	0.012	0.011
	30000	N/A	N/A		81.2	73.2	71.9
					14.5	3.6	1.8
					0.011	0.012	0.010
	40000	N/A	N/A		81	73.6	71.4
					14.4	4.1	1.2
					0.011	0.013	0.012
Permeable Gravel	10000	N/A	N/A		79.3	72.4	63.8
					11.9	2.6	0
					0.011	0.01	0.001
	15000	N/A	N/A		80.6	72.1	63.6
					13.6	2.3	0.2
					0.011	0.013	0.009
	20000	N/A	N/A		73.5	72.2	69.7
					5	2.5	2.1
					0.011	0.013	0.011
Permeable Asphalt	N/A	-22	64		76.9	66.5	63.7
					15.5	3.1	0
					0.001	0.001	0.001
		-22	70		68.9	66.5	65
					5.8	3.1	0
					0.001	0.001	0.001
		-22	76		68.9	66.7	62.6
					5.7	3.4	0.4
					0.001	0.001	0.001

Note: (1) IRI (in/mi)
(2) Transverse Cracking (% Slabs Cracked)
(3) Joint Faulting (in)

Table 3-35. Predicted Distresses of Type II Concrete Pavement Using Brooksville Aggregate and a 8 inch Base Layer (with High Erodibility and No Friction)

Brooksville Aggregate PCC

	Modulus (psi)	Asphalt Properties Superpave Binder Grading			PCC Slab Thickness (Inches)			
		Low Temp (*C)	High Temp (*C)		11	12	13	14
Crushed Stone	20000	N/A	N/A	(1)	83.1	75.3	70.8	64.7
				(2)	***17***	2.8	2.5	1.2
				(3)	0.009	0.008	0.005	0.001
	30000	N/A	N/A			80.3	73.4	63.4
						13.8	4.1	3.2
						0.01	0.012	0.001
	40000	N/A	N/A			80.2	71.9	69.2
						13.6	2.4	1.8
						0.01	0.012	0.011
Permeable Gravel	10000	N/A	N/A			73.3	72.2	63.9
						4.7	2.5	0
						0.011	0.013	0.001
	15000	N/A	N/A			80.5	72.5	63.7
						13.7	3	0
						0.011	0.013	0.001
	20000	N/A	N/A		81.1	72.3	69.8	63.5
					14.6	2.8	2.2	1.7
					0.011	0.012	0.003	0.002
Permeable Asphalt	N/A	-22	64			73	63.5	63.2
						10.9	2.3	2.1
						0.001	0.001	0.001
		-22	70			71.8	66.8	59.6
						5.4	3.4	2.8
						0.001	0.001	0.001
		-22	76		70.9	66.5	64.6	63.5
					8.3	3	2.6	2.1
					0.001	0.001	0.001	0.001

Note: (1) IRI (in/mi)
(2) Transverse Cracking (% Slabs Cracked)
(3) Joint Faulting (in)

Table 3-36. Predicted Distresses of Type II Concrete Pavement Using Brooksville Aggregate and a 10 inch Base Layer (with High Erodibility and No Friction)

Brooksville Aggregate PCC

	Modulus (psi)	Asphalt Properties Superpave Binder Grading		PCC Slab Thickness (Inches)				
		Low Temp (*C)	High Temp (*C)	10	11	12	13	14
Crushed Stone	20000	N/A	N/A		(1)	87.7	73.4	70.5
					(2)	***9.7***	2.1	1.2
					(3)	0.01	0.011	0.009
	30000	N/A	N/A			80.3	71.9	65.4
						13.9	2.6	2.3
						0.01	0.012	0.009
	40000	N/A	N/A			80.7	73.5	64
						14.5	2.4	2
						0.01	0.009	0.008
Permeable Gravel	10000	N/A	N/A			73.1	72.1	71.4
						4.6	2.5	1.3
						0.011	0.013	0.013
	15000	N/A	N/A			77.7	71.8	71.5
						10.5	2.3	1.6
						0.011	0.012	0.013
	20000	N/A	N/A		77.7	76.3	71.8	71.2
					10.7	3.2	2.5	1.4
					0.01	0.009	0.008	0.008
Permeable Asphalt	N/A	-22	64			71.7	66.2	62.3
						9.3	2.7	2.2
						0.001	0.001	0
		-22	70		74.8	65.2	62.5	59.5
					13	2.7	2	1.5
					0.001	0.001	0.001	0
		-22	76		73.9	67.1	65.7	60.7
					12	3.8	2.8	2.1
					0.001	0.001	0.002	0.001

Note: (1) IRI (in/mi)
(2) Transverse Cracking (% Slabs Cracked)
(3) Joint Faulting (in)

3.7.5 Effects of Erodibility Factor

In the analyses as presented in the previous sections, a high erodibility factor of 5 and zero friction between the base and the concrete slab were used. Additional MEPDG analyses were performed to see how the predicted performance would be affected if a low erodibility factor of 1 was used. Table 3-37 presents the predicted terminal distresses of concrete pavements of Type I-A design using concrete containing Brooksville limestone aggregate with a base thickness of 8 inches and varying the stiffness of the base materials. A low erodibility factor of 1 and zero friction between the base and the concrete slab were used in the analyses. The results in this table can be compared with the corresponding results in Table 3-32, where similar designs are analyzed except that a high erodibility factor of 5 was used there. It can be seen that the predicted performance as shown in Table 3-37 are not much different from those in Table 3-32. The erodibility factor appears to have minimal effects on the predicted performance from the MEPDG analyses.

Table 3-37. Predicted Distresses of Type I-A Concrete Pavement Using Brooksville Aggregate and a 8 inch Base Layer (with Low Erodibility and No Friction)

Brooksville Aggregate PCC

	Modulus (psi)	Asphalt Properties Superpave Binder Grading		PCC Slab Thickness (Inches)				
		Low Temp (*C)	High Temp (*C)	10	11	12	13	14
Crushed Stone	20000	N/A	N/A		(1)	63.3	63.1	61.1
					(2)	***8.5***	3.2	1.2
					(3)	0	0	0
	30000	N/A	N/A			63.1	63.5	60.2
						4.7	3.3	0.3
						0	0	0.001
	40000	N/A	N/A		62.2	64.1	63.1	59.8
					4.8	3.9	2.8	1.7
					0	0.001	0	0
Permeable Gravel	10000	N/A	N/A			65.6	63.7	60.8
						10.5	2.2	1.8
						0.002	0.001	0.001
	15000	N/A	N/A			67.5	63.1	59.6
						9.6	2.3	2
						0.003	0	0.001
	20000	N/A	N/A			64.1	63.4	61.6
						8.5	3.2	2.9
						0.001	0	0.001
Permeable Asphalt	N/A	-22	64		65.1	63.8	63.4	62.2
					13.5	3.2	3	2.7
					0	0	0.002	0.001
		-22	70		63.2	63.1	64	66
					12.4	3.1	3.1	2.8
					0	0	0.003	0.001
		-22	76			65.4	65.9	69.7
						12.1	3.1	2.7
						0	0.001	0.001

Note: (1) IRI (in/mi)
(2) Transverse Cracking (% Slabs Cracked)
(3) Joint Faulting (in)

3.7.6 Effects of Friction

In the analyses as presented in the previous sections, zero friction between the base and the concrete slab was used. Additional MEPDG analyses were performed to see how the predicted performance would be affected if a high friction factor of 5 was used. Table 3-38 presents the predicted terminal distresses of concrete pavements of Type I-A design using concrete containing Brooksville limestone aggregate with a base thickness of 8 inches and varying the stiffness of the base materials. A low erodibility factor of 1 and a friction factor of 5 were used in the analyses. The results in this table can be compared with the corresponding results in Table 3-37, where similar designs are analyzed except that no friction was used there. It can be seen that the predicted performance as shown in Table 3-38 are not much different from those in Table 3-37. The friction factor appears to have minimal effects on the predicted performance from the MEPDG analyses.

Table 3-38. Predicted Distresses of Type I-A Concrete Pavement Using Brooksville Aggregate and a 8 inch Base Layer (with Low Erodibility and Full Friction)

Brooksville Aggregate PCC

	Modulus (psi)	Asphalt Properties Superpave Binder Grading		PCC Slab Thickness (Inches)				
		Low Temp (*C)	High Temp (*C)	10	11	12	13	14
Crushed Stone	20000	N/A	N/A		(1)	64.6	72.7	63.6
					(2)	***7.1***	3.1	0
					(3)	0.002	0.013	0.001
	30000	N/A	N/A			71.9	72.6	63.3
						4.5	3.1	0.2
						0.01	0.012	0
	40000	N/A	N/A		78	65.3	69.9	71.6
					4.7	3.6	2.6	1.7
					0.003	0.002	0.01	0.013
Permeable Gravel	10000	N/A	N/A			63.2	72.4	63.1
						9.9	2.3	1.2
						0	0.014	0
	15000	N/A	N/A			76.5	63.3	68.9
						8.6	2.3	1.3
						0.011	0	0.009
	20000	N/A	N/A			63.3	72.7	71.4
						7.1	3.1	1.3
						0	0.013	0.013
Permeable Asphalt	N/A	-22	64		70.1	65.6	63.4	62.2
					7.2	1.8	1.5	1
					0.001	0.001	0.001	0.001
		-22	70		70.3	65.8	62.4	60.5
					7.6	2.1	1.5	0.9
					0.001	0.001	0.001	0.001
		-22	76			69.7	65.3	64.6
						6.8	1.6	1
						0.001	0.001	0.001

Note: (1) IRI (in/mi)
(2) Transverse Cracking (% Slabs Cracked)
(3) Joint Faulting (in)

3.7.7 Summary of Findings on the Effects of Base Type

MEPDG analyses were performed to evaluate the effects of (1) types of base material, (2) stiffness of the base material, (3) erodibility of the base material, and (4) friction between the concrete and base layer on the predicted performance of Type I-A, Type I-B and Type II concrete pavement designs used in Florida. The predicted performance of the pavement appears to have improved slightly with an increase in base thickness. However, the type of base material and the stiffness of the base material appear to have no significant effect on the predicted performance according to the results of the MEPDG analyses. Using different erodibility factor and friction factor for the base materials appear to have no significant effect on the predicted performance according to the results of the MEPDG analyses.

3.8 Evaluation of Effects of Incremental AADTT on Performance of Florida Concrete Pavement Designs

3.8.1 Analysis to Evaluate the Effects of Incremental AADTT

In all the analyses presented in the previous sections in this chapter, an AADTT (Annual Average Daily Truck Traffic) of 4000 was used as the initial traffic. The effects of increased truck traffic on the performance of the three Florida Concrete Pavement Designs were evaluated using the MEPDG model and are presented in this section.

3.8.1.1 Concrete Slab and Joint

A concrete slab thickness of 13 inches was used in the analysis. The same joint spacing of 15 ft was used. Doweled transverse joints using dowel diameter of 1.5 inches with dowel bar spacing of 12 inches c/c was used.

3.8.1.2 Concrete Properties

Typical properties of pavement concrete made with Brooksville limestone were used in this set of analyses. The following properties of concrete were used:

Unit weight = 145 pcf
Coefficient of thermal expansion = 5.68 X 10^{-6} /°F
Flexural strength = 650 psi
Elastic modulus = 4.77 X 10^{6} psi

3.8.1.3 Edge Support and Base Properties

Table 3-12 shows the inputs used for the edge support and base properties for the two pavement designs analyzed. In order to obtain conservative predictions, the base and subbase materials were modeled as highly erodible with Erodibility Factor of 5. Zero friction was chosen to model the interaction between the concrete and the base layers.

The permeable granular base in Type I-A and Type II designs was modeled to have an elastic modulus of 40,000 psi, a Poisson's ratio of 0.35 and a coefficient of lateral pressure of 0.5.

A widened slab of 13 feet was used in the analysis. The properties of the structural asphalt base material are shown in Table 3-14. The properties of the A-3 soil base are shown in Table 3-15. The properties of the LBR40 stabilized subgrade are shown in Table 3-16.

3.8.1.4 Traffic Inputs

The truck traffic inputs to the MEPDG model are shown in Table 3-39. Analyses were made using different initial Annual Average Daily Truck Traffic (AADTT) varying from 4,000 to 22,000. A 50-year design life was used. An annual traffic growth rate of 4% for all truck types was used. The same climatic conditions as described in Section 3.2.3 were used.

Table 3-39. Truck Traffic Inputs to the MEPDG Model

Initial two-way AADTT:	4,000 to 22,000
Number of lanes in design direction:	2
Percent of trucks in design direction (%):	50
Percent of trucks in design lane (%):	95
Operational speed (mph):	60

3.8.2 Effects of Incremental Truck Traffic on Type I-A Pavement

The result of this sensitivity analysis are presented in Table 3-40 which shows the predicted terminal distresses of the Type I-A pavement at the end of 50-year life for the various values of initial two-way AADTT. If one or more of the predicted distresses at the end of the design period exceed the acceptable threshold values, the analyzed pavement would be considered to have failed for the design period. It can be seen that the predicted performance of Type I-A design was satisfactory up to an initial AADTT of 18,000. When an initial AADTT of

19,000 was used, the predicted transverse cracking exceeded the threshold transverse cracking at 95% reliability.

Table 3-40. Predicted Distresses of Type I-A Concrete Pavement at the End of 50-Year Life

Traffic	Distress	Performance Criteria		
Initial Two-way AADTT		Threshold Distress	Threshold Distress at 95% Reliability	Distress Predicted
	Terminal IRI (in/mi)	172	123	64.9
4000	Transverse Cracking (% slabs cracked)	15	4.3	0.2
	Mean Joint Faulting (in)	0.12	0.034	0.003
	Terminal IRI (in/mi)	172	123	72.6
6000	Transverse Cracking (% slabs cracked)	15	4.3	0.5
	Mean Joint Faulting (in)	0.12	0.034	0.017
	Terminal IRI (in/mi)	172	123	75.3
8000	Transverse Cracking (% slabs cracked)	15	4.3	0.9
	Mean Joint Faulting (in)	0.12	0.034	0.004
	Terminal IRI (in/mi)	172	123	79.6
10000	Transverse Cracking (% slabs cracked)	15	4.3	1.4
	Mean Joint Faulting (in)	0.12	0.034	0.029
	Terminal IRI (in/mi)	172	123	86.6
15000	Transverse Cracking (% slabs cracked)	15	4.3	3
	Mean Joint Faulting (in)	0.12	0.034	0.04
	Terminal IRI (in/mi)	172	123	90.5
18000	Transverse Cracking (% slabs cracked)	15	4.3	4.3
	Mean Joint Faulting (in)	0.12	0.034	0.034
	Terminal IRI (in/mi)	172	123	91.8
19000	Transverse Cracking (% slabs cracked)	**15**	**4.3**	**4.8**
	Mean Joint Faulting (in)	0.12	0.034	0.047

3.8.3 Effects of Incremental Truck Traffic on Type I-B Pavement

The result of this sensitivity analysis are presented on Table 3-41 which shows the predicted terminal distresses of the Type I-B pavement at the end of 50-year life for the various values of initial two-way AADTT. It can be seen that the predicted performance of Type I-B design was satisfactory up to an initial AADTT of 17,000 . When an initial AADTT of 18,000 was used, the predicted transverse cracking exceeded the threshold transverse cracking at 95% reliability.

Table 3-41. Predicted Distresses of Type I-B Concrete Pavement at the End of 50-Year Life

Traffic	Distress	Performance Criteria		
Initial Two-way AADTT		Threshold Distress	Threshold Distress at 95% Reliability	Distress Predicted
4000	Terminal IRI (in/mi)	172	123	66.2
	Transverse Cracking (% slabs cracked)	15	4.3	3.2
	Mean Joint Faulting (in)	0.12	0.034	0.004
6000	Terminal IRI (in/mi)	172	123	80.2
	Transverse Cracking (% slabs cracked)	15	4.3	4.5
	Mean Joint Faulting (in)	0.12	0.034	0.007
8000	Terminal IRI (in/mi)	172	123	76.6
	Transverse Cracking (% slabs cracked)	15	4.3	5.9
	Mean Joint Faulting (in)	0.12	0.034	0.006
10000	Terminal IRI (in/mi)	172	123	84.7
	Transverse Cracking (% slabs cracked)	15	4.3	6.4
	Mean Joint Faulting (in)	0.12	0.034	0.029
15000	Terminal IRI (in/mi)	172	123	90.2
	Transverse Cracking (% slabs cracked)	15	4.3	3.3
	Mean Joint Faulting (in)	0.12	0.034	0.030
17000	Terminal IRI (in/mi)	172	123	93.4
	Transverse Cracking (% slabs cracked)	15	4.3	4.0
	Mean Joint Faulting (in)	0.12	0.034	0.032
18000	Terminal IRI (in/mi)	172	123	95.2
	Transverse Cracking (% slabs cracked)	**15**	**4.3**	**4.7**
	Mean Joint Faulting (in)	0.12	0.034	0.055

3.8.4 Effects of Incremental Truck Traffic on Type II Pavement

The result of this sensitivity analysis are presented on Table 3-42 which shows the predicted terminal distresses of the Type II pavement at the end of 50-year life for the various values of initial two-way AADTT. It can be seen that the predicted performance of Type II design was satisfactory up to an initial AADTT of 20,000. When an initial AADTT of 21,000 was used, the predicted transverse cracking exceeded the threshold transverse cracking at 95% reliability.

3.8.5 Summary of Findings on Effects of Incremental Truck Traffic

The MEPDG analysis results show that Type I-A concrete pavement design can have a 50-year service life with an initial AADTT up to 18,000, Type I-B can have a 50-year service life with an initial AADTT up to 17,000, while Type II design can have a 50-year service life with an initial AADTT up to 20,000. Table 3-43 presents the truck traffic data on roadways in Florida with the highest AADTT values, which were obtained from the 2011 FDOT GIS databank. It can be seen that the top ten highest AADTT values in this table range from 17,408 to 22,110. Thus, an AADTT of 17,000 to 20,000 represents typical high truck traffic in Florida.

Table 3-42. Predicted Distresses of Type II Concrete Pavement at the End of 50-Year Life

Traffic	**Distress**	**Performance Criteria**		
Initial Two-way AADTT		Threshold Distress	Threshold Distress at 95% Reliability	Distress Predicted
	Terminal IRI (in/mi)	172	123	69.6
4000	Transverse Cracking (% slabs cracked)	15	4.3	0.3
	Mean Joint Faulting (in)	0.12	0.034	0.013
	123			
	Terminal IRI (in/mi)	172	123	73.7
6000	Transverse Cracking (% slabs cracked)	15	4.3	0.5
	Mean Joint Faulting (in)	0.12	0.034	0.019
	Terminal IRI (in/mi)	172	123	76.7
8000	Transverse Cracking (% slabs cracked)	15	4.3	0.9
	Mean Joint Faulting (in)	0.12	0.034	0.025
	Terminal IRI (in/mi)	172	123	81.4
10000	Transverse Cracking (% slabs cracked)	15	4.3	1.5
	Mean Joint Faulting (in)	0.12	0.034	0.015
	Terminal IRI (in/mi)	172	123	86.7
15000	Transverse Cracking (% slabs cracked)	15	4.3	2.8
	Mean Joint Faulting (in)	0.12	0.034	0.033
	Terminal IRI (in/mi)	172	123	86.9
18000	Transverse Cracking (% slabs cracked)	15	4.3	3.8
	Mean Joint Faulting (in)	0.12	0.034	0.008
	Terminal IRI (in/mi)	172	123	92.8
20000	Transverse Cracking (% slabs cracked)	15	4.3	4.05
	Mean Joint Faulting (in)	0.12	0.034	0.032
	Terminal IRI (in/mi)	172	123	93.7
21000	Transverse Cracking (% slabs cracked)	**15**	**4.3**	**4.9**
	Mean Joint Faulting (in)	0.12	0.034	0.006

Table 3-43. 2011 FDOT GIS Data Showing Roadways with the Highest AADTT

No.	DIST	ROADWAY	DESC_FRM	DESC_TO	AADTT
1	4	86095000	Bridge No-860535	US 1/SR 5 SB	22110
2	4	93220000	Bridge No-930189	Bridge No-930499	21625
3	4	86070000	86095000/EB-I595	SR 736/DAVIE BLVD	20468
4	5	92130000	RAMP 92473001	N/A	20193
5	5	92130000	Bridge No-920094	RAMP 92473001	20193
6	4	86070000	Bridge No-860554	86095000/EB-I595	18972
7	4	86070000	SR 736/DAVIE BLVD	SR 842/BROWARD BLVD	18088
8	4	86070000	Bridge No-860530	Bridge No-860576	17952
9	4	86070000	SR 838/SUNRISE BLVD	Bridge No-860117	17816
10	4	86070000	Bridge No-860579	Bridge No-860554	17408
11	4	86070000	Bridge No-860531	Bridge No-860530	17340
12	4	86070000	Bridge No-860576	Bridge No-860579	17204
13	4	86070000	Bridge No-860117	Bridge No-860130	17160
14	4	86070000	Bridge No-860124	PALM BCH. CO. LN.	16767
15	4	93220000	N/A	10TH AVE N	16384
16	4	86070000	DADE CO. LN.	Bridge No-860529	15640
17	6	87260000	NW 58 ST	Bridge No-870964	15561
18	5	36210000	Bridge No-360022	Bridge No-360043	15535
19	7	10190000	Bridge No-100599	Bridge No-100601	15424
20	7	10190000	Bridge No-100697	Bridge No-100110	15240
21	7	10190000	Bridge No-100601	N/A	15232
22	2	72280000	Bridge No-720334	SR 5	15191
23	4	86070000	Bridge No-860130	Bridge No-860239	15040
24	4	86070000	SR 842/BROWARD BLVD	SR 838/SUNRISE BLVD	14787
25	5	36210000	Bridge No-360018	Bridge No-360022	14783
26	8	75470000	N/A	RAMP 161 SB ON	14496
27	5	36210000	Bridge No-360001	Bridge No-360063	14419
28	4	93220000	HYPOLUXO RD	CR 812/LANTANA RD	14368
29	1	16320000	HILLSBOROUGH CO LINE	ON RAMP TO I-4	14108
30	4	93220000	N/A	Bridge No-930189	14097

3.9 Predicted Service Lives of Concrete Pavements Using Type I-A, I-B and II Designs

MEPDG analyses were performed to determine the predicted service lives of concrete pavements using Type I-A, I-B and II designs. An initial two-way AADTT of 17,000, which represents high-volume truck traffic, was used in the analysis. The concrete properties and other pavement parameters used in these analyses are the same as those described in Section 3.8. Table 3-44 presents the predicted service lives for these three designs with pavement thickness varying from 10 to 14 inches. It can be seen that when the concrete slab thickness is 13 inches or more, the expected service of all three designs are 50 years or more. Among the three designs, Type II has the best predicted performance, followed by Type I-A and then Type I-B.

Table 3-44. Predicted Service Lives of Concrete Pavements Using Type I-A, I-B and II Designs

Slab Thickness (inch)	Type I-A	Type I-B	Type II
	Predicted Life (years)		
10	27	24	28
11	33	30	36
12	42	40	43
13	51	50	56
14	56	53	60

Note: Initial AADTT = 17,000
Concrete using Brookville aggregate used: modulus of rupture of concrete = 650 psi

CHAPTER 4
EVALUATION OF DRAINAGE OF FLORIDA CONCRETE PAVEMENT DESIGNS

4.1 Overview

This chapter presents (1) some background information on drainage in concrete pavements, (2) DRIP (Drainage Requirements in Pavements) 2.0 software, and the results of analyses using the DRIP 2.0 software to evaluate the adequacy of drainage of typical Florida concrete pavements.

4.2 Drainage in Concrete Pavements

Water in the pavement structure has long been recognized as a leading cause of distress. The presence of water, free and capillary held, in pavement layers, has been documented by a number of investigators. The sources of this water are numerous, some sources are site specific (groundwater) and other are common to any pavement structure. Among the later ones is the infiltration of rainfall through unsealed cracks and through the matrix of the upper pavement layers. A study performed by Grogan (1992) showed that up to 23% of rainfall can infiltrate a pavement structure during a particular rainstorm. Similar findings have been reported by Ridgeway (1976) and by Dempsey and Robnett (1979).

The entrapped water within the pavement layers accelerates the deterioration of the pavement structure by causing premature distress of the pavement. The mechanism by which the pavement layers deteriorate has been attributed to loss of support, weakening of the subgrade, pumping of the base and/or subgrade, etc.

Free water infiltrates through the cracks and joints of a PCC pavement. As the pavement deteriorates and cracks, the amount of free water that infiltrates the structure increases. Also, water can infiltrate the system through the longitudinal/shoulder joints, as well as, through shallow ditches and medians.

In order to reduce the amount of free water which infiltrates the structure, the sealing of joints and adequate drainage systems have been implemented in pavements throughout the United States. With proper maintenance, these systems can help control water infiltration as well as prolong the life of a pavement.

The most common approach to provide drainability has been to include a permeable layer within the pavement structure to permit the speedy removal of water percolating into the pavement layer. A typical drainage system is depicted in Figure 4-1. This drainable system consists of the following elements: A permeable base, a separator layer, and an edgedrain system.

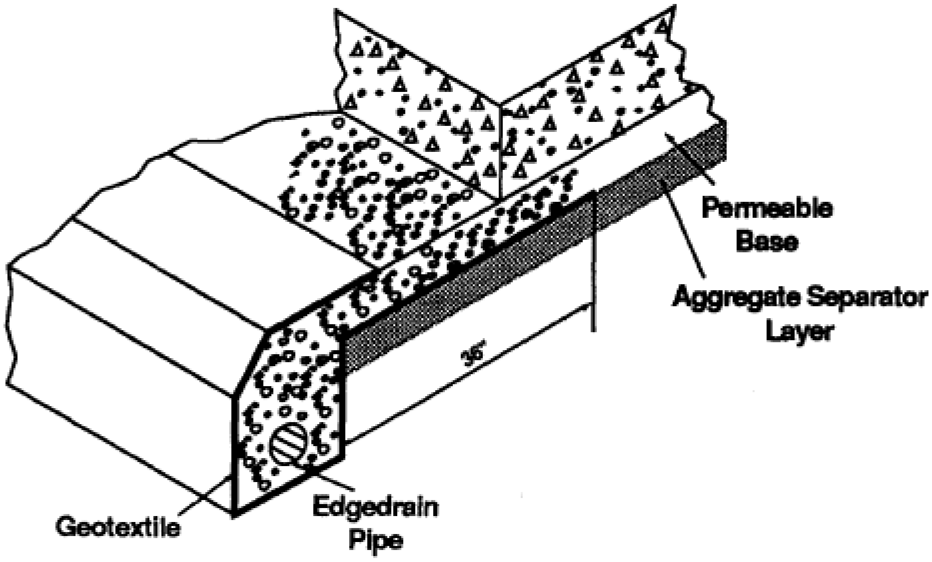


Figure 4-1. Drainable pavement system (FHWA, 1993).

4.2.1 Permeable Base

A permeable base must be permeable enough so that water can drain through it efficiently. The base course must have enough stability to support the pavement construction, and to provide the support necessary for the pavement structure.

Many states require 100-percent crushed stone with a maximum allowable L.A. Abrasion loss of 40 to 45 percent. FHWA (Federal Highway Administration) recommends that only crushed stone be used in permeable bases as it provides stability during the construction phase. Also, the soundness loss should not exceed 12 or 18 percent as determined by the sodium sulfate or magnesium sulfate tests, respectively.

There are two types of permeable bases, namely Unstabilized and Stabilized. Unstabilized bases have aggregate gradations that contain a small amount of finer-sized particles to facilitate load distribution due to the interlock of the aggregates. Stabilized bases are open-graded and thus more permeable.

To increase the permeability of unstabilized base materials, researchers have usually suggested the use of AASHTO No. 57 and 67 grade aggregates. The gradations of both of these aggregates have a 0-5% material passing No. 8 sieve. Aggregates of this gradation have lower strength and stiffness because of poor mechanical interlock between aggregates due to the lack of finer aggregates.

Stabilized base materials provide stability to the permeable base during pavement construction. The amounts of material passing the No. 8 or 16 screens are limited, thus ensuring high permeability. Stabilization of the base material can be done using asphalt at 2 to 2.5 percent by weight. When using Portland cement to stabilize, an application rate of 2 to 3 bags per cubic yard is usually recommended.

A large number of studies have been performed to analyze the minimum coefficient of permeability required in a drainable base, to permit the removal of excess water from the base layer. Table 4-1, shows the current permeability requirements used by different states for unbound permeable base layers.

Table 4-1. Minimum Permeability Requirements for Unbound Permeable Base Layers (Nazef, 2011).

State	Minimum Coefficient of Permeability Requirement (ft/day)
Florida	200 – 300 (recommended)
Texas	1000
New Jersey	>1000
Kansas	1000
Louisiana	1000

It is important to note, however, that the U.S. Department of Transportation specifies a permeability criterion of 3000 ft/day for stabilized permeable bases and a permeability criterion of 1000 ft/day for unstabilized bases. The latter, is the criterion under use by many of the state agencies today as observed in Table 4-1.

4.3 DRIP 2.0 Program

The DRIP (Drainage Requirements in Pavements) 2.0 program is a Windows-based software that is used for the subsurface drainage analysis of pavements. It was developed by FHWA and Applied Research Associates, Inc. to provide design guidance for handling water that infiltrated into the pavement structure from the surface. The following sections present the factors affecting drainage in a concrete pavement as considered by the DRIP 2.0 model.

4.3.1 Water Infiltration

The major sources of inflow into the pavement structure are surface infiltration, water flow from high ground, groundwater seepage, and meltwater from ice. In this case, only surface infiltration is considered in estimating the inflow as this is the predominant factor under Florida conditions. In the case of a high water table, the amount of groundwater seepage entering the permeable base may be a concern, but subsurface drainage layers are normally not installed as a corrective measure for groundwater seepage.

4.3.1.1 Crack Infiltration

The single largest source of infiltrated water in pavements enters through cracks and joints in the surface, cracks or joints between the pavement and shoulder, through the shoulders, and from side ditches. A new pavement may have a virtually impermeable surface, but well before the end of the design life, the pavement will likely contain unsealed cracks and joint openings. The design of the permeable base should be based on the cracked surface condition and should account for the total infiltration that could be expected.

According to the FDOT Rigid Pavement Design Manual, an infiltration rate of 0.7 ft^3/day/ft (28 cc/hr/cm) of joint is assumed, for an average storm duration of 10 hours and an average interval between storms of 100 hours. However, it is important to note that this infiltration rate applies only to a new pavement. As a pavement develops cracks and as joint seals deteriorate with age, the actual infiltration rate will be much higher.

4.3.1.2 Groundwater

Seasonal fluctuations of the water table (most commonly in spring and winter) can be a significant source of water (FHWA, 1993). Rarely is a pavement subsurface drainage system the most efficient way of handling water other than infiltrated free water (AASHTO, 1986). For this reason, groundwater will not be taken into account in the drainage analyses performed in this study.

4.3.2 Computation of Infiltrated Water

Two methods have been used extensively in evaluating surface infiltration: the infiltration ratio method (Cedergren et al., 1973) and the crack infiltration method (Ridgeway, 1976). The infiltration ratio method is highly empirical and depends on both the infiltration ratio and rainfall rate. The crack infiltration method is based on the results of infiltration tests. It has been found that the infiltration is directly related to cracking. Since the crack infiltration method is more rational and is based on field measurements, it is the method that was selected in this research.

4.3.2.1 Crack Infiltration Method

Ridgeway (1976) recommended an inflow rate estimated by the water-carrying capacity of a pavement crack or joint and by an estimated joint or crack length. Ridgeway's research indicated that the condition of the crack or joint (i.e., sealed or unsealed and debris filled, wide or narrow cracks or joints) and the type of base layer underlying the pavement surface (i.e., open-graded or dense-graded) both play a role in defining the infiltration capacity of the joint/crack. The design approach presented in the FHWA Drainage Manual (Moulton and Seals, 1979) uses the crack infiltration method, and Demonstration Project 87 and the National Highway Institute (NHI) Course No. 131026 present it as the preferred method. An equation to compute the infiltration rate for "normal" conditions of a pavement is:

$$q_i = I_C\left[\frac{N_C}{W} + \frac{W_C}{(WC_S)}\right] + k_p \tag{4-1}$$

where:

q_i = Rate of pavement infiltration, m^3/day/m^2 (ft^3/day/ft^2)
I_c = Crack infiltration rate, m^3/day/m (ft^3/day/ft)
N_c = Number of longitudinal cracks
W_c = Length of contributing transverse joints or cracks, m (ft)
W = Width of permeable base, m (ft)
C_s = Spacing of contributing transverse joints or cracks, m (ft)
k_p = Pavement permeability, m/day (ft/day)

The various dimensions of the pavement used in the equation are illustrated in Figure 4-2.

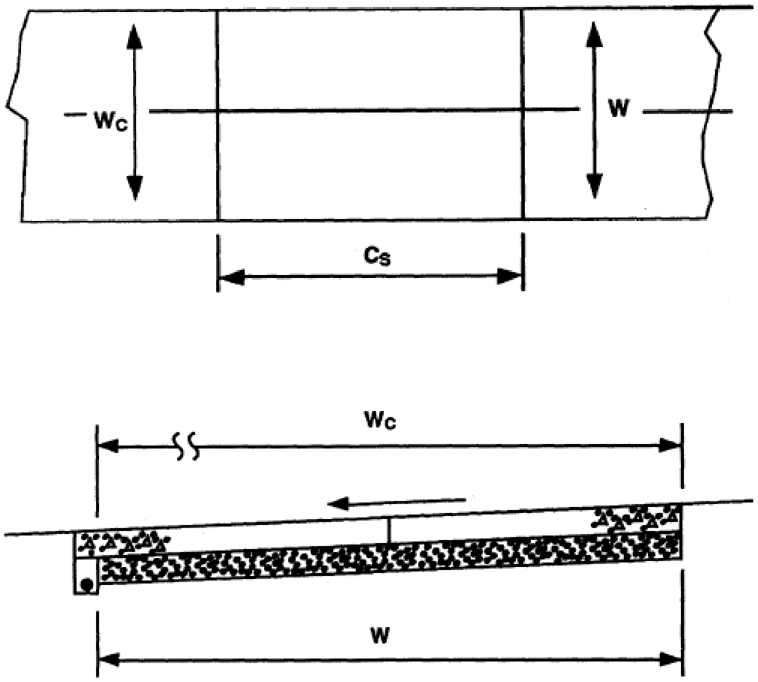


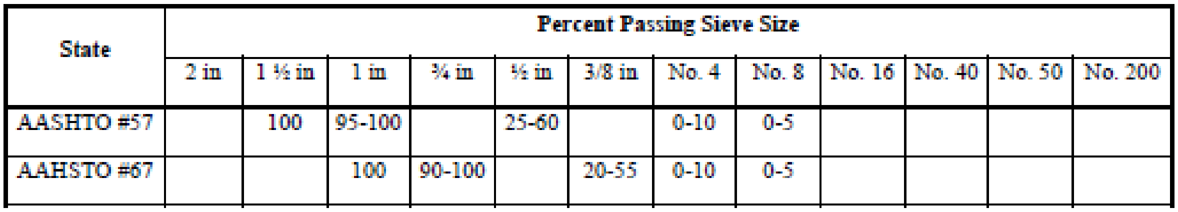
Figure 4-2. Plan and sectional view of a concrete pavement (FHWA, 1993).

