

Model Development and Interim Design Procedure Guidelines for Ultra-thin Whitetopping Pavements

Mr. James W. Mack, Director Of Engineering And Rehabilitation
American Concrete Pavement Association

Dr. Chung Lung Wu, Senior Engineer
PCS/Law

Mr. Scott Tarr, Engineer
Construction Technology Laboratory

Dr. Tarek Refai, Senior Engineer
Construction Technology Laboratory

INTRODUCTION

Ultra-thin whitetopping (UTW) is a relatively new technique for resurfacing deteriorated asphalt pavements. It involves placing very thin concrete slabs (50 to 100 mm (2 to 4 in. thick)) on old asphalt pavements to form bonded, or partially bonded, composite pavements. The reduction in thickness is justified by the use of high quality concrete with relatively high strength, close joint spacings, and bond between the concrete and the existing asphalt pavement.

Although the use of UTW has been increasing, no design method has been available. Because of this, the pavements have probably been either over-designed or under-designed. To address this deficiency, the Portland Cement Association and the American Concrete Pavement Association initiated a research project to develop an Ultra-thin Whitetopping design procedure.¹ The objective of this research was to develop a mechanistic UTW pavement design procedure. It involved:

1. Literature review of past and current work
2. Perform a condition survey of UTW sites in GA and TN
3. Instrument and load test a test sections
4. Develop a 3-dimensional finite element model for UTW analysis
5. Develop of a design procedure and construction guidelines

This paper summarizes those results.

LITERATURE REVIEW

The first experiments using thin whitetoppings were actually conducted in the early 1970's^{2,3} when steel fibers were introduced to the concrete industry. The success of the experiments were limited due to inexperience and difficulties with the steel fiber reinforced concrete. Furthermore, the steel fiber reinforced concrete overlays

were designed and constructed the same as conventional whitetoppings. All in all, these early trials are quite different, with respect to materials, construction, and design from today's UTW projects.

The first modern UTW experimental project was constructed on the access road to a waste disposal landfill in Louisville, KY in September 1991.^{4,5,6,7} Following the success of the UTW project in Louisville, several states, including Tennessee, Georgia North Carolina, Kansas, Iowa, Pennsylvania, New Jersey, Colorado, and Missouri have constructed and are evaluating their own UTW projects^{5,9,10,11,12}. Preliminary evaluations of these UTW projects show remarkable and promising performance. All told, approximately 70 UTW projects were built in the US between 1991 and 1996.⁸

Other countries also have experimented with ultra-thin whitetopping. In 1993, Sweden constructed four test sections and evaluated their performance under heavy traffic for 18 months.^{13,14} Mexico also has constructed a UTW research project on an urban arterial in Tijuana. Early indications are that UTW significantly extends the useful life of flexible pavements.¹⁵ Overall, Mexico has placed over 519,000 m² (621,000 yd²) in 19 different projects.

Some of the basic results found in almost all of these research projects are:

1. Bond between ultra-thin whitetopping and existing asphalt pavement significantly reduces the stresses in the concrete section. This allows the section to perform as a composite section.
2. Bond strength improves by milling and cleaning the old asphalt pavement. This exposes aggregates in the asphalt, which helps lock the pavement layers together.
3. An appropriately thick asphalt layer is needed to insure that the pavement will perform. It is recommended that a minimum asphalt thickness of 75 mm (3 in.) after milling, be used.
4. Panels with short joint spacing perform better. The typical joint spacing that have performed well are somewhere between 0.6 and 1.5 m (2 and 5 feet).
5. The small panels work better because they transfer the wheel load to the flexible pavement through deflection, rather than absorb it by bending.

ULTRA-THIN WHITETOPPING PAVEMENT PERFORMANCE SURVEY

A pavement performance survey was conducted on several UTW pavements located in Tennessee and Georgia during the week of November 13, 1995. Table 1 summarizes the results of the projects reviewed. The sites surveyed were chosen because they either exhibited very good performance, or very poor performance. It was hoped that common elements that contributed to either success or failure could be identified.

The distress parameters surveyed included crack location, crack width, joint spacing, joint/crack faulting, surface wear, and approach and leave edge condition in relation to the surrounding asphalt pavement. When possible, cores were taken to measure concrete and asphalt thickness and quality, as well as the direct shear strength between the concrete and asphalt. Again when possible, a traffic count was done to document the service the pavement provides.

Overall, sections that had small joint spacing, adequate asphalt base, and proper asphalt preparation by milling before construction are performing well. The two sites that are exhibiting extremely poor performance were both placed on inadequate or substandard base material. The UTW at S. Hickory & 28th Street in Chattanooga was on 280 mm (11 in.) of a weak, loose, asphalt treated layer that detached and fell apart during coring. The UTW at 1st Street & Woodland Avenue in Nashville was on an old overlaid cobblestone pavement that was exposed during the milling operations.

FIELD INSTRUMENTATION AND LOAD TESTING

The objectives of the field testing were:

1. To study UTW pavement responses under traffic loading.
2. To evaluate interface bonding strength between the concrete and asphalt layers.
3. To develop data in order to verify the 3-dimensional finite element model being developed for analyzing composite pavement systems.
4. To calibrate stresses calculated by the 3-dimensional model for various interface bonding strengths.

Field instrumentation and load testing was conducted on three different sites, one in St. Louis, Missouri and two in Colorado. The St. Louis site was the primary test site for development of the UTW design procedure, while the Colorado sites were used for model verification. This paper will concentrate on the results from the Spirit of St. Louis Airport. More information on the Colorado sites can be found in reference 1.

Spirit of St. Louis Airport

The test section at Spirit of St. Louis Airport was constructed as part of an apron overlay in early 1995. It was designed as 89 mm (3.5 in.) of concrete over asphalt with joints spaced every 1.27 m (50 in.). To increase interface bonding during construction, the old asphalt pavement surface was milled and thoroughly air blasted prior to concrete placement. The concrete mix design was as follows:

Type I cement	= 303.6 kg/m ³ (510 lb/ yd ³)
Class C fly ash	= 47.6 kg/m ³ (80 lb/ yd ³)
Sand	= 751.8 kg/m ³ (1263 lb/ yd ³)
#57 Limestone	= 1118.5 kg/m ³ (1879 lb/ yd ³)
Water	= 108.9-113.1 kg/m ³ (183-190 lb/ yd ³)
Air entrainment	= 0.084 ml/kgc (5.5 oz/cwt)
Low range water reducer	= 0.261 ml/kgc (17 oz/cwt)
Polypropylene fiber	= 1.79 kg/m ³ (3 lb/ yd ³)

The 28-day modulus of rupture (MR) was specified at 4.65 MPa (675 psi.). However, laboratory test results showed that the MR was consistently greater than 6.90 MPa (1,000 psi.).

A total of six test slabs were instrumented and load tested. The six slabs, labeled as Slabs A to F, had three different joint conditions. Slabs A and B had normal contraction joints. Slabs C and D had one joint completely sawed through the concrete to the bottom of the asphalt layer to simulate a free edge condition. Slabs E and F had all joints completely sawed through the concrete and asphalt layer to form isolated slabs.

Strain gages, located at slab center and joints were, installed onto the top of the asphalt layer and 25 mm (1 in.) above the asphalt surface during construction. Strain gages on top of the concrete were installed before load testing. In addition to the strain gages, two thermocouple trees, with temperature gages at approximately the top, middle and bottom of concrete slabs, and 12 mm (1/2 in.) into the asphalt were installed during construction. Joints and relative test slab locations, as well as plan and section views of the strain gage and embedded thermocouples locations are presented in Figure 1.

Load Testing

Two sets of load tests were conducted: one in early May and the other in late September. The test loads were 44.4 kN (10,000 lb.) for the May test and 53.3 kN (12,000 lb.) for the September test. The load was applied at four different locations on each test slab (over gages 1,2,4,8). Load testing was conducted through the day from early morning to afternoon to cover a range of temperature conditions. Air and pavement temperature were measured and recorded during the entire load testing.

