Pavement Demonstration Program Project Finalization Perpetual Hot Mix Asphalt Pavement Over Rubblized Concrete Project – I-75 (MDOT Job Number 90279)

Final Technical Report

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16. Abstract

All pavement demonstration projects are evaluated to determine whether there is enough information to create appropriate performance curves and/or their applicability as a Michigan Department of Transportation (MDOT) standard practice. This document provides a comprehensive report on the "Perpetual HMA Pavement Over Rubblized Concrete" demonstration project on I-75 Northbound (NB) in Cheboygan County, with MDOT job number 90279, constructed in the fall of 2008. The pavement structure is comprised of three layers of hot mix asphalt (HMA) (total thickness of 8.5 inches) over 9-inch of rubblized Portland cement concrete (PCC) pavement. The design life is 40 years, designed so that the strain at the bottom of the HMA layer is lower than its endurance limit to prevent fatigue cracking. Typically, MDOT HMA over rubblized projects are designed with a 20-year design life with no specific emphasis on the endurance limit. Overall, the perpetual pavement project on I-75 NB displayed adequate performance. Observed distresses have remained low, so performance trends have been positive. However, it is important to note that certain deterioration related to construction quality was observed, such as cracks around the longitudinal joints between lanes and shoulders and in the transition areas at the project start and end. These pavement distresses have contributed to increased Distress Index (DI), introducing some uncertainty regarding the pavement's durability over the 40-year design period. The study recommends conducting thorough comparative analyses, categorizing the factors leading to distress for enhanced evaluation. The data derived from the pavement analysis in this project provide crucial cost and performance insights. These findings serve as valuable guides for future perpetual pavement projects.

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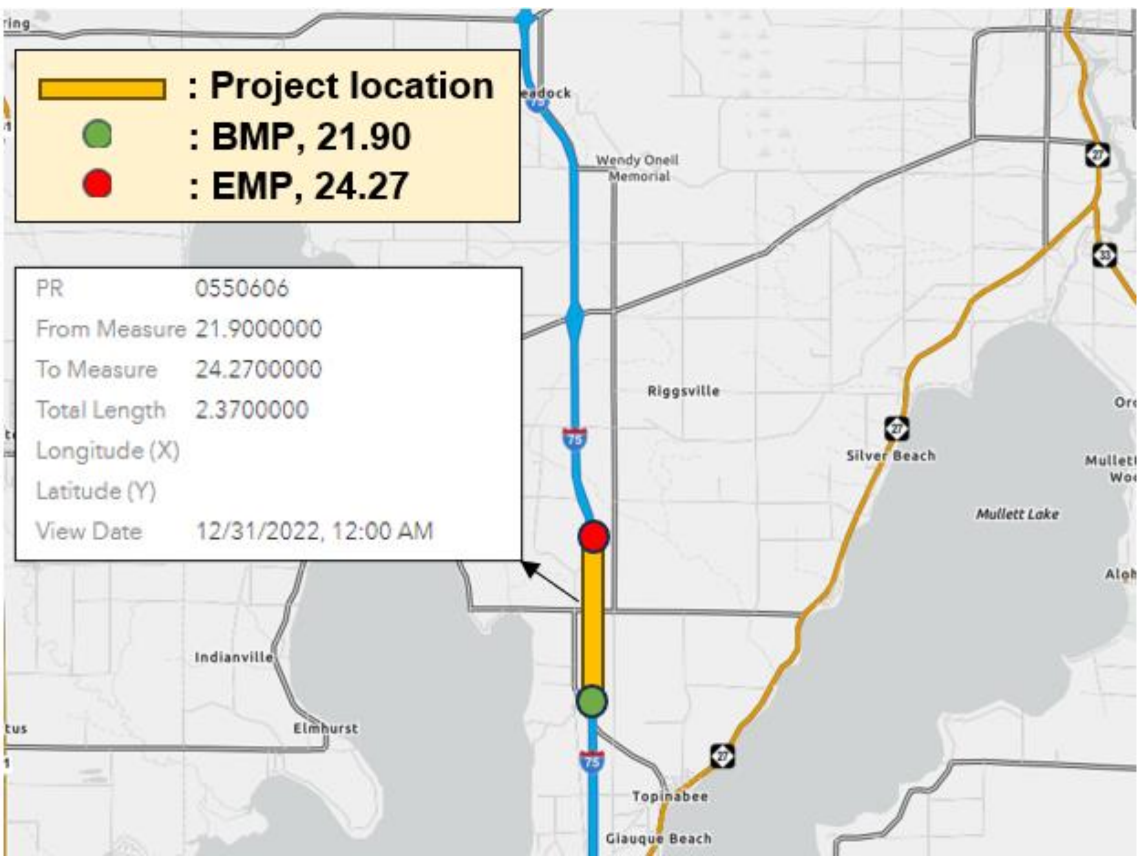
Introduction

Public Act 457 of 2016, MCL 247.651h, contains what is referred to as the pavement life-cycle law. This law requires the Michigan Department of Transportation (MDOT) to conduct a life-cycle cost analysis (LCCA) on projects with pavement costs of $1.5 million or more. The LCCA process is a tool to select the lowest-cost pavement design over the expected service life of the pavement. By law, the LCCA process must include historical information for initial construction and maintenance costs and performance (service life). This information is unavailable for new pavement design types and technologies. Thus, it cannot be used in the pavement selection process until substantial information has been obtained. Accordingly, Public Act 457 of 2016, MCL 247.651i, the pavement demonstration law provides a means for trying new and innovative ideas through demonstration projects. These demonstration projects are not subject to an LCCA process. Pavement demonstration outcomes are intended to increase service life, improve pavement condition, improve ride quality, and/or lower service life costs. Future LCCAs may utilize the cost, performance, and maintenance information from the demonstration projects. Selection of candidate projects is collaborative among MDOT Construction Field Services pavement personnel, MDOT region personnel, and paving industry groups. Once the demonstration project is identified, it goes to MDOT's Engineering Operations Committee for formal approval. Once approved, the project becomes part of the Pavement Demonstration Program. All costs for the demonstration project are funded by the respective MDOT region's rehabilitation and reconstruction template budget. These projects are monitored until a final decision is made regarding the suitability of adopting them as MDOT standard practice. This report evaluates a project for the "Perpetual Hot Mix Asphalt Pavement Over Rubblized Concrete" pavement demonstration fix type on I-75 northbound (NB) in Cheboygan County, MDOT job number 90279.

Project Description

The I-75 NB perpetual hot mix asphalt (HMA) pavement project was constructed in the fall of 2008. This project starts from Topinabee Mail Route Road and continues north for 2.370 miles, as shown in Figure 1. This roadway has two lanes, with each being 12-foot wide. The right shoulder and left shoulder were paved at 10- and 4-foot widths, respectively. The two-lane roadway is comprised of three HMA layers (top, leveling, and base) of 8.5 inches in total thickness over 9-inch of rubblized Portland cement concrete (PCC) for a design life of 40 years to achieve a service life of at least 50 years. In contrast, MDOT's standard practice is to use a 20-year design life for HMA resurfacing over rubblized concrete with current service life estimated at 32 years. Note that the design life of a pavement refers to the theoretical duration until a subsequent major reconstruction or rehabilitation is required, excluding any maintenance, serving as the basis for pavement design. Conversely, the service life pertains to the pavement's life cycle, which encompasses the estimated duration until a major reconstruction or rehabilitation is needed, inclusive of maintenance events. A component of the service life is its initial fix life projection, which is the duration until a subsequent major reconstruction or rehabilitation would be required, excluding any maintenance. However, unlike design life, service and fix life are estimated per the measured data of in-service pavements.

Prior to the demonstration project, the existing pavement was 9 inches of reinforced PCC with 1-3/8 inches of parabolic crown. The existing unbound base material was 3 inches of 23A dense-graded aggregate (noted as "select subbase" in the original plans) over 11 inches of sand subbase. The existing concrete pavement within the project limits was rubblized into a dense-graded unbound base before the HMA resurfacing, using the standard rubblization fix process. In contrast to standard MDOT HMA binder type selection, this project employed high-stress, polymer-modified binder grades for the top and leveling layers to enhance resistance to rutting and improve overall durability. Additionally, the binder high and low-temperature grades of the HMA base course were increased to improve resistance against thermal and fatigue cracking. The HMA asphalt binder improvements and thicker layers are expected to increase the pavement's service life to be considered a "perpetual pavement." This means that the pavement is designed to primarily need only surface repairs, as bottom-up cracking is prevented, and distresses are constrained to the surface. This delays the need for full-depth major fixes such as rehabilitation or reconstruction.



Note: Using Version 23; PR: Physical Road number; BMP: Beginning Mile Point; EMP: Ending Mile Point.

Figure 1. I-75 NB perpetual pavement project location

Table 1 details the pavement cross-section and materials selection for the I-75 NB perpetual pavement project.

Table 1. I-75 NB pavement cross-section

Category	Layer	Thickness (inch)	Material type	Binder PG level
Pavement Resurfacing	Top course	1.5	5E10, high stress	70-28P*
	Leveling course	2.5	4E10, high stress	70-28P*
	Base course	4.5 to 6**	2E10	64-28
Rubblization	Existing concrete pavement	9	Rubblized Concrete Pavement	-
Unbound Layer	Existing base and subbase	14	Granular	-

* "P" refers to polymer modified.
** To achieve a 2% normal crown, the thickness of the HMA base course is estimated to be 6 inches at the centerline and 4.5 inches at the shoulders.

It should be noted that the shoulders did not have existing concrete pavement; instead, the existing shoulders were comprised of 2- to 3-inches of existing HMA pavement. This HMA pavement was crushed and reshaped into a base. The crushed and shaped base for the right shoulder was resurfaced with 4.5 inches of unbound aggregate base and 3.5 inches of HMA pavement (using 5E03 and 4E03 mixes with a PG 58-28 binder grade). The crushed and shaped base for the left shoulder was resurfaced with 8.5 inches of HMA pavement (using the same mixes and binders as the mainline paving).

Traffic Data Assessment and ESAL Estimation

Traffic data plays a crucial role in pavement structure design as it is a significant factor in pavement performance and durability. This section summarizes traffic data used for the original pavement designs as detailed in the 2008 MDOT report, *Structural Analysis of the Pavement Design Recommendations for a Perpetual Pavement Along I-75; Michigan* [1]. For this demonstration project, the original analysis utilized and compared three mechanistic-empirical (ME) based pavement design approaches and the 1993 AASHTO empirical pavement design method. Each design method required specific traffic type parameters, as denoted in Table 2. The traffic data used for all pavement design methods is listed in Table 3.

Table 2. Traffic parameters for different pavement design methods

Design Method	Traffic Parameter
1993 AASHTO pavement design	ESAL, calculated using typical truck factor
Simplistic, Equivalent Annual Modulus Method	ESAL, calculated using traffic and load distribution
PerRoad	ESAL, calculated using traffic and load distribution
MEPDG, Version 1.0	CAADT, traffic, and load distribution

* ESAL: Equivalent Single Axle Load
CAADT: Commercial Average Annual Daily Traffic
MEPDG: Mechanistic-Empirical Pavement Design Guide

Table 3. Traffic data for the original pavement designs (in 2008)

Parameter	Value
Average Annual Daily Traffic (AADT)	7200
Percent of commercial vehicles (%)	13.5
CAADT	988
Traffic growth rate	2%

The ESAL for the 1993 AASHTO design method was calculated using the equation below:

$$ESAL_{Estimated} = CAADT \times 365 \times DD \times LD \times TF \times GF$$

where:
TF = Truck factor
DD = Directional distribution factor
LD = Lane distribution factor
GF = growth factor, $[(1+g)^n - 1]/g$
g = growth rate expressed as a decimal
n = number of years

Table 4 and Figure 2 show the results of $ESAL_{Estimated}$ for the 1993 AASHTO pavement design method. Details of the calculation are shown in Appendix B, Figures 23 to 25.