4.3.3 Materials Requirement for Permeable Base

The quantification of drainage material parameters plays an important role in determining drainage capacity. Porosity and effective porosity define an aggregate material’s ability to store and give up water. Among all these drainage parameters, the coefficient of permeability is the most important in the quantification of the depth of flow in the permeable base.

The gradation of aggregates comprising the permeable base has the greatest influence on permeability. Typical gradation specifications for AASHTO No. 57 and 67 are shown in Table 4-2. The recommended minimum coefficient of permeability is 1000 ft/day for use in high-type highways. The AASHTO No. 57 and AASHTO No. 67 stones, whose gradations are shown in Table 4-2, are typically modified with either asphalt or cement and have shown to provide better structural adequacy due to the interlocking of aggregate.

Table 4-2. Typical Permeable Base Gradations. (FHWA, 1999)



State	Percent Passing Sieve Size											
	2 in	1 ½ in	1 in	¾ in	½ in	3/8 in	No. 4	No. 8	No. 16	No. 40	No. 50	No. 200
AASHTO #57		100	95-100		25-60		0-10	0-5				
AAHSTO #67			100	90-100		20-55	0-10	0-5				

4.3.4 Effects of Roadway Geometry

Geometric design decisions such as maximum and minimum slopes, pavement and shoulder interface, cross-sections, location of filter fabrics, overlap of fabrics, joints, separation layer location, trench dimensions, and so on are critical to pavement performance.

For concrete pavements, the permeable base is generally placed directly beneath the Portland cement concrete (PCC). A separator layer with critical drainage width (W) is placed between the permeable base and the subgrade to prevent fines from migrating into the permeable base. The total width of drainage path (W) can be computed by the following equation:

$$W = b + 2c \tag{4-2}$$

where:

W = Width of Drainage Path (ft)
b = Width of Surface (ft) [number of lanes; each lane being 12ft]
c = Distance from edge of surface to edge of base [shoulder = 3ft]

In designing the drainage of a permeable base, it is important to use the true slope and width of the permeable layer. When the longitudinal slope (S) is combined with the pavement cross slope (S_x), the true or resultant slope (S_R) of the flow path is determined by the equation:

$$S_R = (S^2 + S_x^2)^{1/2} \tag{4-3}$$

where:

S_R = Resultant slope, ft/ft
S = Longitudinal slope, ft/ft
S_x = Cross slope, ft/ft

The resultant length of the flow path is:

$$L_R = W\,[1 + (S/S_x)^2\,]^{1/2} \tag{4-4}$$

where,

L_R = Resultant length of flow path through permeable base, ft
W = Width of permeable base, ft

4.3.5 Design of Permeable Base

The permeable base should have a steady flow capacity equal to or greater than the inflow from the design rainfall. The solution for steady inflow, as developed by Moulton and Seals (1979), presents the required thickness of the permeable base as a function of permeability (k), resultant slope (S_R), length of drainage (L_R), and rate of uniform inflow (q_i). Figure 4-3 presents the relationship between k, S_R, L_R, q_i and the depth of flow (H). The base thickness should equal or exceed the computed depth of flow as obtained from Figure 4-3.

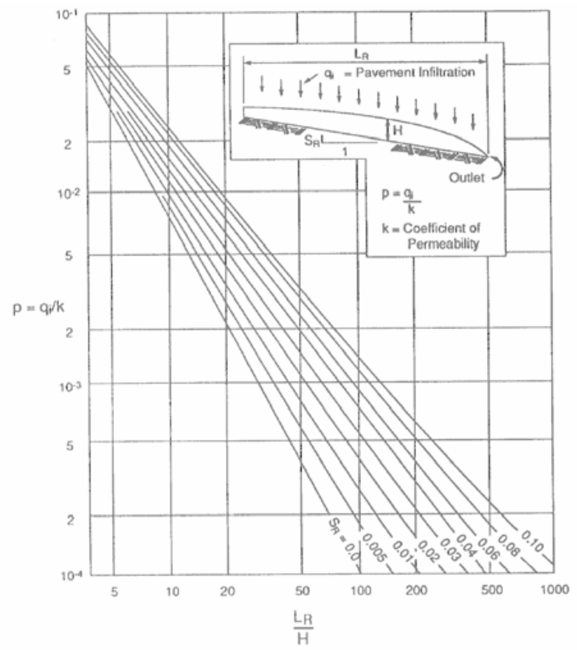


Figure 4-3. Moulton chart for depth-of-flow used in DRIP calculations (Moulton and Seals, 1979).

Under a steady-state flow condition, the following equations are used for the computation for depth of flow (H):

Case 1 where: (S^2 - 4q / k) < 0

$$H_1 = \sqrt{\frac{q_i}{k}} L_R \left[\left\{ \frac{S}{\sqrt{\frac{4q_i}{k - S^2}}} \right\} \left\{ \tan^{-1} \frac{S}{\sqrt{\frac{4q_i}{k - S^2}}} - \frac{\pi}{2} \right\} \right] \qquad (4\text{-}5)$$

Case 2 where: $(S^2 - 4q / k) > 0$

$$H_1 = \sqrt{\frac{q_i}{k}} L_R \left[\frac{S - \sqrt{S^2 - 4q_i / k}}{S + \sqrt{S^2 - 4q_i / k}} \right]^{\frac{S}{2\sqrt{S^2 - 4q_i / k}}} \qquad (4\text{-}6)$$

Case 3 where: $(S^2 - 4q / k) = 0$

$$H_1 = \sqrt{\frac{q_i}{k}} L_R^{-1} \qquad (4\text{-}7)$$

Where H_1 is the depth of water at the upper end of the flow path and is used as the H_{max} value, or the maximum depth of flow necessary for a particular design

Figure 4-4 illustrates how water can flow into a pavement through joints and cracks.

Figure 4-5 illustrates how the infiltrated can flow through a permeable base to an edge-drain.

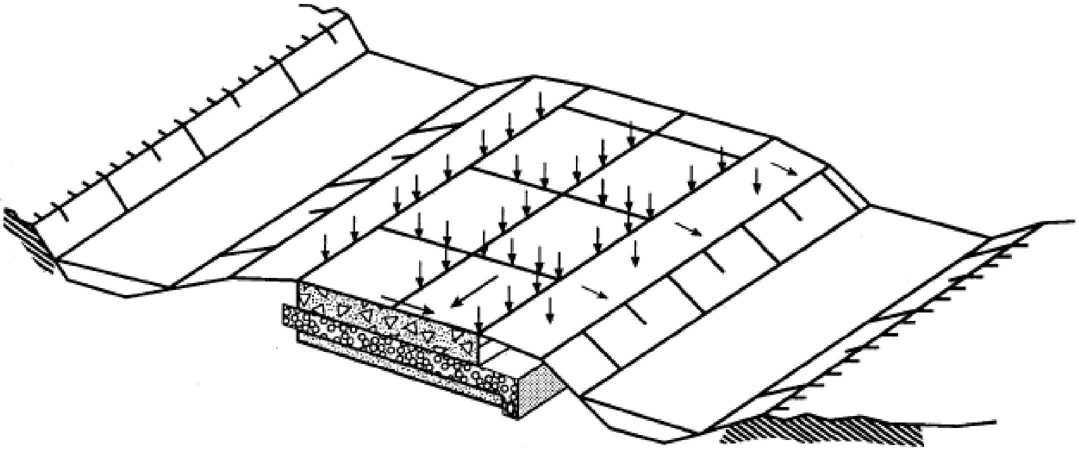


Figure 4-4. Infiltration of free-water (q_i) into PCC pavement with many lanes.

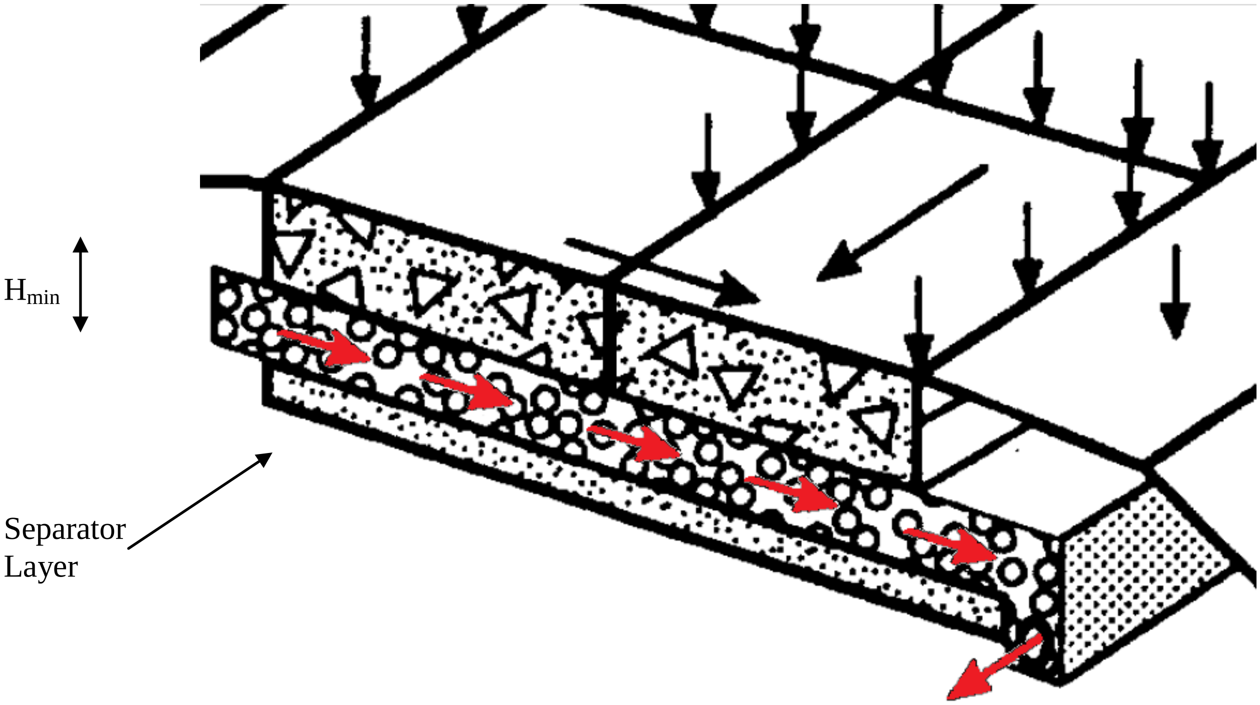


Figure 4-5. **Flow of free water (q_i) through permeable base of height (H_{min}).**

4.4 Steady-Flow Drainage Analysis

A sensitivity analysis was performed using the DRIP 2.0 program to evaluate the drainage characteristics of Type I-A and Type II concrete pavement designs using the steady flow method. The results of this analysis are presented in this section. It is to be noted that since Type I-B design does not have a permeable base, this type of analysis could not be appropriately applied to Type I-B design.

The sensitivity analysis was performed by varying (A) the crack infiltration rate from 0.7 to 2.4 ft^3/day/ft was used to represent the effects of aging and the evolution of cracks, (B) the width of draining path (from 2 to 4 lanes), (C) the pavement slope (from 2 to 6%), and (D) permeability of base material (from 200 to 1000 ft/day). The program computed the maximum depth of flow for the various combinations of conditions evaluated. The maximum computed depth of flow represents the minimum required base thickness for the drainage to be considered adequate.

The results of this sensitivity analysis on Type I-A design are presented in Tables 4-3 through 4-9, which show the required base thickness for the various combinations of these four parameters. The results of this sensitivity analysis on Type II design are presented in Tables 4-10 through 4-16.

It can be seen from these tables that the required base thickness decreases as the base permeability or the pavement slope increases, and the required base thickness increases as the crack infiltration rate or the number of lanes increases.

From Table 4-3, it can be seen that, for a 4-lane pavement with 4% slope and with a 4-inch base, the permeability of the base material has to be at least 400 ft/day if the crack infiltration rate is 0.7 ft^2/day. An infiltration rate of 0.7 ft^2/day applies to a new pavement with few cracks on it. For an older pavement with more cracks on it, the actual infiltration rate would be higher. If the infiltration rate is 1.04 ft^2/day, the required permeability of the base material would be 600 ft/day. If the infiltration **rate** is 2.4 ft^2/day, the required permeability of the base material would be 1300 ft/day.

From Table 4-4, it can be seen that, for a 3-lane pavement with 4% slope and with a 4-inch base, the permeability of the base material has to be at least 300 ft/day if the crack infiltration rate is 0.7 ft^2/day. If the infiltration rate is 1.04 ft^2/day, the required permeability of the base material would be 500 ft/day. If the infiltration rate is 2.4 ft^2/day, the required permeability of the base material would be 1000 ft/day.

From Table 4-5, it can be seen that, for a 2-lane pavement with 4% slope and with a 4-inch base, the required permeability of the base material is 200 ft/day if the crack infiltration rate is 0.7 ft^2/day. If the infiltration rate is 1.04 ft^2/day, the required permeability of the base material is 300 ft/day. If the infiltration rate is 2.4 ft^2/day, the required permeability of the base material would be 500 ft/day.

From Table 4-6, it can be seen that, for a 3-lane pavement with 2% slope and with a 4-inch base, the required permeability of the base material is 600 ft/day if the crack infiltration rate is 0.7 ft^2/day. If the infiltration rate is 1.04 ft^2/day, the required permeability of the base material is 800 ft/day. If the infiltration rate is 2.4 ft^2/day, the required permeability of the base material would be 1400 ft/day.

From Table 4-7, it can be seen that, for a 3-lane pavement with 3% slope and with a 4-inch base, the required permeability of the base material is 400 ft/day if the crack infiltration rate is 0.7 ft^2/day. If the infiltration rate is 1.04 ft^2/day, the required permeability of the base material is 600 ft/day. If the infiltration rate is 2.4 ft^2/day, the required permeability of the base material would be 1700 ft/day.

From Table 4-8, it can be seen that, for a 3-lane pavement with 5% slope and with a 4-inch base, the required permeability of the base material is 300 ft/day if the crack infiltration rate is 0.7 ft^2/day. If the infiltration rate is 1.04 ft^2/day, the required permeability of the base material is 400 ft/day. If the infiltration rate is 2.4 ft^2/day, the required permeability of the base material would be 800 ft/day.

From Table 4-9, it can be seen that, for a 3-lane pavement with 6% slope and with a 4-inch base, the required permeability of the base material is 200 ft/day if the crack infiltration rate is 0.7 ft^2/day. If the infiltration rate is 1.04 ft^2/day, the required permeability of the base material is 300 ft/day. If the infiltration rate is 2.4 ft^2/day, the required permeability of the base material would be 700 ft/day.

Table 4-3. Results of Steady-Flow Analysis on Type I-A Design Using 4 Lanes and 4% Slope

Type I-A – Steady Flow Method
Sensitivity Analysis (4 lanes; 4% slope)

Crack Infiltration Rate (I_c) (ft^3/day/ft)	**Rate Infiltration (q_i) (ft/day)**	**Permeability of Base (k)(ft/day)**	**Minimum Required Thickness of Permeable Base (H_{min}) (inches)**
0.7	0.1063	200	6.74
		300	4.73
		400	**3.66**
1.04	0.1579	200	9.43
		300	6.69
		400	5.21
		500	4.28
		600	**3.63**
1.38	0.2096	200	11.91
		300	8.51
		600	4.67
		700	4.08
		800	**3.62**
1.72	0.2612	200	14.22
		300	10.23
		600	5.67
		700	4.96
		800	4.40
		900	**3.96**
2.06	0.3128	200	16.41
		300	11.86
		600	6.63
		900	4.65
		1000	4.24
		1100	**3.89**
2.4	0.3644	200	18.49
		300	13.42
		600	7.56
		900	5.32
		1200	4.13
		1300	**3.84**

Table 4-4. Results of Steady-Flow Analysis on Type I-A Design Using 3 Lanes and 4% Slope

Type I-A – Steady Flow Method

Sensitivity Analysis (3 lanes; 4% slope)

Crack Infiltration Rate (I_c) (ft^3/day/ft)	**Rate Infiltration (q_i) (ft/day)**	**Permeability of Base (k)(ft/day)**	**Minimum Required Thickness of Permeable Base (H_{min}) (inches)**
0.7	0.1067	200	5.26
		300	**3.69**
1.04	0.1585	200	7.36
		300	5.22
		400	4.06
		500	**3.34**
1.38	0.2103	200	9.29
		300	6.64
		400	5.20
		500	4.28
		600	**3.65**
1.72	0.2621	200	11.09
		300	7.98
		400	6.27
		500	5.18
		600	4.43
		700	**3.87**
2.06	0.3139	200	12.80
		300	9.25
		400	7.30
		500	6.05
		600	5.17
		700	4.53
		800	4.03
		900	**3.63**
2.4	0.3657	200	14.42
		300	10.47
		400	8.28
		500	6.88
		600	5.90
		700	5.17
		800	4.60
		900	4.15
		1000	**3.79**

Table 4-5. Results of Steady-Flow Analysis on Type I-A Design Using 2 Lanes and 4% Slope

Type I-A – Steady Flow Method

Sensitivity Analysis (2 lanes; 4% slope)

Crack Infiltration Rate (I_c) (ft^3/day/ft)	Rate Infiltration (q_i) (ft/day)	Permeability of Base (k)(ft/day)	Minimum Required Thickness of Permeable Base (H_{min}) (inches)
0.7	0.1478	**200**	**2.97**
1.04	0.2196	200	4.12
		300	**2.95**
1.38	0.2913	200	5.17
		300	**3.73**
1.72	0.3631	200	6.15
		300	4.46
		400	**3.53**
2.06	0.4349	200	7.06
		300	5.15
		400	4.09
		500	**3.41**
2.4	0.5280	200	7.93
		300	5.81
		400	4.63
		500	**3.86**

Table 4-6. Results of Steady-Flow Analysis on Type I-A Design Using 3 Lanes and 2% Slope

Type I-A – Steady Flow Method

Sensitivity Analysis (3 lanes; 2% slope)

Crack Infiltration Rate (I_c) (ft^3/day/ft)	**Rate Infiltration (q_i) (ft/day)**	**Permeability of Base (k)(ft/day)**	**Minimum Required Thickness of Permeable Base (H_{min}) (inches)**
0.7	0.1067	200	8.12
		300	5.92
		400	4.70
		500	3.91
		600	**3.36**
1.04	0.1585	200	10.93
		300	8.06
		400	6.45
		500	5.40
		600	4.66
		700	4.11
		800	**3.68**
1.38	0.2103	200	13.43
		300	9.98
		400	8.03
		500	6.76
		600	5.85
		700	5.17
		800	4.64
		900	4.22
		1000	**3.86**
1.72	0.2621	200	15.71
		300	11.75
		400	9.49
		500	8.01
		600	6.96
		700	6.17
		800	5.55
		900	5.05
		1000	4.63
		1100	4.29
		1200	**3.99**

Table 4-6, continued.

Crack Infiltration Rate (I_c) (ft^3/day/ft)	Rate Infiltration (q_i) (ft/day)	Permeability of Base (k)(ft/day)	Minimum Required Thickness of Permeable Base (H_{min}) (inches)
2.06	0.3139	200	17.81
		300	13.38
		400	10.86
		500	9.19
		600	8.00
		700	7.10
		800	6.40
		900	5.83
		1000	5.36
		1100	4.96
		1200	4.63
		1300	4.33
		1400	4.08
		1500	**3.85**
2.4	0.3657	200	19.78
		300	14.92
		400	12.14
		500	10.30
		600	8.99
		700	7.99
		800	7.21
		900	6.58
		1000	6.05
		1100	5.61
		1200	5.23
		1300	4.91
		1400	4.62
		1500	4.37
		1600	4.14
		1700	**3.94**

Table 4-7. Results of Steady-Flow Analysis on Type I-A Design Using 3 Lanes and 3% Slope

Type I-A – Steady Flow Method

Sensitivity Analysis (3 lanes; 3% slope)

Crack Infiltration Rate (I_c)	**Rate Infiltration (q_i)**	**Permeability of Base**	**Minimum Required Thickness of Permeable Base (H_{min}) (inches)**
(ft^3/day/ft)	**(ft/day)**	**(k)(ft/day)**	
0.7	0.1067	200	6.41
		300	4.56
		400	**3.57**
1.04	0.1585	200	8.82
		300	6.35
		400	5.00
		500	4.14
		600	**3.54**
1.38	0.2103	200	11.01
		300	8.00
		400	6.33
		500	5.26
		600	4.51
		700	3.95
		800	**3.52**
1.72	0.2621	200	13.04
		300	9.53
		400	7.57
		500	6.31
		600	5.43
		700	4.77
		800	4.26
		900	3.85
		1000	**3.51**

Table 4-7, continued.

Crack Infiltration Rate (I_c) (ft^3/day/ft)	Rate Infiltration (q_i) (ft/day)	Permeability of Base (k)(ft/day)	Minimum Required Thickness of Permeable Base (H_{min}) (inches)
2.06	0.3139	200	14.93
		300	10.97
		400	8.75
		500	7.32
		600	6.30
		700	5.55
		800	4.96
		900	4.49
		1000	4.10
		1100	3.78
		1200	**3.51**
2.4	0.3657	200	16.72
		300	12.34
		400	9.87
		500	8.27
		600	7.14
		700	6.30
		800	5.64
		900	5.11
		1000	4.68
		1100	4.31
		1200	4.00
		1300	3.74
		1400	**3.50**

Table 4-8. Results of Steady-Flow Analysis on Type I-A Design Using 3 Lanes and 5% Slope

Type I-A – Steady Flow Method

Sensitivity Analysis (3 lanes; 5% slope)

Crack Infiltration Rate (I_c) (ft^3/day/ft)	**Rate Infiltration (q_i) (ft/day)**	**Permeability of Base (k)(ft/day)**	**Minimum Required Thickness of Permeable Base (H_{min}) (inches)**
0.7	0.1067	200	4.46
		300	**3.32**
1.04	0.1585	200	6.30
		300	4.42
		400	**3.42**
1.38	0.2103	200	8.02
		300	5.66
		400	4.40
		500	**3.60**
1.72	0.2621	200	9.64
		300	6.85
		400	5.34
		500	4.39
		600	**3.73**
2.06	0.3139	200	11.19
		300	7.99
		400	6.25
		500	5.14
		600	4.38
		700	**3.82**
2.4	0.3657	200	12.67
		300	9.08
		400	7.12
		500	5.88
		600	5.01
		700	4.38
		800	**3.88**

Table 4-9. Results of Steady-Flow Analysis on Type I-A Design Using 3 Lanes and 6% Slope

Type I-A – Steady Flow Method

Sensitivity Analysis (3 lanes; 6% slope)

Crack Infiltration Rate (I_c) (ft^3/day/ft)	**Rate Infiltration (q_i) (ft/day)**	**Permeability of Base (k)(ft/day)**	**Minimum Required Thickness of Permeable Base (H_{min}) (inches)**
0.7	0.1067	**200**	**3.86**
1.04	0.1585	200	5.49
		300	**3.82**
1.38	0.2103	200	7.04
		300	4.93
		400	**3.81**
1.72	0.2621	200	8.51
		300	5.99
		400	4.64
		500	**3.80**
2.06	0.3139	200	9.91
		300	7.01
		400	5.45
		500	4.47
		600	**3.79**
2.4	0.3657	200	11.27
		300	8.00
		400	6.23
		500	5.12
		600	4.35
		700	**3.78**

From Table 4-10, it can be seen that, for a 4-lane pavement with 4% slope, the permeability of the base material has to be at least 300 ft/day if the crack infiltration rate is 0.7 ft^2/day. An infiltration rate of 0.7 ft^2/day applies to a new pavement with few cracks on it. For an older pavement with more cracks on it, the actual infiltration rate would be higher. If the infiltration rate is 1.04 ft^2/day, the required permeability of the base material would be 400 ft/day. If the infiltration rate is 2.4 ft^2/day, the required permeability of the base material would be 800 ft/day.

From Table 4-11, it can be seen that, for a 3-lane pavement with 4% slope, the permeability of the base material has to be at least 200 ft/day if the crack infiltration rate is 0.7 ft^2/day. If the infiltration rate is 1.04 ft^2/day, the required permeability of the base material would be 300 ft/day. If the infiltration rate is 2.4 ft^2/day, the required permeability of the base material would be 600 ft/day.

From Table 4-12, it can be seen that, for a 2-lane pavement with 4% slope, the required permeability of the base material is 200 ft/day if the crack infiltration rate is 0.7 ft^2/day. If the infiltration rate is 1.04 ft^2/day, the required permeability of the base material is also 200 ft/day. If the infiltration rate is 2.4 ft^2/day, the required permeability of the base material would be 300 ft/day.

From Table 4-13, it can be seen that, for a 3-lane pavement with 2% slope, the required permeability of the base material is 300 ft/day if the crack infiltration rate is 0.7 ft^2/day. If the infiltration rate is 1.04 ft^2/day, the required permeability of the base material is 500 ft/day. If the infiltration rate is 2.4 ft^2/day, the required permeability of the base material would be 1100 ft/day.

From Table 4-14, it can be seen that, for a 3-lane pavement with 4% slope, the required permeability of the base material is 300 ft/day if the crack infiltration rate is 0.7 ft^2/day. If the infiltration rate is 1.04 ft^2/day, the required permeability of the base material is 400 ft/day. If the infiltration rate is 2.4 ft^2/day, the required permeability of the base material would be 800 ft/day.

From Table 4-15, it can be seen that, for a 3-lane pavement with 5%, the required permeability of the base material is 200 ft/day if the crack infiltration rate is 0.7 ft^2/day. If the infiltration rate is 1.04 ft^2/day, the required permeability of the base material is 300 ft/day. If the infiltration rate is 2.4 ft^2/day, the required permeability of the base material would be 500 ft/day.

From Table 4-16, it can be seen that, for a 3-lane pavement with 6% slope, the required permeability of the base material is 200 ft/day if the crack infiltration rate is 0.7 ft^2/day. If the infiltration rate is 1.04 ft^2/day, the required permeability of the base material is 200 ft/day. If the infiltration rate is 2.4 ft^2/day, the required permeability of the base material would be 500 ft/day.

In comparison, it can be seen that the Type II design with a 6-inch permeable base has a lower required base permeability as compared with the type I design with a 4-inch permeable bases. It can thus be stated that the Type II design has a better drainage condition than the Type I-A design if the same base permeability material, pavement slope and number of lanes are used in both designs.

Table 4-10. Results of Steady-Flow Analysis on Type II Design Using 4 Lanes and 4% Slope

Type II - Steady Flow Method

Sensitivity Analysis (4 lane; 4% slope)

Crack Infiltration Rate (I_c) (ft^3/day/ft)	**Rate Infiltration (q_i) (ft/day)**	**Permeability of Base (k)(ft/day)**	**Minimum Required Thickness of Permeable Base (H_{min}) (inches)**
0.7	0.1063	200	6.74
		300	**4.73**
1.04	0.1579	200	9.43
		300	6.69
		400	**5.21**
1.38	0.2096	200	11.91
		300	8.51
		400	6.66
		500	**5.49**
1.72	0.2612	200	14.22
		300	10.23
		400	8.04
		500	6.64
		600	**5.67**
2.06	0.3128	200	16.41
		300	11.86
		400	9.35
		500	7.75
		600	6.63
		700	**5.80**
2.4	0.3644	200	18.49
		300	13.42
		400	10.62
		500	8.82
		600	7.56
		700	6.62
		800	**5.90**

Table 4-11. Results of Steady-Flow Analysis on Type II Design Using 3 Lanes and 4% Slope

Type II - Steady Flow Method

Sensitivity Analysis (3 lane; 4% slope)

Crack Infiltration Rate (I_c) (ft^3/day/ft)	**Rate Infiltration (q_i) (ft/day)**	**Permeability of Base (k)(ft/day)**	**Minimum Required Thickness of Permeable Base (H_{min}) (inches)**
0.7	0.1067	**200**	**5.26**
1.04	0.1585	200	7.36
		300	**5.22**
1.38	0.2103	200	9.29
		300	6.64
		400	**5.20**
1.72	0.2621	200	11.09
		300	7.98
		400	6.27
		500	**5.18**
2.06	0.3139	200	12.80
		300	9.25
		400	7.30
		500	6.05
		600	**5.17**
2.4	0.3657	200	14.42
		300	10.47
		400	8.28
		500	6.88
		600	**5.90**

Table 4-12. Results of Steady-Flow Analysis on Type II Design Using 2 Lanes and 4% Slope

Type II - Steady Flow Method

Sensitivity Analysis (2 lane; 4% slope)

Crack Infiltration Rate (I_c) (ft^3/day/ft)	**Rate Infiltration (q_i) (ft/day)**	**Permeability of Base (k)(ft/day)**	**Minimum Required Thickness of Permeable Base (H_{min}) (inches)**
0.7	0.1478	**200**	**2.97**
1.04	0.2196	**200**	**4.12**
1.38	0.2913	**200**	**5.17**
1.72	0.3631	200	6.15
		300	**4.46**
2.06	0.4349	200	7.06
		300	**5.15**
2.4	0.5280	200	7.93
		300	**5.81**

Table 4-13. Results of Steady-Flow Analysis on Type II Design Using 3 Lanes and 2% Slope

Type II – Steady Flow Method **Sensitivity Analysis (3 lanes; 2% slope)**			
Crack Infiltration Rate (I_c) (ft^3/day/ft)	**Rate Infiltration (q_i) (ft/day)**	**Permeability of Base (k)(ft/day)**	**Minimum Required Thickness of Permeable Base (H_{min}) (inches)**
0.7	0.1067	200	8.12
		300	**5.92**
1.04	0.1585	200	10.93
		300	8.06
		400	6.45
		500	**5.40**
1.38	0.2103	200	13.43
		300	9.98
		400	8.03
		500	6.76
		600	**5.85**
1.72	0.2621	200	15.71
		300	11.75
		400	9.49
		500	8.01
		600	6.96
		700	6.17
		800	**5.55**

Table 4-13, continued.

Crack Infiltration Rate (I_c) (ft^3/day/ft)	Rate Infiltration (q_i) (ft/day)	Permeability of Base (k)(ft/day)	Minimum Required Thickness of Permeable Base (H_{min}) (inches)
2.06	0.3139	200	17.81
		300	13.38
		400	10.86
		500	9.19
		600	8.00
		700	7.10
		800	6.40
		900	**5.83**
2.4	0.3657	200	19.78
		300	14.92
		400	12.14
		500	10.30
		600	8.99
		700	7.99
		800	7.21
		900	6.58
		1000	6.05
		1100	**5.61**

Table 4-14. Results of Steady-Flow Analysis on Type II Design Using 3 Lanes and 3% Slope

Type II – Steady Flow Method

Sensitivity Analysis (3 lanes; 3% slope)

Crack Infiltration Rate (I_c) (ft^3/day/ft)	**Rate Infiltration (q_i)** (ft/day)	**Permeability of Base** (k)(ft/day)	**Minimum Required Thickness of Permeable Base (H_{min}) (inches)**
0.7	0.1067	200	6.41
		300	**4.56**
1.04	0.1585	200	8.82
		300	6.35
		400	**5.00**
1.38	0.2103	200	11.01
		300	8.00
		400	6.33
		500	**5.26**
1.72	0.2621	200	13.04
		300	9.53
		400	7.57
		500	6.31
		600	**5.43**
2.06	0.3139	200	14.93
		300	10.97
		400	8.75
		500	7.32
		600	6.30
		700	**5.55**
2.4	0.3657	200	16.72
		300	12.34
		400	9.87
		500	8.27
		600	7.14
		700	6.30
		800	**5.64**

Table 4-15. Results of Steady-Flow Analysis on Type II Design Using 3 Lanes and 5% Slope

Type II - Steady Flow Method

Sensitivity Analysis (3 lane; 5% slope)

Crack Infiltration Rate (I_c) (ft^3/day/ft)	**Rate Infiltration (q_i) (ft/day)**	**Permeability of Base (k)(ft/day)**	**Minimum Required Thickness of Permeable Base (H_{min}) (inches)**
0.7	0.1067	**200**	**4.46**
1.04	0.1585	200	6.30
		300	**4.42**
1.38	0.2103	200	8.02
		300	**5.66**
1.72	0.2621	200	9.64
		300	6.85
		400	**5.34**
2.06	0.3139	200	11.19
		300	7.99
		400	6.25
		500	**5.14**
2.4	0.3657	200	12.67
		300	9.08
		400	7.12
		500	**5.88**

Table 4-16. Results of Steady-Flow Analysis on Type II Design Using 3 Lanes and 6% Slope

Type II - Steady Flow Method

Sensitivity Analysis (3 lane; 6% slope)

Crack Infiltration Rate (I_c) (ft^3/day/ft)	**Rate Infiltration (q_i) (ft/day)**	**Permeability of Base (k)(ft/day)**	**Minimum Required Thickness of Permeable Base (H_{min}) (inches)**
0.7	0.1067	**200**	**3.86**
1.04	0.1585	**200**	**5.49**
1.38	0.2103	200	7.04
		300	**4.93**
1.72	0.2621	200	8.51
		300	**5.99**
2.06	0.3139	200	9.91
		300	7.01
		400	**5.45**
2.4	0.3657	200	11.27
		300	8.00
		400	6.23
		500	**5.12**

4.5 Time-to-Drain Drainage Analysis

A sensitivity analysis was performed using the DRIP 2.0 program to evaluate the time-to-drain drainage characteristics of Type I-A and Type II designs. This section presents the results from the analyses performed.

