PERFORMANCE OF ULTRA-THIN WHITETOPPING (UTW) IN OKLAHOMA

FINAL REPORT ~ FHWA-OK-10-05
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BY

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16 . ABSTRACT

With the current level of deterioration of pavements in Oklahoma and the United States a satisfactory repair technique that is economical and can be applied rapidly while resisting a significant volume of traffic is becoming important. Thin concrete overlays have been used in increasing numbers over hot-mix asphalt (HMA) pavements and at intersections as a rapid and economical method of repair. These repairs have shown outstanding service in the state of Oklahoma with service lives over 10 years when used in areas with moderate truck traffic. These overlays are commonly referred to as white toppings as the overlay material is much lighter than the asphalt it is overlaying.

This report is organized in three major sections. In section 2 the current condition of whitetopping projects is reviewed in Oklahoma. The inspection of these projects was primarily done with visual inspection, but some work was done with cores from the projects and also with Falling Weight Deflectrometer (FWD) measurements. In section 3 a review of the different whitetopping design methodologies is presented. In section 4 specific unanswered questions over whitetoppings are covered that the Oklahoma DOT felt were important to address from the existing literature.

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SI (METRIC) CONVERSION FACTORS

Approximate Conversions to SI Units					***Approximate Conversions from SI Units***				
Symbol	**When you know**	**Multiply by**	**To Find**	**Symbol**	**Symbol**	**When you know**	**Multiply by**	**To Find**	**Symbol**
		LENGTH					***LENGTH***		
in	*inches*	*25.40*	*millimeters*	*mm*	*mm*	*millimeters*	*0.0394*	*inches*	*in*
ft	*feet*	*0.3048*	*meters*	*m*	*m*	*meters*	*3.281*	*feet*	*ft*
yd	*yards*	*0.9144*	*meters*	*m*	*m*	*meters*	*1.094*	*yards*	*yds*
mi	*miles*	*1.609*	*kilometers*	*km*	*km*	*kilometers*	*0.6214*	*miles*	*mi*
		AREA					***AREA***		
in^2	*square inches*	*645.2*	*square millimeters*	mm^2	mm^2	*square millimeters*	*0.00155*	*square inches*	in^2
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yd^2	*square yards*	*0.8361*	*square meters*	m^2	m^2	*square meters*	*1.196*	*square yards*	yd^2
ac	*acres*	*0.4047*	*hectacres*	*ha*	*ha*	*hectacres*	*2.471*	*acres*	*ac*
mi^2	*square miles*	*2.590*	*square kilometers*	km^2	km^2	*square kilometers*	*0.3861*	*square miles*	mi^2
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ft^3	*cubic feet*	*0.0283*	*cubic meters*	m^3	m^3	*cubic meters*	*35.315*	*cubic feet*	ft^3
yd^3	*cubic yards*	*0.7645*	*cubic meters*	m^3	m^3	*cubic meters*	*1.308*	*cubic yards*	yd^3
		MASS					***MASS***		
oz	*ounces*	*28.35*	*grams*	*g*	*g*	*grams*	*0.0353*	*ounces*	*oz*
lb	*pounds*	*0.4536*	*kilograms*	*kg*	*kg*	*kilograms*	*2.205*	*pounds*	*lb*
T	*short tons (2000 lb)*	*0.907*	*megagrams*	*Mg*	*Mg*	*megagrams*	*1.1023*	*short tons (2000 lb)*	*T*
		TEMPERATURE (exact)					***TEMPERATURE (exact)***		
°F	*degrees*	*(°F-32)/1.8*	*degrees*	*°C*	*°C*	*degrees*	*9/5(°C)+32*	*degrees*	*°F*
	Fahrenheit		*Celsius*			*Fahrenheit*		*Celsius*	
		FORCE and PRESSURE or STRESS					***FORCE and PRESSURE or STRESS***		
lbf	*poundforce*	*4.448*	*Newtons*	*N*	*N*	*Newtons*	*0.2248*	*poundforce*	*lbf*
lbf/in^2	*poundforce per square inch*	*6.895*	*kilopascals*	*kPa*	*kPa*	*kilopascals*	*0.1450*	*poundforce per square inch*	lbf/in^2

Performance of Ultra-Thin Whitetopping (UTW) in Oklahoma

FINAL REPORT

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1 Introduction

With the current level of deterioration of pavements in Oklahoma and the United States a satisfactory repair technique that is economical and can be applied rapidly while resisting a significant volume of traffic is becoming important. Thin concrete overlays have been used in increasing numbers over hot-mix asphalt (HMA) pavements and at intersections as a rapid and economical method of repair. These repairs have shown outstanding service in the state of Oklahoma with service lives over 10 years when used in areas with moderate truck traffic. These overlays are commonly referred to as white toppings as the overlay material is much lighter than the asphalt it is overlaying.

The depth of the concrete overlay depends on the traffic demands, expected bond between the overlay and the HMA and the condition of the HMA pavement. The classification of these overlays depend on the thickness of the concrete overlay. Overlays that are between 2" and 4" are called "ultra thin whitetoppings" (UTWs). Overlays that are between 4" and 8" are called "thin whitetoppings" (TWs) and overlays thicker than this are just whitetoppings. While there is a difference in the naming of these overlays the behavior and design for the different whitetoppings are the same. Because of this all of these overlays will be treated the same and just referred to generically in this document as whitetoppings.

The behavior of these bonded overlays and the importance of a high quality bond between the materials is shown in Fig. 1.1. In this illustration the bending stresses in a pavement is shown with and without bonding. The UTW with no bond experiences a high level of compression at the surface and a high level of tension at the interface between the HMA and the overlay. If a high quality bond is provided between the overlay and the HMA then a much more beneficial stress distribution is obtained as the compressive stresses and the tension forces in the overlay are reduced to very low levels. This low level of tension in the overlay helps prolong the long term performance of UTW which will reduce the need for repairs.

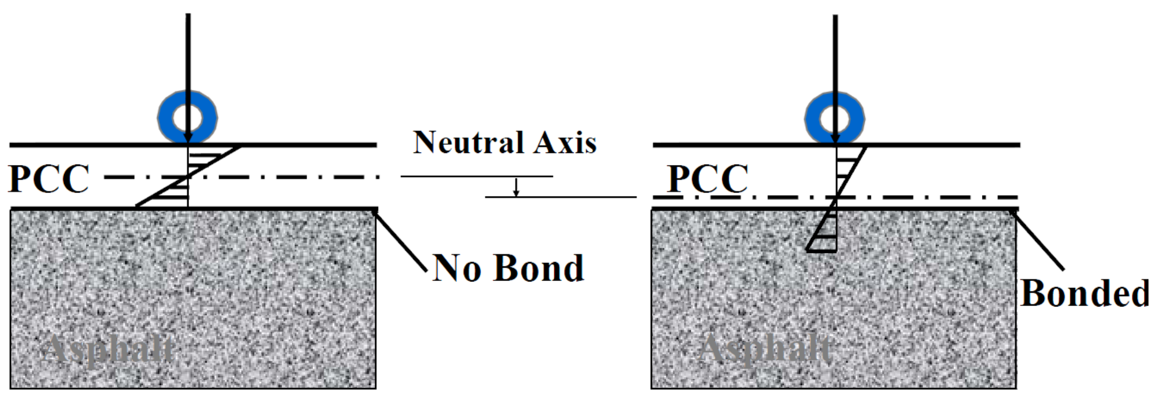


Figure 1.1 – Two UTW pavements are shown with a poor and high quality bond between the underlying HMA. (after Won, 2007)

One major concern with the construction of UTW is that the overlay has a large surface area to volume ratio. This creates unique problems for curing, shrinkage and the timing of when saw-joints are made.

To combat this it is often necessary to use a low shrinkage concrete mixture as well as plastic fibers to prevent plastic shrinkage cracking. Also the correct timing for the sawing of the control joints is important. Furthermore, the panel sizes for these overlays are usually much smaller than typical jointed pavements to minimize the curling and warping that takes place.

This report is organized in three major sections. In section 2 the current condition of whitetopping projects is reviewed in Oklahoma. The inspection of these projects was primarily done with visual inspection, but some work was done with cores from the projects and also with Falling Weight Deflectrometer (FWD) measurements. In section 3 a review of the different whitetopping design methodologies is presented. In section 4 specific unanswered questions over whitetoppings are covered that the Oklahoma DOT felt were important to address from the existing literature.

2 Field Inspection

2.1 Overview

2.1.1 Visual Inspection

Visual inspection is one of the most important ways of determining the health of concrete overlays. In this project the visual inspection was completed by driving the vehicle along the shoulder of the pavement at a low speed while visually inspecting the pavement. Photographs were taken where distress was observed and also at places where future deterioration may be expected to occur. Care was taken so that exit and approach ramps were included. GPS technology was used to insure that the location of each test was accurately recorded so that future testing can be completed in the same location. During the inspection locations were determined for Falling Weight Deflectometer (FWD) testing and coring.

2.1.2 Falling Weight Deflectometer

Falling weight deflectometer testing is the experimental method of determining the load transfer efficiency of the panels of the overlays. The FWD testing was done with a DYNATEST deflectometer as shown in Fig. 2.1. In FWD testing, a heavy load is used as the drop weight and subsequent deflections are measured on the panels in front and back of the dropping locations. It is customary to use 6, 9, and 12 kips as drop loads. A 9 kips load was used for the drops in the case of the overlays in this project. The instrument had 9 sensors laid out as shown in Fig. 2.2. The deflectometer had 9 sensors; two behind the loading plate, one inside the load plate and the remaining six spanning over about 5 feet in the longitudinal direction. Fig. 2.3a shows the sensor layout and Fig. 2.3b shows the loading plate location to measure load transfer efficiency. The joint should be placed in between the loading plate and first sensor in front of the plate. The deflection data which is received is stored in the computer which is mounted inside the vehicle that pulls the FWD. The data is then analyzed to find the load transfer efficiency.

Figure 2.1 - The FWD behind the truck had a computer and camera assembly.

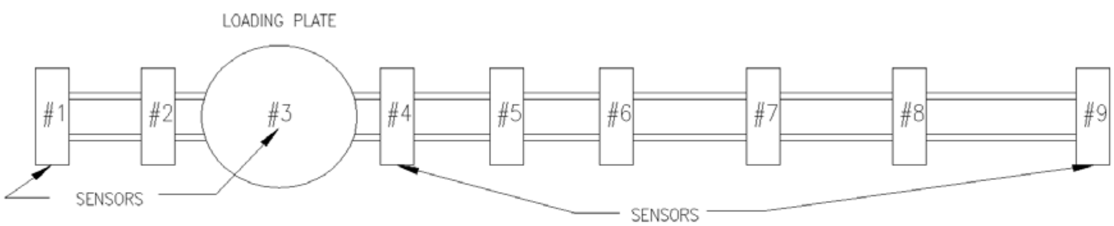


Figure 2.2 - Location of deflection sensors.

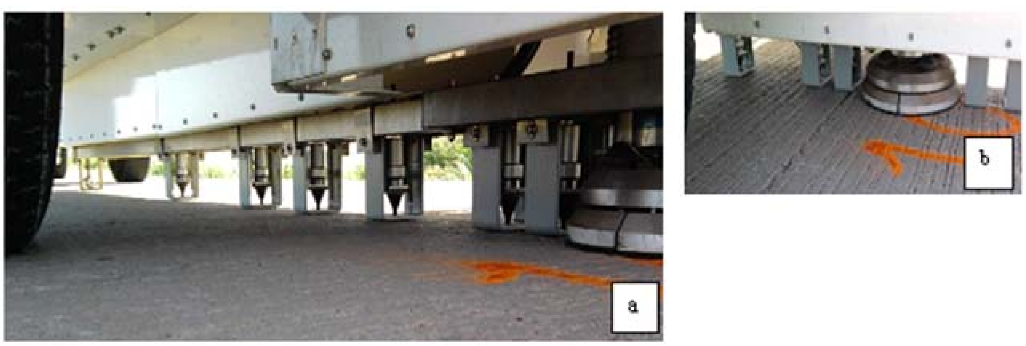


Figure 2.3 – The photos show the layout of the FWD and sensors on the whitetopping.

It is advised to carry out the FWD testing early in the morning or late in the afternoon so as to minimize the effects of temperature. Likewise all the FWD was carried out in the morning between 8 a.m. and noon.

2.1.3 Coring

Coring gives the details about the depth of the different layers present below the overlay. The detailed information about the different layers gives a better understanding about the present and future performance of the overlay. It also provides the opportunity to run tests like dynamic cone penetrometer to determine the foundation properties of the subbase layer. In this project DCP tests were run at all the coring locations. However, different types of soil conditions were observed. So it was hard to decide the exact expressions to be used for finding the bearing capacity. Also it was thought that the number of DCP tests was less for such lengths of the overlays. However, a fair estimate of modulus of rupture has been given in Appendix – A for three overlays in McAlester area. Because of the violent nature of the coring it is hard to comment on the bond between concrete and asphalt. It was common for debonding to be observed between the two layers. Fig. 2.4 shows a typical core photo.

Figure 2.4 – A typical core from an Oklahoma whitetopping.

2.2 McAlester

2.2.1 Overview

The series of overlays which were studied are located along US – 69 near the town of McAlester. The average daily traffic on the McAlester overlays is estimated to be just larger than 15,000, with 37% of them being trucks and 28.5% being trucks with 3 or more axles according to ODOT's 2007 Annual Truck Volume Report. The overlays in this area vary in age from 9 to 3 years old. The layout of overlays visited is shown in Fig. 2.5. Each square in Fig. 2.5 represents a one mile by one mile area.

The thicknesses of the overlays are 4", 5" and 6". Some overlays had different thicknesses in the passing and driving lanes; while others showed a consistent thickness.

A visual inspection was completed for the overlays on May 19 and 20, 2010. One unique aspect of this inspection is that it rained during the inspection. This allowed the research team to inspect the pavements in a highly saturated condition.

Falling Weight Deflectometer, coring and Dynamic Cone Penetrometer Testing were all done on the overlays. Each overlay visited is discussed in separate sections of the report.

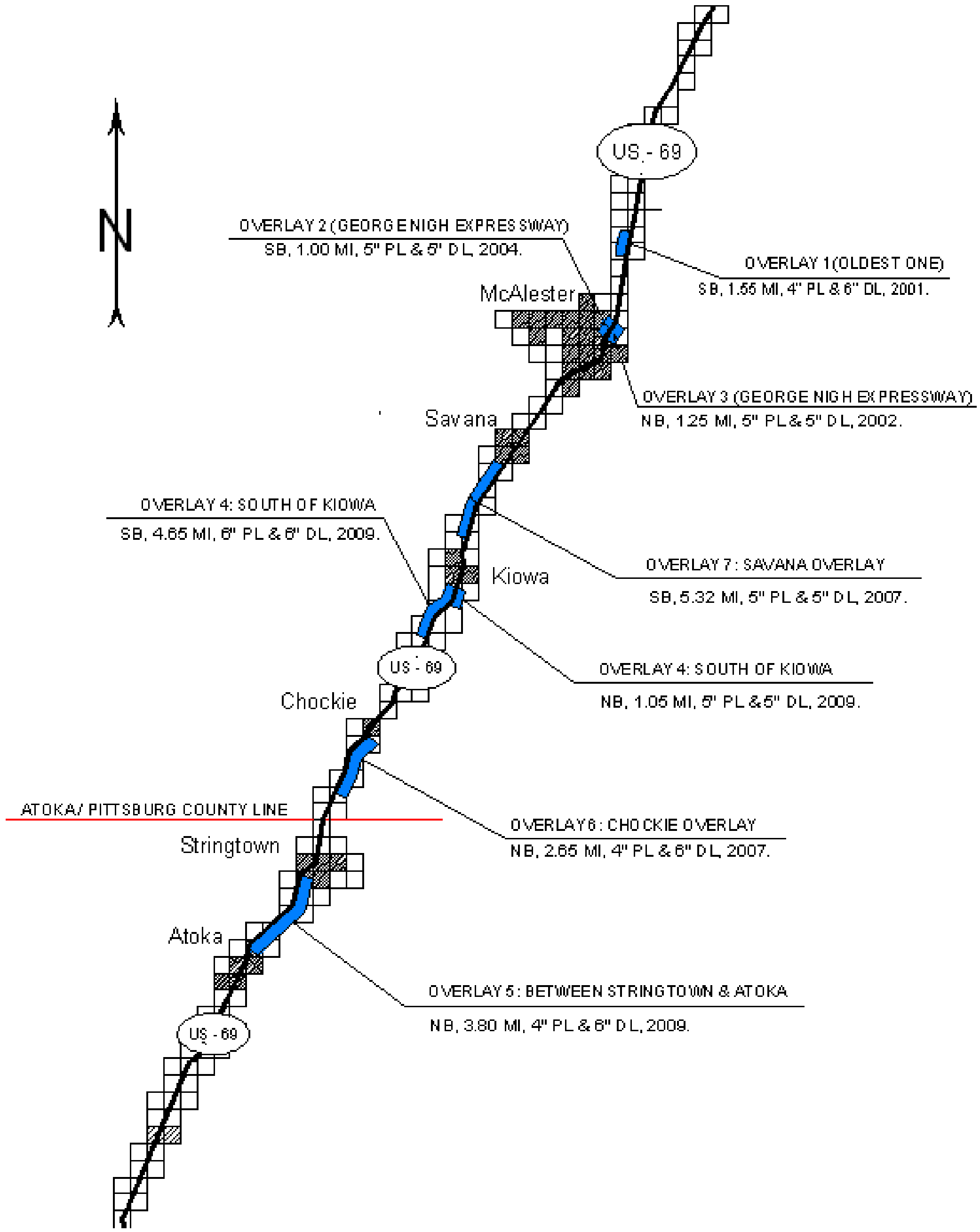


Figure 2.5 - Layout of overlays visited during the inspections. Each overlay with its designation location followed by direction, length, thickness of concrete in the passing lane (PL) and driving lane (DL) and year of construction.