Table 4. $\mathbf{ESAL_{Estimated}}$ for 1993 AASHTO pavement design method

TF	DD	LD	Growth Rate	ESAL (20 years)	ESAL (40 years)
0.86	0.50	0.95	2.0%	3,579,325	8,898,014

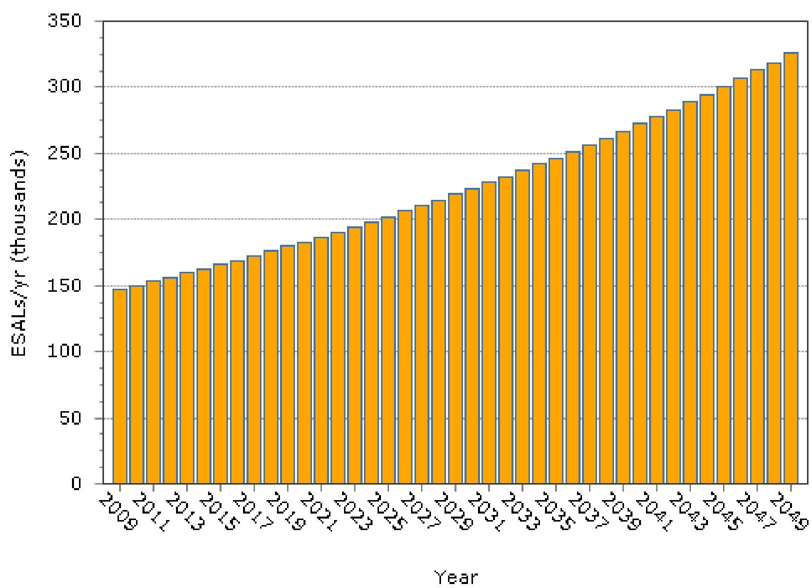


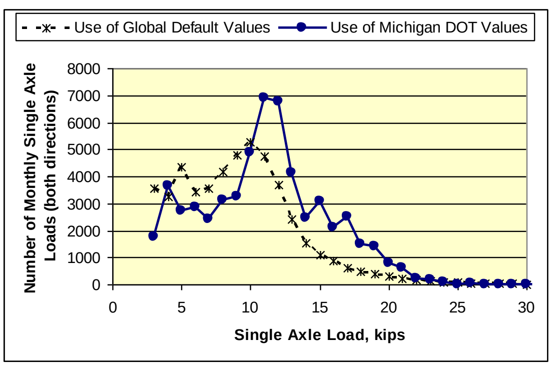
Figure 2. 2009 to 2049 ESAL for 1993 AASHTO pavement design

For the three ME-type methods, the truck class and axle load distributions are needed in addition to the traffic data shown in Table 3. Accordingly, the weigh-in-motion (WIM) site located near Vanderbilt, Michigan, was used to obtain this information. The data was processed through TrafLoad to format the data as MEPDG inputs. Table 5 shows the truck vehicle classification distribution used for the ME methods. Additionally, for informational purposes, the MEPDG global default values for a TTC-11 group (which is described as mixed truck traffic with a higher percentage of single-trailer trucks) are shown.

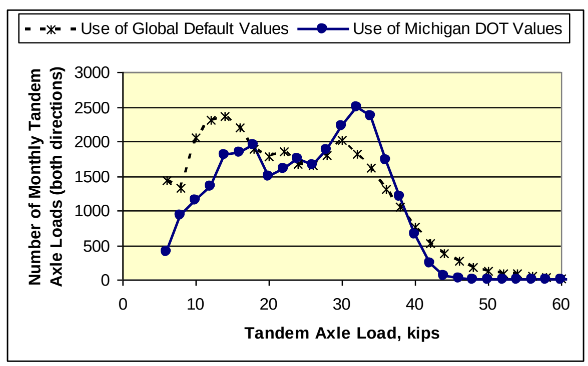
Table 5. Truck vehicle classification normalized volume distribution (in 2009)

Vehicle Classification	Normalized volume distribution, %	
	Michigan I-75 values	Global default values for TTC-11
4	2.0	1.8
5	32.6	24.6
6	4.5	7.6
7	0.5	0.5
8	5.3	5.0
9	31.6	31.3
10	9.4	9.8
11	0.5	0.8
12	0.2	3.3
13	13.4	15.3

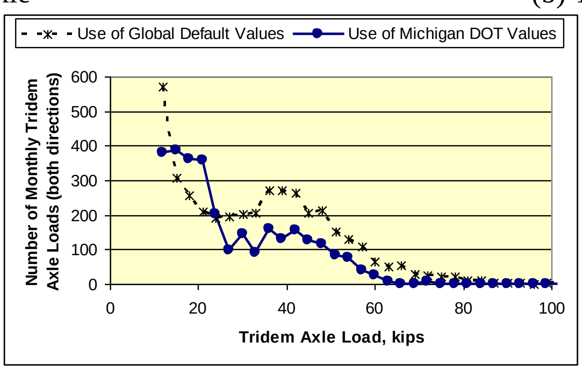
Figure 3 shows the axle load distribution for the single, tandem, and tridem axles. For informational purposes, the MEPDG global default values are also shown.



(a) Single axle



(b) Tandem axle



(c) Tridem axle

Figure 3. Load distribution or spectra used for ME pavement design

In addition to the MEPDG load and distribution data, the simplistic, equivalent annual modulus and PerRoad ME procedures also require a unique $ESAL_{Estimated(ME)}$ as calculated per the load and distribution data. The initial year ESAL in both directions ($ESAL_{InitialY}$) using the traffic volume and axle load distributions was estimated to be 411,330, as shown in Appendix B, Figures 26 to 28. Accordingly, using the $ESAL_{InitialY}$, the TF was calculated as 1.140619. The TF calculation process is shown in the equation below, with the result presented in Table 6.

$$TF = ESAL_{InitialY}/(CAADT \times 365)$$

Table 6. TF calculation process using traffic and load distribution

Type	CAADT	Initial year ESALs ($ESAL_{InitialY}$)	TF
Traffic and load distribution	988	411,330	1.140619

It's worth noting that a higher directional distribution factor (DD) of 0.53 was used for a rural interstate in the MEPDG. Therefore, the base year design lane ESAL was recalculated using the $ESAL_{InitialY}$ of 411,330 per DD of 0.53 and LD of 0.95, which resulted in a base year design lane ESAL of 207,105. Then, multiplied by the GF, the $ESAL_{Estimated(ME)}$ of the design period is obtained. Although the expected design life is 40 years, the ESAL used for the ME methods used design periods of 20 and 50 years. These were calculated using the equation below and are shown in Table 7 and Figure 4.

$$ESAL_{Estimated(ME)} = ESAL_{InitialY} \times DD \times LD \times GF$$

Table 7. The calculated $ESAL_{Estimated}$ for ME methods in different design periods.

DD	LD	$ESAL_{InitialY}$	Growth rate	ESAL (20 years)	ESAL (50 years)
0.53	0.95	411,330	2.0%	8.4 million*	29.3 million*

* These are the ESAL values shown in the 2008 MDOT report [1] that were used for the design evaluation, but per the base year, design lane ESAL and growth rate, the resulting ESAL values should be 5 million and 17.5 million for 20- and 50-year ESAL, respectively.

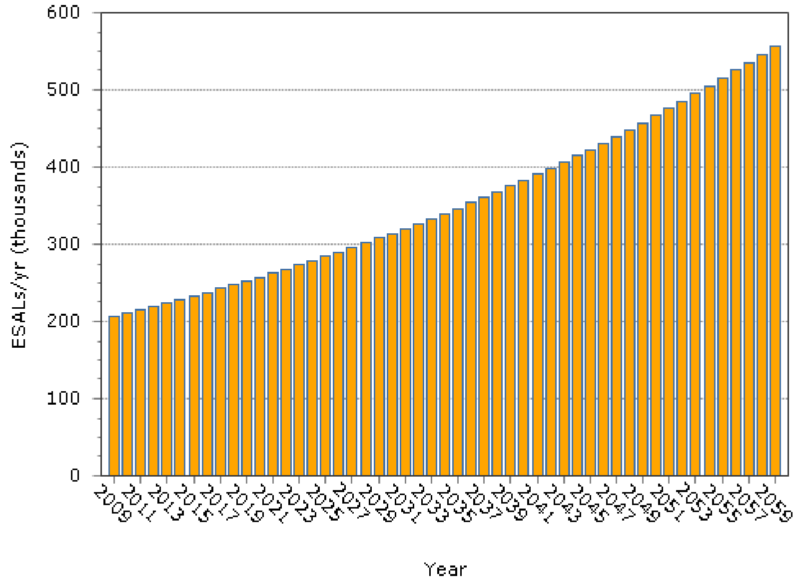


Figure 4. 2009 to 2059 ESAL per Year for ME pavement design

Since the future projections of traffic data used for the original designs were estimated for the pavement design period, assumptions such as growth rate may be inconsistent with actual conditions, potentially leading to inaccurate traffic predictions. Therefore, the actual measured traffic data will be compared with these traffic estimations and predictions. The actual measured traffic data was obtained from the MDOT Transportation Data Management System (TDMS) within the project limits as per TDMS location number 16-0041, located on I-75 south of the Riggsville/I-75 Ramp. This traffic information is shown in Table 8. The TDMS data measurement location is the closest recorded point to the project, with no interchange ramps between them. The comparison between the TDMS recorded traffic data and the traffic for design is shown in Figure 5. The results show that the traffic estimates on the project location were overestimated before 2017 and underestimated after that. If future traffic continues to increase above the estimated prediction, then there is an increased risk of unanticipated pavement distress and a potential reduction in the anticipated service life. However, so far, the initial projected traffic estimate appears reasonable, aligning with the overall yearly average actual two-way CAADT since 2008.

Table 8. Traffic data from TDMS

Year	2-Way		NB	
	AADT	CAADT (FHWA Class 4 and above)	AADT	CAADT (FHWA Class 4 and above)
2022	9,695*	1,649	4,442	755
2021	9,985	1,698	4,575	389
2020	12,075*	2,052	5,133	437
2019	13,659*	1,503	5,806	494
2018	13,825*	1,203	5,877	500
2017	13,702	1,166	5,825	496
2016	6,037	646	N/A	N/A
2015	5,878	N/A	N/A	N/A
2014	5,723	682	N/A	N/A
2013	7,917*	828	N/A	N/A
2012	7,957	848	N/A	N/A
2011	6,364	779	N/A	N/A
2010	7,701	810	N/A	N/A
2009	7,694*	650	N/A	N/A
2008	7,272*	773	N/A	N/A

* MDOT estimated per assumed growth rate.

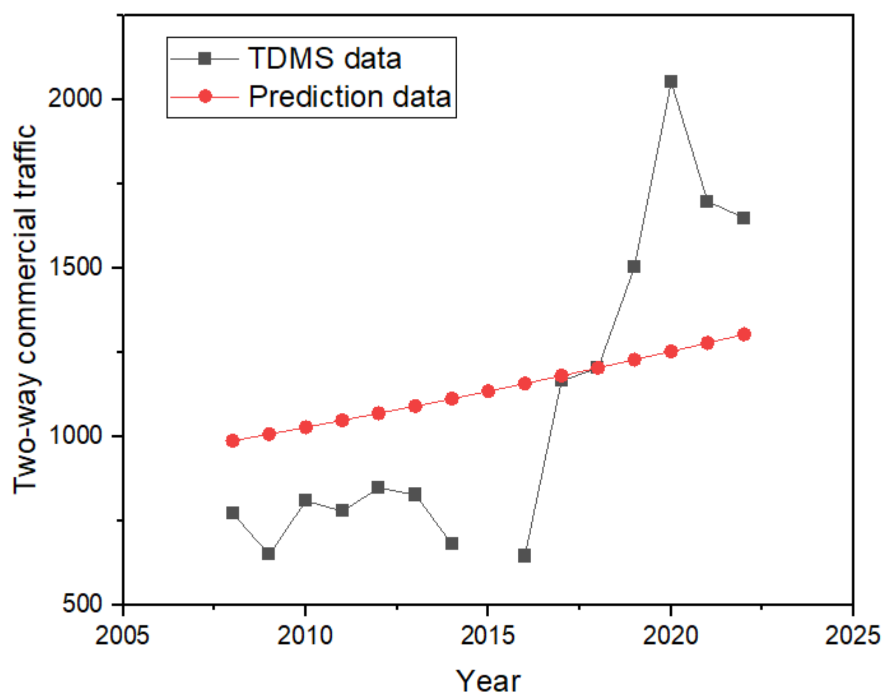


Figure 5. Comparison between TDMS measurements versus predicted two-way CAADT

Pavement Design and Distress Prediction

As introduced in the previous section, the 1993 AASHTO pavement design method and three ME-based methods were used to evaluate the original HMA perpetual pavement design for the rubblized PCC pavement along the I-75 demonstration project. Since the previous section detailed the traffic related design parameters, this section will detail the remaining design aspects. Accordingly, Table 9 denotes the parameters for the existing structure (prior to rubblization and new pavement construction) for the pavement designs as detailed in the 2008 MDOT pavement design report [1].

Table 9. Parameters of the existing structure for pavement design

Existing Layer	Layer Property	Value/Assumption
Subgrade	Subgrade soil type	Loose to moderately compact fine sand (from 20, 5-foot borings)
	AASHTO subgrade soil classification	A-3
	Density of subgrade soil	120 pcf
	Effective resilient modulus of the subgrade soil (for 1993 AASHTO design)	3,800 psi (transformed from falling weight deflectometer, adjusted value to lab test conditions for the spring-thaw season)
	In-place resilient modulus of the subgrade soil (for ME designs)	7,600 psi (estimated per the effective resilient modulus and applying an adjustment factor of 2)
	Poisson's ratio	0.40
	Frost susceptibility of subbase and subgrade soil *	Very low to low (Corps of Engineers classification system)
	Water table depth	15 feet
Subbase (includes base)	Granular thickness	14 inches
	Density	126 pcf
	Resilient modulus	15,000 psi
PCC Pavement	Slab thickness	9 inches

* Shown in Appendix C, Figure 31.

The data parameters for the rubblized PCC layer and resurfacing HMA layers are shown in Tables 10 to 12. Figure 32 in Appendix C shows the equivalent annual modulus values for the HMA layers. A Superpave mixture design procedure was used to determine the target asphalt content. It is worth noting that some materials shown in this design, e.g., SMA and leveling base, were not fully adopted in the final plans used for construction.

