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Evaluation of Composite Pavements Using a Heavy Vehicle Simulator

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Abstract: A full-scale experiment was performed at the APT facility located at the Florida Department of Transportation (FDOT) Material Research Park to evaluate the feasibility of using whitetopping (WT) pavements in Florida. A total of nine full-scale and instrumented WT test sections were constructed and tested using a Heavy Vehicle Simulator (HVS). A 3-D finite element model was developed to analyze the behavior of the WT pavement test sections. Six of the WT pavement test sections were constructed to have a bonded concrete-asphalt interface, while three of the test sections were intended to have an unbonded concrete-asphalt interface. Using the 3-D model, maximum tensile stresses in the pavement were computed for the critical condition when a 106.8-kN (24-kip) single axle load was placed at the mid-edge of the slab and when the temperature differential in the concrete slab was +11.1℃ (+20°F). Based on the computed maximum stresses in the concrete, the expected numbers of repetitions of the 106.8-kN (24-kip) single axle loads at the critical thermal condition were computed for the nine test sections. Results show that the 10cm (4inches) slabs can be used for heavy 106.8-kN (24-kip) single axle loads, but only for low-volume traffic conditions. In order to withstand the critical load without fear of fatigue failure (for an infinite number of critical load repetitions), a minimum slab thickness of 6 inches would be needed for a joint spacing of 1.2m (4ft), and a minimum slab thickness of 20cm (8inches) would be needed for a joint spacing of 1.8m (6ft).

Key words*: 3-D Finite Element Mode; Bonded Slab; Composite Pavement; Concrete-asphalt Interface; Critical Loading Condition; Partially Bonded Slab; Shear Strength; Temperature Differentia; Whitetopping.*

Introduction

Background

Increasing truck weights and tire pressures on pavements in recent years have pushed the demand on the performance of our pavements to a higher level. Many asphalt pavements have experienced rutting while many others have experienced longitudinal cracking. One of the possible solutions to this problem is the use of whitetopping (WT), which is placing a concrete layer over an existing asphalt pavement. WT has an advantage over an asphalt overlay in that the concrete surface is stronger and thus is more resistant to rutting and surface-initiated cracking. The better durability and long-term performance characteristics of concrete pavement surfaces can significantly reduce traffic delays associated with the frequent maintenance of asphalt pavements. In addition, when concrete surfaces are used, skid resistance and safety can be substantially improved, especially under wet conditions. In recent years, with the sky-rocketing price of asphalt, concrete is becoming more competitive in cost with that of asphalt. This makes the use of WT a more economically viable alternative for rehabilitation of asphalt pavements.

There are three type of WT based on the thickness of the concrete slab. Ultra-Thin Whitetopping (UTW) is a relatively new technique for resurfacing deteriorated asphalt pavements. It involves placing very thin concrete slabs, 5 to 10cm (2 to 4inches) thick, on an old asphalt pavement to create a bonded (or partially bonded) composite pavement. Reduction of thickness is justified by the use of a high quality concrete, shorter joint spacing, and good bond between the concrete and the existing asphalt pavement.

Thin Whitetopping (TWT) involves placing relatively thicker concrete slabs, normally 12.5 to 20cm (5 to 8inches) thick, bonded (or partially bonded) over an existing asphalt pavement. Similar to UTW pavements, TWT pavements use short joint spacing and good bond between the concrete and the asphalt layer.

Conventional whitetopping (CWT) involves placing concrete slabs which are typically greater than 20cm (8inches) in thickness. These concrete slabs are typically not bonded to the underlying asphalt layer.

Experimental UTW pavements have been constructed in many states [1, 2], including Colorado, Georgia, Iowa, Kansas, Kentucky, Missouri, New Jersey, North Carolina, Pennsylvania, and Tennessee. Preliminary evaluations of these recently constructed UTW projects have shown that UTW is a viable rehabilitation method for asphalt pavements. The Florida Department of Transportation (FDOT) has also experimented with UTW in recent years. Three UTW test tracks were constructed behind the FDOT State Materials Office in Gainesville in 1996. An experimental UTW project was also constructed at the Ellaville Truck Weigh Station on I-10 in

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northwest Florida in 1997. However, the performance of these test sections was less than ideal, with the observation of some early cracking on the concrete surface. These problems were attributed mainly to the fact that all of the UTW test sections were inadequately designed for the traffic at the Ellaville Weigh Station [3, 4].While the UTW technique may provide a durable wearing surface for normal traffic loads on residential and city streets, low-volume roads, street intersections, general aviation airports, and parking areas, the UTW technique was probably not an appropriate rehabilitation alternative for weigh stations subjected to frequent applications of heavy truck traffic. The use of TWT or CWT might have been a more appropriate choice in such an application.

Given the potential economical and technical benefits of WT pavements, there was a need to effectively evaluate the feasibility and proper application of UTW, TWT, and CWT pavements in Florida, so that the WT techniques could be properly and effectively utilized to achieve the maximum benefits to the traveling public.

Objectives of Research

Main objectives of this research are as follows:

1. To develop analytical models for analysis of the behavior of UTW, TWT, and CWT pavements. These models were to be verified and fine-tuned by full-scale experimental results.
2. To evaluate the potential performance of the WT pavement test sections for use under Florida conditions.
3. To assess the applicability of UTW, TWT, and CWT techniques for rehabilitation of asphalt pavements in Florida.

Approach and Significance of Research

The FDOT Materials Office has a Heavy Vehicle Simulator (HVS) and an Accelerated Pavement Testing (APT) facility for operation of this HVS. The HVS can apply realistic, full-size wheel loads to full-size pavements to assess their behavior and performance directly. The HVS has the capability to simulate 20 years of interstate traffic on a pavement test section within a period of 1 to 4 months. This APT facility was used in this study to evaluate the behavior and performance of WT pavements in Florida in a direct and effective manner.

In the past, there have been many studies in which WT pavements were constructed and their performance observed. There have also been some studies in which WT pavements were modeled and analyzed with respect to the various factors which may affect their performance [5, 6]. However, little work has been done where the WT pavements were instrumented and the measured responses were compared to the analytical results to validate the models used.

The significance of this research work is that the analytical model developed was validated and fine-tuned by measured responses from full-scale and instrumented WT pavements, which had not been done before. The 3-D finite element model used also had further refinements from previous work in this area. In the past research work, 2-D models using 4-node elements and 3-D models using 8-node elements had been used to analyze this type of pavements[7, 8]. In this research, a 3-D finite element model with 20-node 3D solid elements was used to model the pavement structure. Previous models had used a 4-slab system to model the

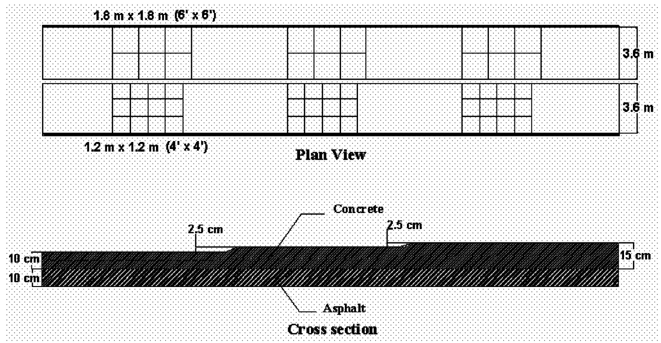


Fig. 1. Layout of the Test Sections for Phase I.

