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RESEARCH ARTICLE

Feasibility of whitetopping pavements for resurfacing thin asphalt pavements

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Abstract: Many asphalt pavements have experienced rutting (wheel path deformation) while many others have experienced cracking. One of the possible solutions to this problem is the use of whitetopping, which is a composite section with a concrete layer over an existing asphalt pavement. Whitetopping 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.

This research attempts to study the feasibility of whitetopping for resurfacing of thin asphalt pavements (less than 100 mm) in tropical and sub-tropical countries. Whitetopping pavement test track constructed at the acceleration testing facility in a tropical climate, at the Florida Department of Transportation (FDOT), was used for the study. A three-dimensional (3-D) finite element model (FEM) was developed to simulate the field conditions using SAP2000 structural analysis software. The developed model was verified with field measured strain data. The verified analytical model was used to perform a parametric study in order to find the sensitivity and effects of loading conditions, joint spacing, joint conditions, whitetopping thickness, asphalt thickness, interface bond between asphalt and whitetopping, elastic modulus of concrete and asphalt, base and subgrade layers and temperature differentials in concrete. This study found that the developed model can be used to estimate the maximum anticipated stresses in whitetopping pavements.

The research revealed that ultra thin whitetopping pavement (less than 50 mm thick) is not suitable for resurfacing of thin asphalt pavements. A minimum concrete thickness of 60 mm and 80 mm is required for a 80 mm asphalt thickness with a joint spacing of 0.9 m for axle load controlled at 80 KN (18 kips) and 98 KN (22 kips), respectively. The parametric study revealed that the minimum thickness needs to be adjusted with other design parameters.

Keywords: Asphalt pavements, concrete overlays, conventional whitetopping, overlays, thin whitetopping, ultra-thin whitetopping.

INTRODUCTION

Better durability and long-term performance characteristics of concrete pavement surfaces can significantly reduce traffic delays associated with the frequent maintenance of the asphalt pavements. In addition, concrete surfaces are skid resistant and safety can be substantially improved, especially under wet conditions. In recent years, with the skyrocketing price of asphalt, concrete is becoming more competitive in cost compared with that of asphalt. Further, pavements at intersections, pedestrian crossings and approaches to them are more susceptible to distresses due to the effects of decelerating and accelerating of vehicles at those locations. Therefore, frequent maintenance and rehabilitation is required to correct such distress in hot mix asphalt (HMA) pavements, resulting in high maintenance costs and severe traffic delays. To overcome this problem, a long lasting pavement rehabilitation method must be adopted. The use of whitetopping is a more economically viable alternative for rehabilitation of asphalt pavements.

The concept of resurfacing the existing asphalt pavement using Portland cement concrete (whitetopping) is not new. In fact, the first reported use of whitetopping dates back to 1918 (Hutchinson, 1982). However, this technology has improved over the years with the improvement of the concrete paving technology.

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Plain concrete, reinforced concrete and fibrous (fiber-reinforced) concrete have been used to resurface flexible pavements (Hutchinson, 1982; McGhee, 1994). In the 1940's and 1950's, plain concrete was mainly used at the civil and military airports and the concrete thickness used in these projects ranged from 8 to 18 in. (200 to 460 mm). Since 1960, plain concrete has been extensively used to resurface existing highway pavements in states such as California, Utah, and Iowa in the USA. The concrete thickness of these resurfacing projects ranged from 7 to 10 in. (175–250 mm). Continuously the reinforced concrete and fiber-reinforced concrete were also used in a limited number of projects. NCHRP Synthesis 204 listed 189 whitetopping projects in the United States between 1918 and 1992. This list included street, highway, and airfield projects.

Ultra-thin whitetopping (UTW)

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

The first UTW experimental project was constructed on the access road to a waste disposal landfill in Louisville, Kentucky in September 1991(Cole & Mohsen, 1993; Risser *et al*., 1993). The concrete mixture was designed to provide a relatively high early compressive strength of 3,500 psi at 24 hours. A low water-cement ratio of 0.33 was selected to achieve a higher strength and to reduce drying shrinkage. Two concrete slab thicknesses; 2 in. (50 mm) and 3.5 in. (87.5 mm), and two joint spacings; 2 ft (0.6 m) and 6 ft (1.8 m) were used. The Louisville UTW pavement has performed well, carrying many more traffic loads than that were predicted by the design procedures available at that time. Following the success of the Louisville UTW project, many other states, including Tennessee, Georgia, North Carolina, Kansas, Iowa, Pennsylvania, New Jersey, Colorado, Missouri, Mississippi, Virginia and Florida, have constructed UTW projects and are currently evaluating these projects. Over 200 UTW pavements have been built in the last decade. The development of a mechanistic design procedure for UTW pavements in 1997 represented another major step in advancing this promising technique (Mack *et al*., 1997; Wu *et al*., 1997; ACPA, 1998).