Laboratory Tests of Pavement Cores

A total of 22 pavement cores were taken during May load testing. The average concrete thickness measured 91 mm (3.6 in.) and the average asphalt thickness measured 79 mm (3.1 in.). In addition to thickness, other laboratory tests were run on the cores to determine the concrete and asphalt material properties. All results are shown in Table 2. Since asphalt is temperature dependent, its properties were measured at 5 and 25 °C (41 and 77 °F).

Also shown in Table 2 is the measured interface bond strength between the concrete and asphalt layers. Both the direct shear test and the bond pull-off test were used to measure the bond strength. Of the fifteen cores subjected to direct shear, the strength ranged from 0.50 to 1.61 MPa (73 to 234 psi) with an average value of 0.70 MPa (102 psi). The seven remaining cores measured with the bond pull-off test had a strength between 0.31 and 0.77 MPa (45 and 111 psi) and an average value of 0.51 MPa (74 psi). This is about 73% of the shear strength.

Results of Load Testing

From strains measured at top of concrete slabs and 25 mm (1 in.) above asphalt surface, strains at bottom of concrete slabs can be extrapolated. For a fully bonded composite pavement, strains at bottom of concrete slabs would be the same as those at top of the asphalt layer. For a completely unbonded system, strains at concrete slab top would be the same as those at concrete slab bottom.

Table 3 shows the strains from the load testing on Slab D (one free edge) for the May test. The other slabs and tests had similar results. By comparing concrete top strains with concrete slab bottom strains, and concrete bottom strains with asphalt top strains, it was found that UTW pavements should be treated as partially bonded composite pavements.

Although the pavement temperature and temperature differential in the slabs increased throughout the testing, no consistent trend in the measured strains, either increasing or decreasing, was observed for all the slabs. The maximum temperature differential measured in the concrete slab during testing was 7.2 °C (13 °F). The maximum temperature difference between top of concrete slab and bottom of asphalt layer was 11.2 °C (20.3 °F). There was no appreciable difference between the May and September test results.

Comparing strains caused by load applied at Location 1 (normal joint) and 3 (free edge) for Slab D, it is clear that a better joint condition can reduce load induced stresses. Free edge joints had a much higher stress than a normal joint. In all test slabs, the normal joint conditions produced similar pavement stresses as the center loading conditions.

Other slabs were specifically set-up to compare stress load transfer (stress of unloaded side divided by stress of loaded side) between a normal joint and a free joint. It was found that normal joints had about 68% load transfer for the May test and 39% for the September test. Cooler pavement temperature may have caused the decrease. As expected, the free joints had very little stress load transfer (close to zero).

During both load tests, slab movements were measured in Slabs B, C and F. Since no reference point was established, the first set of measurements, taken in early morning were used as the base line. Relative movements were determined by subtracting the base line measurements from subsequent measurements. Although pavement temperatures were different for each test date, no appreciable difference in relative slab movements was observed. The maximum relative movements were 0.48, 0.41 and 0.63 mm (0.019, 0.016, and 0.025 in.) for Slabs B, C and F, respectively. These values are very small and seem to indicate that the pavement layers maintained full contact with their foundation.

Colorado Test Pavements

The Colorado UTW projects all had the new concrete placed on top of a newly placed asphalt layer. In one test section, the new asphalt was milled and in the other 2 test sections, it was placed directly on the new asphalt (no milling). The test sections placed directly on top of the asphalt had average shear strengths 0.23 and 0.45 MPa (36 and 65 psi). The test slabs built on the new asphalt surface that was milled had no bond. Apparently, a new asphalt surface, or the milling of a new asphalt surface, adversely affect the ability of the whitetopping to bond to the underlying layers.

DEVELOPMENT OF A 3-D MODEL FOR COMPOSITE PAVEMENT ANALYSIS

Traditionally, two-dimensional (2-D) finite element models (FEM) such as ILLISLAB, J-SLAB, and FEACONS have been used to analyze concrete pavement systems. These programs calculate deflections and stresses in jointed concrete pavements, with and without load transfer, for various subgrade support conditions. The pavement systems can have one or two layers above the subgrade and these layers can be bonded or unbonded. Two-dimensional models can also analyze linear temperature differentials. However, 2-D finite element programs are limited in their analysis capabilities in several areas. Some are:

- Can only model interface conditions between layers as fully bonded or fully unbonded. The actual condition is partial bond.
- Can only model vertical boundaries (cracks, joints, and edges) that coincide in both the concrete and underlying asphalt. In reality, the joints only go through the concrete layer. Likewise, the deteriorated cracks in the asphalt pavement may not align with the joints in the concrete overlay.
- Can only model slab curling and warping conditions where the top layer curvature is the same as the underlying layer. Because the asphalt is unjointed thermoplastic base, this is not realistic.
- Can only analyze a linear temperature distribution between the top of the surface layer and the bottom of the base layer. Due to asphalt's thermoplastic properties and the partial bonding conditions, non-linear or unequal temperature distributions may exist.
- Assumes that there is no compression in any layer. There is some compression in the asphalt layer.

Because of these inherent limitations, it was necessary to develop a three-dimensional (3-D) model to analyze the unique characteristics of the UTW pavement system.

Three - Dimensional Model

The 3-D FEM of the pavement slab system, shown in Figure 2, was developed using NISA STATIC program.¹⁸ NISA STATIC is the linear and nonlinear static analysis solver. The element library contains a large variety of elements including higher order elements. Material and geometric nonlinearity as well as hyperelastic materials are covered in nonlinear analysis. Also, the shear-friction gap element is covered in nonlinear analysis.

A section of nine slabs were modeled. Each slab was 1.27 m (50 in.) square and each consisted of two separate layers. The top layer represented the concrete overlay and the bottom layer represented an existing asphalt pavement. Each layer consisted of a 10 x 10 mesh of 8-node solid elements with three translational degrees of freedom at each node. Each of the solid elements had physical thickness equivalent to the layer thickness.

The nine slabs were connected to each other by vertical three-dimensional spring elements simulating potential load transfer between the concrete layers in the slab sections. The spring element used for load transfer between the concrete slabs had a 2-node uniaxial tension-compression capability with three translational degrees of freedom at each node. Load transfer was assumed to be the spring stiffness multiplied by the tributary horizontal length. For the cases where the joints between the concrete slabs were free, a very low value was assigned for the axial stiffness of these springs. The adjoining nodes between the nine asphalt slab sections were rigidly connected with rigid link members to accomplish complete continuity in the bottom layer.

Similar three-dimensional, 2-node vertical spring elements were used to simulate the soil response below the eight edge and corner slab sections. Each of the soil spring elements had stiffness equivalent to the assumed subgrade modulus (MPa/m^3 (lb/in.^3)) multiplied by its tributary plan area. The vertical response of the soil below the center slab section was modeled using a three-dimensional, 2-node vertical gap element capable of transferring axial compression forces only.

The interface between the concrete and asphalt layers was modeled using three-dimensional horizontal spring elements located at the interface of the concrete and asphalt layers. Two horizontal sets of springs were used at the interface with one set in each direction. Behavior of fully bonded concrete overlay was obtained through the use of very high stiffness for the horizontal interface springs. Similarly, the behavior of unbonded

concrete overlay is modeled when a very low value of stiffness is assigned to the horizontal interface springs. A partially bonded condition can be achieved when a moderate value of horizontal interface spring stiffness is utilized. This is a trial and error process. By changing the value of the stiffness for the horizontal interface springs, the corresponding in-plane stresses (flexural stresses) change for the concrete and asphalt layers. Experimental runs of this model indicated reasonable and acceptable results.

Parametric Study

Once the 3-D model was developed, a parametric study was performed to evaluate the relative impact of various whitetopping pavement characteristics. The parameters studied were:

1. Location of the load, center or edge
2. Joint edge support due to cracks in asphalt aligned with PCC slab edges (no crack, one free edge, and four free edges)
3. Load transfer between slabs
4. Degree of bonding at the interface between the concrete and asphalt.