Table 10. Parameters of the rubblized PCC layer for pavement design

Layer Property	Value/Assumption
Elastic modulus (psi)*	50,000 (if crushed in high-quality)
	35,000 (if over-rubblization)

* Shown in Appendix C, Figure 33.

Table 11. Parameters of the HMA layers for pavement design

Layer	Layer Description	Thickness, in.	Design air voids, %	In-place air voids, %	Effective asphalt Content by Volume, %
HMA-top layer	Stone matrix asphalt (SMA) provides a rust-resistant, durable mixture, 12.5mm*	2.0	4.0	6.0	10.0
HMA-leveling layer	A gap or coarse-graded aggregate blend has more resistance to rutting, 12.5mm	2.5	4.0	7.5	9.5
HMA-base layer	Dense-coarse-graded high binder content, stiff base HMA mixture provides more crack resistant, 25mm	4	3.0	6.0	9.0
Leveling base	It is recommended to fill in depressions or low spots along the rubblized surface*	As thin as possible	4.0	6.5	11.0

* The construction plans replaced the SMA layer with standard dense-grade HMA, and the leveling base was replaced with an additional variable thickness of the HMA base to fill depressions and/or correct for the crown.

Table 12. Parameters of all HMA layers for pavement design

Layer Property	Value/Assumption
HMA mixtures feature	Not susceptible to moisture damage and stripping, minimum fracture resistance
HMA Dynamic modulus	ME default values for Superpave mix (25 and 12.5 mm) with PG 70-28 asphalt
Layer bond assumption	Full bond maintained between HMA layers over time
Poisson's ratio	0.30

Other remaining parameters used in the ME designs are shown in Table 13. The design period for this project is 40 years; however, a longer analysis period of 50 years was used to determine the increase in distress beyond the design period. In addition, the weather station of Pellston, Michigan, which is within 20 miles of the project, was used for the ME design climate inputs.

Table 13. General ME pavement design parameters

Parameter	Value/Assumption
Design life	20 and 50 years
Tire pressure	120 psi (827 kPa)
Equivalent seasonal temperatures (for PerRoad design)	76°F (summer), 52°F (fall), 49°F (spring), 26°F (winter)

The result of the 1993 AASHTO pavement design for a 40-year design life is 7.5-inch HMA over the rubblized PCC layer, with details presented in Appendix C, Figure 30.

The analysis results for the simplistic, equivalent annual modulus ME method include bottom-up cracking and distortion, as shown in Tables 14 and 15. As shown, the computed strains are less than the permissible strains for both design lives, so the predicted bottom-up cracking and distortion are considered to be acceptable for the HMA pavement design.

Table 14. Simplistic, Equivalent Annual Modulus method predictions for bottom-up fatigue cracking

Total HMA structural overlay thickness, in.			7.5	8.5	9.0
Estimated endurance limit for the high asphalt content, stiff asphalt HMA base, in./in.			0.000045		
Permissible tensile strain, in./in.		20-year design traffic	0.000138		
		50-year design traffic	0.000094		
Rubblized layer E = 50 ksi	Computed tensile strain at the bottom of HMA base; in./in.		0.0000679	0.0000582	0.0000540
	Predicted bottom-up fatigue cracks, %	20-year design traffic	1.2	0.7	0.6
		50-year design traffic	4.2	2.5	2.0
Rubblized layer E = 35 ksi	Computed tensile strain at the bottom of HMA base; in./in.		0.0000728	0.0000619	0.0000573
	Predicted bottom-up fatigue cracks, %	20-year design traffic	1.5	0.9	0.7
		50-year design traffic	5.3	3.1	2.4

Table 15. Simplistic, Equivalent Annual Modulus design predictions for distortion

Total HMA structural overlay thickness, in.			7.5	8.5	9.0
Sand subbase layer	Permissible vertical strain, in./in.	20-year design traffic	0.000408		
		50-year design traffic	0.000300		
	Computed vertical strain at the top of sand subbase, in./in.		0.000184	0.000156	0.000141
AASHTO A-3 subgrade soil	Permissible vertical strain, in./in.	20-year design traffic	0.000348		
		50-year design traffic	0.000256		
	Computed vertical strain at the top of subgrade, in./in.		0.000157	0.000136	0.000127

* The computed vertical strains for the sand subbase and subgrade soil are for the condition with the lower modulus of the rubblized PCC layer (35,000 psi).

The analysis results for the PerRoad ME method show the reliability of the pavement achieved per the target strain, as presented in Table 16.

Table 16. PerRoad design predictions of reliability

Total HMA structural overlay thickness, in.		7.5	8.5	9.0
Endurance limit (tensile strain at the bottom of the HMA base layer)	100 micro-strains	94.6%	96.8%	99.1%
	75 micro-strains	88.3%	91.5%	94.5%
Vertical compressive strain at the top of subbase layer, 400 micro-strains		99.9%	100%	100%
Vertical compressive strain at the top of subgrade, 300 micro-strains		90.2%	95.4%	99.5%
Surface deflection, 18 mils		89.7%	91.9%	92.9%

* The reliability levels included in this table are for the condition for which the modulus of the rubblized PCC slab is 50,000 psi.

The analysis results for the MEPDG, Version 1.0, include the prediction of cracking, rutting, and IRI at 95% reliability using a 50-year design life. Note that the endurance limit was not included as an input, and an elastic modulus of 50,000 psi was used for the rubblized PCC layer. The MEPDG design results are shown in Table 17.

Table 17. Summary of the predicted distresses using the MEPDG, 95% reliability

Performance indicator (threshold value)	**Distress value predicted at year 50**		**Year in which the threshold distress value is exceeded**	
Total HMA structural overlay thickness, in.	**8.5**	**7.5**	**8.5**	**7.5**
Bottom-Up area fatigue cracks, % (2)	1.86	1.90	50+	50+
Surface initiated longitudinal cracks, ft./mi. (1,250)	2,069	2,827	24	13
Thermal cracks, ft./mi. (1,000)	108	108	50+	50+
Total rutting, in. (0.40)	0.61	0.64	18*	15
IRI, in./mi. (170)	250	251	29	29

* NOTE: All other ME-based methods indicate sufficient structure to protect the subgrade soils from excessive distortions for the 7.5 and 8.5-inch overlays.

As a result of the original design evaluation using the four pavement design methods, the 8.5-inch HMA surfacing was recommended. This design thickness was best suited to meet the perpetual HMA requirements for this project on I-75 to resist long-term bottom-up fatigue cracking. While the MEPDG design predicted increased rutting, all other designs did not. Additionally, while top-down cracking and IRI thresholds may be exceeded before 40 years in service, these could be mitigated with surface repairs. It was noted that the 7.5-inch HMA surfacing might also be sufficient, but this would depend on the achieved strength of the rubblized layer, which could vary due to construction.

Construction and Quality Control

According to the construction plans and May 12, 2008 pre-construction meeting notes, the I-75 NB perpetual over rubblized concrete project is from Station (Sta) Point of Beginning (POB) 525+66.99 to Points of Ending (POE) 651+00, as shown in Figure 6. The rubblized sections of the PCC are from Sta 530+00 to 601+00 and Sta 609+00 to 651+00. The segments from Sta 525+66.99 to 530+00 and Sta 601+00 to 609+00 were designated for full-depth reconstruction and pavement removal to preserve the under clearance of the bridges. The HMA pavement resurfacing encompasses the entire length from Sta 525+66.99 to 651+00 (around 2.37 miles).

The rubblization process commenced in September 2008, as documented in the inspector's daily reports (IDRs). The construction was completed with the final surface HMA paving in October 2008. Therefore, the entirety of the demonstration project, from rubblization to the final layer of HMA, spanned approximately one and a half months. See Appendix D, Figures 34 to 37 for the IDRs that denote initial rubblization and final paving. The record of material collection during the construction is shown in Appendix D, Figure 38.

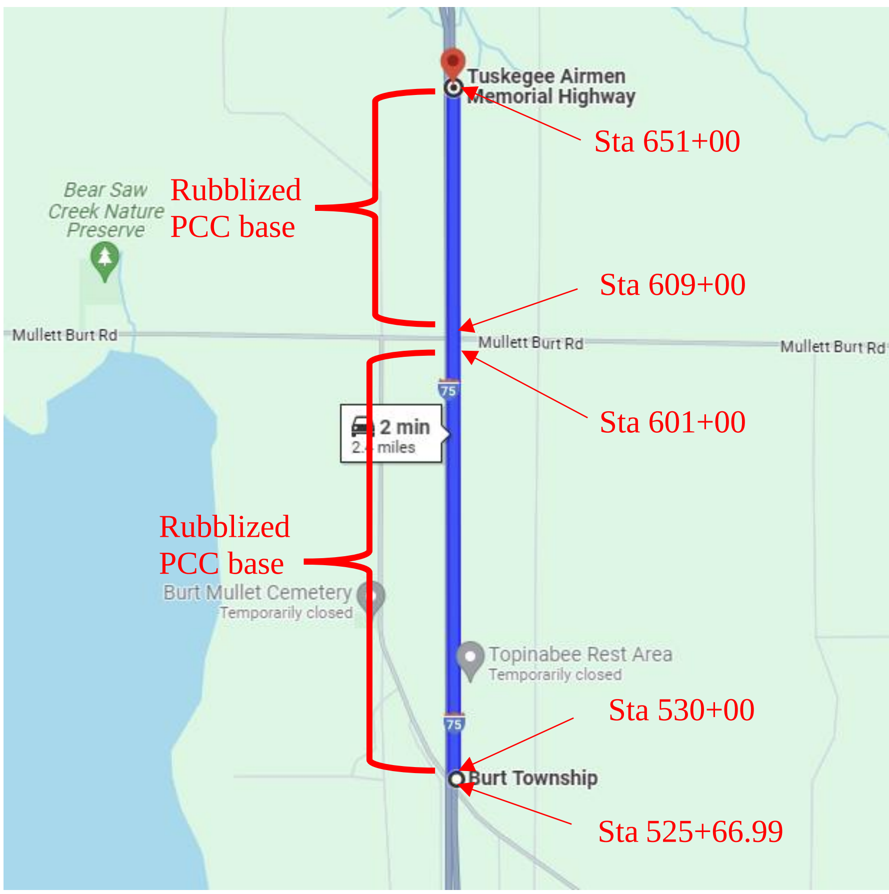


Figure 6. Schedule of the I-75 NB project construction

Figure 7 presents the active construction occurring near Sta 601+00 in September 2008 from Google Maps. In the right lane, the boundary between the new aggregate base (for full-depth reconstruction) and rubblized concrete is shown. In this Figure, the concrete in the left lane has not yet been rubblized or removed.

Figure 7. Construction around Sta 601+00 in September 2008 from Google Maps

As previously noted, the HMA used in construction slightly differs from those recommended by the pavement design. Specifically, the HMA top course layer for construction is a standard dense graded, 5E10, whereas the design recommended SMA. Also, instead of a leveling base layer above the rubblized concrete, the HMA base course layer thickness was increased to achieve the normal crown and fill rubblized imperfections. Accordingly, the thickness of the HMA base course is estimated to be 6 inches at the centerline and 4.5 inches at the shoulders. These are minor changes that will not significantly impact the structural-related characteristics.

According to the field evaluation report during the construction in 2008 (shown in Appendix E, Figure 39), a Multi-Head Breaker (MHB) was used for rubblizing, requiring a second pass to cover the full lane width. Observations included a contractor's backhoe dig at Sta 621+20, revealing larger pieces stuck. A subsequent hand dig exposed steel and mesh near the contractor's hole. Samples were collected from rubblized material at Sta 621+15, and the MHB operator noted "moon-shaped" cracks due to insufficient shoulder support during the rubblizing process. Additional digs at Sta 631+60 and 625+83 displayed painted steel with effective debonding. The MHB operator reported that full-depth repairs were rubblizing well, similar to the old concrete, but the operator sometimes faced challenges in breaking the material further at the shoulder due to the lack of shoulder support. This is likely because the shoulder material was different (existing HMA that was crush and shaped) from the mainline lanes, so the edge was less confined for rubblization and had less support. Some field rubblization pictures for the I-75 NB perpetual pavement project are provided in Figure 8.

Figure 8. Field rubblization process for the I-75 NB project

After the construction, the Ground Penetrating Radar (GPR) test was conducted on the I-75 NB perpetual pavement project to measure pavement thickness (HMA layers) from BMP (where distance = 0), as presented in Figure 9. Further, the HMA thickness frequency histogram was plotted, as shown in Figure 10. It should be noted that GPR measurements are estimates since core data is not available to validate them. The results indicate that the resurfaced HMA thickness meets the designed 8.5-inch HMA requirements, with a few measured points falling below 8.5 inches. Most points tested had thicknesses between 9 to 11.5 inches, resulting in an average HMA thickness of 10.27 inches.