Thin whitetopping (TWT)

Thin Whitetopping is a variation of the UTW method where thicker concrete slabs are used. The slab thicknesses in the range 5 to 7 in. are normal for this type of pavements. TWT may have a bonded or an unbonded interface between the concrete slab and the AC layer. While more attention has been paid to investigate the behaviour of UTW pavements, a few studies have focused on the alternative of using TWT when the conditions for the thinner slabs cannot be met. Among the states that have undertaken projects involving TWT are Colorado (Tarr *et al*., 1997), Minnesota (Vandenbossche & Fagerness, 2002) and Mississippi.

Conventional whitetopping (CWT)

Pavements with a slab thicknesss greater than 8 in. is commonly known as the conventional whitetopping (CWT). CWT pavements are generally used for pavements subject to heavier traffic loads, and have been designed based on the assumption that the existing asphalt concrete (AC) layer does not contribute directly to the load-carrying capacity of the pavement structure. Rather, the AC layer is considered to serve as a base layer for the new concrete overlay, and no bond is considered to exist between the overlay and the existing asphalt. A longer joint spacing [comparable to those of conventional jointed concrete pavements (JCP)] is generally incorporated in CWT.

Whitetopping construction procedures

UTW pavements are constructed with slipform or fixed form pavers in essentially the same way as the conventional concrete pavements with some special provisions. The construction procedures consist of the following steps: preparing asphalt surface, placing the concrete, finishing, surface texturing, curing, and sawing the joints. Asphalt pavement surface preparation prior to concrete placement is a very important procedure to achieve better bonds and good performance of UTW. Milling, followed by cleaning with compressed air to remove all laitance, dust, grit and all foreign materials, is the best way to prepare the asphalt surface. It is recommended that an adequate asphalt thickness of a minimum of 3 in. (75 mm) after milling, be used, if possible (Mack *et al*., 1997).

The concrete used in UTW can be produced at a ready-mix plant and delivered to the site by ready-mix trucks. Normal slipform pavers can be used to spread, screed, and consolidate the concrete in an efficient manner. After the surface is finished and textured, a curing compound is immediately sprayed on the entire surface to achieve adequate curing. The curing compound is generally applied at a rate twice the normal application rate for thicker concrete pavements because thin concrete slabs can lose water rapidly (ACPA, 1998). Joint sawing must be performed as soon as the surface conditions permit, or when the concrete is able to support the equipment and the operator. Usually joints are not sealed because joint openings are generally narrow due to the short joint spacing.

There are several advantages of using whitetopping, a thin layer of concrete on asphalt, for rehabilitating asphalt pavements (Hutchinson, 1982; Rasser *et al*., 1993; Mack *et al*., 1997). Whitetopping provides long-term benefits to the travelling public, and to the roadway or airport agencies. The use of a durable concrete with good long-term performance characteristics can decrease the required maintenance and life cycle costs of pavements. As a result, concrete surfaces can significantly reduce traffic delays associated with the frequent maintenance of asphalt pavements. In addition, when concrete surfaces are used, there is a substantial improvement in skid resistance and safety, especially under wet conditions. These advantages promote and contribute to the use of concrete pavements over asphalt surfaces. Thin asphalt overlays usually correct only functional distress, but whitetopping overlays correct both functional and structural failures (Rasser *et al*., 1993).

Hot mix asphalt (HMA) pavements with surface problems are good candidates for thin whitetopping (TWT), which has a 50–100 mm concrete layer, while HMA pavements with rutting problems are good candidates for TWT and ultra thin whitetopping (UTW), which has 50 mm or less concrete thickness. TWT and UTW provide the functional improvement and increase the structural capacity of HMA pavements without much change in the profile to handle increased traffic loads (Chunhua, 2005).

The main objectives of this research are:

(1) to develop analytical models for analysis of the behaviour of whitetopping pavements.
(These models are to be verified and fine-tuned by full-scale experimental results).
(2) to evaluate the potential performance of the whitetopping pavements using a parametric analysis.
(3) to assess the applicability of whitetopping techniques for rehabilitation of thin asphalt pavements.

METHODS AND MATERIALS

Instrumentation of test slabs

The FEACONS programme was used to analyze the stresses in the test slabs when subjected to a 53-kN (12-kip) single wheel load with a tire pressure of 827 kPa (120 psi) and a contact area of 645 cm^2 (100 in^2), and applied along the edge of the slab, which represents the most critical loading location. Similar to the case of bonded interface, the analysis was performed for two different load positions at the corner of the slab and at the middle of the edge for the same temperature differentials in the concrete slabs. No load transfer at the joints was assumed in the analysis, which represents the worst condition. With the results from the stress analysis, it was possible to identify the locations where the maximum stresses and strains in the test slab would occur so that strain gauges could be placed to monitor these maximum induced strains.