The stresses were evaluated at three critical locations; the surface of the concrete, the bottom of the concrete layer at the interface, and the bottom of the asphalt layer. The concrete and asphalt properties were based on the field evaluations and were held constant throughout the study. The properties used were:

Concrete thickness	= 89 mm (3.5 in.)
Concrete modulus of elasticity	= 23,443 MPa/m (3.4 million psi)
Concrete unit weight	= 2,130 kg/m ³ (133 pcf.)
Concrete coefficient of thermal expansion	= $6.8 \times 10^{-6}/^{\circ}\text{C}$ ($3.8 \times 10^{-6}/^{\circ}\text{F}$)
Concrete Poisson's ratio	= 0.15
Asphalt thickness	= 76 mm (3.0 in.)
Asphalt modulus of elasticity	= 6,895 MPa/m (1.0 million psi)
Asphalt unit weight	= 2,147 kg/m ³ (134 pcf.)
Asphalt coefficient of thermal expansion	= $3.6 \times 10^{-6}/^{\circ}\text{C}$ ($2.0 \times 10^{-5}/^{\circ}\text{F}$)
Asphalt Poisson's ratio	= 0.35
Modulus of subgrade reaction	= 67.9 MPa/m (250 pci.)
Wheel Load	= 4,000 KN (5,000-lb) single wheel

Figures 3 to 5 show the results of the parametric study. As can be seen, the interface bonding strength is the most important factors affecting load induced stresses in the pavement. When the system is fully bonded, minimal stresses at the bottom of the concrete and asphalt exist. As the bond decreases, the stress at the bottom of the concrete increases substantially. However, the stress in the bottom of the asphalt layer remains relatively constant.

For the center and joint-loading with no asphalt crack conditions, the load transfer between concrete slabs has very little effect on the stresses. This is because the asphalt layer provides sufficient support for load transfer. For the conditions where a crack exists below a loaded edge, the load transfer efficiency begins to have an affect, especially for a completely unbonded system. As can be seen Figures 4 and 5, as the load transfer decreases, the tensile stress increase for both the concrete and asphalt layer. This indicates that high load transfer may compensate for situations where the interface bonding conditions have become weak.

DEVELOPMENT OF UTW DESIGN PROCEDURE

If one looks at any design procedure, it involves the selection of the proper concrete thickness in order to carry the anticipated traffic. Accordingly, the UTW design procedure developed here is based on the same principal. It uses the existing asphalt properties and an iterative process to determine the concrete thickness and joint spacing. The procedure is based on and is similar to the PCAPAV Design procedure.¹⁹ It uses load spectra and stress ratio to determine the allowable load applications for a given thickness, joint spacing, asphalt properties, etc.

The philosophy and procedure adopted for the UTW design procedure are:

1. Determine critical load induced stress or strain caused by and 80-KN (18-kip) single axle load and 160 KN (36-kip) tandem axle load.
2. Determine critical temperature induced stress or strain or strain.
3. Determine axle load for the design traffic and determine the number of repetitions expected for each axle load category.
4. Convert the stress/strain caused by 80-KN (18-kip) single axle load and 160 KN (36-kip) tandem axle load and temperature to cover the axle spectra of the design traffic.
5. Using fatigue relationships for concrete and asphalt to determine allowable loads for each axle category.
6. Calculate the stress ratio for each axle category.
7. Determine the amount of fatigue used by each axle load category.
8. Sum up fatigue for all axle load categories and check to see if below 100%. If the fatigue is below 100% for both concrete and asphalt, the design is adequate.
9. Iterate if necessary.

The final design is determined by a trial and error method that iterates concrete thickness and spacing until the total fatigue used for all load applications is below 100%. For this procedure, two design criteria were developed: portland cement concrete fatigue under corner loading and asphalt fatigue under joint loading. Note that steps 3 through 9 are essentially the same steps as the PCAPAV design procedure. The new part for the UTW design procedure has been the development of design equations .

Development of Predication Equations for Design Stresses and Strains

The development of a design equations requires hundreds of computer runs in order to develop a full factorial of pavement parameters. Unfortunately, due to the lengthy computer processing time to model and run the 3-D FEM, it was determined that developing a procedure based entirely on the 3-D model was unfeasible. Therefore, an alternate approach was used to develop design equations. The alternate approach developed and used consisted of:

- A. Verify 3-D FEM program by field load testing data.
- B. Determine effective bonding of composite pavement system.
- C. Develop adjustment factors to correlate 2-D stresses & 3-D stresses for fully bonded pavement systems.
- D. Perform a full-matrix parametric study using 2-D model with fully bonded interface, considering joint spacing, concrete and asphalt layer thickness, concrete and asphalt material properties, load positions, and subgrade material properties
- E. Convert 2-D stresses to 3-D stresses using the adjustment factors developed in Step 3.
- F. Adjust computed stresses to account for the effective bond developed in Step 2.
- G. Develop design equations to correlate the converted and adjusted stresses to different pavement design parameters.

Note: Only the results of the St. Louis site was used in the development of the design equation. Its test sections were constructed under a more controlled environment and used a more common surface preparation technique

Once the design equations have been developed, the concrete thickness and joint spacing can be determined using the analysis described in steps 3 through 9 above to limit the fatigue used by each load to within acceptable limits.

A. Verify the 3-D Finite Element Model

For most cases, the measured stresses in the slabs were greater than computed fully bonded values. At the top of concrete, the average percent difference between the computed and the measured stresses was 14%. At the bottom of concrete slab, the average percent difference was 34%. When partially bonded systems were assumed, the discrepancies dropped to 2% and 7% for stresses at the concrete slab top and bottom, respectively. Because of time required for running the 3-dimensional model, the analysis was conducted with only one level of the partial bonding. However, the model was found to predict the field data reasonably well, and therefore was used for the development of the UTW design guidelines.

B. Determine Effective Interface Bonding from Field Data

The effects of interface bonding were quantified by comparing the measured stresses against computed stresses for fully bonded pavement systems. Comparisons were made for stresses at both the top and the bottom of the concrete slabs. As discussed in the previous section, measured stresses were greater than computed stresses due to partial bonding.

A paired-t test at 75% confidence level found no difference between the May and September test data. From both sets of test data, the average stress increase at the top of the concrete slabs was 19% with a standard deviation of 17%. For stresses at the bottom of concrete slabs, the average increase was 57% with a standard deviation of 32%. Though a large discrepancy between the slab top and bottom stress increase seemed to exist, a closer look of the data revealed that the much larger percent increase at the bottom was due to the much smaller stress values developed at the bottom of concrete slabs.

Considering the variations involved in the field testing, data obtained from concrete slab top was used. From the analysis, an average stress increase from a fully bonded system to a partially bonded system was 19% with a standard deviation of 17%. To account for the variation in the results, the stress increase factor of 36% ($19\% + 17\%$) was used in the design process.

C. Develop Correlation between 2-D Stresses & 3-D Stresses for Fully Bonded Pavement Systems

Due to the processing time for 3-D model computer runs, prediction equations that would convert stresses and strains computed by the 2-D finite element model ILSL2²⁰ to those computed by the new 3-D model were developed. Maximum stresses at the bottom the asphalt layer caused by joint loads and at the top of the concrete slab caused by corner loads were computed for identical UTW pavements using both the 2-dimensional and the 3-dimensional models.

Because the surface profiles measured at the St. Louis site showed very little slab vertical displacements, it is believed that the short joint spacing and the bonding between the concrete and asphalt minimized slab curl. With very little slab curl, it is reasonable to assume that the pavement can maintain constant contact with the foundation underneath. As a result, the load and the temperature induced stresses can be considered separately, with the final stresses as the sum of these two stresses. Accordingly, the 2-D to 3-D conversion equations for load and for temperature were derived separately.

From the stresses computed by both the 2-D and 3-D models, multiple linear regression techniques were used to develop the conversion equations stress at bottom of asphalt layer due to joint loading, stress at top of concrete slab due to corner loading, stress at bottom of asphalt layer at joint due to temperature differential, and stress at top of concrete slab due to corner under temperature differential.