Eight overlays were surveyed for this study. The tables and photographs provide information about the condition of the overlay and its construction details. The overlays will be discussed starting with the one that is farthest north and then moving south.

2.2.2 MC-161N(181) SB – Oldest McAlester Whitetopping

Table 2.1 – Details of the oldest McAlester whitetopping.

Project	**MC - 161N(181) s**outh bound
Location	Begin 0.7 miles north of Jct SH-113/US-69 and extend south 1.55 miles on US - 69: Pittsburg County
GPS coordinates at the start of the project	N 35°00’36.0”
	W 95°43’08.2”
Shoulder	asphalt 14' wide
Length	1.55 miles
Thickness of Concrete	4" in passing lane 12' wide; 6" in driving lane 14' wide
Panel Size	6' x 6' in passing lane; 6' x 7' in driving lane
Year of Construction	November, 2001

The overall condition of this overlay looks excellent. However, a few problems were observed. Joint raveling was a common problem which was observed as shown in Fig. 2.6 b. Some longitudinal cracks were seen covering multiple panels from 2 panels to 9 panels long. In total, 38 panels had visible cracks, which is about 1% of the total number of panels. Some examples of the longitudinal cracks are shown in Fig. 2.7. The passing lane was in better shape as compared to the driving lane. This is as anticipated as the travel lane would be expected to see significantly more traffic than the passing lane. Also in the driving lane the right wheel path showed more wear when compared to left wheel path. This may be due to the presence of an asphalt shoulder for this overlay. An asphalt shoulder would not provide as much support as the adjacent concrete lane. At about 0.2 miles from the start of the overlay a core was taken for previous testing. However, it was not refilled well; the filler material has been removed over time, as shown in Fig. 2.8. The drainage for this system was excellent.

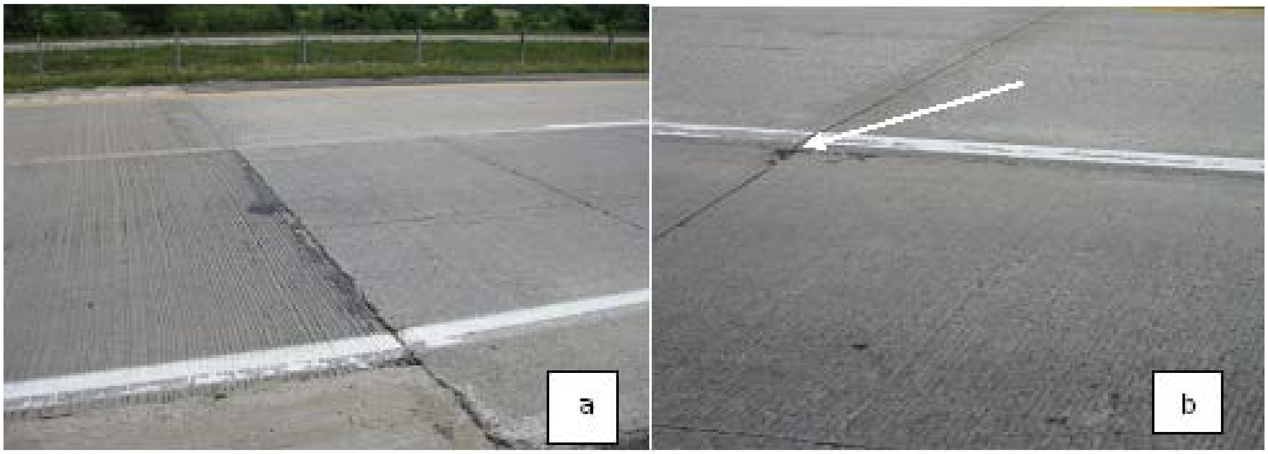


Figure 2.6 - Photo a showing the start of the overlay at north end. Photo b shows typical joint raveling seen.

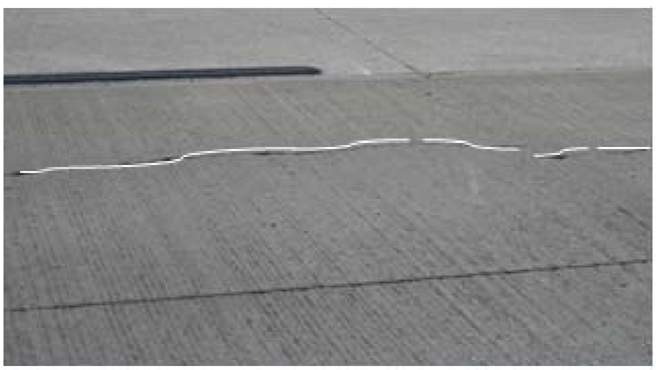


Figure 2.7 - Longitudinal Cracking spanning over panels

Figure 2.8 - Testing core not refilled well.

A longitudinal crack that covered 13 panels on the outside of the pavement was seen, as shown in Fig. 2.9 a. Faulting was seen at the end of the overlay. The last seven panels had problems with faulting. Occasional grinding on the surface was seen on these panels. This south bound overlay ended at an asphalt pavement as shown in Fig. 2.9 b. The end was also the top of a hill as shown in Fig. 2.9a.

Figure 2.9 - Photo a shows the 13 panel long longitudinal crack at the end of the overlay. Photo b shows the transition of concrete into asphalt at the end of the overlay. Photo c shows overall good condition of the overlay.

2.2.3 MC-161N(132) SB – George Nigh Expressway

Table 2.2 – Details of the George Nigh Expressway south bound.

Project	**MC - 161N(132) s**outh bound
Location	Begin 0.3 miles north of Jct US-270/US-69 (north end of railroad bridge) and extend north 1.0 Miles on US - 69: Pittsburg County
GPS coordinates at the start of the project	N 34°56´10.9"
	W 95°44´05.4"
Shoulder	asphalt 13' wide
Length	1.0 mile
Thickness of Concrete	5" in passing lane 13' wide; 5" in driving lane 13' wide
Panel Size	6' x 6' ; 6' x 7'
Year of Construction	March, 2004

The overall condition of this overlay looked very good. However, there were some problems observed. The initial panels had been replaced with a new concrete panel (Fig. 2.10 a). A definite elevation difference was seen between the asphalt pavement and the approach thin white-topping panel at the beginning of the project. The elevation difference was very significant, with some cracking observed in the approach panels even after the replacement (Fig. 2.10 b). Some heavy polishing was seen in both driving and passing lanes of the overlay, stretching to over 30 panels in length. Longitudinal cracking was also seen, but it was insignificant as it was just at a couple of places over the overlay and also spanned only for two panels. At about 0.7 miles after the start of the overlay (at exit for US 270 west on the highway), two punch outs were seen, as shown in Fig. 2.11. Some corner cracking was also observed where the approach ramp meets the overlay after this exit. None of the problems stated above were extensive, but they should be monitored for future deterioration. The end of the overlay looked good without any sign of distress, as shown in Fig. 2.12, as it ends in a concrete bridge. Also, drainage of this overlay looked quite good and satisfactory.

Figure 2.10 - Initial panel replacement at the start of overlay as seen in photo a, and in photo b we see how there is an elevation difference between the leaving asphalt panel and approaching concrete panel.

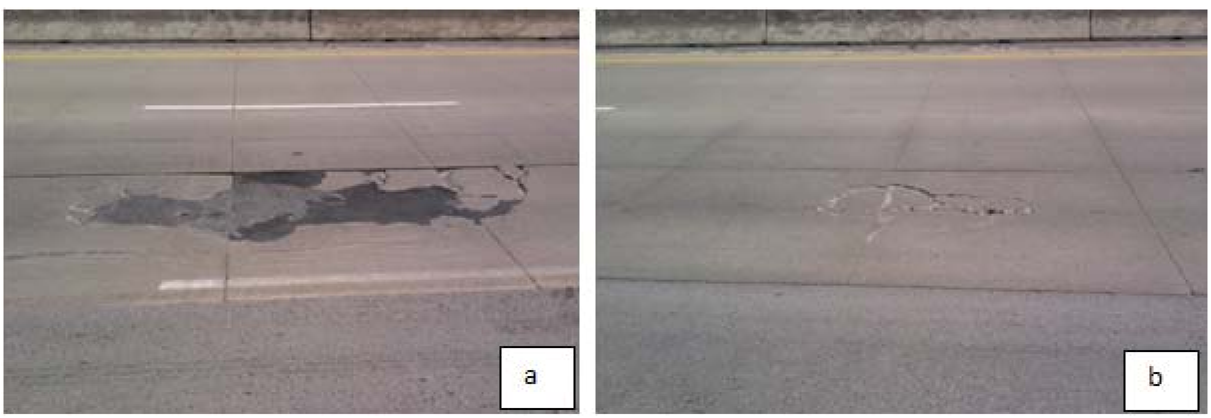


Figure 2.11 - Punch outs at the exit for US 270 west. These two punch (photos a and b) outs are about ten panels apart from each other.

Figure 2.12 – The overlay terminates at a bridge and does not show any distress.

2.2.4 MC – 161N(110)NB – George Nigh Expressway

Table 2.3 – Details of the George Nigh Expressway north bound.

Project	**MC - 161N(110)** north bound
Location	Begin 0.3 miles south of Jct US-270/US-69 (north end of railroad bridge) and extend North 1.25 Miles on US - 69: Pittsburg County
GPS coordinates at the start of the project	N 34°55´27.5"
	W 95°44´32.0"
Shoulder	asphalt 13' wide
Length	1.25 miles
Thickness of Concrete	5" in passing lane 13' wide; 5" in driving lane 13' wide
Panel Size	6' x 6' ; 6' x 7'
Year of Construction	December, 2002

This overlay starts at the end of a concrete bridge in McAlester. The overlay has shown good performance apart from the initial panels. The initial panels have heavy block cracking, as shown in Fig. 2.13 a. The driving lane of the overlay has a continuous replacement panel and not the thin white-topping overlay seen in the passing lane, as shown in Fig. 2.13 b. The reason for these failed initial panels could not be determined. Some minor longitudinal cracking, scaling and polishing were observed in the passing lane. However, they probably are caused by normal wear. At about 0.2 miles from the south end of the overlay, heavy surface scaling and corner cracks were observed in the driving lane. Similar heavy abrasion and scaling was again observed in the passing lane about 0.8 miles from the south end of the overlay.

Some pop outs were seen about 0.5 miles from the south end of the overlay. About 24 panels showed pop outs with approximately 20 per panel, as shown in Fig. 2.14. This is likely caused by an unsound aggregate being used in these panels. The overlay ended in a jointed concrete pavement. Significant scaling and faulting were observed where the overlay ends. However, repairs have been done for both panels, as shown in Fig. 2.15.

Figure 2.13 - Initial panels facing block cracking as shown in photo a. The first panel is replaced after initial failures with a full size panel as shown in photo b.

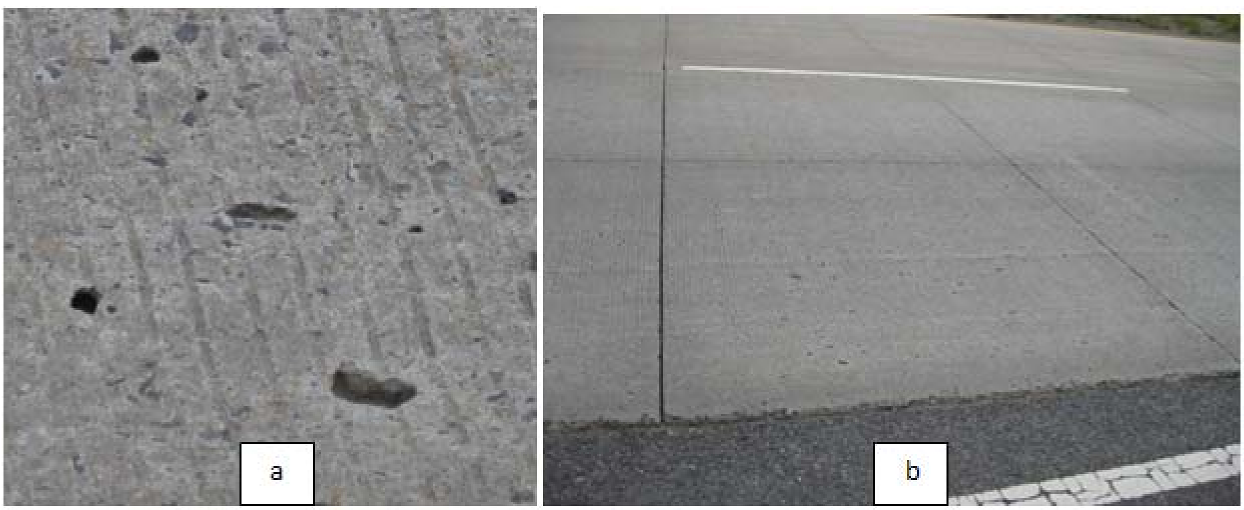


Figure 2.14 - Photo a showing pop outs (about 20 pop outs/panel) of larger aggregates (on about 24 panels) at about 0.5 miles from the south end of the overlay. Photo b is just the closer view of photo a.

Figure 2.15 – Photo a shows the repaired panels at the north end of the overlay. Photo b shows a cracked panel at the west end of the overlay.

2.2.5 NH-STIM(178)NB/SB – South of Kiowa

Table 2.4 – Details of the overlay south of Kiowa south bound.

Project	**NH – STIM(178) s**outh bound
Location	Begin 0.3 Miles North of the Atoka/Pittsburg County line and extend north 4.65 miles on US - 69: Pittsburg County
GPS coordinates at the start of the project	N 34°42´53.1"
	W 95°54´08.2"
Shoulder	concrete 8' wide
Length	4.65 miles
Thickness of Concrete	6" in passing lane 16' wide; 6" in driving lane 14' wide
Panel Size	6' x 6' ; 6' x 7'
Year of Construction	November, 2009

There are actually two overlays south of Kiowa, one going towards the north and the other going towards the south. The tables give us further information about these two different overlays.

These overlays, being new, are in good shape and have not shown any notable sign of distress. The figures below show the good condition of the overlays. Figures 2.16 a and b show the start and the end of the south bound overlay. The good condition of the overlays is seen in all the photographs.

Figure 2.16 - Photo a showing start and photo b showing end of the south bound overlay.

Table 2.5 - Details of the overlay south of Kiowa north bound.

