Trinidad Lake Asphalt Overlay Performance Final Report

WA-RD 710.2

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July 2014

Washington State Department of Transportation
Office of Research & Library Services

WSDOT Research Report

Experimental Feature Report

Final Report
In-House Experimental Feature

TRINIDAD LAKE ASPHALT OVERLAY PERFORMANCE FINAL REPORT

Contract 6441
SR-16
Tacoma Narrows Bridge – Eastbound
MP 7.28 to 8.41

Washington State Department of Transportation

Engineering and Regional Operations
Construction Division
State Materials Laboratory

1. REPORT NO.	2. GOVERNMENT ACCESSION NO.	3. RECIPIENT'S CATALOG NO.
WA-RD 710.2		
4. TITLE AND SUBTITLE Trinidad Lake Asphalt Overlay Performance Final Report		5. REPORT DATE July 2014 6. PERFORMING ORGANIZATION CODE In-House
7. AUTHOR(S) Keith Anderson, Mark Russell, Jeff S. Uhlmeyer, Dave Luhr, Bryan Dias, and Jim Weston		8. PERFORMING ORGANIZATION REPORT NO.
. PERFORMING ORGANIZATION NAME AND ADDRESS Washington State Department of Transportation Materials Laboratory, MS-47365 Olympia, WA 98504-7365		10. WORK UNIT NO. 11. CONTRACT OR GRANT NO. Contract 6441
12. SPONSORING AGENCY NAME AND ADDRESS Washington State Department of Transportation Transportation Building, MS 47372 Olympia, Washington 98504-7372 Project Manager: Kim Willoughby, 360-705-7978		13. TYPE OF REPORT AND PERIOD COVERED Final 14. SPONSORING AGENCY CODE
15. SUPPLEMENTARY NOTES		

16. ABSTRACT

Construction of the new Tacoma Narrows Bridge (TNB) included a steel orthotropic bridge deck. The higher flexibility of an orthotropic deck causes pavement placed upon it to fatigue and crack more quickly than pavement placed on a normal roadway. The HMA overlay placed on the new bridge incorporated Trinidad Lake Asphalt (TLA) to help resist the stresses of an orthotropic deck.

The performance of the overlay after eight years of traffic was disappointing. Problems with the mix design, achieving specified densities, temperature differentials, and issues with the paver resulted in a pavement with a higher than desired void content. The result was severe rutting caused by raveling in the wheel paths possibly exacerbated by wear from studded tires. Cracking and delamination, the initial concern which prompted the use of the TLA modified HMA, were not a problem on the bridge deck.

As a result of the severe raveling, consideration is being given to the use of a conventional HMA when replacement of the deck wearing surface becomes necessary.

17. KEY WORDS	18. DISTRIBUTION STATEMENT
Tacoma Narrows Bridge, Trinidad Lake Asphalt, orthotropic bridge deck	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22616

19. SECURITY CLASSIF. (of this report)	20. SECURITY CLASSIF. (of this page)	21. NO. OF PAGES	22. PRICE
None	None	76	

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Introduction

The opening of the second Tacoma Narrows Bridge (TNB) in July of 2007 marked the use of two transportation technologies new to the State of Washington. The new bridge was the first in the state constructed with a steel orthotropic deck in place of a traditional concrete deck. Large bridges in other parts of the world routinely incorporate orthotropic decks to reduce weight and lower cost. The drawback is the higher flexibility of the steel deck causes pavement placed upon it to fatigue and crack more quickly than pavement placed on a normal roadway. This leads to the second new technology, Trinidad Lake Asphalt (TLA), which was added to the hot mix asphalt (HMA) overlay on the new bridge. TLA is a naturally occurring asphalt binder used to increase the durability and stability necessary for a pavement to withstand the stresses on an orthotropic deck. This is the final report on this project. It includes background information on orthotropic bridge deck overlay construction practices and documents the construction of the overlay on the TNB. In addition, it includes a summary of the seven year performance of the overlay and future recommendations for use of this process. The location of the bridge is shown in Figure 1 and the layout of the deck is shown in Appendix A (Figure 63).

Figure 1. Tacoma Narrows Bridge vicinity map.

Orthotropic Bridge Deck

The word orthotropic is derived from the words orthogonal anisotropic, meaning different elastic properties in perpendicular directions. An orthotropic bridge deck is one in which a steel deck plate is supported by longitudinal ribs and transverse crossbeams (1). The different geometries of the ribs and crossbeams give the deck different flexural stiffness in the transverse and longitudinal directions making it orthotropic. Figure 2 is a schematic of a section of an orthotropic bridge deck.

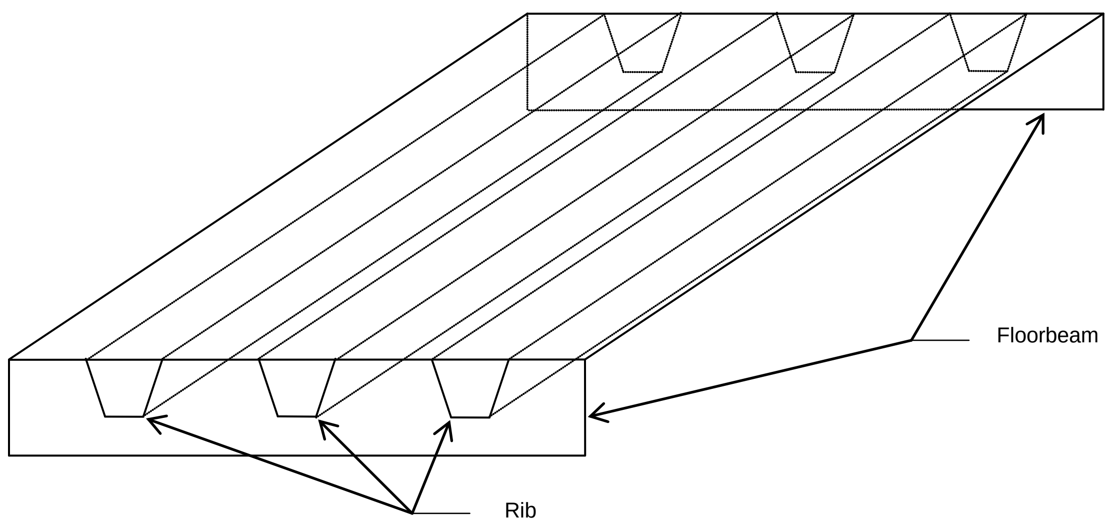

Figure 2. Schematic of an orthotropic deck with closed ribs.

The primary advantage of an orthotropic bridge deck is that it is lighter than a traditional concrete deck which reduces the total dead load carried by the rest of the structure. The reduced dead load allows the towers, cables, and other supporting members to be smaller reducing overall cost. The reduction in dead load comes at a price. Deflections due to traffic loading in the relatively thin steel deck plate are much greater than on a concrete deck. The greater deflections translate into higher strains in the pavement which leads to reduced life due to fatigue in traditional HMA overlays.

Pavement Properties

Pavements most often used on orthotropic bridge decks can be divided into three main categories. They include mastic asphalt, HMA with a modified binder and epoxy asphalt. Regardless of the type of system used, the pavement should be designed for the following properties (2):

Lightweight – The pavement has to be lightweight or the primary reason for choosing an orthotropic bridge deck, lower dead load, will be negated.

Impervious – Water has to be kept away from the steel deck in order to prevent corrosion.

Stable – The pavement needs to be able to resist plastic flow in order to prevent rutting and shoving.

Flexible – The pavement must be flexible in order to resist fatigue cracking due to the higher strains inherent with an orthotropic deck.

Skid Resistant – Enough friction must be provided by the pavement to prevent skidding.

Durable – The pavement must be able to stand up to the environmental conditions it is exposed to.

Smooth-riding – A smooth driving surface must be provided by the pavement.

To attain these properties the typical deck overlay system consists of four layers: the bonding layer, the isolation layer, the adhesion layer, and the wearing course. According to Medani the purpose of these layers are as follows (3):

Bonding layer - Binds the overlay system to the steel deck. It must provide a strong bond and protect the steel against corrosion.

Isolation layer - Transfers the loads from the wearing course to the much stiffer steel deck. To do this it must be flexible and resistant to fatigue. It must also be able to keep moisture from reaching the steel.

Adhesion layer - Binds the wearing course to the layers below.

Wearing course – The wearing course needs to be able to sustain traffic loads and provide a smooth and safe driving surface.

Paving Systems

Three types of asphalt based overlay systems have been used successfully on orthotropic bridge decks; (1) mastic asphalt, (2) HMA with modifiers, and (3) epoxy asphalt. Each type is discussed below.

Mastic Asphalt

Mastic asphalt has been successfully used on many orthotropic bridge decks in Europe. It is durable and provides a long service life on orthotropic decks. Mastic asphalt is impermeable to moisture and provides a good bond with other layers. However, it has a poor skid resistance, has a higher tendency toward plastic deformation and is more difficult to apply (3, 4).

Mastic asphalt consists of an asphalt and aggregate mixture with overfilled voids so that the aggregate is suspended within the pavement. The lack of aggregate on aggregate contact requires that the binder provide the stability instead of the aggregate. This calls for binders that are stiffer than normally used with hot mix asphalt to provide additional stability. TLA is often added to increase the stability of the binder.

A mastic asphalt overlay system was used on the Forth, Severn, and Humber bridges in the United Kingdom. The bonding layer consisted of a zinc primer applied directly to the steel deck plate followed by a 0.02 to 0.04 inch thick elastomeric adhesive. A 0.12 inch layer of rubberized asphalt applied on top of the elastomeric adhesive served as the isolation layer. The mastic asphalt acted as both the adhesion layer and wearing course bringing the total thickness of the surfacing to about 1.6 inches (5).

HMA with Modifiers

HMA has been used in both France and the United States. In order to perform satisfactorily on orthotropic bridge decks, modified binders are necessary to increase fatigue resistance. HMA has been used alone or in combination with an underlying course of mastic asphalt. It has the advantage of rapid placement with conventional paving equipment but is subject to fatigue cracking when used with unmodified binders. Modifiers used on orthotropic bridge decks include styrene butadiene styrene (SBS) and ethyl vinyl acetate (3).

The Millau Viaduct in France is a recent example of the use of modified HMA on an orthotropic bridge deck. A bonding layer consisting of a bituminous primer followed by a 0.12 inch bituminous sealing sheet acted as the isolation layer. The wearing course consisted of 2.4 inches of HMA with an SBS modified binder (6).

Epoxy Asphalt

Epoxy asphalt has been used in the USA, Canada and China. Epoxy asphalt is essentially an HMA mixture with the asphalt replaced by a two part epoxy resin. It has the advantage of easy placement with conventional paving equipment allowing it to be placed quickly and achieve a better ride quality than mastic asphalt. Epoxy asphalt is very stable, resists cracking and bonds well to the underlying layer; however, its one disadvantage is a cure time which is dependent on temperature. If conditions are good it can cure within a few hours, however, cooler temperature can extend the cure time which can result in cracking if traffic is allowed on the surface too early (4, 5). A two inch epoxy asphalt overlay was placed on the San Mateo – Hayward Bridge in California in 1969. The overlay is performing well with only a few fatigue cracks visible as of 2002 (7).

Tacoma Narrows Bridge Overlay Design

HMA with a modified binder was chosen for the orthotropic deck on the TNB. The system consists of five layers with the first three (the bonding, isolation and adhesion layers) comprised of a bridge deck waterproofing membrane system manufactured by Stirling Lloyd. The HMA overlay system made up the remaining two layers. The bottom layer or base course consisted of a sand HMA and the top layer or course consisted of a dense graded HMA, both modified with Trinidad Lake Asphalt.

Three Part Bridge Deck Waterproofing Membrane System

Sterling Lloyd's Eliminator® three part bridge deck waterproofing system was used to bond the overlay to the deck and to protect the steel. The first layer consisted of a rust inhibiting acrylic-based prime coat. The specifications required that the prime coat achieve bond strength of 290 psi in order to secure the overlay to the steel. The second layer was two applications of a methyl methacrylate waterproofing membrane spray applied 50 mils thick resulting in a total of thickness 100 mils. A polymer modified bituminous hot melt adhesive applied by hand with squeegees at a rate of 25 – 35 square feet per gallon made up the final layer.

Base Course

The base course consists of a No. 4 nominal maximum aggregate size (NMAS) gradation HMA with a binder made up of a blend of PG 64-22 asphalt, TLA and SBS polymer. The polymer was to be added to the PG 64-22 binder at a rate of 3.0 percent but was changed to 1.5 percent at the request of the asphalt supplier, U.S. Oil. Testing by U.S. Oil indicated the asphalt could not be produced with 3.0 percent SBS polymer (8). The final blend consisted of 60 percent of the polymer modified PG 64-22 asphalt and 40 percent TLA.

The project specifications required the mix to achieve an air void content of 0.5 to 1.0 percent after 75 blows in a six inch Marshall mold. Table 1 shows the gradation and asphalt content for the base course.

Table 1. Base course gradation and asphalt content requirements.

Sieve	Percent Passing		
	Control Points	Proposed Mix	Tolerance Limits
3/8	100	100.0	93-100
No. 4	95-100	98	91-100
No. 8	71-79	77	73-81
No. 16	59-67	52	48-56
No. 30	47-55	35	31-39
No. 50	25-35	24	20-28
No. 100	14-22	17	15-19
No. 200	12-16	12.1	10.1-14.1
Binder %	10.0-11.2	10.8	10.5-11.1

The second column lists the control points from the original specifications. The control points represent the allowable range when developing the gradation for the mix design. The third column shows the proposed mix design. The proposed gradation was developed from tests of aggregate from pit site B-333. The gradation results from pit B-333 did not fall within the control points for the No. 16, No. 30 and No. 50 sieves so the specifications were revised to accommodate the new gradation. The fourth column shows the tolerance limits based on the new gradation which represent the range in which the production gradation results must fall to be within specification.

In addition to gradation requirements, aggregates used for TLA had to meet the following requirements:

- Natural sand is not allowed.
- The sand equivalent (SE) shall be 40 or above.
- Ninety-seven percent of the aggregates retained on the U.S. No. 4 sieve shall have a minimum of two fractured faces.
- The uncompacted void ratio shall be at least 45%.

Table 2. Base course mix design testing.

Mix Property	Mix Testing			Approved Mix Design
P_b	10.0	10.5	11.0	10.8
Percent V_a	2.3	1.9	1.5	1.7
Percent VMA	18.6	19.4	19.9	19.7
Percent VFA	87.7	90.2	92.5	91.6
Dust/Asphalt	1.519	1.407	1.342	1.368
P_{be}	7.966	8.602	9.016	8.850
G_{mm}	2.379	2.358	2.349	2.353
G_{mb}	2.324	2.314	2.314	2.314
G_{se}	2.710	2.701	2.708	2.705
Stability (lbs.)	6,990	6,395	6,395	6,395
Flow (0.01")	29	37	45	41

The proposed gradation was combined with 10.0, 10.5 and 11.0 percent asphalt binder contents and tested to determine the mix properties shown in Table 2. The specified 0.5 to 1.0 percent air void content could not be achieved using the proposed gradation without increasing the asphalt content percentage beyond the allowable limit. Therefore, a revision to the specification to allow the greater air void percentage was approved. The binder content chosen for the production mix was 10.8 percent. The fifth column shows the estimated properties for the 10.8 percent binder content.