D. Perform Parametric Study Using the 2-Dimensional Model ILSL2

Stresses caused by an 80 KN (18-kip) single axle load (SAL) and a 160 KN (36-kip) tandem axle load (TAL) at both the mid-joint and slab corner, as well the temperature differential in the pavements, were computed using the 2-D FEM computer program ILSL2, assuming fully bonded conditions. For each

loading and for each temperature condition, a total of 80 UTW pavements with the following combinations of pavement parameters were investigated:

Concrete slab dimension	= 0.61 X 0.61 m and 1.27 X 1.27 m (24 X 24 in. and 50 X 50 in.)
Concrete slab thickness, h1	= 51, 76, and 102 mm (2, 3 and 4 in.)
Concrete modulus of elasticity, E1	= 27,580 MPa/m (4 million psi)
Concrete Poisson's ratio, μ_1	= 0.15
Concrete unit weight, g1	= 2,403 kg/m ³ (150 pcf.)
Concrete coefficient of thermal expansion, α_1	= 9.9×10^{-6} /°C (5.5×10^{-6} /°F)
Asphalt layer thickness, h2	= 76 to 229 mm (3 to 9 in.)
Asphalt modulus of elasticity, E2	= 345 to 13,790 MPa/m (50,000 to 2 million psi)
Asphalt Poisson's ratio, μ_2	= 0.35
Asphalt unit weight, g2	= 2,243 kg/m ³ (140 pcf.)
Asphalt coefficient of thermal expansion, α_2	= 3.6×10^{-6} /°C (2.0×10^{-5} /°F)
Temperature differentials in the concrete slab	= +8.3, +2.8, and -5.6°C (+15, +5 and -10 °F)
Modulus of subgrade reaction, K	= 20.4 to 217.2 MPa/m (75 to 800 pci)

Mid-joint loading. Under joint loading, the critical stresses occurred at the bottom of the asphalt layer. However, since asphalt pavements are generally designed by limiting asphalt, both the major and minor principal stresses at the critical locations were recorded. After converting to the 3-D model stresses, the load induced strains were computed from the recorded stresses.

Corner loading. Same as the joint loading case, both an 80 KN (18-kip) SAL and a 160 KN (36-kip) TAL were applied to UTW slab corners. Maximum tensile stresses at top of concrete slabs under the loading were recorded. Locations of the maximum tensile stress were in general away from the load.

Temperature differential. From pavement temperature data recorded from the three testing sites, it was seen that most of the temperature differential occurred in the concrete slab. Furthermore, as a thermo-plastic material, asphalt would not curl as concrete would. It was therefore assumed that all the temperature differential existed in the concrete. The asphalt layer was not subjected to temperature differential. Both the major and minor principal stresses at bottom of asphalt layer at slab joint and the maximum tensile stresses at top of concrete slab were recorded for the different temperature conditions.

E. Convert 2-D Stresses to 3-D Stresses using the Adjustment Factors

Using equations developed in step C, load- and temperature-induced stresses computed by the 2-dimensional model were converted to equivalent 3-dimensional model stresses.

F. Adjust Computed Stresses to Account for the Effective Bond

Because the stresses computed and converted were for UTW pavements with fully bonded concrete and asphalt layers, all the converted load- and temperature-induced stresses were increased by 36%. This was the increase in stresses required to account for partial bonding as determined in step B.

G. Develop Design Equations to Correlate Stresses to Different Pavement Parameters

Using the multiple linear regression technique with the data obtained in the preceding section, prediction equations were derived for computing design stresses and strains. A total of six equations were developed:

AC Strain for 18-kip SAL at joint (microstrain)

$$\text{Log}_{10}(\text{JT}_{18}) = 5.267 - 0.927 * \text{Log}_{10}(\text{K}) + 0.299 * \text{Log}_{10}(\text{L}/\text{le}) - 0.037 * \text{le} \quad (1)$$

$$\text{R}^2_{\text{adj}} = 0.892$$

$$\text{N} = 67.$$

AC Strain for 36-kip TAL at joint (microstrain)

$$\text{Log}_{10}(\text{JT}_{36}) = 6.070 - 0.891 * \text{Log}_{10}(\text{K}) - 0.786 * \text{Log}_{10}(\text{le}) - 0.028 * \text{le} \quad (2)$$

$$\text{R}^2_{\text{adj}} = 0.870$$

$$\text{N} = 69$$

Concrete Stress for 18-kip SAL at corner (psi)

$$\text{Log}_{10}(\text{COR}_{18}) = 5.025 - 0.465 * \text{Log}_{10}(\text{K}) + 0.686 * \text{Log}_{10}(\text{L}/\text{le}) - 1.291 * \text{Log}_{10}(\text{le}) \quad (3)$$

$$\text{R}^2_{\text{adj}} = 0.966$$

$$\text{N} = 80$$

Concrete Stress for 36-kip TAL at corner (psi)

$$\text{Log}_{10}(\text{COR}_{36}) = 4.898 - 0.559 * \text{Log}_{10}(\text{K}) - 0.963 * \text{Log}_{10}(\text{le}) + 1.395 * \text{Log}_{10}(\text{L}/\text{le}) - 0.088 * (\text{L}/\text{le}) \quad (4)$$

$$\text{R}^2_{\text{adj}} = 0.978$$

$$\text{N} = 80$$

AC Strain for temperature differential at joint (microstrain)

$$(\text{JT}_T) = (-28.698) + 2.131 * (a_1 * \Delta T) + 17.692 * \text{L}/\text{le} \quad (5)$$

$$\text{R}^2_{\text{adj}} = 0.788$$

$$\text{N} = 240$$

Concrete Stress for temperature differential at corner (psi)

$$(\text{COR}_T) = 28.037 - 3.496 * (a_1 * \Delta T) - 18.382 * \text{L}/\text{le} \quad (6)$$

$$\text{R}^2_{\text{adj}} = 0.788$$

$$\text{N} = 240$$

where

JT_n = strains at bottom of asphalt layer for axle load or temperature load, microstrain

COR_n = stress at top of concrete slab for axle load or temperature load, psi

K = Modulus of subgrade reaction

L = concrete slab joint spacing, in.

l_e = effective radius of relative stiffness for a fully bonded composite pavement

= $((Eh^3/12)_e / (1 - \mu^2) / K)^{0.25}$, in.

$$(Eh^3/12)_e = (E1 * h1^3) / 12 + E1 * h1 * (NA - h1 / 2)^2 + (E2 * h2^3) / 12 + E2 * h2 * (h1 - NA + h2 / 2)^2 \quad (7)$$

μ = Concrete Poisson's ratio = .15

$E1$ = Concrete modulus of elasticity

$h1$ = Concrete slab thickness

$E2$ = Asphalt modulus of elasticity

$h2$ = Asphalt layer thickness,

NA = neutral axis from top of concrete slab, in.

$$= (E1 * h1 * (h1 / 2) + E2 * h2 * (h1 + h2 / 2)) / (E1 * h1 + E2 * h2)$$

ΔT = temperature differential in concrete slabs, °F

$a1$ = concrete coefficient of thermal expansion, /°F

$a1 * \Delta T$ had no unit and was expressed as 10^{-6}

R^2_{adj} = adjusted coefficient of correlation

N = number of data points

Selection of Fatigue Equations for Portland Cement Concrete and Asphalt

Fatigue equations developed at the Portland Cement Association (PCA)¹⁹ were used for governing the PCC fatigue cracking for the corner loading case. The equations have the following form:

For $SR > 0.55$

$$\text{Log}_{10}(N) = (0.97187 - SR) / 0.0828 \quad (8)$$

For $0.45 < SR < 0.55$

$$N = (4.2577 / (SR - 0.43248))^{3.268} \quad (9)$$

For $SR < 0.45$

$$N = \text{unlimited} \quad (10)$$

where

SR = stress to strength ratio

N = number of allowable load repetitions

The asphalt concrete fatigue equations developed by the Asphalt Institute²⁰ were used as the design criterion for the joint loading case in the UTW design procedure. The asphalt concrete fatigue equation is:

$$N = C * 18.4 * (4.32 \times 10^{-3}) * (1/\epsilon)^{3.29} * (1/Ea)^{0.854} \quad (11)$$

where

N = number of load repetitions for 20% or greater AC fatigue cracking

ϵ = maximum tensile strain in the asphalt layer

Ea = asphalt mixture dynamic modulus, psi

C = a correction factor = 10^M

M = $4.84 * ((Vb / (Vv + Vb)) - 0.69)$

Vb = volume of asphalt, percent

Vv = volume of air voids, percent

For typical asphalt concrete mixtures, M would be equal to zero and the correction factor C would become one.