The sensitivity analysis was performed by varying (A) the width of draining path (from 2 to 4 lanes), (B) the pavement slope (from 2 to 6%), and (C) permeability of base material (ft/day). The program computed the time to drain for the various combinations of conditions evaluated. The time-to-drain in hours represents a specific time to obtain 50% drainage for a saturated base layer. FHWA rates the quality of drainage based on a scale that ranges from very poor (drains in more than 1 month), poor (from 7 days to 1 month), fair (from 1 to 7 days), good (from 2 to 24 hours) and excellent (less than 2 hours).

Equations 4-8 and 4-9 show the solutions for time to drain as utilized by the DRIP program.

Case 1: 0.5≤ U ≤1.0

$$T = 0.5S - 0.48S_1^{\,2} Log(1 + \frac{2.4}{S_1}) + 1.15S_1 Log\left[\frac{S_1 - US_1 + 1.2}{(1-U)(S_1 + 2.4)}\right] \qquad (4\text{-}8)$$

Case 2: 0≤ U ≤0.5

$$T = US - 0.48S_1^{\,2} Log(1 + \frac{4.8U}{S_1}) \qquad (4\text{-}9)$$

Where:

U = Percent drainage (expressed as a fraction, e.g., 1 percent = 0.01),
S_1 = Slope factor = H/DS,
S = Slope of granular layer, ft/ft,
T = Time factor = (tkH)/($n_e L^2$), hrs,

The results of time-to-drain analysis on Type I-A design are presented in Tables 4-17 through 4-31, which show the required time to drain for the various combinations of base permeability, pavement slope and number of lanes. The results of time-to-drain analysis on Type II design are presented in Tables 4-32 through 4-46.

It can be seen from these tables that the time to drain decreases (or drainage condition improves) as the base permeability or the pavement slope increases. The time to drain increases as the number of lane increases. In comparison, the Type II design with a 6-inch permeable base has a lower time to drain than the Type I-A design with a 4-inch permeable base if the same base permeability, pavement slope and number of lanes are used in both designs.

Table 4-17. Results of Time-to-Drain Analysis on Type I-A Design Using 4 Lanes and 2% Slope

Type I-A – Time-to-Drain Method

Sensitivity Analysis (4 lanes; 2% slope)

Permeability of Base (k)(ft/d)	Time to Drain (T)(hr)	Quality of Drainage
200	41.13	Fair
300	27.42	Fair
400	20.57	Good
500	16.45	Good
600	13.71	Good
700	11.75	Good
800	10.28	Good
900	9.14	Good
1000	7.48	Good
1500	5.48	Good
2000	4.11	Good
2500	3.29	Good
3000	2.74	Good
4000	**2.00**	**Excellent**

Table 4-18. Results of Time-to-Drain Analysis on Type I-A Design Using 4 Lanes and 3% Slope

Type I-A – Time-to-Drain Method

Sensitivity Analysis (4 lanes; 3% slope)

Permeability of Base (k)(ft/d)	Time to Drain (T)(hr)	Quality of Drainage
200	28.91	Fair
300	19.28	Good
400	14.46	Good
500	11.57	Good
600	9.64	Good
700	8.26	Good
800	7.23	Good
900	6.43	Good
1000	5.78	Good
1500	3.86	Good
2000	2.89	Good
2500	2.31	Good
2900	**1.99**	**Excellent**

Table 4-19. Results of Time-to-Drain Analysis on Type I-A Design Using 4 Lanes and 4% Slope

Type I-A – Time-to-Drain Method

Sensitivity Analysis (4 lanes; 4% slope)

Permeability of Base (k)(ft/d)	**Time to Drain (T)(hr)**	**Quality of Drainage**
200	22.30	Good
300	14.86	Good
400	11.15	Good
500	8.92	Good
600	7.43	Good
700	6.37	Good
800	5.57	Good
900	4.95	Good
1000	4.46	Good
1500	2.97	Good
2000	2.23	Good
2200	2.03	Good
2400	**1.94**	**Excellent**

Table 4-20. Results of Time-to-Drain Analysis on Type I-A Design Using 4 Lanes and 5% Slope

Type I-A – Time-to-Drain Method

Sensitivity Analysis (4 lanes; 5% slope)

Permeability of Base (k)(ft/d)	**Time to Drain (T)(hr)**	**Quality of Drainage**
200	18.19	Good
300	12.12	Good
400	9.09	Good
500	7.27	Good
600	6.06	Good
700	5.20	Good
800	4.55	Good
900	4.04	Good
1000	3.64	Good
1500	2.42	Good
1900	**1.91**	**Excellent**

Table 4-21. Results of Time-to-Drain Analysis on Type I-A Design Using 4 Lanes and 6% Slope

Type I-A – Time-to-Drain Method

Sensitivity Analysis (4 lanes; 6% slope)

Permeability of Base (k)(ft/d)	**Time to Drain (T)(hr)**	**Quality of Drainage**
200	15.35	Good
300	10.23	Good
400	7.67	Good
500	6.14	Good
600	5.12	Good
700	4.38	Good
800	3.84	Good
900	3.41	Good
1000	3.07	Good
1500	2.05	Good
1600	**1.92**	**Excellent**

Table 4-22. Results of Time-to-Drain Analysis on Type I-A Design Using 3 Lanes and 2% Slope

Type I-A – Time-to-Drain Method

Sensitivity Analysis (3 lanes; 2% slope)

Permeability of Base (k)(ft/d)	**Time to Drain (T)(hr)**	**Quality of Drainage**
200	30.74	Fair
300	20.49	Good
400	15.37	Good
500	12.29	Good
600	10.25	Good
700	8.78	Good
800	7.68	Good
900	6.83	Good
1000	6.15	Good
1500	4.1	Good
2000	3.07	Good
2500	2.46	Good
3000	**2.00**	**Excellent**

Table 4-23. Results of Time-to-Drain Analysis on Type I-A Design Using 3 Lanes and 3% Slope

Type I-A – Time-to-Drain Method
Sensitivity Analysis (3 lanes; 3% slope)

Permeability of Base (k)(ft/d)	Time to Drain (T)(hr)	Quality of Drainage
200	21.81	Good
300	14.54	Good
400	10.91	Good
500	8.73	Good
600	7.27	Good
700	6.23	Good
800	5.45	Good
900	4.85	Good
1000	4.36	Good
1500	2.91	Good
2000	2.18	Good
2200	**1.98**	**Excellent**

Table 4-24. Results of Time-to-Drain Analysis on Type I-A Design Using 3 Lanes and 4% Slope

Type I-A – Time-to-Drain Method
Sensitivity Analysis (3 lanes; 4% slope)

Permeability of Base (k)(ft/d)	Time to Drain (T)(hr)	Quality of Drainage
200	16.92	Good
300	11.28	Good
400	8.46	Good
500	6.77	Good
600	5.64	Good
700	4.83	Good
800	4.23	Good
900	3.76	Good
1000	3.38	Good
1500	2.26	Good
1700	**1.99**	**Excellent**

Table 4-25. Results of Time-to-Drain Analysis on Type I-A Design Using 3 Lanes and 5% Slope

Type I-A – Time-to-Drain Method

Sensitivity Analysis (3 lanes; 5% slope)

Permeability of Base (k)(ft/d)	**Time to Drain (T)(hr)**	**Quality of Drainage**
200	13.85	Good
300	9.23	Good
400	6.92	Good
500	5.54	Good
600	4.62	Good
700	3.96	Good
800	3.46	Good
900	3.08	Good
1000	2.77	Good
1400	**1.98**	**Excellent**

Table 4-26. Results of Time-to-Drain Analysis on Type I-A Design Using 3 Lanes and 6% Slope

Type I-A – Time-to-Drain Method

Sensitivity Analysis (3 lanes; 6% slope)

Permeability of Base (k)(ft/d)	**Time to Drain (T)(hr)**	**Quality of Drainage**
200	11.72	Good
300	7.81	Good
400	5.86	Good
500	4.69	Good
600	3.91	Good
700	3.35	Good
800	2.93	Good
900	2.60	Good
1000	2.34	Good
1200	**1.95**	**Excellent**

Table 4-27. Results of Time-to-Drain Analysis on Type I-A Design Using 2 Lanes and 2% Slope

Type I-A – Time-to-Drain Method

Sensitivity Analysis (2 lanes; 2% slope)

Permeability of Base (k)(ft/d)	Time to Drain (T)(hr)	Quality of Drainage
200	20.57	Good
300	13.71	Good
400	10.29	Good
500	8.23	Good
600	6.86	Good
700	5.88	Good
800	5.14	Good
900	4.57	Good
1000	4.11	Good
1500	2.74	Good
2000	2.06	Good
2100	**1.96**	**Excellent**

Table 4-28. Results of Time-to-Drain Analysis on Type I-A Design Using 2 Lanes and 3% Slope

Type I-A – Time-to-Drain Method

Sensitivity Analysis (2 lanes; 3% slope)

Permeability of Base (k)(ft/d)	Time to Drain (T)(hr)	Quality of Drainage
200	14.82	Good
300	9.88	Good
400	9.88	Good
500	5.93	Good
600	4.94	Good
700	4.24	Good
800	3.71	Good
900	3.29	Good
1000	2.96	Good
1500		**Excellent**

Table 4-29. Results of Time-to-Drain Analysis on Type I-A Design Using 2 Lanes and 4% Slope

Type I-A – Time-to-Drain Method
Sensitivity Analysis (2 lanes; 4% slope)

Permeability of Base (k)(ft/d)	Time to Drain (T)(hr)	Quality of Drainage
200	11.60	Good
300	7.73	Good
400	5.80	Good
500	4.64	Good
600	3.87	Good
700	3.31	Good
800	2.90	Good
900	2.58	Good
1000	2.32	Good
1200	**1.93**	**Excellent**

Table 4-30. Results of Time-to-Drain Analysis on Type I-A Design Using 2 lanes and 5% Slope

Type I-A – Time-to-Drain Method
Sensitivity Analysis (2 lanes; 5% slope)

Permeability of Base (k)(ft/d)	Time to Drain (T)(hr)	Quality of Drainage
200	9.55	Good
300	6.37	Good
400	4.78	Good
500	3.82	Good
600	3.18	Good
700	2.73	Good
800	2.39	Good
900	2.12	Good
1000	**1.91**	**Excellent**

Table 4-31. Results of Time-to-Drain Analysis on Type I-A Design Using 2 Lanes and 6% Slope

Type I-A – Time-to-Drain Method

Sensitivity Analysis (2 lanes; 6% slope)

Permeability of Base (k)(ft/d)	**Time to Drain (T)(hr)**	**Quality of Drainage**
200	8.11	Good
300	5.41	Good
400	4.06	Good
500	3.25	Good
600	2.70	Good
700	2.32	Good
800	2.03	Good
900	**1.80**	**Excellent**

Table 4-32. Results of Time-to-Drain Analysis on Type II Design Using 4 Lanes and 2% Slope

Type II – Time-to-Drain Method

Sensitivity Analysis (4 lanes; 2% slope)

Permeability of Base (k)(ft/d)	**Time to Drain (T)(hr)**	**Quality of Drainage**
200	15.74	Good
300	10.49	Good
400	7.87	Good
500	6.30	Good
600	5.25	Good
700	4.50	Good
800	3.93	Good
900	3.50	Good
1000	3.15	Good
1500	2.10	Good
1600	**1.97**	**Excellent**

Table 4-33. Results of Time-to-Drain Analysis on Type II Design Using 4 Lanes and 3% Slope

Type II – Time-to-Drain Method

Sensitivity Analysis (4 lanes; 3% slope)

Permeability of Base (k)(ft/d)	**Time to Drain (T)(hr)**	**Quality of Drainage**
200	13.28	Good
300	8.85	Good
400	6.64	Good
500	5.31	Good
600	4.43	Good
700	3.79	Good
800	3.32	Good
900	2.95	Good
1000	2.66	Good
1400	**1.90**	**Excellent**

Table 4-34. Results of Time-to-Drain Analysis on Type II Design Using 4 Lanes and 4% Slope

Type II – Time-to-Drain Method

Sensitivity Analysis (4 lanes; 4% slope)

Permeability of Base (k)(ft/d)	**Time to Drain (T)(hr)**	**Quality of Drainage**
200	11.39	Good
300	7.59	Good
400	5.70	Good
500	4.56	Good
600	3.80	Good
700	3.25	Good
800	2.85	Good
900	2.53	Good
1000	2.28	Good
1100	2.07	Good
1200	**1.90**	**Excellent**

Table 4-35. Results of Time-to-Drain Analysis on Type II Design Using 4 Lanes and 5% Slope

Type II – Time-to-Drain Method

Sensitivity Analysis (4 lanes; 5% slope)

Permeability of Base (k)(ft/d)	**Time to Drain (T)(hr)**	**Quality of Drainage**
200	10.07	Good
300	6.72	Good
400	5.04	Good
500	4.03	Good
600	3.36	Good
700	2.88	Good
800	2.52	Good
900	2.24	Good
1000	2.01	Good
1100	**1.83**	**Excellent**

Table 4-36. Results of Time-to-Drain Analysis on Type II Design Using 4 Lanes and 6% Slope

Type II – Time-to-Drain Method

Sensitivity Analysis (4 lanes; 6% slope)

Permeability of Base (k)(ft/d)	**Time to Drain (T)(hr)**	**Quality of Drainage**
200	9.04	Good
300	6.02	Good
400	4.52	Good
500	3.61	Good
600	3.01	Good
700	2.58	Good
800	2.26	Good
900	2.01	Good
1000	**1.81**	**Excellent**

Table 4-37. Results of Time-to-Drain Analysis on Type II Design Using 3 Lanes and 2% Slope

Type II – Time-to-Drain Method

Sensitivity Analysis (3 lanes; 2% slope)

Permeability of Base (k)(ft/d)	Time to Drain (T)(hr)	Quality of Drainage
200	10.40	Good
300	6.93	Good
400	5.20	Good
500	4.16	Good
600	3.47	Good
700	2.97	Good
800	2.60	Good
900	2.31	Good
1000	2.08	Good
1100	**1.89**	**Excellent**

Table 4-38. Results of Time-to-Drain Analysis on Type II Design Using 3 Lanes and 3% Slope

Type II – Time-to-Drain Method

Sensitivity Analysis (3 lanes; 3% slope)

Permeability of Base (k)(ft/d)	Time to Drain (T)(hr)	Quality of Drainage
200	8.97	Good
300	5.98	Good
400	4.48	Good
500	3.59	Good
600	2.99	Good
700	2.56	Good
800	2.24	Good
900	**1.99**	**Excellent**

Table 4-39. Results of Time-to-Drain Analysis on Type II Design Using 3 Lanes and 4% Slope

Type II – Time-to-Drain Method
Sensitivity Analysis (3 lanes; 4% slope)

Permeability of Base (k)(ft/d)	**Time to Drain (T)(hr)**	**Quality of Drainage**
200	7.89	Good
300	5.26	Good
400	3.95	Good
500	3.16	Good
600	2.63	Good
700	2.25	Good
800	**1.97**	**Excellent**

Table 4-40. Results of Time-to-Drain Analysis on Type II Design Using 3 Lanes and 5% Slope

Type II – Time-to-Drain Method
Sensitivity Analysis (3 lanes; 5% slope)

Permeability of Base (k)(ft/d)	**Time to Drain (T)(hr)**	**Quality of Drainage**
200	7.06	Good
300	4.71	Good
400	3.53	Good
500	2.83	Good
600	2.35	Good
700	2.02	Good
800	**1.77**	**Excellent**

Table 4-41. Results of Time-to-Drain Analysis on Type II Design Using 3 Lanes and 6% Slope

Type II – Time-to-Drain Method
Sensitivity Analysis (3 lanes; 6% slope)

Permeability of Base (k)(ft/d)	Time to Drain (T)(hr)	Quality of Drainage
200	6.04	Good
300	4.27	Good
400	3.20	Good
500	2.56	Good
600	2.13	Good
700	**1.83**	**Excellent**

Table 4-42. Results of Time-to-Drain Analysis on Type II Design Using 2 Lanes and 2% Slope

Type II – Time-to-Drain Method
Sensitivity Analysis (2 lanes; 2% slope)

Permeability of Base (k)(ft/d)	Time to Drain (T)(hr)	Quality of Drainage
200	5.85	Good
300	3.90	Good
400	2.93	Good
500	2.34	Good
600	**1.95**	**Excellent**

Table 4-43. Results of Time-to-Drain Analysis on Type II Design Using 2 Lanes and 3% Slope

Type II – Time-to-Drain Method

Sensitivity Analysis (2 lanes; 3% slope)

Permeability of Base (k)(ft/d)	Time to Drain (T)(hr)	Quality of Drainage
200	2.34	Good
300	3.46	Good
400	2.59	Good
500	2.08	Good
600	**1.73**	**Excellent**

Table 4-44. Results of Time-to-Drain Analysis on Type II Design Using 2 Lanes and 4% Slope

Type II – Time-to-Drain Method

Sensitivity Analysis (2 lanes; 4% slope)

Permeability of Base (k)(ft/d)	Time to Drain (T)(hr)	Quality of Drainage
200	4.66	Good
300	3.11	Good
400	2.33	Good
500	**1.87**	**Excellent**

Table 4-45. Results of Time-to-Drain Analysis on Type II Design Using 2 Lanes and 5% Slope

Type II – Time-to-Drain Method

Sensitivity Analysis (2 lanes; 5% slope)

Permeability of Base (k)(ft/d)	**Time to Drain (T)(hr)**	**Quality of Drainage**
200	4.24	Good
300	2.83	Good
400	2.12	Good
500	**1.70**	**Excellent**

Table 4-46. Results of Time-to-Drain Analysis on Type II Design Using 2 Lanes and 6% Slope

Type II – Time-to-Drain Method

Sensitivity Analysis (2 lanes; 6% slope)

Permeability of Base (k)(ft/d)	**Time to Drain (T)(hr)**	**Quality of Drainage**
200	3.89	Good
300	2.60	Good
400	**1.95**	**Excellent**

4.6 Drainage Requirements for Type I-A and Type II Designs

If a crack infiltration rate of 0.7 ft^3/day/ft is used, the required base permeability for Type I-A and Type II designs for the various combinations of pavement cross slope and number of lanes are summarized in Tables 4-47 and 4-48, respectively. The corresponding times to drain for the various conditions are also shown on these tables. In comparison, Type II design has better drainage performance than Type I-A design for the same combination of pavement slope and number of lanes used.

Table 4-47. Required Base Permeability and Time to Drain for Type I-A Design (with Crack Infiltration Rate of 0.7 ft^3/day/ft)

Cross-Slope	**Permeability Required (ft/day)** **[Time-to-Drain (hrs)]**		
	2-lane	3-lane	4-lane
2%	400 [20.6]	600 [10.25]	700 [11.75]
3%	300 [9.88]	400 [10.91]	500 [11.57]
4%	200 [11.6]	300 [11.3]	400 [11.2]
5%	200 [9.6]	300 [9.2]	300 [12.1]
6%	200 [8.1]	200 [11.7]	300 [10.2]

Table 4-48. Required Base Permeability and Time to Drain for Type II Design (with Crack Infiltration Rate of 0.7 ft^3/day/ft)

Cross-Slope	Permeability Required (ft/day) [Time-to-Drain (hrs)]		
	2-lane	3-lane	4-lane
2%	200 [5.85]	300 [6.93]	500 [6.30]
3%	200 [2.34]	300 [5.98]	400 [6.64]
4%	200 [4.7]	200 [7.9]	300 [7.6]
5%	100 [4.9]	200 [7.1]	200 [10.1]
6%	100 [3.1]	200 [6.0]	200 [9.0]

4.7 Drainage Issues for Type I-B Design

Type I-B concrete pavement has a 4-inch asphalt concrete base layer under the concrete slab. Since the asphalt concrete layer is a non-permeable layer, water does not drain well through it. Thus the steady-flow analysis and the time-to-drain analysis could not be appropriately performed on this type of pavement. In order for this type of pavement to not have serious issues with drainage, the following conditions must exist throughout the life of the pavement:

(1) The concrete layer remains well bonded to the asphalt concrete layer, so that little water would go between these two layers.

(2) The pavement has good surface drainage characteristics, so that most of the water would run off the surface instead of seeping through the concrete into the asphalt concrete layer.

(3) The asphalt concrete is resistant to stripping action, even if some water gets between the concrete and the asphalt layer.

In the new long-life concrete pavement design adopted by Washington DOT, a 4-inch dense-graded HMA (hot mix asphalt) is used as the base layer under a 13-inch concrete layer (Muench et al. 2012). Prior to the adoption of this new design, Washington DOT had used an asphalt-treated base which had shown stripping problems. However, no stripping or pumping problem has been reported with the use the HMA base layer.

Long-term monitoring of Type I-B pavement is needed to determine whether or not there will be drainage related issue with the use of asphalt concrete base in concrete pavement in Florida.

CHAPTER 5
EVALUATION OF FACTORS AFFECTING THE PERFORMANCE OF JOINTED PLAIN CONCRETE PAVEMENTS

5.1 Selection of LTPP Test Sections

5.1.1 LTPP Data Set Overview

To meet the needs for information on long-term performance of a wide range of pavement designs, the Long-Term Pavement Performance (LTPP) program was planned and launched in 1987 as a part of the Strategic Highway Research Program (SHRP). There are more than 2,500 pavement sections which have been monitored in the U.S. and Canada and are included in the LTPP database.

5.1.2 Classification of LTPP Test Sections

The 2500+ test sections in the LTPP program are classified as either General Pavement Studies (GPS) or Specific Pavement Studies (SPS). The data for the GPS test sections contain more general variables affecting the performance of pavements, while the data for the SPS test sections contains more specific variables. Thus, the data for the GPS test sections are more useful for the evaluation of the effects of different pavement variables on the pavement performance.

There are 800+ GPS test sections, which are classified into 18 different experimental sections, as shown in Table 5-1. The data from Experimental Section GPS-3 for Jointed Plain Concrete Pavements (JPCP) were used for the analysis as presented in this chapter.

Table 5-1. Classifications of GPS Test Sections

Experimental Section Designation	Type of Pavement
GPS-1	Asphalt Concrete (AC) Pavement on Granular Base
GPS-2	AC Pavement on Bound Base
GPS-3	Jointed Plain Concrete Pavement (JPCP)
GPS-4	Jointed Reinforced Concrete Pavement (JRCP)
GPS-5	Continuously Reinforced Concrete Pavement (CRCP)
GPS-6 (A,B,C,D,S)	Existing AC Overlay of AC Pavement
GPS-7 (A,B,C,D,F,R,S)	Existing AC Overlay on PCC Pavement
GPS-9	Unbonded PCC Overlay on PCC Pavement

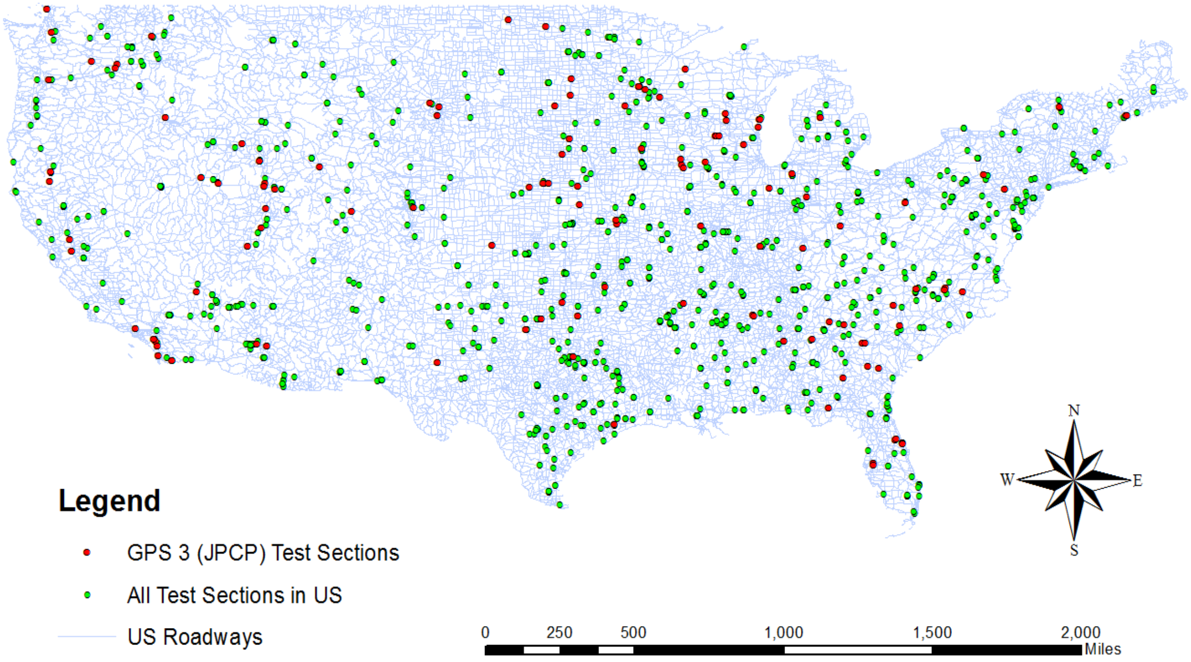


Figure 5-1. LTPP test sections in the U.S..

Figure 5-1 shows the locations of all the LTPP test sections in the United States. The locations of the GPS-3 (JPCP pavements) test sections are shown in red in this figure.

5.1.3 Selection of JPCP Test Sections for Analysis

In the LTPP database, the ride quality of the GPS test sections is characterized by the International Roughness Index (IRI), in units of meter per kilometer (m/km). An examination was made on the IRI data of the JPCP (GPS-3) Test Sections to see if the data was reasonable. The IRI data for each test section were plotted versus time to observe how IRI vary as a function of time, as shown in Figure 5-2 and 5-3. The plot in Figure 5-2 represents a reasonable trend as IRI generally increases with time as a pavement deteriorates. It was found that some of the IRI data show a decrease in IRI with time, as shown in Figure 5-3, which does not appear to be reasonable. The GPS-3 test sections with unreasonable IRI trend and missing traffic data were excluded from the analysis.

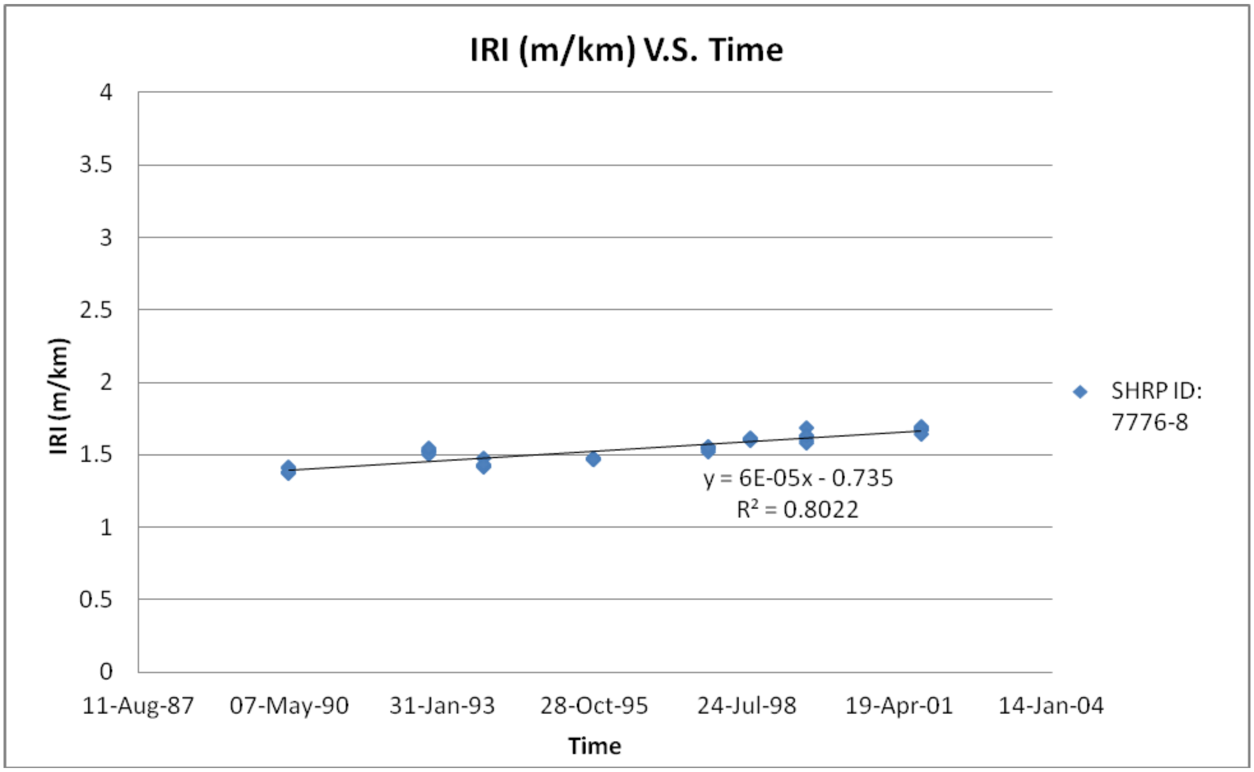


Figure 5-2. **Historical IRI data for test sections with reasonable trend.**

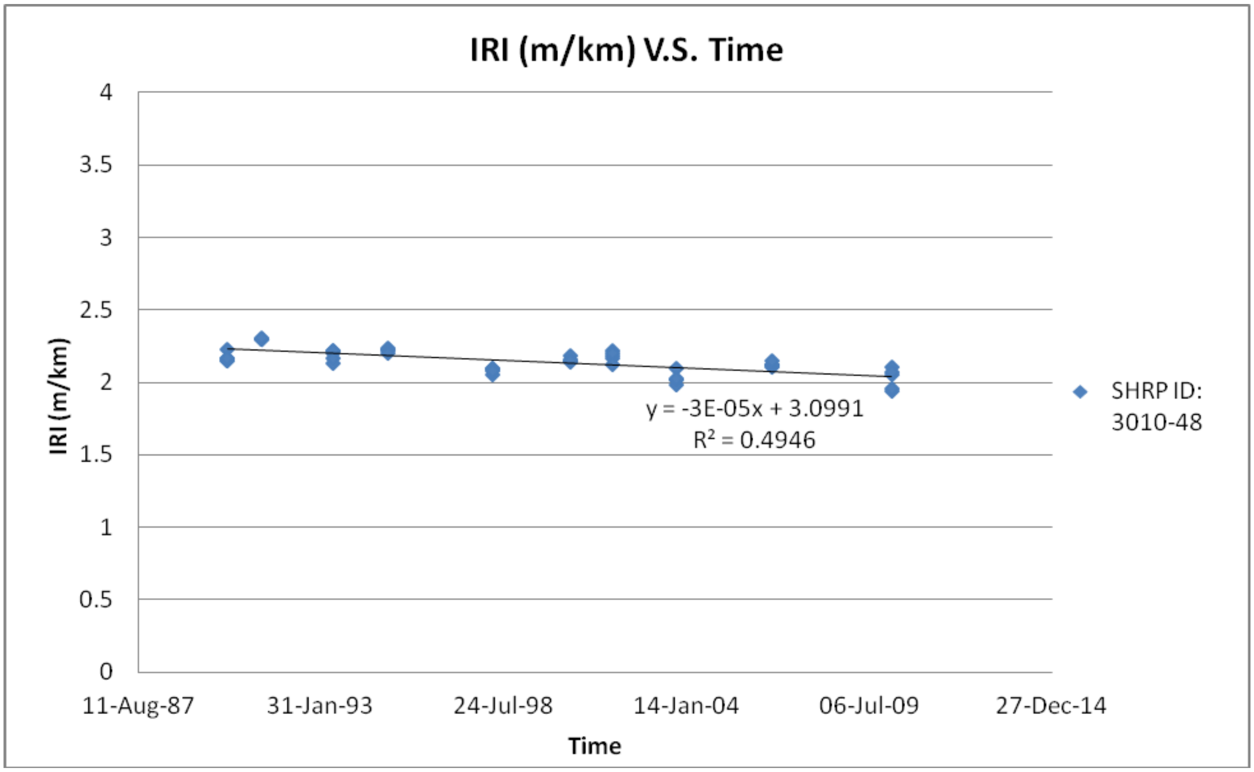


Figure 5-3. **Historical IRI data for test sections with unreasonable trend.**

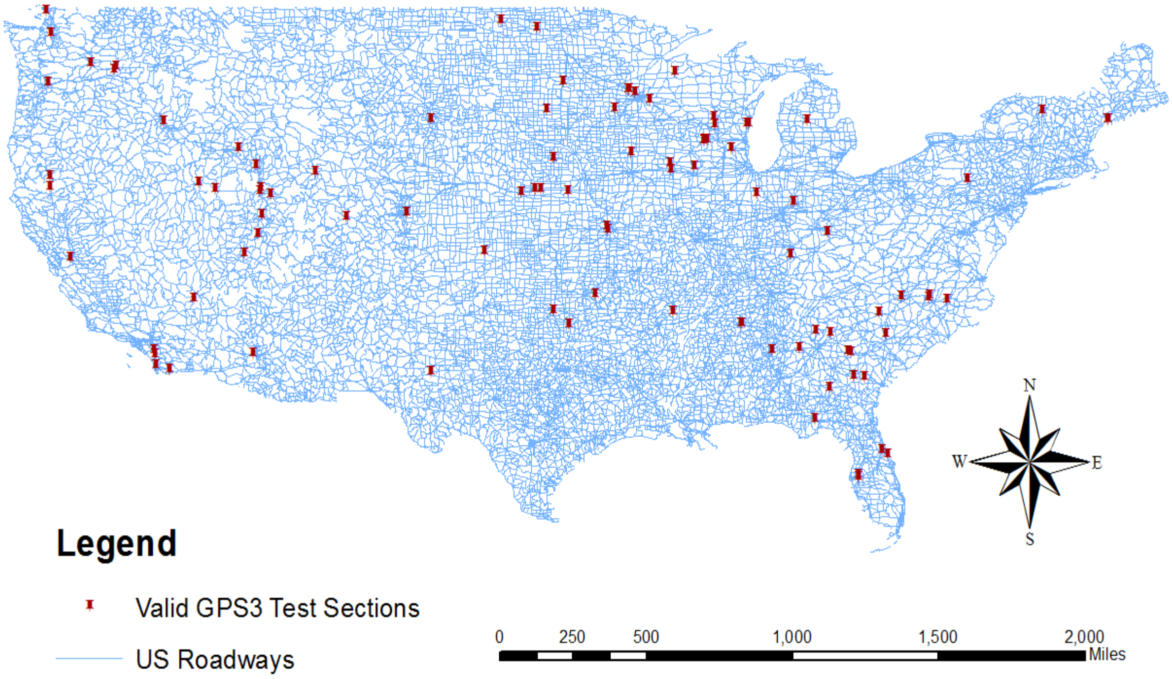


Figure 5-4. **Locations of valid GPS-3 test sections used in analysis.**

Out of 126 GPS-3 test sections, 97 test sections were found to have reasonable IRI trends versus time. Table 5-2 presents the information on these 97 test sections which are evaluated in this study. The locations of these 97 GPS-3 test sections are shown in Figure 5-4.