Top Course

The top course consisted of a 1/2 inch NMAS gradation with a binder made up of a blend of PG 64-22 asphalt, TLA and SBS polymer. The polymer was to be added to the PG 64-22 binder at a rate of 3.0 percent but was changed to 1.5 percent for the same reason as the base course binder. The final blend consisted of 75 percent of the polymer modified PG 64-22 asphalt and 25 percent TLA. The project specification required the mix to be designed to achieve an air void content of 3.0 to 5.0 percent after 75 blows in a six inch diameter Marshall mold. Table 3 shows the gradation and asphalt content for the top course.

Table 3. Top course gradation and asphalt content requirements.

Sieve	Percent Passing		
	Control Points	Proposed Mix*	Tolerance Limits*
3/4	100	100	100
1/2	95-100	95	88-100
3/8	76-86	87	80-94
No. 4	45-57	53	46-60
No. 8	41-49	43	39-47
No. 16	-	33	29-37
No. 30	29-35	26	22-30
No. 50	16-22	20	16-24
No. 100	9-13	14	12-16
No. 200	5-9	8.1	6.1-10.1
Binder %	5.0-5.6	5.6	5.3-5.9

*From CTL production test reports

The aggregate for the top course was required to meet the same additional requirements as the base course. The proposed JMF for the top course did not fall within the control points for the 3/8, No. 30 and No. 100 sieves. Apparently this change was also approved resulting in the

JMF and tolerance limits which are outside the control points but no documentation of the change is available. Testing performed in developing the mix design was not available for the wearing course.

Tacoma Narrows Bridge Overlay Construction

The next section covers the construction of the Trinidad Lake Asphalt HMA overlay of the bridge. The paving of the overlay was preceded by the construction of calibration strips at an off-site location followed by test strips on the bridge itself designed to assure that materials and methods were optimized to produce the best possible pavement.

Calibration Strips

Prior to actual paving on the bridge, calibration strips were paved at Woodworth & Company's asphalt plant facility in Lakewood, WA. The purpose of the calibration strips was to calibrate the nuclear density gauges and to show that the contractor's methods and equipment could achieve the required results. At least one calibration strip was specified for both the base course and top course, but more could be required if the Engineer determined that they were necessary.

Calibration Strip Requirements

The calibration strip was required to consist of a 5/8 inch thick steel plate at least 10 feet wide and 20 feet long to simulate an orthotropic bridge deck. Preparation of the plate was to be identical to the actual bridge deck consisting of sand blasting the steel to a near white condition then applying the three component waterproofing deck seal.

Calibration of the nuclear density gauges involved taking density readings in five locations on the plate after each lift. Readings were to be taken for the following conditions:

- When the plate is at least 16 inches off the ground.
- Over a simulated orthotropic rib.

Cores were taken from the simulated orthotropic bridge deck at each of the five gauge reading locations to determine the bulk specific gravity of the HMA. The bulk specific gravity of the core divided by the average gauge reading determined the correlation factor for the gauge. The steps were repeated for the top course.

Calibration Strip Setup

The simulated bridge deck was made out of six 5/8 inch thick steel plates eight feet wide by twenty feet long. The plates were butted together to form a 24 feet wide by 40 feet long simulated orthotropic bridge deck. Woodworth and Company constructed a 25 foot wide HMA over crushed rock roadway on both sides of the simulated bridge deck. The roadway extended approximately 250 feet ahead of the bridge deck and approximately 100 feet beyond. The calibration strip ran in a more or less north and south direction with paving on all calibration strips starting at the south end. To simulate the bridge deck, the steel plates were placed over an approximately two foot deep pit and supported by timber blocking. Blocking consisted of 4 X 6's placed longitudinally at 17.5 inch spacing. The 4 X 6's rested on 8 X 10 transverse beams spaced at 3 feet 7-5/8 inches. Supporting the beams were four 10 X 24 timbers resting on the bottom of the pit. The steel plates were coated with the waterproofing deck seal as required but simulated ribs were not used.

Figure 3. Calibration strip looking south.

Figure 4. Simulated orthotropic bridge deck.

Calibration Strip Paving

A total of five calibration strips were constructed. Paving on all of the strips started at the south end and proceeded north on the westernmost half of the strip. The transfer vehicle and the truck operated on the eastern side of the strip while paving the west half of the strip. After paving the western half of the strip the paving train reversed direction and paved the eastern side of the strip with the transfer vehicle and truck operating in the same lane as the paver. Delivery trucks carried partial loads with tight tarps except on the first strip where loose tarps were used. Trucks were driven on local roads for about 30 minutes to simulate the haul to the bridge. A CSS-1 tack coat was placed between all lifts of HMA. No tack coat was applied over the steel plates prior to the first lift. Each strip took 10 to 30 minutes to pave in both directions. Table 5 shows the date and course paved for each calibration strip.

Table 4. Calibration strip construction dates.

Strip No.	Date	Course
1	May 25, 2007	Base
2	May 25, 2007	Top
3	May 29, 2007	Base
4	May 29, 2007	Top
5	May 31, 2007	Base

Membrane Placement

Overlay construction on the bridge began by placing the three part deck protection membrane system. To prepare the surface for membrane application the steel deck plates were cleaned to a near white surface using a Blastrac 2-4800 DH shot blaster. Membrane application proceeded with roller application of the prime coat followed by spraying two coats of the methyl methacrylate waterproofing membrane. Completion of the membrane system consisted of spreading the polymer modified bituminous hot melt adhesive over the methacrylate waterproofing membrane with squeegees. On the morning of paving, areas of the polymer modified bituminous hot melt adhesive that had been damaged by equipment were repaired by applying additional adhesive and the entire deck surface was cleaned with hand brooms to prevent further damage from equipment.


Figure 5. Blastrac 2-4800 DH shotblaster.


Figure 6. Steel deck after shotblasting.


Figure 7. Applying acrylic based prime coat bonding layer.


Figure 8. Applying methyl methacrylate isolation layer.

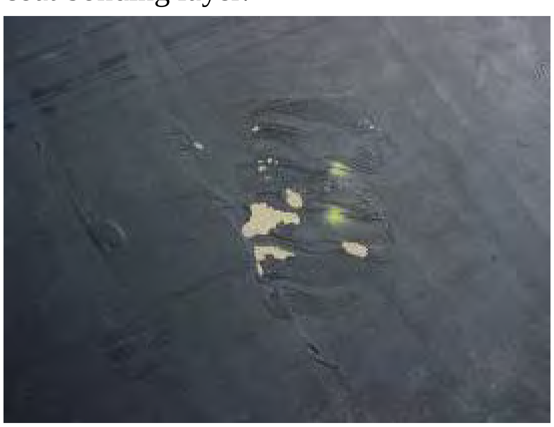
Figure 9. Damage to adhesion layer.


Figure 10. Repairing adhesion layer.

Paving Equipment

Paving was accomplished using an Ingersoll-Rand/Blaw-Knox PF-4410 paver (Woodworth equipment number 703) and a Terex/Cedarapids CR 662RM Road Mix transfer vehicle. Inspection of the paver revealed that it was missing a section of the mix management kit (MMK). The MMK is a retrofit which consists of a series of chains that help to contain the mix in front of the screed in order to prevent segregation. Without the MMK retrofit, less mix is distributed to the area in front of the gear box which can leave a segregated streak in the center of the mat. It is not clear if the missing section contributed to the streaks in the mat (See Streaks in Pavement). From June 21 onward, the paver was replaced by another Blaw-Knox PF-4410 paver (Woodworth equipment number 710) with an intact mix management kit.

Figure 11. Ingersoll-Rand/Blaw Knox PF-4410 paver.


Figure 12. Terex/Cedarapids CR662RM RoadMix transfer vehicle.


Figure 13. Missing chains on MMK retrofit on I-R/Blaw-Knox PF-4410 paver.


Figure 14. Missing chains on MMK retrofit.

The mix was delivered using single unit (no trailer) end dumps with the loads tightly tarped to prevent loss of heat from the mix. The mix was end dumped directly into the transfer vehicle.

Different combinations of rollers were used on each pass. All rollers were used in static mode only. The roller models used on one or more paving passes are listed in Table 5.

Table 5. Roller descriptions.

Manufacturer	Model	Type	Weight (lb.)	Width (in.)
Ingersoll-Rand	D-24	Double Drum Vibratory	5,000	48
Ingersoll-Rand	DD-28HF	Double Drum Vibratory	6,000	48
Ingersoll-Rand	DD-31HF	Double Drum Vibratory	7,000	48
Ingersoll-Rand	DD-110HF	Double Drum Vibratory	25,000	78
Ingersoll-Rand	DD-130	Double Drum Vibratory	28,000	84

The rollers can be divided by size and weight into two categories. The model D-24, DD-28HF and DD-31HF with weights of 5,000 to 7,000 lbs. and approximately 48 inch drums are the smaller rollers. The model DD-110HF and DD-130 with weights of 25,000 and 28,000 lbs. and 78 and 84 inch drums are the larger rollers. The Contractor used the rollers within each category interchangeably but did not substitute a large roller for a small roller after establishing their pattern.

Paving

HMA Placement

HMA placement on the TNB began on June 1, 2007 with paving of the job control strip for the base course. The purpose of the job control strip is to verify that the equipment and methods achieve the desired compaction and level of quality to allow paving to go into full production. A control strip is required for both the base course and the top course. Additional control strips are required if the job mix formula changes or if there is a change in material. Figure 15 shows the base course paving sequence.

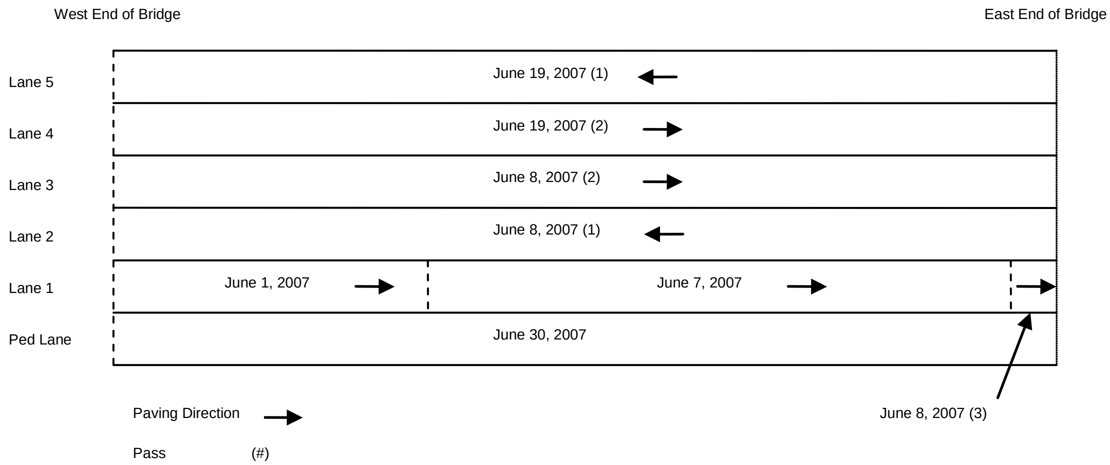

Figure 15. Base course paving sequence.

During placement of the job control strip for the base course the large rollers began picking up mix from the newly laid mat. Pickup occurred between 300 and 700 feet from the west end of the bridge. The mix had to be removed from the roller drums before the roller could continue, delaying the compaction operation. Voids left in the mat were repaired by carrying mix back to the damaged area in shovels and a wheelbarrow. Despite the repair efforts voids and open areas remained in the mat. The voids were more or less centered about 6.5 feet from the face of the barrier. The problem ceased when the large rollers were removed from the paving operation and replaced by smaller rollers.

Figure 16. Mix pickup area immediately after rolling.

Figure 17. Raking in mix to repair pickup area.

Figure 18. Area of mix pickup after completion of rolling.

Figure 19. Area of mix pickup prior to the next day's paving.

An additional short section of base course was paved on June 7th. This section was not formally designated a job control strip but it served to verify that a satisfactory level of quality could be achieved and base course placement went into full production on June 8th.

During placement of the base course the Contractor applied water using hand held sprayers ahead of truck tires and vehicle tracks in order to prevent damage to the polymer modified bituminous hot melt adhesive. One area of tack coat incurred damaged by the track on the Terex transfer vehicle. The area may have been more susceptible to damage since the adhesion layer had been repaired earlier in the day. Otherwise this method appeared successful in preventing damage.

Figure 20. Applying water to prevent damage to adhesion layer.

Figure 21. Adhesion layer damaged by track.

After completing paving the base course in Lane 2 at the west end of the bridge the Contractor turned the paving train around and began paving eastbound in Lane 3. A water truck sprayed water on Lane 2 to cool it enough to allow trucks which were delivering mix to pave Lane 3 to drive on Lane 2. At this time Lane 1 had only been paved to within about 250 feet of the east end of the bridge which left the bridge drains two inches higher than the current grade. The water flowed to the east end of the bridge and accumulated adjacent to the barrier in Lane 1 since it could not get into the bridge drains. After completion of paving Lane 3 the contractor proceeded to pave the remaining base course in the east end of Lane 1. The contractor attempted to remove the excess water that had accumulated in Lane 1 using leaf blowers prior to paving, but water was still present near the curb and over parts of the lane during paving.

Portions of the base course and tack layer were affected by the water and had to be removed around the bridge scuppers. The repair consisted of reapplying the adhesion layer and replacing the base course material at the end of the next day's paving.

Figure 22. Water being removed prior to paving.

Figure 23. Triangle of adhesion layer removed near scupper due to water damage.

Figure 24. Adhesion layer repaired.

Figure 25. Repaired adhesion layer.

After completion of placing base course in Lanes 1 through 5, the Contractor switched over to placement of the top course. Base and top course on the Pedestrian Lane were placed later after completion of all of the other paving on the bridge. A CSS-1 tack coat was placed between the base and top courses using a distributor truck. Figure 26 shows the sequence of paving the top course.

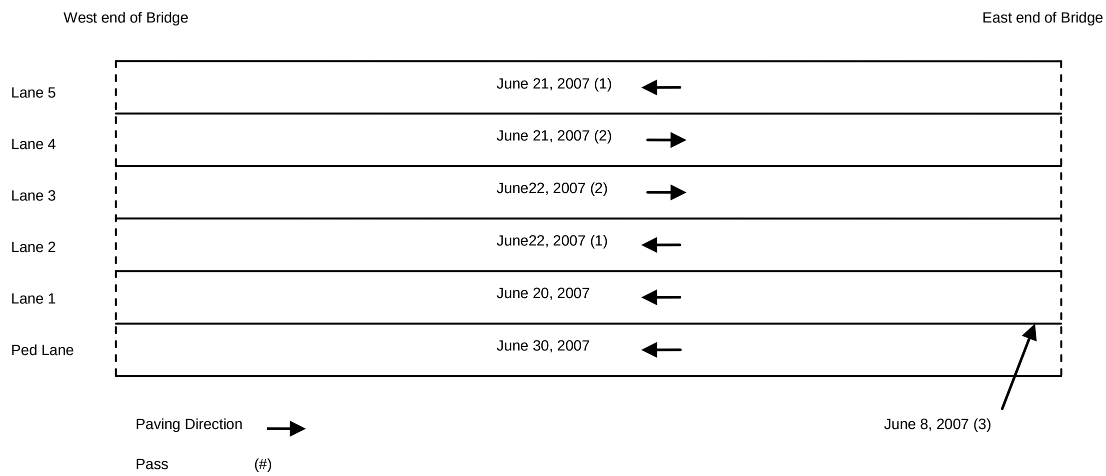

Figure 26. Top course paving sequence.