UTW PAVEMENT THICKNESS DESIGN PROCEDURE

Using equations 1-11, the design of a Ultra-thin Whitetopping can be determined. Separate fatigue analyses must be done for the concrete and asphalt layers. For a given set of parameters, one of the two analyses will govern and determine the required concrete thickness and joint spacing. The example problem given here illustrates the steps involved in the design procedure. The example represents design of a whitetopping project for a rural roadway where minimal truck traffic is expected.

Note: Equations 1-6 were developed for U.S. customary units; therefore the example presented here will use these units.

Design Procedure Parameters.

Parameters and material properties used in the design are the following:

Asphalt Modulus of Elasticity, E2	= 600,000 psi
Asphalt Thickness, h2	= 5 in.
Existing modulus of subgrade reaction, K	= 130 pci
Concrete Modulus of Elasticity, E1	= 4,000,000 psi
Concrete Modulus of Rupture, MR	= 650 psi
Temperature Differential, ΔT	= -8° F throughout the day
Coefficient of Thermal Expansion, α	= 5.5×10^{-6} in./in./°F
Trial Concrete Thickness	= 2.5 in.
Trial Joint Spacing	= 48 in.
Load Safety Factor, LSF	= 1.0

Procedure Steps.

- Using equation 7, determine l_e and L/l_e .

$$l_e = 24.245$$

$$L/l_e = 1.98$$

- Using equations 1 and 3, along with the modulus of subgrade reaction, compute the load induced critical asphalt strains and concrete stresses for 18-kip single axle loads (SAL).

$$JT_{18} = 315.5$$

$$COR_{18} = 287.0$$

Use proportions to calculate the critical asphalt strains and concrete stresses for the SAL load spectra. Put the results in columns 3 and 4 in the upper portion of Table 4a.

- Using equations 2 and 4, repeat step 2 for the anticipated tandem axle loads (TAL) and place the computed asphalt strains and concrete stresses in the lower portion of columns 3 and 4 in Table 4a.

$$JT_{36} = 262.6$$

$$COR_{36} = 419.3$$

- Using equations 5 and 6, compute the critical temperature induced asphalt strains and concrete stresses and place in Columns 5 and 6 of Table 4a, respectively. These values are not load related and, therefore, are equal for all loading conditions.

$$JT_T = 87.4$$

$$COR_T = 145.4$$

- Once the concrete stresses and asphalt strains have been computed for the loading and temperature conditions, they can be summed as the Total Stresses and Total Strains in columns 7 and 8, respectively.
- In Table 4b, compute the concrete stress ratio, SR, in Column 10, by dividing the total concrete stresses in Column 7 (Table 4a) by the design concrete modulus of rupture.
- Using the stress ratio and equations 8, 9, or 10, determine the allowable repetitions for the concrete layer in Column 11.
- Compute the percent fatigue in Column 12 by dividing Column 9 by Column 11, multiplying by 100. Total the concrete fatigue damage for both single and tandem axle loadings at the bottom.
- Enter the maximum tensile microstrain in the asphalt layer from Column 8 into Column 13 as shown.
- Using the existing asphalt modulus of elasticity and the microstrains in Column 13, compute the allowable load repetitions for the asphalt layer from equation 11 and enter these values into Column 14.
- Calculate the percent fatigue for the asphalt layer and the total asphalt fatigue damage in the same manner as the concrete fatigue computation in Step 9. Sum the percent fatigue for the given load cases to compute the total asphalt fatigue damage at the bottom.

Example Summary.

In this case, the concrete fatigue analysis governed. The stresses at the top extreme fiber of the concrete slab under corner loading cause greater damage than the strains at the bottom extreme fiber of the asphalt under joint loading conditions. For the existing asphalt and subgrade conditions, a concrete whitetopping thickness of 2.5 in. with a joint spacing of 48 in. is shown to be sufficient to carry the anticipated traffic loading.

SUMMARY

A mechanistic pavement design procedure for UTW pavement was developed through a comprehensive study involving extensive field load testing and theoretical analysis of UTW behavior using both 3-dimensional and 2-dimensional finite element models. Two types of pavement failure were considered in this procedure, portland cement concrete fatigue under corner loading and asphalt concrete fatigue under joint loading. Temperature induced stresses and strains were also included in the design. The design example presented in the previous section showed that this procedure gave reasonable results. However, the design procedure is viewed as the first generation design method. It will be refined as more field performance data (especially long-term performance data) become available.

From the field and the theoretical analyses, the following statements can be made:

1. A 3-dimensional finite element model was developed and was shown to model the UTW pavement behavior properly.
2. UTW pavements behaved as partially bonded systems, and should be designed accordingly.
3. Good interface bonding between concrete and asphalt layers is essential for successful UTW pavement performance.
4. Joint spacing is an important factor in determining UTW pavement performance. From field data, a maximum concrete slab joint spacing of 4 ft may be advisable.
5. The most commonly used asphalt pavement surface preparation (milling and cleaning) seemed to produce adequate interface bonding. However, long term interface performance can not be evaluated at this time because of the lack of data.
6. As illustrated in the design example, stresses at top of concrete under corner loading and temperature can be the governing design criterion.

Further Research:

There are several areas that need further research. Some of the more important are:

Stress Design Criteria for the bottom of Concrete:

Under certain conditions with thick asphalt bases and thicker concrete slabs, the critical load location may be in the concrete at the edge. Further investigation to develop stress prediction equations for the bottom of the concrete at the edge are needed.

Bonding Conditions:

All current equations are based on the bonding conditions at the Spirit of St. Louis Airport (36%). Other equations to predict concrete stress and asphalt strain for different bonding conditions need to be developed.

Surface Preparation:

What asphalt surface preparation techniques improve interface bonding. An earlier Iowa study²² showed that old asphalt pavements with milled and cleaned surfaces had interface direct shear strength in the range between about 130 and 220 psi. Comparing the results of this Iowa Study, the Spirit of St. Louis, and the Colorado sites, it quite apparent that many factors can affect bond.

Traffic Axle Load Distributions:

The axle load categories used in this design are from the PCAPAV design procedure and are approximately 15 years old. The axle load distributions may or may not be realistic for current traffic on the lower volume roads where UTW will most often be used.

TABLE 1. Ultrathin Whitetopping Project Field Evaluations

TABLE 2. Summary of Test Results on Cores - Spirit of St. Louis Airport

TABLE 3. Measured Strains of Slab D (5/6/1995) - St. Louis Airport

TABLE 4a. Design Example

TABLE 4b. Design Example

TABLE 1. Ultrathin Whitetopping Project Field Evaluations

Location	Construct Date	Concrete Thick (mm)	Asphalt Thick (mm)	Joint Spacing (m)	Traffic	Condition	Comments
Lavonia, Ga - DOT I-85 Truck Scales (Approach)	5/26/93	64	276	0.6 x 0.6	351,000 80-KN ESALS ¹⁶	2% Slabs cracked	Contains 3 lbs./cubic yard of polypropylene fibers. ¹⁰
Lavonia, Ga - DOT I-85 Truck Scales (Exit)	5/26/93	79	283	0.6 x 0.6	351,000 80-KN ESALS ¹⁶	5% Slabs cracked	No Fibers
Chattanooga, TN - S. Hickory & E. 28 th Street	7/28/92	67	280 of loose Asphalt Treated Base	1.14 x 1.26"	475,000 buses annually ¹⁷	18% Slabs replaced 68% Slabs cracked	Poor Performance attributed to substandard base material, which fell apart during coring.
Chattanooga, TN - Belvoir & Brainerd	11/10/93	76	NA	0.9 x 1.2	Heavily Trafficked	5% Slabs cracked	Unable to core due to traffic
Nashville, TN - N. 1 st St. & Woodland Ave.	5/2/92	76	Asphalt/ cobblestone	1.40 x 1.42	4,000,000 80-KN ESALS ¹⁷	Severe cracking and rutting in wheelpaths	Poor Performance attributed to insufficient asphalt section on cobblestone, traffic, and long joint spacing
Nashville, TN - Deaderick between 4 th & 5 th	10/27/94	152 121	140 101	1.2 x 1.2	Bus Stop Lanes	1% Slabs cracked	3 out of 5 slabs cracked are over steam line and cracked shortly after construction.