WT pavements. In this research, a 12-slab system was used. Using a 12-slab system can give better modeling of the effects of the adjacent slabs. The bond interface between the concrete slab and the AC layer was also modeled with special elements to cover the range from a fully bonded to fully un-bonded condition. How well this can model the actual behavior of the WT pavements was validated by measured responses from the full-scale test sections.

Construction and Instrumentation of Test Sections

Description of the Test Sections

The HVS testing of the test sections in this study was divided into two main phases, namely, Phase I using bonded composite pavements, and Phase II using un-bonded composite pavements. Phase I was divided in two sub phases, I-a and I-b, by using two different joint spacings.

Test Sections in Phase I

The test track in Phase I-a (on Lane 6) consisted of three test sections of 10, 12.5, and 15cm (4, 5, and 6inches) of concrete placed bonded to the existing AC layer, with 1.8 x 1.8m (6 x 6ft) joint spacing. The test track in Phase I-b (on Lane 7) consisted of three test sections with the same thicknesses as those used on Lane 6, but with 1.2 x 1.2m (4 x 4ft) joint spacing. The concrete overlay in Lane 7 was also bonded to the existing AC layer. While the 10cm (4inches) concrete slabs are considered UTW, the 12.5 and 15cm (5 and 6inches) slabs fall in the category of TWT. Fig. 1 shows the layout of the test sections in Lanes 6 and 7. The test sections are confined by two ends and transition concrete slabs constructed to support the HVS.

Test Sections in Phase II

After the removal of the test sections for Phase I-a, Lane 6 was overlaid with 15, 20, and 25cm (6, 8, and 10inches) of concrete placed un-bonded to the existing AC layer, with 1.8 x 1.8m (6 x 6ft) joint spacing. Because it was not possible to remove the concrete slab without damaging the AC layer (a very strong bond was observed), the existing asphalt layer was also removed and replaced with a new one with the same thickness and properties. To ensure an un-bonded condition in the interface in these test sections, a white pigmented curing compound was sprayed on the asphalt surface

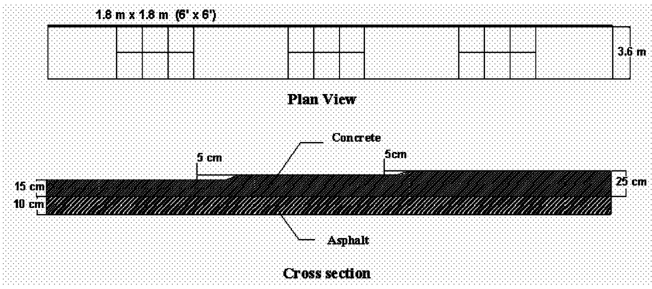


Fig. 2. Layout of the Test Sections for Phase II.

prior to the concrete placement. Fig. 2 shows the test section layout for Phase II.

Instrumentation of the Test Sections

Fig. 3 shows the instrumentation layout for the 1.8 x 1.8m (6 x 6ft) test sections (Phase I-a), which were placed on Lane 6 of the APT test area. Three locations (Location 1, 2, and 3) were identified to have the maximum anticipated strains due to the HVS load from results of stress analyses using a finite element model, which is described in section of "Finite Element Model for Analysis of WT Pavements". Thus, strain gages were placed at these three locations. While Location 1 had two gages, the other two locations had only one.

At Location 1, one embedded strain gage was placed at a depth of 2.54cm (1inch) from the concrete surface, while the other embedded strain gage was placed 1.27cm (0.5inch) from the bottom of the concrete layer. Location 2 had a strain gage embedded 2.54cm (1inch) from the surface of the concrete slab. Location 3 had a strain gage embedded 1.27cm (0.5inch) from the bottom of the concrete slab. Two surface gages were also used to monitor any micro cracks that may occur in the concrete surface. These two gages were located next to the transversal joint at the middle of the slab, in the adjacent panels. These surface gages, located in the adjacent slabs were also used to evaluate the load transfer at the joints.

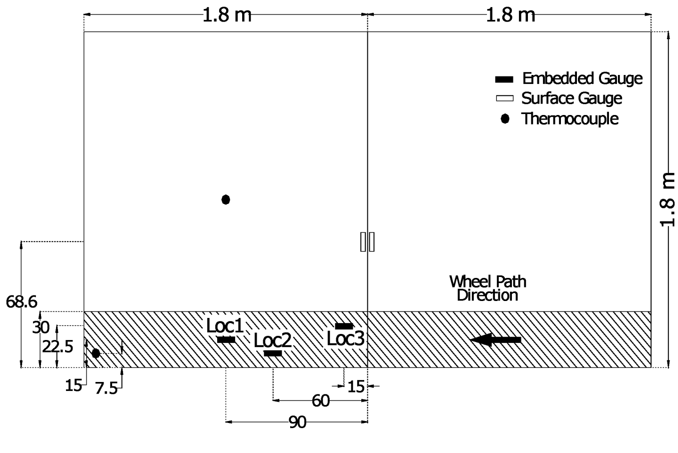
Fig. 3 also shows the locations of thermocouples to monitor the temperature in the slab. Two positions were considered for the thermocouples, one at the center of the slab and the other in the corner. The thermocouples were placed 2.54cm (1inch) apart along the depth of the slab with the first starting at 2.54cm (1inch) from the surface. An additional thermocouple was placed on the surface of the AC layer to monitor daily variation of temperature in the asphalt layer.

Fig. 4 shows the instrumentation layout for the 1.2 x 1.2m (4 x 4ft) slabs in Phase I-b. In this case only two locations (namely Loc1 and Loc2) were identified as having maximum stresses and for placement of embedded gages. Four strain gages were placed on the surface of the concrete slab to monitor any micro cracks occurring in the slabs and load transfer at the joints. At Location 1, three gages were used; one at 2.54cm (1inch) from the surface of the concrete slab, the second at 1.27cm (0.5inch) from the bottom of the concrete slab, and the third one at 1.27cm (0.5inch) below the surface of the AC layer. The placement of these three gages allowed not only for the monitoring of the maximum strain at the top and bottom of the concrete slab, but also for the comparison of the strain values near the interface of the layers. By comparing the strain values near the interface, one could ascertain the bonding condition in the composite pavement.