Figure 1 shows the instrumentation layout for the 1800 mm × 1800 mm test section, which was placed on lane 6 of the accelerated pavement testing (APT) area. Three locations (location 1, 2 and 3) were identified to have the maximum anticipated strains due to the heavy vehicle simulator (HVS) load. Thus, strain gauges were placed at these three locations. While location 1 had two gauges; at the top and the bottom, the other two locations had only one. At location 1, one embedded strain gauge was placed at a depth of 25 mm (1 in.) from the concrete surface, while the other embedded strain gauge was placed 12.5 mm (0.5 in.) from the bottom of the concrete

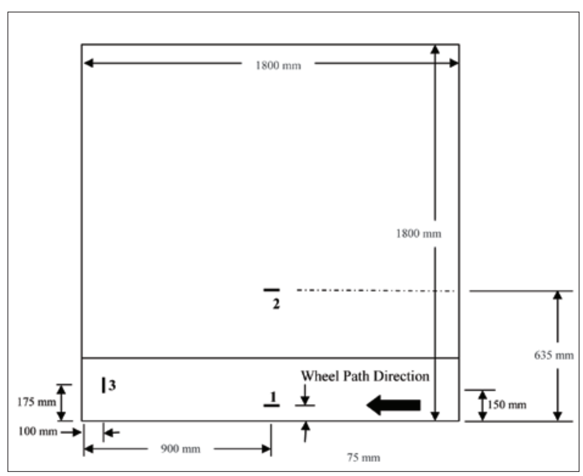


Figure 1: Strain gauge locations

layer. Location 2 had a strain gauge embedded 1 in. from the surface of the concrete slab and location 3 had a strain gauge embedded 0.5 in. from the bottom of the concrete slab. Two surface gauges located next to the transversal joint at the middle of the slab were also used to monitor any micro cracks that may occur in the concrete surface. These surface gauges located in the adjacent slabs were also used to evaluate the load transfer at the joints. Figure 1 shows the locations of thermocouples used to monitor the temperature in the slab. Two positions were considered for the thermocouples, one at the centre of the slab and the other in the corner. They were placed 25 mm apart along the depth of the slab with the first starting at 25 mm from the surface. An additional thermocouple was placed on the surface of the AC layer to monitor the daily variation of temperature in the asphalt layer.

Data collection

For each test slab, the strain gauges were connected to a strain indicator unit, Vishay System 6000 (Model 6100) for strain reading and data acquisition. This system has the ability to take individual strain readings at a very high frequency and 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 s at 1 h intervals. The rate of data collection was 100 strain values per second. This rate allowed the capture of the progression of the strain and to especially observe the strain reversal phenomenon. Strain gauge readings due to a static wheel load were also taken for two wheel loading positions, namely, corner (pt 1), and mid-edge (pt 2). The static readings were recorded while the wheel was traveling at slow speed towards the static loading position and while it stayed at the two load positions (pt 1 and pt 2) for 20 s each. All the thermocouples were connected to the same data acquisition system. The temperature data were collected during the entire day at 5-min intervals. In both phases, data collection for temperature was started before the loading period.

ANALYSIS

3-D finite element model development

Experimental investigations were performed in the accelerated pavement testing facility at the Florida Department of Transportation (FDOT). The strain data were observed at several gauge locations under a dynamic loading condition. Field strain data from dynamic loading conditions were used to verify a finite-element model, which uses static loading conditions with the parameters listed below (Appea *et al*., 2005; Kumara, 2005; Tapia *et al*., 2007; Tia *et al*., 2007; Tapia *et al*., 2008). Parameters 1- 4 were assigned based on the field conditions and the other parameters were assigned based on the laboratory test results and theoretical investigations of previous studies at the same test location (Appea *et al*., 2005; Kumara, 2005; Tapia *et al*., 2007; Tia *et al*., 2007; Tapia *et al*., 2008).