Temperature Differentials

Infrared imaging was used during HMA placement to check for differences in temperature that could lead to potential density differentials. The imaging revealed temperature differentials ranging from 10°F to more than 40°F. Temperature differentials can lead to areas of inadequate compaction in the mat. They are usually caused by mix inside the haul vehicle cooling on the outside of the load faster than in the center. If the mix is not remixed before it passes through the paver, areas of cooler mix will be present in the mat. Temperature differentials on the TNB appeared as longitudinal streaks as opposed to periodic cooler patches which are usually associated with end dumping directly into the paver.

Thermal imaging revealed three identifiable patterns of temperature differentials across the pavement surface during top course paving. The most common pattern is a warmer streak down the center of the mat flanked by cooler streaks on each side. Temperature differentials were greatest with this pattern, some being over 40°F. A second pattern with the cooler streak in the center is also apparent on some of the images. Most of the temperature differentials were within about 15°F in this pattern. Uniform temperature was seen across the mat on a few occasions.

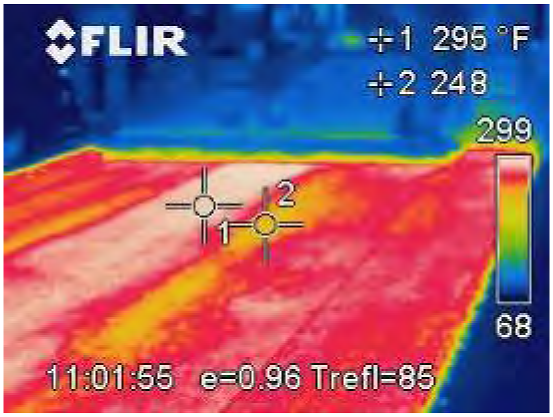

Figure 27. Temperature differential of 47°F in top course.

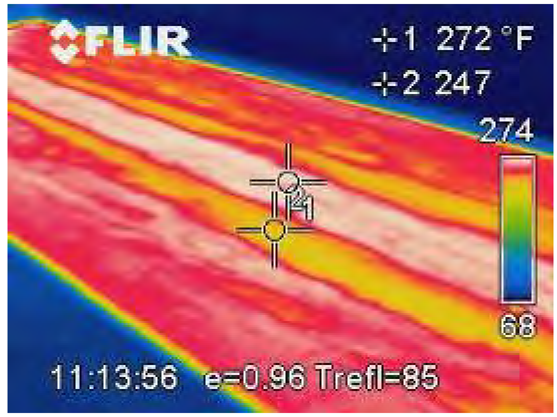

Figure 28. Warmer center in base course.

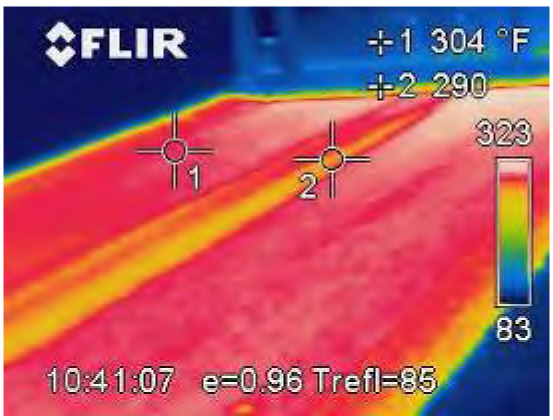

Figure 29. Mat with cooler center recorded on June 20.

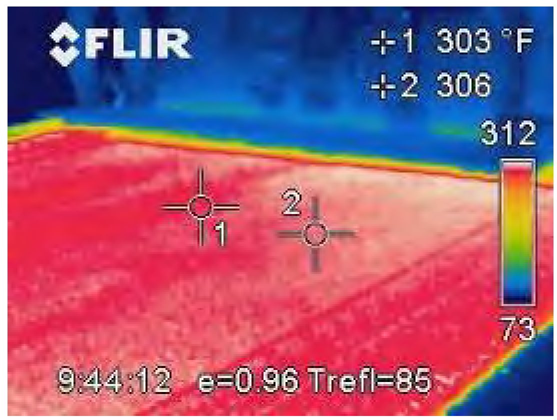

Figure 30. Mat with uniform temperature recorded on June 22.

Compaction

Compaction of the base course began on the control strip with a small roller as the breakdown roller followed by two large rollers as intermediate rollers and a second small roller as the finish roller. This pattern lasted for about the first 2.5 hours of base course placement at which time the larger rollers were removed due to the problems with them picking up parts of the mat. The remainder of the base course was compacted using only the small rollers.

The remainder of Lane 1 as well as the base course on Lanes 2, 3 and 5 were compacted using four of the small rollers. The arrangement generally consisted of a breakdown roller operating from within several feet of the screed to about 100 feet behind the paver. Two small rollers worked the intermediate position between about 100 and 300 feet behind the paver. The finish roller worked the section approximately 200 feet beyond the area covered by the intermediate rollers. The spacing of the rollers tended to increase as the speed of the paver increased.

A fifth small roller was used during paving of Lanes 2 and 3 to compact the joint between the lane being paved and the previously paved lane. The roller operated back and forth on the joint within about 40 feet of the paver. Most of the roller operated on the cold side of the joint with only about six inches of the drum on hot side.

Five small rollers were used to compact the base course in Lane 4. Two worked side by side as breakdown rollers within 100 feet of the paver and two others worked as intermediate rollers from 100 to 450 feet behind the paver. The fifth roller stayed from 450 to 650 feet behind the paver to finish the mat.

Roller speeds were measured several times during placement of the base course. Time and distance were measured from the time the roller stopped to change direction until it stopped again at the end of the pass. Table 6 shows the range of roller speeds recorded.

Table 6. Roller speeds on base course.

Roller	Speed (fps)
Breakdown	7-9
Intermediate (Closest to Paver)	4-5
Intermediate (Farthest from Paver)	8-11
Finish	7-10
Joint	4

Two roller arrangements were used to compact the top course. On Lanes 1 and 5 a large roller and small roller worked together near the paver as breakdown rollers. The small roller compacted the mix near the barrier with the large roller compacting the rest of the mat. Breakdown rolling occurred within about 200 feet of the paver with the rollers coming within several feet of the screed on their forward pass. Intermediate rolling was accomplished by a large roller which operated from about 200 to 400 feet behind the paver. Two small rollers finished the pavement with a 250 to 500 foot gap between them and the intermediate roller.

Breakdown rolling on Lanes 2, 3 and 4 consisted of a large roller operating from within several feet of the screed to about 200 feet back. A second large roller operated between 200 and 400 feet behind the paver. Three small rollers followed with the last being about 1000 feet behind the paver. A sixth small roller compacted the joint between the lane being paved and the previously paved lane (this was the joint between Lane 2 and 3 when Lane 3 was being paved). The roller worked with only about six inches of the drum on the new mat as before. Table 7 gives the range of roller speeds recorded during placement of the top course.

Table 7. Roller speed on top course.

Roller	Speed (fps)
Large Breakdown	4-8
Small Breakdown	4
Large Intermediate	4-7
Small Intermediate	8-11
Finish	5-11
Joint	6-8

Testing

Testing during HMA placement consisted of gradation, asphalt content and density. Construction Testing Laboratories, Inc. (CTL) provided the quality control testing and Professional Service Industries, Inc. (PSI) provided quality assurance testing. WSDOT's testing was informational.

Gradation and Asphalt Content

Tables 8 and 9 show the average gradation and asphalt content test results for the base course and top course, respectively. The results represent an average of a total of 32 tests by CTL, two tests by PSI and four tests by WSDOT on the base course and 24 tests by CTL, four tests by PSI and three tests by WSDOT on the top course.

Table 8. Base course gradation and asphalt content test results.					
Sieve	JMF	Tolerance Limits	CTL	PSI	WSDOT
3/8	100	100	100.0	100.0	100.0
#4	98	91-100	99.3	98.9	99.0
#8	77	73-81	79.8	76.5	78.5
#16	52	48-56	53.7	50.4	52.8
#30	35	31-39	37.2	34.6	36.5
#50	24	20-28	25.7	23.2	26.0
#100	17	15-19	18.6	16.1	18.5
#200	12.1	10.1-14.1	13.5	11.2	13.3
% Asphalt	10.8	10.5-11.1	10.8	11.2	11.0

Table 9. Top course gradation and asphalt content test results.					
Sieve	JMF	Tolerance Limits	CTL	PSI	WSDOT
3/4	100	100	100.0	100.0	100.0
1/2	95	88-100	94.3	94.9	96.0
3/8	87	80-94	86.2	87.7	86.2
#4	53	46-60	56.3	57.8	56.3
#8	43	39-47	44.6	45.7	44.6
#16	33	29-37	33.9	34.6	33.9
#30	26	22-30	26.7	26.9	26.7
#50	20	16-24	20.0	20.1	21.0
#100	14	12-16	14.6	14.3	15.0
#200	8.1	6.1-10.1	9.2	9.2	9.9
% Asphalt	5.6	5.3-5.9	5.5	5.7	6.0

All of the average gradation results were within the tolerance bands during construction. Based on CTL's test results, ten out of 32 base course gradation samples were outside the tolerance limits on at least one sieve. Only one sieve was outside of tolerance limits for twenty four top course gradation samples tested by CTL.

Base course asphalt content was out of specification on several tests. Most of the out of specification results were near the start of paving and were above the upper tolerance band. All of the top course asphalt content test results were within the tolerance limits. Individual asphalt content and gradation test results can be found in Appendix B.

Compaction Testing

Table 10 summarizes the average density results. The results for CTL, PSI and WSDOT did not meet the target value (97.0 percent) for the base course compaction. The average density value for WSDOT was the only one to meet the target minimum for top course compaction (94.0 percent). Individual compaction test results are included in Appendix C.

Table 10. Compaction test results.

Testing Entity	Base Course Compaction	Base Course Standard Deviation	Top Course Compaction	Top Course Standard Deviation
CTL	94.4	1.72	93.4	1.26
PSI	N/A	N/A	93.7	0.95
WSDOT	95.4	1.51	95.3	1.93
Target	97.0	N/A	94.0	N/A

Density test results from cores taken from the top course in Lane 1 are shown in Table 11. CTL's density results for Lane 1 averaged 92.3 percent which correlates well with the average core density of 92.9 percent.

Table 11. Core density results.

Core No.	Percent of Maximum Density
1	92.5
2	93.5
3	93.8
4	91.8
5	92.9
Average	92.9
Target	94.0

Appearance of Finished Mat

Streaks in Pavement

Both lifts in all lanes displayed some level of visible streaking after paving. Some of the streaks appear to be located at the screed extensions while others seem to be associated with cool streaks in the mat visible on the infrared photographs. It was not clear if the streaks are superficial or represent defects in the mat.

Figure 31. Steaks in base course on Lane 1.

Figure 32. Streaks in top course on Lane 1.

Figure 33. Streaks in top course on Lanes 4 and 5.

Figure 34. Streak left by screed extension.

Flushing

Slight flushing occurred after placement of the job control strip. Flushing appeared to be isolated to the section between 3.5 feet from the face of barrier to the edge of the paved lane at 10.5 feet. The flushing seems to correspond to high asphalt content test results on Lane 1.

Figure 35. Flushing in Lane 1 immediately after paving.

Figure 36. Flushing in Lane 1 several days after paving.

Indentations in Base Course

There were many areas where tires from equipment sitting on the bridge left indentations in the base course. These areas were repaired by heating with a weed burner and reshaping before placing the next lift.

Figure 37. Indentation in base course left by equipment tire.

Figure 38. Repairing indentation.

Indentations in Completed Overlay

An inspection on July 9th revealed that there were many indentations in the top course caused by construction activities. Some of the indentations were clearly due to equipment tires or the outrigger supports for a 300 ton Demag crane while the cause of others could not be identified. Cracks were associated with some of the indentations where the mix appeared to have been sheared by the object causing the indentation.


Figure 39. Tire indentation near west end.


Figure 40. Indentation left by Demag crane pad.


Figure 41. Indentation with crack near midspan.


Figure 42. Possible repaired area.

Spill at West Tower

Solvent spilled from the west tower onto the completed overlay in Lanes 2 and 3 which required the pavement to be removed and replaced. Repair included rotomilling the damaged pavement, repairing the three layer membrane system and repaving with conventional HMA.

Figure 43. Solvent spill.

Figure 44. Damaged pavement removed from Lane 2 and 3.

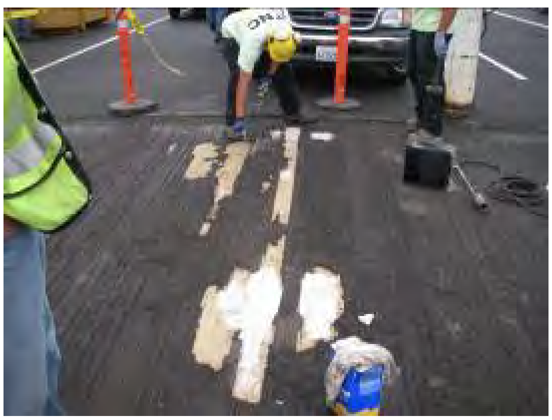

Figure 45. Repairing isolation layer.

Figure 46. Repairing base course with conventional HMA.

Figure 47. Replacing top course with conventional HMA.

Figure 48. Finished conventional HMA patch.

Construction Problems

The problems with paving, mix design, compaction, damage during construction, and the final appearance of the mat detailed in the post-construction report are repeated verbatim in the following sections. Figure 49 shows the final configuration of the lanes and pedestrian path on the deck.

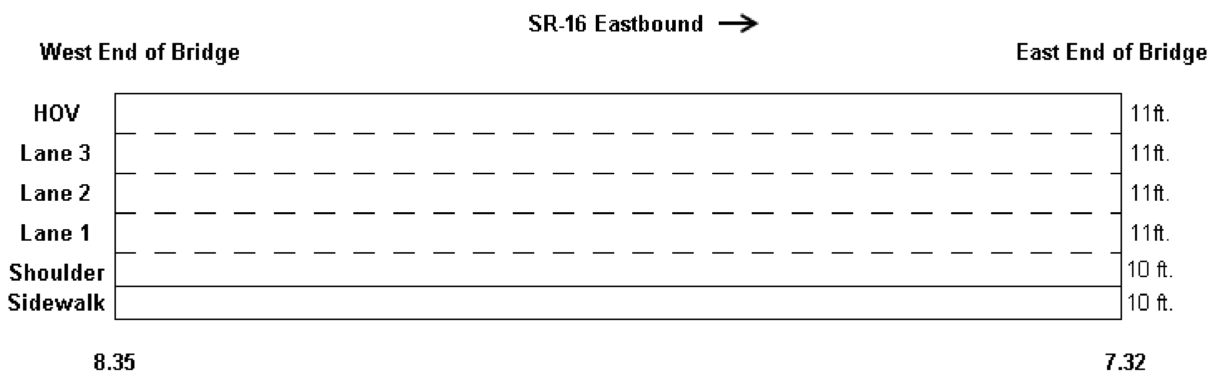

Figure 49. Plan map of finished bridge deck.