Location 2 had two gages; one at 1.27cm (0.5inch) above the bottom of the concrete slab, and the other at 1.27cm (0.5inch) below the surface of the AC layer. Fig. 4 also shows the two locations for the thermocouples in Phase I-b. The vertical positions of the thermocouples in Phase I-b were the same as those used in Phase I-a.

Fig. 5 shows the instrumentation plan adopted for Phase II. Taking advantage of the upgrade to the data collection equipment that allowed for more channels for data acquisition, five locations were selected for placement of strain gages. For each of these locations, a set of three strain gages were installed to monitor maximum strains in the concrete slab and the strain at the surface of the AC layer. One strain gage was placed at 2.54cm (1inch) under the surface of the concrete slab. A second strain gage was placed 2.54cm (1inch) from the bottom of the concrete slab. The third strain gage was located 1.27cm (0.5 inch) below the top of the asphalt layer. Unlike the previous phase, surface gages were not used in Phase II.

In Phase II, three locations were used to monitor the temperature in the slabs. For each of the three thermocouple locations, a set of



(Note: Dimensions are in cm unless indicated)

Fig. 3. Instrumentation Layout for the Test Slabs in Phase I-a.

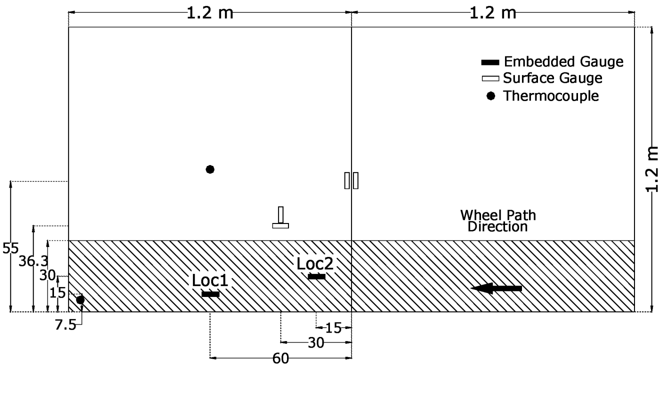


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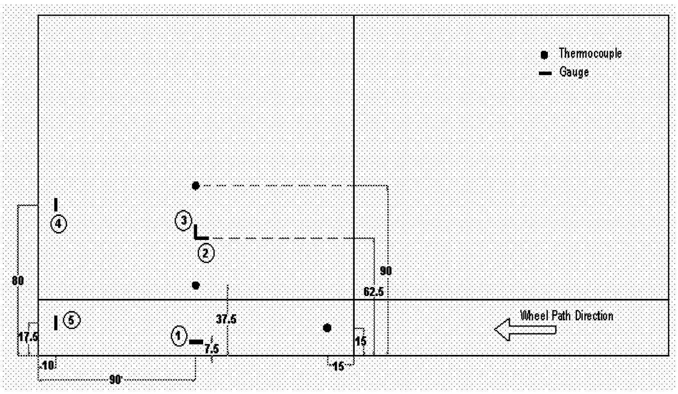
Fig. 4. Instrumentation Layout for the Test Slabs in Phase I-b.



(Note: Dimensions are in cm)
Fig. 5. Instrumentation Layout for Phase II.

Fig. 6. After Placement of Concrete on Lane 6 in Phase I-a.

thermocouples were placed at depths of 2.5, 7.5, 12.5, 17.5, and 22.5cm (1, 3, 5, 7, and 9inches) for the 25cm (10inches) slabs, at depths of 2.5, 7.5, 12.5, and 17.5cm (1, 3, 5, and 7inches) for the 20cm (8inches) slabs, and at depths of 2.5, 7.5, and 12.5cm (1, 3, and 5inches) for the 15cm (6inches) slabs. At each of the locations, a thermocouple was also placed in the asphalt layer at a depth of 1.27cm (0.5inch) from the top of the asphalt layer.

Construction of Test Sections

Construction of Test Sections in Phase I

The concrete test track for Phase I-a was constructed on Lane 6 on June 10, 2004, and that for Phase I-b was constructed on Lane 7 on August 10, 2004. These two concrete test tracks were constructed over an existing 10cm (4inches) asphalt surface (two 5cm (2inches) lifts of asphalt) at the APT test area at the FDOT State Materials Research Park. The asphalt surface was milled and cleaned prior to the placement of formworks for the test track.

A minimum 24-hour compressive strength of 17.24MPa (2,500psi) and minimum 28-day strength of 40MPa (5,800psi) were specified for the concrete for the test tracks. Before placing the concrete on top of the asphalt layer, water was sprayed on the asphalt surface to promote a good bond between the concrete and the asphalt and to prevent the reduction of water from the concrete. Samples of concrete were taken from a selected truck during concrete placement. The slump, air content and temperature of the fresh concrete were measured. Samples were fabricated for compressive strength and elastic modulus, maturity test, and flexural strength tests. Fig. 6 shows the concrete test track after placement of the concrete. After placement and finishing of the concrete on the test track, saw cuts were made to a one third (1/3) of the thickness to form the joints for the slabs the following day. A diamond-bladed saw was used for these cuts to ensure a smooth, straight vertical surface. The concrete slabs were kept moist by sprinkling with water for at least 3 days to ensure adequate curing.

Construction of Test Sections in Phase II

After testing on the concrete slabs in Phase I-a was finished, the concrete slabs were removed from Lane 6, and the test sections in Phase II were placed on Lane 6. When attempts were made to remove the concrete slabs without removing the underlying asphalt layer, it was found that the concrete slabs were bonded so well to the asphalt layer that a lot of the underlying asphalt concrete was also removed at the same time. Due to this situation, a new asphalt layer was placed on Lane 6 before the concrete slabs in Phase II were placed. Before the concrete was placed, a white pigmented curing compound was applied to the asphalt surface to act as a debonding agent between the asphalt and the concrete slab. Similarly, the joints for the concrete slabs were sawed the following day.

Finite Element Model for Analysis of WT Pavements

Finite Element Program

The multi-purpose finite element program, ADINA version 8.2, was used to build the model for the analysis of WT pavements in this study [9]. The capability of the ADINA program for 3-D finite element analysis, its versatility in modeling materials behaviors under load and temperature effects, and its capability in modeling the interface condition between two layers make this program very appropriate to model composite pavements.

Six-Slab and Twelve-Slab 3-D Finite Element Models

Fig. 7 shows a 6-slab 3-D FE model developed for the analysis of the composite pavement test sections with a joint spacing of 1.8m (6ft). The 6-slab model was used to analyze the test sections in Phase I-a and Phase II. The number of slabs used in this model corresponds to the actual number of slabs in each test section in these two phases of the study. A 12-slab 3-D FE model was developed for analysis of the composite pavement test sections with a joint spacing of 1.2m (4ft), which were used in Phase I-b. The number of slabs used in this model also corresponds to the actual number of slabs in each test section in Phase I-b of the study.