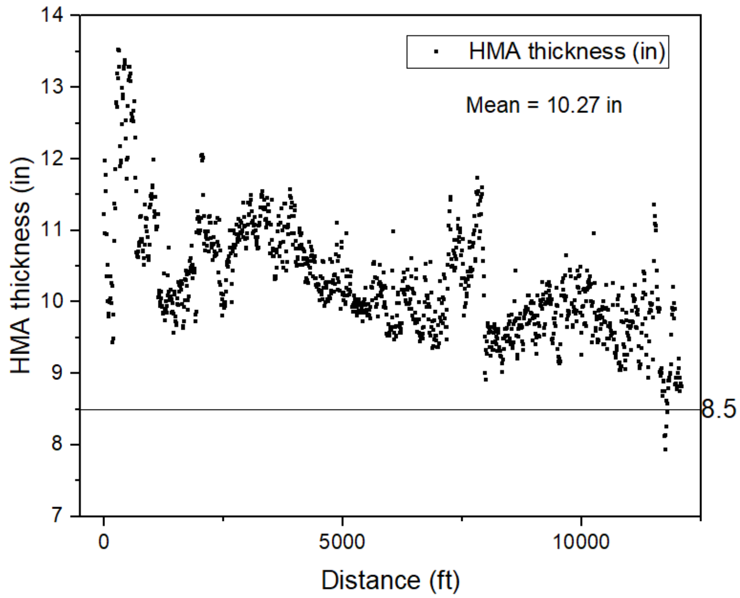


Figure 9. HMA thickness along the distance for the I-75 NB project

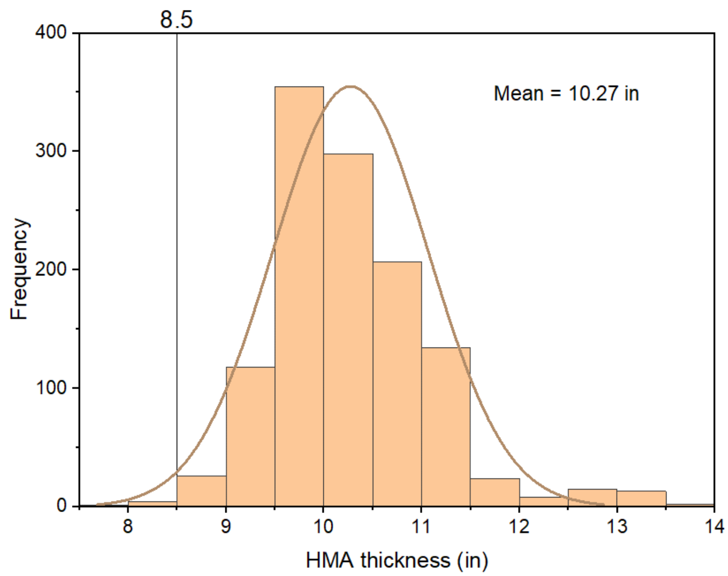


Figure 10. HMA thickness distribution for the I-75 NB project

Pavement Condition Data Analysis

For MDOT roadways, pavement condition (used for performance assessment) for each project is measured by a variety of methods, including rutting, MDOT's Distress Index (DI), and International Roughness Index (IRI). Rutting is the difference in elevation across the pavement surface plane defined by its transverse cross slope, measured in each wheel path separately in inches. The DI measurement is the total accumulated distress point value for a given pavement section normalized to a 0.1-mile length, collected per a sampling of the 0.1-mile length. It is a unitless value that indicates a pavement's 2-dimensional surface distress condition (so faulting and rutting are not included). The IRI measurement is the roughness of the road profile in inches/mile (so that physical distresses such as faulting and rutting can impact its measurement). Condition data measurements are to be taken in the rightmost lane (outside lane) unless this lane is unavailable due to construction or other lane obstruction. The lane configuration of the I-75 NB perpetual over rubblized concrete project is presented in Figure 11.

Figure 11. Lane configuration for the I-75 NB project, Google Maps Image, October 2023

Note that historically through 2019, MDOT network-level data collection for DI, IRI, and rut-or-fault was intended to be obtained every other year for any given route segment (including both directions of divided routes). However, the following is a list of exceptions to that biennial schedule:

- Starting in 2009, the annual IRI collection began in at least one direction of all National Highway System (NHS) routes.

- Starting in 2018, the annual IRI collection on at least one direction of all NHS routes was reduced to only Interstate routes.
- Also, starting in 2018, the annual collection of DI and rut-or-fault began (in addition to IRI) on one direction of the Interstate routes.
- Schedules for data collection are subject to roadway availability, so construction or similar operations may prevent data collection for that anticipated year.

A summary of yearly IRI, rutting, and DI on the I-75 NB project is presented in Table 18 and Figures 12 to 14.

The pavement has remained very smooth, with the IRI consistently well below 95 inches/mile, which is the FHWA threshold for good condition (per FHWA 23 CFR 490.313). Over time, the IRI has increased very slowly, but there was a distinct decrease in 2021. This decrease can be attributed to a chip seal with fog coat capital preventive maintenance project (MDOT job number 204267) that occurred in August 2020.

The overall average rutting is low, remaining below 0.2 inches, which also meets the FHWA threshold for good condition (per FHWA 23 CFR 490.313). Although early rutting values were higher than later ones, this may be attributed to factors such as traffic compaction after construction and/or data noise. Since the rutting has remained low and has not shown an increase with pavement age, this indicates a strong structure.

For DI, values remain low and far below 50, which is the value used in the *MDOT Pavement Selection Manual* [2] to approximate the end of service life. This indicates that this project has been in good to fair condition. There was a DI spike in 2018, but then it decreased in 2019. A crack treatment was conducted in 2017 (MDOT job number 200432), so the increase in 2018 may be due to the cracks being more visible due to the treatment or excessive sealing. However, this does not explain why the DI decreased from 2018 to 2019 since no maintenance event was observed. Therefore, the DI after 2017 may be inaccurate, particularly the 2018 DI.

Table 18. Yearly Progression of IRI, rutting, and DI for the I-75 NB project

Data Year (Pavement Age)	**IRI**	**Rutting**	**DI**
2009 (1)	42	-	0.033
2010 (2)	43	0.11	-
2011 (3)	45	0.13	0.052
2012 (4)	46	-	-
2013 (5)	45	0.05	1.416
2014 (6)	48	-	-
2015 (7)	47	0.13	0.633
2016 (8)	48	-	-
2017 (9)	49	0.05	1.382
2018 (10)	54	0.04	13.159
2019 (11)	60	0.04	8.4
2020 (12)	-	-	-
2021 (13)	48	0.07	-
2022 (14)	49	0.07	-

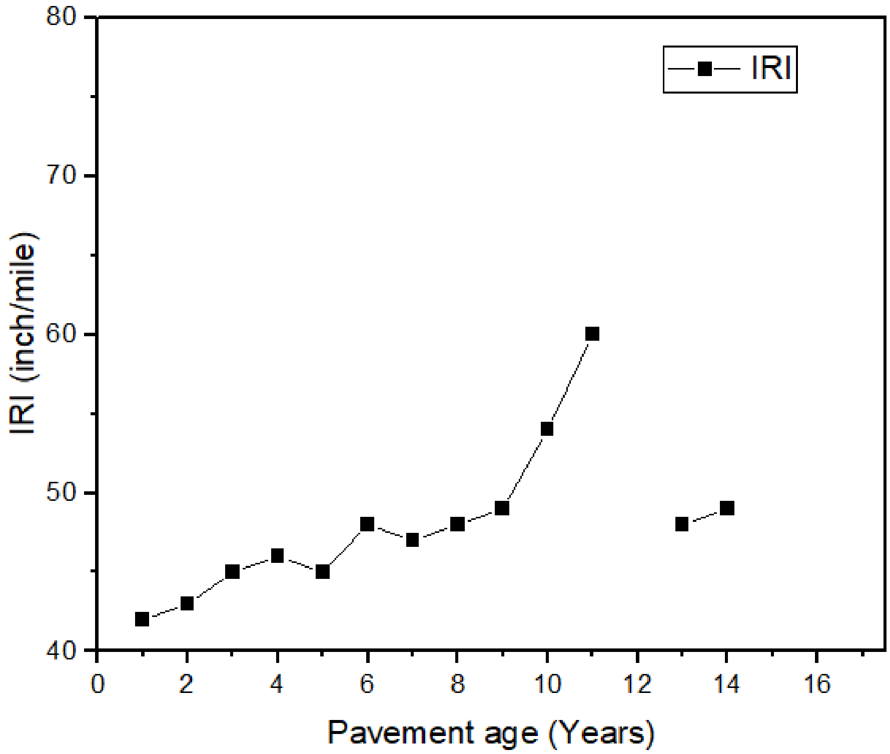


Figure 12. Yearly IRI data for the I-75 NB project

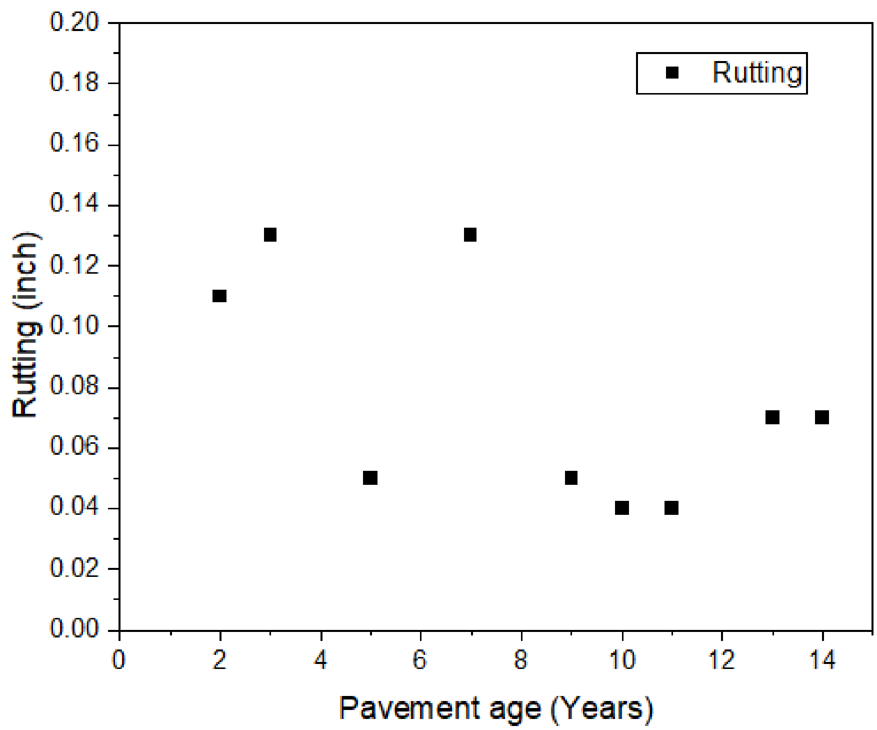


Figure 13. Yearly rutting data for the I-75 NB project

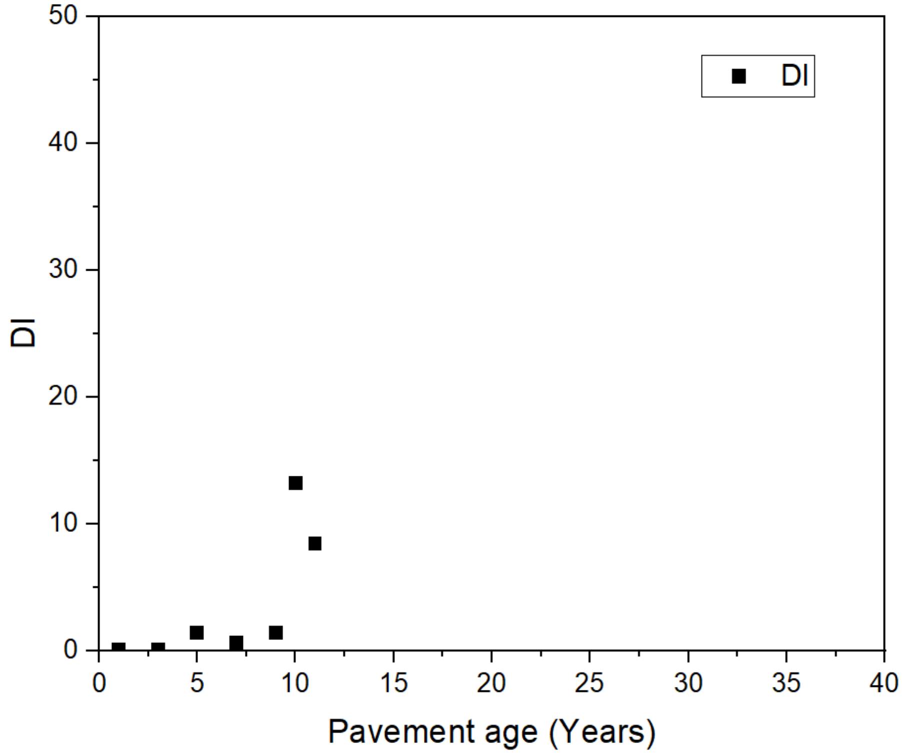


Figure 14. Yearly DI data for the I-75 NB project

Detailed breakdown of the yearly rutting and IRI data per tenth mile along the project length are shown separately in Tables 19 and 20, and Figures 15 and 16, respectively. Note that the IRI and rutting values are an average of the data from the right and left wheel paths. The DI breakdown per tenth mile for 2018 and 2019 are shown in Table 21 and Figure 17.