<u>Paving</u>

The TLA modified HMA overlay was placed using conventional equipment and construction methods. Once the contractor became familiar with working with the new material, placement proceeded as efficiently as conventional HMA. However, using these conventional methods it was impossible to achieve the target densities for both the base and top courses. The Special Provisions currently require pneumatic rollers for bridges that are 125 feet or longer (9).

Mix Design

The design documents specify restrictive values for air voids and tight control points for gradation. It is not clear how these values were determined but it can be assumed they were chosen in order to produce a mix that will achieve the desired longevity on an orthotropic deck. During the mix design process the specifications for both the air voids and gradation requirements were changed to allow use of the available materials. It is not clear how these changes may affect the performance of the pavement. Prior to constructing future overlays, more investigation into the reasoning behind the mix design specification should occur in order to determine the consequences of changing the requirements.

Compaction

The target of 97 percent of maximum theoretical density for the base course and 94 percent of theoretical maximum density for the top course were difficult to attain leading to many failing tests. Ninety six percent of base course density tests and 63.5 percent of top course density tests did not meet the target. Adequate density is important in making a pavement impermeable to moisture and higher densities also tend to make a pavement more fatigue resistant. As will be seen in the following sections, the low densities did affect the HMA performance on the TNB.

Future overlays of this kind should review the density requirements to determine if the high density targets are necessary and if they are attainable. The performance of the existing overlay may give evidence as to whether lower densities are adequate. If the overlay performs adequately the densities achieved on the existing overlay could be used as a basis for future overlays. Otherwise more mix testing would be necessary to determine if the density requirements may be lowered.

If it is determined that the higher densities are required, procedures need to be put into place to ensure the densities are met. Although calibration strips were constructed to show that the Contractor could achieve the desired results, the densities during actual paving on the bridge did not consistently achieve the targets. This is likely due to the differences in construction at the calibration strip site and at the bridge. The calibration strip is only 40 feet long which allowed better control of roller passes and less cooling time prior to completion of rolling than on the bridge. Paving speeds on the first two calibration strips which were used to verify that the equipment could achieve the desired results ranged between 7 and 13 feet per minute. Paving on the bridge was much faster ranging between 27 to 36 feet per minute which may have affected the Contractor's ability to obtain the target densities. If this type of overlay is used in the future, the compaction equipment lay down temperature, and paving speed necessary to meet density requirements established by the test sections should be closely followed during actual paving to ensure compaction is achieved.

Damage to the Mat

Numerous indentations, cracks and spills were present on the mat after it was complete. Most if not all of these can be attributed to the construction activities occurring on the bridge after placement of the overlay. Some of the damaged areas were superficial but many required repair by methods which may affect the performance of the pavement. In order to avoid this type of damage in the future, paving should not begin until work on or above the bridge deck is complete.

Appearance of the Mat

The finished top course looked more open than other 1/2 inch HMA pavements. It is not clear if this is just the perception of the viewer or an actual problem with the mat; however, there were several factors that could lead to an open mat. These include the picking up of individual rocks by the rollers, temperature differentials in the mat and the streaks in the mat that may be due to faulty paver operations including the damaged MMK retrofit. Another possibility is that the depth of the top course may have affected the mix quality. The minimum recommended thickness of an HMA layer is three times the NMAS with four times the NMAS being preferred. The minimum recommended thickness allows the aggregate to properly orient itself during compaction. The top course was placed at a depth of 1-1/4 inches; only 2.5 time the NMAS, which may have contributed to the open mat. It is recommended that either a smaller NMAS be used to achieve the minimum requirement of three times the thickness; or that the top course be increased to a minimum of 1 ½ inches, if this type of overlay is used in the future.

Summary

In summary, a review of all of the problems repeated verbatim from the post-construction report indicated that the long-term performance of the TLA modified pavement may be negatively affected by; (1) the failure to meet mix design targets, (2) the failure to achieve density targets for both base and top courses, (3) the difficulties during paving with rock pick-up, temperature differentials, and streaking, (4) damage to the finished overlay in the form of indentations, cracks, and spills, and (5) the open appearance of the finished mat.

Pavement Performance

The long-term performance of the TLA modified HMA pavement was determined by periodic measurements of rutting/wear, ride, friction resistance, and visual condition on all lanes following completion of construction. Testing done in November of 2012 was used to evaluate the 5-year performance of the overlay as required by the warranty included in the contract documents (see Appendix D and E). Finally, a visual survey of the deck was conducted on May 1, 2014 to document the condition of the overlay prior to the issuing of this final report. The evaluation performed for the warranty will be addressed first followed by the long-term performance.

Five-Year Warranty Evaluation

The contract for the project required a five year warranty on the performance of the TLA overlay (see Appendix D, Pavement Warranty). A report on the evaluation is included as Appendix E. The cracking distress, pavement friction, and rutting/wear did not exceed the warranty specifications except for one 1/10 mile section of the bridge deck overlay. Raveling caused rutting that exceeded the 1/4 inch warranty limit was found on approximately 500 lineal feet in Lane 2 on the bridge. The Contractor placed a patch on the rutting in Lane 2 to fulfill the warranty requirements.

The design of the warranty requirements focused on the possibility that the TLA modified HMA could have early distress due to the flexibility of the orthotropic deck. Therefore, cracking, delamination, and rutting and wear were the focus of the warranty requirements. What was not anticipated was the possibility that other distress categories such as the raveling and potholing noted during the five-year evaluation would be the deciding issues leading to the replacement of the overlay in less than ten years.

Rutting/Wear

Rutting/wear measurements for each year since construction are listed in Table 12 and shown in the Figure 50 bar graph. The maximum rutting wear for any lane was 5.1 mm (between 3/16 and 13/64 inches) with the average for all lanes at 4.0 mm (5/32 inch). The most heavily trafficked middle lanes (2 and 3) had the highest rutting/wear, as would be expected. The increase in rutting/wear over the 83 months of data collection was, on the average, 2.4 mm (4.0-1.6).

Table 12. Rutting/wear measurements.

Lane	August 2007 (mm)	March 2008 (mm)	April 2009 (mm)	April 2010 (mm)	October 2011 (mm)	November 2012 (mm)	May 2014 (mm)
1	-	-	2.4	2.3	2.5	2.7	3.3
2	-	1.6	2.8	2.7	3.5	3.7	5.1
3	1.7	2.2	3.2	3.3	3.8	4.0	4.8
HOV	1.4	1.6	2.1	2.1	2.5	2.6	2.8
Average	1.6	1.8	2.6	2.6	3.1	3.2	4.0
Age (months)	2	9	22	34	52	65	83

Note: Lanes are numbered from the outside toward the median.

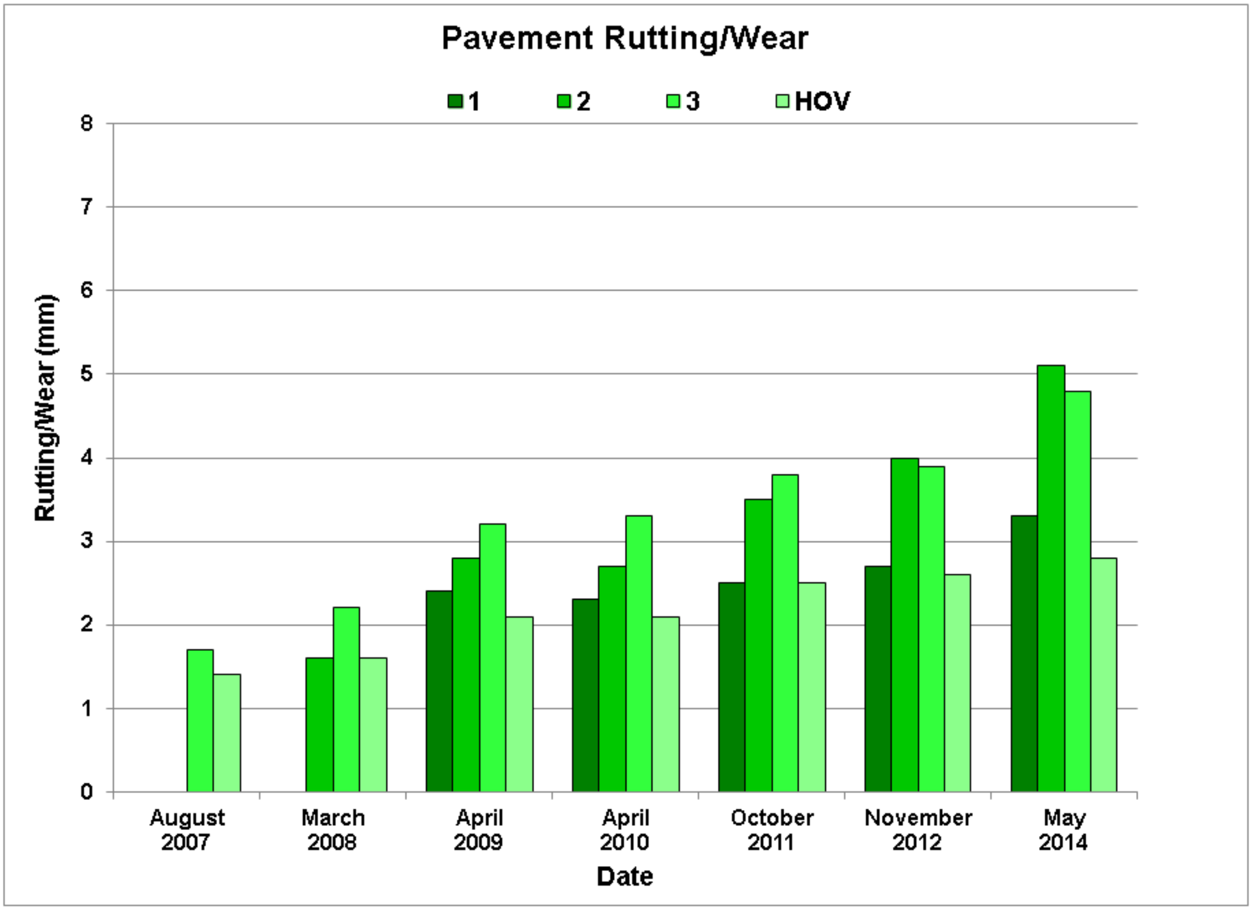

Figure 50. Rutting/wear measurements.

Ride

Ride measurements for each year since construction are listed in Table 13 and shown in the Figure 51 bar graph. The ride measurements were excellent for all lanes and did not vary to a great degree from lane to lane or from year to year which was not unexpected given the steel deck upon which the overlay rests. Individual values ranged from a high of 84 inches/mile to a low of 56 inches/mile. Averages for each measurement period ranged from 61 to 78 inches/mile. The increase in roughness for Lane 2 may be due to the large number of patches needed to repair deep raveling in the wheel paths.

Table 13. Ride measurements.							
Lane	August 2007 (IRI)	March 2008 (IRI)	April 2009 (IRI)	April 2010 (IRI)	October 2011 (IRI)	November 2012 (IRI)	April 2010 (IRI)
1	-	-	68	70	71	70	70
2	74	67	69	79	79	70	91
3	74	60	61	78	69	68	71
HOV	84	56	57	85	68	67	66
Average	77	61	64	78	72	71	75
Age (months)	2	9	22	34	52	65	83

Note: Lanes are numbered from the outside toward the median.

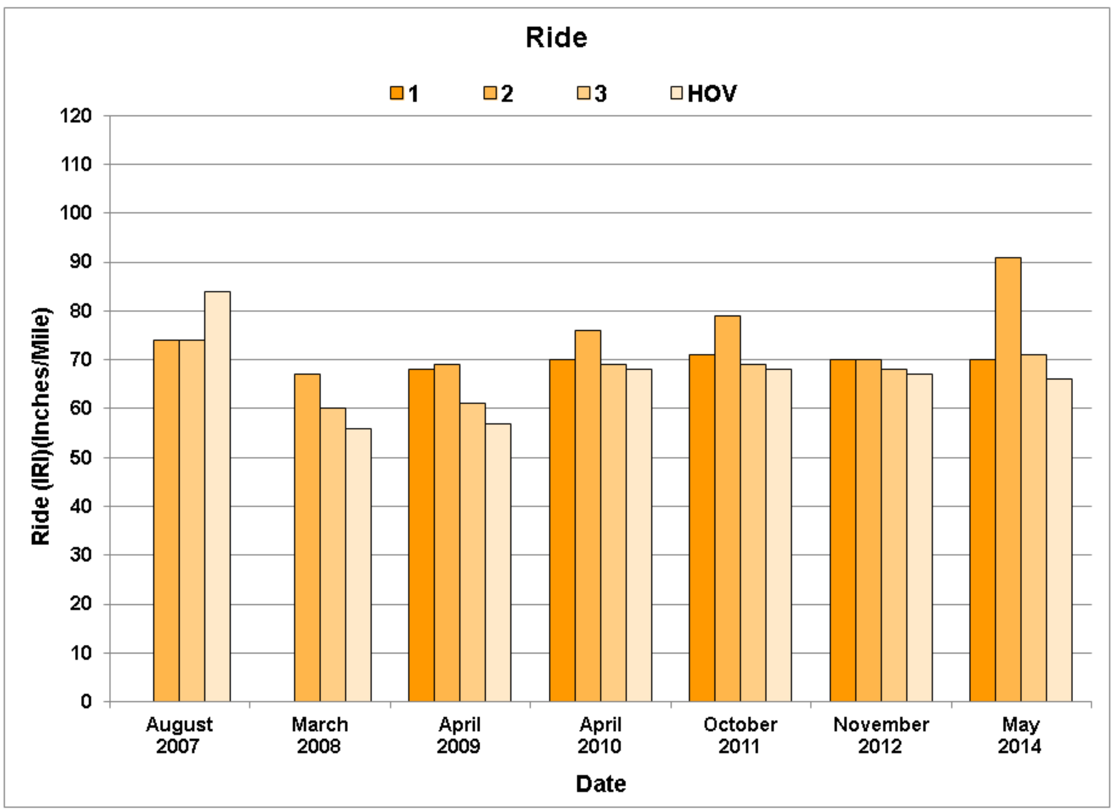

Figure 51. Ride measurements.

Friction Resistance

Friction measurements are listed in Table 14 and shown graphically in Figure 52. The friction measurements were excellent for all lanes with individual values ranging from a low of 45.6 to a high of 66.3 and yearly friction number averages ranging from 48.8 to 61.3.

Table 14. Friction measurements.