Table 5-2. Information on 97 Valid GPS Test Sections

SHRP ID	State Code	Average Annual Traffic[1] (× 10^3 ESAL)	IRI Deterioration Rate[2]	Slab Thickness (in)	Slab Length (ft)	Base Type[3]	Subgrade Type[4]	Drainage Type[5]	Average Temperature (˚C)	Days under 0˚C	Annual Precipitation (mm)	Wet Days
3052	46	14.6	576.89	9.0	20.0	303	109	1	7.5	164.5	572.0	109.4
3053	46	21.5	646.12	8.0	15.0	303	145	1	6.8	183.8	531.7	118.4
3012	55	24.0	847.10	7.0	15.0	303	216	1	7.2	157.2	771.7	132.1
3010	46	38.8	1226.70	9.5	15.0	302	216	1	6.6	167.0	589.6	98.7
3019	55	48.8	312.64	8.0	15.1	306	205	1	5.4	182.8	847.6	150.3
3013	39	63.3	903.12	8.0	17.0	339	108	1	11.9	110.6	1145.7	161.0
3030	18	65.2	571.01	8.0	15.5	319	114	1	10.9	115.1	987.1	148.9
3069	26	68.5	65.54	9.0	14.5	N/A	203	1	7.3	164.6	818.8	165.7
4138	12	71.0	687.85	8.0	20.0	339	214	1	21.6	4.8	1334.8	166.1
4059	12	71.4	89.95	6.0	14.0	303	202	2	21.9	3.6	1357.6	182.5
3020	13	72.9	49.10	9.0	20.0	339	216	1	18.7	33.7	1174.2	119.7
3028	1	91.2	754.81	10.0	20.0	303	216	4	16.3	63.6	1393.2	121.6
3010	35	95.8	32.37	8.0	13.5	308	202	1	16.6	77.4	396.4	68.2
3006	38	96.0	84.41	8.0	14.0	302	145	1	4.5	189.9	480.0	86.1
3019	13	98.1	201.01	9.0	20.0	308	114	1	15.1	63.0	1432.6	151.5
3010	55	115.7	1578.76	10.0	14.9	302	146	1	8.3	132.6	834.7	130.2
4160	40	124.6	292.88	9.0	15.0	320	108	1	16.4	63.4	1024.8	83.5
3060	20	138.4	432.56	10.0	15.0	334	102	1	13.0	99.5	987.5	136.0
3005	38	141.1	358.85	8.0	13.8	308	114	1	3.8	189.9	507.2	106.6
1682	50	141.5	567.49	8.0	N/A	302	215	7	7.2	153.8	889.6	186.5
3816	37	150.3	33.77	9.0	30.0	331	141	1	15.2	71.1	12.0	157.1
3006	19	156.5	702.01	8.5	20.0	331	114	1	9.8	131.0	141.1	198.3
3007	13	160.1	41.05	9.0	20.0	308	145	1	14.8	58.6	1451.3	109.7
3003	27	160.6	320.49	7.5	15.0	302	114	1	7.3	157.3	715.6	118.4
3015	20	173.5	122.02	9.0	15.0	319	102	1	12.1	142.5	492.5	79.7
3813	53	178.2	335.91	7.8	15.0	308	214	1	12.0	32.7	1234.8	182.7
3018	28	184.5	272.61	9.0	20.0	339	214	1	14.9	87.4	1544.2	130.5
3013	27	190.8	172.14	8.0	15.0	302	204	1	7.2	155.2	758.9	134.7
3008	37	201.8	99.46	8.0	21.3	332	215	1	15.4	69.2	1183.9	150.2
7086	49	229.0	326.75	10.0	12.5	334	267	1	11.7	105.6	446.8	109.2
3021	6	233.7	19.99	8.4	15.5	331	215	1	16.0	56.3	383.4	49.7
3033	31	239.6	265.12	9.0	15.5	319	214	1	9.5	150.6	657.6	96.0
3055	19	254.2	334.60	10.0	20.0	302	216	1	8.5	152.5	856.9	128.0
7085	49	258.2	18.80	10.0	12.5	334	265	1	7.5	192.2	488.6	110.0
3028	31	261.6	64.18	8.5	15.5	331	102	1	11.1	131.5	739.6	111.5
3014	55	269.3	186.94	10.0	15.5	304	267	1	8.9	140.7	885.9	151.1
6353	55	271.5	102.17	9.0	15.5	331	282	2	7.7	157.8	904.4	140.2
3009	55	272.2	1206.16	8.0	15.3	304	108	1	8.0	139.9	867.7	131.4
3013	20	279.9	86.07	10.0	15.0	332	102	1	13.0	97.7	1002.9	120.0
6351	55	293.5	78.35	9.0	15.5	303	265	2	7.7	157.9	907.2	132.6
3019	28	322.5	257.25	9.0	20.0	339	265	1	14.9	87.4	1544.1	130.3
3011	53	323.8	177.00	9.0	11.5	307	215	2	10.0	52.8	1151.1	162.9
6352	55	337.0	22.74	9.0	15.5	303	282	3	7.9	156.1	897.3	155.3
3016	55	339.8	15.15	9.0	15.5	302	204	1	7.5	157.2	830.2	139.6
3028	19	339.8	80.57	9.5	20.0	334	114	2	10.5	126.9	913.5	123.0
7409	53	353.3	130.18	9.0	11.5	304	255	1	11.1	121.9	212.0	75.4
3812	53	365.5	31.46	9.0	15.0	308	265	1	11.2	37.6	1012.6	178.3
7083	49	374.9	82.76	10.0	12.5	334	267	3	9.5	171.8	217.8	73.2

Table 5-2, continued.

3807	37	380.0	30.74	9.0	21.3	339	214	1	14.8	78.7	1136.0	142.1
3002	18	393.0	98.45	9.0	15.5	303	114	1	10.6	122.1	968.0	152.2
3009	19	397.8	34.32	10.0	20.0	331	216	2	9.3	135.4	884.7	126.2
3015	55	405.0	85.98	9.0	15.3	302	202	1	7.9	156.6	874.6	147.3
3017	16	421.5	58.88	10.0	14.5	321	143	7	9.2	150.8	315.7	87.9
3019	53	421.6	66.66	10.0	11.5	304	141	1	12.0	76.3	243.8	95.7
3011	5	434.0	24.39	10.0	15.0	331	133	1	16.1	62.2	1247.9	128.7
3014	53	449.3	29.24	10.0	11.5	308	214	1	11.9	80.2	243.7	94.9
4057	12	483.6	22.64	13.0	15.5	306	202	6	22.7	1.8	1256.2	147.6
3013	23	490.3	22.70	10.0	20.0	308	204	1	7.6	144.4	1175.5	145.9
3014	23	492.8	41.48	10.0	20.0	308	204	1	7.6	144.3	1176.1	145.0
3015	49	536.2	52.41	11.0	12.5	321	215	1	11.2	110.7	511.4	109.5
1623	42	544.6	12.73	9.0	20.0	334	217	2	10.3	118.2	1079.2	143.6
3032	8	573.3	7.07	8.0	15.5	334	255	1	9.1	173.5	348.9	95.2
4157	40	573.8	20.35	9.0	15.0	319	214	1	15.9	70.1	1110.0	122.3
3811	12	582.3	60.55	9.0	20.0	339	216	2	19.5	24.3	1500.2	156.5
3804	12	599.7	73.64	12.0	19.5	339	202	1	22.7	2.3	1276.4	149.9
3005	27	625.9	100.89	7.5	20.0	308	113	1	6.6	164.1	770.1	126.0
3007	27	625.9	29.16	7.5	19.7	321	113	2	6.6	164.1	770.1	126.0
3012	27	634.3	2.01	9.9	15.1	308	133	2	6.6	164.1	770.1	126.0
3011	13	701.2	19.26	10.0	20.0	319	214	4	18.5	34.6	1180.8	139.8
3013	32	702.6	74.81	8.0	15.5	331	255	1	10.8	127.0	133.0	62.1
7082	49	723.2	41.39	10.0	12.5	334	267	1	9.2	146.7	429.8	105.5
3015	13	760.1	44.18	10.0	20.0	339	202	2	19.1	27.6	1228.1	116.9
3023	16	800.0	5.79	9.0	13.5	304	214	1	10.6	131.8	281.6	98.8
3010	32	830.4	43.82	9.0	15.5	331	264	1	7.1	201.5	271.5	76.3
3011	49	836.3	82.49	10.5	15.0	331	267	1	10.3	154.1	347.3	89.6
3027	56	878.8	169.97	10.0	13.8	304	215	1	6.2	197.7	220.1	93.5
3016	21	919.6	4.41	11.0	15.0	303	111	1	14.1	88.0	1237.2	147.2
3010	6	977.3	3.47	9.0	15.5	331	217	1	17.7	0.4	287.1	46.6
7776	8	1017.1	22.97	10.5	13.0	307	113	1	10.1	159.9	419.0	101.8
3018	31	1038.3	91.05	12.0	15.5	339	202	1	10.1	150.3	620.6	90.6
3011	37	1040.5	6.67	10.0	30.0	321	216	7	15.7	65.2	1181.0	141.1
7614	4	1053.3	32.92	10.0	15.0	331	215	1	22.0	12.6	209.4	40.0
7456	6	1096.4	35.56	11.4	15.5	331	267	1	16.1	16.9	308.8	61.1
3044	37	1103.3	23.16	9.0	30.0	307	143	1	15.0	75.5	1173.2	157.7
3018	13	1154.4	23.71	10.0	19.5	331	216	4	16.7	55.2	1223.9	127.9
3017	13	1183.0	3.67	10.0	19.5	331	214	4	16.6	52.5	1222.6	138.7
3012	45	1195.6	3.97	10.0	21.5	334	145	1	16.7	54.7	1143.4	130.3
3023	31	1229.0	7.42	12.0	15.5	302	202	1	10.5	142.3	679.6	116.0
7493	6	1232.1	1.72	9.6	13.5	331	215	1	17.3	0.7	355.0	48.3
3018	40	1243.9	65.73	9.0	15.0	320	102	1	15.8	72.6	878.2	112.2
3010	49	1337.7	10.72	9.5	15.0	331	267	3	8.9	184.4	306.2	83.3
7084	32	1344.5	3.99	10.5	13.5	322	280	1	20.0	12.8	128.9	36.4
3030	6	1370.1	19.18	7.8	15.5	331	267	2	14.2	39.3	1327.4	114.1
3024	31	1511.3	14.01	14.0	15.5	307	102	1	10.5	141.4	714.8	109.8
3016	13	1590.9	9.41	11.0	20.0	319	214	2	16.3	59.3	1348.1	137.9
3019	6	1934.0	21.70	8.4	15.5	331	216	2	18.1	12.3	332.5	48.9
3005	6	2023.4	107.08	8.4	15.5	331	265	2	9.8	132.0	986.8	109.4

Note 1: Annual ESALs in thousands in the LTPP lane

Note 2: IRI Deterioration Rate = ΔIRI/year/annual ESALs × 10^6

Note 3: Index for drainage type

Code	Detail
1	No subsurface drainage
2	Longitudinal drains
3	Transverse drains
4	Drainage blanket
6	Drainage blanket with longitudinal drains
7	Other

Note 4: Index for base type

Code	Detail
302	Gravel (Uncrushed)
303	Crushed Stone
304	Crushed Gravel
306	Sand
307, 308	Soil-Aggregate Mixture
319	Fine-Grained Soils
320	Sand Asphalt
321	Asphalt Treated Mixture
322	Dense Graded, Hot Laid, Central Plant Mix
331	Cement Aggregate Mixture
332	Econocrete
334	Lean Concrete
339	Soil Cement

Note 5: Index for subgrade type

Code	Detail
102	Fine-Grained Soils: Lean Inorganic Clay
108	Fine-Grained Soils: Lean Clay with Sand
109	Fine-Grained Soils: Fat Clay with Sand
111	Fine-Grained Soils: Gravelly Lean Clay
113	Fine-Grained Soils: Sandy Clay
114	Fine-Grained Soils: Sandy Lean Clay
133	Fine-Grained Soils: Silty Clay with Sand
141	Fine-Grained Soils: Silt
143	Fine-Grained Soils: Silt with Sand
145	Fine-Grained Soils: Sandy Silt
146	Fine-Grained Soils: Gravelly Silt with Sand
202	Coarse-Grained Soils: Poorly Graded Sand
203	Coarse-Grained Soils: Poorly Graded Sand with Gravel
204	Coarse-Grained Soils: Poorly Graded Sand with Silt
205	Coarse-Grained Soils: Poorly Graded Sand with Silt and Gravel
214	Coarse-Grained Soil: Silty Sand
215	Coarse-Grained Soil: Silty Sand with Gravel
216	Coarse-Grained Soil: Clayey Sand
217	Coarse-Grained Soil: Clayey Sand with Gravel
255	Coarse-Grained Soil: Poorly Graded Gravel with Silt and Sand
264	Coarse-Grained Soil: Silty Gravel
265	Coarse-Grained Soil: Silty Gravel with Sand
267	Coarse-Grained Soil: Clayey Gravel with Sand
280	Rock and Stone
282	Rock

5.2 Evaluation of Effects of Various Design and Environment Factors on Performance of GPS-3 (JPCP) Test Sections

5.2.1 Pavement Performance Indicator

To evaluate the effect of various factors on performance of valid GPS-3 test sections, a IRI deterioration rate was used as a performance indicator for the test sections. IRI was used as a performance indicator since it is directly related to the serviceability of pavements. The IRI deterioration rate is computed as follows:

$$\textit{IRI Deterioration Rate} = \textit{Rate of Change in IRI} / \textit{Traffic} \times 10^{6} \qquad (5\text{-}1)$$

Where:

Rate of Change in IRI = Δ IRI (m/km) / Age (years)

Traffic = Average annual ESALs

Due to the variation of the IRI data, it was difficult to determine the change in IRI consistently. To resolve this problem, a linear regression analysis was performed on the plot of IRI versus time data, and the slope of the regression line was used to determine the change in IRI over the analysis period.

5.2.2 The Effect of Slab Design and Environment Factors on the Pavement Performance

In order to determine the correlation between each parameter and the IRI deterioration rate, descriptive statistical analysis for factors related with pavement design and environment was conducted.

5.2.2.1 The Effect of Concrete Slab Thickness

Figure 5-5 shows a plot of IRI deterioration rate versus concrete slab thickness for pavements with and without drainage systems. It can be seen that the deterioration rate generally decreases with increasing concrete slab thickness. The pavements with drainage systems generally perform better than those without.

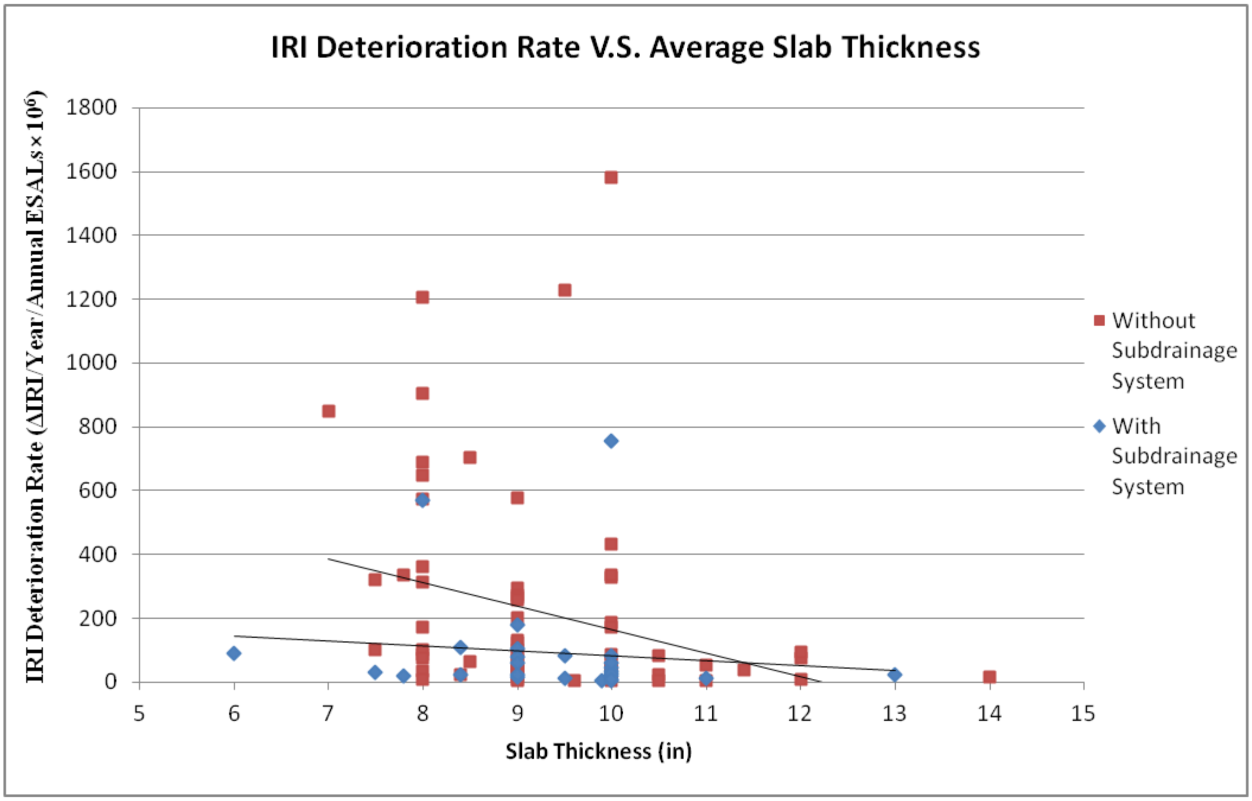


Figure 5-5. IRI deterioration rate versus concrete slab thickness.

5.2.2.2 The Effect of Concrete Slab Length

Figure 5-6 shows a plot of IRI deterioration rate versus concrete slab length for pavements with and without drainage systems. It can be seen that the deterioration rate generally increases with increasing concrete slab length for the test sections with drainage system. On the other hand, in case of the test sections without drainage systems, the deterioration rate generally decreases with increasing concrete slab length. With the inconsistent results, it can be concluded that the slab length does not have a strong correlation with the pavement performance expressed by a change in IRI.

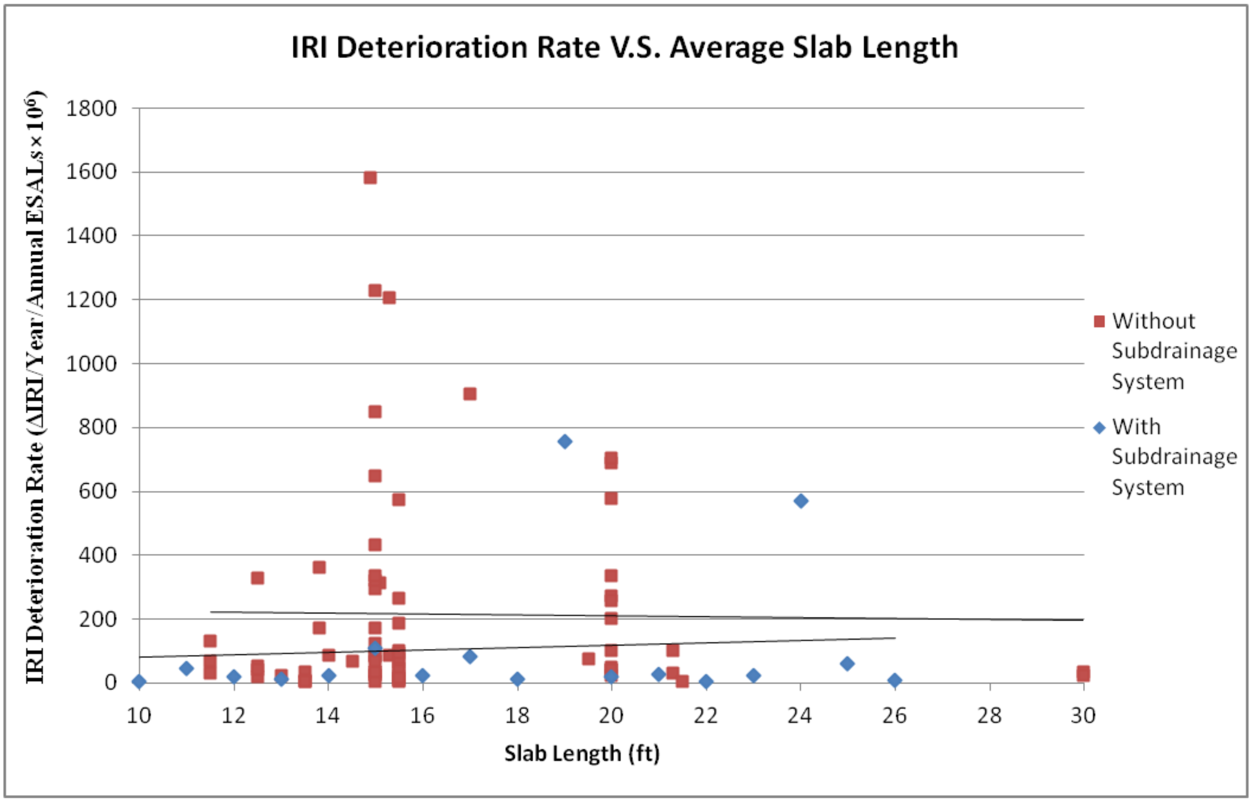


Figure 5-6. IRI deterioration rate versus concrete slab length.

5.2.2.3 The Effect of Average Temperature

Figure 5-7 shows a plot of IRI deterioration rate versus annual average temperature for pavement with and without drainage systems. It can be seen that the deterioration rate generally decreases with increasing temperature. Similarly, the pavements with drainage systems generally perform better than those without.

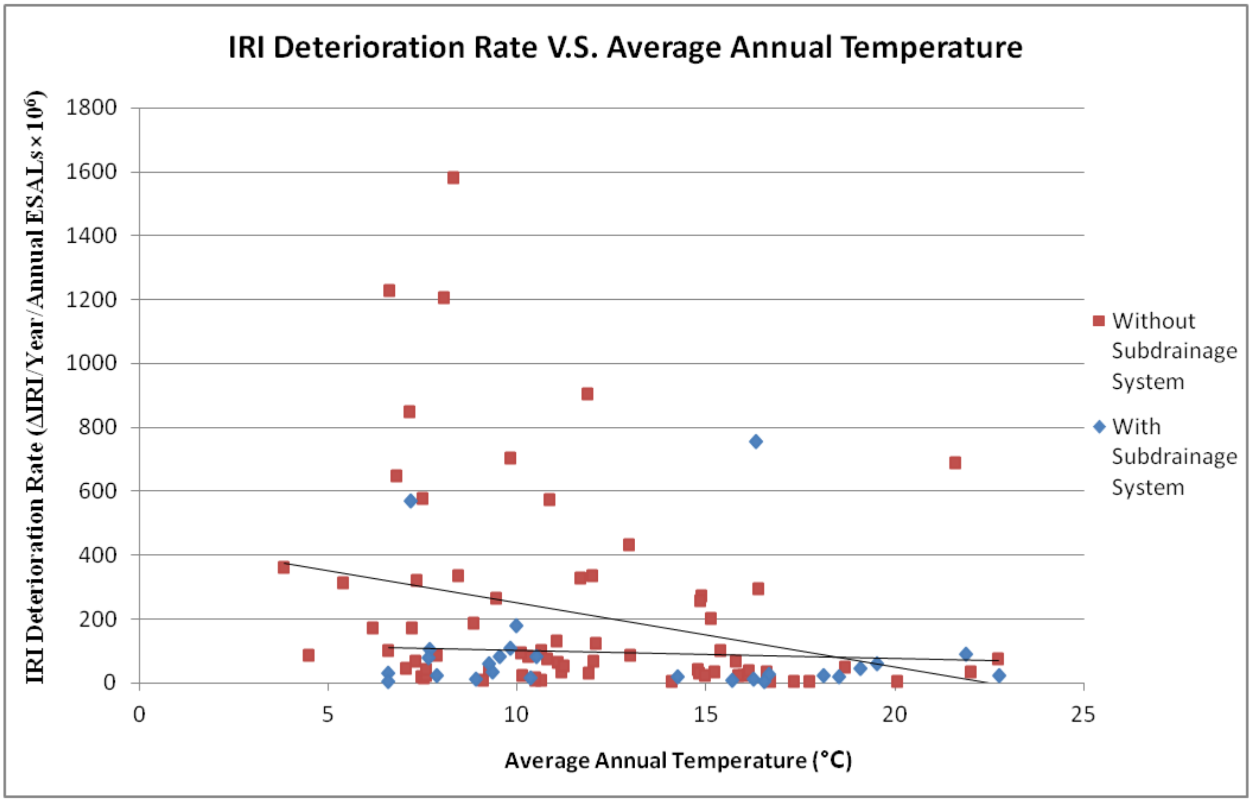


Figure 5-7. IRI deterioration rate versus average temperature.

5.2.2.4 The Effect of Number of Cold Days

Figure 5-8 shows a plot of IRI deterioration rate versus number of cold days per year under 0°C for pavement with and without drainage systems. It can be seen that the deterioration rate generally increases with increasing number of cold days. Similarly, the pavements with drainage systems generally perform better than those without.

5.2.2.5 The Effect of Precipitation

Figure 5-9 shows a plot of IRI deterioration rate versus annual precipitation for pavement with and without drainage systems. It can be seen that the deterioration rate generally increases with increasing precipitation. Similarly, the pavements with drainage systems generally perform better than those without.

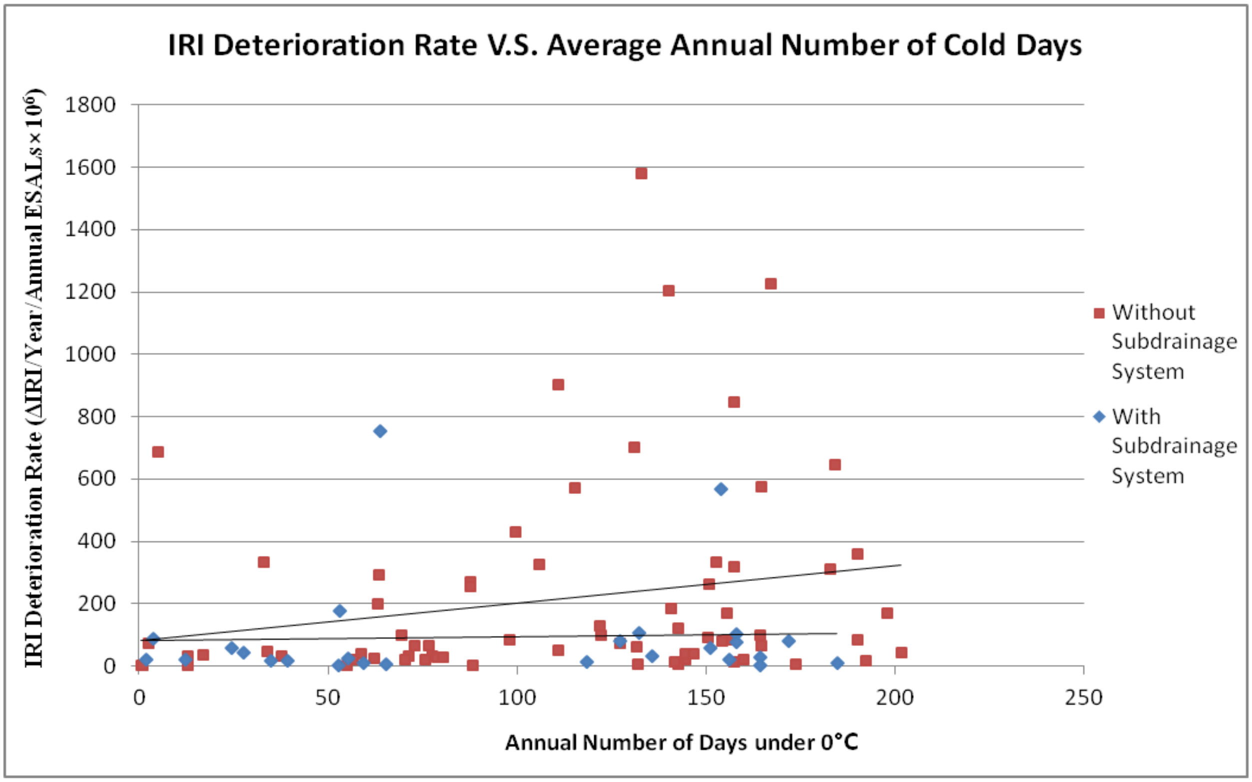


Figure 5-8. IRI deterioration rate versus number of cold days.

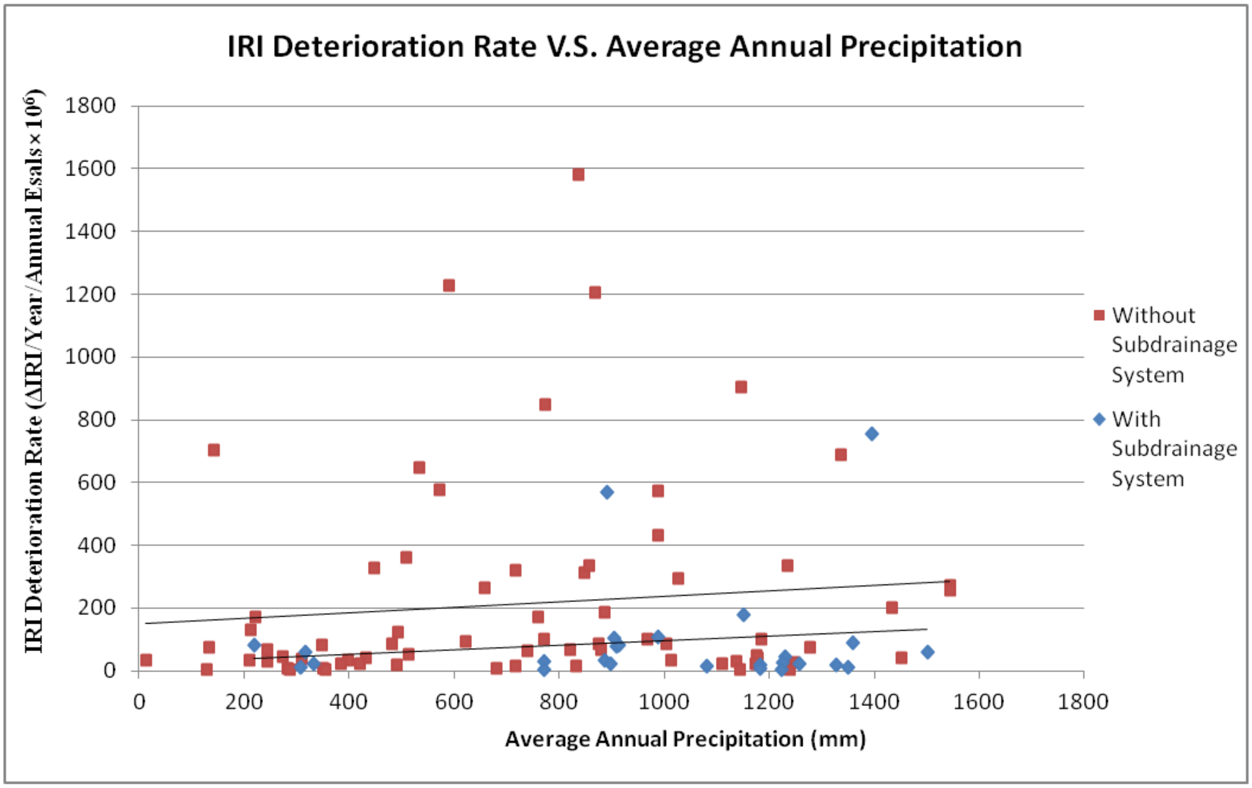


Figure 5-9. IRI deterioration rate versus precipitation.

5.2.2.6 The Effect of Number of Wet Days

Figure 5-10 shows a plot of IRI deterioration rate versus number of wet days (days with 5 mm or more of precipitation) per year for pavement with and without drainage systems. It can be seen that the deterioration rate generally increases with increasing number of wet days. Similarly, the pavements with drainage systems generally perform better than those without.