TABLE 2. Summary of Test Results on Cores - Spirit of St. Louis Airport

Sample No.	Interface bond strength, MPa		Cement concrete properties				Asphalt Concrete Properties						
			Elastic modulus, MPa	Compressive strength, MPa	Indirect tensile strength, MPa	Unit weight, kg/m^3	Resilient modulus, MPa, at 25 °C		5 °C		Indirect tensile strength at 25 °C, MPa	Unit weight, kg/m^3	
	Shear test	Pull off test					Instant	Total	Instant	Total			
	1B	0.774				4.63							
2B		0.549											
3B	0.796			32.69			5,545	5,131	9,793	9,517	0.920		
4B	0.708			30.55									2,210
5B		0.659											
6B		0.549											
1C	0.505			29.79			4,669	4,400	15,379	14,897	1.129		
2C													
3C	0.840												
4C	0.923						5,055	4,821	12,483	12,207	1.142		
5C		0.286			3.74								
6C		0.439											
1F	0.702					4.08							
2F		0.768											
3F	0.620			41.52									2,059
4F	0.549				4.52								2,162
5F	0.592			31.72			5,724	5,379	11,586	11,103	0.858		
6F		0.308											
1S			23103										
2S				33.66									
3S				32.62									
4S			23793										
Average	0.701	0.508	23,448	33.22	4.24	2,137	5,248	4932.759	12,310	11,931	1.012		2,147

TABLE 3 Measured Strains of Slab D (5/6/1995) - St. Louis Airport

Load Location	Gage location		Micro strain		
	Layer	No.	Test starting time		
			7:41 AM	11:01 AM	2:05 PM
1 (Edge) Left Rear Wheel N-S	Interface	1	18	25	27
	Concrete	1	6	9	9
	Surface	1	-40	-37	-36
2 (Center) Left Rear Wheel N-S	Interface	2	28	29	27
	Interface	3	19	19	14
	Concrete	2	11	16	15
	Concrete	3	12	13	13
	Surface	2	-45	-43	-40
	Surface	3	-39	-33	-31
3 (Edge) Left Rear Wheel N-S	Interface	4	26	19	14
	Concrete	4	19	21	23
	Surface	4	-67	-72	-68
3 (Edge) Left Rear Wheel W-E (B)	Interface	4	27	26	13
	Concrete	4	28	32	28
	Surface	4	-92	-89	-88
4 (Corner) Left Rear Wheel N-S	Interface	3	3	-14	-12
	Concrete	3	2	-8	-7
	Surface	3	-8	32	34
4 (Corner) Left Rear Wheel W-E (B)	Interface	3	-13	-12	-11
	Concrete	3	-2	-5	-8
	Surface	3	25	35	30

Note: N-S = truck traveled from north to south
W-E (B) = truck backed from west to east

TABLE 4a. Design Example

Axle Load, kips	Multiplied by LSF	Critical Concrete Stresses and Asphalt Strains					
		Load Induced		Temperature Induced		Total	
		Stress, psi	Microstrain	Stress, psi	Microstrain	Stress, psi	Microstrain
1	2	3	4	5	6	7	8

Single Axles

$$l_e = \frac{24.245}{1.98}$$

20	20	319	350	145	-87	464	263
18	18	287	315	145	-87	432	228
16	16	255	280	145	-87	401	193
14	14	223	245	145	-87	369	158
12	12	191	210	145	-87	337	123
10	10	159	175	145	-87	305	88
8	8	128	140	145	-87	273	53
6	6	96	105	145	-87	241	18
4	4	64	70	145	-87	209	-17
2	2	32	35	145	-87	177	-52

Tandem Axles

40	40	466	292	145	-87	611	204
36	36	419	263	145	-87	565	175
32	32	373	233	145	-87	518	146
28	28	326	204	145	-87	472	117
24	24	279	175	145	-87	425	88
20	20	233	146	145	-87	378	58
16	16	186	117	145	-87	332	29
12	12	140	88	145	-87	285	0
8	8	93	58	145	-87	238	-29
4	4	47	29	145	-87	192	-58

TABLE 4b. Design Example

Axle Load, kips	Expected Repetitions	Concrete Fatigue Analysis			Asphalt Fatigue Analysis		
		Concrete Stress Ratio	Allowable Repetitions, N	Fatigue Percent, %	Asphalt microstrain	Allowable Repetitions, N	Fatigue Percent, %
1	9	10	11	12	13	14	15
Single Axles							
20	20	0.714	1,285	1.6	263	433,317	0.0
18	95	0.665	5,028	1.9	228	693,593	0.0
16	1,750	0.616	19,677	8.9	193	1,201,069	0.1
14	15,000	0.567	77,010	19.5	158	2,322,532	0.6
12	50,000	0.518	349,038	14.3	123	5,303,079	0.9
10	150,000	0.469	5,606,775	2.7	88	16,008,692	0.9
8	500,000	0.420	unlimited	0.0	53	85,518,963	0.6
6	875,000	0.371	unlimited	0.0	18	unlimited	0.0
4	1,250,000	0.322	unlimited	0.0	-17	unlimited	0.0
2	1,750,000	0.273	unlimited	0.0	-52	unlimited	0.0
Tandem Axles							
40	0	0.940	2	0.0	204	995,390	0.0
36	0	0.869	18	0.0	175	1,652,475	0.0
32	5	0.797	129	3.9	146	3,009,420	0.0
28	50	0.725	946	5.3	117	6,267,278	0.0
24	175	0.654	6,942	2.5	88	16,133,602	0.0
20	1,500	0.582	50,937	2.9	59	61,133,561	0.0
16	70,000	0.510	475,457	14.7	30	594,696,502	0.0
12	500,000	0.439	unlimited	0.0	0	unlimited	0.0
8	950,000	0.367	unlimited	0.0	-29	unlimited	0.0
4	1,250,000	0.295	unlimited	0.0	-58	unlimited	0.0
Total Concrete Fatigue, % =				78.2	Total Asphalt Fatigue, % =		
					3.3		

Figure 1. Test Site and instrumentation layout for Spirit of St. Louis Airport.

Figure 2. Three dimensional finite element analysis slab system model

Figure 3. Induced Stresses vs. interface for center loading and joint loading with no cracks in asphalt.

Figure 4. Induced Stresses vs. interface for joint loading with one crack in asphalt.

Figure 5. Induced Stresses vs. interface for joint loading with four cracks in asphalt.

Figure 1. Test Site and instrumentation layout for Spirit of St. Louis Airport.