Lane	August 2007 (FN)	January 2008 (FN)	May 2009 (FN)	October 2010 (FN)	December 2011 (FN)	December 2012 (FN)
1	-	-	56.5	51.2	50.9	55.2
2	45.6	57.9	53.3	44.8	46.6	52.0
3	47.0	59.7	53.3	46.1	48.9	53.6
HOV	58.2	66.3	60.3	53.1	55.1	59.8
Average	50.3	61.3	55.9	48.8	50.4	55.2
Age (months)	2	7	23	40	54	66

Note: Lanes are numbered from the outside toward the median.

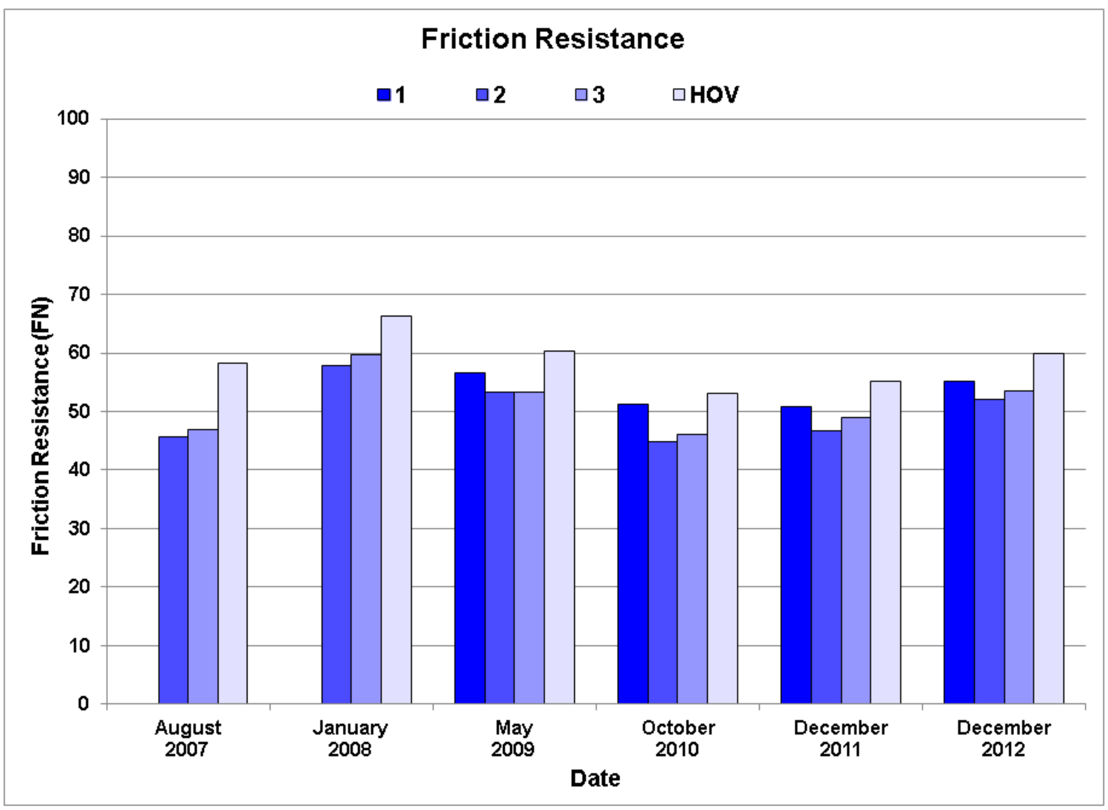

Figure 52. Friction measurements.

Visual Inspection

A visual survey was conducted on May 1, 2014 to document the condition of the wearing course at the end of 82 months (2 months shy of seven years). The most noticeable feature of the wearing course was raveling in the wheel paths on all of the lanes and patching in Lanes 2 and 3. The raveling was consistent in the wheel paths of all lanes with linear areas where the aggregates were missing giving the impression of it being a longitudinal crack. Lane 2 of the three general purpose lanes contained a multitude of patches in the wheel paths indicating that the raveling was greatest in this lane. Photos of the pavement on May 1, 2014 are included in Figures 53 to 62.

Figure 53. Severe raveling in the wheel paths of Lane1 and patch in Lane 2. Note accumulation of gravel on the shoulder. (Photo from 5/1/2014)

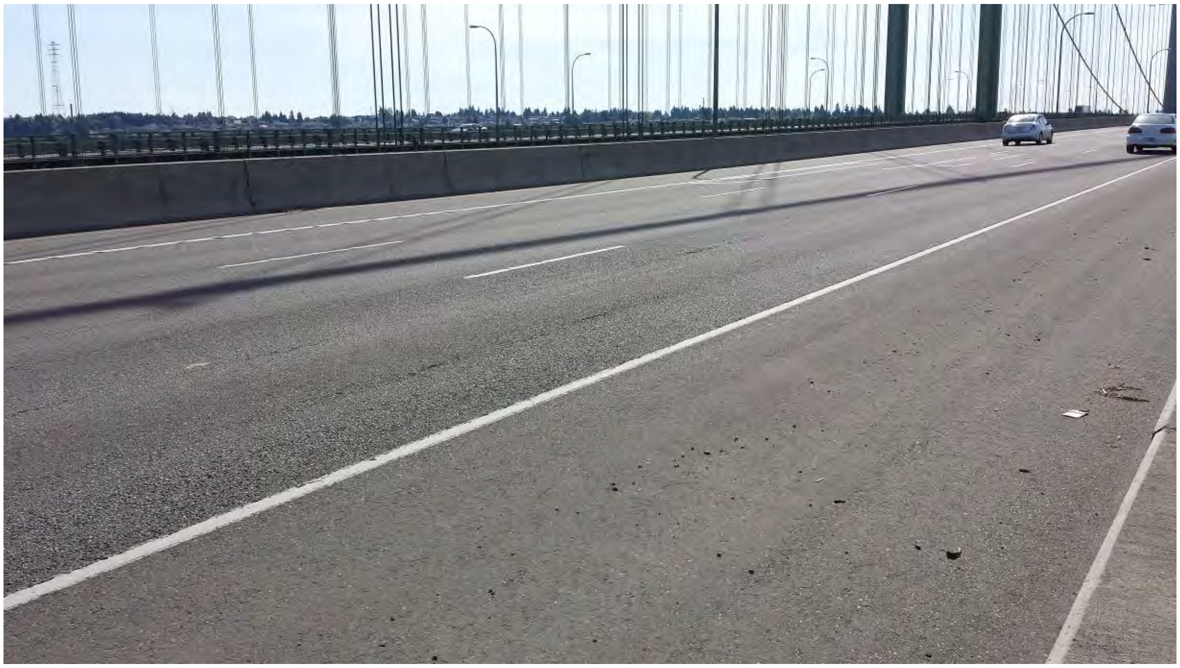

Figure 54. Severe raveling in the wheel paths of Lane 1. Note linear nature of the raveling in left wheel path of Lane 1. (Photo from 5/1/2014)

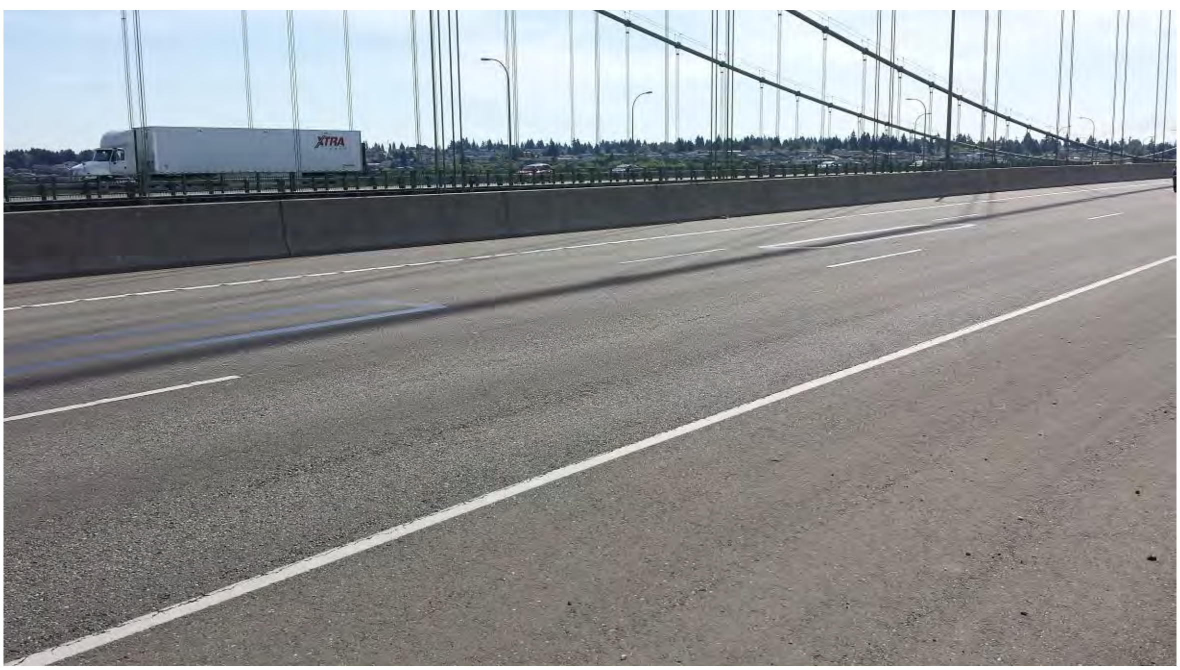

Figure 55. Severe raveling in Lane 1, patch in Lane 2. (Photo from 5/1/2014)

Figure 56. Moderate raveling in all lanes. (Photo from 5/1/2014)

Figure 57. Patches in Lane 3, pothole and raveling in HOV Lane. (Photos from 5/1/14)

Figure 58. Patches in Lanes 2 and 3. (Photos from 5/1/14)

Figure 59. Patch and severe raveling in Lane 2. (Photos from 5/1/14)

Figure 60. Severe raveling Lane 1, patches Lanes 2 and 3. (Photos from 5/1/14)

Figure 61. Patch Lane 1 and 2. Patch was done in July 2007 to repair problems caused by a solvent spill on the original overlay. Patch material was conventional HMA. (Photos from 5/1/14)

Figure 62. Close-up of HMA patch showing minimal raveling in the wheel paths. (Photos from 5/1/14)

The visual survey revealed the damage done to the wearing course over the seven years since it was constructed. All lanes are exhibiting areas of severe raveling which are characterized by linear areas of missing aggregate that resemble a longitudinal crack. Lanes two and three have experienced the greatest degree of damage as witnessed by the large number of patches in those lanes. Figures 61 and 62 showed above, a conventional HMA patch that was placed to repair an area of the original TLA mix that was damaged by a solvent spill. It appears that the raveling on the patched area is less than the surrounding TLA mix pavement.

The source of the damage is unknown, but studded tires have been linked to accelerated pavement wear on other roadways in the state. Studded tire damage caused the early failure due to raveling and rutting of open-graded friction course quieter pavements that were placed between 2006 and 2009 on I-405, I-5, and SR-520 (10). The problems with the mix design, meeting density requirements, temperature differentials, streaking, and open appearance of the finished mat resulted in a pavement with a higher void content which is more susceptible to studded tire damage. It would not be unreasonable to conclude that at least a portion of the raveling on the TLA overlay is due to studded tires.

Conclusions

The performance of the Trinidad Lake Asphalt mix appeared to be satisfactory with respect to the measurements of cracking, delamination, ride and friction. However, excessive rutting caused by raveling is a major problem that is shortening the life of the overlay. The disappointing performance of the overlay after only seven years of traffic indicate that the documented problems during construction resulted in a pavement that was susceptible to premature raveling possibly exacerbated by studded tires.

Future Action

Correspondence from the Olympic Region Materials Engineer, Bryan Dias, indicated that the Region is considering using a conventional HMA mix as a replacement for the TLA modified pavement (11). The good performance of the patch placed to repair an area of the original pavement would seem to indicate that a conventional HMA pavement could be a less expensive option then the TLA modified HMA. Further evidence for this choice is provided by rutting/wear measurements made on the conventional HMA placed adjacent to the bridge when the deck was overlaid. Tables 15 thru 17 show the rutting wear measurements on the each lane of the TNB deck and on the pavement east and west of the bridge.

Table 15. Rutting/wear on the TLA overlay on 5/12/2014.

Lane	LWP Rutting/Wear (mm)	RWP Rutting/Wear (mm)	Average Rutting/Wear (mm)
1	2.3	4.2	3.3
2	5.2	5.0	5.1
3	6.2	3.3	4.8
HOV	2.2	3.4	2.8
AVE	4.0	4.0	4.0

Table 16. Rutting/wear on the HMA east of the bridge on 5/12/2014.			
Lane	LWP Rutting/Wear (mm)	RWP Rutting/Wear (mm)	Average Rutting/Wear (mm)
1	1.5	3.1	2.3
2	5.2	4.8	5.0
3	5.5	3.2	4.4
HOV	3.6	2.9	3.3
AVE	4.0	3.5	3.7

Table 17. Rutting/wear on the HMA west of the bridge on 5/1//2014.			
Lane	LWP Rutting/Wear (mm)	RWP Rutting/Wear (mm)	Average Rutting/Wear (mm)
1	N/A	N/A	N/A
2	6.2	5.1	5.6
3	6.2	4.2	5.2
HOV	4.2	3.2	3.7
AVE	5.5	4.1	4.8

The rutting/wear measurements from the pavements east and west of the bridge bracket the measurements on the bridge with the east section showing slightly lower wear and the west section slightly higher than the pavement on the bridge. The use of a conventional HMA mix as a replacement on the bridge would not seem to be an issue with respect to rutting/wear. The only issue with using a conventional HMA mix would seem to be the original reason for specifying the TLA modified HMA – flexibility.