Solid 20-Node Finite Element

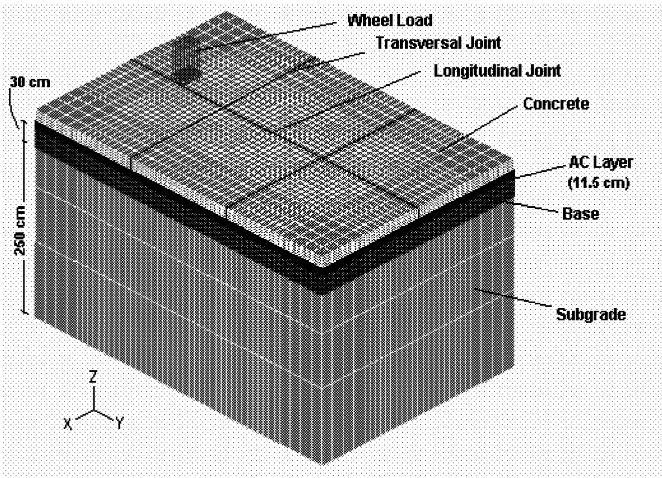


Fig. 7. Six-Slab 3-D Finite Element Model.

All 3-D solid elements were modeled as 20-node elements. The 20-node element has a hexahedral shape, with one node at each of its 8 vertices and one node at the middle of each of its 12 edges. Each node has six degrees of freedom (three translations and three rotations). This type of node configuration has been shown to give a high level of accuracy in combination with an acceptable computing time demand.

Modeling of Concrete Slab Joints

Load transfer across the joints between two adjoining concrete slabs is modeled by translational springs connecting the slabs at the nodes of the finite elements along the joint. Three values of spring constants are used to represent the stiffnesses along three different directions.

Modeling of Materials

The concrete, AC, base and subgrade materials are modeled as isotropic and linearly elastic, and are characterized by their elastic modulus and Poisson's ratio. Since dynamic pavement responses due to moving wheel loads were measured and analyzed in this study, it is adequate to model the pavement materials as elastic rather than as viscoelastic materials, since viscous responses would be minimal. The AC material is also modeled as a temperature dependent material with an elastic modulus which varies with temperature. The contraction and expansion of the concrete due to temperature effects is also considered in the analysis and characterized by the coefficient of thermal expansion of the concrete.

Modeling of Concrete-Asphalt Interface

The concrete-asphalt interface in the composite pavement can be modeled as fully bonded, partially bonded or un-bonded. By default, the ADINA program treats adjacent nodes as rigidly connected to one another, so that the fully bonded condition would be implicitly considered in the model with no special treatment of the interface between the concrete slab and the AC layer. The composite pavement test sections in Phase I-a and Phase I-b, which were constructed to have bonded interface between the concrete and the AC, were modeled to have a fully bonded interface in this fashion.

Cores taken from the test sections in Phase II indicated that, though the concrete-asphalt interface was intended to be unbonded, there was partial bonding between the concrete and asphalt layers in these test sections. Analysis of the results of Falling Weight Deflectometer (FWD) tests on these test sections also indicated that the computed FWD deflections matched better with the measured deflections when the concrete-asphalt interface was modeled as partially unbonded rather than fully unbonded. Similarly, the strains calculated using the fully unbonded model resulted in strain values 25% higher than the measured strains. Thus, the model for the fully unbonded condition was not used in the analysis of the data in this study.

To model the partial bond condition in the interface, translational spring elements were used to connect the bottom of the concrete layer with the top of the AC layer at the nodes, with zero distance between the two layers. Each spring was modeled with three spring constants to represent stiffness along the three different directions.

Modeling of Loads and Temperature Effects

To model a moving HVS wheel load on a test section in this study, computations were done for the responses of the pavement subjected to static loads placed at different consecutive locations along the wheel path. The calculated response, such as strain at a particular location, could then be plotted versus time, as the wheel passed over the various locations at the various times. This computed strain versus time plot could then be compared to the measured strain versus time plot as obtained from the strain gage measurements during HVS loading.

The applied HVS wheel load was modeled as a uniformly distributed load over a square area. The contact area was taken to be the wheel load divided by the contact tire pressure. For example, for a 53.4kN (12kips) wheel load with a tire pressure of 0.83MPa (120psi), the contact area would be 645cm^2 (100in^2).

The concrete is modeled as a material which contracts or expands according to its temperature change and coefficient of thermal expansion. An initial temperature is first specified for the entire concrete layer. The effects of the change in temperature from the initial temperature were then duly considered in the analysis when the information on the temperature distribution in the concrete layer was provided in the input. The temperature at the bottom of the concrete slab is set to be unchanged from the initial temperature. The temperature in the concrete layer was assumed to vary linearly from the top to the bottom of the slab, for ease of analysis. From the temperature at the top and the bottom of the concrete slabs, the temperature of the concrete at all the nodes of the finite element mesh for the concrete slabs were computed and entered as inputs to the model.

Materials and Pavement Characterization

Interface Bond Strength

Results from Phase I

Table 1. Properties of Hardened Concrete Sampled from Truck in Phase I-a.

Curing Time, days	Compressive Strength, MPa (psi)	Elastic Modulus, GPa (ksi)	Flexural Strength, MPa (psi)
1	11.65 (1,690)	-	-
3	20.27 (2,940)	-	-
7	27.10 (3,930)	23.72 (3,440)	-
14	32.75 (4,750)	25.76 (3,737)	5.05 (732)
28	41.23 (5,980)	27.17 (3,940)	5.32 (772)
56	46.54 (6,750)	30.20 (4,380)	5.84 (847)

Iowa shear tests were performed on both the 10cm (4inches) and the 15cm (6inches) diameter core samples extracted from the 10cm (4inches) slabs in Phase I-a before loading started (at 28 days or later). The average shear strength for two 15cm (6inches) diameter core samples was 1.43MPa (207.5psi), while the shear strength for one 10cm (4inches) diameter sample was 1.14MPa (165psi).

Six 15cm (6inches) diameter cores of the concrete/asphalt composite layer were extracted from each of the test sections at the end of the HVS testing in Phase I-a. The locations of the cores were selected so that the bond strength for different conditions could be evaluated. For each test section, two cores were obtained from the wheel path (loaded area); one from the corner and one from the mid edge of the slab. Four cores were obtained outside the wheel path; one from the center of an unloaded slab, one from the center of the loaded slab, one from the mid edge of the longitudinal joint, and one from the mid edge of the transverse joint.

For the test sections in Phase I-b, a total of 7 cores of the concrete/asphalt composite layer were extracted from each test section. Six of these samples were tested for interface bond condition after loading. The average shear strength was 1.34MPa (195psi). Similar to the case for Phase I-a, no significant difference in the shear strength was observed among the cores taken from the corner, mid edge and center of the slabs both in and out the wheel path. This means the area of the slab interface loaded for a short period of time did not experience more deterioration than that out of the wheel path.