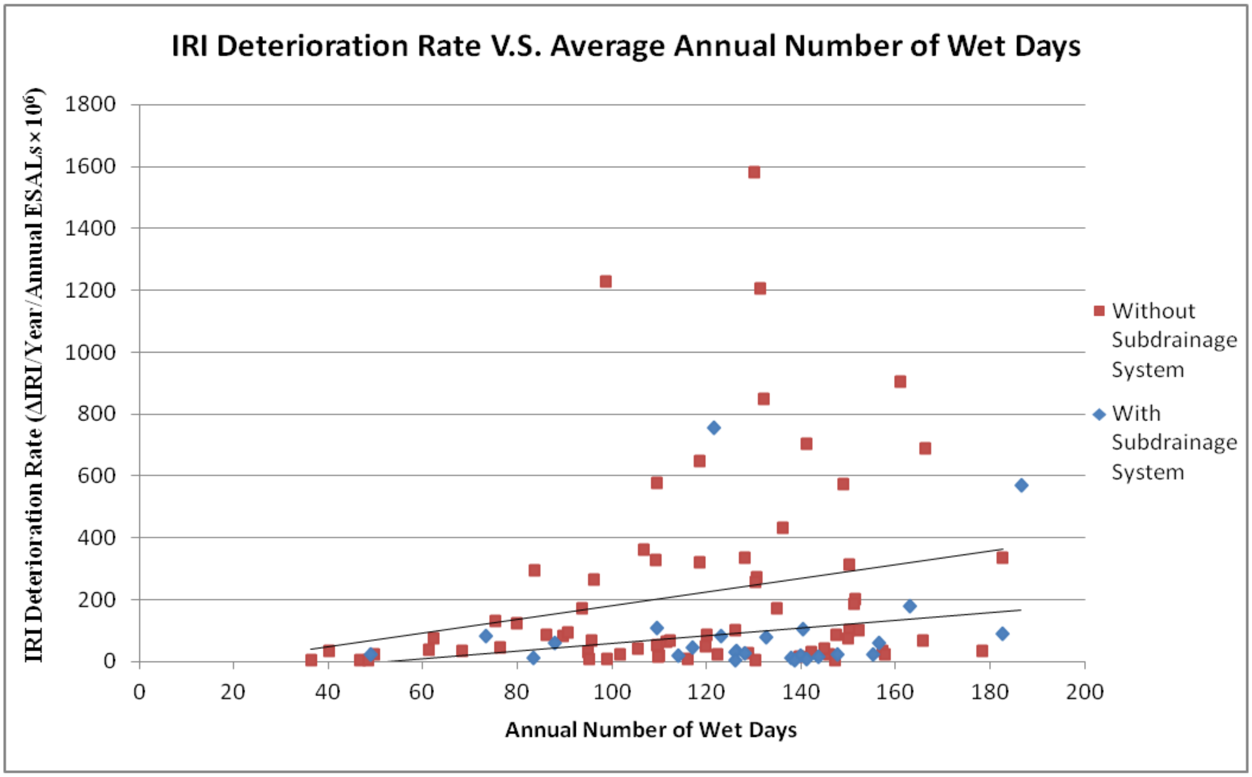


Figure 5-10. IRI deterioration rate versus number of wet days.

5.2.2.7 The Effect of Different Drainage Systems

Figure 5-11 shows the average IRI deterioration rate of pavements with different drainage systems. It can be seen that the deterioration rate of pavements with no drainage system is substantially higher than those with drainage systems. Drainage blankets appear to give slightly better performance than longitudinal drains and all other drainage systems. Other drainage types include granular borrow, 4 inches of sand with longitudinal drain and longitudinal ditch.

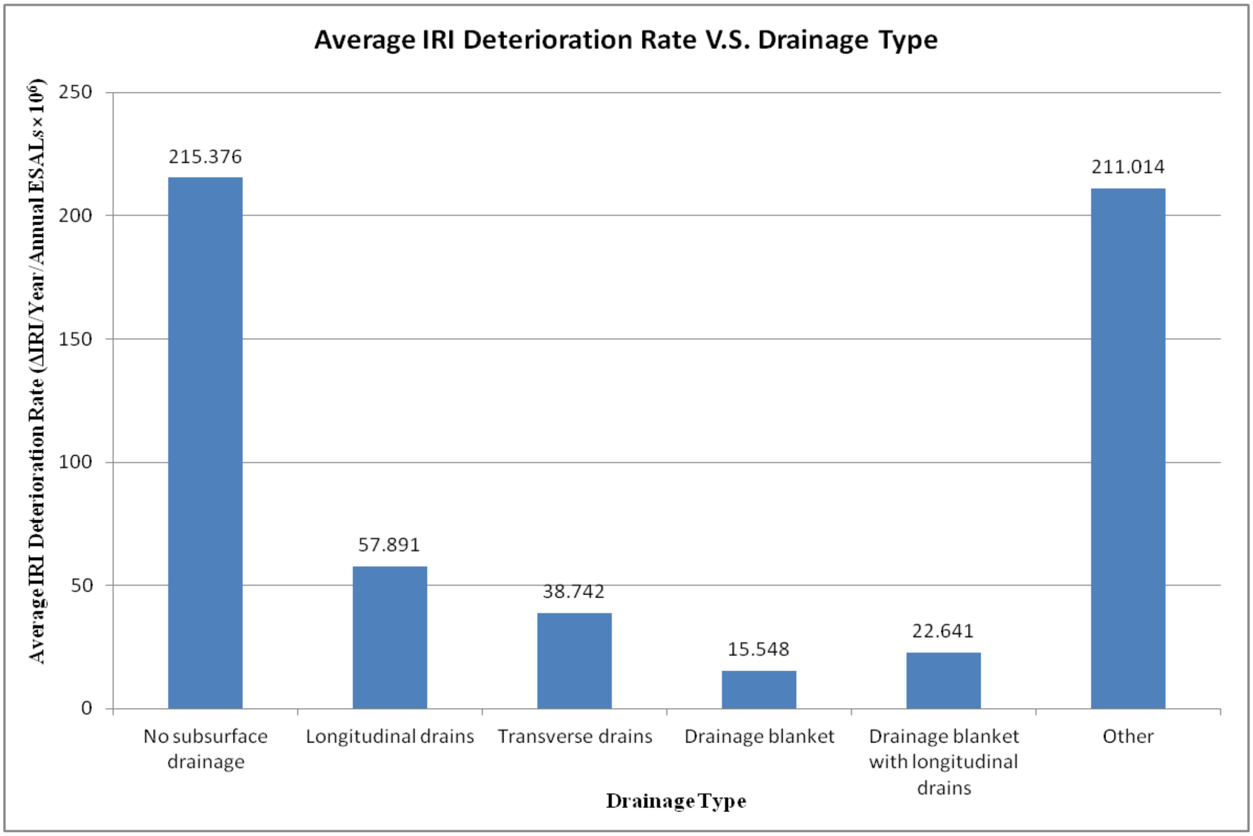


Figure 5-11. Average IRI deterioration rate of pavements with different drainage types.

5.2.2.8 The Effect of Base Type

Table 5-3 shows the average IRI deterioration rates of concrete pavements with different base types. As shown in Table 5-3, pavements with dense graded, hot laid, central plant mix base appear to have the best performance while pavements with gravel base appear to have the worst performance.

5.2.2.9 The Effect of Subgrade Type

Table 5-4 shows the average IRI deterioration rates of concrete pavements with different subgrade types. The pavements with gravelly lean clay subgrade appear to have the best performance while the pavements with gravelly Silt with Sand subgrade appear to have the worst performance.

Table 5-3. Average IRI Deterioration Rate with Different Base Types

Base Type	Average IRI Deterioration Rate¹
Gravel (Uncrushed)	439.314
Crushed Stone	346.5347
Crushed Gravel	294.2849
Sand	167.6409
Soil-Aggregate Mixture	84.0522
Fine-Grained Soils	177.0287
Sand Asphalt	179.3026
Asphalt Treated Mixture	36.78105
Dense Graded, Hot Laid, Central Plant Mix	3.98945
Cement Aggregate Mixture	72.08385
Econocrete	92.76475
Lean Concrete	111.8433
Soil Cement	230.5121

Note 1: IRI Deterioration Rate = ΔIRI/year/annual ESALs × 10^6

Table 5-4. Average IRI Deterioration Rate with Different Subgrade Types

Subgrade Type	Average IRI deterioration Rate¹
Fine-Grained Soils: Lean Inorganic Clay	130.7613
Fine-Grained Soils: Lean Clay with Sand	800.7224
Fine-Grained Soils: Fat Clay with Sand	576.8935
Fine-Grained Soils: Gravelly Lean Clay	4.40555
Fine-Grained Soils: Sandy Clay	51.0051
Fine-Grained Soils: Sandy Lean Clay	333.2012
Fine-Grained Soils: Silty Clay with Sand	13.20205
Fine-Grained Soils: Silt	50.2167
Fine-Grained Soils: Silt with Sand	41.02235
Fine-Grained Soils: Sandy Silt	193.8844
Fine-Grained Soils: Gravelly Silt with Sand	1578.756
Coarse-Grained Soils: Poorly Graded Sand	55.9034
Coarse-Grained Soils: Poorly Graded Sand with Gravel	65.5394
Coarse-Grained Soils: Poorly Graded Sand with Silt	62.8676
Coarse-Grained Soils: Poorly Graded Sand with Silt and Gravel	312.6444
Coarse-Grained Soil: Silty Sand	152.7233
Coarse-Grained Soil: Silty Sand with Gravel	140.1199
Coarse-Grained Soil: Clayey Sand	335.9278
Coarse-Grained Soil: Clayey Sand with Gravel	277.7431
Coarse-Grained Soil: Poorly Graded Gravel with Silt and Sand	198.2133
Coarse-Grained Soil: Silty Gravel	303.107
Coarse-Grained Soil: Silty Gravel with Sand	238.2063
Coarse-Grained Soil: Clayey Gravel with Sand	284.5577
Rock and Stone	490.7425
Rock	111.0513

Note 1: IRI Deterioration Rate = ΔIRI/year/annual ESALs × 10^6

5.3 Evaluation of Performance of JPCP Test Sections in Wet and No-Freeze Climate Zone

5.3.1 Overview

In the previous section, the Long-Term Pavement Performance (LTPP) database was studied and used to evaluate the effects of environmental conditions and pavement designs on performance of Jointed Plain Concrete Pavements (JPCP) in the U.S. In this section, additional analysis was performed on LTPP JPCP sections in the southeastern region of the U.S., which is classified as Wet and No-Freeze Climate Zone and which is more applicable to Florida climate condition. Critical stress analysis was performed on the selected LTPP JPCP sections to determine the maximum stress in the concrete slab under a specific load and temperature condition. The results of critical stress analysis were compared to the observed performance of the selected pavements to determine if they can be used as predictors for the performance of these pavements.

5.3.2 Valid Test Sections in the Wet and No-Freeze Climate Zone

There are four LTPP climate zones in U.S. as shown in Figure 5-12, (1) Dry and No-Freeze, (2) Wet and Freeze, (3) Wet and No-Freeze, and (4) Dry and Freeze. Florida lies in the Wet and No-Freeze Zone. Thus, for the critical stress analysis, only test sections in the Wet and No-Freeze Zone were selected to reduce the effect caused by different environment conditions.

Among the 97 valid GPS-3 sections in the U.S., there are 26 test sections located in this climate zone. Figure 5-13 shows all sections in the Wet and No-Freeze Climate Zone and excluded sections. Out of all 26 valid GPS-3 test sections, 2 test sections (SHRP ID: 3019-28 and 4059-12) were excluded due to their missing material property data in LTPP data set. Table 5-5 shows the 24 valid test sections used in the analysis, along with their material properties

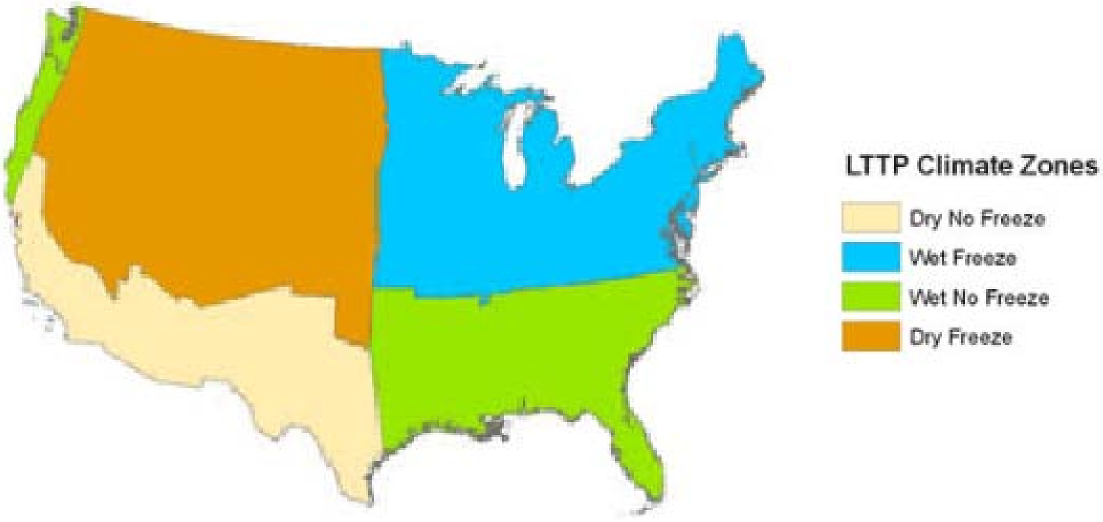


Figure 5-12. LTPP climate zones.

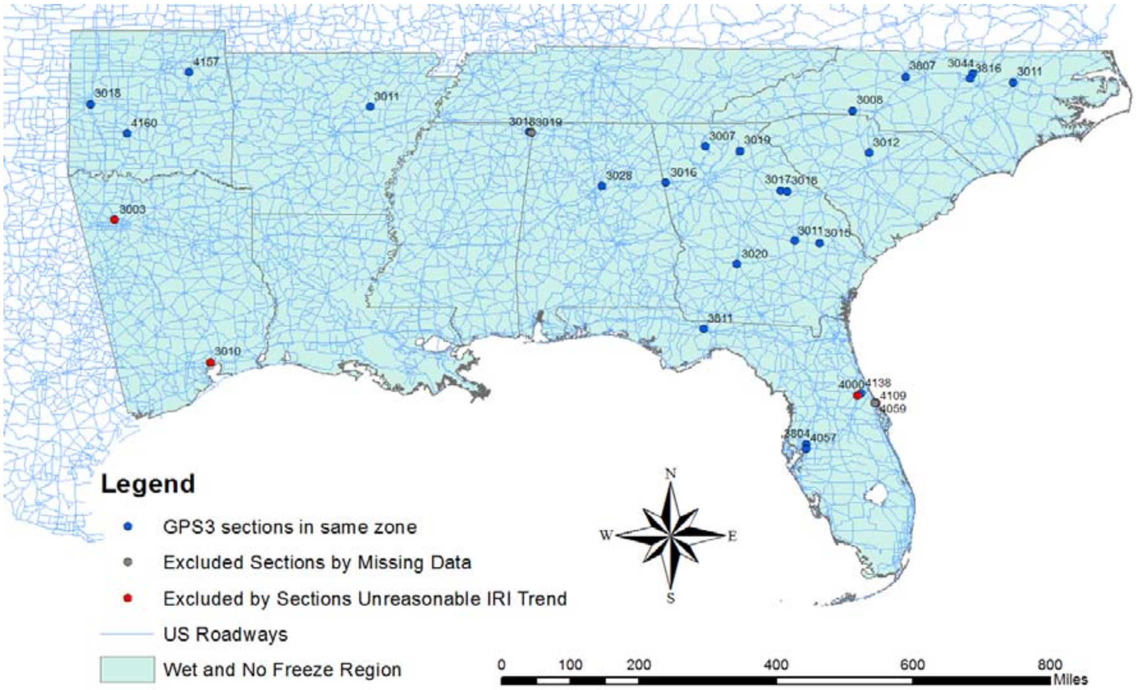


Figure 5-13. Locations of LTPP JPCP test sections in Wet and No-Freeze Climate Zone.

Table 5-5. Data on the LTPP JPCP Test Sections in Wet and No-Freeze Climate Zone

SHRP ID	States Code	Comment	Calculated Maximum Stress (psi)	Stress-to-Strength Ratio	Coefficient of Thermal Expansion (1/°F)	Average Poisson's Ratio	Average Elastic Modulus (psi)	Slab Length (in)	Slab Thickness (ft)	Average Annual Traffic[1] (× 10^3 ESAL)	Compressive Strength (psi)	Tensile Strength (psi)	Calculated MR (psi)	IRI Deterioration Rate[2]
3017	GA13	Valid	377.66	0.68	4.495E-06	0.145	3.45E+06	19.5	10	1183.00	5695	494.5	553.5448	3.67
3012	SC45	Valid	488.06	0.75	4.989E-06	0.160	4.73E+06	21.5	10	1195.57	8645	584.5	654.291	3.97
3011	NC37	Valid	399.27	0.69	4.667E-06	0.140	3.78E+06	30.0	10	1040.50	6725	515.5	577.0522	6.67
3016	GA13	Valid	422.82	0.57	5.127E-06	0.165	4.05E+06	20.0	11	1590.93	8150	664.5	743.8433	9.41
3011	GA13	Valid	379.76	0.81	4.642E-06	0.110	3.45E+06	20.0	10	701.21	5230	421.0	471.2687	19.26
4157	OK40	Valid	478.67	0.58	4.048E-06	0.225	4.73E+06	15.0	9	573.82	8125	732.5	819.9627	20.35
4057	FL12	Valid	318.29	0.59	4.382E-06	0.170	3.78E+06	15.5	13	483.63	7040	485.5	543.4701	22.64
3044	NC37	Valid	450.67	0.65	5.278E-06	0.135	3.73E+06	30.0	9	1103.33	5790	620.0	694.0299	23.16
3018	GA13	Valid	275.62	0.53	4.359E-06	0.130	2.00E+06	19.5	10	1154.40	5735	467.0	522.7612	23.71
3011	AR5	Valid	478.20	0.72	5.993E-06	0.150	4.03E+06	15.0	10	434.00	7985	591.5	662.1269	24.39
3807	NC37	Valid	440.67	0.66	4.376E-06	0.190	3.95E+06	21.3	9	380.00	7345	594.0	664.9254	30.74
3816	NC37	Valid	455.40	0.78	4.889E-06	0.135	4.03E+06	30.0	9	150.25	6795	519.0	580.9701	33.77
3007	GA13	Valid	516.65	0.76	3.952E-06	0.250	5.43E+06	20.0	9	160.05	7590	609.5	682.2761	41.05
3015	GA13	Valid	444.71	0.73	5.037E-06	0.115	4.13E+06	20.0	10	760.05	6705	545.0	610.0746	44.18
3020	GA13	Valid	423.03	0.51	4.750E-06	0.125	3.58E+06	20.0	9	72.86	8965	738.0	826.1194	49.10
3811	FL12	Valid	377.27	0.73	4.357E-06	0.165	3.05E+06	20.0	9	582.33	5955	460.0	514.9254	60.55
3018	OK40	Valid	487.84	0.69	4.367E-06	0.235	4.58E+06	15.0	9	1243.92	6770	631.0	706.3433	65.73
3804	FL12	Valid	362.27	0.63	4.358E-06	0.185	3.95E+06	19.5	12	599.71	6095	514.0	575.3731	73.64
3008	NC37	Valid	518.00	0.72	5.144E-06	0.180	4.03E+06	21.3	8	201.83	8885	646.0	723.1343	99.46
3019	GA13	Valid	437.15	0.80	4.556E-06	0.210	3.75E+06	20.0	9	98.05	6740	487.5	545.709	201.01
3018	MS28	Valid	467.91	0.74	4.517E-06	0.195	4.25E+06	20.0	9	184.50	8450	563.0	630.2239	272.61
4160	OK40	Valid	424.90	0.64	3.925E-06	0.205	3.95E+06	15.0	9	124.63	6105	591.5	662.1269	292.88
4138	FL12	Valid	441.45	0.91	4.246E-06	0.260	3.33E+06	20.0	8	71.00	5770	432.0	483.5821	687.85
3028	AL1	Valid	487.79	0.85	4.083E-06	0.190	5.53E+06	20.0	10	91.20	7220	514.5	575.9328	754.81
3019	MS28	Missing data	N/A											
4059	FL12	Missing data	N/A											

Note 1: Annual ESALs in thousands in the LTPP lane

Note 2: IRI Deterioration Rate = ΔIRI/year/annual ESALs × 10^6

5.3.3 The Effect of Concrete Properties on the Pavement Performance

To figure out the relationship between each material property and IRI deterioration rate, descriptive statistical analysis was conducted.

5.3.3.1 The Effect of Compressive Strength

Figure 5-14 shows a plot of IRI deterioration rate versus compressive strength of the concrete slab. Though the trend of fitted line seems to be reasonable, the IRI deterioration rate did not correlate well with the compressive strength of concrete slab, with an R^2 value of 0.0108.

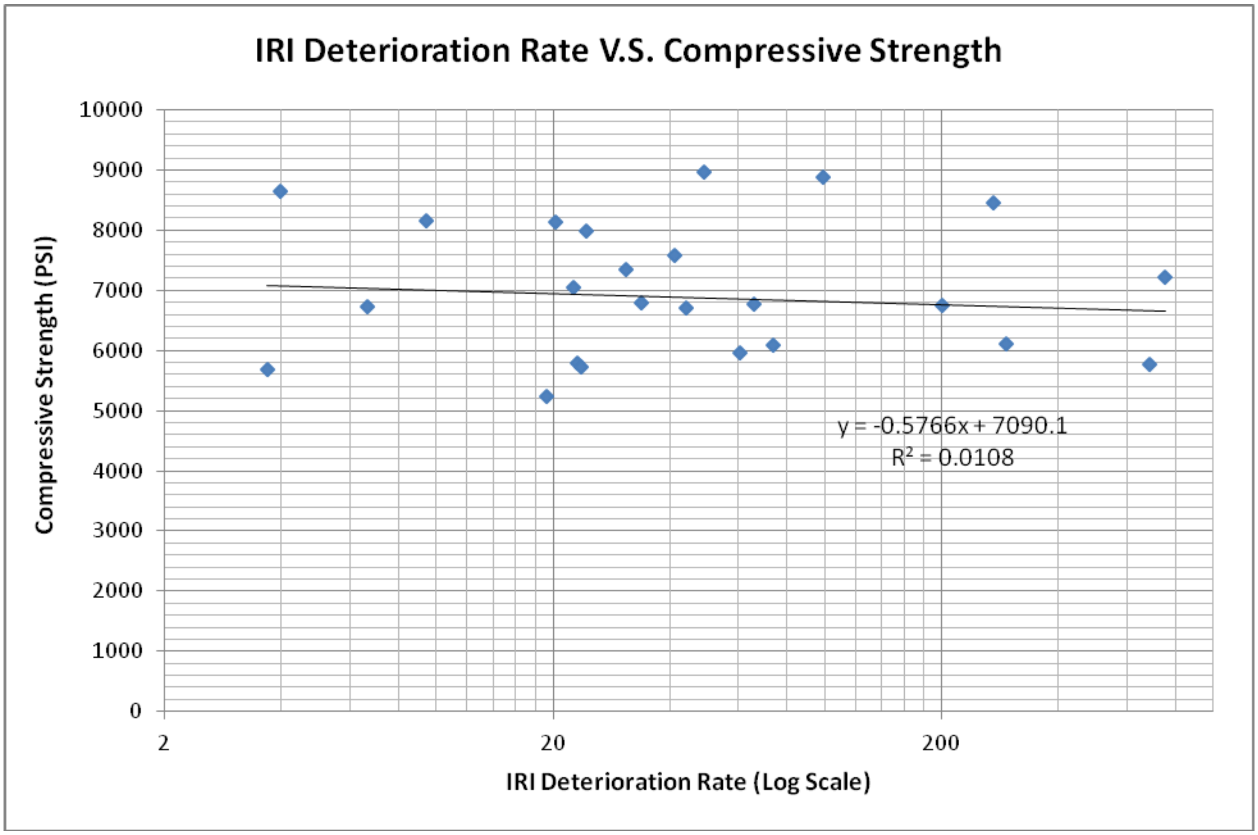


Figure 5-14. IRI deterioration rate versus compressive strength of concrete.

5.3.3.2 The Effect of Tensile Strength

Figure 5-15 shows a plot of IRI deterioration rate versus tensile strength of the concrete slab. The trend of fitted line seems to be reasonable but the IRI deterioration rate does not correlate well with the compressive strength of concrete slab, with an R^2 value of 0.0772

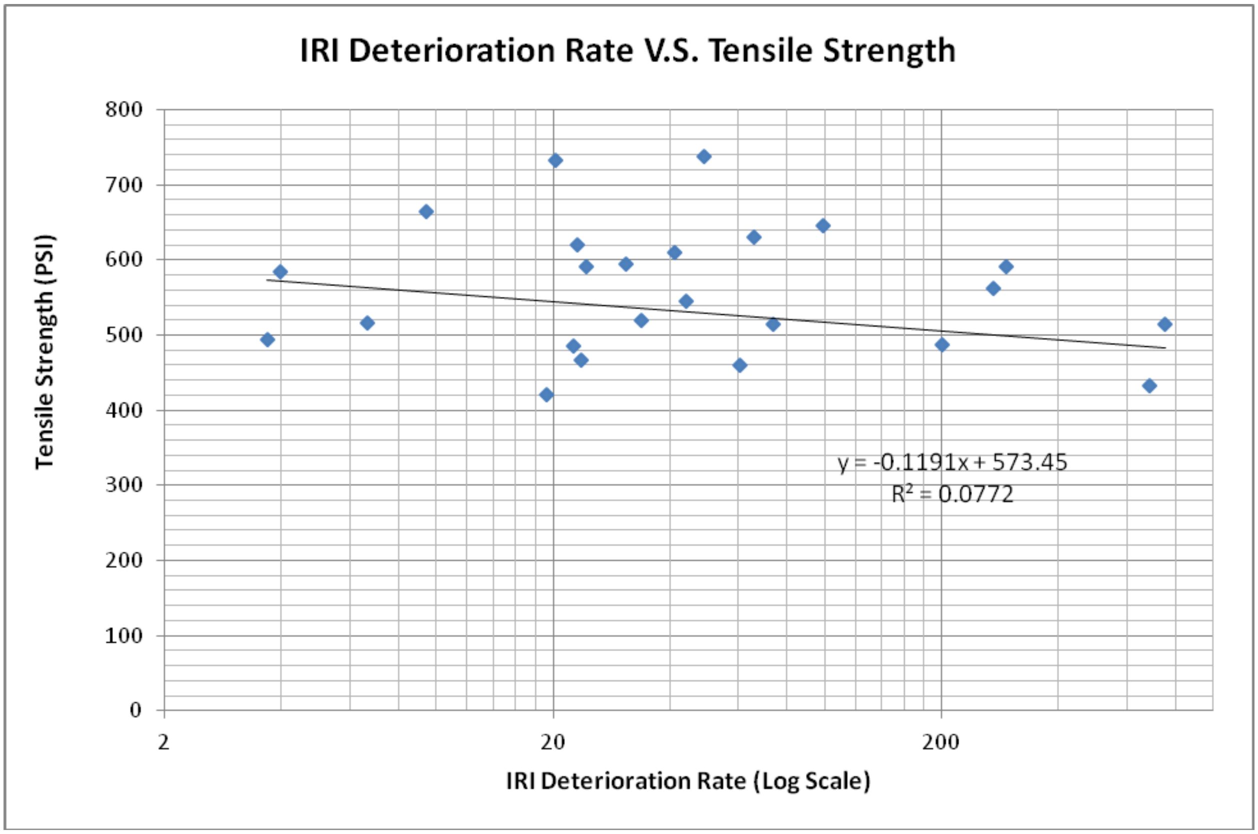


Figure 5-15. IRI deterioration rate versus tensile strength of concrete.

5.3.3.3 The Effect of Coefficient of Thermal Expansion of Concrete

Figure 5-16 shows a plot of IRI deterioration rate versus coefficient of thermal expansion of the concrete slab. Similarly, the coefficient of thermal expansion shows a reasonable trend along with the IRI deterioration rate but it did not have a strong correlation with a R^2 value of 0.0139.

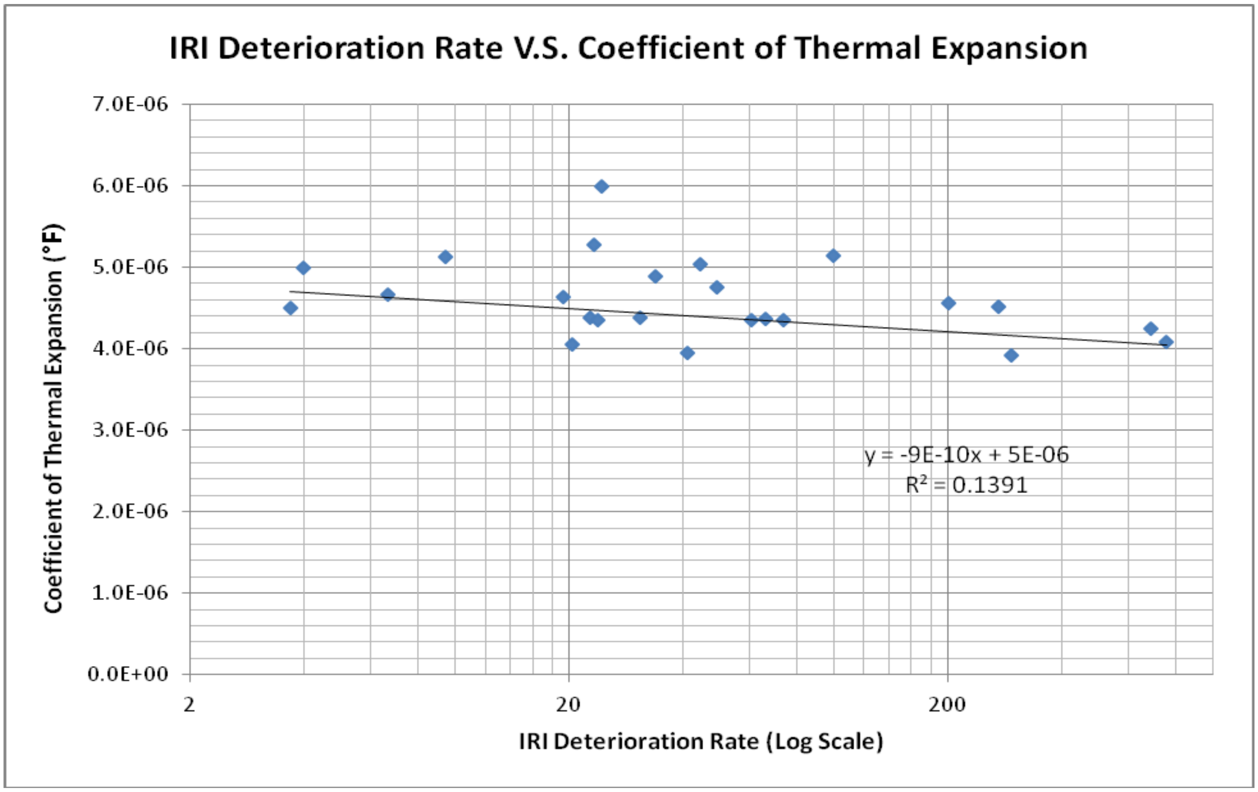


Figure 5-16. IRI deterioration rate versus coefficient of thermal expansion.

5.3.3.4 The Effect of Elastic Modulus

Figure 5-17 shows a plot of IRI deterioration rate versus elastic modulus of concrete. The IRI deterioration rate did not correlate well with the elastic modulus of concrete, with an R^2 value of 0.0497. Similarly, the trend of the fitted line seems to be reasonable.

5.3.3.5 The Effect of Slab Thickness

Figure 5-18 shows a plot of IRI deterioration rate versus slab thickness for the 24 test sections in the Wet and No-Freeze Climate Zone. Similar to the Figure 5-5 which shows 97 test sections in the U.S., slab thickness does not have a strong correlation with the IRI deterioration rate.

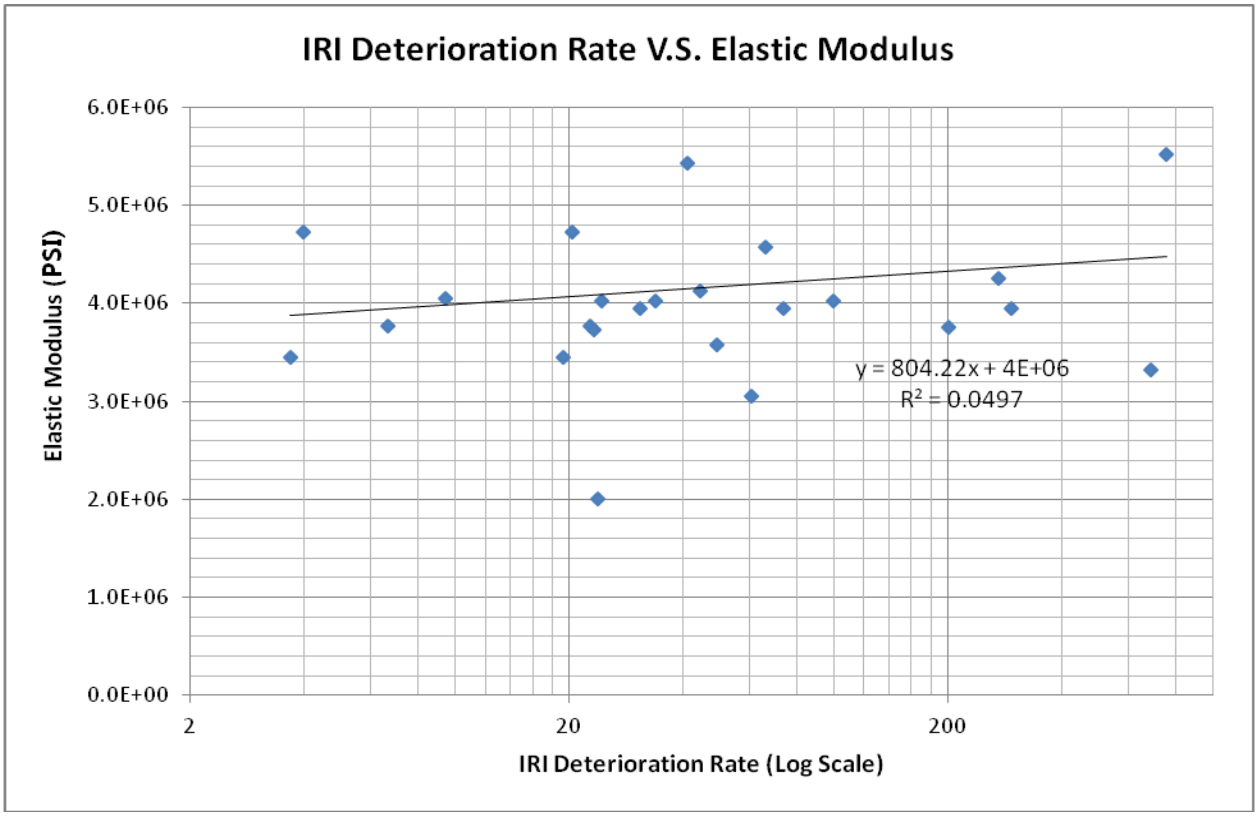


Figure 5-17. IRI deterioration rate versus elastic modulus.