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Appendix A

Bridge Deck Plan Sheet

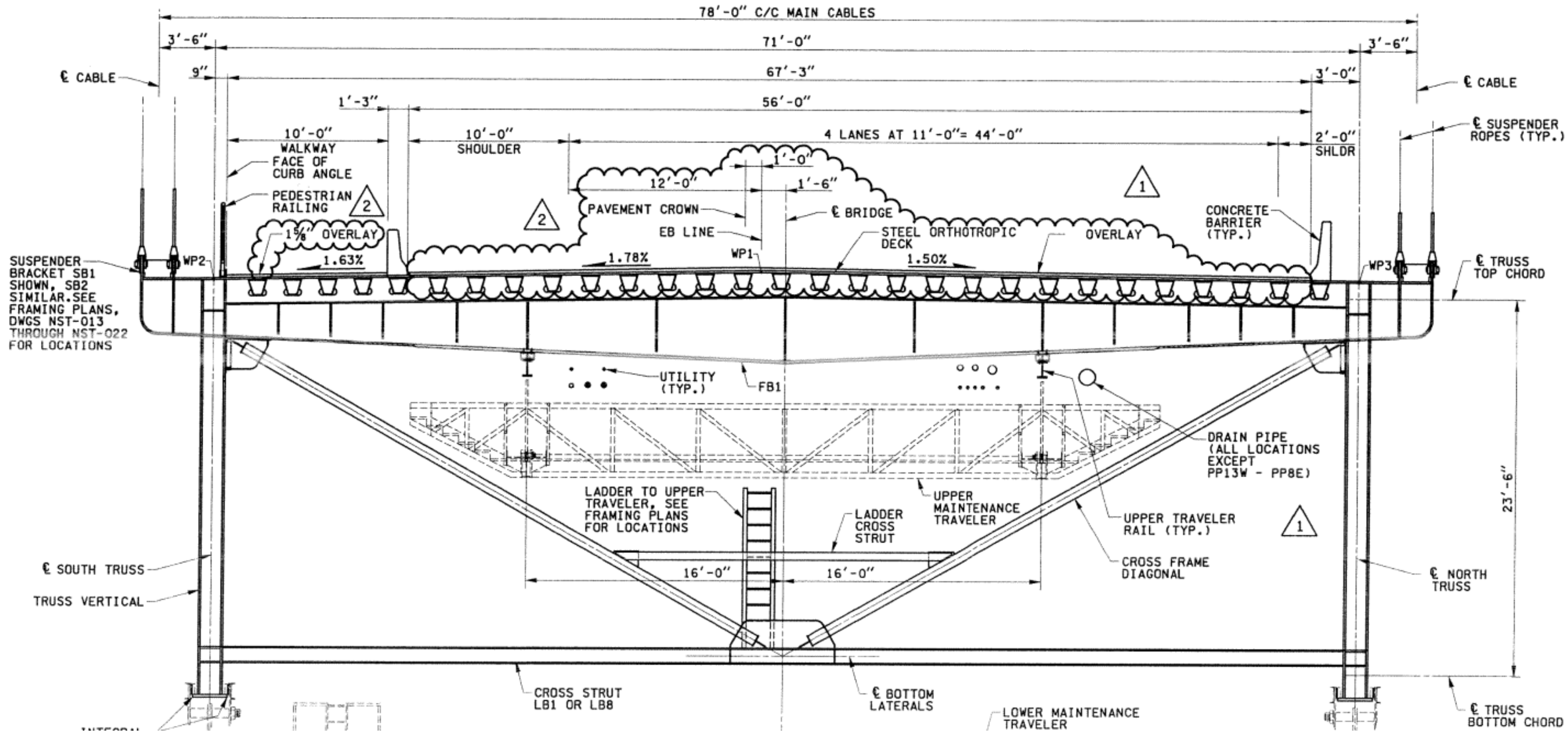

Figure 63. Plan sheet for Tacoma Narrows Bridge.

Appendix B

Gradation and Asphalt Content Test Results

Test Date	Sieve / Percent Passing								
	3/8"	#4	#8	#16	#30	#50	#100	#200	% Asphalt
JMF	100	98	77	52	35	24	17	12.1	10.8
Tolerance	100	91 - 100	73 - 81	48 - 56	31 - 39	20 - 28	15 - 19	10.1-14.1	10.5 - 11.1
6/1/2007	100	99	80	54	38	27	20	14.3	11.30
6/1/2007	100	99	81	55	38	26	19	13.5	11.35
6/1/2007	100	99	80	54	37	26	18	12.8	11.24
6/7/2007	100	100	81	54	37	25	18	13.3	10.75
6/7/2007	100	99	81	55	38	27	20	14.4	12.02
6/7/2007	100	99	78	52	36	25	18	13.4	10.94
6/7/2007	100	100	81	54	37	25	18	13.5	10.85
6/7/2007	100	100	82	55	38	26	19	13.6	11.05
6/7/2007	100	99	81	55	38	26	19	14.2	11.05
6/7/2007	100	100	81	55	38	26	19	13.8	10.82
6/7/2007	100	99	79	53	36	25	18	13.0	10.52
6/8/2007	100	98	74	50	35	25	18	13.1	10.51
6/8/2007	100	98	74	50	35	25	18	13.4	10.66
6/8/2007	100	98	75	51	36	25	19	13.5	10.64
6/8/2007	100	98	76	52	36	25	18	13.4	10.65
6/8/2007	100	98	76	51	36	25	18	13.0	10.67
6/8/2007	100	99	78	53	37	26	19	13.6	10.63
6/8/2007	100	99	80	54	38	27	20	14.7	10.57
6/8/2007	100	99	81	55	38	27	19	14.3	10.62
6/8/2007	100	100	83	56	39	27	20	14.6	10.87
6/19/2007	100	100	79	51	35	24	17	12.2	11.15
6/19/2007	100	100	80	53	36	25	18	13.2	11.04
6/19/2007	100	100	81	54	37	25	17	12.3	11.14
6/19/2007	100	100	80	53	36	24	17	12.1	10.89
6/19/2007	100	100	81	54	38	26	19	13.9	10.60
6/19/2007	100	100	82	55	38	26	19	13.6	10.76
6/19/2007	100	100	80	54	38	26	19	13.5	10.61
6/19/2007	100	100	82	54	38	26	19	13.2	10.00
6/19/2007	100	100	81	55	38	27	19	13.7	10.72
6/30/2007	100	100	84	57	39	26	19	13.5	10.87
6/30/2007	100	97	81	55	38	25	17	13.8	10.71
6/30/2007	100	100	81	54	39	27	19	13.9	10.55
Average	100.0	99.3	79.8	53.7	37.2	25.7	18.6	13.5	10.84
Std. Dev.	0.00	0.85	2.46	1.70	1.18	0.89	0.88	0.63	0.35

Table 18. CTL base course gradation and asphalt content test results.

CTL = Construction Technology Laboratories, Inc.
Out of tolerance values are shown in red.

Table 19. PSI base course gradation and asphalt content test results.									
Test Date	Sieve / Percent Passing								
	3/8"	#4	#8	#16	#30	#50	#100	#200	% Asphalt
JMF	100	98	77	52	35	24	17	12.1	10.8
Tolerance	100	91 - 100	73 - 81	48 - 56	31 - 39	20 – 28	15 - 19	10.1-14.1	10.5 - 11.1
6/8/2007	100.0	97.9	74.4	50.1	34.8	23.8	16.7	11.4	11.22
6/19/2007	100.0	99.8	78.6	50.7	34.3	22.5	15.5	11	11.23
Average	100.0	98.9	76.5	50.4	34.6	23.2	16.1	11.2	11.2
Std. Dev.	0.00	1.34	2.97	0.42	0.35	0.92	0.85	0.28	0.01

PSI = Professional Service Industries, Inc.
Out of tolerance values are shown in red.

Table 20. WSDOT base course gradation and asphalt content test results.									
Test Date	Sieve / Percent Passing								
	3/8"	#4	#8	#16	#30	#50	#100	#200	% Asphalt
JMF	100	98	77	52	35	24	17	12.1	10.8
Tolerance	100	91 - 100	73 - 81	48 - 56	31 - 39	20 - 28	15 - 19	10.1-14.1	10.5 - 11.1
6/1/2007	100	99	80	54	38	27	19	13.9	11.64
6/8/2007	100	98	74	50	35	25	18	12.8	10.63
6/8/2007	100	99	81	55	38	27	19	13.6	10.83
6/19/2007	100	100	79	52	35	25	18	12.9	11.09
Average	100.0	99.0	78.5	52.8	36.5	26.0	18.5	13.3	11.0
Std. Dev.	0.00	0.82	3.11	2.22	1.73	1.15	0.58	0.54	0.44

Out of tolerance values are shown in red.

Table 21. CTL top course gradation and asphalt content test reports.											
Test Date	Sieve / Percent Passing										
	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	% Asphalt
JMF	100	95	87	53	43	33	26	20	14	8.1	5.6
Tolerance	100	88 - 100	80 - 94	46 - 60	39 - 47	29 - 37	22 - 30	16 - 24	12 - 16	6.1 - 10.1	5.3 - 5.9
6/20/2007	100	92	84	49	41	32	25	19	14	9.2	5.35
6/20/2007	100	93	85	58	43	32	25	18	13	8.5	5.52
6/20/2007	100	97	89	59	44	33	26	19	14	9.3	5.71
6/20/2007	100	93	84	55	43	32	26	19	14	9.0	5.58
6/21/2007	100	92	86	57	44	33	26	19	14	9.2	5.54
6/21/2007	100	96	87	61	46	35	28	21	15	9.1	5.56
6/21/2007	100	95	88	58	46	35	28	21	16	9.8	5.45
6/21/2007	100	93	85	55	44	34	26	20	14	9.0	5.50
6/21/2007	100	93	85	55	45	35	28	21	16	9.9	5.44
6/21/2007	100	94	86	55	45	35	27	21	15	9.4	5.41
6/21/2007	100	95	87	58	46	35	27	20	15	9.3	5.57
6/21/2007	100	97	89	60	47	36	28	21	15	9.5	5.78
6/21/2007	100	96	86	56	45	35	27	21	15	9.5	5.54
6/22/2007	100	95	86	57	46	35	28	21	15	10.0	5.61

Table 22. CTL top course gradation and asphalt content test reports (continued).											
Test Date	Sieve / Percent Passing										
	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	% Asphalt
JMF	100	95	87	53	43	33	26	20	14	8.1	5.6
Tolerance	100	88 - 100	80 - 94	46 - 60	39 - 47	29 - 37	22 - 30	16 - 24	12 - 16	6.1 - 10.1	5.3 - 5.9
6/22/2007	100	95	88	57	45	34	27	21	15	9.7	5.62
6/22/2007	100	94	87	56	46	35	28	21	16	9.9	5.49
6/22/2007	100	95	87	56	45	35	27	21	15	8.1	5.57
6/22/2007	100	94	87	55	45	34	27	20	15	9.7	5.53
6/22/2007	100	92	83	55	44	33	26	20	14	9.0	5.47
6/22/2007	100	94	86	55	44	34	27	21	15	9.5	5.43
6/22/2007	100	96	86	55	43	32	25	19	14	9.2	5.44
6/30/2007	100	94	84	57	45	34	26	19	14	9.2	5.69
6/30/2007	100	95	86	58	45	34	26	19	14	8.7	5.72
6/30/2007	100	94	86	55	45	34	27	20	14	8.1	5.64
Average	100.0	94.3	86.2	56.3	44.6	33.9	26.7	20.0	14.6	9.2	5.5
Std. Dev.	0.00	1.48	1.52	2.31	1.28	1.13	0.98	0.96	0.69	0.51	0.11

Table 23. PSI top course gradation and asphalt content test results.											
Test Date	Sieve / Percent Passing										
	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	% Asph.
JMF	100	95	87	53	43	33	26	20	14	8.1	5.6
Tolerance	100	88 - 100	80 - 94	46 - 60	39 - 47	29 - 37	22 - 30	16 - 24	12 - 16	6.1 - 10.1	5.3 - 5.9
6/20/2007	100.0	93.3	84.9	52.1	43.3	32.7	25.8	19.4	14.1	9.3	5.69
6/20/2007	100.0	95.2	87.7	59.3	45.9	34.4	26.8	20.0	14.1	8.9	5.74
6/21/2007	100.0	95.6	89.7	59.6	46.9	34.7	26.6	19.5	13.8	9.0	5.89
6/22/2007	100.0	95.4	88.6	60.0	46.6	36.6	28.5	21.3	15.1	9.6	5.60
Average	100.0	94.9	87.7	57.8	45.7	34.6	26.9	20.1	14.3	9.2	5.7
Std. Dev.	0.00	1.06	2.05	3.78	1.64	1.60	1.14	0.87	0.57	0.32	0.12

Table 24. WSDOT top course gradation and asphalt content test results.											
Test Date	Sieve / Percent Passing										
	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	% Asphalt.
JMF	100	95	87	53	43	33	26	20	14	8.1	5.6
Tolerance	100	88 - 100	80 - 94	46 - 60	39 - 47	29 - 37	22 - 30	16 - 24	12 - 16	6.1 - 10.1	5.3 - 5.9
6/20/2007	100	95	87	54	45	34	27	21	15	10.0	5.84
6/21/2007	100	96	88	61	46	30	26	20	14	9.4	5.99
6/22/2007	100	97	89	59	47	36	28	22	16	10.2	6.18
Average	100.0	96.0	88.0	58.0	46.0	33.3	27.0	21.0	15.0	9.9	6.0
Std. Dev.	0.00	1.00	1.00	3.61	1.00	3.06	1.00	1.00	1.00	0.42	0.17

Out of tolerance values are shown in red.

Appendix C

Compaction Test Results

Table 25. CTL base compaction test results.

Date	Lane	Percent of Maximum Density				
		Test 1	Test 2	Test 3	Test 4	Test 5
6/30/2007	Ped.	92.0	94.6	96.5	94.3	94.7
6/30/2007	Ped.	94.4	92.2	95.6	93.8	92.5
6/30/2007	Ped.	95.0	93.8	94.4	96.3	93.4
6/30/2007	Ped.	95.9	98.7	93.1	94.9	93.8
6/30/2007	Ped.	91.3	94.6	96.8	92.2	95.7
Average	94.4	Std. Dev.	1.72	Target	97.0	

CTL = Construction Technology Laboratories, Inc.
Ped. = Pedestrian Lane
Test results that did not meet minimum target density are displayed in red.

Table 26. WSDOT base course compaction test results.

Date	Lane	Percent of Maximum Density				
		Test 1	Test 2	Test 3	Test 4	Test 5
6/8/2007	2	93.7	94.6	95.5	96.3	94.4
6/8/2007	3	96.1	96.6	96.4	96.1	95.7
6/19/2007	5	94.1	95.9	95.8	96.1	99.2
6/19/2007	4	95.7	95.4	96.4	96.9	96.2
6/30/2007	Ped.	94.7	95.0	90.8	94.0	94.4
Average	95.4	Std. Dev.	1.51	Target	97.0	

Test results that did not meet minimum target density are displayed in red.
Ped. = Pedestrian Lane

Table 27. CTL top course compaction test results.