Rutting tenth mile data was inconclusive and did not indicate any unique trends. However, the IRI tenth mile data was found to be significantly higher at the start and end of the project. This corresponds with the project construction joints and field investigation pictures, as shown in Appendix F, Figures 47 to 49. As shown, these joints display cracking and raveling distress.

The DI tenth mile data found results similar to IRI, where DI along the project length was highest near the project start and end construction joints. It also showed increased cracking near the Bullett Burt Road bridge and rest area ramps. Sympathy cracks from the longitudinal joint, shoulders, and ramps may have started to progress into the lane and accounted for the increased DI.

Table 19. Yearly pavement rutting data per 0.1 mile for the I-75 NB project

Pavement length (mile, south to north direction)	2009	2010	2017	2018	2019	2021	2022
0.1	0.12	0.13	0.07	0.02	0.03	0.11	0.10
0.2	0.1	0.12	0.07	0.04	0.04	0.08	0.08
0.3	0.11	0.12	0.08	0.05	0.06	0.09	0.09
0.4	0.11	0.12	0.06	0.04	0.04	0.08	0.08
0.5	0.1	0.1	0.05	0.03	0.05	0.06	0.08
0.6	0.11	0.12	0.06	0.04	0.05	0.08	0.08
0.7	0.09	0.1	0.05	0.04	0.03	0.05	0.07
0.8	0.08	0.09	0.06	0.04	0.04	0.06	0.07
0.9	0.1	0.11	0.05	0.04	0.05	0.06	0.07
1.0	0.1	0.1	0.04	0.04	0.04	0.05	0.07
1.1	0.1	0.08	0.04	0.04	0.04	0.05	0.06
1.2	0.09	0.1	0.04	0.04	0.04	0.04	0.06
1.3	0.12	0.11	0.05	0.03	0.04	0.06	0.08
1.4	0.1	0.13	0.05	0.03	0.04	0.07	0.08
1.5	0.11	0.12	0.05	0.04	0.04	0.08	0.08
1.6	0.12	0.12	0.06	0.03	0.05	0.07	0.07
1.7	0.12	0.13	0.07	0.04	0.05	0.07	0.07
1.8	0.11	0.13	0.05	0.03	0.04	0.08	0.08
1.9	0.11	0.11	0.05	0.05	0.06	0.07	0.07
2.0	0.1	0.11	0.05	0.04	0.05	0.06	0.07
2.1	0.12	0.11	0.06	0.04	0.04	0.08	0.08
2.2	0.11	0.11	0.05	0.04	0.05	0.07	0.07
2.3	0.11	0.12	0.05	0.05	0.05	0.07	0.07
2.4	0.11	0.13	0.07	0.03	0.02	0.08	0.07
Average	0.11	0.11	0.06	0.04	0.04	0.07	0.08

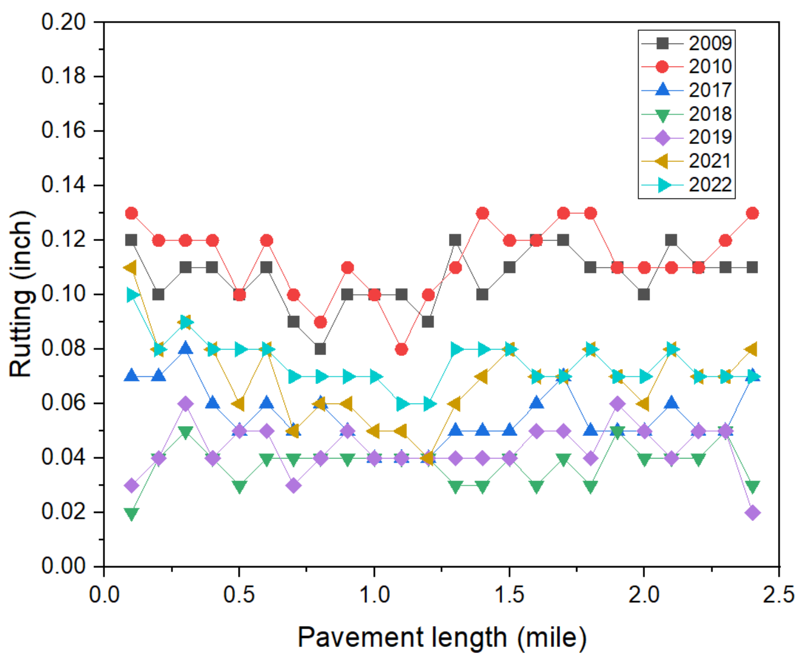


Figure 15. Rutting per 0.1 mile along the I-75 NB project

Table 20. Yearly pavement IRI data per 0.1 mile for the I-75 NB project

Pavement length (mile, south to north direction)	2009	2010	2017	2018	2019	2021	2022
0.1	55	57	82	89	92	77	80
0.2	41	46	50	54	61	48	47
0.3	44	41	48	51	57	48	48
0.4	42	43	46	51	48	43	43
0.5	40	39	50	48	54	44	43
0.6	58	60	60	64	57	58	57
0.7	45	48	46	55	56	45	45
0.8	36	36	39	45	50	48	49
0.9	46	46	53	55	61	57	56
1.0	41	43	43	49	51	42	42
1.1	34	33	37	44	48	37	39
1.2	33	36	50	55	63	48	52
1.3	42	45	45	52	52	44	46
1.4	35	34	41	47	47	45	46
1.5	49	48	57	58	67	55	56
1.6	39	39	49	49	64	47	48
1.7	40	44	46	48	58	45	47
1.8	39	39	44	48	59	45	45
1.9	34	37	42	45	60	38	39
2.0	43	43	52	61	64	51	52
2.1	40	42	47	53	68	49	49
2.2	38	38	46	49	55	43	44
2.3	34	38	49	49	61	43	45
2.4	62	58	77	83	92	73	74
Average	42	43	50	54	60	49	50

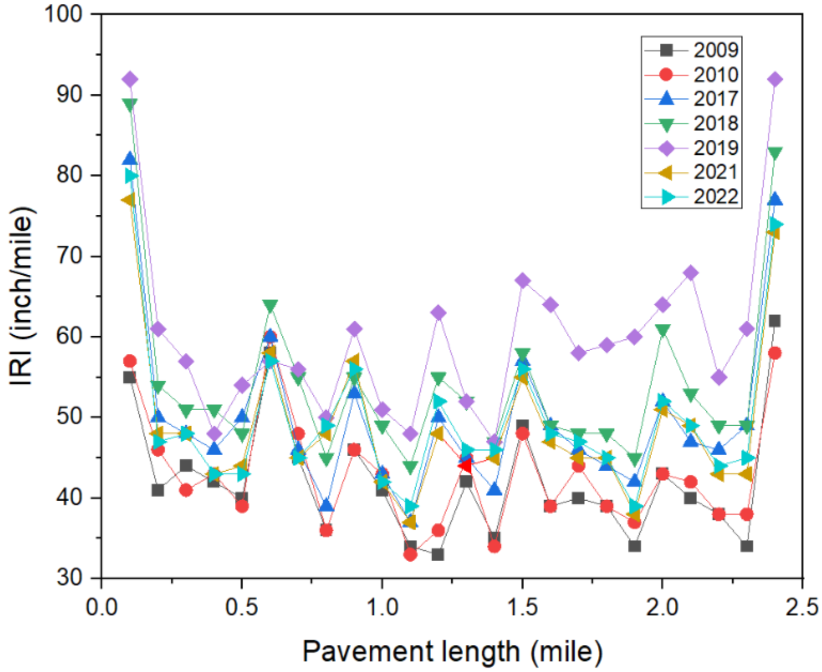


Figure 16. IRI per 0.1 mile along the I-75 NB project

Table 21. Yearly pavement DI data per 0.1 mile for the I-75 NB project

Pavement length (mile, south to north direction)	2018	2019
0.1	15.1	11.89
0.2	14	7.08
0.3	8.64	9.2
0.4	11.12	7.94
0.5	14.34	7.78
0.6	31.84	11.36
0.7	23.6	7.84
0.8	13.44	7.4
0.9	16.06	8.64
1.0	8.42	7.12
1.1	15.52	8
1.2	7.54	7.38
1.3	11.2	7.5
1.4	8.93	9.04
1.5	9.06	8.41
1.6	13.61	10.07
1.7	15.89	9.02
1.8	16	8.02
1.9	10.6	7.56
2.0	7.14	8.6
2.1	9.47	8.43
2.2	10.84	7.16
2.3	7.4	6.8
Average	13.0	8.4

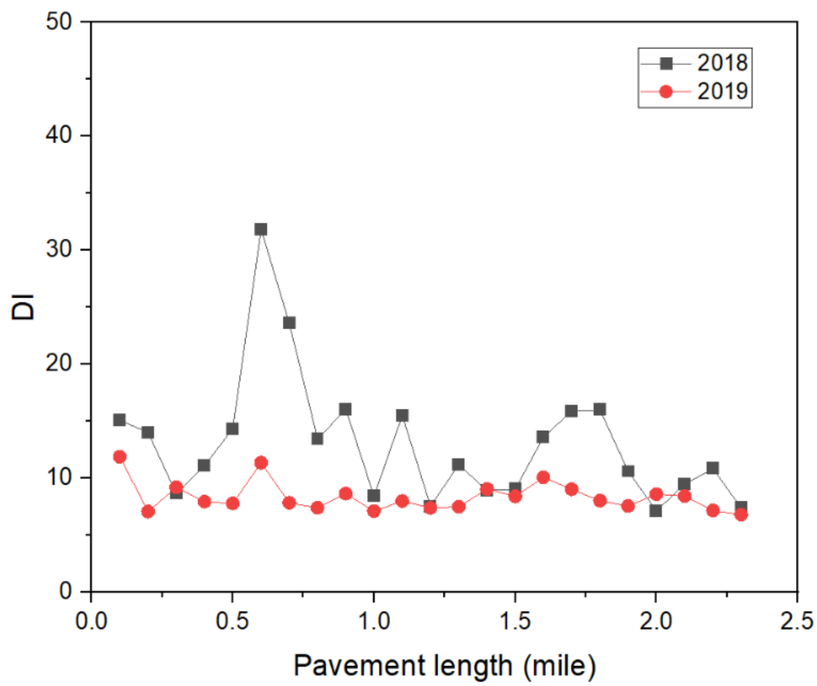


Figure 17. DI per 0.1 mile along the I-75 NB project

Pavement Condition Survey Findings

Annual pavement condition field assessments of all MDOT demonstration projects are documented in the MDOT Pavement Demonstration Program Legislative Status Report, *Pavement Demonstration Program Status Report Public Act 457 of 2016* [3]. Typically, this annual report includes a summary of visual distress conditions, including cracking and repairs. These reports are derived from the field survey notes. As an example, the 2022 field evaluation notes are shown in Appendix E, Figures 40-41. Survey pictures are shown in Appendix F. Annual surveys collected data in both lanes of this project, so the pavement condition data measurements (used for performance assessments) may not be directly comparable to the annual site surveys since condition data measurements are taken in one lane. Key notes from the annual status reports are shown in Table 22. The associated observed cracking lengths are shown in Table 23 and Figure 18. Note that crack lengths exclude those at the longitudinal construction joint since this is a common crack occurrence due to construction operations and may not indicate the pavement's structural characteristics.

Table 22. Summary of the I-75 NB perpetual demonstration program status reports

Report Date	Key Observations
Mar. 2010	Pavement in as-constructed condition with no distress.
Feb. 2011	No distress noted, but noticeable longitudinal paving joints and potential raveling issues.
Jan. 2012	No distress was noted.
Feb. 2013	Construction related longitudinal cracking observed at the paving joints.
Jan. 2014	Slight longitudinal cracking at paving joints and crack at the transition area.
Jan. 2015	No distress except raveling in spot locations along longitudinal joints.
Jan. 2016	3 transverse cracks across both lanes (6 total) and separating longitudinal joints.
Jan. 2017	4 transverse cracks across both lanes (8 total), separating longitudinal joints, localized segregation, and signs of age-related oxidization.
Jun. 2018	5 transverse cracks in the left lane and 9 in the right lane (14 total), separating longitudinal joints, localized segregation, and a pothole observed in right lane.
Jun. 2019	28 transverse cracks in total, separating longitudinal joints, localized segregation, some potholes observed at the start and ending transitions (mostly in the right lane).
Jun. 2020	45 transverse cracks in total, separating longitudinal joints (up to 2-inch width), localized segregation, some potholes and delamination observed at the start and ending transitions (mostly in the right lane).
Jun. 2021	Chip seal maintenance occurred prior to this survey, transverse cracking decreased to 18 locations in total, no longitudinal cracking, longitudinal joint widening reduced (down to 1-inch width), and potholes and delamination reduced.
Jun. 2022	Minimal change in distress; 20 transverse cracks, no longitudinal cracking, longitudinal joint mostly unchanged from prior year, some increased raveling with potholes and delamination at the start and ending transitions (mostly in the right lane).