Project	**NH - STIM(178)** north bound
Location	Begin 3.08 miles north of the Atoka/Pittsburg County line and extend North 1.05 Miles on US - 69: Pittsburg County
GPS coordinates at the start of the project	N 34°40´50.8"
	W 95°55´45.0"
Shoulder	concrete 8’ wide
Length	1.05 miles
Thickness of Concrete	5"
Panel Size	6' x 6' ; 6' x 7'
Year of Construction	November, 2009

Figure 2.17 shows the start of the north bound overlay. The initial panel has block cracking, as shown in the photo. The reason for this block cracking could not be determined. This is surprising, as the overlay is just one year old.

Figure 2.17 – Photo a shows a cracked panel at the start of an overlay. Photo b shows a closer view of that panel.

Figure 2.18 - Photo a shows the end of the overlay. The asphalt pavement looks good where it starts. But there is bit of distress initiation as shown in photo b.

2.2.6 NH – STIM(150) – Between Stringtown and Atoka

Table 2.6 - Overlay between Stringtown and Atoka.

Project	**NH - STIM(150)** north bound
Location	Begin 1.44 miles north of SH - 75 west and extend north 3.80 miles on US - 69, with a 412 feet exception for the North Boggy River Bridge: Atoka County
GPS coordinates at the start of the project	N 34°24´30.7"
	W 96°00´33.2"
Shoulder	asphalt 6' wide
Length	3.80 miles
Thickness of Concrete	4" in passing lane 16' wide; 6" in driving lane 14' wide
Panel Size	6' x 6' ; 6' x 7'
Year of Construction	November, 2009

This overlay showed some distress where it was adjacent to a CRCP. This distress included corner cracking and some initial cracking at the interface between the two systems, as shown in Fig. 2.19 a and b. But all of those remain at a very rudimentary level and pose no issue of concern. The overlay otherwise was in good shape. The end of the overlay is shown in Fig. 2.20. Some polishing was seen in the intermediate stage of the overlay, but it looked like mechanical polishing done for smoothing the surface and not the wear of the overlay. Fig. 2.21 shows one of these areas.

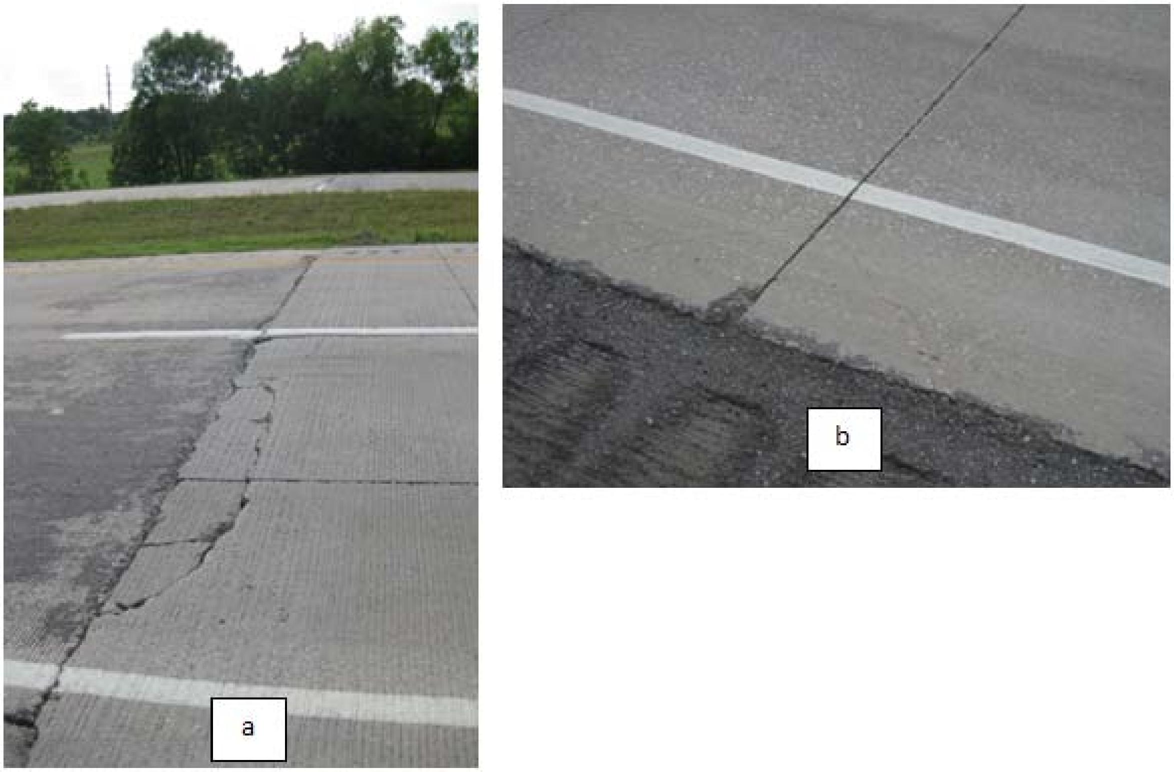


Figure 2.19 – Photo a shows the cracked panels at the junction of the overlay and the existing asphalt pavement. Photo b shows a rare corner crack on the overlay.

Figure 2.20 – The interface of the overlay and the adjacent pavement is in good condition.

Figure 2.21 – Polishing of the overlay surface.

2.2.7 SSR-103N(084)SR NB – Overlay near Chockie

Table 2.7 – Details of the overlay near Chockie.

Project	**SSR-103N(084) SR** north bound
Location	Begin 0.5 Miles North of Jct. SH- 43/US - 69 and extend north 2.65 miles on US - 69: Atoka County
GPS coordinates at the start of the project	N 34°32´46.0"
	W 96°01´29.5"
Shoulder	asphalt 8' wide
Length	2.65 miles
Thickness of Concrete	4" in passing lane 16' wide; 6" in driving lane 14' wide
Panel Size	6' x 6' ; 6' x 7'
Year of Construction	early 2007

There was very few corner or longitudinal cracks seen on the pavement. Some polishing in the driving lane and some abrasion were seen in a few spots. Grinding was performed at some places to make the surface of the overlay smoother. The initial panel is without any distress, as shown in Fig. 2.22 a. There was some shoulder alligator cracking as shown in Fig. 2.23 a, but it was only seen at two places. The end of the overlay looked excellent, as it ended into a CRCP. The overall health of the overlay is excellent.

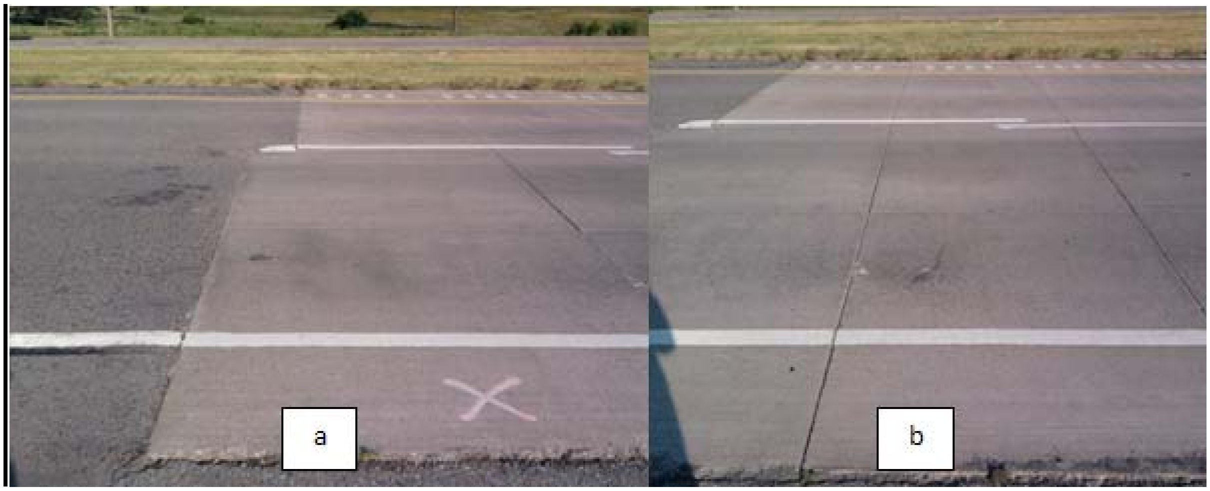


Figure 2.22 - Photo a showing start of the overlay where the asphalt pavement ends. A corner crack was seen on the second panel right after the start of the overlay is shown in photo b.

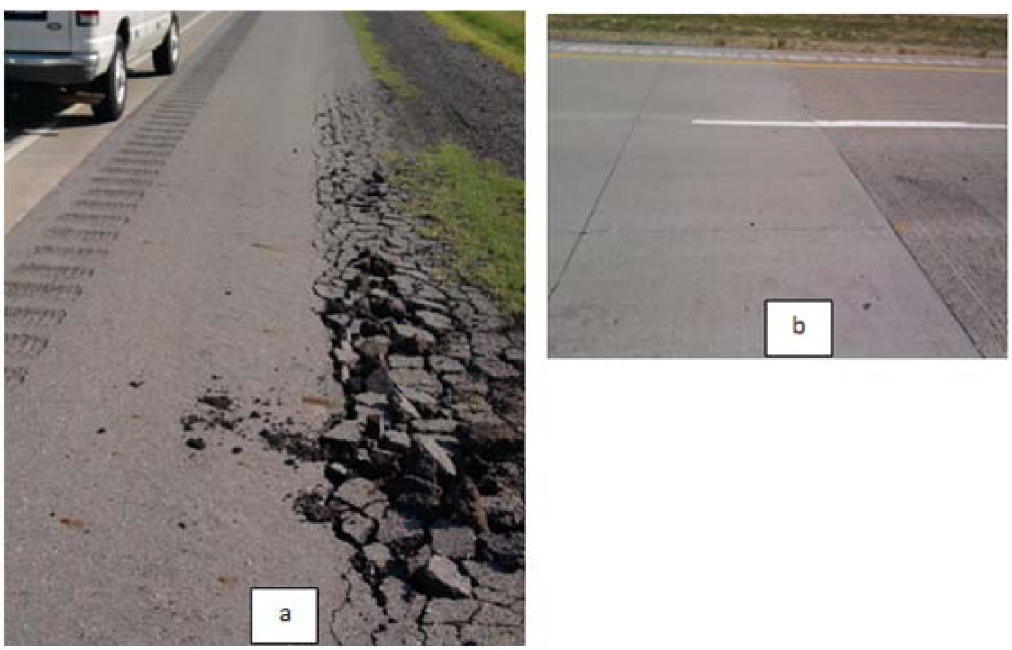


Figure 2.23 – Photo a shows the deterioration of the asphalt shoulders. Photo b shows the interface of the overlay with a CRCP pavement. The end of the overlay was in excellent condition.

2.2.8 NHY – 013N(113R)SB – Unbonded Overlay near Savana

Table 2.8 – Details of Savana Overlay (Un-bonded concrete overlay).

Project	**NHY - 013N(113R) s**outh bound
Location	Begins 1.0 mile north of Kiowa and extends north 5.32 miles on US - 69: Pittsburg County
GPS coordinates at the start of the project	N 34°48´35.1"
	W 095°51´38.8"
Shoulder	asphalt 8' wide
Length	5.32 miles
Thickness of Concrete	5" thick and 30' wide
Panel Size	6' x 6' in passing lane; 6' x 7' in driving lane
Year of Construction	November, 2007

This is an un-bonded concrete overlay on the asphalt surface with a bond breaker. This overlay is technically not part of this study but the researchers thought that it may be of interest to ODOT, as it was very young and yet showed such poor performance.

Most of the problems on this overlay were in the right wheel path of the driving lane. The passing lane did not show significant deterioration. Longitudinal cracks extending though two panels on average were seen all along the overlay; some cracks extended through up to 4 panels in a row. The number of cracked panels was significant, as shown in Fig. 2.24. About 200 panels (about 6%) were cracked per mile for the initial two miles. Then this crack pattern stopped abruptly.

Figure 2.24 - Photo a showing the type of longitudinal cracking pattern. Photo b shows the cracks after heavy rains overnight. The seepage of water through the cracks is clearly visible.

It was noticed that the deterioration of the overlay seemed to correlate with the intersections. Sometimes the failure was seen just as an individual panel failure. Sometimes it was seen as a series of panel failures covering up to seven panel failures in a single stretch. There were also large pieces of concrete that were from the failed panels. The drainage for this overlay was not good. Several places were seen to retain water instead of letting it flow through the system. However, it is unclear how much of an effect this poor drainage has on the failures. A total of 19 such individual or series panel failures were seen along the 5 mile stretch of the overlay, some of them are shown in Fig. 2.25.

Figure 2.25 - Photos showing the failures of panels in the right wheel path of driving lane of the overlay. Photo a shows failures observed in continuous panels. Photo b shows the pieces of concrete that came from the failed panels. Photo c shows individual panel failures, typically seen in at the intersections. Photo d shows drainage problems in the pavement.

2.2.9 Falling Weight Deflectometer

Falling Weight Deflectometer (FWD) analysis along with coring operations were done on three of the aforementioned overlays. All the experimentation was carried out by ODOT and coordinated by Chris Westlund. The three selected overlays were MC-161N(181) SB (Oldest McAlester Whitetopping), MC-161N(132) SB (George Nigh Expressway), and SSR-103N(084)SR NB (near Chockie). MC-161N(181) SB was selected because it is the oldest overlay that was in service and is in good condition. MC-161N(132) SB is in McAlester city and is showing good service. SSR-103N(084)SR NB was chosen because it is very young and it will give a good baseline data for further studies.

The Falling Weight Deflectometer testing was started at about 9 am every day and ended at 11 am. The pavement surface temperature range during testing was 92.5 °F to 120.5 °F. The pavement temperature was recorded at each drop location. Tables 2.9 through 2.11 give the location of FWD drops; the GPS coordinates of the test locations and the load transfer efficiencies. All but two of the Load Transfer Efficiencies (LTE) were above 70% this is very good for a whitetopping that has been in service for over nine years.

Table 2.9 - FWD locations and LTE for MC-161N(181) SB - Oldest McAlester Whitetopping.

Sr No	Location and notation on pavement	GPS Coordinates	Drop Description	Remarks/Comment	Load Transfer Efficiency in %	
					Right Lane	Left Lane
1	Initial Panel	N 35°00.595' W 095°43.136'	Joint Drop	1^{st}	93	93
				3^{rd}	95	91
				5^{th}	95	91
				7^{th}	91	89
2	At 0.2 miles (1)	N 35°00.413' W 095°43.144'	Joint Drop		94	93
3	At 0.4 miles (2)	N 35°00.199' W 095°43.182'	Joint Drop	At 0.4 miles + 50 panels (300 feet) to south of ramp.	94	90
4	At 0.6 miles (3)	N 35°00.071' W 095°43.207'	Joint Drop		95	90
5	At 0.8 miles (4)	N 34°59.877' W 095°43.244'	Joint Drop		93	85
6	At 1.0 miles (5)	N 34°59.678' W 095°43.283'	Joint Drop		94	90
7	At 1.2 miles (6)	N 34°59.559' W 095°43.303'	Joint Drop		95	89
8	At 1.4 miles (7)	N 34°59.384' W 095°43.320'	Joint Drop		89	87
9	At 1.5 miles (8)	N 34°59.275' W 095°43.323'	Joint Drop		84	68
10	Last Panel	N 34°59.210' W 095°43.323'	Joint Drop	7^{th}	93	95
				5^{th}	95	96
				3^{rd}	95	51
				Last	91	83
			Average		**92.9**	**86.4**

Table 2.10 - FWD locations and LTE for MC-161N(132) SB - George Nigh Expressway south bound.