Results from Phase II

Four cores were taken from the 25cm (10inches) slabs in Phase II before the HVS loading was started, and Iowa shear tests were run on these cores to determine the interface shear strength. Though the concrete-asphalt interface was intended to be unbonded, the cores indicated that the concrete was partially bonded to the asphalt layer. The average interface shear strength from the four cores was 0.82MPa (118.6psi).

After HVS loading, six cores were taken from each test slab, and Iowa shear tests were run on these cores for comparison purpose. The average shear strength for these cores was 0.93MPa (135psi).

Table 2. Properties of the Hardened Concrete Sampled from Truck in Phase II.

Age, days	Compressive Strength, MPa (psi)	Elastic Modulus, GPa (ksi)	Flexural Strength, MPa (psi)
1	13.33(1,933)	-	-
3	24.88 (3,608)	-	-
7	32.07 (4,651)	22.80 (3,307)	-
14	-	26.72 (3,875)	5.57 (808)
28	41.94 (6,083)	27.60 (4,004)	5.89 (855)
56	45.59 (6,612)	29.45 (4,272)	-

Properties of the Concrete

Tables 1 and 2 show the compressive strength, elastic modulus and flexural strength at various curing times of the concrete sampled from the truck in Phase I-a and Phase II, respectively.

Splitting tensile strength test was run on the concrete portion of the core samples after the Iowa bond strength test. From the core samples obtained from the test sections in Phase I-a before the start of the HVS loading, the average indirect tensile strength of concrete from three samples was 4.21MPa (610psi). Eighteen core samples (6 from each test section) were taken from the test sections in Phase I-a after the HVS loading. The average indirect tensile strength ranged from 3.26MPa (473psi) for the 10cm (4inches) concrete slabs to 3.51MPa (509psi) for the 12.5cm (5inches) slabs.

Properties of the Asphalt Concrete

Resilient modulus test was performed on the asphalt portion of a core that was obtained from a 10cm (4inches) slab in Phase I-a before HVS loading. The resilient modulus at 10°C was determined to be 8.71GPa (1,263ksi). Resilient modulus and indirect tensile strength tests were run on the asphalt portion of the cores which were taken from the test sections in Phase I-a at the end of the HVS loading period. Results of these tests are summarized in Table 3. Additional resilient modulus tests performed at 25ºC on three samples of AC gave an average resilient modulus of 5.17GPa (750ksi).

Table 3. Results of Resilient Modulus and Indirect Tensile Strength Tests on the Asphalt Concrete Samples Obtained from Test Sections in Phase I-a After HVS Loading.

	Resilient Modulus, GPa (psi)					
Temp.	5° C			15° C		
Slab Thickness	10 cm (4”)	12.5 cm (5”)	15 cm (6”)	10 cm (4”)	12.5 cm (5”)	15 cm (6”)
1st cycle	12.82 (1.86E+06)	11.58 (1.68E+06)	12.69 (1.84E+06)	8.48 (1.23E+06)	8.07 (1.17E+06)	7.72 (1.12E+06)
2nd cycle	12.68 (1.84E+06)	11.58 (1.68E+06)	12.69 (1.84E+06)	8.48 (1.23E+06)	8.00(1.16E+06)	7.65 (1.11E+06)
3rd cycle	12.76 (1.85E+06)	11.38 (1.65E+06)	12.76 (1.85E+06)	8.41 (1.22E+06)	8.00(1.16E+06)	7.72 (1.12E+06)
Average	12.76 (1.85E+06)	11.51 (1.67E+06)	12.71 (1.84E+06)	8.46 (1.23E+06)	8.02 (1.16E+06)	7.70(1.12E+06)
	Indirect Tensile Strength MPa (psi)					
	-	-	-	2.01 (292)	1.81 (263)	1.81 (263)

FWD Tests

FWD tests were performed on the composite pavement test sections in all phases of the study. The measured FWD deflection basins were used to estimate the elastic modulus of the pavement materials and the stiffness of the springs used to model the load transfer at the joints and concrete-asphalt interface through a back-calculation process. This back-calculation process also allowed for the verification of the elastic modulus of the concrete and the asphalt layer, previously evaluated from laboratory testing.

FWD tests were run at midday between 1:30PM and 3:30PM and at early morning between 7AM and 8AM. At midday, the temperature differential tends to be positive and slab tends to curl down at the edges and joints. This is the best time to run the FWD test for evaluation of joints because the slab is more likely to be in full contact with the layer underneath at both the edges and joints. From midnight to early morning, the temperature differential tends to be negative and the slab tends to curl down at the center of the slab. This is an ideal time to run the FWD test at the center of the slab for evaluation of the condition of the concrete slab and the layer underneath. In order to reduce the effects of the joints, the FWD test run in the morning was performed on a transition slab with the same slab thickness, but with a larger panel size.

HVS Testing and Data Collection

HVS Testing

Testing of the composite pavements was performed using a HVS, Mark IV model using a super single tire with a contact pressure of 0.83Mpa (120psi). The wheel load traveled at a speed of 5km/hr (8mph) in a uni-directional mode with no wander, and along the longitudinal edge of the test slab. Loading along the edge was chosen because it represents the most critical loading condition for a concrete slab.

HVS Testing of Test Sections in Phase I-a

HVS loading of the test section with 10cm (4inches) slabs was started on July 11, 2004. This test section performed well with no observed cracking after three days of loading at 40kN (9kips) with a total of 36,407 passes, followed by 12 days of loading at 53.4kN (12kips) with a total of 146,748 passes. Then the load was increased to 66.7kN (15kips) for three days with 35,918 passes and then to 80kN (18kips) for two days with 21,727 passes, Finally the load was increased to 93.4kN (21kips)) and corner cracks developed after 12,187 passes.

HVS loading of test selection with 12.5cm (5inches) slabs was done from November 1, 2004 through January 14, 2005. This test section was loaded with 79,014 passes of 40kN (9kips), 130,186 passes of 53.4kN (12kips), 25,638 passes of 66.7kN (15kips) and finally 128,817 passes of 80kN (18kips) HVS wheel load. No load-induced cracks were observed during and at the end of this testing period. Only a few small shrinkage cracks were observed on the surface of the concrete.

HVS testing of the test section with 15cm (6inches) slabs was done from May 23, 2005 through May 26, 2005. This test section was loaded for three days with a total of 41,239 passes of 53.4kN (12kips) wheel load. No load-induced cracks were observed, and only a few small shrinkage cracks were observed on the surface of the concrete.

Table 4. Summary of HVS Loading on Test Sections in Phase II.