Table 23. Annual survey observed crack lengths on the I-75 NB project

Year	Pavement Age	Transverse Cracking (feet/lane-mile)	Longitudinal Cracking (feet/lane-mile)
2008	0	0	0
2009	1	0	0
2010	2	0	0
2011	3	0	0
2012	4	0	0
2013	5	0	0
2014	6	0	0
2015	7	0	0
2016	8	15	0
2017	9	20	0
2018	10	35	0
2019	11	51	1
2020	12	60	5
2021	13	46	0
2022	14	48	0

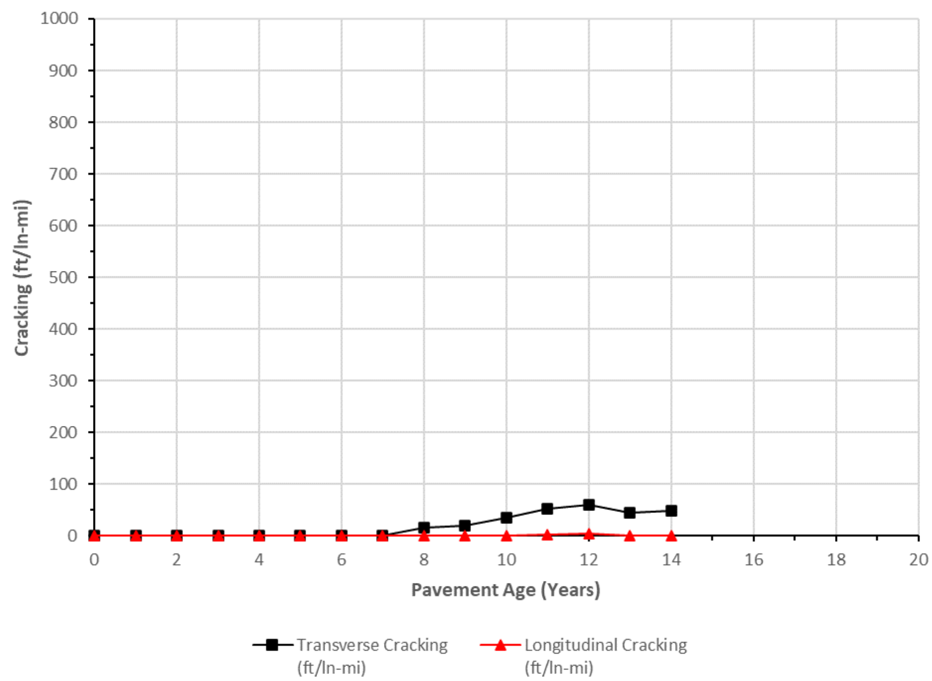


Figure 18. Annual survey observed crack lengths on the I-75 NB project

According to the 2022 field survey of I-75 NB, after 14 years of service, approximately 48 feet/lane-mile transverse cracking and no longitudinal cracking were observed. This is very low over this timespan. Even considering the highest observed cracking of 60 feet/lane-mile in 2020 (prior to the maintenance project in August 2020), total cracking was low. Furthermore, cracks have remained tight (less than 1-inch wide). While localized issues have been observed, such as potholes, surface delamination, and longitudinal joint separation (up to 2-inch wide), these are largely construction-related issues due to inadequate density at construction joints and are not representative of the integrity of the pavement structure. Furthermore, the maintenance project in 2020 has largely mitigated these localized issues. Accordingly, field surveys have described the pavement's overall performance as good. It should be noted that annual reporting condition ratings of good, fair, and/or poor are assigned to each project based on a subjective evaluation of the condition at the time of the latest field visit and are only intended to provide a general sense of the performance (in terms of anticipated distress and ride quality per the design type), so this qualifier may not reflect the final recommendation of this pavement after all relevant information is obtained to make a final determination. The annual field condition survey observations are mostly consistent with the progression performance data measurements, except for the inconsistent DI value in 2018.

Performance Comparison and Evaluation

To assess the relative pavement performance, the I-75 NB perpetual HMA over rubblized concrete project will be compared with standard MDOT HMA over rubblized concrete pavement data per the *MDOT Pavement Selection Manual* [2]. For comparison, the estimated fix life (estimated life pavement would last without maintenance, occurring at 50 DI) of the I-75 NB demonstration and standard pavement are shown in Figure 19. The service life (estimated life pavement would last with maintenance, occurring at 50 DI) of the standard pavement with the I-75 NB DI values are shown on Figure 20. It should be noted that the demonstration project DI values may exhibit more variability than the statewide project values since its data is derived from a single project rather than a broad set of values.

As shown in Figures 19 and 20, DI within the first 10 years indicates that the I-75 NB project values are much lower than those of the standard alternative. However, I-75 NB DI notably increased at the pavement age of 10 years (2018). As previously described, construction-related longitudinal joint separation and transition area distresses largely contribute to this increase.

As shown in Figure 19, due to the DI increase at year 10, the trend of DI values for the I-75 NB project would forecast that the fix life at 50 DI would occur at the age of 14 years, whereas this is 16 years for the standard alternative. However, The I-75 NB pavement has consistently performed well, and the elevated DI does not seem to correlate with visual surveys or other performance measurements. Its latest DI measurement is still very low (8.4 DI at age 11 years), so it is unlikely that future distress will unexpectedly accelerate.

Furthermore, its first maintenance event occurred at 9 years, which is 2 years after the first maintenance event for the standard alternative. As shown in Figure 20, if we assume the same number of maintenance events, timing between each, and similar improvements to DI as estimated for the standard alternative, then the I-75 NB demonstration could be estimated to have 2 more years of service life beyond that of the standard version. However, this estimation of 34 years is much lower than the anticipated 50 years of service life.

Therefore, both the fix and service life projections seem too low. Consequently, this single demonstration project may not produce an adequate performance curve for sufficient comparison to its standard version.

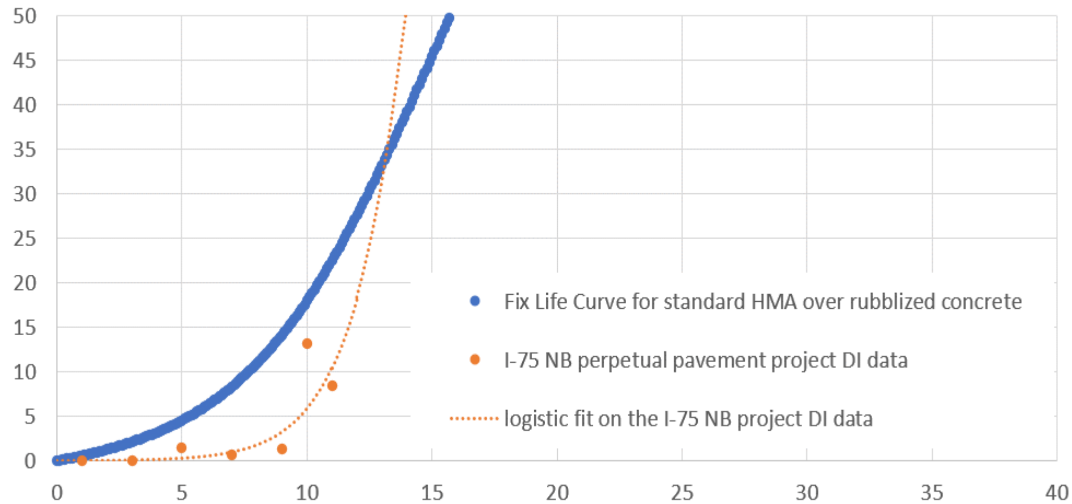


Figure 19. Comparison on I-75 NB DI trend with fix life of standard pavement

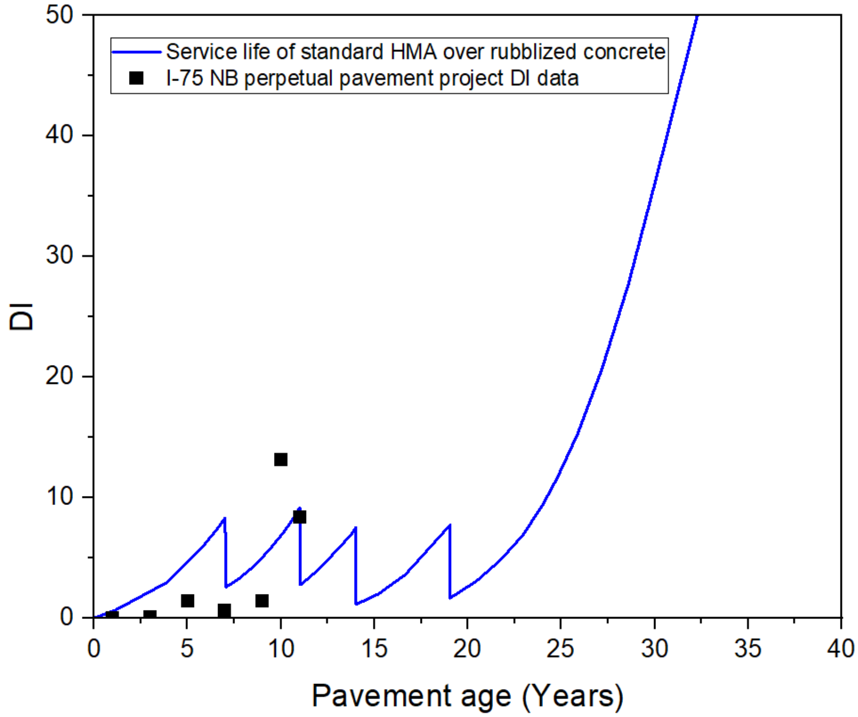


Figure 20. Comparison on I-75 NB DI trend with service life of standard pavement

Cost Comparison and Evaluation

Costs included in this report were adjusted to 2019 dollars for comparison with the standard costs included in the *MDOT Pavement Selection Manual* [2] by using the procedure as denoted in Chapter 6, Section F of that manual. This manual explains the Life Cycle Cost Analysis (LCCA) procedure and MDOT's guidelines for pavement selection. The initial cost for construction was approximated by using MDOT LCCA unit prices and the estimation method for the pavement surface cost as described in Chapter 2, Section A of the *MDOT Pavement Selection Manual*. Note that this method does not consider any base and subbase materials, rubblization, embankment, pre-repair/prep work, or HMA separator layers. This is consistent with the fact that the standard rubblization process was used in the I-75 NB demonstration project, so the only difference between the I-75 NB perpetual project and its standard alternative is the HMA. To facilitate the following comparisons, the perpetual HMA over rubblized demonstration project will be evaluated against the standard HMA over rubblized concrete performance curves and cost data provided in the *MDOT Pavement Selection Manual* [2].

Historical unit prices for HMA mixes from September 2009 will be used to estimate the initial construction cost of the I-75 NB project, since these prices captured construction in 2008 (cost will be inflated to 2019 dollars). The actual project bid prices will not be used since these can be highly variable due to the project quantities and do not provide costs for other mix types needed for comparison. Additionally, it is not clear if binder grade adjustments alone have significant cost impacts, so this will not be included in the cost analysis. However, since mix types and pavement thickness have a significant impact on overall cost, the cost comparison will use these parameters. Accordingly, as shown in Table 24, the perpetual HMA pavement cost is estimated to be approximately $256,300 per lane-mile. In contrast, to estimate the standard alternative, the standard 1993 AASHTO pavement design using a 20-year design life for this project as shown in Appendix C will be used. As a result, this pavement would have been 6.5-inch HMA (as increased from 5.5-inch due to MDOT minimum thickness requirements). Accordingly, as shown in Table 25, this is estimated to be approximately $171,400 per lane-mile. Therefore, approximately $84,900 per lane-mile, or about 1.5 times the initial pavement cost, is added using a perpetual HMA versus the standard HMA over rubblized concrete.

Table 24. Estimated initial cost for the I-75 NB perpetual pavement per unit prices

HMA Layer	Mix Type	Thickness (inch)	Application Rate (lbs/syd)	Total Tons	Unit Price in 2009	Cost per Lane-Mile
Top	5E10	1.5	165	580.8	$63.64	$36,962.11
Leveling	4E10	2.5	275	968	$57.34	$55,505.12
Base	2E10	4.5	495	1,742.4	$65.50	$114,127.20
Total Cost per Lane-Mile (Per 2009 unit prices)						$206,594.43
Total Cost per Lane-Mile (Adjusted to 2019)						**$256,305.90**

Table 25. Estimated initial cost for the I-75 NB theoretical standard pavement per unit prices

HMA Layer	Mix Type	Thickness (inch)	Application Rate (lbs/syd)	Total Tons	Unit Price in 2009	Cost per Lane-Mile
Top	5E10	1.5	165	580.8	$63.64	$36,962.11
Leveling	4E10	2	220	774.4	$57.34	$44,404.10
Base	3E10	3	330	1,161.6	$48.92	$56,825.47
Total Cost per Lane-Mile (Per 2009 unit prices)						$138,191.68
Total Cost per Lane-Mile (Adjusted to 2019)						**$171,443.84**

While the initial paving cost for the I-75 NB perpetual pavement project is higher than the standard HMA over rubblized concrete project, the anticipated service life for the I-75 NB perpetual HMA over rubblized concrete pavement is longer than its standard alternative. Per the *MDOT Pavement Section Manual*, the standard HMA over rubblized concrete pavement is 32 years, while the anticipated service life of the perpetual alternative is at least 50 years. However, as observed in the previous section, this I-75 NB project currently suggests a lower service life of roughly 34 years. Therefore, as shown in Table 26, the perpetual alternative initial cost per year of its service life may range from $5,130 to $7,540 per lane-mile, while the standard alternative is $5,360 per lane-mile. It is important to note that this per year cost does not include the benefit of delayed major rehabilitation or reconstruction, which becomes more significant with longer service life. Therefore, in terms of the initial paving cost, the perpetual alternative is more cost-effective than the standard if at least 50 years of service life is achieved. However, it may not be reasonable to assume this if the service life is less than 50 years.