<table>
<tr><th rowspan="2">Sr No</th><th rowspan="2">Location and notation on pavement</th><th rowspan="2">GPS Coordinates</th><th rowspan="2">Drop Description</th><th rowspan="2">Remarks/ Comment</th><th colspan="3">Load Transfer Efficiency in %</th></tr>
<tr><th>Right Lane</th><th colspan="2">Left Lane</th></tr>
<tr><td rowspan="4">1</td><td rowspan="4">The initial Panel</td><td rowspan="4">N 34°56.211'
W 095°44.067'</td><td rowspan="4">Joint Drop</td><td>1st</td><td rowspan="4"></td><td>51</td><td rowspan="4">61.8</td></tr>
<tr><td>3rd</td><td>80</td></tr>
<tr><td>5th</td><td>51</td></tr>
<tr><td>7th</td><td>65</td></tr>
<tr><td>2</td><td>At 0.1 miles (1)</td><td>N 34°56.126'
W 095°44.133'</td><td>Joint Drop</td><td></td><td></td><td colspan="2">92</td></tr>
<tr><td>3</td><td>At 0.2 miles (2)</td><td>N 34°56.048'
W 095°44.190'</td><td>Joint Drop</td><td></td><td></td><td colspan="2">87</td></tr>
<tr><td>4</td><td>At 0.3 miles (3)</td><td>N 34°55.982'
W 095°44.234'</td><td>Joint Drop</td><td></td><td></td><td colspan="2">92</td></tr>
<tr><td>5</td><td>At 0.4 miles (4)</td><td>N 34°55.908'
W 095°44.278'</td><td>Joint Drop</td><td></td><td></td><td colspan="2">90</td></tr>
<tr><td>6</td><td>At 0.5 miles (5)</td><td>N 34°55.822'
W 095°44.327'</td><td>Joint Drop</td><td></td><td>93</td><td colspan="2">89</td></tr>
<tr><td>7</td><td>At 0.6 miles (6)</td><td>N 34°55.740'
W 095°44.381'</td><td>Joint Drop</td><td></td><td>93</td><td colspan="2">83</td></tr>
<tr><td>8</td><td>At 0.7 miles (7)</td><td>N 34°55.657'
W 095°44.425'</td><td>Joint Drop</td><td>Below the bridge</td><td>95</td><td colspan="2">92</td></tr>
<tr><td>9</td><td>At 0.8 miles (8)</td><td>N 34°55.594'
W 095°44.467'</td><td>Joint Drop</td><td>Charlie's Chicken on right</td><td>92</td><td colspan="2">91</td></tr>
<tr><td>10</td><td>At 0.9 miles (9)</td><td>N 34°55.509'
W 095°44.520'</td><td>Joint Drop</td><td>At the milage sign of ATOKA 45 and Durant 78</td><td>93</td><td colspan="2">88</td></tr>
<tr><td rowspan="4">11</td><td rowspan="4">Last Panel @ 1.0 miles</td><td rowspan="4">–</td><td>Joint Drop</td><td>7th</td><td></td><td colspan="2">93</td></tr>
<tr><td>Joint Drop</td><td>5th</td><td></td><td colspan="2">92</td></tr>
<tr><td>Joint Drop</td><td>3rd</td><td></td><td colspan="2">92</td></tr>
<tr><td>Joint Drop</td><td>Last</td><td></td><td colspan="2">91</td></tr>
<tr><td colspan="3"></td><td colspan="2">Average (excluding first four)</td><td>93.3</td><td colspan="2">90.1</td></tr>
</table>

Due to entry and exit ramps FWD could not be performed on the initial panels of the driving lane. However FWD was run on the initial panels of the passing lane. This is the same overlay which has a replaced initial panel and is also severely distressed, so lower FWD values for those panels was not a surprise. However, the remedy for such a failure remains a matter of further studies. Apart from initial panels, the FWD results showed high values of LTE.

Table 2.11 - FWD locations and LTE for SSR-103N(084)SR NB near Chockie.

<table>
<tr><th rowspan="2">Sr No</th><th rowspan="2">Location and notation on pavement</th><th rowspan="2">GPS Coordinates</th><th rowspan="2">Drop Description</th><th rowspan="2">Remarks/Comment</th><th colspan="4">Load Transfer Efficiency in %</th></tr>
<tr><th colspan="2">Right Lane</th><th colspan="2">Left Lane</th></tr>
<tr><td rowspan="4">1</td><td rowspan="4">Initial Panel 0.0 mile</td><td rowspan="4">N 34°32.773'
W 096°01.494'</td><td rowspan="4">Joint Drop</td><td>1st</td><td>47</td><td rowspan="4">61.25</td><td>55</td><td rowspan="4">76</td></tr>
<tr><td>3rd</td><td>54</td><td>62</td></tr>
<tr><td>5th</td><td>51</td><td>93</td></tr>
<tr><td>7th</td><td>93</td><td>94</td></tr>
<tr><td>2</td><td>At 0.1 miles (1)</td><td>N 34°32.862'
W 096°01.442'</td><td>Joint Drop</td><td></td><td colspan="2">82</td><td colspan="2">87</td></tr>
<tr><td>3</td><td>At 0.2 miles (2)</td><td>N 34°32.946'
W 096°01.395'</td><td>Joint Drop</td><td></td><td colspan="2">92</td><td colspan="2">87</td></tr>
<tr><td>4</td><td>At 0.3 miles (3)</td><td>N 34°33.014'
W 096°01.357'</td><td>Joint Drop</td><td></td><td colspan="2">94</td><td colspan="2">88</td></tr>
<tr><td>5</td><td>At 0.4 miles (4)</td><td>N 34°33.094'
W 096°01.313'</td><td>Joint Drop</td><td></td><td colspan="2">94</td><td colspan="2">87</td></tr>
<tr><td>6</td><td>At 0.5 miles (5)</td><td>N 34°33.184'
W 096°01.261'</td><td>Joint Drop</td><td></td><td colspan="2">88</td><td colspan="2">85</td></tr>
<tr><td>7</td><td>At 0.6 miles (6)</td><td>N 34°33.252'
W 096°01.222'</td><td>Joint Drop</td><td></td><td colspan="2">93</td><td colspan="2">90</td></tr>
<tr><td>8</td><td>At 0.7 miles (7)</td><td>N 34°33.325'
W 096°01.181'</td><td>Joint Drop</td><td></td><td colspan="2">92</td><td colspan="2">86</td></tr>
<tr><td>9</td><td>At 0.8 miles (8)</td><td>N 34°33.405'
W 096°01.144'</td><td>Joint Drop</td><td></td><td colspan="2">87</td><td colspan="2">86</td></tr>
<tr><td>10</td><td>At 0.9 miles (9)</td><td>N 34°33.489'
W 096°01.093'</td><td>Joint Drop</td><td></td><td colspan="2">94</td><td colspan="2">89</td></tr>
<tr><td>11</td><td>At 1.0 miles (10)</td><td>N 34°33.567'
W 096°01.048'</td><td>Joint Drop</td><td></td><td colspan="2">91</td><td colspan="2">86</td></tr>
<tr><td></td><td></td><td></td><td colspan="2">Average (excluding first four)</td><td colspan="2">92.9</td><td colspan="2">87.2</td></tr>
</table>

For the Chockie overlay, the initial FWD readings were lower than for the rest of the whitetopping. However, the initial panels did not show any sign of distress. This area should be watched carefully as further distress may occur soon. This is discouraging for an overlay that is so young. However, it highlights that more attention may be needed at the interface of whitetoppings with other pavements or bridges. Other than the initial panels all the FWD values are in the higher range. A higher LTE emphasizes the good condition of the overlay.

2.2.10 Coring

The cores give important information about depths of concrete and asphalt layers on the pavement. The thickness of the asphalt was significant for all the overlays. It was observed to be more than 9” for all the locations where cores were extracted for these three overlays. Because of the violent nature of the coring process it is hard to comment on the bond between concrete and asphalt. It was common for debonding to be observed.

Fig. 2.26 shows the cores extracted from Oldest McAlester Whitetopping. The thickness of asphalt was found to be, at minimum, 9” and was on an average 11”. This is likely one contributing factor into the positive performance of the overlay. Four cores were taken at the locations 1 and 5 from the driving lane and 3 and 7 from the passing lane as described in Table 2.9.

Figure 2.26 - Cores extracted on the oldest McAlester Overlay.

Three cores were taken at locations 2 and 7 from in the passing lane and location 5 in the driving lane for George Nigh Expressway south bound. The minimum depth of asphalt was found to be 9 ¼” from the coring operations. The depth of asphalt on an average was 10.5”.

Coring on the Chockie Overlay showed that the asphalt that was placed on top of an old concrete layer. The asphalt layer was 8” to 9” in the driving lane and 10” to 12” in the passing lane. The constant depth of the old concrete pavement was 6”. Four cores were taken at the locations 2 and 7 in the passing lane and 3 and 9 in the driving lane from Table 2.11. Figure 2.27 shows the core at location 9.

Figure 2.27 – Core from location 9.

Figure 2.28 shows a comparison of the cores extracted from the overlays. As seen in the figure there is a difference between the overall depth of the material (concrete and asphalt) below each overlay. The George Nigh SB Whitetopping had the least amount of support material when compared to the other overlays but still shows satisfactory performance. It will be interesting to see how this material impacts the long term performance of this overlay compared to the others. The thickness of aggregate base is not included as the coring operations were only carried out from concrete overlay to the end of asphalt layer or to the end of the older concrete pavement for the Chockie Overlay.

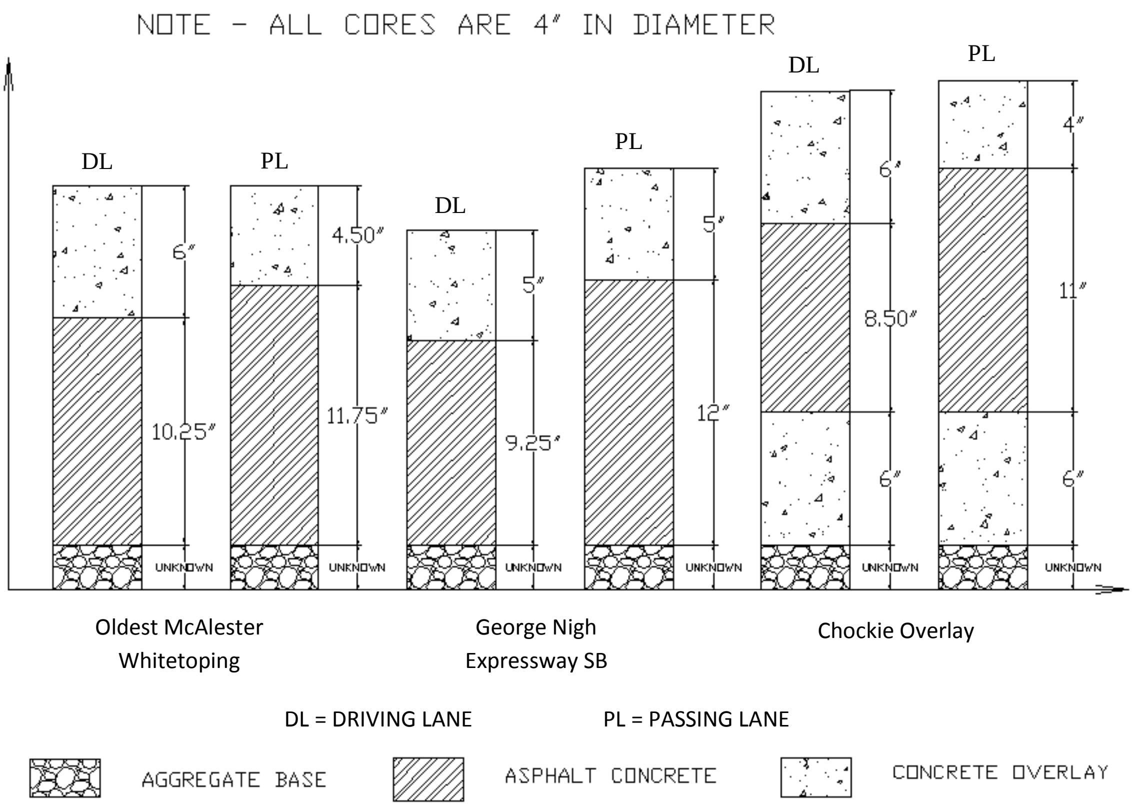


Figure 2.28 - Details of different cores extracted from Overlay 1, Overlay 2 and Overlay 6 from left to right respectively. The values shown are an average of two core measurements.

2.2.11 Discussion

Important observations from these investigations include:

- Satisfactory performance was observed for all overlays inspected. This includes overlays that had been in service for almost 9 years.
- The thickness of the asphalt for all of the whitetoppings investigated was significant. Therefore, it is difficult to determine how these whitetoppings would have performed with thinner support sections or with asphalt that had more degradation.
- The crack patterns observed were similar between all of the overlays investigated.
- The most consistent areas of distress were observed at the interface between the overlay and the adjacent pavement. This area could use better construction details.

Based on the observations made in this project the following recommendations can be made:

1) A whitetopping is a good candidate for an area that has a thick layer of asphalt.
2) Thin Whitetoppings have shown good performance in areas with significant truck traffic in highway conditions in Oklahoma when the overlay was used over at least 8” of asphalt.
3) Better construction details should be devised for areas at the interface between the overlay and the adjacent pavement as these areas showed the most distress. This distress was most likely attributed to the difference in height between the overlay and the adjacent pavement or bridge. It is hard to determine if this height difference existed at construction or has developed over time.

2.3 Stillwater

2.3.1 Overview

The overlay is located in Stillwater. It is at the intersection of south Perkins Road (US – 177) and east 6th Avenue (SH – 51). This is a very busy intersection in Stillwater.

The intersection was constructed in July 1999. Oklahoma DOT class AA concrete was used with fiber reinforcing. However, at the time of inspection, no fibers were seen either on the surface of the overlay or in the joints. At the time of inspection the intersection had been in service for eleven years. The total cost of construction was $152,000. The total construction time was seventeen days. This intersection at US – 177 is 61.8’ wide with five lanes and at SH – 51 it is 52.2’ wide, again with five lanes as shown in Fig. 2.29. The panels had spacing of 6’ x 6’and 6’ x 6.3’.6’ x 5.7’sized panels were used for a few corner panels.

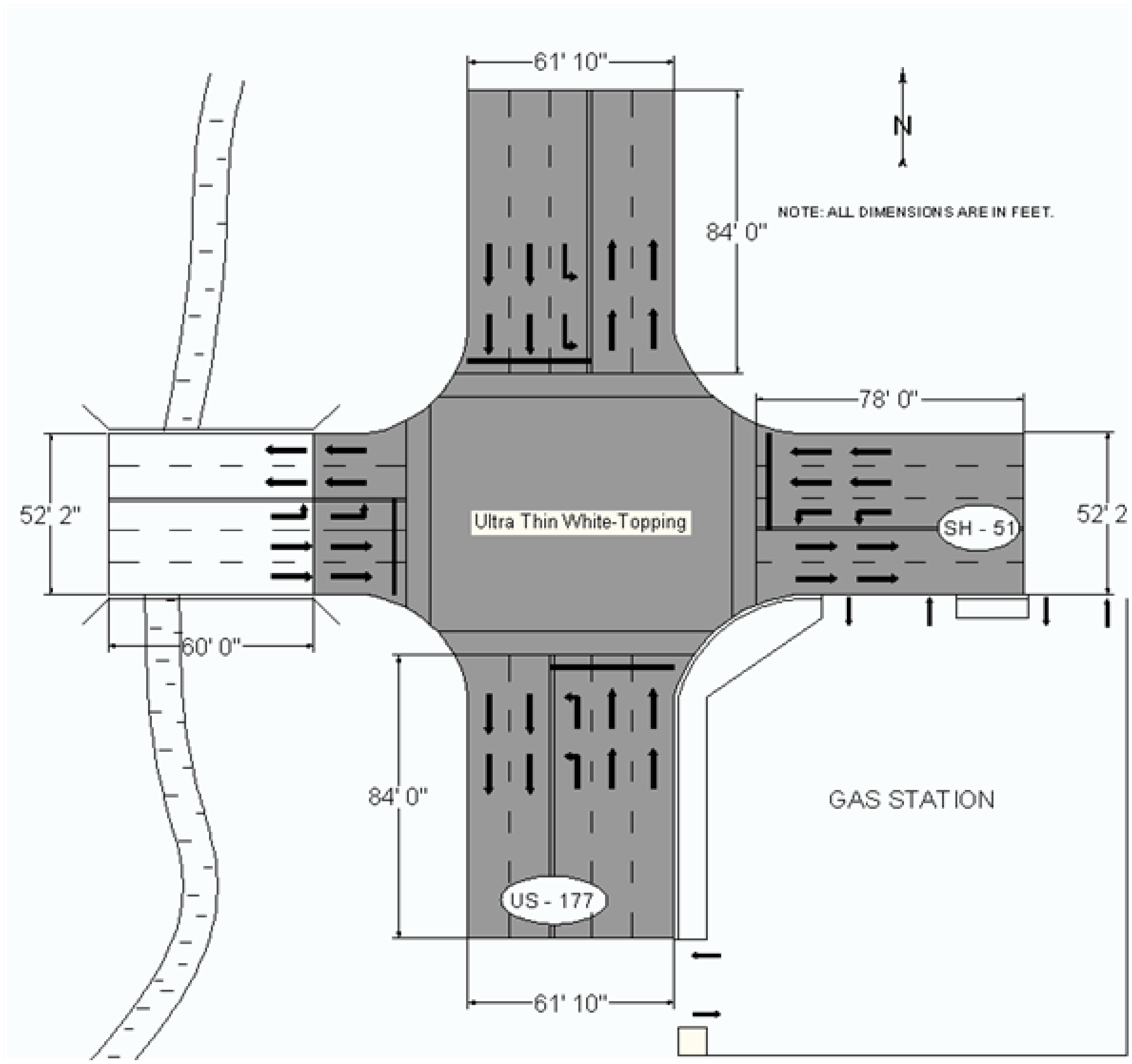


Figure 2.29 - Layout of Intersection of US-177 and SH – 51 in Stillwater, OK.