Slab	Load	Starting Date	Ending Date	# of Passes
25 cm (10”)	53.4 kN (12 kips)	November 14, 2005	December 4, 2005	87,508
	66.7 kN (15 kips)	December 4, 2005	December 11, 2005	86,954
	80 kN (18 kips)	December 11, 2005	December 16, 2005	72,554
20 cm (8”)	53.4 kN (12 kips)	January 9, 2006	January 15, 2006	73,662
	66.7 kN (15 kips)	January 15, 2006	January 21, 2006	60,923
	80 kN (18 kips)	January 21, 2006	January 28, 2006	67,015
15 cm (6”)	53.4 kN (12 kips)	January 30, 2006	February 5, 2006	73,108
	66.7 kN (15 kips)	February 5, 2006	February 10, 2006	67,015
	80 kN (18 kips)	February 10, 2006	February 15, 2006	65,908

HVS Testing of Test Sections in Phase I-b

The three test section on Lane 7 (in Phase I-b), with a panel size of 1.2 x 1.2m (4 x 4ft) and concrete slab thickness of 10, 12.5, and 15cm (4, 5, and 6 inches), were loaded with 40,650, 38,800, and 26,040 passes, respectively, of 53.4kN (12kips) HVS wheel loads at 0.83MPa (120psi) tire pressure. No load-induced or shrinkage crack was observed in any of the three test sections at the end of each loading period.

HVS Testing of Test Sections in Phase II

The test sections in Phase II with a panel size of 1.8 x 1.8m (6 x 6ft), concrete slab thickness of 15, 20, and 25cm (6, 8, and 10 inches), and un-bonded concrete-asphalt interface were loaded by three levels of HVS wheel loads. Table 4 summarizes the HVS loads, numbers of wheel passes and loading periods for the test sections in Phase II. No load-induced or shrinkage cracks were observed in any of the slabs at the end of the tests.

Data Collection

For each test slab, the strain gages were connected to a strain indicator unit, Vishay System 6000 (Model 6100) for strain reading and data acquisition. This enabled the recording of dynamic strains as the wheel passed over the pavement. Data collection for load-induced strain was started immediately after the start of HVS loading.

Strain data were collected for 30 seconds at one-hour intervals. The rate of data collection was 100 strain values per second. This rate allowed for the capture of the progression of the strain and to especially observe the strain reversal phenomenon.

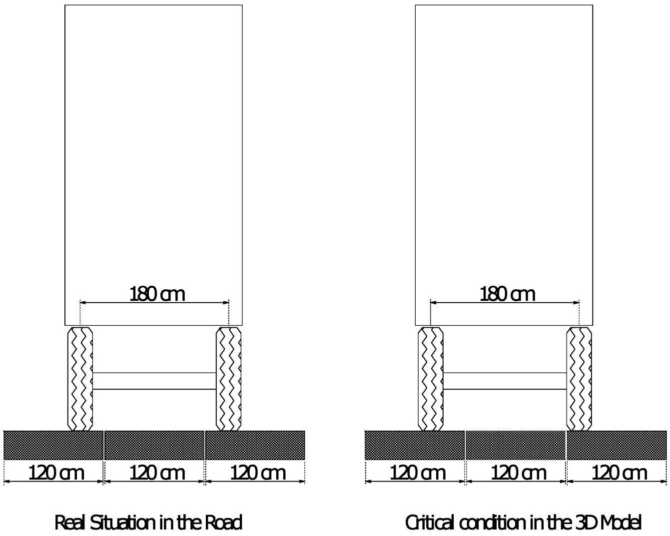


Fig. 8. Axle Load Positioned on Slabs with 1.2m (4ft) Joint Spacing.

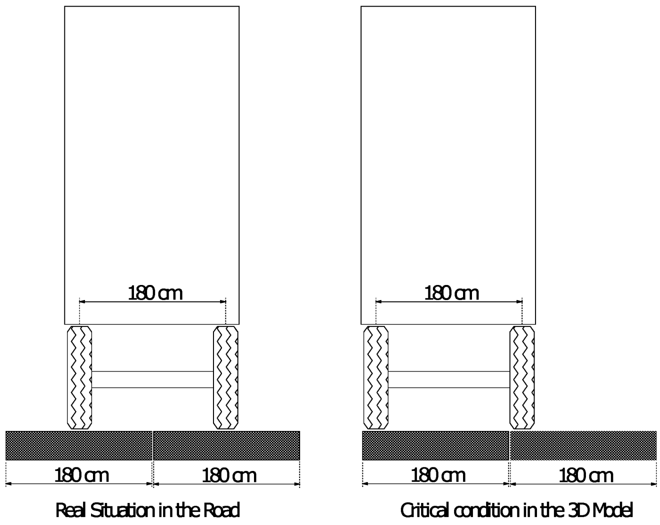


Fig. 9. Axle Load Positioned on Slabs with 1.8m (6ft) Joint Spacing.

All the thermocouples were connected to the same data acquisition system. Temperature data were collected during the entire day at 5-minute intervals. Data collection for temperature was started before the loading period.

Evaluation of Potential Performance of WT Designs

Method of Analysis

In order for the 3-D analytical model to accurately analyze the behavior of WT pavements, it needs to have the correct properties of the pavement materials and the correct values of spring stiffness for modeling the behavior of joints and concrete-asphalt interface. The elastic moduli of the concrete and asphalt materials were initially estimated from the results of laboratory tests on these materials. The results of the FWD tests on the composite pavement test sections were used to estimate the elastic moduli of the other pavement materials and the joint and interface spring stiffnesses, and also to adjust the values of elastic moduli of the concrete and asphalt materials by back-calculation method (of matching the analytically computed deflections with the measured FWD deflections).

The estimation of the properties and parameters of the pavement materials was further refined by matching the analytically computed strains with the measured strains in the test sections caused by HVS wheel loads.

The 3-D finite element model with the model parameters for each test section, as determined from the deflection-based and strain-based calibration, was used to perform a stress analysis to determine the maximum stresses in each WT pavement under typical critical temperature-load conditions in Florida. The potential performance of each WT pavement was assessed based on (1) the maximum tensile stress in the concrete, (2) the maximum shear stress at the concrete-asphalt interface, and (3) the maximum tensile stress in the AC.

A 106.8-kN (24-kip) single axle load, slightly higher than the maximum legal single axle load of 97.9kN (22kips) in Florida, was used as the applied load in the analysis. The two critical loading positions used in the analysis were (1) the mid-edge and (2) the corner of the slab. Figs. 8 and 9 show the positions of the axle load used for the slabs with joint spacings of 1.2m (4ft) and 1.8m (6ft), respectively. These figures also show the comparison between a typical load position with the critical load position.

Minimum and maximum temperature differential in the concrete slab as observed during the HVS loading were around -5.6ºC (-10ºF) and +11.1ºC (+20ºF), respectively. These two extreme temperature differentials were used in the critical stress analysis.