1. Whitetopping joint spacing – 1800 mm
2. Thickness of whitetopping layer – 100 mm
3. Thickness of asphalt concrete layer – 112 mm
4. Thickness of lime rock base – 300 mm
5. Thickness of subgrade – 2000 mm
6. Elastic modulus of concrete – 30 GPa
7. Elastic modulus of asphalt – 3 GPa
8. Elastic modulus of lime rock base – 0.3 GPa
9. Elastic modulus of subgrade – 0.05 GPa

The SAP2000 nonlinear 8.1.2 structural analysis programme was used to develop a three dimensional finite-element model with eight node hexagonal finite elements.The programme can perform both static and dynamic analyses of structures (SAP2000, 2002). To obtain accurate results, the aspect ratio (the ratio of the longest dimension of 3-D element to its shortest dimension) should not exceed ten (Risser *et al*., 1993). The asphalt concrete layer was bonded to the lime rock base layer, which was on top of a subgrade material. The material properties were assigned based on the results of laboratory and field tests and considered as linear elastic. The static load was applied along a wheel path while strains at various gauge locations were measured in the field experiment. Comparison was carried out between the measured and computed strains. A fully and partially bonded condition was adopted for whitetopping- asphalt concrete interface while considering the full load transfer condition for whitetopping joints.

Figures 2 and 3 show the comparison of measured and computed strains at the two gauge locations. The computed strains from the analytical model were compared with the measured strains from the field tests. There is a considerable variance in compressive strains at gauge location 1. However, software strains are always higher than field strains, which is more conservative and also it should be noted that the field load was a dynamic load while a static load was used in the analysis by the 3-D finite element model. It was found in a previous study that static strains are higher than dynamic strains (Kumara, 2005). Figure 4 shows the distribution of field strain and model strain data at all the locations. It can be seen that there is a strong linear correlation between

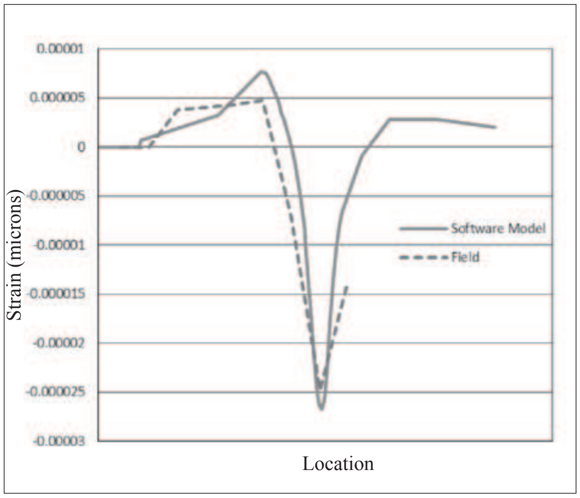


Figure 2: Strains induced at gauge location 1 (top)

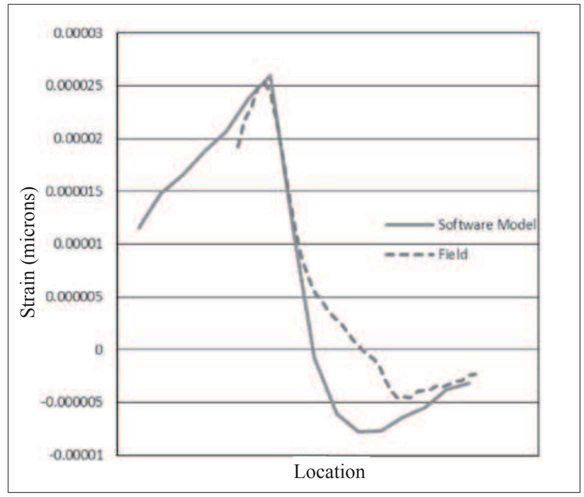


Figure 3: Verification of strains at gauge location 2 (bottom)

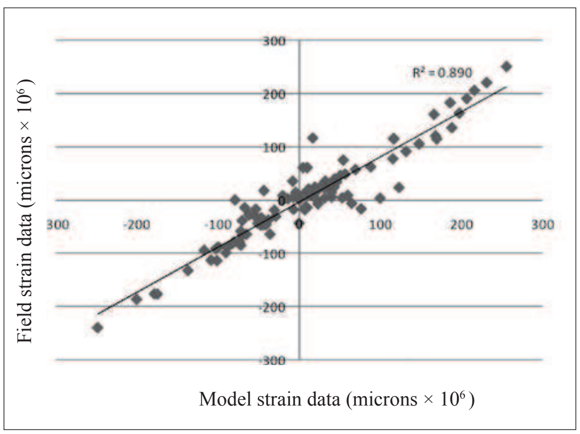


Figure 4: Correlation between model strain data and the field strain data

the model strain data and the field strain data. The other important fact is that the analytical models did not consider the effect of minor cracks and local defects in the concrete, which can affect the actual measured strains.

Parametric study

A comprehensive parametric analysis was performed by using the developed 3-D finite element model in order to find the optimum dimensions of a whitetopping, and to investigate the applicability of whitetopping for intersections and raised pedestrian crossings in Sri Lanka. Asphalt core samples collected from roads in Colombo metropolitan area revealed that usable thickness of remaining asphalt (removing surface distresses) has been less than 100 mm and therefore the parametric analysis were focused on HMA roads with less than 4 inches of asphalt concrete. The effects of the parameters listed below were studied. The layer coefficients (elastic modulus and poisson ratios) assigned for the subgrade, base and other layers in the parametric analysis were the same as those in the verified software model.