The depth of the overlay is 4”. The joints of the panels are not sealed.

Table 2.12 - The AADT Data for the Stillwater overlay.

Direction	ODOT AADT State Records*		MacArthur Consultants Survey**
	2005	2008	2004
North	20300	15500	22473
East	9200	9300	17774
South	13300	10400	16772
West	13600	10400	16550

*ODOT, annual survey (2005 and 2008)
** David Cline, MacArthur Associated Consultants.

The percentage of trucks in all directions did not exceed 5 %. Numbers of trucks were measured at the intersection for ten minutes in every direction on 6/11/10.

2.3.2 Visual Inspection

Inspection was done on June 6 and 11, 2010. The overall condition of the intersection was good, but some corner and longitudinal cracking, faulting, and fines movements were noteworthy. On the US – 177 approaches of the intersection, less damage was observed, as shown in Figs. 2.31 through 2.35; however, on SH 51 the approaches of the intersection showed considerable damage. This damaged area can be seen in Figs. 2.36 through 2.44. A summary of the picture locations is shown in Fig. 2.30.

Fig. 2.31 is a photo taken from the edge of the gas station towards the north-west. The panels of the intersection seen in this photo look excellent. There was negligible amount of distress on this side of the intersection. Also the approach from south as shown in Fig. 2.32 does not show any damage. One explanation for this is that there was no elevation difference between the asphalt and concrete overlay. The panels, as seen in Fig. 2.33, on the approach lanes from south to the intersection on US – 177 look in good condition. Again Fig. 2.34 shows the same good conditions of the panels on the approach from the north to the intersection on US – 177. The area shown in Fig. 2.35 is the approach on US – 177 from the north side. Fig. 2.35 supports the hypothesis that if there is no elevation difference between the asphalt pavement and the concrete overlay, the initial panels will show good performance.

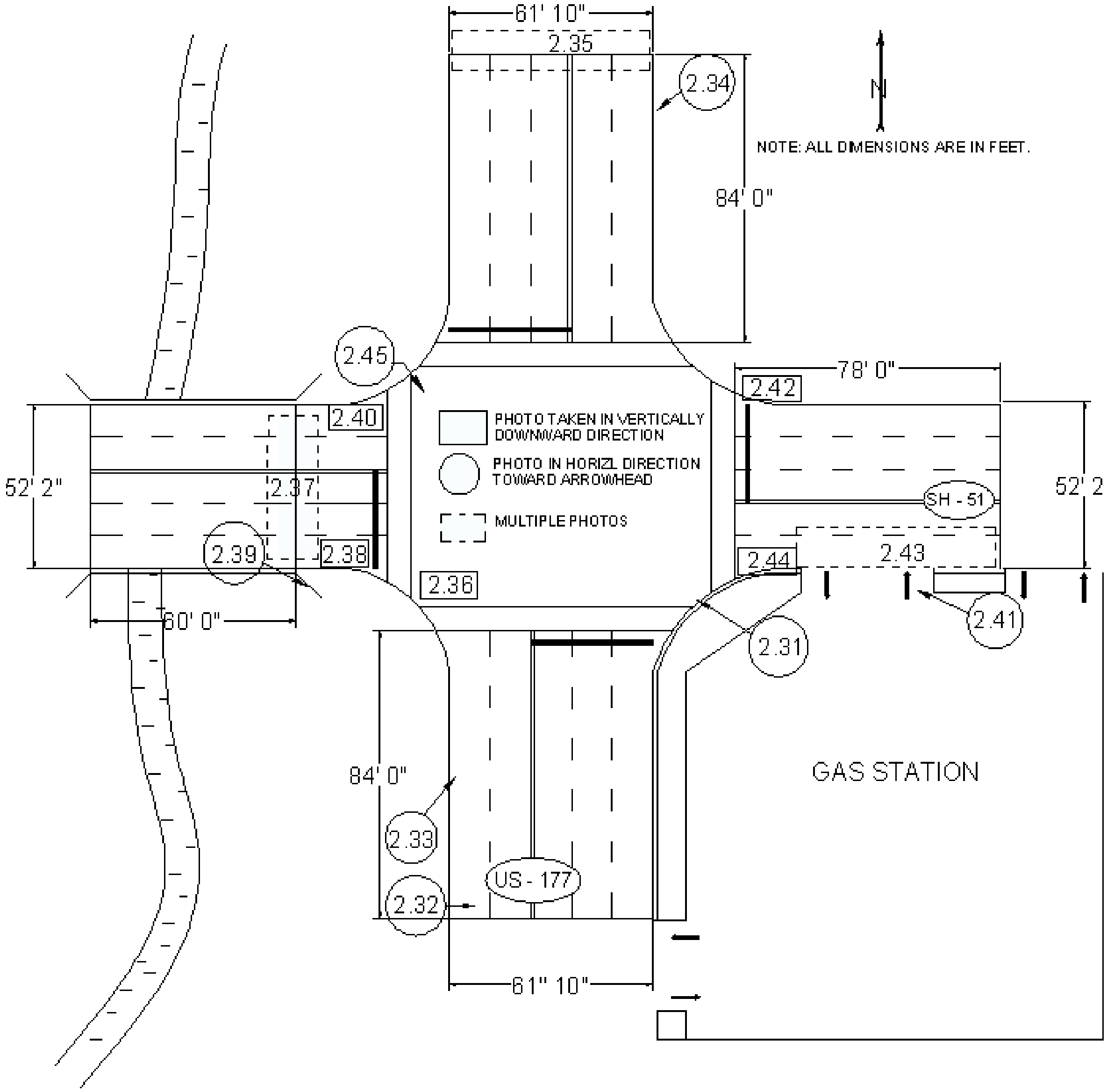


Figure 2.30 - Layout of the intersection of US – 177 and SH – 51 depicting the location where the photographs were taken.

Figure 2.31 - Intersection as seen from the edge of gas station towards north-west.

Figure 2.32 - Approach on US 177 from south showing excellent condition of initial panels.

Figure 2.33 - Excellent condition of panels on US 177 from the south.

Figure 2.34 - Photo of intersection seen on US 177 towards south-west showing excellent condition of overlay.

Figure 2.35 - Approach on US 177 from north showing absence in panel failures. Note the small difference in the elevation between the approach and leave pavements.

Figures 2.36 through 2.40 show the problems observed primarily at the west approach on SH – 51. The problems observed were cracking in the shoulder, approach panel failures, fines movements, and abutment cracking. Figure 2.36 shows the cracking in the shoulder on the US – 177 south side turn from west SH – 51. Figure 2.37 shows the approach panel failures from west SH – 51. The first panel is longer (6’ x 9’) than all of the other overlay panel dimensions. The 9’ dimension is in the east-west direction, the same direction as the traffic. Also, there is considerable rutting (refer Fig. 2.37 b) on the adjacent concrete leave slab on the east side of the concrete bridge connecting to the intersection (refer Fig. 2.29 for layout of bridge and overlay). This rutting has induced an elevation difference which may have decreased the performance of the approach panel. In addition, since this panel is longer, one would expect the length to increase the bending stresses in the panel from center loadings as well as the curling of the panel. Severe cracking of the panel can be seen in Fig. 2.37.

Figure 2.36 – Distress cracking in the shoulder.

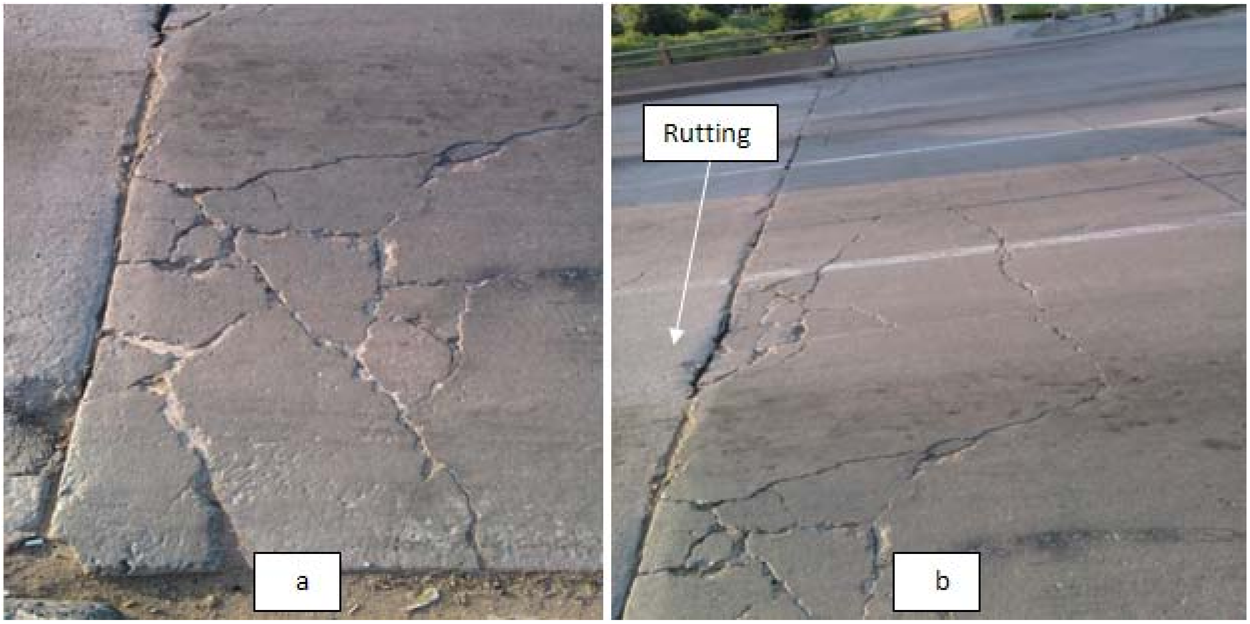


Figure 2.37 - Photos from west side of intersection on SH – 51. Photo a is at the intersection of the overlay and the concrete bridge. Rutting can be clearly seen in photo b.

Figures 2.38, 2.39 and 2.40 show the fines movement seen on the pavements, the abutment cracking and the impact loading due to an insufficient turning radius for large trucks. Figure 2.39 shows the abutment cracking which may have been caused by thermal movements of the pavement or perhaps settlement of the compacted soil behind the bridge abutment. However, it is not clear when the cracking occurred. It is also not clear whether there is any type of relation between the fines movement and the bridge abutment. Figure 2.40 is really interesting as it shows how the tire impacts the ground as it leaves the curb and creates stresses on the surface of the pavement.

Figure 2.38 - Fines movement in first panels of intersection. **Figure 2.39 -** Cracking in abutments of the bridge.

Figure 2.40 - Photo showing the impact loading due to lack of radius of curvature for large trucks.

Figures 2.41 through 2.44 show the problems with the east end of the intersection. Again, this is also on SH – 51. Some of the problems observed were faulting and panel cracking where the concrete overlay interfaced the asphalt shoulder. Figures 2.41 through 2.42 show the faulting which is taking place on the eastern end of the overlay. Predominantly, faulting was seen only in east bound lanes and not in west bound lanes. However, the west bound lanes showed quite a bit of longitudinal cracking.

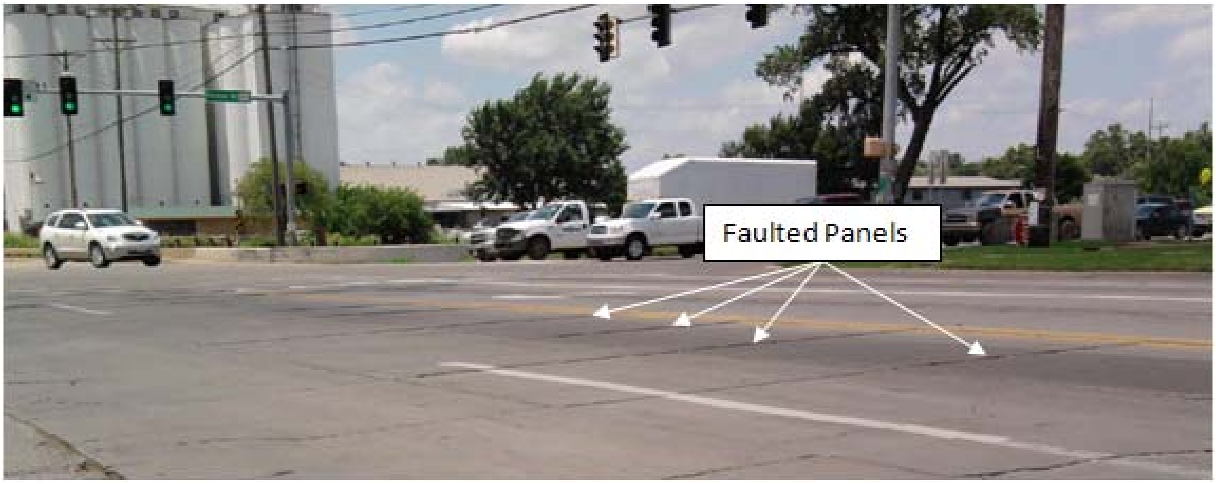


Figure 2.41 - Photo showing faulting as seen from east end of the intersection. A repetitive noise was heard from the tires of the vehicles when they passed on the faulted panels.

Figure 2.42 - Photos a and b showing faulting as overlay turns from west SH – 51 to north US – 177.

Figures 2.43 through 2.44 show how the cracking has increased. This increase in damage is apparent at the interface of concrete and asphalt. Figure 2.43 shows a crack that has extended from the shoulder since 2004. We know this crack has extended as we have photographs taken by MacArthur and Associates from 2004. Some sections were seen in which some parts of panels have been elevated, as shown in Fig.2.44.

Figure 2.43 - Photos at the entrance to the gas station showing how the cracking has changed between 2004 and 2010. Photo a was taken in 2004 (David Cline, MacArthur Associated Consultants) and photo b is taken by the research team in 2010.

Figure 2.44 - Large block cracking at the east end of the intersection where the overlay is adjacent to the gas stations. The shadows shown in photo a show the difference in height between to the overlay and the adjacent asphalt. Additional block cracking can be seen in photo b.

However, overall condition of the overlay in the intersection looked satisfactory as shown in Fig. 2.45.

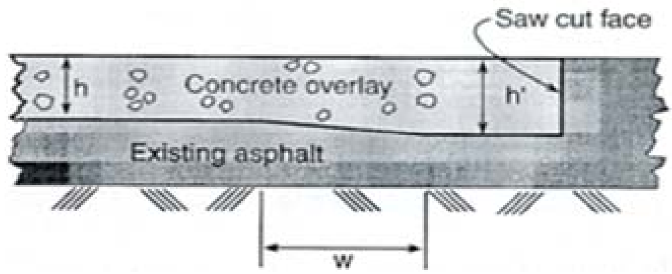
Figure 2.45 - Overview of the intersection.

2.3.3 Discussion

Based on the field investigation in this study, the following observations can be made:

1. The approach panels that had a longer aspect ratio (6' x 9' compared to 6' x 6') showed more distress than the other panels in the overlay. Distress in the overlay was observed at the interface between the concrete overlay and adjacent asphalt slab. Other types of interfaces such as overlay-concrete pavement are not available in this project so the effect of such an interface could not be studied. Improved details should be used in these areas. One possible detail that may improve this situation is shown in Fig. 2.46. However, more research is needed to further investigate the performance. One area of concern for this detail is that if an increased overlay thickness is used then this will require more milling of the asphalt. This additional milling may cause loss of support and premature failure of the overlay.