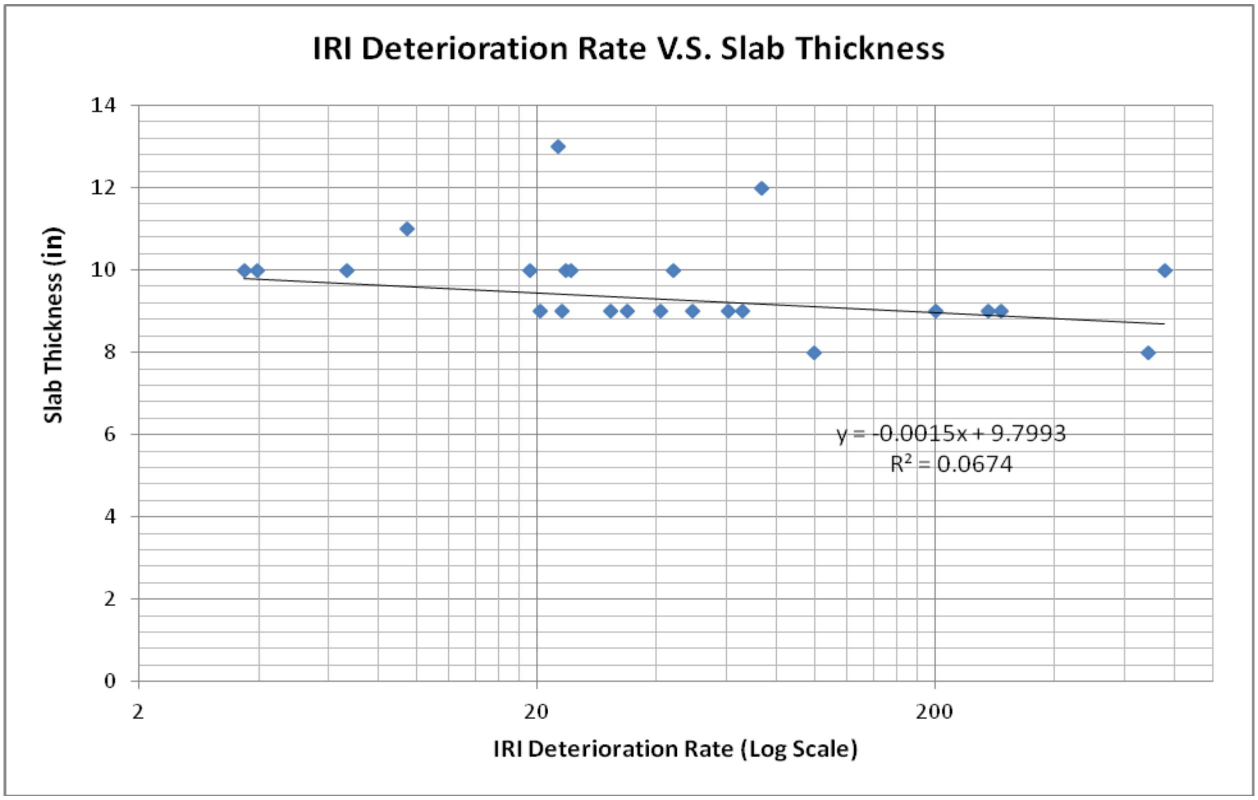


Figure 5-18. IRI deterioration rate versus slab thickness.

5.3.3.6 The Effect of Slab Length

Figure 5-19 shows a plot of IRI deterioration rate versus slab length for the 24 test sections in the Wet and No-Freeze Climate Zone. Similar to the Figure 5-6 which shows 97 test sections in the U.S., slab length does not correlate well with the IRI deterioration rate with an R^2 value of 0.0136.

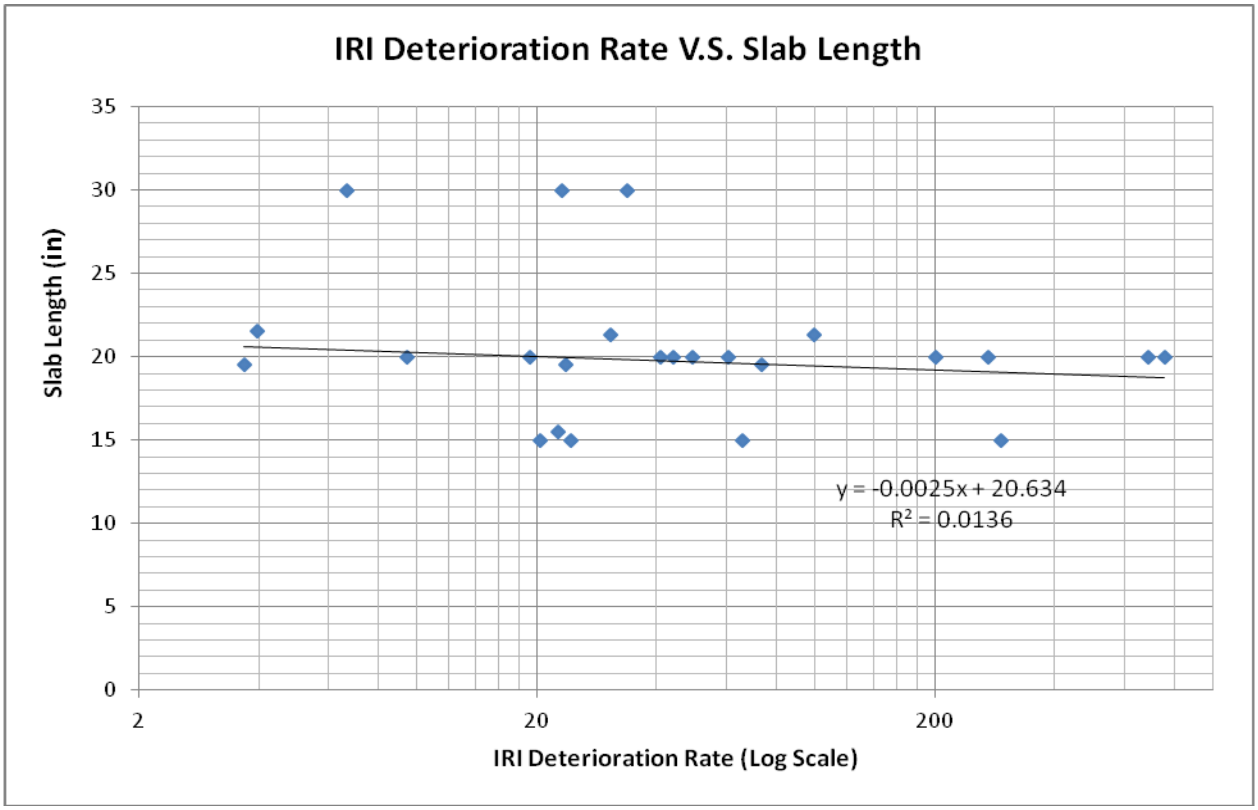


Figure 5-19. IRI deterioration rate versus slab length.

5.3.4. Critical Stress Analysis

Each single concrete property showed reasonable trend but not strong correlation with IRI deterioration rate. This can be explained by the fact that the pavement deterioration process is very complex, and all material factors interact with one another in affecting the performance of the pavement. This section represents the results of a critical stress analysis to determine the maximum stresses and the stress-to-strength ratios for these pavements.

5.3.4.1 Method of Analysis

FEACONS (Finite Element Analysis of Concrete Slab) program, which was developed at the University of Florida for stress analysis of concrete pavements, was used to perform the critical stress analysis. Previous studies (Wu and Larsen, 1993) have shown that the most critical loading condition on a concrete pavement occurs when there is a positive temperature differential in the concrete slab (i.e., when the temperature at the top of the slab is higher than that at the bottom) and a heavy load is applied to the middle edge of the slab. The maximum stress caused by a 22-kip axial load which is applied at the middle of the edge of the concrete slab and 20 °F of the temperature differential between top and bottom of the concrete slab was computed for each of these 24 test sections using the FEACONS program. Figure 5-20 shows the finite element meshes for the different slab length and the locations of applied load.

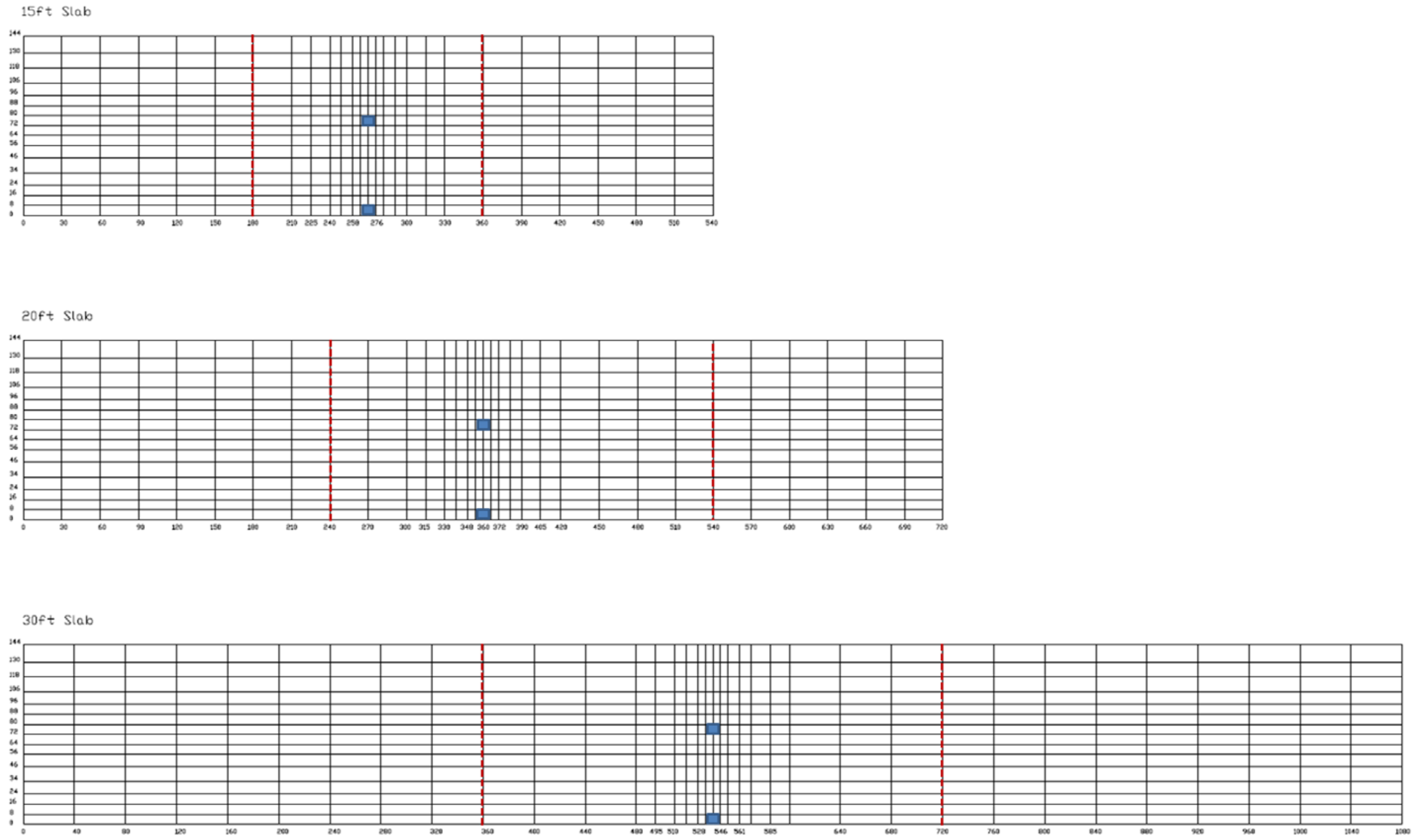


Figure 5-20. The finite element meshes and locations of applied loads for different slab length.

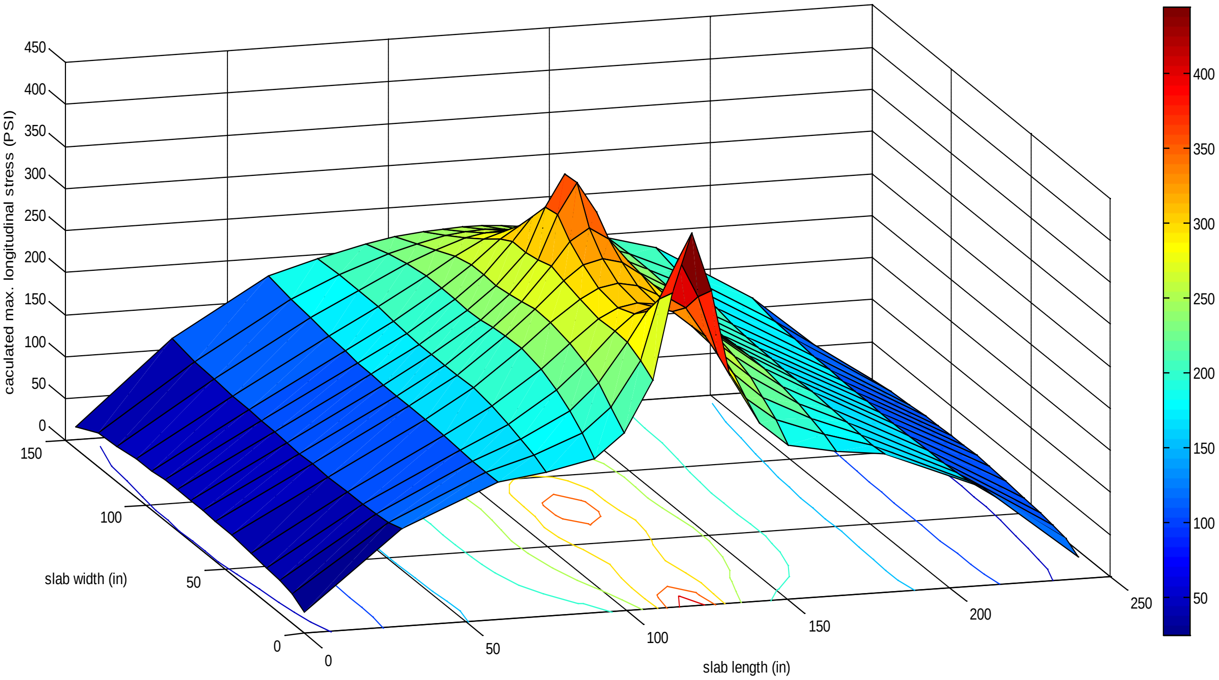


Figure 5-21. Calculated critical stress for a test section.

Figure 5-21 shows calculated longitudinal stresses for a typical test section (GA3015). The two points having the highest longitudinal stress are the locations of applied load.

5.3.4.2 The Relationship between Maximum Computed Stress and Pavement Performance

The maximum computed stresses from the critical stress analysis were compared to the IRI deterioration rate of the selected test sections. Figure 5-22 shows a plot of maximum computed stress versus the IRI deterioration rate of the selected test sections. It can be seen that the IRI deterioration rate increases as the computed maximum stress increases. However, the R^2 of the correlation is only 0.0501, which is fairly low.

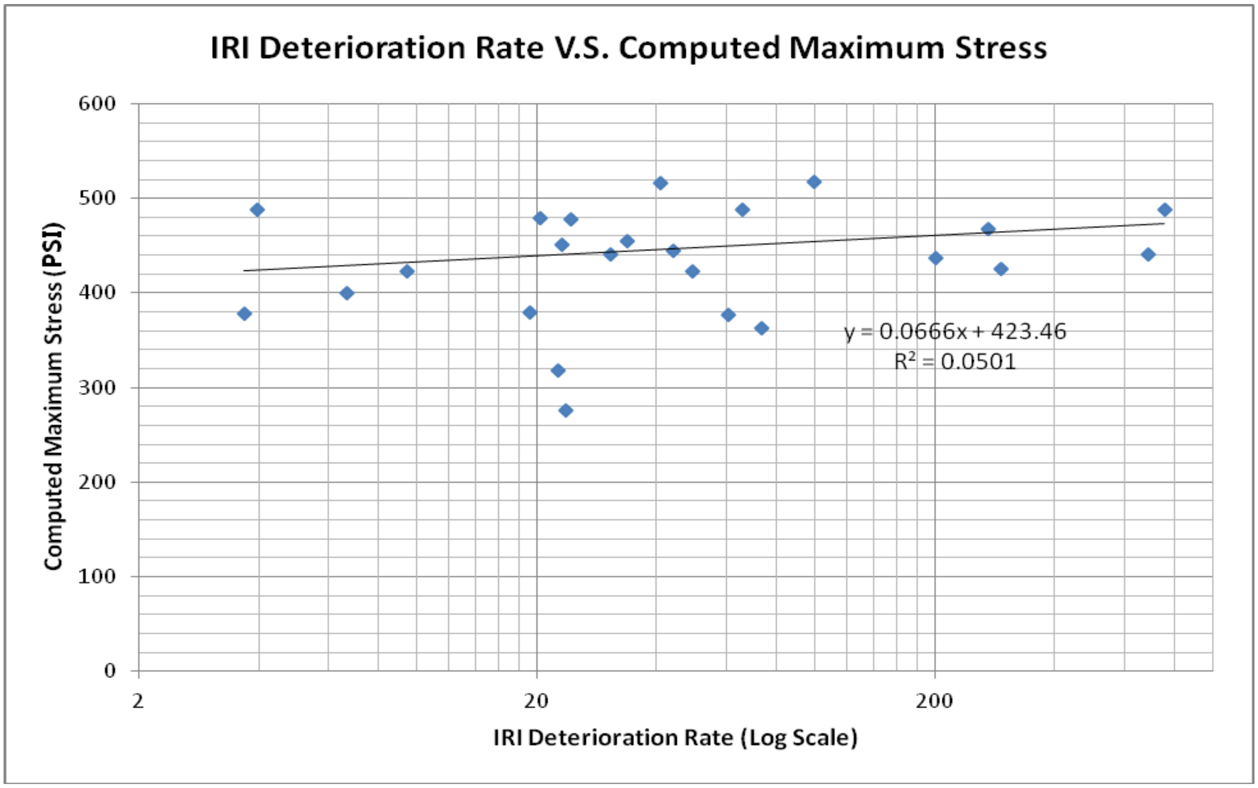


Figure 5-22. IRI deterioration rate versus maximum computed stress.

5.3.4.3 The Relationship between Stress-Strength Ratio and Pavement Performance

The flexural strength (or modulus of rupture) of the concrete for each test section was estimated from the tensile strength of the concrete, which was available in the LTPP database, by the following equation (Raphael, 1984):

$$MR = 7.5/6.7 \times Fst \tag{5-2}$$

Where:

MR = Flexural Strength (PSI)

Fst = Split Tensile Strength (PSI)

The maximum computed stress from the critical stress analysis was divided by the flexural strength to obtain the stress-to-strength ratio for each test section. The computed stress-to-strength ratios are presented in Table 5-5.

According to fatigue failure theory, the stress-to-strength ratio is related to the number of cycles to failure for the concrete. A lower stress-to-strength ratio would predict a higher number of cycles to failure for the concrete slab, and thus, it would be a better predicted performance for the concrete pavement. The stress-to-strength ratios for these selected JPCP test sections were compared to the observed performance of these pavements to determine if they could be used as predictors for the performance of these pavements.

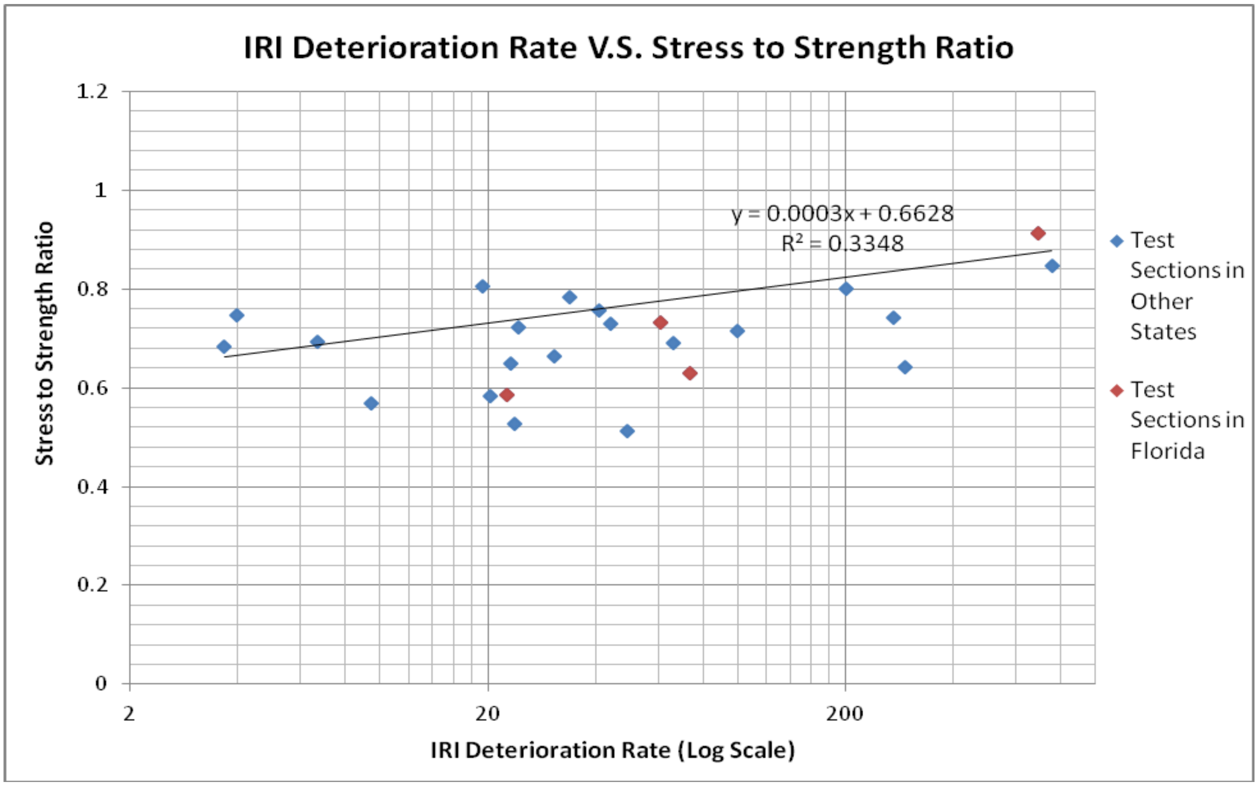


Figure 5-23. IRI deterioration rate versus stress-to-strength ratio.

Figure 5-23 shows a plot of IRI deterioration rate versus stress-to-strength ratio. It can be seen that the stress-to-strength ratio correlates well with the IRI deterioration rate, with a R^2 value of 0.3348. A lower stress-to-strength ratio is related to better pavement performance with a lower IRI rate of deterioration.

5.4. Evaluation of Florida Concrete Pavement Designs by Critical Stress Analysis

5.4.1 Overview

In the last section, critical stress analysis was performed on the LTPP JPCP test sections in the Wet and No-Freeze Zone in the U.S. and it was found that the computed maximum stress to flexural strength ratios relate well to the deterioration rate of the pavement. The IRI deterioration rate increases as the stress-to-strength ratio increases, as fatigue theory would predict. Thus, the computed stress-to-strength ratio can be used as a performance indicator of concrete pavements. In this section, critical stress analysis is applied to the Florida concrete pavement designs, Types I-A, I-B and II.

5.4.2 Evaluation of Effect of Various Factors

The FEACONS program was used to perform critical stress analysis on the Florida concrete pavement designs subject to the variation of various factors. The three designs considered in this study, namely Types I-A, I-B and II differ mainly in the base and subbase structure. When they are modeled by the FEACONS program, the differences among these three designs are mainly in the effective modulus of subgrade reaction used.

5.4.2.1 The Effect of Subgrade Modulus

Figure 5-24 shows the calculated maximum stress in the concrete slab with thickness of 11, 12 and 13 inches versus subgrade modulus in the range of LTPP database. A slab length of 15 ft and slab width of 12 ft, and typical properties of concrete made with Brooksville coarse aggregate were used in this analysis. It can be seen that for these three slab thicknesses, the difference in subgrade modulus does not have any significant effect on the maximum stresses.

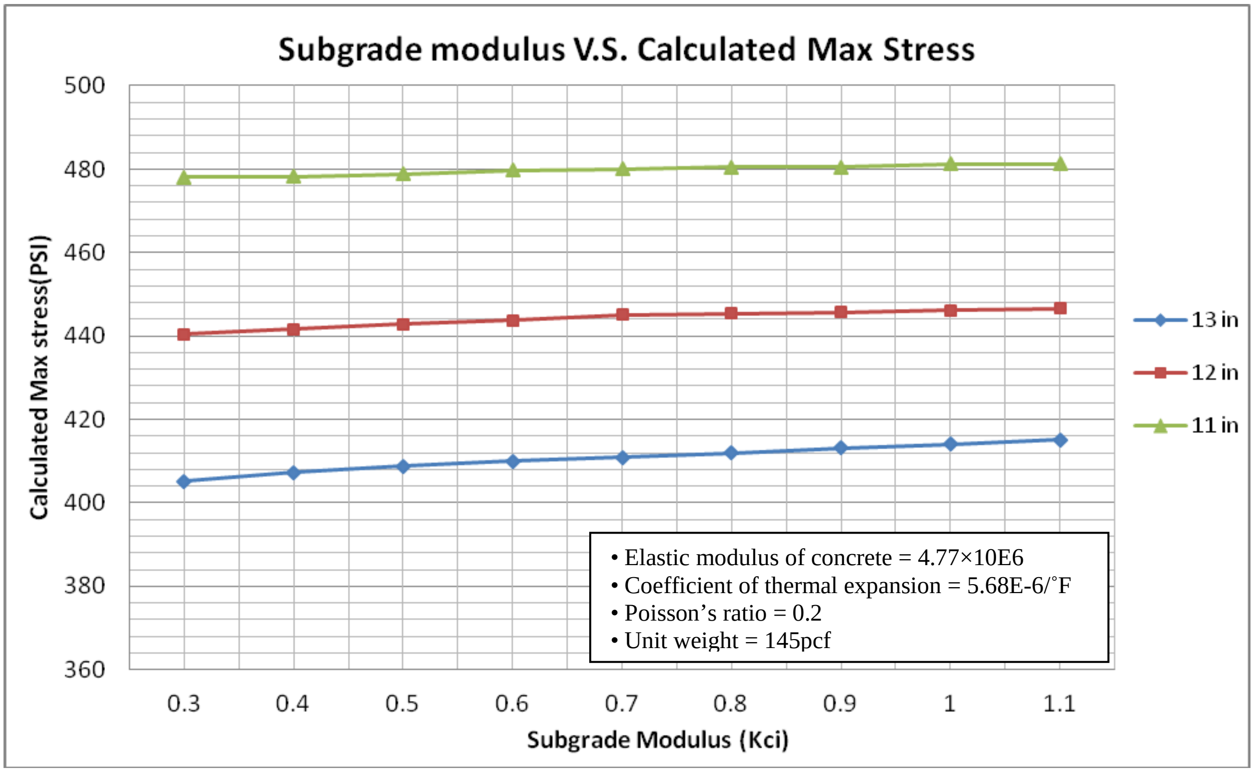


Figure 5-24. Calculated maximum stress for different subgrade modulus and slab thickness.

5.4.2.2 The Effect of Elastic Modulus and Coefficient of Thermal Expansion

The elastic modulus and the coefficient of thermal expansion of the concrete pavement in the LTPP database ranged approximately from 3000 to 6000 ksi and 4.0E-6 to 6.0E-6/˚F, respectively. Using the different values of elastic modulus and coefficient of thermal expansion in the range of LTPP database, the maximum stresses in 13-inch slabs under the critical loading conditions were computed by FEACONS program and presented in Figure 5-25. The figure shows that the calculated maximum stress increases by around 30% when the elastic modulus changes from 3000 to 6000 ksi. Also, the calculated maximum stress increases by around 25% when the coefficient of thermal expansion changes from 4.0E-6 to 6.0E-6/˚F.

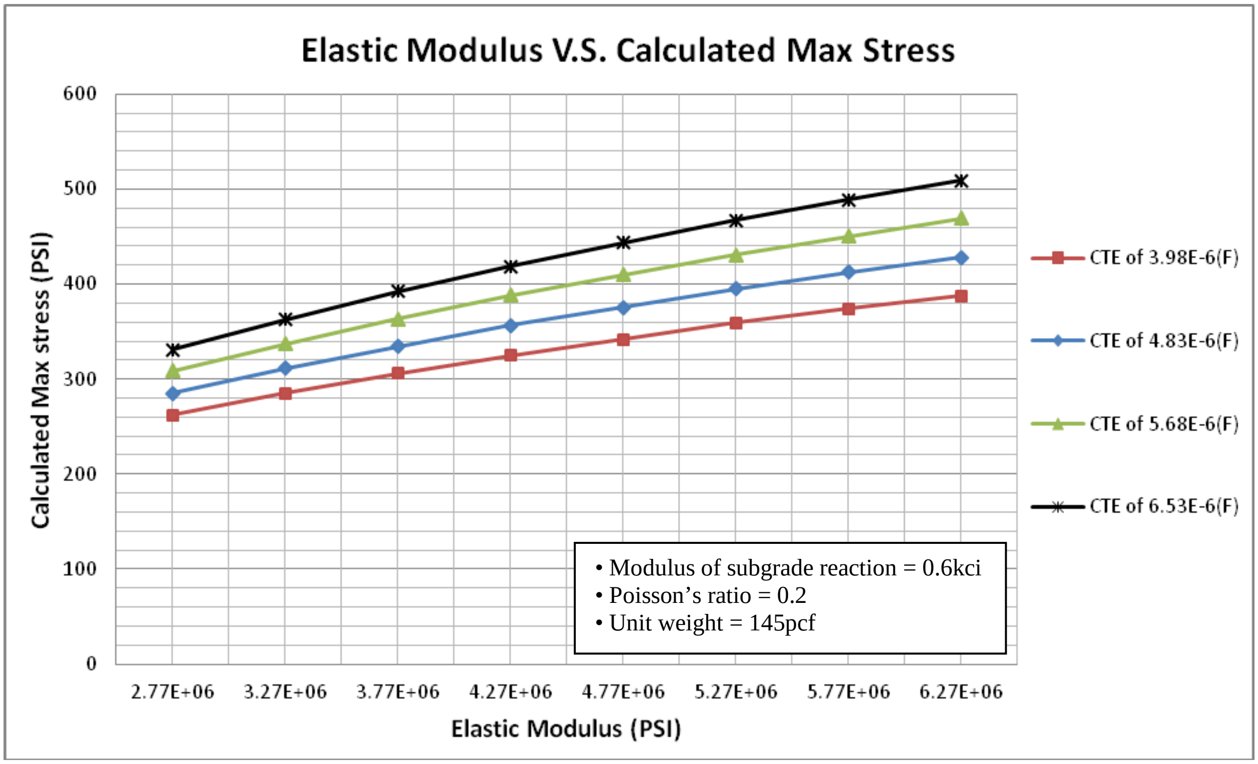


Figure 5-25. Calculated maximum stresses for different elastic modulus and CTE.

5.4.2.3 The Effect of Poisson's Ratio and Unit Weight

The Poisson's ratio and unit weight for the concrete pavement in the LTPP database ranged from 0.1 to 2.5 and 135 pcf to 155 pcf, respectively. Using the different values of Poisson's ratio and unit weight in the range of LTPP database, the maximum stresses in 13-inch slabs under the critical loading conditions were computed by FEACONS program and presented in Figure 5-26. The figure shows that the calculated maximum stress increases by less than 1% when the unit weight changes from 135 to 155 pcf. Also, the calculated maximum stress increases by less than 3% when the Poisson's ration changes from 0.10 to 0.25.