- Loading conditions
- Whitetopping thickness
- Joint spacing
- Joint condition
- Asphalt thickness
- Interface bond between asphalt and whitetopping layers
- Elastic modulus of whitetopping, asphalt concrete, base and subgrade layer
- Temperature effect

Since the stresses causing cracking in concrete pavements are flexural rather than compressive, flexural strength is used in the concrete pavement design.

Determination of design flexural strength: The following approximate relationship between flexural and compressive strength was used in the study.

$$f_r = C\sqrt{f_{cr}'} \quad ...(1)$$

f_r - Flexural strength (modulus of rupture) (MPa)

C - Constant- 0.75 (metric)

f_{cr}' - Compressive strength (MPa)

Compressive strength - 30 MPa (Laboratory test results for concrete mix)

Flexural strength = f_r = 0.75 x $(30)^{0.5}$ = 4.10 MPa

The stress to flexural strength ratio can be used to evaluate the potential performance of concrete pavements (Nevill & Brooks, 2004). Maximum principal stresses were considered in the analysis. Shear stresses at asphalt and whitetopping interface were also analysed. The interface shear stress should be lower than the bond strength to avoid bond failure in thin whitetopping pavements. The interface bond strength estimation was done through the Iowa shear test, which was performed at the Florida Department of Transportation laboratories. The test results indicated that interface bond strength is about 1.40 MPa (Kumara, 2005) and was considered as the bond strength in this study. In a thin whitetopping pavement, it is desirable to have a stress/ strength ratio of less than 0.5 and a shear stress at interface of less than the bond strength to avoid fatigue failure and bond failure, respectively. The American Society for Testing and Materials (ASTM) defines fatigue strength as the value of stress at which failure occurs after a number of load cycles, and fatigue limit as the limiting value of stress at which failure occurs as fatigue strength becomes very large. ASTM does not define endurance limit but implies that it is similar to fatigue limit. However, it has been revealed from a previous study that the typical value of the fatigue limit is 0.5 of the tensile strength. Therefore, based on fatigue criteria, flexural stresses must be maintained lower than 2.05 MPa (0.5 x 4.10 Mpa).

Effect of loading position: Wheel load can be applied at different positions in a slab. Three different loading positions were chosen to find out the most susceptible places for loading, namely, the centre, mid edge and corner. The results show that the corner loading induces the highest stresses in a concrete slab.

Effect of asphalt-whitetopping interface bond: Obtaining a good bond between the two layers is essential for long-term performance of thin and ultra-thin whitetopping. Bonding the two layers increases curling/ warping stresses, which can be partly offset by using a smaller joint spacing, but it significantly decreases the stresses from the combined effect of curling and applied load. Principles of superposition can be used to quantify the combined effects of curling and applied load once it has been determined that the two layers are bonded. This assumption significantly underestimates the stresses if the overlay is only partially bonded. For this reason, it is extremely important to characterize accurately the bond between the two layers. The maximum principal stress variations at different bonding conditions were analyzed. The partially bonded condition was modelled by placing an interface layer between the asphalt and whitetopping layers with a very low elastic modulus. It is clearly seen that stresses have considerably increased when the interface bond is weak. Furthermore, the standard specification for construction of the whitetopping pavement should place primary emphasis on achieving maximum bond between the whitetopping and asphalt surface.

Effect of temperature: During day time, the temperature at the top of the whitetopping pavement is higher than the bottom and *vice versa* at night time. Therefore, the slab starts to curl and induce tensile stress in the area above the neutral axis at day time and below that at night.

Figure 5 shows the stress variation with temperature differential. A positive temperature differential (top temperature higher than bottom temperature) induces higher stress than that is induced by a negative temperature differential (bottom temperature is higher than top temperature).

Figure 6 shows the combined effect of the temperature and vehicle loading position. It can be inferred from the figure that corner loading is critical during night time, and center load during the day time. It also shows the importance of considering the effects of temperature