2. Maintaining close elevation difference at the approach panels may reduce the possibility of initial panel failures due to impact loading.



w = 6 ft

h' = h + 3 in (minimum of 6 in)

Figure 2.46: Transition from UTW to adjoining asphalt pavement.

3. Adequate geometric design of the intersection can avoid unnecessary load induced stresses like impact loads from the tires of large trucks as is shown in Fig. 2.40.

2.4 Okarche

2.4.1 Overview

This overlay is near Okarche, Oklahoma and starts approximately 8 miles north of Calumet on county road N2740 at the Canadian and Kingfisher County line and continues north for 5 miles. The project was designed by Russell Engineering Inc. of Oklahoma City and was constructed by Duit Construction in 2009. The thickness of the overlay is 5”. It is overlaid on chip sealed asphalt pavement without milling. The panel size is 6’ x 6’ in both north and south bound lanes. The total width of the new pavement is 24’. The width of old chip sealed asphalt pavement was 20’. The cross sectional details are as shown in Fig. 2.47.

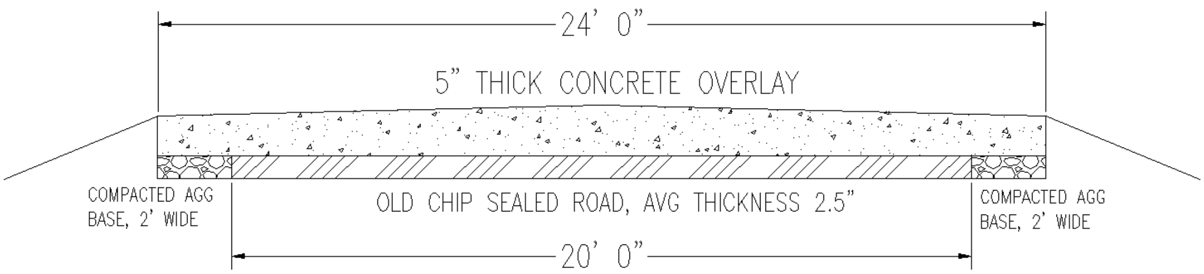


Figure 2.47 - Cross section showing the details of Okarche overlay.

In 2006 the ADT was observed to be 700. According to communication with Chris Westlund of ODOT (2010) about 10-15% of the total ADT is large truck traffic from local oil fields. Also, the overlay sees significant truck traffic during the wheat harvest in June. Figures 2.48 & 2.49 show the photos taken at the time of the construction of the overlay.

Figure 2.48 - Photos show the construction of the overlay.

Figure 2.49 - Photos show the construction of the overlay. (Photos in Figs 2.48 & 2.49 courtesy of Chris Westlund, ODOT).

2.4.2 Visual Inspection

A visual inspection was completed on June 22, 2010. The overall condition of the overlay looked excellent. This is not surprising since the overlay was just a year old.

Figs 2.50 & 2.51 show the beginning of the overlay on its south side. The initial panels, which from previous experiences are the most critical, look un-cracked.

Figure 2.50 - Photos a and b show approach panels in un-distressed condition. Elevation difference between leaving asphalt pavement and approaching concrete panels is almost zero.

Figure 2.51 - Photo shows the overlay from south direction (looking towards north).

Fig. 2.52 shows the condition of the overlay where it ends on the north side. The elevation difference in between concrete and asphalt pavements was thought to be insignificant. The good condition of the initial panels suggests that if the same elevation for approach and leave pavements is maintained, the initial panels on the overlay are less likely to fail assuming that there is good bond between the concrete and the underlying asphalt.

Figure 2.52 - North end of overlay. Photo a is taken looking towards south direction. Photo b is seven panel series from the end of the overlay.

Figure 2.53 - Photo showing the excellent condition of the overlay.

The overall condition of the thin white topping is excellent as shown in Fig. 2.53. The research team did not notice any longitudinal cracks, transverse cracks, fatigue distress or pop outs.

2.4.3 Falling Weight Deflectometer

On the day of inspection ODOT carried out FWD testing on the pavements. The FWD was used to determine load transfer efficiency (LTE) of the joints for the overlay. The FWD testing started at 8:30 am and ended at 11 am. The initial surface temperature of the pavement was recorded at 89 °F and the final temperature at 108 °F. The pavement temperature was recorded at each drop location. Tables 2.13 and 2.14 give the location of FWD drops and LTE. The GPS coordinates of the test locations were also recorded so that future testing can be carried out at the same coordinates. The FWD test was performed for the second and fourth mile of the project. The distance between two consecutive readings was 500', and readings alternated between the northbound lane and the southbound lane. In total, 10 readings were taken on each mile: five in the northbound lane and five in the southbound lane. Also, four additional readings were taken in the driving lane at the beginning and ending of the overlays. Fig. 2.54 shows the typical FWD drops locations for a mile and at the respective ends of the overlay.

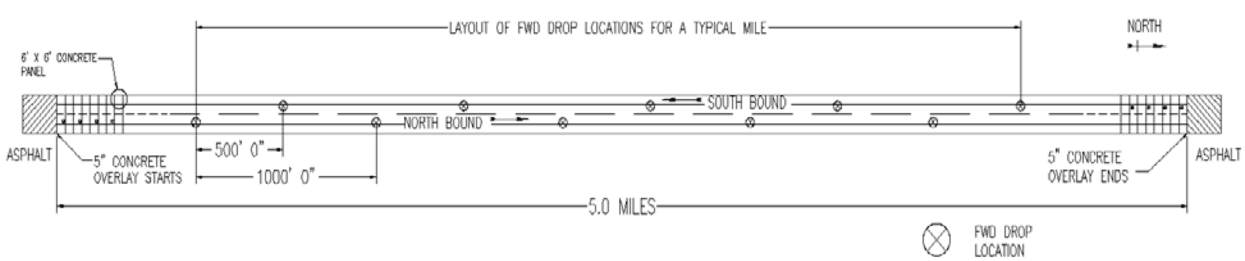


Figure 2.54 - Details of location of FWD drops.

Table 2.13 - Location of FWD drops and LTE for north bound lane.

Sr No	Test Location	Station No. As per ODOT records	GPS coordinates	Surface Temp in ° F	Load Transfer Efficiency, %
			NORTH BOUND		
1	First Panel where Overlay starts	10 + 55	N 35°43.531' W 98°07.104'	89.5	
2	Third Panel	10 + 67	N 35°43.533' W 98°07.106'	89	99
3	Fifth Panel	10 + 79	N 35°43.535' W 98°07.109'	90	95
4	Seventh Panel	10 + 91*	N 35°43.537' W 98°07.107'	90	114
5	At the start of second mile (NB)	66 + 80	N 35°44.537' W 98°07.111'	91	93
6	Panel at 0.2 mile of 2nd mile (NB)	76 + 90	N 35°44.703' W 98°07.110'	92	93
7	Panel at 0.4 mile of 2nd mile (NB)	86 + 75	N 35°44.868' W 98°07.109'	94	99
8	Panel at 0.6 mile of 2nd mile (NB)	96 + 50	N 35°45.019' W 98°07.109'	94	99
9	Panel at 0.8 mile of 2nd mile (NB)	106 + 00	N 35°45.175' W 98°07.106'	95	97
10	Panel at start of 4th mile (NB)	172 + 50	N 35°46.301' W 98°07.104'	100	91
11	Panel at 0.2 mile of 4th mile (NB)	183 + 65	N 35°46.474' W 98°07.107'	96	95
12	Panel at 0.4 mile of 4th mile (NB)	194 + 60	N 35°46.662' W 98°07.110'	97	95
13	Panel at 0.6 mile of 4th mile (NB)	205 + 25	N 35°46.830' W 98°07.108'	98.5	96
14	Panel at 0.8 mile of 4th mile (NB)	214 + 40	N 35°46.957' W 98°07.110'	97.5	96

Note: * = illogical data.

Table 2.14 - Location of FWD drops and LTE for south bound lane.

Sr No	Test Location	Reference No as per ODOT records	Location GPS coordinates	Surface Temp in ° F	Load Transfer Efficiency, %
SOUTH BOUND					
1	End Panel	273 + 92	N 35°47.862' W 98°07.110'	103	99
2	Third Panel from End	273 + 86	N 35°47.860' W 98°07.109'	103.5	88
3	Fifth Panel from End	273 + 80*	N 35°47.858' W 98°07.109'	103	114
4	Seventh Panel from End	273 + 74*	N 35°47.856' W 98°07.109'	103.5	17
5	Panel at start of 4th mile (SB)	218 + 65	N 35°46.904' W 98°07.112'	103	92
6	Panel at 0.2 mile of 4th mile (SB)	210 + 36	N 35°46.740' W 98°07.110'	103	90
7	Panel at 0.4 mile of 4th mile (SB)	200 + 40	N 35°46.565' W 98°07.108'	104	92
8	Panel at 0.6 mile of 4th mile (SB)	189 + 10	N 35°46.390' W 98°07.106'	107	95
9	Panel at 0.8 mile of 4th mile (SB)	178 + 50	N 35°46.200' W 98°07.112'	104	93
10	At the start of second mile (SB)	109 + 95	N 35°45.100' W 98°07.110'	104.5	92
11	Panel at 0.2 mile of 2nd mile (SB)	101 + 00	N 35°44.944' W 98°07.111'	105	93
12	Panel at 0.4 mile of 2nd mile (SB)	91 + 91	N 35°44.785' W 98°07.110'	106	91
13	Panel at 0.6 mile of 2nd mile (SB)	82 + 35	N 35°44.620' W 98°07.114'	107	94
14	Panel at 0.8 mile of 2nd mile (SB)	71 + 80	N 35°44.453' W 98°07.114'	108	92

Note: * = illogical data.

The majority of the joints showed excellent joint load transfers (more than 90 %). There were three exceptions. One joint showed very low load transfer and two joints showed more than 100 percent load transfer, which is not possible. The reasons for this abnormality in the readings are unknown.

2.4.4 Discussion:

Though the overlay is in good condition currently, there are some areas where the overlay may see some future problems. These are discussed as follows:

1) The original chip sealed road was just 20 feet wide. When it was overlaid with concrete it was widened to 24 feet. So there are extensions of 2 feet on both the west and east side of the overlay as shown in Fig. 2.47. The sub-bases of these concrete extensions were made with compacted aggregate base of thickness 4” to 6”. Though it may have been compacted, it was observed to look like loose gravel before placement of concrete (Westlund, 2010). It is probable that there is a difference in stiffness between the old chip-sealed road and the newly laid sub-base layer. So this interface may be a problematic area in the future. Also, it is seen that the compaction done was not sufficient where the concrete conduits were placed (Westlund, 2010). This area might also show distress if the existing compaction is not high enough. However, since a low load would be expected in the shoulder areas and the road sees a low volume of traffic, then the damage may not occur immediately.
2) The last 100 feet of the overlay on the north and south end of the project did not have the asphalt chip-sealed road as a sub base. The asphalt base was removed during milling to achieve the taper so that the overlay would tie in to the adjacent road elevations (Westlund, 2010). So it is possible that signs of future distress may be seen in these areas. During the inspection of the overlay it was observed that some of the panels showed a significant amount of vibration when trucks passed on an adjacent panel. This phenomenon was not observed in other areas of the overlay or any other overlay in this study. This observation suggests that the base may not be sufficient to support the overlay in this area or there is little bond between the overlay and the underlying layers.
3) During the construction the old asphalt pavement surface was not milled and may not have been accurately cleaned. This can be seen in Fig. 2.48. Therefore, it is possible that the asphalt and concrete may not have a good bond. If de-bonding occurs then one would expect a significant decrease in performance for the overlay.

3 Thin Whitetopping Design Manuals

In this section an overview is given of the major design manuals currently in use. The manuals coverd are the Colorado, PCA, ACPA, and Texas. Other design manuals were investigated by the research team but were not presented as they were typically improved upon by the design manuals presented. It should be noted that the University of Illinois has recently begun working on a new method for UTW design. The finalized work has not yet been finalized and so it was not included. This work may make modifications to the most recent ACPA work.

3.1 Colorado DOT

3.1.1 Introduction

To develop a design method, the Colorado DOT hired the CTLGroup to develop a new design procedure for thin white-topping (Sheehan et al. 2004). First a number of finite element models were developed to describe the performance of UTW. Four UTWs were then constructed and instrumented to monitor their performance based on these models. Three were evaluated in 1998 and one in 2004.

The objectives of the field testing were to identify the critical load location, the effects of the asphalt surface preparation, the response of UTW to traffic loading, the interface bonding strength, the effect of pavement age on load-induced stresses, and the calibration of design guidelines. Laboratory tests were conducted on compressive strength, modulus of elasticity, and flexural strength of specimens cast during the construction. Cores were used to measure thickness. Direct shear testing (Iowa 406C) was performed to determine the interface shear strength between the concrete and asphalt layers. Old test sections were revisited and were found to be in good condition. At the test locations visited, there was minimal or no faulting irrespective of the use of dowel bars. Some distresses were observed which were due to snow plows and at stop signals. A summary of the UTWs constructed for this study is presented in Table 3.1.

Table 3.1: Table giving the details of the parameters varied in the CDOT study.

Parameters	Range of Variation
Joint spacing	48, 72, and 144”
Concrete slab thickness	4.1” to 7.4”
Asphalt thickness	2.8” to 7”
Concrete elastic modulus	4000 ksi
Asphalt elastic modulus	350 and 800 ksi
Concrete Poisson’s ratio	0.15
Asphalt Poisson’s ratio	0.35
k-value	150, 225, 340 and 500 psi
Truck axle configuration	Single Axle Load and Tandem Axle Load
Slab loading locations	Corner & Edge

3.1.2 Field Instrumentation

The critical load location was determined by using a heavy loaded truck that was parked in different locations. The worst case loading was determined based on the highest strains measured for the slab. The effects of asphalt concrete surface preparation were studied by combining various surface treatments such as milling of existing asphalt, newly placed asphalt that is milled, existing asphalt pavement that is un-milled and newly placed asphalt which is un-milled. It was found that best performance was observed when the existing asphalt was milled. Joint spacing was changed over a wide range from 60”, 72” to 144”. Joints at 60” were observed to perform well. Temperature gradients were measured with the help of embedded thermocouples. Consequent curling and warping was measured with the help of the reference rods, and a falling weight deflectometer was used to test if the curling has any effect on joint load transfer efficiency.

Summary of field instruments used:

- embedded concrete strain gages
- embedded thermocouples
- reference rods to measure changes in the slab profile
- surface strain gages
- crack gages at joints
- falling weight deflectometer
- ground penetrating radar

3.1.3 Design Philosophy

A set of equations were developed using finite element software called ILLISLAB (ILSL2) and linear regression. Based on the findings from field instrumentation these equations were modified. Significant modification was made to several of the equations in 2004 based on the field instrumentation. The equations where modifications were needed included bond adjustment for concrete stress and asphalt strain and the temperature effect on concrete stress. The research team felt that the modification factors made the design equations better match the field performance.

3.1.4 Sensitivity Analysis

Sensitivity analysis was completed for the following parameters:

- asphalt thickness
- modulus of sub-base/sub-grade reaction
- asphalt modulus of elasticity
- concrete flexural strength
- expected number of 18-kip ESALs
- temperature differential

Other input parameters such as the material Poisson's ratio or existing asphalt fatigue damage were not considered as they were assumed to be less likely to affect the overlay performance. The panel spacing is assumed to be governed by the intersection geometry and avoiding placing a joint in the wheel paths.

The study showed that the concrete thickness design is not sensitive to asphalt thickness, but still CDOT recommends a minimum asphalt layer thickness of 5" (Goldbaum 2010). The concrete thickness was found to be sensitive to the subgrade/sub-base modulus. The sensitivity was found to decrease as the concrete thickness increases. It was also found to be sensitive to the asphalt modulus of elasticity, but as before, this sensitivity decreased as the concrete thickness increased. The concrete thickness is sensitive to the flexural strength of the concrete and hence a minimum of 650 psi of flexural strength was recommended. The concrete thickness required was not found to be sensitive to temperature gradients.