Date	Lane	Percent of Maximum Density				
		Test 1	Test 2	Test 3	Test 4	Test 5
6/20/2007	1	92.3	91.8	91.0	91.5	93.2
6/20/2007	1	93.3	93.4	93.5	92.5	92.0
6/20/2007	1	93.7	92.7	92.7	92.5	90.3
6/20/2007	1	92.5	94.4	92.9	92.1	92.6
6/20/2007	1	91.4	90.9	90.9	91.2	92.5
6/21/2007	5	93.6	93.2	94.4	94.2	94.6
6/21/2007	5	94.1	94.1	93.0	93.3	93.9
6/21/2007	5	93.7	93.7	93.7	94.7	94.8
6/21/2007	5	94.0	94.0	94.1	93.5	95.9
6/21/2007	5	93.9	94.2	94.3	94.1	94.9
6/21/2007	4	93.9	94.3	94.0	95.6	94.8
6/21/2007	4	94.1	91.8	92.3	91.1	92.1
6/21/2007	4	92.9	92.3	94.9	94.1	94.7
6/21/2007	4	92.5	93.3	92.0	93.7	94.2
6/21/2007	4	92.5	93.5	92.3	93.8	94.5
6/22/2007	2	92.1	93.1	92.5	91.8	94.6

Table 27. (Continued)						
Date	Lane	Percent of Maximum Density				
		Test 1	Test 2	Test 3	Test 4	Test 5
6/22/2007	2	93.8	92.4	93.3	93.5	93.8
6/22/2007	2	91.9	91.6	93.7	93.2	93.3
6/22/2007	2	94.6	93.0	93.3	94.6	93.6
6/22/2007	2	93.1	92.2	94.5	93.2	92.6
6/22/2007	3	92.6	93.2	93.4	93.6	93.5
6/22/2007	3	93.3	93.9	93.3	93.1	94.0
6/22/2007	3	94.1	93.1	93.7	92.9	92.9
6/22/2007	3	93.9	93.8	93.0	93.6	93.1
6/22/2007	3	92.9	92.2	93.0	94.6	90.2
6/30/2007	Ped.	92.0	96.0	92.7	93.9	97.3
6/30/2007	Ped.	92.8	93.6	95.6	96.6	94.4
6/30/2007	Ped.	95.6	95.7	92.9	92.6	95.9
6/30/2007	Ped.	93.0	92.7	93.9	93.2	91.8
6/30/2007	Ped.	93.4	96.5	97.0	95.2	95.7
Average	93.4	Std. Dev.	1.26	Target	94.0	

Test results that did not meet minimum target density are displayed in red.
Ped. = Pedestrian Lane

Table 28. PSI top course compaction test results.						
Date		Percent of Maximum Density				
	Lane	Test 1	Test 2	Test 3	Test 4	Test 5
6/20/2007	1	93.6	94.2	94.5	93.5	93.6
6/20/2007	1	95.4	94.5	94.1	94.8	92.6
6/20/2007	1	95.9	94.1	92.3	93.7	92.7
6/20/2007	1	92.9	94.5	93.4	93.1	93.5
6/20/2007	1	93.0	93.7	92.1	92.4	93.9
Average	93.7	Std. Dev.	0.95	Target	94.0	

PSI = Professional Service Industries, Inc.
Test results that did not meet minimum target density are displayed in red.

Table 29. WSDOT top course compaction test results.						
Date	Lane	Percent of Maximum Density				
		Test 1	Test 2	Test 3	Test 4	Test 5
6/21/2007	5	96.3	96.8	96.6	96.2	96.2
6/21/2007	4	97.7	97.7	95.9	95.4	92.5
6/22/2007	2	92.2	93.0	93.5	95.3	91.7
6/22/2007	3	96.1	96.0	94.6	94.4	95.1
6/30/2007	Ped.	96.3	91.4	97.4	98.2	96.0
Average	95.3	Std. Dev.	1.93	Target	94.0	

Test results that did not meet minimum target density are displayed in red.
Ped. = Pedestrian Lane

Appendix D

Pavement Warranty

PAVEMENTS

Description of Extended Warranty Components

All Construction Pavements

Performance Parameters

The parameters that will be used to evaluate performance of all constructed pavements for this project are ride quality, pavement friction, and pavement surface condition.

Asphalt Concrete Pavement

Ride Quality

Ride quality will be evaluated according to the International Roughness Index (IRI). A baseline measure of the IRI will be conducted using WSDOT's South Dakota Type Profiler. Acceptance will be based on the following criteria:

A. Ride quality value of less than 95 inches per mile. If the limit is exceeded the defective pavement shall be replaced (minimum depth of .15 feet) for the full lane width over the section.

B. The ride quality value at the end of the warranty period shall not exceed 95 inches per mile. If this criteria is not met, corrective action shall be taken as indicated above to bring this parameter within the limits.

Pavement Friction

Pavement friction shall meet the following performance criteria:

A. The requirement for acceptance is a friction number greater than 50. Pavement exhibiting values less than 50 will require corrective action to provide values that exceed 50.

B. The friction value at the end of the warranty period shall be no less than 40. Pavements with a friction number less than 40 will require corrective action. If at any time during the warranty period it is determined, in accordance with ASTM E2743-90, that this criterion is not met, upon receipt of notice to such effect corrective action shall be taken to provide values that meet or exceed 40.

Pavement Surface Condition

Pavement surface condition shall meet the following performance criteria:

A. Acceptance will permit no identifiable distress as defined by the WSDOT Pavement Surface Condition Rating Manual. If these criteria are not met, corrective action shall be taken as outlined in Table 1.

B. Distress types exceeding the allowable level of severity at the end of the warranty period shall require corrective action as outlined in Table 1.

<u>Portland Cement Concrete</u>

Ride Quality

A baseline measure of the International Roughness Index (IRI) will be conducted using WSDOT's South Dakota Type Profiler. Ride quality will be evaluated using a profilograph as indicated in the 1998 Standard Specifications for Road, Bridge and Municipal Construction (Attachment No. 4, Exhibit D, Standards and Criteria) and as follows:

A. The requirement for acceptance on newly constructed concrete pavement will be satisfaction of Standard Specification Section 5-05.3(12). If said criteria are not met, the profile shall be diamond ground back to acceptable limits provided the area requiring grinding does not exceed five percent of the surface area of a day's production and does not compromise the structural capacity of the section. If this limit is exceeded or the section thickness is reduced by more than five percent, the defective pavement shall be replaced for the full lane width over the section.

B. The IRI value at the end of the warranty period shall not increase by more than 25 percent from the IRI value determined at Project Substantial Completion. If at any time during said period this criterion is not met, upon receipt of notice to such effect, corrective action shall be taken as indicated above to bring this parameter within the limits.

Pavement Friction

Pavement friction shall meet the following performance criteria:

A. The requirement for acceptance is a friction value greater than 50. Pavement exhibiting values less than 50 will require corrective action to profile values that exceed 50.

B. The friction value at the end of warranty period shall be no less than 40. If at any time during said five-year period it is determined, in accordance with ASTM D274-90, that this criterion is not met, upon receipt of notice to such effect, corrective action shall be taken to provide that meet or exceed 40.

Pavement Surface Condition

Pavement shall meet the following performance criteria:

A. Acceptance shall not identifiable distress as defined by the WSDOT Pavement Surface Condition Rating Manual. If this criterion is not met, Design-Builder shall take corrective action as outlined in Table 2.

B. Distress types exceeding the allowable level of severity at the end of the warranty period shall require corrective action as outlined in Table 2.

Required Corrective Action

TABLE 1. Required Corrective Action for Pavement Distress Levels – Asphalt Concrete Pavements

Distress Type	Allowable Level of Severity	Allowable Extent of Severity	Corrective Action
Rutting and Wear	Less than ¼ inch	Project Length	Mill and fill with 2 inches of asphalt concrete pavement required
Alligator Cracking	Less than ¼ inch	Less than ten percent of project length of both wheel paths	Pavement repair required
Longitudinal Cracking	Less than ¼ inch	Less than 100 percent of project length (single crack)	Crack seal required
Transverse Cracking	Less than ¼ inch	Less than 4 cracks per 100 feet	Crack seal required

TABLE 2. Required Corrective Action for Pavement Distress Levels – New Concrete Pavements

Distress Type	Allowable Level of Severity	Allowable Extent of Severity	Corrective Action
Cracking	One crack per panel	Less than ten percent of project length	Full depth repair required
Joint and Crack Spalling	Spall less than ¼ inch wide	Less than ten percent of joints and cracks	Partial depth repair required
Pumping and Blowing	Slight shoulder depression, no staining	Less than ten percent of joints and cracks	Full depth panel replacement and repair of underlying base material required
Faulting	Less than 1/8 inch	Less than ten percent of joints	Diamond grinding back to zero tolerance without compromising pavement section. If the structural integrity of the pavement section is compromised then full depth slab replacement is required.

Patching	Less than ten percent of panel area is patched	Less than ten percent of all panels in travel lane are patched	Full depth panel repair required
Scaling	Pavement appears slightly rough	Less than ten percent of pavement surface	Diamond grinding back to zero tolerance without compromising pavement section. If the structural integrity of the pavement section is compromised then full depth slab replacement is required.
Wear	Less than ¼ inch	Less than ten percent of one lane mile	Diamond grinding back to zero tolerance without compromising pavement section. If the structural integrity of the pavement section is compromised then full depth slab replacement is required.
Joint Seal Damage	Hardening adhesive failure, cohesive failure, complete loss of sealant	Less than ten percent of joint length per one lane mile	Joint resealing required

ORTHOTROPIC DECK PLAT OVERLAY

Description of Extended Warranty Components

Orthotropic Deck Plate Overlay

Performance Parameters

The parameters that will be used to evaluate performance of the orthotropic deck plate overlay for this Project are rutting, shoving, tearing, cracking, bleeding of binder with a rut depth of ¼ inch maximum and delamination checked by chain drag techniques with no cases of delaminations identified as set forth in Table 1.

Required Corrective Action

TABLE 1. Required Corrective Action for Pavement Distress Levels – Orthotropic Deck Plate Overlay

Distress Type	Allowable Level of Severity	Allowable Extent of Severity	Corrective Action
Rutting and Wear	Less than ¼ inch	Project length	Full depth asphalt concrete pavement repair required.
Alligator Cracking	Less than ¼ inch	Less than ten percent of project length of both wheel paths.	Pavement repair required.
Delamination	None identified using chain drag technique	None identified using chain drag technique.	Removal and replacement required.

Appendix E

Pavement Warranty Evaluation

Tacoma Narrows SR 16 Warranty Evaluation

Background

The physical completion date for the Tacoma Narrows warranty project was May 16, 2008. In anticipation of the end of warranty period in May 2013, a condition evaluation was performed of the asphalt pavement on November 11, 2012. The results of this condition evaluation are summarized in this document, and compared with the warranty requirements. The roughness (IRI) specification was removed from the warranty by change order.

All Asphalt Pavement

The warranty specifications for asphalt pavement are shown in the table below. These specifications were used to evaluate the data collected by WSDOT's pavement condition survey Pathway van on 11/8/12.

Table 30. Required corrective actions for all asphalt pavement distress levels.

Distress Type	Allowable Level of Severity	Allowable Extent of Severity	Corrective Action
Rutting and Wear	Less than ¼ inch.	Project length.	Mill and fill with 2 inches of asphalt concrete pavement required.
Alligator Cracking	Less than ¼ inch.	Less than ten percent of project length of both wheel paths.	Pavement repair required.
Longitudinal Cracking	Less than ¼ inch.	Less than 100 percent of project length (single crack).	Crack seal required.
Transverse Cracking	Less than ¼ inch.	Less than 4 cracks per 100 feet.	Crack seal required.

Cracking Distress

The summary of the asphalt pavement cracking results are shown in the following table. On the right side of the table the measured cracking distresses are compared with the allowable distress, as indicated in the specification. There were no instances where the measured distress exceeded the allowable for alligator cracking, longitudinal cracking or transverse cracking.

Table 31. Sample of crack measurement for 5-year warranty evaluation.													
Road	Lane	Direction	Begin SRMP	End SRMP	Dir.	Patching (ft.)	Total Length (ft.)	Total Alligator Cracking (ft.)	Allowable Alligator Cracking (ft.)	Total Long. Cracking (ft.)	Allowable Long. Cracking (ft.)	Total Transverse Cracks (ft.)	Allowable Transverse Cracks (ft.)
16	1	WB	4.41	8.58	I	60	10,394	6	2,079	1,578	4,188	12	168
16	2	WB	4.41	10.14	I	406	18,525	11	3,705	345	12,359	16	494
16	3	WB	4.41	10.14	I	9	18,555	0	3.711	250	12,397	11	496
16	HOV	WB	4.41	10.14	I	13	18.570	51	3.714	417	12,609	19	504
16	1	EB	8.48	7.28	D	0	6,446	0	1,289	11	818	2	33
16	2	EB	10.74	4.41	D	95	21,658	0	4.332	662	16,039	12	642
16	3	EB	10.74	4.41	D	0	21,775	0	4,385	741	16,149	20	646
16	HOV	EB	10.74	4.41	D	4	21,695	20	4,339	792	16,065	29	643

Rutting Distress

As indicated in the specification, rutting is limited to ¼ inch over the entire project length. However, there are a couple of areas where the specification is open to interpretation.

a) The specification does not indicate over what distance the rut measurement is to be evaluated. Individual measurements are taken every few feet, but normally ruts are averaged over a given distance. For this warranty evaluation a distance of 0.1 mile was used to evaluate the rut measurements.

b) The specification does not indicate if the rut depth is evaluated in each wheel path separately, or if the average of the two wheel path ruts is considered. WSDOT typically uses the average of wheel path ruts in annual pavement condition evaluations. For this warranty evaluation the average rut depth of both wheel paths was used.

The rut depths were evaluated for all lanes using the above criteria, and in all cases the rut depth limits were not exceeded, except for one 1/10 mile segment in Lane 2 (decreasing direction). The data for this segment is shown in the graph below. As the data shows, several of the 0.1 mile rut measurements for the left wheel path slightly exceeded the 0.25 in. limit. However, the average rut depth is exceeded only for one 0.1 mile segment. WSDOT did not require the Contractor to repair this section.

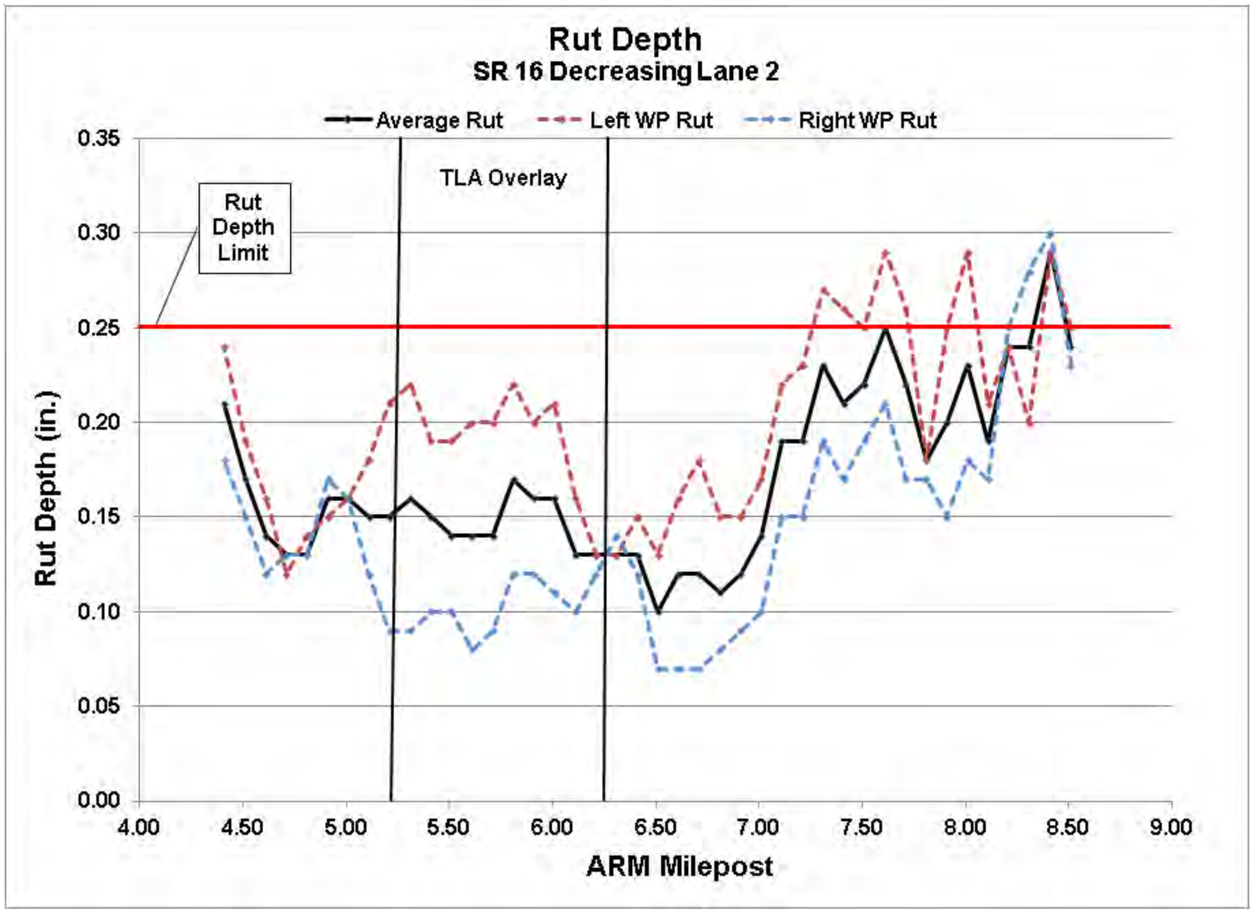

Figure 64. Rut depth for Lane 2.