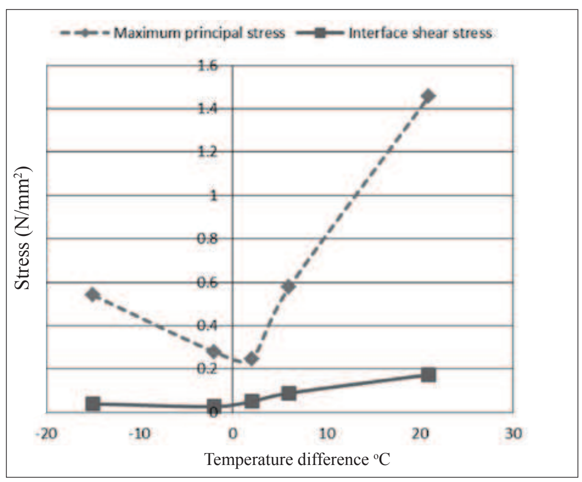


Figure 5: Stresses variation with temperature difference

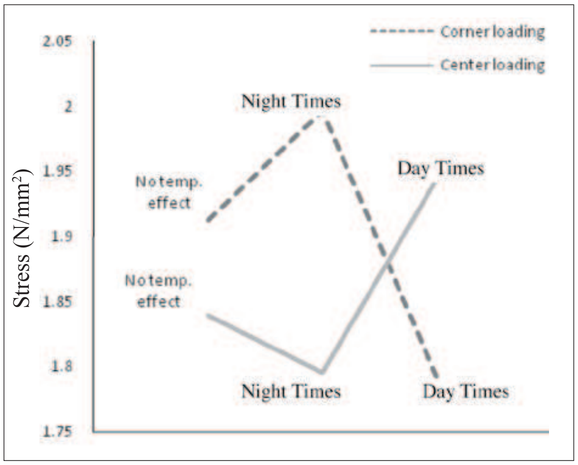


Figure 6: Stresses variation with temperature and loading position

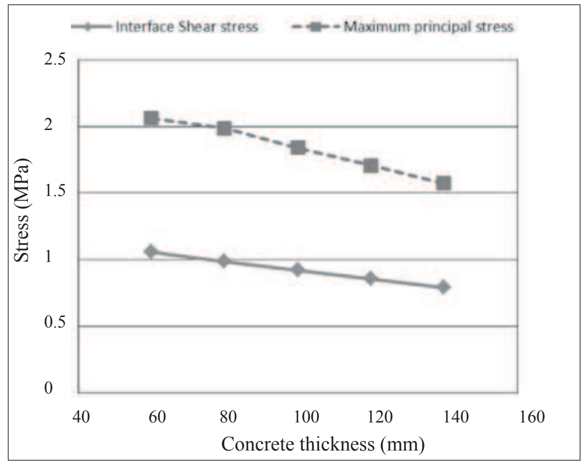
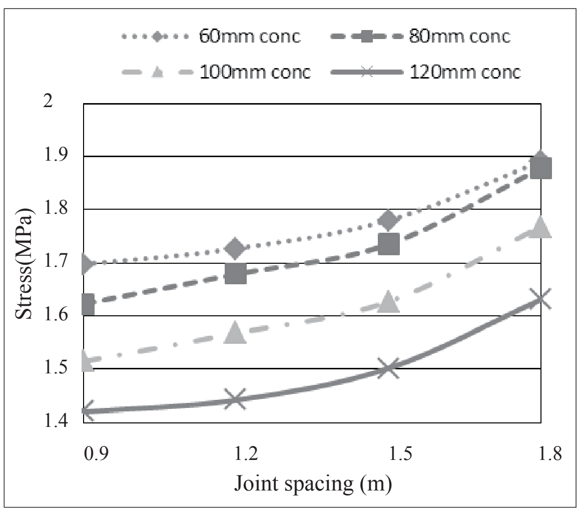
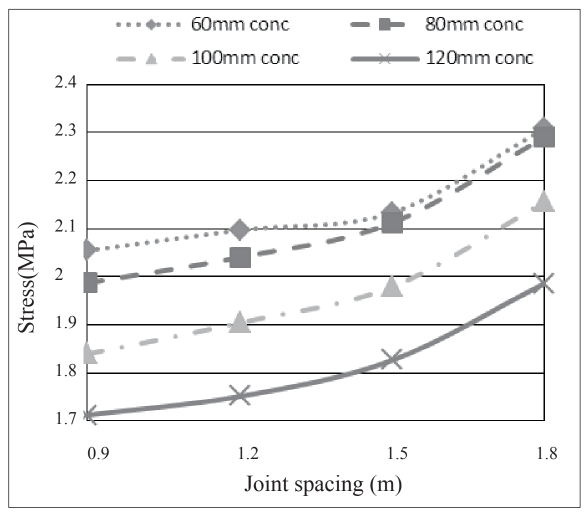


Figure 7: Stress variation with concrete thickness



a.) Load case 1



b.) Load case 2

Figure 8: Maximum principal stress variation with joint spacing

differentials in the analysis of thin whitetopping pavements.

Effect of whitetopping layer thickness: The dimensions of slabs have an important role for stresses in concrete. Field experiments in Mexico showed that deformations on concrete for square slabs with 1.20 m width and 90 mm thickness were 73 % higher than the measured values for a slab with 0.90 m width and 65 mm thickness (Balbo & Radolfo, 1992).