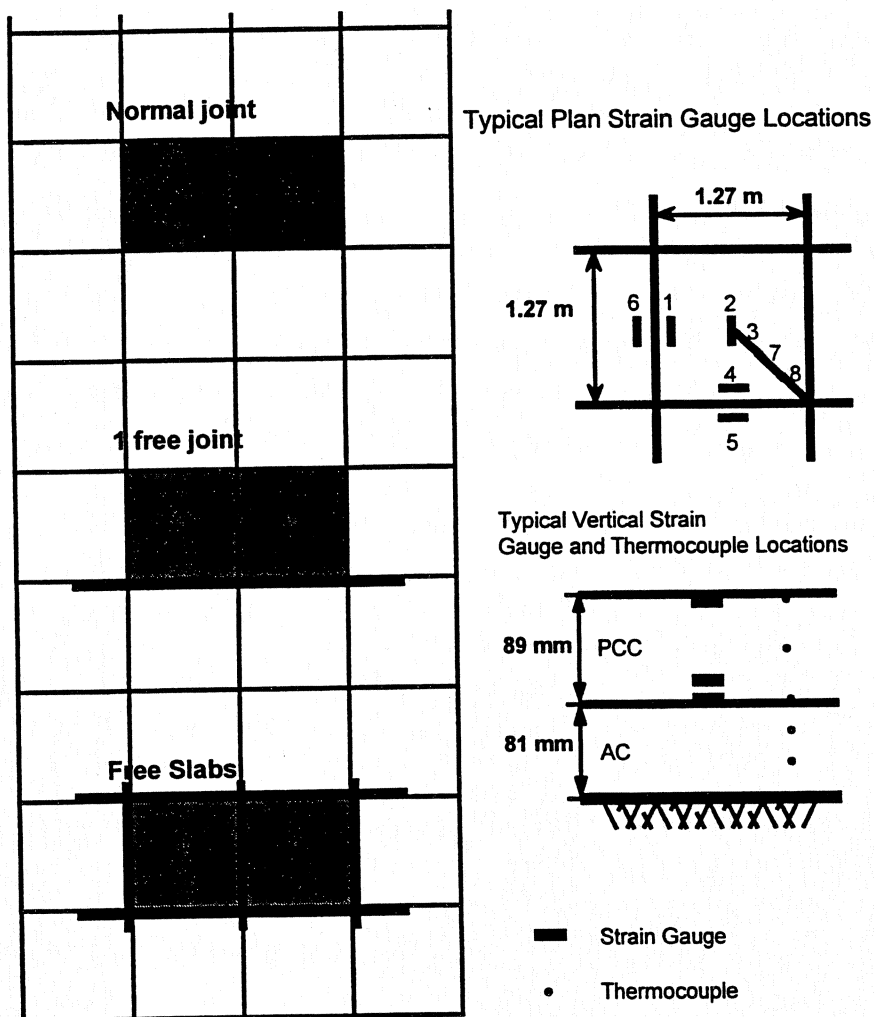
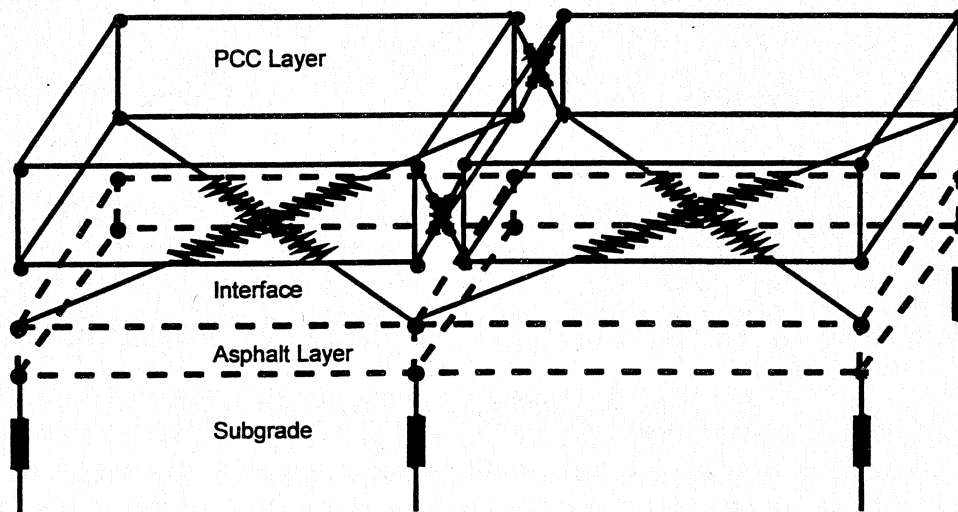


Figure 2. Three dimensional finite element analysis slab system model



Note: Compression gap elements are also used between asphalt layer and soil foundation enabling the separation caused by temperature.

Figure 3. Induced Stresses vs. interface for center loading and joint loading with no cracks in asphalt.

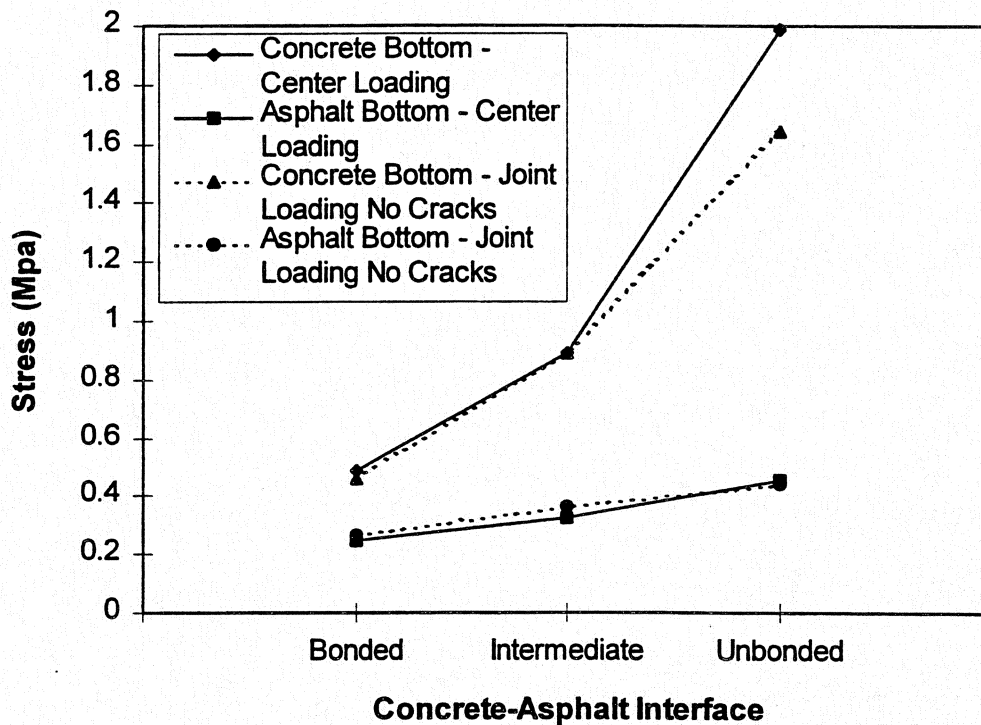


Figure 4. Induced Stresses vs. interface for joint loading with one crack in asphalt.