3.1.5 Design Procedure

The material properties needed for Colorado white-topping design procedure include

- asphalt modulus of elasticity
- asphalt thickness
- existing modulus of subgrade reaction
- concrete modulus of elasticity
- concrete modulus of rupture
- concrete and asphalt Poisson's ratio
- temperature differential
- trial concrete thickness and spacing
- existing asphalt fatigue
- expected repetitions of different axle categories

1. First the effective radius of relative stiffness for a fully bonded slab (l_e) and the joint spacing (L) are used to find the load induced stress for a 20-kip single axle loads (SAL) and/or 40-kip tandem axle loads (TAL). These stresses and strains are then converted into equivalent stresses and strains developed because of a range of different axle loads.

2. Calibration factors are then used for less than perfect interface bond, loss of support, and temperature induced stress variations to modify the calculated stresses and strains.
3. Fatigue analyses for concrete and asphalt layers are conducted separately. Portland Cement Association's concrete fatigue and the Asphalt Institute's asphalt fatigue equations are used for fatigue analyses.

Concrete thickness and joint spacing are determined so that they satisfy the fatigue failure criteria. It is then possible to reduce the thickness of the pavement and analyze the section again to get closer to the fatigue limit for that white topping. The report also provides a method to convert the conventional calculation of ESALs to numbers that would be more appropriate for white-topping ESALs. The damage caused by axle loads is a direct function of the overlay thickness. The original ESAL equation given in AASHTO is derived for an 8" concrete pavement and conversion factors are available for a 6" pavement thickness. However sections even thinner than 6" are sometimes used with white toppings and so new factors have to be used.

3.2 Portland Cement Association

3.2.1 Introduction

The Portland Cement Association sponsored the development of design guidelines for concrete overlays with a special emphasis on UTWs (Wu et al. 1999). The CTLGroup was again the agency responsible for the research. This research was based on the two test sections in Colorado used for the CDOT design guide and a test section constructed at an airport in Chesterfield in Missouri. Other case studies include an early UTW in Louisville, Kentucky, projects from Tennessee, and Georgia, and an experimental section in Sweden. Three test sections were constructed and instrumented in order to validate the equations. One section was constructed in a parking area at an airport in Chesterfield, Missouri in early 1995 and two sections were constructed in Denver and Longmont, Colorado in the summer of 1996. These are the same test sections which were used for the development of the CDOT design manual. However, the CDOT manual also included two more test sections, and the manual was revised in the year 2004. This document does not contain those modifications. A summary of the parameters varied in UTWs constructed for this study is given in Table 3.2.

Table 3.2: Range of parameters varied in the experimental sections for PCA study.

Parameters	Range of Variation	
	Chesterfield, Missouri	**Colorado Test sections**
Joint spacing	50" x 50"	60, 72, 120, 144"
Concrete slab thickness	3.6"	4, 4.5, 5, 6"
Asphalt thickness	3.1"	2.9 to 5"
Concrete elastic modulus	3400 ksi	3300-3700 ksi
Asphalt elastic modulus	715 ksi (at 41°F) 1730 ksi (at 77°F)	480.3 ksi (at 41°F) 1455 ksi (at 77°F)
Temperature Differential over the entire pavement	20.3 °F	22-31 °F
Truck axle configuration	10 kips and 12 kips Single Axle Load	18 kips and 20 kips Single Axle Load
Slab loading locations	Center, Corner & Edge	Center, Corner & Edge

The objectives of the field testing were to study white-topping pavement response under traffic loadings, to evaluate interface bond between the concrete and the asphalt layers, to verify the FEM model developed for analyzing composite pavement systems, and to calibrate stresses calculated by the FEM model for interface bonding strengths.

3.2.2 Field Instrumentation

Internal and external strain gages were used to measure the strains in the asphalt, concrete and surface of the pavement. Temperature changes were measured at different times of the day with thermocouples. The profile of the pavement was also determined by using a hand held measuring device. The following material properties were measured during the testing: compressive and flexural strength, modulus of elasticity for concrete and asphalt, indirect tensile strength, and the Iowa 406C shear test was performed for bond strength.

3.2.3 Design Philosophy

Prediction equations were developed using FEM analysis. A range of factors were considered based on the field data and theoretical analysis. The parameters considered are as shown in Table 3.3.

Table 3.3: Parameters considered for generation of prediction equation.

Parameters	**Range of Variation**
Square Concrete Slab Dimension	24” and 50".
Concrete slab thickness	2”, 3” and 4"
Concrete elastic modulus	4000 ksi
Concrete Poisson’s ratio	0.15
Concrete Unit Weight	150 pcf
Coefficient of thermal expansion for concrete	5.5 x 10^{-6}/°F
Asphalt thickness	3” to 9"
Asphalt elastic modulus	50 to 2000 ksi
Asphalt Poisson’s ratio	0.35
Asphalt Unit Weight	140 pcf
Coefficient of thermal expansion for asphalt	2.0 x 10^{-6}/°F
Slab loading locations	Joint and Corner
k-value	75 to 800 pci
Truck axle Load	18 kips for SAL and 36 kips for TAL
Temperature differential only for concrete	(+15, +5 and -10˚F)

Finite element models were developed to simulate the constructed concrete overlays. A parametric study was performed to consider the effects of various parameters on the composite pavement system. The parameters investigated include:

- location of load (center or edge)
- location of cracks in the asphalt and the impact on the pavement support
- load transfer between panels slabs
- bonding between the asphalt and concrete interface

Stresses were calculated at the top and bottom of the concrete layer and at the top of asphalt layer. Assumptions were made about the material properties based on the field studies. The design equations were then developed using multiple linear regression analysis and validated using the test data from field testing.

The prediction equations were developed for concrete stresses at the corner and asphalt strains at the bottom of the asphalt layer for the joint loading. A separate set of equations were developed for the effect of temperature at the locations for concrete maximum stress (at the corner) and asphalt maximum strain (at the bottom of asphalt layer at the joint). The models were then adjusted to take into account that there was only partial bonding between the different layers. This lead to a 36% increase in the estimated stresses in the overlay. The temperature stresses and strains were then added to these values to find the total stresses and strains. These stresses and strains were used to estimate the fatigue life of the composite pavement system and, if those criteria are satisfied, then the final design is established for the final UTW system.

The fatigue equations used were developed by PCA for concrete under corner loading and by Asphalt Institute for asphalt under joint loading. These are the same equations used in the Colorado Design Guide.

3.2.4 Sensitivity Analysis

The parametric study showed that the degree of bonding between the asphalt and concrete has the maximum impact on the stresses. Also the condition of the old asphalt plays an important role in defining the performance of the pavement. It was observed as the cracking increased in the asphalt layer that the load transfer efficiency was decreased. Therefore the group concluded that having sound asphalt under the overlay is important to the performance.

3.2.5 Design Procedure

The parameters required for the PCA design procedure are:

- asphalt modulus of elasticity
- asphalt thickness
- existing modulus of subgrade reaction
- concrete modulus of elasticity
- concrete modulus of rupture
- temperature differential
- trial concrete thickness and spacing
- expected repetitions of different axle categories

1. Using the design parameters, the depth of the neutral axis and the radius of relative stiffness (l_e) are calculated.
2. Using l_e, the modulus of subgrade reaction (k) and joint spacing (L), the load induced concrete stresses and asphalt strains are calculated for either 18 kip or 36 kip axles. These stresses and strains are converted into equivalent stresses and strains for other axle loads.
3. Temperature induced concrete stresses and asphalt strains are calculated at the locations where maximum values are found. These will be the same for both single and tandem axle loading conditions as they are not load dependent.
4. These stresses and strains are added together to get the total stresses and strains.
5. Fatigue analysis is carried out separately for asphalt under joint loading and for concrete under corner loading. It is necessary that both asphalt and concrete satisfy the fatigue failure criteria.

The design procedure is deemed to be first generation, hence long term performance data can enhance the procedures developed in this report for the design of Ultra-Thin White topping.

3.3 American Concrete Pavement Association

3.3.1 Introduction

This design manual is a combination of the PCA design manual with new information regarding materials, construction, and repairs for white toppings collected by ACPA (ACPA 1999). This document is organized to allow a designer to use tables to quickly arrive at a design for a white topping. Because of this simplification in the design procedures, the method does not allow for a large number of variables to be investigated. A range of different UTW projects have been considered for this report as shown in Table 3.4.

Table 3.4: Ultra-Thin White topping projects considered for the ACPA study.

Project No	Location (Date)	Concrete Thickness	Panel size	Asphalt Treatment	Fiber reinforced PCC
1	Kentucky, Louisville road to disposal facility (1991)	2”, 3.5”	2’x2’ , 6’x6’	milled	yes
2	Iowa Rt. 21 between Victor and Belle Plain (1994)	2”, 4”, 6”, 8”	2’x2’, 4’x4’, 6’x6’, 12’x12’	patch & scarify, patch only, cold in-place recycling	yes, no
3	Missouri, Spirit of St. Louis Airport (1994)	3.5”	4’x4’	lightly milled	yes
4	Colorado, Denver area, W. of Santa Fe, frontage rd. of S. Santa Drive (1996)	4”, 5”	4’x4’, 5’x5’, 5.5’x5.5’	milled, unmilled	no
5	Colorado, Denver area, Colo. 119 E. of Longmont (1996)	4”, 5”, 6”	6’x6’, 12’x12’	milled, unmilled	no
6	Colorado, SE corner of state, US 287, N. of Campo (1997)	6”	6’x6’, 8’x12’, 10’x12’, 12’x12’	milled	no
7	Virginia, McLean, FHWA Turner-Fairbank Research Center (1998)	2.5”, 3.5”	3’x3’,4’x4’, 6’x6’	milled	yes, no
8	Ohio, Lancester, Ohio University Accelerated Load Facility (1998)	3”	3’x3’	milled	yes, no

3.3.2 Design Philosophy

The design procedure was developed through the extensive field and theoretical analysis of white toppings by CTLGroup with additional calibration to field performance completed by ACPA. Theoretical analysis was done by finite element analysis using a combination of 2D and 3D models. Two types of pavement failure were considered: concrete fatigue under corner loading (prevalent UTW distress mode) and asphalt concrete fatigue under joint loading. Temperature induced stresses and strains were also included in the design. A parametric study showed that the interface bonding strength was one of the most important factors affecting load induced stresses in the pavement. The condition of the old asphalt pavement also played a major role in determining the stresses in the pavement. To be conservative, it is assumed that a considerable amount of asphalt fatigue life has been consumed before concrete placement. Field load testing data from the Missouri Airport overlay was used to validate the finite element models.

3.3.3 Design Procedure

According to the ACPA, a UTW is essentially used as a maintenance strategy. Hence the thickness of the concrete and asphalt can be determined by looking at the geometry available at the project. Thus the design guide has been designed to determine the load carrying capacity and expected service life based on these numbers. Instead of using equations, the designs have been produced in a tabular format. The variables used in the ACPA design are summarized in Table 3.5.

Table 3.5: Variables investigated for generating tables.

Description	Range of variation
UTW thickness	2”, 3” and 4"
Asphalt Thickness	3”, 4”, 5” and 6"
Joint Spacing	2’, 3’, 4’ and 6'
Design Flexural Strength	700, 800 psi
Modulus of Subgrade/ Subbase reaction (k- value)	100, 200 pci
Maximum Truck Axle Load	Category A (Low Truck volume Facility) 18 kips (SAL); 36 kips (TAL)
	Category B (Medium Truck volume Facility) 22 kips(SAL); 44 kips(TAL)

In order to determine the design thickness only four inputs are required. These inputs are:

- truck axle category
- subgrade / sub-base modulus (k)
- average flexural strength
- asphalt thickness

With these four inputs one can select the joint spacing or a thickness for the overlay. The joint spacing is recommended to be 12 to 16 times the overlay thickness. The tables also give the design fatigue life of the overlay. The report ends with instructions about materials to be used, construction and repairs for the overlays and a list of all the UTW and TWT overlays in the United States.

3.4 Texas DOT

3.4.1 Introduction

This work was completed by Dr. Moon Won at the University of Texas (Won et al. 2009). This study started with an in-depth review of the design procedures provided by Colorado, New Jersey, American Concrete Pavement Association (ACPA) and Portland Cement Association (PCA).

Full scale field testing was carried out to achieve the following objectives:

- Measurement of the response of instrumented white-topping pavement under static and repetitive traffic
- Establish the fatigue performance of white-topping pavement using full-scale fatigue testing data
- Calibration of the developed mechanistic design procedure to develop design guidelines

Cores and beams were used to determine the mechanical properties of the concrete. Falling weight deflectometer (FWD), Dynamic Cone Penetrometer (DCP) and Plate Load test (k-value test) were conducted to come up with the foundation properties.

3.4.2 Field Instrumentation

A full-scale concrete slab was constructed that was 18' x 18' x 6" with a 6' joint spacing. The slab was placed on 2" of four year old asphalt concrete pavement.

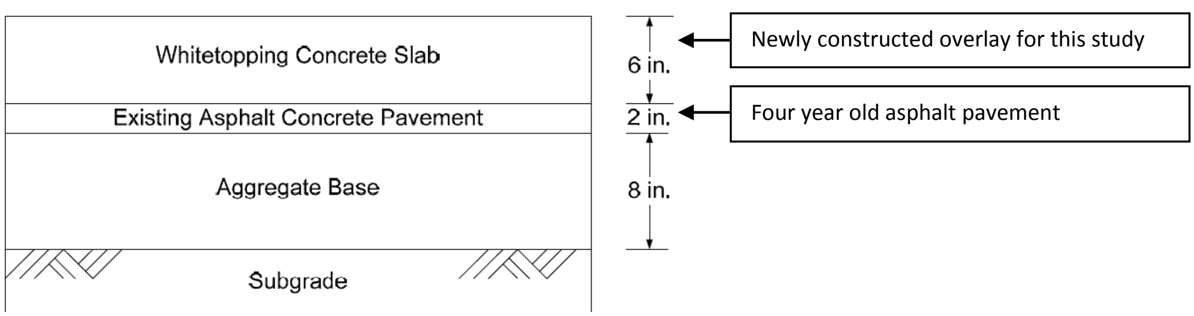


Figure 3.1: Cross section depicting the details of the test slab.

The test slab was instrumented with vibrating wire gages, dynamic strain gages, linear variable differential transformers (LVDTs), and a stress meter for monitoring slab behavior due to both environmental and traffic loading. Mechanical properties were determined from both cylinders (4" x 8") and beams (6" x 6" x 20") cast at the test location. The 28 day strengths were about 5.1 ksi for compressive strength, 650 psi for modulus of rupture, and 5400 ksi for modulus of elasticity.

The values from FWD and DCP data for the elastic modulus of underlying layers of AC, aggregate base and sublayer was quite variable. The primary reasons given for this variability include that there are a number of assumptions in the calculations one makes in order to determine the material properties from the FWD readings. Also, FWD is more effective at making measurements of materials at greater depths. In the current study the depth of the asphalt layer is only 2”, so it is possible that the FWD would give erroneous results. On the other hand, results from DCP made on the base material would be more reliable as it is a more direct method of measurement of foundation properties. Hence data from DCP was used to calculate the modulus of subgrade. The values from the DCP were then converted to California Bearing Ratios (CBR) values and then were converted into resilient moduli. However, only one relationship is given for DCP to resilient modulus conversion, in the field this might change as the type of soil changes.

The use of the plate load test gave more accurate calculations of the design values for the subgrade and base. This is because the k-value determination is more reliable than a method that uses back calculation. One interesting discovery by the research group was the relation between values from a popular evaluation layered program in pavement design called ELSYM5 and the measured values. Based on this information, the authors have suggested that the k-values should be reduced by 50% when estimating the modulus of subgrade reaction from a layered elastic program using the estimated modulus value of each layer.

Stationary Dynamic Deflectometer (SDD) was used for super accelerated testing as shown in Figure 3.2. The SDD is a mobile accelerated pavement testing device, which can apply sinusoidal loads at a range of load levels and load frequencies. It is actually a Rolling Dynamic Deflectometer (RDD) developed at UT that was used in a stationary mode. In this study, fatigue failure of a full-scale slab was defined as the occurrence of the first visible crack which was usually associated with the abrupt changes in the dynamic displacement.