The maximum computed tensile stresses in the various bonded concrete slabs (in Phases I-a and I-b) and partially bonded concrete slabs (in Phase II) caused by a 106.8-kN (24-kip) single axle load placed at two different critical positions (mid-edge or corner), and for three different temperature differentials in the concrete slab (-5.5, 0, or +11.1ºC (-10, 0, or +20ºF)) are shown in Table 6. Three different AC moduli, namely 2.07GPa (300ksi), 4.83GPa (700ksi), or 7.58GPa (1,100ksi), which represent the condition of the AC at different temperatures, were used in the analysis.

Model parameters of the 3-D model for each test section used in the critical stress analysis are displayed in Table 5. Note that all the model parameters, except for the joint spring stiffnesses, are from the results of deflection-based and strain-based calibration. All the joint spring stiffnesses are set to be zero in this analysis, based on the expectation that all the joints will eventually crack all the way through, and there will eventually be less load transfer across the joints as compared with their initial conditions

The maximum computed stresses in the concrete slab caused by the critical loading condition (of a 106.8-kN (24-kip) single axle load, placed at the mid-edge of the slab, and at a temperature differential of +11.1ºC (+20°F) in the concrete slab) were used to assess the potential performance of the WT pavement test sections evaluated in this study. The fatigue curve given by the PCA, which relates the stress/strength ratios with the number of repetitions to produce fatigue failure in concrete, was used to estimate the number of load repetitions to failure.

The following equations were used to calculate the maximum

Table 5. Model Parameters of the 3-D Model for Each Test Section Used in the Analysis.

		Phase I-a			Phase I-b			Phase II		
		10cm	12.5cm	15cm	10cm	12.5cm	15cm	15cm	20cm	25cm
Material Elastic Moduli [GPa]	Concrete	30	30	30	30	30	30	28.96	30	30
	AC Layer	2.07–7.58	2.07-7.58	2.07-7.58	2.07-7.58	2.07-7.58	2.07-7.58	2.07-7.58	2.07-7.58	2.07-7.58
	Base	1.1	1.1	1.1	0.9	0.9	0.9	1.1	1.1	1.1
	Subgrade	0.21	0.21	0.21	0.19	0.19	0.19	0.21	0.21	0.21
Spring Constants [kN/m 10^5]	Interface X							5.25 – 6.13	3.5 – 5.25	1.75 – 5.25
	Interface Y							5.25 – 6.13	3.5 – 5.25	1.75 – 5.25
	Interface Z							8.75×10^{12}	1.75×10^{13}	1.75×10^{13}
	Trans. Joint X	0	0	0	0	0	0	0	0	0
	Trans. Joint Y	0	0	0	0	0	0	0	0	0
	Trans. Joint Z	0	0	0	0	0	0	0	0	0
	Long. Joint X	0	0	0	0	0	0	0	0	0
	Long. Joint Y	0	0	0	0	0	0	0	0	0
	Long. Joint Z	0	0	0	0	0	0	0	0	0

Table 6. Maximum Tensile Stresses in the Concrete Slabs Caused by a 106.8-kN (24-kip) Single Axle Load at Various Critical Loading Conditions.

Tensile Stress (MPa)			AC Elastic Modulus (GPa)					
			Mid-Edge			Corner		
Phase	Slab	Temp	2.07	4.83	7.58	2.07	4.83	7.58
I-a (bonded) 1.8 x 1.8 m	10 cm	-5.5	1.70	2.12	2.62	1.79	2.40	2.89
		0	2.30	1.52	1.09	1.41	0.93	0.66
		11.1	3.92	3.05	2.53	2.25	2.01	1.85
	12.5 cm	-5.5	1.54	1.97	2.49	1.73	2.30	2.82
		0	2.02	1.44	1.09	1.18	0.82	0.62
		11.1	3.66	2.99	2.56	2.01	1.85	1.72
	15 cm	-5.5	1.39	1.80	2.32	1.71	2.18	2.70
		0	1.77	1.31	1.03	1.01	0.72	0.56
		11.1	3.37	2.84	2.49	1.93	1.80	1.70
I-b (bonded) 1.2 x 1.2 m	10 cm	-5.5	2.23	2.06	2.10	2.28	2.09	2.35
		0	1.97	1.29	0.90	1.42	1.05	0.86
		11.1	3.83	3.01	2.52	2.84	2.34	2.04
	12.5 cm	-5.5	2.17	2.04	2.00	2.35	2.18	2.27
		0	1.68	1.18	0.88	1.38	1.11	0.93
		11.1	3.36	2.80	2.43	2.53	2.18	1.96
	15 cm	-5.5	2.05	1.94	1.91	2.35	2.18	2.13
		0	1.45	1.06	0.83	1.35	1.11	0.96
		11.1	2.87	2.50	2.24	2.20	1.96	1.79
II (partially bonded) 1.8 x 1.8 m	15 cm	-5.5	1.57	-	-	1.50		
		0	1.75	-	-	1.26		
		11.1	3.28	2.75	2.49	1.88		
	20 cm	-5.5	1.28	-	-	-		
		0	1.38	-	-	-		
		11.1	2.76	2.43	2.23	-		
	25 cm	-5.5	1.14	-	-	-		
		0	1.09	-	-	-		
		11.1	2.20	2.00	1.89	-		

number of load repetitions as a function of the stress/strength ratio:

$$NLR = 10^{(96.5 - 100\,r)/8.1} \qquad \text{If} \quad r > 0.5$$

$$NLR = \text{infinite} \qquad \text{If} \quad r < 0.5$$

Where NLR = number of load repetitions to failure, and

r = stress/strength

The average flexural strength at 56 days of the concrete used in the test sections was 5.8MPa (842psi), as presented in Table 1. This flexural strength value was used in the computation of the stress to flexural strength ratios for the WT pavement test sections.

Table 7 displays the computed maximum stresses and stress to flexural strength ratios for all the WT pavement test sections evaluated in this study. The allowable number of 106.8-kN (24-kip) single axle loads under critical loading conditions were also computed and shown in this table. It is to be stressed that the results of these analyses are only applicable to the condition of the test sections, which had 11.4cm (4.5inches) of AC layer over 30.5cm (12inches) of limerock base. Evidence indicates that the maximum computed stresses were all below the flexural strength of the concrete for all the test sections. This means that all the WT pavement test sections with a concrete slab thickness of 10cm (4inches) or higher can withstand certain number of repetitions of the 106.8-kN (24-kip) single axle load under the critical loading condition without cracking. The allowable number of repetitions of this critical load increases with slab thickness. The allowable number of load repetitions also increases with smaller joint spacing.

To be able to withstand the critical load without fear of fatigue failure (for an infinite number of critical load repetitions), a minimum slab thickness of 15cm (6inches) would be needed for a joint spacing of 1.2m (4ft), and a minimum slab thickness of 20cm (8inches) would be needed for a joint spacing of 1.8m (6ft).