<u>Pavement Friction</u>

The warranty specification states that “The friction value at the end of the warranty period shall be no less than 40”. Friction testing was done on 12/3/12 every 0.1 mile on all lanes, with average friction numbers ranging from 52.0 to 59.8 and no individual number less than 45.

Table 32. Friction number results from 12/3/2012.				
Lane	Direction	Minimum (FN)	Maximum (FN)	Average (FN)
1	EB	53.7	57.0	55.2
2	EB	45.6	54.1	52.0
3	EB	48.1	55.1	53.6
HOV	EB	56.8	62.0	59.8
1	WB	54.9	65.7	57.4
2	WB	51.4	63.4	54.6
3	WB	52.9	67.0	56.3
HOV	WB	52.2	63.9	58.1

Orthotropic Bridge Deck Overlay

The warranty specification for the asphalt overlay on the bridge deck is shown in Table 17. The alligator cracking was tabulated with the other portion of the project in the previous table, and was not in excess of the allowable.

There was no evidence of delamination on the new bridge span. However, there was no evaluation by chain dragging as listed in the specification, since chain dragging is appropriate only for concrete bridge decks.

There was visual evidence of wear (raveling) in excess of the allowable wear of 1/4 inch stated in the table. Detailed examination of the data revealed intermittent rutting that exceeding 1/4 inch occurred over approximately 500 lineal feet, primarily in Lane 2 on the bridge. Images of the raveling/wear and the rut depth measurements from the Pathway Van are shown in Figures 65 to 67. The Contractor was required to patch this section on the deck.

Table 33. Required corrective actions for orthotropic deck pavement distress levels.			
Distress Type	Allowable Level of Severity	Allowable Extent of Severity	Corrective Action
Rutting and Wear	Less than 1/4 inch.	Project length.	Full depth asphalt concrete pavement repair required.
Alligator Cracking	Less than 1/4 inch.	Less than ten percent of project length of both wheel paths.	Pavement repair required.
Delamination	None identified using chain drag technique.	None identified using chain drag technique.	Removal and replacement required.

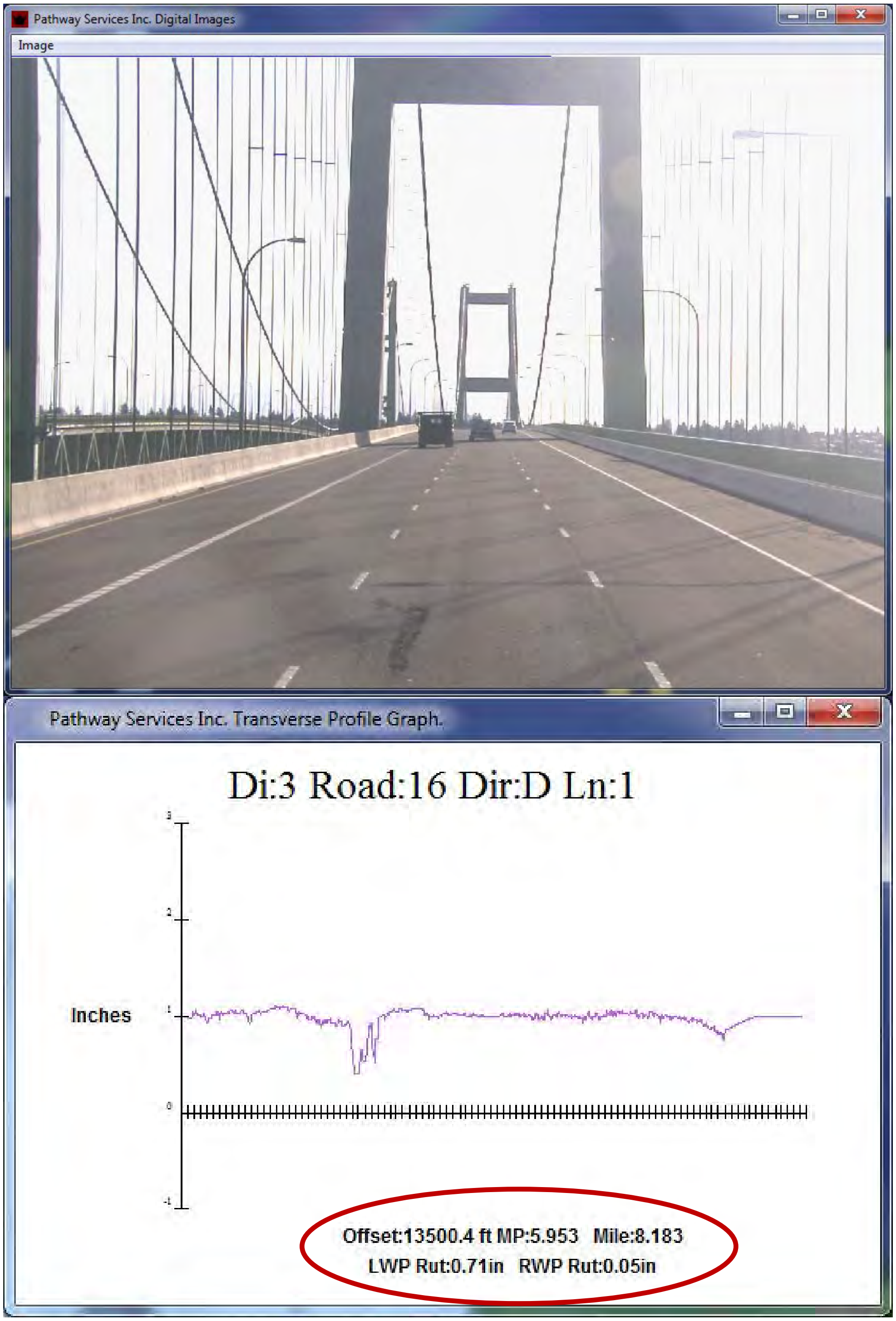

Figure 65. Left wheel path wear depth is 0.71 in. Note that the graph is incorrectly labeled as Lane 1.

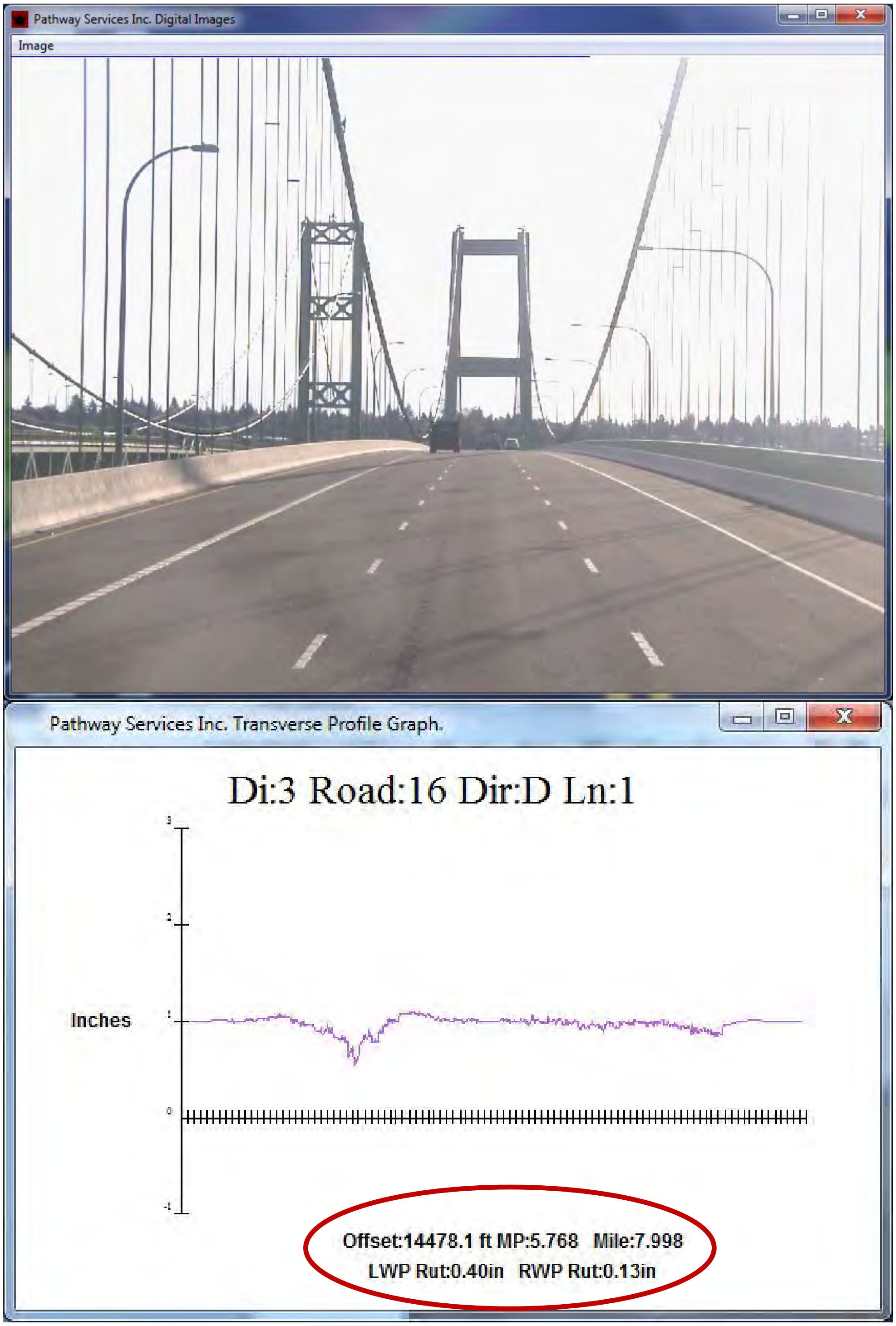

Figure 66. Left wheel path wear depth is 0.40 in. Note that the graph is incorrectly labeled as Lane 1.

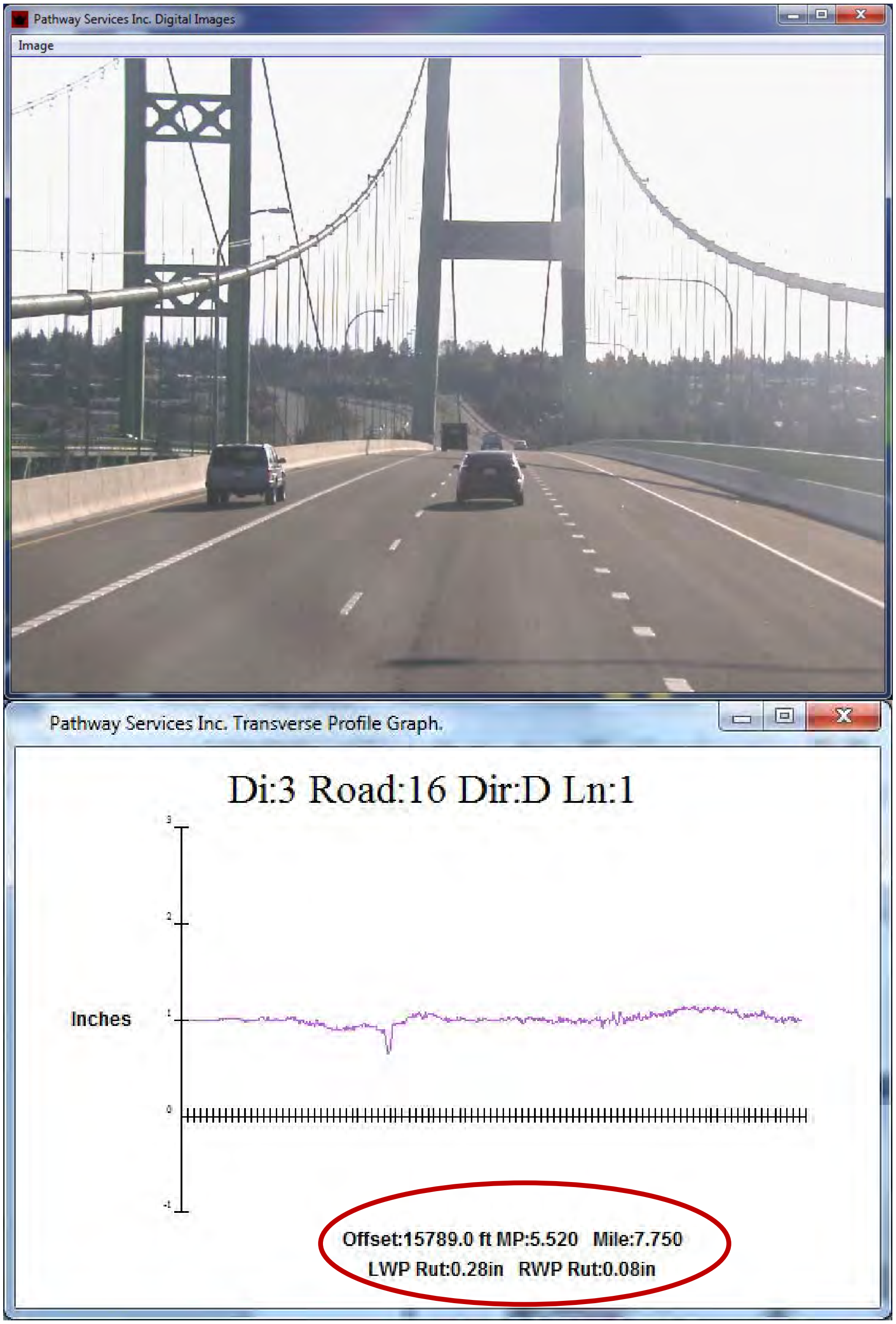

Figure 67. Left wheel path wear depth is 0.28 in. Note that the graph is incorrectly labeled as Lane 1.