Whitetopping thickness was varied from 60 mm to 120 mm while observing the stresses induced due to corner loading, which is the critical loading condition. Figure 7 indicates that increasing the thickness of whitetopping substantially reduces flexural stresses. According to a previous study, the interface bond strength requirement of 1.40 MPa has to be maintained to preserve the bond between the asphalt and whitetopping layers (Tia *et al.*, 2007). As shown in Figure 7, interface shear stresses do not exceed the interface bond strength.

Effect of joint spacing of whitetopping layer: Two loading cases were considered in this analysis as standard axle load (80 kN) and the overloading limit (98 kN). Figures 8a and 8b show the effect of joint spacing on corner loading for load case I [40 kN (9 kips)] where the axle load is 80 kN and load case II [49 kN (11 kips)] where the axle load is 98 kN, respectively. Joint spacing was varied from 0.9 m to 1.8 m for a selected asphalt thickness of 80 mm.

It can be clearly seen in Figures 8a and 8b that flexural stresses increase when joint spacing increases. For the 49 kN (11 kips) loading (load case II), the thickness of whitetopping should be 80 mm or above while maintaining a joint spacing of 0.9 m. All the above results are applicable for asphalt concrete thickness of 80 mm. For higher asphalt concrete thicknesses, the whitetopping thickness can be further reduced while maintaining a longer joint spacing.

Effect of asphalt layer thickness: After milling and surface preparation, the final asphalt concrete thickness might be less than 100 mm in most cases. For a 100 mm thick whitetopping layer with a joint spacing of 0.9 m, the asphalt concrete layer thickness was varied from 60 mm to 120 mm to represent realistic asphalt concrete thicknesses.

Figure 9 shows that there is a significant effect of asphalt concrete thickness on the flexural stresses of the whitetopping layer. Flexural stresses for the overload limit of 49 kN load have exceeded the flexural strength

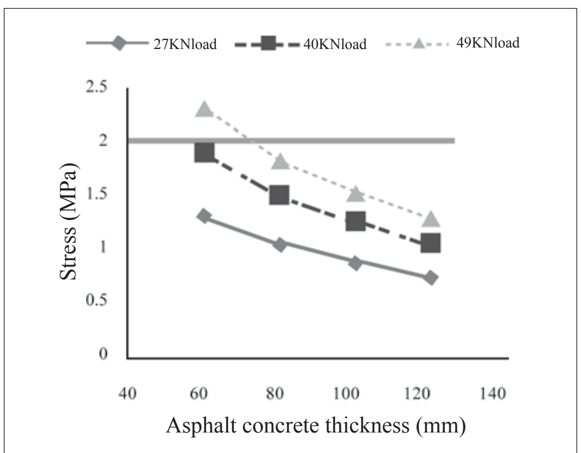


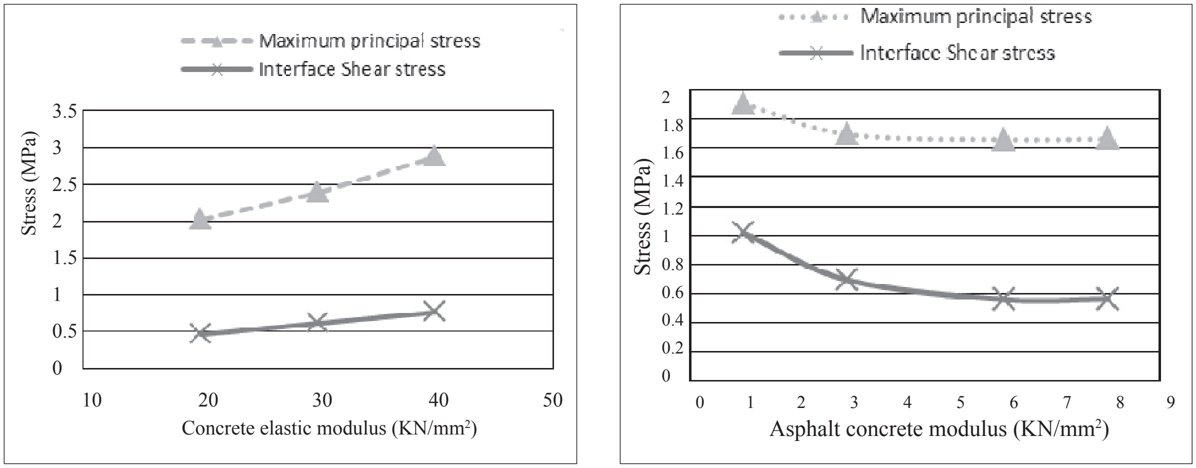
Figure 9: Maximum principal stress variation with asphalt concrete thickness

(2.05 MPa as drawn in the graph) when the thickness is less than 80 mm and the 40 kN load flexural stresses are very close to the flexural strength. Therefore, it can be identified that for higher loads such as 49 kN (11 kips), asphalt thicknesses must be higher than 80 mm, in order to construct a 100 mm thick whitetopping on the existing asphalt pavement. To construct a whitetopping with a thickness lower than 100 mm, the existing asphalt concrete thickness must be higher than 100 mm.