Table 26. Initial paving cost per year of service life

Type	Initial Pavement Cost	Service life (years)	Yearly average cost
I-75 perpetual HMA over rubblized	$256,300	34	$7,540
		50	$5,130
Standard HMA over rubblized	$171,400	32	$5,360

In addition to the pavement's initial cost, its maintenance is a major contributing factor to the overall cost of a pavement. In comparison, per the *MDOT Pavement Selection Manual*, for the MDOT standard HMA over rubblized concrete pavement, on average, preventive maintenance cycles occur after 7, 11, 14, and 19 years, with rehabilitation or reconstruction estimated to occur after 32 years. Accordingly, the cost per lane-mile of these maintenance fixes is estimated at $25,844, $45,335, $29,389, and $49,158, respectively, so their total cost is $149,726 per lane-mile.

For the I-75 NB perpetual project, two maintenance projects, crack treatment (in 2017) and chip/fog seal (in 2020), were implemented after construction. Considering that the pavement life is 15 years in 2023, the number of maintenance events is 1 less than that of the standard HMA over rubblized concrete projects over that same time period (at ages 7, 11, and 14). The per lane-mile costs of the I-75 NB maintenance fixes in 2017 and 2020 are $6,224 and $51,458 (as adjusted to 2019 cost), respectively, so their total cost is $57,682. This is $42,886 less than the standard version over the same timeframe.

To date, the maintenance cost of the I-75 NB perpetual HMA over rubblized concrete project is lower than standard HMA over rubblized concrete projects. While its initial cost is higher, the potential increased service life would reduce the overall long-term cost of the pavement. Therefore, this demonstration fix type should provide a cost-effective option if the anticipated service life is achieved, but based on current projections, the I-75 NB project does not appear to be achieving this.

Conclusions and Recommendations

This report presents a final evaluation of the "Perpetual Hot Mix Asphalt Pavement Over Rubblized Concrete" pavement demonstration project on I-75 NB in Cheboygan County, MDOT job number 90279. It includes a summary of its design, construction, performance, condition, and costs. Conclusions and recommendations are presented as follows.

The evaluation of the I-75 NB demonstration project indicates both positive aspects and areas of concern. Compared to standard HMA over rubblized concrete, perpetual pavement offers potential advantages in terms of longer service life and lower overall cost due to reduced long-term maintenance needs. This highlights its promise as a sustainable and cost-effective pavement solution.

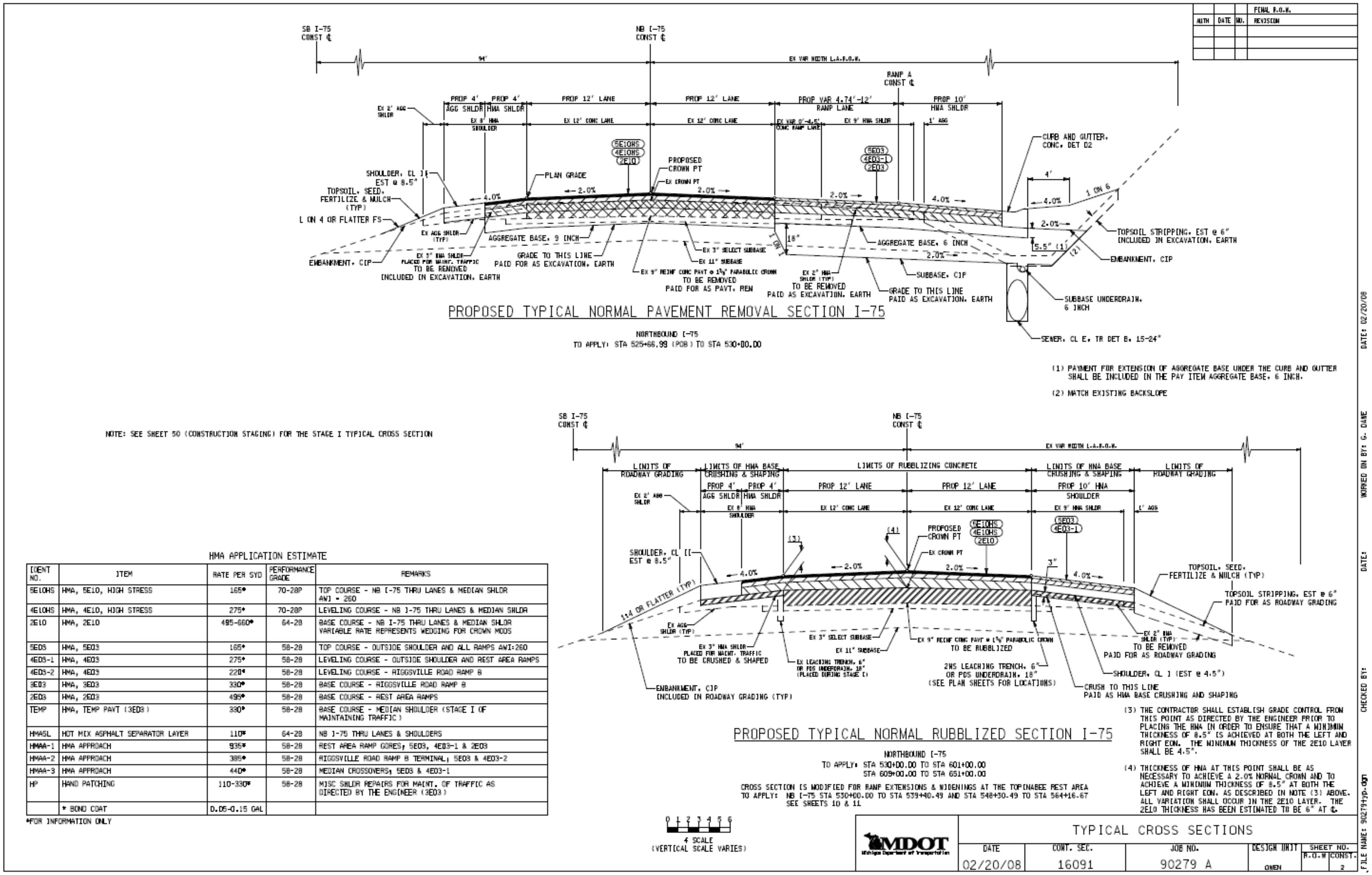
However, the evaluation of this project found some failures due to construction quality and existing base inconsistencies. Issues such as poor existing shoulder base condition, longitudinal joint failure (attributable to insufficient density and/or inadequate bonding), and transverse joint failures (due to paving and construction breakpoints) were observed. While MDOT has since implemented changes to its construction requirements to ensure the density of the longitudinal construction joints, these findings highlight the importance of thorough project scoping to identify and plan for existing pavement cross-section irregularities and address construction issues in future projects. These issues can significantly impact the performance of perpetual HMA rehabilitation projects and raise questions about the cost-effectiveness of this fix type. For future implementations, to ensure the durability and success of perpetual HMA over rubblized concrete, addressing construction quality and base irregularities is crucial for maximizing the benefits of perpetual rehabilitation as a sustainable, cost-effective solution.

Therefore, per the findings of this report, perpetual HMA pavement over rubblized concrete may provide an acceptable, cost-effective construction approach compared with traditional HMA pavement over rubblized concrete. However, due to the questionable forecast of DI, limited dataset from this single project, and the concerns noted above, MDOT should consider constructing additional projects using this demonstration fix type before standardizing. If more projects are to be constructed, then additional construction parameters and pre-construction investigation should be utilized to ensure the design. Consequently, since the primary issue is that of establishing and validating the performance curve and because additional detailed annual reviews would not enhance the conclusions from this project, it is recommended that the MDOT end its annual monitoring and status reporting of the I-75 NB demonstration project. Future data collection needed for fix type evaluation can be solely facilitated by the standard networkwide MDOT condition data measurements and standard MDOT project tracking.

References

1. Von Quintus H. Structural Analysis of the Pavement Design Recommendations for a Perpetual Pavement Along I-75; Report No. 15953-3/2, Michigan. Michigan Department of Transportation Office Memorandum. 2008.
2. Michigan Department of Transportation Pavement Selection Manual. Pavement selection. 2021.
3. Schenkel J. Pavement Demonstration Program Status Report Public Act 457 of 2016 [Internet]. 2022. Available from: https://www.michigan.gov/mdot/about/governmental-affairs

Appendix A: Proposed Pavement Construction Plans



HMA APPLICATION ESTIMATE

IDENT NO.	ITEM	RATE PER SYD	PERFORMANCE GRADE	REMARKS
5E10HS	HMA, 5E10, HIGH STRESS	165*	70-28P	TOP COURSE - NB I-75 THRU LANES & MEDIAN SHLDR AWI = 260
4E10HS	HMA, 4E10, HIGH STRESS	275*	70-28P	LEVELING COURSE - NB I-75 THRU LANES & MEDIAN SHLDR
2E10	HMA, 2E10	495-660*	64-28	BASE COURSE - NB I-75 THRU LANES & MEDIAN SHLDR VARIABLE RATE REPRESENTS WEDGING FOR CROWN MODS
5E03	HMA, 5E03	165*	58-28	TOP COURSE - OUTSIDE SHOULDER AND ALL RAMPS AWI=260
4E03-1	HMA, 4E03	275*	58-28	LEVELING COURSE - OUTSIDE SHOULDER AND REST AREA RAMPS
4E03-2	HMA, 4E03	220*	58-28	LEVELING COURSE - RIGGSVILLE ROAD RAMP B
3E03	HMA, 3E03	330*	58-28	BASE COURSE - RIGGSVILLE ROAD RAMP B
2E03	HMA, 2E03	495*	58-28	BASE COURSE - REST AREA RAMPS
TEMP	HMA, TEMP PAVT (3E03)	330*	58-28	BASE COURSE - MEDIAN SHOULDER (STAGE I OF MAINTAINING TRAFFIC)
HMASL	HOT MIX ASPHALT SEPARATOR LAYER	110*	64-28	NB I-75 THRU LANES & SHOULDERS
HMAA-1	HMA APPROACH	935*	58-28	REST AREA RAMP GORES; 5E03, 4E03-1 & 2E03
HMAA-2	HMA APPROACH	385*	58-28	RIGGSVILLE ROAD RAMP B TERMINAL; 5E03 & 4E03-2
HMAA-3	HMA APPROACH	440*	58-28	MEDIAN CROSSOVERS; 5E03 & 4E03-1
HP	HAND PATCHING	110-330*	58-28	MISC SHLDR REPAIRS FOR MAINT. OF TRAFFIC AS DIRECTED BY THE ENGINEER (3E03)
	* BOND COAT	0.05-0.15 GAL		

*FOR INFORMATION ONLY

Figure 21. JN 90279 Typical Cross-Section for I-75 NB perpetual over rubblized concrete project: (upper one: pavement removal section, Sta 525+66.99 to 530+00; Lower one: rubblized section, Sta 530+00 to 601+00, and Sta 609+00 to 651+00)