Figure 3.2: Field Testing Arrangement for super accelerated load testing.

One of the slabs was tested until it failed. Then the other three slabs were tested to different stress levels (65, 80 and 90%) of the failed slab. Usually a corner crack was the source of failure. Apart from the effect of stress level, the fatigue behavior of concrete is highly influenced by the service stress level. The service stress level is the ratio of maximum flexural stress expected in the life of pavement, that is, the stress associated with the maximum axle load divided by the modulus of rupture of the concrete. A slight increase of this ratio decreases the fatigue life of concrete significantly. Some researchers have proposed a concept of equivalent fatigue life to overcome this problem (Shi et al. 1993). The equivalent fatigue life is defined as follows:

$$EN = N^{1-R}$$

where,
EN = equivalent fatigue life,
N = number of cycles to failure, and
R = applied minimum-to-maximum stress ratio.

The results from this testing were found to be close to the Thompson and Barenburg's S-N curve (S is cyclic stress and N is the logarithmic scale of cycles to failure) after the application of equivalent fatigue life concept (Thompson and Barenberg, 1992). The equation is given as:

$$\text{Log } N = -1.7136 \;\rule{2em}{0.4pt}\; + 4.284 \qquad \rule{2em}{0.4pt} \geq 1.25$$

$$\text{Log } N = 2.8127(\;\rule{2em}{0.4pt}\;)^{-1.2214} \qquad \rule{2em}{0.4pt} < 1.25$$

where,
N = number of load repetition to failure,
σ = maximum stress of white topping, and
MR = modulus of rupture of concrete.

The reliability of the fatigue equation that was used in this project does not seem appropriate to be used for all white toppings as it was derived for one type of white topping design and materials (6" of concrete and 2" of 4 year old asphalt and panel sizes of 6'x 6'). This equivalent fatigue life concept is deemed useful though. Also, the results of this fatigue equation gives values that are close to the values predicted with the Colorado Design Methodology fatigue equations.

3.4.3 Design Philosophy

It is the overall design philosophy of this work that the white topping will behave more like small independent slabs, where stresses due to bending or warping/curling are relatively small compared with those in other concrete pavement types. Failure modes and analysis methodologies were thought to be similar for overlays and traditional pavement types.

In this study the load induced concrete stresses is the most significant variable. The most important parameters are concrete overlay thickness, asphalt concrete layer thickness, thickness of base layer, elastic moduli of concrete, asphalt and base; k-value, load transfer efficiency and bonding conditions with type of traffic loads, namely single or tandem axle.

In the overall design philosophy, the spacing was kept constant at 6’ x 6’. This may be a good assumption for a thin white topping but does not seem appropriate for UTW, as when thicknesses are lower than 4” we need to reduce the panel size. Also the report does not address temperature or environmental stresses. These stresses may be significant for these pavements.

A finite element program, ISLAB2000 was used to model concrete pavement behavior under load. Model generation consisted of concrete topping, HMA, base, and subgrade. Each white topping slab has symmetrical 18’ x 18’ dimensions with 6’ joint spacing. The edge loading at the center panel was assumed for a maximum loading condition with a 20-kip single axle load (SAL) and 34-kip tandem axle load (TAL). The values of each variable selected in this study are summarized in Table 3.6.

Table 3.6: Summary of input variables used for ISLAB2000

Variable	Selected Input Levels	Number of Cases
Traffic Load	20-kips SAL and 34-kips TAL	2
Bond Conditions	Bonded, Unbonded	2
Thickness of Concrete Layer	4”, 6”, and 8”	3
Elastic Modulus of Concrete (million psi)	4 and 5	2
Thickness of Asphalt Layer	3”, 6”, and 9”	3
Elastic Modulus of Asphalt	200 ksi , 500 ksi, and 800 ksi	3
Thickness of Base Layer	5” and 10”	2
Elastic Modulus of Base	100 ksi and 1000 ksi	2
Subgrade Reaction Modulus , k-value (pci)	50, 100, and 200	3
Load Transfer Efficiency - LTE (%)	1, 50, and 99	2
	Total Number of Run	**7,776**

The ISLAB2000 calculation model generated a large number of data sets based on variable selections. To establish a statistically meaningful regression equation, a log-log expression was used for data analyses. One equation was finalized for the critical depth of the concrete as

$$\log(t_{PCC}) = 3.5615 + 0.1017 \times \log(ESALs) + 0.4982 \times \log(E_{PCC}) - 0.7232 \times \log(t_{AC}) - 0.3624 \times \log(E_{AC}) - 0.2695 \times \log(t_{BS}) - 0.0891 \times \log(E_{BS}) - 0.0287 \times \log(k) - 1.2250 \times \log(MR)$$

where,

t_{PCC} = required thickness of the white topping concrete, in,

ESALs = expected number of 18-kip ESALs,

E_{PCC} = concrete modulus of elasticity, psi,

t_{AC} = thickness of the asphalt layer, in,

E_{AC} = asphalt modulus of elasticity, psi,
t_{BS} = thickness of the base layer, in,
E_{BS} = base modulus of elasticity, psi,
k = modulus of subgrade reaction, pci, and
MR = modulus of rupture of white topping concrete, psi.

This equation was calibrated for no bonding condition between the concrete and asphalt. The equation from the CDOT manual in which stress for an unbounded condition is increased 1.65 times compared to a bonded condition was used. This prediction equation was also calibrated for fatigue behavior using Thompson and Barenburg's S-N curve as shown above.

Here three safe assumptions have been used in this analysis:

- no bond between the concrete and asphalt layers
- a 1 % LTE across joints
- an 18 kips axle is the standard load but a 20 kips axle was used in the study

3.4.4 Sensitivity Analysis:

A sensitivity analysis was conducted for the calculated white topping thickness. Design variables included the expected number of 18-kip ESALs, concrete elastic modulus, asphalt thickness, asphalt elastic modulus, base thickness, base elastic modulus, and modulus of subgrade reaction.

For most cases, white topping thicknesses are not sensitive to the number of expected load repetitions (ESALs), especially when projected ESALs exceed 2 million. The geometric property (thickness of each layer) of the white topping pavement is more sensitive to the white topping thickness compared to the material properties (elastic modulus of each layer and k-value) except for the white topping concrete strength (modulus of rupture). The asphalt thickness is more sensitive to the white topping thickness than the base thickness. The asphalt elastic modulus has the most influence on the required white topping thickness compared to other material properties. The elastic modulus of concrete has a slight impact on the white topping thickness while the base elastic modulus and k-value shows minimal sensitivity to the white topping thickness.

The subgrade reaction modulus (k-value) has the least effect on the calculation of the white topping thickness. Concrete flexural strength has one of the greatest impacts on the white topping thickness.

3.4.5 Design Steps

The design of the overlay should be carried out as per the following steps

1) Evaluate the structural condition and material properties using FWD and DCP.
2) Estimate subgrade resilient modulus using DCP by converting it to an equivalent CBR.
3) Use this modulus value to estimate the modulus of the subgrade reaction using a layered program such as ELSYM5. Divide this value by 2 to get the static modulus of subgrade reaction (k).
4) Use the resilient modulus obtained in Step 2 as an input value for a back-calculation program to estimate the resilient modulus of the sub-base layer.
5) Using design traffic and modulus values obtained in above steps, estimate the slab thickness in accordance with the prediction equation given above. The spacing is constant at 6' x 6'.

3.5 Discussion

Of the design methods presented they can be placed in two distinct design philosophies. The work by the CTLGroup was the foundation for the Colorado, PCA and ACPA. This work focused on using several theoretical equations to describe the stresses and damage accumulation for a whitetopping that were then tuned and updated based of field observations. These design methods are currently used by the Colorado DOT and have been performing satisfactory. The other method developed by the Texas DOT uses experimental data to better understand how the accumulation of damage impacts whitetopping performance. However the work goes on to make some very conservative assumptions on the load transfer and behavior of whitetoppings.

Based on the review in this document the research team would recommend using the latest Colorado design method as it has the most calibration from actual whitetoppings in the field.

4 ODOT Concerns and Questions

4.1 Overview

This section covers several questions that ODOT raised in the initial meeting for the project. With each document that was reviewed they were reviewed in hopes of finding insight into these questions.

4.2 How thin is it feasible to construct an UTW?

Illinois DOT does not recommend constructing a UTW below 3.5”. As per the interaction of the research team with the Colorado DOT they have successfully used a UTW that was 2” thick. As per personal interaction with Jay Golbaum of CDOT the 2” overlay saw severe damage immediately after the construction. Jay felt that the premature failure was due to the asphalt layer being too thin under the concrete layer.

On one of the overlays (US – 69) investigated by research team, it was seen that at one of the locations where cores were taken, the thickness of concrete was 4”. This overlay sees considerable truck traffic (almost 40% of 15000 ADT) and is showing satisfactory performance after 3 years of service.

In conclusion it seems that a minimum concrete overlay thickness of 3.5” is the lowest recommended concrete thickness for highways. A thinner section may be possible in areas where lighter traffic is expected.

4.3 What specifications are suggested for the concrete mixtures to be used in an UTW?

Typically a whitetopping is expected to be opened to traffic as soon as possible. Because of this high strength concrete mixtures are typically used as they will gain some minimum useable strength as soon as possible. The owner should be able to specify any workable concrete mixture which satisfies the condition of early strength gain and, acceptable amount of shrinkage. Common cementitious contents are typically between 500 and 650 lb/cy.

In the past shrinkage cracking has been a problem for high strength overlay mixes (Lin and Wang, 2005). A low cement content with low water to cementitious ratio is thus recommended. Blending of portland cement with one or more secondary cementitious material has been found to help in reducing the shrinkage of the mixture but this may also impact the strength gain. The use of optimized graded aggregates, plasticizers, and fibers have been important in past mixtures.

Since overlays have such a high surface to volume ratio the early age shrinkage of the concrete is of concern. To combat this some owners have specified a double layer of curing compound. The tensile strength of concrete plays an important role in the performance of the overlay. Typically plastic fibers have shown improvements in resisting plastic shrinkage cracks and steel fibers for cracking that occur after a few days of age (Roesler et al., 2008). Thus it is recommended to use plastic fibers in the concrete mixtures and perhaps steel fibers if the underlying asphalt is in poor condition.

Finally it is suggested that the whitetopping is sawed within the first twelve hours after the placement of concrete (Lin and Wang, 2005). This will ensure that the overlay does not induce the unnecessary curling stresses within the initial hours of setting of concrete mixture.

4.4 How can you insure a good bond between the interface of the concrete and asphalt?

The most common practice used to construct whitetoppings is to mill the asphalt before placing the concrete topping. Milling the surface improves bond because it exposes the fractured aggregate and creates a rough surface essential to the development of mechanical bond. Milling also helps remove any rutting in the existing asphalt surface and restores the proper grade and cross slope. If the surface is highly distressed, patching may need to be done prior to milling. A clean surface is paramount for proper bond. Cleaning of the surface of the asphalt by either the use of a broom or a low pressure wash has been successful. Once a surface is cleaned it is extremely important to keep it clean until paving commences. If the surface is cleaned more than a few hours prior to paving, cleaning with compressed air may be required again just before paving in order to remove any dust, dirt, or debris falling or blowing onto it. If traffic is allowed on the milled surface, the surface must be cleaned again before paving. No specification was found that specified a depth of milling or any quantifiable parameter for the milling process.

5 Conclusions

In this report a summary is provided of the inspection of 9 whitetoppings and one unbonded overlay in the state of Oklahoma, in addition 4 whitetopping design methods were reviewed, and several ODOT questions were addressed.

The findings from the field inspection indicate that the whitetoppings can be an effective pavement rehabilitation technique for highways and intersections that experience significant truck traffic. One of these has showed satisfactory performance for over 9 years. This whitetopping had a 4” thickness with 6’x6’ panels in the passing lane and used a 6” thickness with 6’ x 7’ panels in the driving lane. However this whitetopping was used on top of over 10” of asphalt. This thickness of asphalt is not common to have available to use in a whitetopping application. According to the design procedures the overlay performance is dependent on the thickness and quality of the asphalt. Most design procedures show that little benefit in performance is obtained after 6” of asphalt are used.

The primary areas of distress for the whitetoppings were either discrete corner cracking or failures at the interface of the whitetopping with the adjacent pavement. There failures were fairly common when there was a significant difference in elevation. It is assumed by the research team that these height differences (as much as ¼” in some cases) were not present at the construction of the whitetopping and have developed over years of service. The research team feels that these failures can likely be prevented if improved details can be used to stiffen these areas to minimize differential deflection during service. More work is needed to determine what types of details would be beneficial. However if this could be done it would reduce the maintenance needed for whitetoppings.

Although the whitetopping used on Okarche road is currently in good condition the research team shows concern for future performance of this overlay. The road has been widened by 2’ in each direction over the existing asphalt road. This additional 2’ on each side was cast directly on compacted aggregate base. This is concerning as there should be little bond between these materials. However since these areas in the shoulder they might not receive direct loading. Another area of concern is that based on the pictures of the construction provided by Chris Westlund of ODOT it does not appear that the asphalt surface was cleaned before the concrete overlay was placed. This could be significant as this area needs to be clean and free of debris to insure a good bond between the underlying asphalt and the concrete. In addition the last 100’ of the overlay on the north and south end of the project was not cast on asphalt but instead existing fill. While the research team visited the site it was noticed that there was significant vibration in the panels over this last 100’ when a truck traveled on the roadway. These vibrations were not noticed on the portions of the overlay that were founded on asphalt on this same overlay or on any other overlay that was inspected for this project. This vibration implies that there is either poor bond with the surface under the concrete and so the thin overlay is showing more bending in adjacent sections when it is loaded. This puts more stresses in the concrete overlay and should lead to an accelerated deterioration when compared to the other portions of the overlay.

Of the whitetopping design procedures that were reviewed for this project it was determined that the updated Colorado design procedure from 2004 gets the highest recommendations. While this method does make some suspect assumptions it has seen the heaviest calibration from field data and according to conversations with the Colorado DOT it does a good job predicting field performance.

Finally the some answers are given to several ODOT questions in the final section of the document that address typical concrete specifications, insuring good bond, and minimizing the thickness of a whitetopping.

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Appendix – A

The following formulas are used for converting the Dynamic Cone Penetration (DCP) readings to a resilient modulus. Conversion of DCP to California Bearing Ratio (CBR) is done by using the formula from the TxDOT design manual. The further conversion of CBR to resilient modulus (M_R) is done by using formula from NCHRP design guide 1 – 37A.

$$\mathbf{CBR = 292/DCP^{1.12}}$$

where, CBR is California Bearing Ratio
DCP is DCP index in mm/blow.

$$\mathbf{M_R = 2550 \times (CBR)^{0.64}.}$$

This expression has been shown to work well for both clays and granular materials.

The results of this expression were tested against the results given by conventional $\mathbf{M_R = 1500 \times CBR}$**,** and were found to give conservative results and this method was used for this study. The tables below give the MR values for the pavements tested with DCP. The variation among the individual values is quite alarming and hence it is recommended to use the data presented in Appendix – A with caution. Because the number of tests is low it may be difficult to draw a strong conclusion.

Table A 1: Overlay 1: MC-161N(181) SB

Location	DCP index (mm/blow)	California Bearing Ratio (CBR)	Resilient Modulus (psi)
1	9.75	22.78	18860
5	17.83	11.59	12230
3	8.85	25.40	20210
7	11.125	19.65	17160
		Average	**17115**

Table A 2: Overlay 2: MC-161N(132) SB

Location	DCP index (mm/blow)	California Bearing Ratio (CBR)	Resilient Modulus (psi)
5	9.02	24.86	19940
2	11.08	19.75	17210
8	65	2.72	4840
		Average	**13997**

Table A 3: Overlay 3: SSR-103N(084)SR NB

Location	DCP index (mm/blow)	California Bearing Ratio (CBR)	Resilient Modulus (psi)
2	27.86	7.03	8880
7	13.24	16.18	15140
3	26.04	7.58	9325
9	7.32	31.4	23160
		Average	**14126**

So the average values for the three overlays fall in the 14000- 17000 psi range. However, it is difficult to say if these numbers apply to any areas besides the one mile stretch sampled for this project.