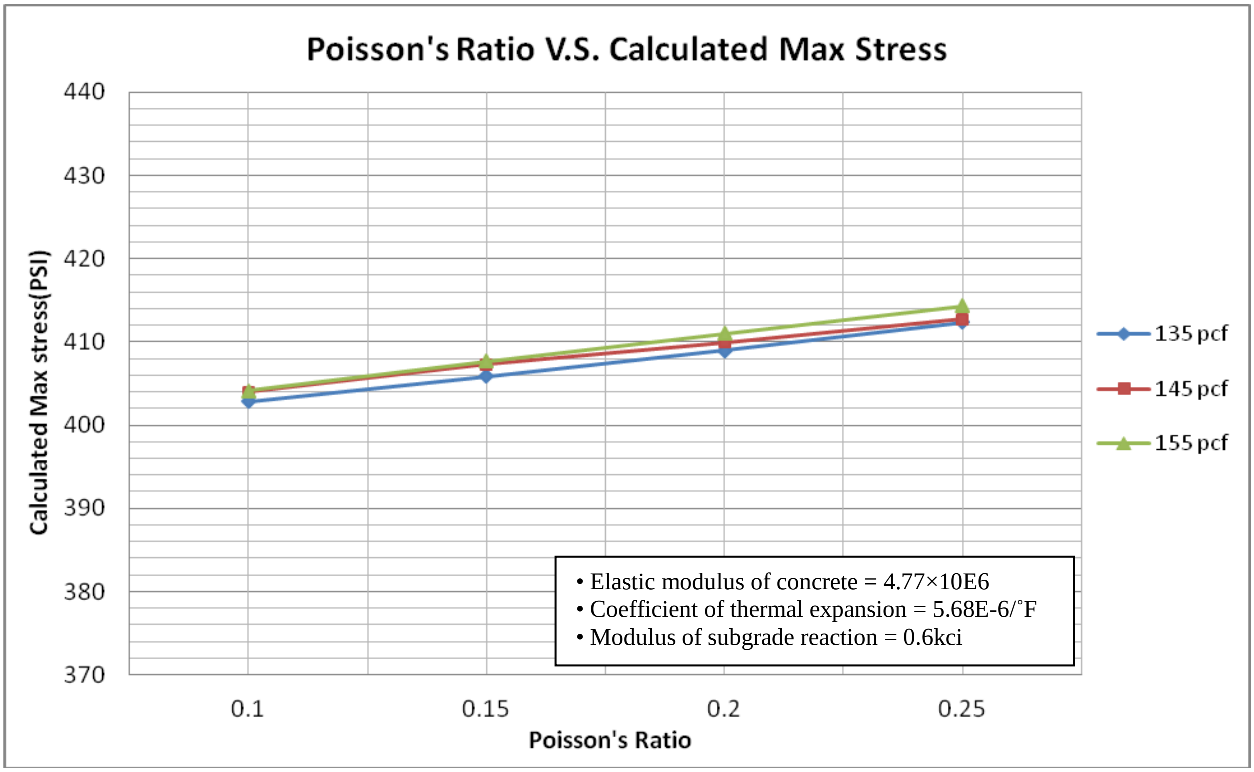


Figure 5-26. Calculated maximum stress for different Poisson's ratio and unit weight.

5.4.3 Evaluation of the Recommended Florida Concrete Pavement Designs

From the result of critical stress analysis, the modulus of subgrade reaction has little effect on the maximum stress in the concrete slabs when the slab thickness is 11 inches or higher. Thus the maximum stresses in Types I-A, I-B and II pavements would be affected mainly by the elastic modulus and coefficient of thermal expansion of the concrete as well as the slab thickness. The extent of the effects of these three factors on the Florida designs were evaluated and presented in this section. The following parameters were used in the critical stress analysis using the FEACONS program:

Slab length = 15 ft

Slab width = 12 ft

Modulus of subgrade reaction, k_s = 0.6 kci

Edge stiffness, k_e = 30 ksi

Joint linear stiffness, k_l = 500 ksi

Joint torsion stiffness k_t = 1000 k-in/in

Applied load = 22 kips

Temperature differential = 20˚F

Poisson's ratio = 0.2

Unit weight =145 pcf

Concrete using Brookville aggregate is used in the analysis. The coefficient of thermal expansion of the concrete is 5.68 X 10^{-6} /°F. Since the elastic modulus of the concrete changes as the strength of the concrete changes, the estimated elastic modulus (E) of concrete is estimated from the modulus of rupture (f_r) from the following regression equation (Tia et al., 1989).

$$E = 4.20(w^{1.5}) f_r \tag{5-3}$$

Where:

E = Elastic modulus, psi,

w = Unit weight, pcf; and,

f_r = Modulus of rupture, psi.

Table 5-6. Maximum Computed Stresses and Stress-to-strength Ratios for Concrete Pavement with Different Concrete Flexural Strength and Slab Thicknesses

	Maximum Computed stress (PSI)				**Stress-to-strength Ratio**			
Flexural Strength (PSI)	500	600	700	800	500	600	700	800
11 inch thick slab	413	457	500	537	0.83	0.76	0.71	0.67
12 inch thick slab	385	425	461	494	0.77	0.71	0.66	0.62
13 inch thick slab	358	394	425	454	0.72	0.66	0.61	0.57
14 inch thick slab	333	365	392	417	0.67	0.61	0.56	0.52

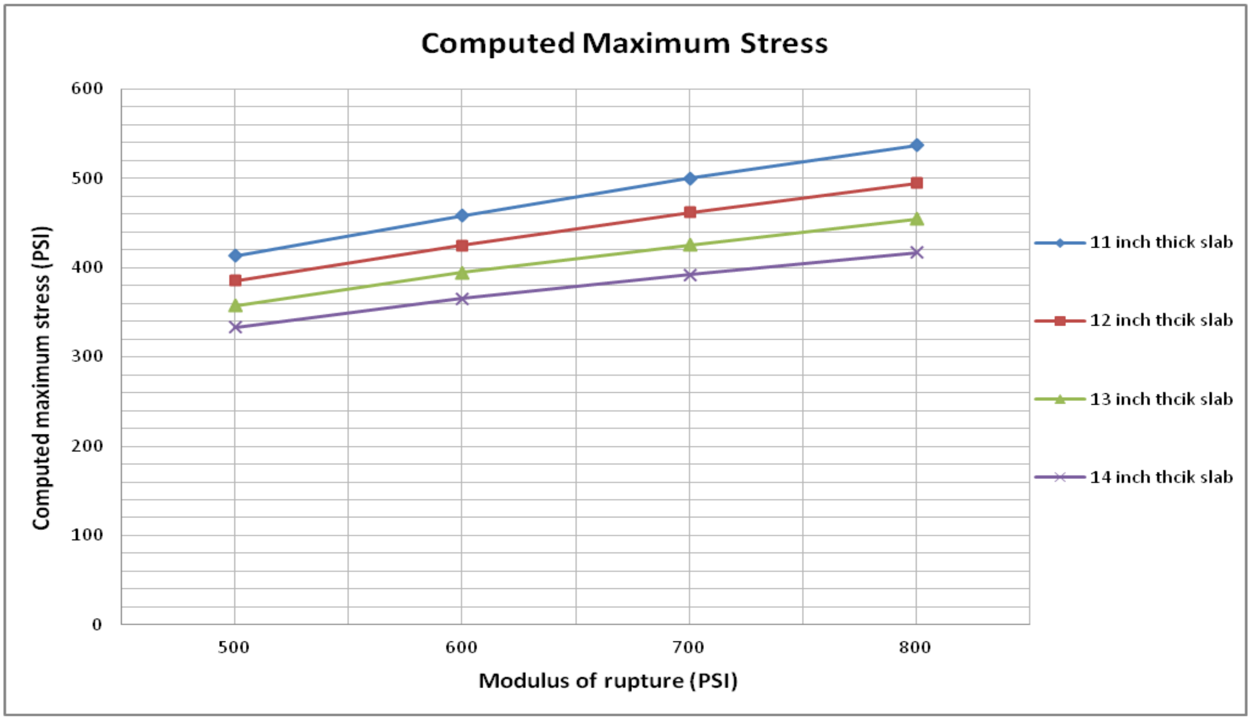


Figure 5-27. Calculated maximum stress with different modulus of rupture and unit slab thickness.

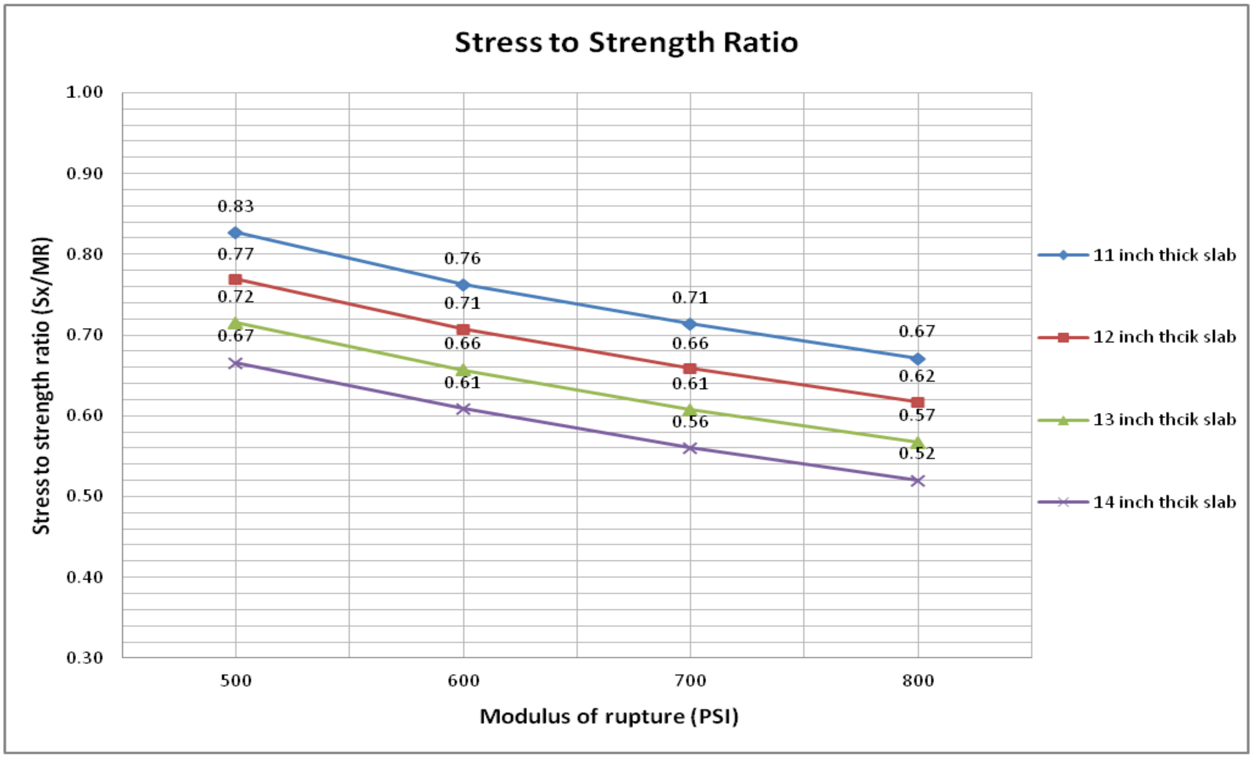


Figure 5-28. Calculated stress-to-strength ratio with different modulus rupture and unit slab thickness.

Figure 5-27 shows plots of computed maximum stress versus modulus of rupture for different slab thicknesses. It can be seen that the maximum stress generally increases with increasing modulus of rupture. This is due to the increase in elastic modulus of the concrete with increase in flexural strength. Figure 5-28 shows plots of maximum stress-to-strength ratio versus modulus of rupture of concrete for different concrete slab thicknesses. It can be seen that the stress-to-strength ratio decreases as the modulus of rupture of the concrete increases, if the concrete is made of the same type of aggregate as in the case of this analysis. For concrete slab thickness of 13 inches, the computed stress-to-strength ratios are 0.72, 0.66, 0.61 and 0.57 for concrete flexural strength of 500, 600, 700 and 800 psi, respectively. For concrete slab thickness of 14 inches, the computed stress-to-strength ratios are 0.67, 0.61, 0.56 and 0.52 for concrete flexural strength of 500, 600, 700 and 800 psi, respectively.

CHAPTER 6
LIFE CYCLE COST OF LONG-LIFE CONCRETE PAVEMENTS FOR FLORIDA

6.1 Concrete Pavement Designs

From the results of MEPDG analysis performed in this study, it appears that three typical Florida concrete pavement designs, which have been referred to as Type I-A, Type I-B and Type II designs in this study, could possibly be used as long-life pavements if the slab thickness was adequate and the concrete properties were right – low elastic modulus, low coefficient of thermal expansion and adequate flexural strength.

Table 6-1 shows the features of these three designs. Type I-A pavement has a PCC slab of varied thickness with a 4-inch treated permeable base over a 2-inch asphalt structural course. Type I-B pavement consists of a PCC slab of varied thickness with a 4-inch asphalt concrete base. Both Type I-A and I-B pavements have a 12-inch Type B stabilized subgrade with a minimum LBR of 40. Type II pavement has a PCC slab of varied thickness with a 6-inch granular permeable base over 54-inch A-3 soil.

Table 6-1. Features of Type I-A, Type I-B and Type II Concrete Pavement Designs

Type	Design Slab Thickness		Base Type	Subbase Type
	Minimum (inch)	Maximum (inch)		
Type I-A	8	N/A	4 inch treated permeable base over 2 inch asphalt structural course	12 inch Type B stabilized subgrade (LBR 40)
Type I-B			4 inch asphalt concrete base	12 inch Type B stabilized subgrade (LBR 40)
Type II			6 inch special stabilized sub base over 54 inch special select embankment	None

6.2 Concrete Pavement General Structure

To estimate the cost of construction for pavements with these three types of designs previously described, a general construction design was selected. This general design is shown in Figures 6-1 and 6-2. The cost estimates for these three types of pavement were developed using a 4-lane highway design with 2 lanes traveling in each direction for a length of 10 miles. Concrete pavement slabs with a width of 13 ft and a length of 15 ft were used. Concrete shoulders with a width of 3 ft and joint spacing of 15 ft were used. Concrete pavement and shoulder thickness was varied from 10 to 14 inches, increasing by 1 inch increments, and the total construction costs of the pavements with the various concrete thicknesses were estimated.

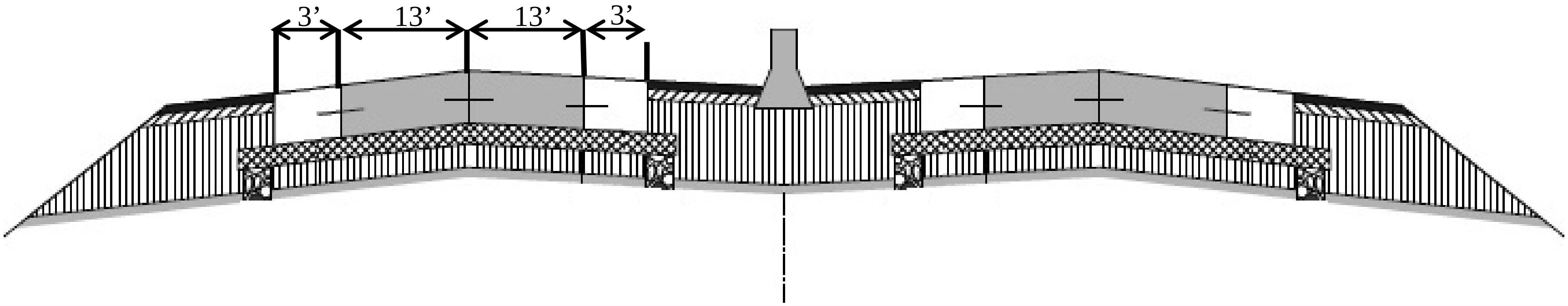


Figure 6-1. Type I-A, Type I-B and II concrete general structural design – sectional view.

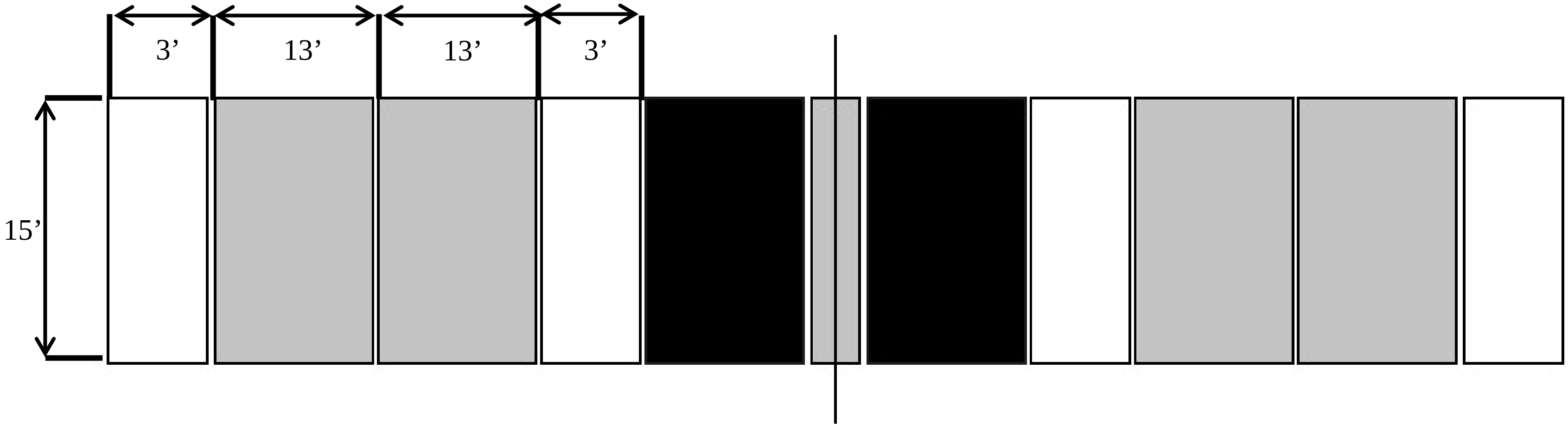


Figure 6-2. Type I-A, Type I-B and II concrete general structural design – plan view.

6.3 Estimation of Construction Cost of Florida Concrete Pavements

6.3.1 Construction Cost of Concrete Slabs and Shoulders

Historical construction costs of concrete slabs and shoulders were provided by Concrete Paving Alliance of Florida and were used in the estimation of the construction costs of concrete slabs and shoulders. Table 6-2 shows the historical unit cost of concrete construction in terms of dollars per square yard of concrete for various slab thicknesses. These unit costs include the project overhead costs for the PCC pavement work, the cost of manufacturing the concrete on site in a concrete batch plant, furnishing and placement of all reinforcement, placement and finish of the concrete pavement, sawing of the joints and clean-up costs.

Table 6-2. PCC Pavement Historical Cost Data

PCC Pavement Thickness (inches)	Cost per SY ($)
8.5	$ 38.94
9	$ 48.17
10	$ 48.27
10.5	$ 48.38
11.5	$ 53.28
12	$ 64.20
14	$ 64.87

The unit costs were plotted against concrete thickness as shown in Figure 6-3, and a trend line was developed through linear regression analysis. The developed trend line was then used to estimate the construction cost of the concrete slabs and shoulders of various thicknesses.

Table 6-3 shows the estimated construction cost for the concrete slabs for 10 miles of 4-lane concrete pavement of various thicknesses as described in Section 6.2. Table 6-4 shows the estimated construction cost for the concrete shoulders of various thicknesses. The detailed calculations are shown in Appendix A.

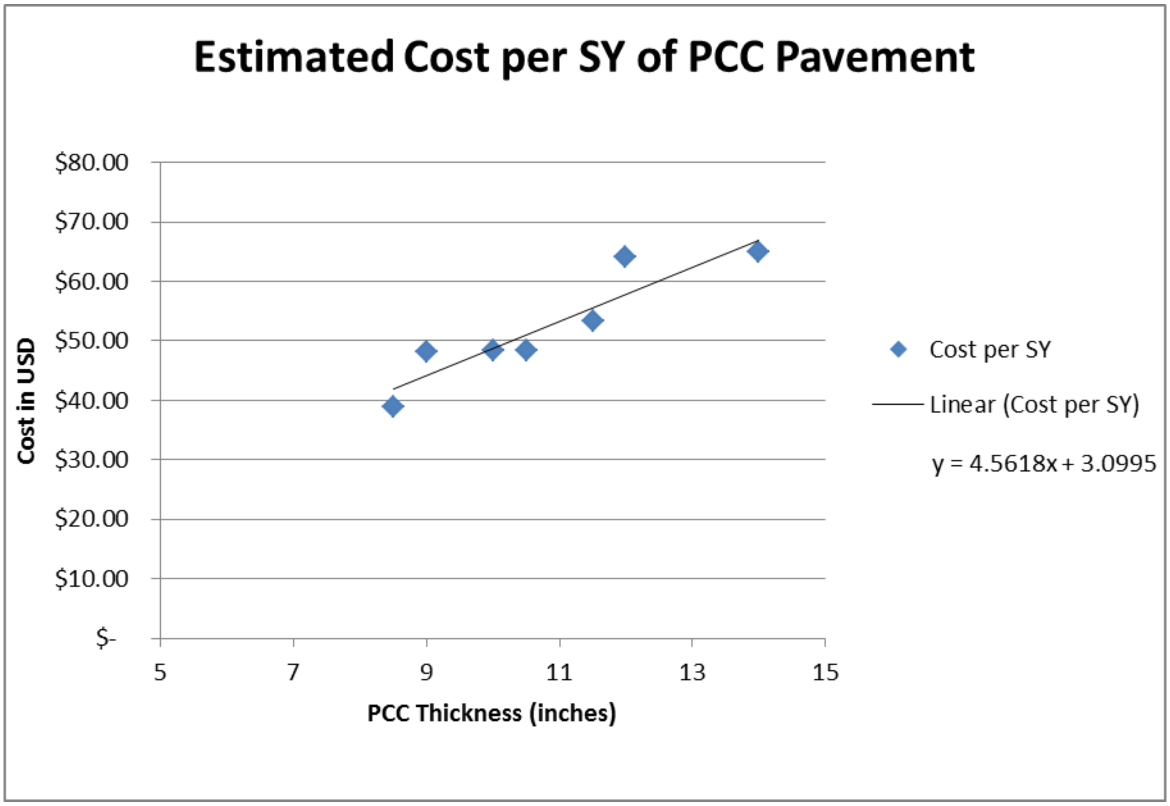


Figure 6-3. Estimated unit cost of concrete slabs and shoulders

Table 6-3. Estimated Cost of Concrete Slabs for 10 Miles of 4-Lane Pavement

Mainline Pavement Thickness	Cost/Unit	Units	Quantity/ Direction	Total Quantity	Cost ($)
10 inch	$48.72	SY	152533	305067	$14,862,085
11 inch	$53.28	SY	152533	305067	$16,253,738
12 inch	$57.84	SY	152533	305067	$17,645,392
13 inch	$62.40	SY	152533	305067	$19,037,045
14 inch	$66.96	SY	152533	305067	$20,428,698

Table 6-4. Estimated Cost of Concrete Shoulders for 10 Miles of 4-Lane Pavement

Shoulder Pavement Thickness	Cost/Unit	Units	Quantity/ Direction	Total Quantity	Cost ($)
10 inch	$ 48.72	CY	35200	70400	$ 3,429,712
11 inch	$ 53.28	CY	35200	70400	$ 3,750,863
12 inch	$ 57.84	CY	35200	70400	$ 4,072,013
13 inch	$ 62.40	CY	35200	70400	$ 4,393,164
14 inch	$ 66.96	CY	35200	70400	$ 4,714,315

6.3.2 Construction Cost of Base and Subbase

The historical cost data, as provided by FDOT, were used in estimating the construction cost of the base and subbase for Type I-A, Type I-B and Type II pavements. Tables 6-5, 6-6 and 6-7 show the estimated costs of the individual items as well as the total cost of the base and subbase for 10 miles of 4-lane concrete pavements with Type I-A, Type I-B and Type II designs, respectively. The detailed calculations are shown in Appendix A.

Table 6-5. Estimated Cost of Base and Subbase for 10 Miles of 4-Lane Pavement with Type I-A Design

Pavement Items:	Cost/Unit	Units	Quantity/ Direction	Total Quantity	Cost ($)
4" Permeable Base Layer	$ 6.61	SY	152533	305067	$2,016,491
2" Structural AC Inter-Layer	$ 85.00	Ton	15307	30613	$2,602,112
Prime Coat (Mainline and Shoulders)	$ 4.00	Gallons	93867	187733	$ 750,933
Type B Stabilized	$ 4.00	SY	152533	305067	$1,220,267
Total					**$6,589,803**

Table 6-6. Estimated Cost of Base and Subbase for 10 Miles of 4-Lane Pavement with Type I-B Design

Pavement Items:	Cost/Unit	Units	Quantity/ Direction	Total Quantity	Cost ($)
4" Structural AC Inter-Layer	$ 85.00	Ton	61226	122452	$10,408,449
Prime Coat (Mainline and Shoulders)	$ 4.00	Gallons	93867	187733	$ 750,933
Type B Stabilized	$ 4.00	SY	152533	305067	$1,220,267
Total					**$12,379,649**

Table 6-7. Estimated Cost of Base and Subbase for 10 Miles of 4-Lane Pavement with Type II Design

Pavement Items:	Cost/Unit	Units	Quantity/ Direction	Total Quantity	Cost ($)
6" Permeable Base Layer	$ 11.84	SY	152533	305067	$3,611,989
54 " Prepared Soil	$ 8.06	SY	152533	305067	$2,458,837
Total					**$6,070,827**

6.3.3 Total Construction Cost of Pavement

The estimated total costs of construction for the entire pavement were obtained by adding the construction cost of the concrete slabs and shoulders to the construction of the base and subbase for the pavements with the three designs.

Tables 6- 8, 6-9 and 6-10 show the estimate total costs of pavements with concrete thicknesses of 10 through 14 inches, for Type I-A, Type I-B and Type II designs, respectively. The predicted service lives of these pavements as predicted by MEPDG and presented in Section 3.9 are also shown on these tables.

Table 6-8. Total Cost of Construction for 10 Miles of 4-Lane Pavement with Type I-A Design.

Pavement Design Thickness	Cost ($)	Predicted Life (yrs)
10 inch	$24,881,600	27
11 inch	$26,594,404	33
12 inch	$28,307,208	42
13 inch	$30,020,012	51
14 inch	$31,732,816	56

Table 6-9. Total Cost of Construction for 10 Miles of 4-Lane Pavement with Type I-B Design.

Pavement Design Thickness	Cost ($)	Predicted Life (yrs)
10 inch	$30,671,446	24
11 inch	$32,384,250	30
12 inch	$34,097,054	40
13 inch	$35,809,858	50
14 inch	$37,522,662	53

Table 6-10. Total Cost of Construction for 10 Miles of 4-Lane Pavement with Type II Design.

Pavement Design Thickness	Cost ($)	Predicted Life (yrs)
10 inch	$24,362,624	28
11 inch	$26,075,428	36
12 inch	$27,788,232	43
13 inch	$29,501,036	56
14 inch	$31,213,839	60

6.4 Life Cycle Cost Analysis

Using the predicted lives of the pavement of different concrete slab thicknesses, the total estimated costs for the various types of pavements were converted to equivalent annual costs using different interest rates. The conversion from total construction cost (or present worth) to equivalent annual cost (or annual cost) was done using the following equation:

$$A = PW / \{ [(1 + i)^n – 1] / i (1 + i)^n \} \qquad (6\text{-}1)$$

Where:

A = Annual cost

PW = Present worth (or total construction cost)

n = Number of year (or expected life)

I = Interest rate

When the interest of construction cost was not considered, the annual cost was calculated by dividing the total construction cost by the expected life of the pavement in years.

Table 6-11 presents the computed annual costs for 10 miles of 4-lane Type I-A pavements of various concrete slab thicknesses using interest rates of 3.5% and 5% and also for the case when interest of cost is not considered. Similarly, Tables 6-12 and 6-13 present the computed annual costs for 10 miles of 4-lane Type I-B and Type II pavements, respectively.

In comparing among the annual costs for the three designs, it can be seen that the Type II design gives the lowest annual cost, followed by Type I-A and Type I-B. The most cost effective slab thickness to be used depends on the interest to be used in the analysis. When cost of interest is not considered, using 14-inch concrete slab gives the least annual cost for all three designs. With concrete slab thickness of 14 inches, the expected service for Type I-A, I-B and II designs are 56, 53 and 60 years, respectively. When an interest rate of 3.5% is considered, the cost effective slab thickness for all three designs is 13 inches. With concrete slab thickness of 13 inches, the expected service for Type I-A, I-B and II designs are 51, 50 and 56 years, respectively.

Table 6-11. Computed Annual Cost for 10 Miles of 4-Lane Type I-A Pavement

Concrete Slab Thickness (inch)	Total Cost ($)	Expected Life (year)	No Interest	I= 3.5%	I = 5%
			Annual Cost ($)		
10	24,881,600	27	921,541	1,439,461	1,699,211
11	26,594,404	33	805,891	1,371,538	1,661,885
12	28,307,208	42	673,981	1,296,421	1,624,684
13	30,020,012	51	588,628	1,270,494	1,636,951
14	31,732,816	56	566,657	1,300,008	1,697,074

Table 6-12. Computed Annual Cost for 10 Miles of 4-Lane Type I-B Pavement

Concrete Slab Thickness (inch)	Total Cost ($)	Expected Life (year)	No Interest	I= 3.5%	I = 5%
			Annual Cost ($)		
10	30,671,446	24	1,277,977	1,909,998	2,222,787
11	32,384,250	30	1,079,475	1,760,775	2,106,642
12	34,097,054	40	852,426	1,596,672	1,987,114
13	35,809,858	50	716,197	1,526,707	1,961,547
14	37,522,662	53	707,975	1,566,233	2,028,976

Table 6-13. Computed Annual Cost for 10 Miles of 4-Lane Type II Pavement

Concrete Slab Thickness (inch)	Total Cost ($)	Expected Life (year)	No Interest	I= 3.5%	I = 5%
			Annual Cost ($)		
10	24,362,624	28	870,094	1,378,989	1,635,281
11	26,075,428	36	724,317	1,285,106	1,575,854
12	27,788,232	43	646,238	1,259,512	1,583,744
13	29,501,036	56	526,804	1,208,578	1,577,718
14	31,213,839	60	520,231	1,251,320	1,648,970

CHAPTER 7
SUMMARY AND RECOMMENDATIONS

7.1 Findings from MEPDG Analyses

The MEPDG model, calibrated for the Florida conditions, was used to analyze the performance of three typical concrete pavement designs in Florida to evaluate their suitability for use as long-life concrete pavements and the effects of various design parameters on their performance. These three designs are referred to as Types I-A, I-B and II in this report and are presented in Table 3-1 and Figure 3-1.

Concrete slab thickness, concrete flexural strength, and the aggregate used in the concrete were found to be the three most significant factors affecting the predicted performance of the pavement evaluated. The three aggregates used in the analysis included Brooksville limestone, Calera limestone and river gravel. For concrete with the same design flexural strength, Brooksville limestone was shown to give the best predicted performance, followed by Calera limestone, and the river gravel. The better predicted performance was due to the relatively low elastic modulus and low coefficient of thermal expansion of the concrete made with Brooksville limestone. When the same Brooksville aggregate was used in the concrete, increasing the modulus of rupture of the concrete gave improved predicted performance and increased service life to the pavement.

MEPDG analyses were performed to evaluate the effects of (1) types of base material, (2) stiffness of the base material, (3) erodibility of the base material, and (4) friction between the concrete and base layer on the predicted performance of these three concrete pavement designs. The predicted performance of the pavement appeared to have improved slightly with an increase in base thickness. However, the type of base material and the stiffness of the base material appeared to have no significant effect on the predicted performance. Using different erodibility factor and friction factor for the base materials appeared to have no significant effect on the predicted performance according to the results of the MEPDG analyses.

MEPDG analyses were performed to determine the predicted service lives of concrete pavements using Type I-A, I-B and II designs. An initial AADTT of 17,000, which represents high-volume truck traffic, was used in the analysis. The concrete made with Brooksville aggregate and with a modulus of rupture of 650 psi was used. The predicted service lives for these three designs with pavement thickness varying from 10 to 14 inches are presented in Table 3-44. When the concrete slab thickness is 13 inches or more, the expected service of all three designs are 50 years or more. Among the three designs, Type II has the best predicted performance, followed by Type I-A and then Type I-B.

7.2 Findings from Drainage Evaluation

Sensitivity analysis was performed using the DRIP 2.0 program to evaluate the drainage characteristics of Type I-A and Type II concrete pavement designs using the steady flow method and the time-to-drain method. The required base permeability and the corresponding times-to-drain for various combinations of number of lanes and pavement cross slopes for Type I-A and Type II designs are presented in Table 4-31 and 4-32, respectively. In comparison, the Type II design with a 6-inch permeable base shows better drainage characteristics than the Type I-B design with a 4-inch permeable base. For pavements with 2 to 4 lanes and pavement cross-slope from 2% to 6%, the required base permeability varies from 200 to 700 ft/day for Type I-A design, and varies from 100 to 500 ft/day for Type II design. If the same base permeability, pavement slope and number of lanes are used in both designs, Type II design has a lower time-to- drain than the Type I-A design.

Type I-B concrete pavement has a 4-inch asphalt concrete base layer. Since the asphalt concrete layer is a non-permeable layer, the steady-flow analysis and the time-to-drain analysis could not be appropriately performed on this type of pavement. Long-term monitoring of Type I-B pavement is needed to determine whether or not there will be drainage related issue with the use of asphalt concrete base in concrete pavement in Florida.

7.3 Findings from Analysis of LTPP Data and Critical Stress Analysis

The Long-Term Pavement Performance (LTPP) database was used to evaluate the effects of various factors, which included environmental conditions, drainage types, concrete properties, base types, subgrade types and concrete slab length, on performance of Jointed Plain Concrete Pavements (JPCP) in the US with emphasis on Florida and its neighboring states. Critical stress analysis was also performed on the selected LTPP JPCP sections to determine the maximum stress in the concrete slab under a critical load and temperature condition. The maximum computed critical stress for each condition was divided by the modulus of rupture of the concrete to determine the stress-to-strength ratio.

The computed critical stress-to-strength ratio was found to be the most significant parameter which can be related to the performance of the LTPP pavements. This relationship is presented in Figure 5-23. A lower stress-to-strength ratio is related to better observed pavement performance. The better performing pavements were noted to have a computed stress-to-strength ratio of less than 0.70.

Critical stress analysis was also performed on the three Florida concrete pavement designs. Results from the critical stress analysis show that the most significant factors affecting the stress-to-strength ratios are the concrete slab thickness and the concrete properties which include the elastic modulus, modulus of rupture and coefficient of thermal expansion. Variations in the base and subbase properties were found to have minimal effects on the stress-to-strength ratios for concrete slab thickness of 11 inches or higher. This observed result agrees well with the findings from the MEPDG analysis that the most significant factors affecting the performance of the concrete pavement are the concrete slab thickness and the concrete properties.

Similar to the results from MEPDG analysis, when the same aggregate is used in the concrete, increasing the flexural strength of the concrete will result in better predicted pavement performance. When the concrete made with Brooksville aggregate was used, for concrete slab thickness of 13 inches, the computed stress-to-strength ratios were 0.72, 0.66, 0.61 and 0.57 for concrete flexural strength of 500, 600, 700 and 800 psi, respectively.