Summary of Findings

A full scale experiment was performed at the APT facility located at the FDOT Material Research Park to evaluate the feasibility of using WT pavements in Florida. A total of nine instrumented WT test sections were constructed and tested using a HVS. A 3-D finite element model was developed to analyze the behavior of the WT pavement test sections. The model was verified and calibrated using the measured FWD deflections and HVS load-induced strains from the test sections. The model was then used to evaluate the potential performance of these test sections under a typical critical

Table 7. Computed Stress Ratio in the Concrete and Allowable Number of 106.8-kN (24-kip) Single Axle Loads under Critical Loading Conditions.

Phase	Slab Thickness	Stress MPa, (psi)	Stress-strength Ratio	# of Repetitions of 106.8-kN (24-kip) Axle Loads to Failure
I-a	10.0 cm (4")	3.92 (568.30)	0.675	3,810
	12.5 cm (5")	3.66 (531.43)	0.631	13,231
	20.0 cm (6")	3.36 (488.60)	0.580	56,178
I-b	10.0 cm (4")	3.83 (555.06)	0.659	5,958
	12.5 cm (5")	3.36 (486.94)	0.578	59,416
	20.0 cm (6")	2.87 (416.15)	0.494	no limit
II	15.0 cm (4")	3.28 (476.00)	0.565	85,963
	20.0 cm (5")	2.76 (400.42)	0.476	no limit
	25.0 cm (6")	2.20 (318.40)	0.378	no limit

temperature-load condition in Florida. A summary of the findings from this study is presented in the following section.

Bond Strength at the Concrete-Asphalt Interface

In the construction of the test sections with a bonded concrete-asphalt interface (in Phases I-a and I-b), the asphalt surface was milled, cleaned and sprayed with water before the placement of concrete. This method was found to produce excellent bonding at the interface. The average shear strength from the Iowa shear test on the cores from these test sections was 1.52MPa (220psi) for Phase I-a, and 1.34MPa (195psi) for Phase I-b. The maximum computed shear stress at the interface under the critical temperature-load condition for all cases is only 0.59MPa (85psi).

In the Construction of the test sections with an unbonded concrete-asphalt interface (in Phase II), a white-pigmented curing compound was sprayed on the surface of the asphalt to act as a debonding agent before the placement of concrete. Results of Iowa shear test on the cores from these test sections indicated an average shear strength of 0.82MPa (119psi) before the HVS loading and 0.93MPa (135psi) after the HVS loading. This indicates that partial bonding existed at the interface though an unbonded condition was intended, and that the bonding improved with additional loading on the pavement.

Development of the 3-D Finite Element Model

A 3-D finite element model was developed for the analysis of WT pavements. The model was verified and calibrated with the measured FWD deflections and HVS load-induced strains. It was found that the bonded interface (as existed in the test sections in Phases I-a and I-b) could be modeled well by modeling the concrete as perfectly bonded to the asphalt. Partially-bonded interface (as existed in the test sections in Phase II) could be modeled well by vertical and horizontal springs connecting the concrete layer with the asphalt layer.

For the conditions at the time of the HVS loading on the test sections, it was found that joints could be modeled well by springs connecting the slabs at the joints. It is postulated that load transfer at the joints will eventually decrease with time. In the analysis for long-term performance of the test sections under the critical condition, the worst joint condition was assumed and thus springs of zero stiffness were used to model the joint behavior in this analysis.

Effects of Elastic Modulus of AC

The elastic modulus of the AC layer was found to have great influence on the maximum tensile stresses in the concrete slab. Thus, for the analysis for the most critical loading condition, the lowest possible elastic modulus of the AC (at the highest temperature) was used.

Effects of Concrete Panel Size

Maximum stresses in the concrete were found to decrease as the joint spacing decreases. At the most critical loading condition, the concrete slabs with 1.2m (4ft) joint spacing had lower maximum stresses than those with 1.8m (6ft) joint spacing.

Effects of Bonded versus Partially Bonded Interface

At the condition of negative temperature differentials in the concrete slab, the concrete slabs with a partially bond interface were found to have higher maximum stresses than those with a fully bonded interface. However, at the condition of zero or positive temperature differential in the slab, the maximum stresses in the partially bonded slabs are about the same as those in the fully bonded slabs.

Potential Performance of the Test Sections

The verified and calibrated 3-D finite element model was used to evaluate the potential performance of the nine test sections under a critical temperature-load condition. Maximum tensile stresses in the pavement were computed for the critical condition when a 106.8-kN (24-kip) single axle load (which is higher than the legal limit of 97.9kN (22kips) in Florida) was placed at the mid-edge of the slab (which is the most critical loading position) and when the temperature differential in the concrete slab was +11.1°C (+20°F) (which is a typical severe temperature condition in the summer time in Florida.)

The maximum computed stresses in the concrete slabs were all below the flexural strength of the concrete for all the 9 test sections. Similarly the maximum tensile stresses in the AC layer were all below the tensile strength measured in the IDT test. Based on the computed maximum stresses in the concrete, the expected numbers of repetitions of the 106.8-kN (24-kip) single axle loads at the critical thermal condition were computed for the nine test sections. Results show that the 10cm (4inches) slabs can be used for heavy (106.8-kN (24-kip) single axle) load but only for low-volume traffic condition. The allowable traffic volume increases as the concrete slab thickness increases. In order to be able to withstand the critical load without fear of fatigue failure (for an infinite number of critical load repetitions), a minimum slab thickness of 15cm (6inches) would be needed for a joint spacing of 1.2m (4ft), and a minimum slab thickness of 20cm (8inches) would be needed for a joint spacing of 1.8m (6ft).

Recommendations

The developed 3-D finite element model is recommended for use for analysis of WT pavements subjected to load and temperature effects. The model parameters needed in the analysis include the elastic moduli of the concrete, AC, base, and effective subgrade layers. For analysis of long-term behavior, the joint stiffness can be assumed to be zero. For partially bonded interface condition, the stiffness values of the springs for modeling the interface are also needed as model parameters. The elastic moduli of concrete and AC can be determined by testing in the laboratory, while the other parameters can be determined through the back-calculation method of matching analytical deflections and strains due to an applied load with measured values.

The WT pavement test sections in this study were limited to the following conditions:

1. When each test section was tested by the HVS, the test section was shaded by the HVS. As a result, the temperature differential in the concrete slabs was much lower than its maximum potential amount if the slabs were unshaded. The test sections could not be tested and monitored for the most critical temperature condition.
2. Due to the high demand on the use of the HVS, it was not possible to load the test sections for an extended period of time to evaluate the long-term performance of the WT pavements and the modes of failures under actual traffic and weather conditions.

It is recommended that experimental WT pavement test sections of various designs be constructed on actual roadways to evaluate their behavior and performance under actual environmental and traffic conditions. The experimental pavement sections will be instrumented for monitoring of temperature and strains on a long-term basis. This will enable the monitoring of the behavior of the WT pavements under critical load and temperature conditions, and the verification of the predicted response from the analytical model. It will also enable the evaluation of the long-term behavior of the WT pavements under actual traffic and weather conditions.

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