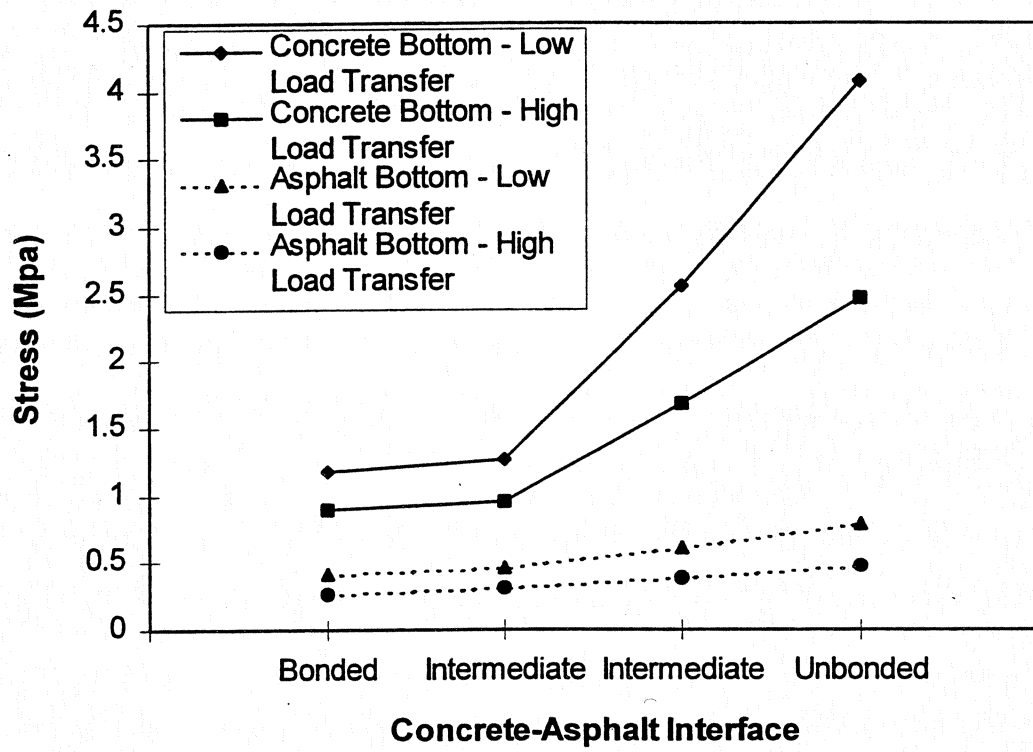
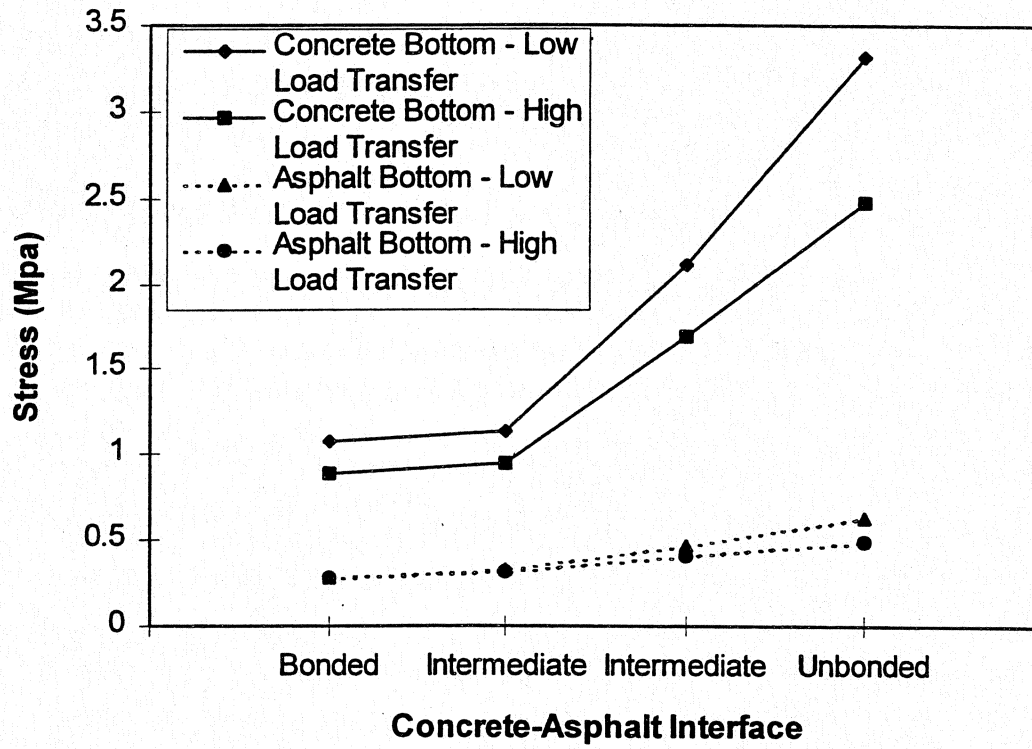


Figure 5. Induced Stresses vs. interface for joint loading with four cracks in asphalt.



REFERENCES

1. Wu, Chung-Lung, Tarr, S., Refai, T, Nagi, M, and Sheehan, M, "Development of Ultra-thin Whitetopping Design Procedure", PCA R&D Serial No. 2124, Portland Cement Association, Skokie, Illinois, 1997.
2. Hutchinson, R.L., "Resurfacing with Portland Cement Concrete," Synthesis of Highway, Practice, NCHRP 99, Transportation Research Board, 1982.
3. McGhee, Kenneth, H., "Portland Cement Concrete Resurfacing," Synthesis of Highway Practice, NCHRP 204, 1994
4. Cole, L.W., and Mohsen, J.P., "Ultra-Thin Concrete Overlays on Asphalt." Paper prepared for presentation at the 1993 TAC Annual Conference, Ottawa, Ontario.
5. Brown, D., "Ultra-Thin Whitetopping Emerges as Rehab Technique," Transportation Builder, V7, No. 1, Jan. 1995, pp 37-41.
6. Risser, R.J., LaHue, S.P., Volgt, G.F., and Mack, J., "Ultra-Thin Concrete Overlays on Existing Asphalt Pavement," 5th International Conference on Concrete Pavement Design and Rehabilitation, Vol. 2, April 1993, Purdue University, Lafayette, IN., pp. 247-254.
7. Mack, J.W., Cole, L.W., and Mohsen, J.P., "Analytical Considerations for Thin Concrete Overlays on Asphalt," Transportation Research Board, 72nd Annual Meeting, Washington, D.C., Jan., 1993, Paper No. 930752, pp.26.
8. Mack, J.W. "Ultra-Thin Whitetopping the World," Concrete Pavement Progress, Vol. 35, No. 2, American Concrete Pavement Association, Skokie, IL, 1996
9. Speakman, J., and Scott III, H., "Ultra-Thin, Fiber-Reinforced Concrete Overlays for Urban Intersections," TRR 1532, Advancements in Concrete Materials Technology, Transportation Research Board, Washington, DC, 1996, pp.15-20.
10. Cown, R.M., "Experimental Concrete Inlay on Existing Asphalt Pavement," Georgia Department of Transportation, Office of Materials and Research, Concrete Branch, Forest Park, Georgia, 1993
11. Tritsch, S., "Whitetopping, Technique Revives Burgeoning Kansas Thoroughfare," Roads and Bridges, Sept. 1995, pp. 52-55.
12. Packard, R.G., "UTW Proves its Worth in Worldwide Tests," Roads and Bridges, July 1996, pp. 15.
13. Slifwerbrand, J., "Whitetoppings-Swedish Field Tests 1993-1995," Swedish Cement and Concrete Research Institute, S-100 44 Stockholm, CBI Report, pp.57.
14. Petersson, O., and Silfwerbrand, J., "Thin Concrete Overlays on Asphalt Roads," 5th International Conference on Concrete Pavement Design and Rehabilitation, Vol. 2, April 1993, Purdue University, Lafayette, IN., pp. 241-246
15. Salcedo Guerrero, M. A. "Rehabilitación de Pavimentos Flexibles Mediante Losas Delgadas de Concreto, Resultados Experimentales," Cementos Mexicanos
16. Letter from Clair G. Baumgard to Ronald Collins documenting 2 year follow-up inspection of experimental concrete inlay, Georgia Department of Transportation, Office of Materials and Research, Concrete Branch, Forest Park, Georgia, May 23, 1995.
17. Speakman, J., and Scott III, H., "Ultra-Thin, Fiber-Reinforced Concrete Overlays for Urban Intersections," TRR 1532, Advancements in Concrete Materials Technology, Transportation Research Board, Washington, DC, 1996, pp.15-20.
18. NISA STATIC, NISA II Finite Element Application Programs, Engineering Mechanics Research Corporation (EMRC), Troy MI, 1992.
19. Packard, R.G., *Thickness Design for Concrete Highway and Street Pavements*, Portland Cement Association, Skokie, Illinois, 1984.

20. Khazanovich, Lev, "Structural Analysis of Multi-layered Concrete Pavement Systems," thesis presented to the University of Illinois as partial fulfillment of the requirements for the Degree Doctor of Philosophy, University of Illinois at Urbana-Champaign, 1994
21. *Thickness design - Asphalt Pavements for Highway and Streets*, Manual Series No. 1, (MS-1), Lexington, Kentucky, Asphalt Institute, 1991.
22. Grove, James D., Harris, G. K., and Skinner, B. J., "Bond Contribution to Whitetopping Performance on Low Volume Roads," Paper Presented at Transportation Research Board 72nd Annual Meeting, Washington, D. C., 1993.

ACKNOWLEDGEMENTS

The research reported in this paper was conducted at Construction Technology Laboratories, Inc., with the sponsorship of the Portland Cement Association (PCA Project Index No. 95-01). Additional field instrumentation and load testing conducted in Colorado was sponsored by PCA and Colorado Department of Transportation. The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented. The contents do not necessarily reflect the views of the Portland Cement Association or the Colorado Department of Transportation.