7.4 Findings from Life Cycle Cost Analysis

The cost estimates for Type I-A, Type I-B and Type II pavement with concrete slab varying from 10 inches to 14 inches were developed. The predicted service lives of these pavements were based on the results of MEPDG analysis using a concrete made with Brooksville aggregate and modulus of rupture of 650 psi. The estimated total costs, predicted service lives and annual costs for these three designs are shown in Tables 7-11, 7-12 and 7-13. Type II design has the lowest cost estimate, which is slightly more than that for Type I-A design, while Type I-B design has the highest cost estimate.

When cost of interest was not considered, using 14-inch concrete slab gave the least annual cost for all three designs. With concrete slab thickness of 14 inches, the expected service for Type I-A, I-B and II designs are 56, 53 and 60 years, respectively. When an interest rate of 3.5% was considered, the most cost effective slab thickness for all three designs was 13 inches. With concrete slab thickness of 13 inches, the expected service for Type I-A, I-B and II designs are 51, 50 and 56 years, respectively.

7.5 Recommended Long-Life Concrete Pavement Designs for Florida

From the results of in this study, it appears that the three typical Florida concrete pavement designs, which have been referred to as Type I-A, Type I-B and Type II designs in this study, can be used as long-life pavements if the slab thickness was adequate and the concrete has low elastic modulus, low coefficient of thermal expansion and adequate flexural strength. Among the three designs evaluated, Type II pavement has the best predicted performance from the MEPDG analysis and the best drainage characteristics from the results of the drainage evaluation using the steady flow method and the time-to-drain method. Type II pavement also has the lowest cost estimate.

Type II design is recommended as the preferred design for use as long-life concrete pavements in Florida. However, if the special select A-3 soil is not available, Type I-A and Type I-B can also be used. A concrete slab thickness of 13 or 14 inches is recommended to be used. The present FDOT construction specifications for these three types of design are to be followed. In addition to meeting the present FDOT specification requirements for these three designs, the concrete mixture to be used must be designed and evaluated by the following procedure:

(1) Design the concrete mix to give a flexural strength of at least 650 psi at 28 days. Use an aggregate which has a past history of producing concrete of low elastic modulus and low coefficient of thermal expansion.

(2) Measure the flexural strength, elastic modulus and coefficient of thermal expansion the designed concrete mix at 28 days.

(3) Perform MEPDG analysis to evaluate the predicted performance of the designed pavement for a design life of 50 years, using the measured concrete flexural strength, elastic modulus and coefficient of thermal expansion as input properties for the concrete. If the predicted life of the pavement is at least 50 years, the concrete mix would be acceptable for the project. If the predicted life is less than 50 years, a new concrete mix can be designed by either specifying a higher flexural strength or using a different aggregate. Steps 1 through 3 would be repeated until an acceptable concrete mix for the project is obtained.

It is recommended that FDOT establish a database of concrete mix designs which are acceptable for use in long-life concrete pavements, so that optimum concrete mixes can be designed and selected efficiently for this purpose. A research study to evaluate the drainage requirements for these concrete pavement designs is also recommended.

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APPENDIX A
TYPE I COST ESTIMATE SAMPLE CALCULATION (12” PCC)

Main Pavement – PCC

$= lenght * Lanes * slab\ width$

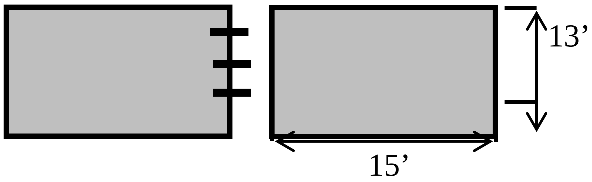
$= 10\ miles \left(\frac{1760\ yd}{1\ mile}\right) * 4\ Lanes * 13ft \left(\frac{1\ yd}{3\ ft}\right)$

= 305,067 yd^2

Main Pavement – Dowel Bar Information

Dowel Information:

- Diameter: 1.5”
- Spacing: 12” c/c
- Length: 18”



No. of Slabs:

$= 10\ miles \left(\frac{1760\ yd}{1\ mile}\right) = 52{,}800ft$

$= \frac{52{,}800\ ft}{15\ ft/_{slab}} = 3520\ slabs$

No. of Dowels:

$= 3520\ slabs * \left(\frac{13\ Dowels}{slab}\right) = 45{,}760\ Dowels/lane$

Dowel Length:

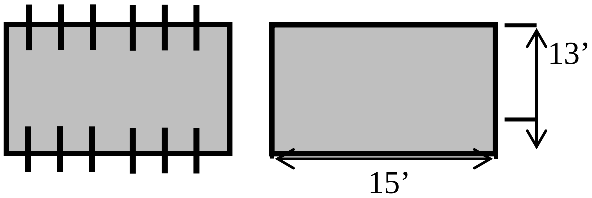
$= 45{,}760\ dowels * 4\ lanes * 18\ inches * \left(\frac{1\ ft}{12\ inches}\right)$

$= 247{,}560\ ft$

Main Pavement – Tie Bar Information

Tie bar Information:

- Diameter: 1.5”
- Spacing: 24” c/c
- Length: 25”



No. of Slabs:

$= 10\ miles \left(\frac{1760\ yd}{1\ mile}\right) = 52{,}800ft$

$= \frac{52{,}800\ ft}{15\ ft/_{slab}} = 3520\ slabs$

No. of Dowels:

$$= 3520\ slabs * \left(\frac{24\ bars}{slab}\right) = 84{,}480\ bars$$

Shoulder Pavement – PCC

$$= lenght * Lanes * shoulder\ width$$

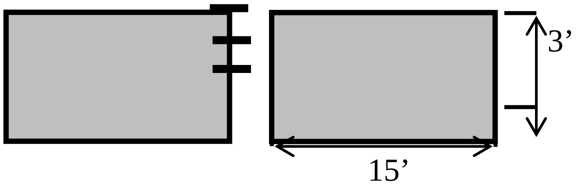
$$= 10\ miles \left(\frac{1760\ yd}{1\ mile}\right) * 4\ shoulders * 3ft \left(\frac{1\ yd}{3\ ft}\right)$$

= 70,400 yd^2

Shoulder Pavement – Dowel Bar

Dowel Information:

- Diameter: 1.5”
- Spacing: 12” c/c
- Length: 18”



No. of Slabs:

$$= 10\ miles \left(\frac{1760\ yd}{1\ mile}\right) = 52{,}800ft$$

$$= \frac{52{,}800\ ft}{15\ ft/_{slab}} = 3520\ slabs$$

No. of Dowels:

$$= 3520\ slabs * \left(\frac{3\ Dowels}{slab}\right) = 10{,}560\ Dowels/shoulder$$

Dowel Length:

$$= 10{,}560\ dowels * 4\ shoulders * 18\ inches * \left(\frac{1\ ft}{12\ inches}\right)$$

$$= 63{,}360\ ft$$

4 inch Permeable Base

$$= slab\ lenght * Lanes * slab\ width$$

$$= 10\ miles \left(\frac{1760\ yd}{1\ mile}\right) * 4\ Lanes * 13ft \left(\frac{1\ yd}{3\ ft}\right)$$

$$= 305{,}066.67\ yd^2$$

1 inch Structural AC Inter-Layer

General Information:		Sp. Gra
- Air Voids:	7%	
- Asphalt Content:	6%	1.04
- Aggregate:		2.50

- Density of Water: 62.4 lb/ft^3

$$MTD = \frac{100}{\left(\frac{6}{1.04}\right) + \left(\frac{94}{2.5}\right)} = 2.31\ {}^{g}/_{cc}$$

$$Density\ = MTD * (1 - 0.07)$$
$$= 2.14 * 62.4 = 133.81\ {}^{lb}/_{ft^3}$$
$$= 133.81\frac{lb}{ft^3} * \left(\frac{1\ TN}{2000\ lbs}\right) = 3.61\ \frac{TN}{yd^3}$$

Amount of asphalt necessary:

$$= \frac{3.61\ TN}{yd^3} * 10\ miles\left(\frac{1760\ yd}{1\ mile}\right) * 4\ lanes * 1\ inch\left(\frac{1\ yd}{36\ inches}\right) * 13\ ft\left(\frac{1\ yd}{3\ ft}\right)$$
$$= 30{,}613\ TN$$

Prime Coat

General Information:

- Approximated use by Asphalt Institute: $\frac{0.5\ gal}{yd^2}$

$$= 10\ miles\left(\frac{1760\ yd}{1\ mile}\right) * [13ft(2\ lanes) + 3ft(2\ shoulders)]$$
$$= 187{,}733yd^2 * \left(\frac{0.5\ gal}{1\ yd^2}\right) * 2\ directions$$
$$= 187{,}733\ gallons$$

Type B Stabilization

$$= slab\ lenght * Lanes * slab\ width$$
$$= 10\ miles\left(\frac{1760\ yd}{1\ mile}\right) * 4\ Lanes * 13ft\left(\frac{1\ yd}{3\ ft}\right)$$
$$= 305{,}066.67\ yd^2$$

APPENDIX B
FDOT GIS AADTT DATA (TOP 300)

No.	DIST	COSITE	ROADWAY	DESC_FRM	DESC_TO	TruckAADT
1	4	862807	86095000	Bridge No-860535	US 1/SR 5 SB	22110
2	4	932192	93220000	Bridge No-930189	Bridge No-930499	21625
3	4	862493	86070000	86095000/EB-I595	SR 736/DAVIE BLVD	20468
4	5	920321	92130000	RAMP 92473001	N/A	20193
5	5	920321	92130000	Bridge No-920094	RAMP 92473001	20193
6	4	862458	86070000	Bridge No-860554	86095000/EB-I595	18972
7	4	862498	86070000	SR 736/DAVIE BLVD	SR 842/BROWARD BLVD	18088
8	4	862454	86070000	Bridge No-860530	Bridge No-860576	17952
9	4	862500	86070000	SR 838/SUNRISE BLVD	Bridge No-860117	17816
10	4	862456	86070000	Bridge No-860579	Bridge No-860554	17408
11	4	862394	86070000	Bridge No-860531	Bridge No-860530	17340
12	4	862455	86070000	Bridge No-860576	Bridge No-860579	17204
13	4	862501	86070000	Bridge No-860117	Bridge No-860130	17160
14	4	862507	86070000	Bridge No-860124	PALM BCH. CO. LN.	16767
15	4	932199	93220000	N/A	10TH AVE N	16384
16	4	862487	86070000	DADE CO. LN.	Bridge No-860529	15640
17	6	870572	87260000	NW 58 ST	Bridge No-870964	15561
18	5	360438	36210000	Bridge No-360022	Bridge No-360043	15535
19	7	100087	10190000	Bridge No-100599	Bridge No-100601	15424
20	7	102016	10190000	Bridge No-100697	Bridge No-100110	15240
21	7	100086	10190000	Bridge No-100601	N/A	15232

22	2	722361	72280000	Bridge No-720334	SR 5	15191
23	4	862502	86070000	Bridge No-860130	Bridge No-860239	15040
24	4	862499	86070000	SR 842/BROWARD BLVD	SR 838/SUNRISE BLVD	14787
25	5	360439	36210000	Bridge No-360018	Bridge No-360022	14783
26	8	972020	75470000	N/A	RAMP 161 SB ON	14496
27	5	360317	36210000	Bridge No-360001	Bridge No-360063	14419
28	4	932197	93220000	HYPOLUXO RD	CR 812/LANTANA RD	14368
29	1	160103	16320000	HILLSBOROUGH CO LINE	ON RAMP TO I-4	14108
30	4	932191	93220000	N/A	Bridge No-930189	14097
31	7	100112	10190000	N/A	Bridge No-100607	14065
32	7	105352	10190000	Bridge No-100137	10320000 MAINLINE	14000
33	7	105609	10190000	N/A	Bridge No-100697	14000
34	7	105609	10190000	Bridge No-100123	N/A	14000
35	6	870573	87260000	Bridge No-870964	Bridge No-870975	13908
36	7	102015	10190000	Bridge No-100110	Bridge No-100137	13800
37	6	872525	87260000	Bridge No-870778	Bridge No-870982	13680
38	6	870570	87260000	N/A	Bridge No-870778	13671
39	7	102018	10190000	Bridge No-100120	Bridge No-100123	13640
40	5	770268	77160000	N/A	N/A	13625
41	5	770268	77160000	LAKE MARY BLVD	N/A	13625
42	6	870553	87260000	Bridge No-870975	Bridge No-870757	13623
43	5	770267	77160000	SR 436	Bridge No-770022	13550
44	2	720860	72280000	Bridge No-720219	Bridge No-720220	13346

45	2	722353	72280000	Bridge No-720220	Bridge No-720331	13205
46	4	860331	86070000	Bridge No-860529	Bridge No-860531	13124
47	6	870571	87260000	Bridge No-870982	NW 58 ST	13110
48	4	932198	93220000	CR 812/LANTANA RD	N/A	13088
49	5	180188	36210000	SUMTER COUNTY LINE	Bridge No-360001	13028
50	5	180188	18130000	Bridge No-180070	MARION COUNTY LINE	13028
51	5	750196	75280000	Bridge No-750014	SR 408	12921
52	5	750196	75280000	N/A	Bridge No-750014	12921
53	4	932196	93220000	N/A	GATEWAY BLVD/22 AVE	12800
54	5	360440	36210000	Bridge No-360063	Bridge No-360018	12773
55	7	140156	14140000	NB 14075000	RAMP 018	12765
56	4	932201	93220000	N/A	SR 80/SOUTHERN BLVD	12704
57	4	862505	86070000	Bridge No-860120	Bridge No-860121	12702
58	7	105353	10190000	N/A	N/A	12695
59	1	161005	16320000	ON RAMP TO I-4	SR 546	12483
60	4	932200	93220000	10TH AVE N	N/A	12448
61	2	260488	26260000	Bridge No-260054	Bridge No-260057	12420
62	1	160111	16320000	SR 25/US 27	OSCEOLA CO LINE	12320
63	1	160111	92130000	POLK COUNTY LINE	Bridge No-920094	12320
64	4	932193	93220000	Bridge No-930499	Bridge No-930503	12320
65	8	971900	86470000	MIAMI-DADE CO LINE	Bridge No-860407	12204
66	7	102023	10190000	Bridge No-100586	Bridge No-100589	12144
67	2	723896	72001000	Bridge No-720242	Bridge No-720245	12038

68	5	753056	75280000	N/A	N/A	12012
69	5	753056	75280000	Bridge No-750074	N/A	12012
70	2	720895	72001000	Bridge No-720241	Bridge No-720242	11984
71	2	263455	26260000	Bridge No-260057	SR 222/NW 39TH AVE	11970
72	1	160108	16320000	N/A	SR 25/US 27	11854
73	7	100084	10190000	Bridge No-100607	HILLS/POLK CO LINE	11827
74	7	102026	10190000	Bridge No-100658	Bridge No-100672	11818
75	6	870569	87260000	Bridge No-870268	N/A	11799
76	2	725020	72270000	RAMP 380	(72270436)	11736
77	2	720861	72280000	N/A	Bridge No-720219	11685
78	2	723863	72280000	Bridge No-720328	N/A	11685
79	5	920315	92130000	N/A	N/A	11655
80	5	180186	18130000	N/A	Bridge No-180070	11623
81	5	790494	79002000	Bridge No-790071	LPGA BLVD	11622
82	1	160117	16320000	SR 546	SR 539	11600
83	1	160112	16320000	N/A	N/A	11600
84	7	102028	10190000	RAMP 10320182	Bridge No-100658	11515
85	2	722163	72020000	Bridge No-720177	Bridge No-720178	11495
86	4	932172	93220000	Bridge No-930530	N/A	11488
87	2	720900	72001000	Bridge No-720263	Bridge No-720259	11449
88	2	722156	72020000	Bridge No-720301	Bridge No-720174	11448
89	6	870574	87260000	Bridge No-870757	Bridge No-870766	11400
90	2	720894	72001000	N/A	Bridge No-720241	11396

91	5	770266	77160000	Bridge No-770084	VOLUSIA COUNTY LINE	11388
92	1	160116	16320000	SR 539	Bridge No-160310	11340
93	7	100091	10190000	US 301 / SR 43	N/A	11337
94	1	160115	16320000	Bridge No-160310	N/A	11319
95	4	932222	93220000	GATEWAY BLVD/22 AVE	HYPOLUXO RD	11296
96	7	102020	10190000	Bridge No-100115	Bridge No-100117	11280
97	5	790484	79110000	SEMINOLE COUNTY LINE	N/A	11275
98	4	860163	86070000	Bridge No-860121	SW 10 ST/SR 869	11222
99	5	790534	79002000	LPGA BLVD	N/A	11172
100	2	260456	26260000	Bridge No-260063	Bridge No-260054	11160
101	6	870567	87260000	Bridge No-870760	Bridge No-870112	11144
102	4	930198	93220000	Bridge No-930503	N/A	11104
103	2	722159	72020000	Bridge No-720174	Bridge No-720177	11068
104	6	870575	87260000	Bridge No-870766	N/A	11058
105	6	879060	87260000	Bridge No-870468	N/A	11050
106	1	130040	13075000	Bridge No-130067	Bridge No-130084	11040
107	4	930174	93220000	N/A	Bridge No-930530	10969
108	7	100088	10190000	Bridge No-100614	Bridge No-100599	10824
109	2	720887	72290000	N/A	Bridge No-720234	10824
110	4	932203	93220000	Bridge No-930487	N/A	10816
111	2	290257	26260000	N/A	COLUMBIA CO LINE	10800
112	2	290257	29180000	ALACHUA CO LINE	Bridge No-290053	10800
113	1	160114	16320000	N/A	Bridge No-160181	10780

114	5	770286	77160000	N/A	Bridge No-770084	10773
115	1	130041	13075000	Bridge No-130084	Bridge No-130103	10770
116	8	972015	75470000	Bridge No-750626	Bridge No-750610	10721
117	7	100144	10075000	Bridge No-100363	GIBSONTON DR	10679
118	8	970625	75471000	Bridge No-750099	GORE WITH 75002000	10639
119	2	269904	26260000	Bridge No-260061	Bridge No-260063	10637
120	7	102024	10190000	Bridge No-100672	Bridge No-100586	10600
121	5	750586	75008000	N/A	Bridge No-750123	10575
122	2	290256	29180000	Bridge No-290053	Bridge No-290059	10560
123	2	780259	78080000	Bridge No-780116	DUVAL CO LINE	10560
124	1	170047	17075000	Bridge No-170083	MANATEE CO LINE	10512
125	5	753007	75280000	N/A	SR 91	10488
126	2	290320	29180000	Bridge No-290061	N/A	10409
127	2	729905	72280000	ST JOHNS CO LINE	Bridge No-720636	10408
128	5	700366	70225000	N END OF BR 700127	Bridge No-700054	10350
129	7	100151	10075000	Bridge No-100393	Bridge No-100403	10328
130	6	870579	87260000	Bridge No-870239	Bridge No-870104	10316
131	6	870566	87260000	Bridge No-870126	Bridge No-870760	10289
132	1	160113	16320000	Bridge No-160181	N/A	10268
133	2	725514	72020000	Bridge No-720178	Bridge No-720182	10165
134	2	720864	72280000	Bridge No-720636	Bridge No-720328	10165
135	4	862504	86070000	Bridge No-860231	Bridge No-860120	10164
136	8	972004	75470000	Bridge No-750404	RAMP 110 SB OFF	10117

137	8	972025	75470000	N/A	SR 429 SB	10117
138	8	972025	75470000	RAMP 161 SB ON	N/A	10117
139	8	972014	75470000	OSCEOLA COUNTY LINE	Bridge No-750626	10117
140	4	862503	86070000	Bridge No-860239	Bridge No-860231	10105
141	7	100089	10190000	N/A	Bridge No-100614	10000
142	5	753080	77160000	ORANGE COUNTY LINE	SR 436	9986
143	5	753080	75280000	N/A	SEMINOLE COUNTY LINE	9986
144	8	970505	75470000	Bridge No-750610	N/A	9966
145	1	130039	13075000	BEGIN I-275 NB	Bridge No-130067	9947
146	8	970610	75471000	Bridge No-750089	Bridge No-750091	9945
147	5	920316	92130000	N/A	N/A	9943
148	1	160363	16320000	N/A	N/A	9927
149	2	260454	26260000	SR 222/NW 39TH AVE	N/A	9900
150	5	753044	75280000	Bridge No-750064	Bridge No-750066	9900
151	5	753044	75280000	Bridge No-750066	N/A	9900
152	4	932177	93220000	N/A	N/A	9824
153	1	130042	13075000	Bridge No-130103	RAMP#13175310	9779
154	2	740132	74160000	Bridge No-740940	GEORGIA STATE LINE	9749
155	4	932195	93220000	N/A	N/A	9728
156	6	870568	87260000	Bridge No-870112	Bridge No-870268	9719
157	7	100147	10075000	Bridge No-100485	SR 618 X-TOWN REVERS	9680
158	8	972266	87471000	Bridge No-870198	Bridge No-870407	9676
159	8	970533	75471000	END BRIDGE 750180	Bridge No-750088	9647

160	5	700401	70225000	N/A	N/A	9630
161	5	700401	70225000	ST JOHN RD	N/A	9630
162	5	730292	73001000	N/A	N/A	9630
163	7	100153	10075000	Bridge No-100420	Bridge No-100367	9602
164	5	360437	36210000	Bridge No-360043	Bridge No-360037	9579
165	2	720832	72270000	N/A	RAMP 380	9454
166	4	862808	86095000	Bridge No-860378	SR 91/TURNPIKE NB	9453
167	8	970620	75471000	Bridge No-750088	Bridge No-750089	9436
168	5	700368	70225000	Bridge No-700054	N/A	9360
169	5	750584	75008000	N/A	N/A	9353
170	1	120059	12075000	Bridge No-120122	Bridge No-120090	9313
171	5	750530	75008000	N/A	N/A	9306
172	8	971904	86470000	Bridge No-860432	Bridge No-860533	9296
173	7	100146	10075000	GIBSONTON DR	Bridge No-100485	9282
174	4	932187	93220000	N/A	N/A	9280
175	7	109926	10075000	Bridge No-100495	N/A	9253
176	2	290324	29180000	N/A	SUWANNEE CO LINE	9240
177	2	290324	37130000	COLUMBIA CO LINE	N/A	9240
178	5	753064	75280000	N/A	Bridge No-750256	9240
179	2	720902	72001000	Bridge No-720256	Bridge No-720412	9238
180	2	729923	72290000	Bridge No-720237	NASSAU CO LINE	9194
181	2	729923	74160000	DUVAL CO LINE	Bridge No-740034	9194
182	5	790495	79002000	N/A	Bridge No-790082	9188

183	5	790495	79002000	N/A	N/A	9188
184	2	722121	72020000	N/A	Bridge No-720301	9179
185	5	753034	75280000	SR 408	Bridge No-750064	9174
186	7	140190	14140000	RAMP 018	RAMP 001	9152
187	7	100150	10075000	N/A	Bridge No-100393	9146
188	7	080037	08150000	Bridge No-080021	SUMTER CO LINE	9135
189	7	100154	14075000	HILLSBOROUGH COUNTY	SB I-275	9120
190	7	100154	10075000	Bridge No-100367	PASCO CO LINE	9120
191	2	290255	29180000	Bridge No-290059	Bridge No-290061	9120
192	1	130043	13075000	RAMP#13175302	N/A	9120
193	5	360436	26260000	MARION CO LINE	Bridge No-260061	9021
194	5	360436	36210000	Bridge No-360037	ALACHUA COUNTY LINE	9021
195	4	862506	86070000	SW 10 ST/SR 869	Bridge No-860124	9020
196	7	155405	15190000	N/A	4TH ST N	8936
197	5	790496	79002000	Bridge No-790082	FLAGLER COUNTY LINE	8894
198	5	790496	73001000	VOLUSIA COUNTY LINE	N/A	8894
199	2	720898	72001000	Bridge No-720248	Bridge No-720253	8881
200	5	750130	75280000	N/A	N/A	8876
201	7	150106	15190000	54TH AVE N	Bridge No-150100	8873
202	2	780055	78080000	INTL GOLF PKWY	Bridge No-780116	8820
203	4	941901	94001000	MARTIN CO LINE	ST LUCIE W BLVD	8816
204	2	720171	72280000	Bridge No-720332	Bridge No-720334	8782
205	5	753051	75280000	N/A	Bridge No-750074	8778

206	4	932202	93220000	SR 80/SOUTHERN BLVD	Bridge No-930487	8768
207	7	105601	10075000	MANATEE CO LINE	Bridge No-100346	8758
208	8	972003	75470000	SR 429 SB	Bridge No-750404	8758
209	5	791003	79110000	N/A	SR 472	8745
210	8	972250	87471000	N/A	N/A	8732
211	5	750648	75280000	N/A	N/A	8712
212	7	100148	10075000	SR 618 X-TOWN REVERS	Bridge No-100495	8694
213	6	872114	87260000	ON RAMP 87260514	OFF RAMP 518	8693
214	5	753027	75280000	Bridge No-750160	N/A	8679
215	5	753027	75280000	N/A	Bridge No-750160	8679
216	2	720109	72270000	Bridge No-720199	N/A	8678
217	4	862001	86075000	N/A	PINES BLVD/ SR 820	8672
218	8	970534	75471000	Bridge No-750093	Bridge No-750099	8494
219	5	770343	77160000	N/A	LAKE MARY BLVD	8484
220	5	770343	77160000	Bridge No-770022	N/A	8484
221	8	971912	86470000	N/A	COMMERCIAL BLVD	8466
222	8	971908	86470000	Bridge No-860533	N/A	8466
223	4	862806	86095000	SR 91/TURNPIKE NB	Bridge No-860535	8464
224	3	552003	55320000	Bridge No-550074	US27/SR63/MONROE ST	8439
225	6	870405	87260000	Bridge No-870253	Bridge No-870051	8408
226	2	740158	74160000	Bridge No-740034	Bridge No-740940	8408
227	6	875252	87090000	NW 74 ST	W 9 ST/NW 62 ST	8390
228	1	170225	17075000	Bridge No-170085	Bridge No-170145	8343

229	7	100143	10075000	Bridge No-100346	Bridge No-100363	8320
230	2	725515	72020000	Bridge No-720306	N/A	8313
231	5	750535	75280000	Bridge No-750367	N/A	8303
232	8	970598	75471000	Bridge No-750091	Bridge No-750093	8296
233	7	100152	10075000	Bridge No-100403	Bridge No-100420	8295
234	1	170043	17075000	LAUREL ROAD	RP 17005102 VEN CONN	8289
235	7	140086	14140000	RAMP 001	RAMP 007	8262
236	4	862804	86095000	Bridge No-860357	Bridge No-860391	8249
237	5	189920	18130000	Bridge No-180037	N/A	8243
238	2	725511	72020000	SR 5 SB 72070-000	TO PARK ST	8228
239	7	100104	10190000	Bridge No-100589	US 301 / SR 43	8190
240	5	753074	75280000	N/A	N/A	8184
241	5	753074	75280000	N/A	N/A	8184
242	8	972254	87471000	Bridge No-870194	Bridge No-870198	8142
243	2	273134	27090000	Bridge No-270047	NASSAU CO LINE	8120
244	2	273134	72270000	NASSAU COUNTY LINE	Bridge No-720199	8120
245	2	273134	74170000	BAKER CO LINE	DUVAL CO LINE	8120
246	1	175075	17075000	Bridge No-170096	LAUREL ROAD	8118
247	6	870576	87260000	N/A	Bridge No-870468	8094
248	6	870578	87260000	Bridge No-870234	Bridge No-870239	8094
249	3	482006	48260000	N/A	N/A	8062
250	1	120058	12075000	Bridge No-120120	Bridge No-120122	8037
251	4	930217	93220000	N/A	Bridge No-930371	8036

252	7	102009	10320000	Bridge No-100062	Bridge No-100203	8007
253	1	120057	12075000	Bridge No-120107	Bridge No-120120	7998
254	5	180358	18130000	Bridge No-180033	Bridge No-180037	7986
255	5	753069	75280000	Bridge No-750256	N/A	7953
256	6	870577	87260000	Bridge No-870051	Bridge No-870234	7952
257	2	729914	72001000	Bridge No-720412	Bridge No-720396	7946
258	5	927067	92530000	REAVES RD	US 17 / US 92 / OBT	7942
259	7	140094	14140000	RAMP 009	HERNANDO CO LINE	7935
260	7	140094	08150000	PASCO CO LINE	Bridge No-080021	7935
261	2	780258	78080000	Bridge No-780057	INTL GOLF PKWY	7920
262	4	892210	89095000	Bridge No-890117	Bridge No-890129	7914
263	1	120060	12075000	Bridge No-120090	Bridge No-120093	7888
264	5	790486	79110000	SR 44	N/A	7884
265	4	862005	86075000	N/A	N/A	7868
266	6	870554	87260000	ON RAMP 87260337	Bridge No-870253	7866
267	8	972270	87471000	N/A	N/A	7847
268	1	170042	17075000	Bridge No-170090	Bridge No-170096	7839
269	4	860186	86095000	Bridge No-860391	Bridge No-860378	7826
270	6	870581	87260000	Bridge No-870104	ON RAMP 87260514	7809
271	4	862003	86075000	N/A	GRIFFIN RD	7803
272	7	102008	10320000	Bridge No-100203	Bridge No-100210	7775
273	4	892212	89095000	Bridge No-890129	Bridge No-890121	7770
274	7	140093	14140000	RAMP 007	RAMP 009	7763

275	5	790485	79110000	SR 472	N/A	7755
276	8	971902	86470000	Bridge No-860407	Bridge No-860432	7728
277	1	130063	13075000	RAMP#13175310	RAMP#13175302	7719
278	2	720886	72290000	72290239 TO DUNN AVE	N/A	7688
279	6	870137	87260000	N/A	ON RAMP 87260337	7682
280	2	722170	72020000	Bridge No-720182	Bridge No-720306	7656
281	8	971916	86470000	COMMERCIAL BLVD	ATLANTIC BLVD/SR 814	7636
282	7	102006	10320000	Bridge No-100214	Bridge No-100219	7632
283	5	753018	75280000	N/A	N/A	7623
284	5	180208	18130000	HERNANDO COUNTY LINE	N/A	7585
285	4	860268	86010000	MIAMI DADE CO LN	SR 858/HALLANDALE BL	7566
286	7	150062	15190000	4TH ST N	END BRIDGE 150107	7560
287	2	720888	72290000	Bridge No-720234	Bridge No-720237	7503
288	2	320112	32100000	N/A	GEORGIA STATE LINE	7430
289	2	722355	72280000	Bridge No-720331	Bridge No-720332	7412
290	2	722241	72020000	N/A	SB ON FR CLARK (408)	7315
291	5	180194	18130000	N/A	Bridge No-180033	7289
292	6	872537	87090000	NW 95 ST/JOHN HILL R	ON RAMP 87260403	7260
293	5	753061	75280000	N/A	N/A	7224
294	7	106011	10030000	SR 583 / N 56TH ST	N/A	7210
295	5	799906	79110000	N/A	N/A	7209
296	4	862803	86095000	Bridge No-860359	Bridge No-860357	7176
297	2	320236	32100000	SUWANNEE CO LINE	US 129/SR 51	7168

298	2	320236	37130000	N/A	HAMILTON CO LINE	7168
299	8	971930	93470000	NORTH END BR #860184	Bridge No-930416	7138
300	8	971930	86470000	Bridge No-860506	PALM BEACH CO LINE	7138

APPENDIX C
SAMPLE INPUT PARAMETERS FOR ANALYSIS OF TYPE I-A DESIGN USING MEPDG

Structure--Design Features

General Design Features

Pavement Cross-Slope	4%
Crack Infiltration Rate (ft^3/day/ft):	2.06

Joint Design

Joint spacing (ft):	15
Dowel diameter (in):	1.5
Dowel bar spacing (in):	12

Edge Support	Tied PCC shoulder, Widened slab
Long-term LTE(%):	50
Widened Slab (ft):	13

Structure--Layers

Layer 1 -- JPCP

General Properties

PCC material	JPCP
Layer thickness (in):	13
Unit weight (pcf):	145
Poisson's ratio	0.2

Thermal Properties

Coefficient of thermal expansion (per F° x 10- 6):	5.68
Thermal conductivity (BTU/hr-ft-F°) :	1.25
Heat capacity (BTU/lb-F°):	0.28

Mix Properties

Cement type:	Type II
Cementitious material content (lb/yd^3):	470
Water/cement ratio:	0.4
Aggregate type:	Brooksville Limestone
Reversible shrinkage (% of ultimate shrinkage):	50
Time to develop 50% of ultimate shrinkage (days):	35

Strength Properties

28-day PCC modulus of rupture (psi):	700

Layer 2 -- Permeable aggregate

Unbound Material:	Permeable aggregate
Thickness(in):	4

Strength Properties

Poisson's ratio:	0.35
Coefficient of lateral pressure,Ko:	0.5
Modulus (input) (psi):	40000

Drainage Properties

Permeability (ft/day):	900
Time-to-drain 50% saturated layer (hrs):	3.76

General Parameters

Maximum dry unit weight (pcf):	127.2
Specific Gravity	2.7

Layer 3 -- Asphalt Concrete

Material Type:	Asphalt Concrete
Thickness(in):	2

General Properties

PG Grade(C°):	76-22
Effective binder content (%):	11.6
Air Voids (%):	7
Unit weight (pcf):	150
Poisson's ratio:	0.35

Layer 4 -- Type B (LBR 40)

Unbound Material:	Type B
Thickness(in):	12

Strength Properties

Poisson's ratio:	0.35
Coefficient of lateral pressure,Ko:	0.5
CBR:	32
Modulus (input) (psi):	23479

General Parameters

Maximum dry unit weight (pcf):	120
Specific Gravity	2.7

Layer 5 -- Subbase

Unbound Material: A-3

Thickness(in): Semi-infinite

Strength Properties

Poisson's ratio: 0.35

Coefficient of lateral pressure,Ko: 0.5

Modulus (input) (psi): 16000

General Parameters

Maximum dry unit weight (pcf): 120

Specific Gravity 2.7