Effect of length to width ratio: The length to width ratio of the whitetopping layer was varied from 1.0 to 2.0 in order to find the optimum length to width ratio for 0.9 m x 0.9 m slabs. The results showed that the optimum length to width ratio is 1.30 for whitetopping pavements. However, the stress variation is small with other panel sizes. A previous study recommended that the ratio of the length (longest dimension) to width (shortest dimension) of any given panel be not more than 1.5, although some recommend a maximum ratio of 1.25 (ACPA, 1998). It should be noted that large number of joints are typically specified for a UTW or TWT. Therefore, the cost of the additional sawing might be weighed against the cost of a thicker PCC section, which would require less sawing.

Effect of whitetopping elastic modulus: Under the 49 kN(11 kips) load, the maximum principal stress and interface shear stress of the whitetopping layer were observed by varying the elastic modulus of whitetopping for a pavement with a 100 mm whitetopping layer, 80 mm asphalt and 0.9 m joint spacing. Figure 10a shows that the maximum principal stress and interface shear stress increase significantly as the elastic modulus of whitetopping layer increases. Therefore, a good bond between whitetopping and asphalt concrete is required to maintain the composite action.

Effect of asphalt concrete elastic modulus: Under the 49 kN (11 kips) load, the maximum principal stress and interface shear stresses of the whitetopping layer were observed by varying the modulus of asphalt concrete for a pavement with 100 mm whitetopping layer, 80 mm asphalt layer and 0.9 m joint spacing. Figure 10b shows that the maximum principal stress and maximum interface shear stress of the whitetopping layer reduce with the increase of the asphalt concrete elastic modulus. It can be identified from Figure 10b that there should be a sufficient elastic modulus in the existing asphalt layer to mobilize the composite action in whitetopping pavements. A deteriorated pavement with a substantially cracked surface would not be a good candidate for whitetopping.



a) Concrete elastic modulus

b) Asphalt elastic modulus

Figure 10: Stress variation in whitetopping layer with elastic modulus of material

Effect of base and subgrade elastic modulus: The effect of the variation of base layer elastic modulus on stresses of whitetopping layer was examined under the same conditions defined in the previous section. The results showed that the maximum principal stress and the maximum interface shear stress of the whitetopping layer reduced as the elastic modulus of the base layer increases. The maximum principal stress increases rapidly when the base strength is reduced beyond 0.25 x10^3 MPa. So the minimum elastic modulus of the base should be greater than 0.25 x10^3 MPa

The Effect of the subgrade layer elastic modulus on stresses of whitetopping layer was examined under the same conditions defined in the previous section. The maximum principal stress and the maximum interface shear stress of the whitetopping layer reduces as the elastic modulus of subgrade layer increases. However the effect of subgrade layer elastic modulus is not significant for stresses in the whitetopping layer.

CONCLUSION

The maximum stresses in the concrete were observed to increase significantly; as the asphalt thickness and concrete thickness decreases, the concrete stiffness increases, the asphalt stiffness decreases, the base stiffness decreases and the concrete panel size increases. However, the effect of subgrade elastic modulus is insignificant for stresses in the whitetopping layer. The analysis was performed for grade 30 concrete with flexural strength not less than 4.0 MPa. For a 40 kN loading (80 kN axle load), the minimum whitetopping thickness can be 60 mm while maintaining the joint spacing less than 1.5 m.

For the 49 kN (98 kN axle load) loading, the thickness of whitetopping should be not less than 100 mm while maintaining the joint spacing at 0.9 m. All the above conditions are applicable where the asphalt concrete thickness is 80 mm. For higher asphalt concrete thicknesses, the whitetopping thickness can be further reduced while maintaining a longer joint spacing.

It can be identified that for higher loads (49 kN), the asphalt thickness must be higher than 80 mm in order to construct a 80 mm thick whitetopping on top of the existing asphalt pavement. To construct a whitetopping with a lower thickness than 80 mm, the existing asphalt concrete thickness must be greater than 100 mm.

The temperature effect is significant on stresses in concrete and the combined effect of temperature differential and the loading position (corner at night time and centre at daytime) should be considered to estimate the maximum stresses. It is required to estimate the highest temperature differential possible in pavement for better estimation.

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