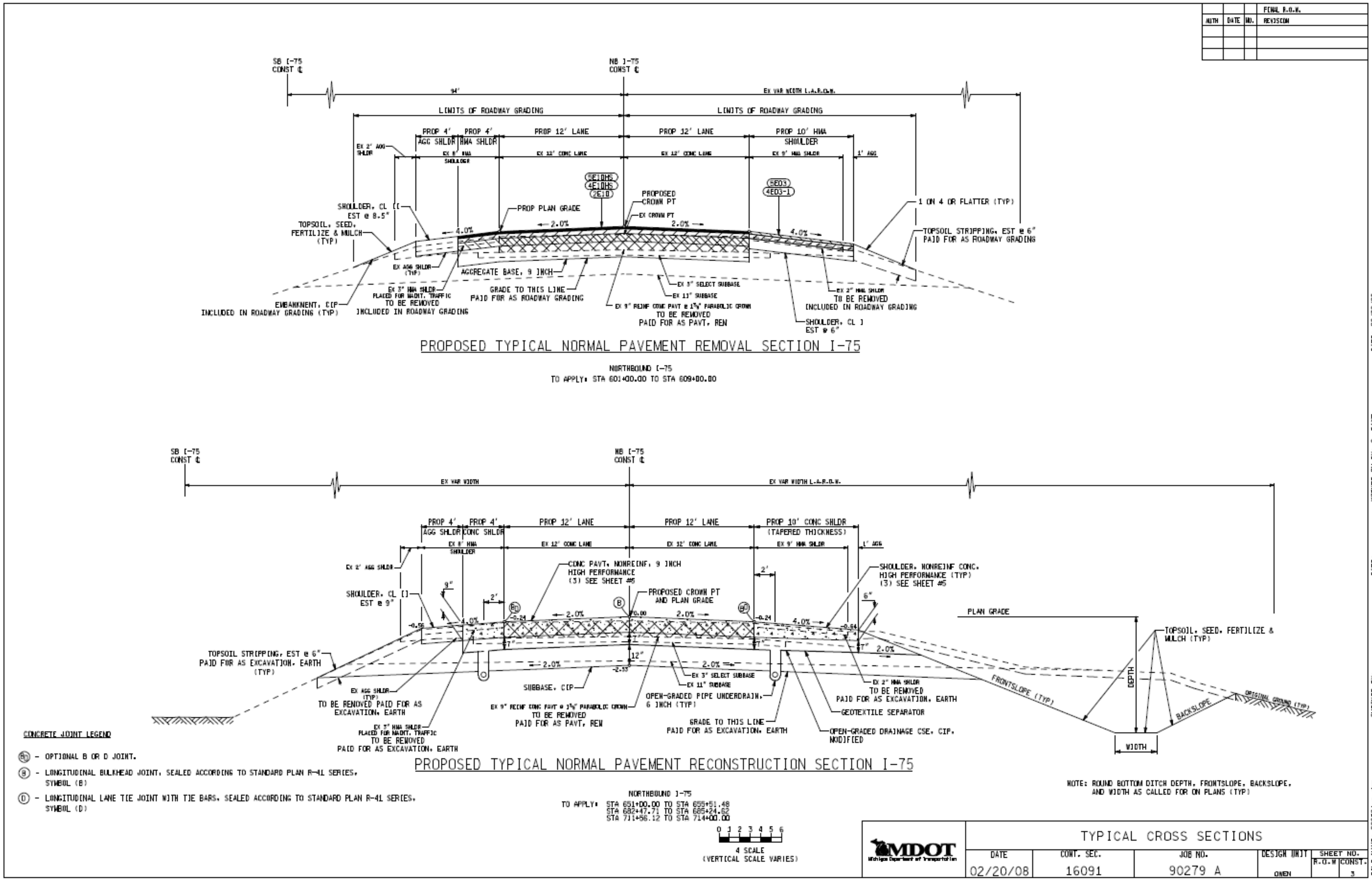


Figure 22. JN 90279 Typical Cross-Section for I-75 NB perpetual over rubblized concrete project: (upper one: pavement removal section, Sta 601+00 to 609+00; lower one: non-related to this demonstration project)

Appendix B: Traffic Data

DATE: July 25, 2006

TO: Hilary Owen
North Region, Grayling TSC

FROM: Ed Waddell
Project Planning Division

SUBJECT: **Traffic Analysis Request (TAR) #1714, Project #75001C**
I-75 from Topinabee Mail Road to Riggsville Road, Cheboygan County

The following tables contain the data requested for this project. If the project will need any additional traffic items, please let me know.

I-75 from Topinabee Mail Road to Riggsville Road, Cheboygan County			
	Base Year	**Construction Year**	**Design Year**
	2006	**2008**	**2028**
ADT	7,100	7,200	8,400
Directional ADT	3,550	3,600	4,200
30th High Hour (DHV)	1,315	1,337	1,555
30th High Hour Directional (DDHV)	743	755	878
% Commercial of ADT	13.5%		
% Commercial of DHV	7.5%		

Equivalent Single Axle Loadings (ESAL's)		
Pavement Type	**Rigid**	**Flexible**
Construction Year Commercial ADT	988	988
Commercial Growth Rate	2%	2%
Lane Distribution	95%	95%
Directional Distribution	50%	50%
Average ESAL	1.19	0.86
Initial Year ESALs	430,000	310,000
Total ESALs 2008-2028	4.9 million	3.5 million

Figure 23. MDOT estimated traffic information for pavement design

ESAL estimate sheet

Date 2/14/2008

- This spreadsheet is intended to allow for a **rough** estimation of ESAL's (for both rigid and flexible pavements).
- Gray boxes require input values - some guidance is given if there are typical values.

Control Section 16091 Job Number 75001
Location I-75 NB from Topinabee to Riggsville
Year of const. 2008

Commercial ADT -	988	Use the CADT for the year of construction. May require using the most recent CADT and multiplying by (1+ g)^n. Where n is the number of years between the CADT year and the construction year and g is the growth rate in decimal form.
Truck Factors -		
HMA	0.86	Typically ranges from 0.6 to 0.9 - use 0.75 in the absence of other information
Concrete	1.19	Typically ranges from 0.9 to 1.25 - use 1.1 in the absence of other information
Analysis Period -	40	Enter the number of years for which you want to estimate the cumulative ESAL's
Directional Dist. -	50	This is the percentage of trucks travelling in each direction. Enter as a %. Almost always 50%, but can be more. If the amount of trucks are not equal between directions, then use the higher value (60% in the case of a 60-40 split).
Lane Distribution -	95	This is the percentage of trucks in the design lane (outside or truck lane). Typical values are:
Truck Growth Rate -	2	Typically ranges from 0% to 3%. A compound growth rate is assumed and used in the calculations.

Typical lane distribution values:

one lane each direction	100%
two lanes each direction	85%
three lanes each direction	70%
four or more lanes each dir.	60%

ESAL's	Flexible	Rigid
	8,898,014	12,312,368

Figure 24. ESAL estimation sheet, page 1

ESAL estimate sheet

Date 2/14/2008

- This spreadsheet is intended to allow for a **rough** estimation of ESAL's (for both rigid and flexible pavements).
- Gray boxes require input values - some guidance is given if there are typical values.

Control Section	16091	Job Number	75001
Location	I-75 NB from Topinabee to Riggsville		
Year of const.	2008		

Commercial ADT -	1000	Use the CADT for the year of construction. May require using the most recent CADT and multiplying by (1+ g)^n. Where n is the number of years between the CADT year and the construction year and g is the growth rate in decimal form.
Truck Factors -		
HMA	0.86	Typically ranges from 0.6 to 0.9 - use 0.75 in the absence of other information
Concrete	1.19	Typically ranges from 0.9 to 1.25 - use 1.1 in the absence of other information
Analysis Period -	40	Enter the number of years for which you want to estimate the cumulative ESAL's
Directional Dist. -	50	This is the percentage of trucks travelling in each direction. Enter as a %. Almost always 50%, but can be more. If the amount of trucks are not equal between directions, then use the higher value (60% in the case of a 60-40 split).
Lane Distribution -	95	This is the percentage of trucks in the design lane (outside or truck lane). Typical values are:

one lane each direction	100%
two lanes each direction	85%
three lanes each direction	70%
four or more lanes each dir.	60%

Truck Growth Rate -	2	Typically ranges from 0% to 3%. A compound growth rate is assumed and used in the calculations.

ESAL's	Flexible	Rigid
	9,006,087	12,461,911

Figure 25. ESAL estimation sheet, page 2

Equivalency factors defined for a terminal serviceabilty index of 2.5 and an SN of 4.0

Load Interval	Total Monthly Single Axles	Single Axle Equivalency Factor	Single Axle ESALs per month
3	1742.217	0.0009	1.567996
4	3664.27	0.003	10.99281
5	2723.225	0.006	16.33935
6	2858.389	0.01	28.58389
7	2407.715	0.02	48.15429
8	3142.231	0.04	125.6892
9	3259.591	0.07	228.1713
10	4890.172	0.1	489.0172
11	6908.509	0.15	1036.276
12	6766.529	0.21	1420.971
13	4137.702	0.29	1199.934
14	2477.83	0.39	966.3538
15	3072.14	0.51	1566.791
16	2093.06	0.65	1360.489
17	2500.03	0.81	2025.024
18	1507.286	1	1507.286
19	1425.293	1.22	1738.857
20	805.5926	1.47	1184.221
21	605.6219	1.76	1065.895
22	201.6308	2.09	421.4083
23	177.7501	2.47	439.0428
24	73.97285	2.89	213.7815
25	15.105	3.37	50.90385
26	22.40443	3.91	87.60133
27	10.38463	4.52	46.93853
28	0	5.21	0
29	0	5.97	0
30	0	6.83	0
31	0	7.79	0
32	0	8.85	0
33	0	10.03	0
34	0	11.34	0
35	0	12.78	0
36	0	14.38	0
37	0	16.14	0
38	0	18.06	0
39	0	20.18	0
40	0	22.5	0
41	0	25.03	0
		Total Single Axle ESALs	17280.29 Average Monthly Total

Figure 26. ESAL estimation using traffic and load distribution, page 1

Load Interval	Total Monthly Tandem Axles	Tandem Axle Equivalency Factor	Tandem Axle ESALs per month
6	403.2718	0.001	0.403272
8	934.9476	0.004	3.739791
10	1145.872	0.01	11.45872
12	1353.611	0.02	27.07223
14	1793.076	0.03	53.79228
16	1830.209	0.06	109.8125
18	1951.604	0.09	175.6443
20	1490.755	0.14	208.7057
22	1598.591	0.21	335.7041
24	1753.673	0.29	508.5653
26	1663.157	0.4	665.2627
28	1873.997	0.53	993.2182
30	2210.687	0.7	1547.481
32	2482.548	0.89	2209.467
34	2364.563	1.11	2624.665
36	1725.629	1.38	2381.368
38	1192.045	1.68	2002.636
40	653.4778	2.03	1326.56
42	227.6897	2.43	553.2859
44	55.99884	2.88	161.2767
46	26.3458	3.4	89.57573
48	6.961577	3.98	27.70708
50	0	4.64	0
52	0	5.39	0
54	0	6.22	0
56	0	7.16	0
58	0	8.22	0
60	0	9.4	0
62	0	10.94	0
64	0	12.17	0
66	0	13.8	0
68	0	15.6	0
70	0	17.59	0
72	0	19.78	0
74	0	22.2	0
76	0	24.85	0
78	0	27.76	0
80	0	30.95	0
82	0	34.43	0
		Total Tandem Axle ESALs	16017.4 Average Monthly Total

Figure 27. ESAL estimation using traffic and load distribution, page 2

Load Interval	Total Monthly Tridem Axles	Tridem Axle Equivalency Factor	Tridem Axle ESALs per month
12	380.6205	0.004	1.522482
15	387.706	0.01	3.87706
18	361.3995	0.02	7.227991
21	359.0083	0.04	14.36033
24	201.1068	0.07	14.07748
27	98.1208	0.11	10.79329
30	145.4253	0.17	24.7223
33	92.16407	0.25	23.04102
36	158.6342	0.35	55.52195
39	128.8604	0.48	61.85299
42	153.9061	0.64	98.49992
45	126.1392	0.84	105.9569
48	115.6454	1.07	123.7405
51	82.30106	1.34	110.2834
54	74.64792	1.66	123.9155
57	40.98297	2.02	82.7856
60	26.19683	2.44	63.92027
63	7.099706	2.92	20.73114
66	0.066042	3.47	0.229164
69	0.066042	4.09	0.27011
72	6.769258	4.8	32.49244
75	0	5.59	0
78	0	6.49	0
81	0	7.5	0
84	0	8.63	0
87	0	9.9	0
90	0	11.32	0
93	0	12.91	0
96	0	14.67	0
99	0	16.63	0
102	0	18.8	0

Total Tridem Axle ESALs	979.8219	Average Monthly Total
Cumulative Monthly ESALs	34277.51	Average Monthly Total - All Axles
Cumulative Annual ESALs	411330.2	Average Annual Total - All Axles
ESALs per Truck **Truck Equivalency Factor**	1.140619	Average Annual Value

Figure 28. ESAL estimation using traffic and load distribution, page 3

Appendix C: Pavement Design Data

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product
Michigan Department of Transportation
8885 Ricks Rd.
Lansing, MI
USA

Flexible Structural Design Module

CS 16091/16092, JN 75001 I-75 from Topinabee Rd. to Riggsville Rd.
Rubblize and HMA Resurface

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	3,500,000
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	95 %
Overall Standard Deviation	0.49
Roadbed Soil Resilient Modulus	3,800 psi
Stage Construction	1
Calculated Design Structural Number	5.48 in

Layered Thickness Design

Thickness precision Actual

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Spec Thickness (Di)(in)	Min Thickness (Di)(in)	Elastic Modulus (psi)	Width (ft)	Calculated Thickness (in)	Calculated SN (in)
1	5E10 Top Course	0.42	1	1.5	-	390,000	-	1.50	0.63
2	4E10 Leveling Course	0.42	1	2	-	390,000	-	2.00	0.84
3	3E10 Base Course	0.36	1	2	-	275,000	-	2.00	0.72
4	Rubblized Concrete	0.18	1	-	-	50,000	-	8.92	1.61
5	Agg. base/sand subbase	0.12	1	14	-	20,000	-	14.00	1.68
Total	-	-	-	-	-	-	-	28.42	5.48

1.5"
2.0"
2.0"
5.5"

Figure 29. 1993 AASHTO pavement design result, 20 years

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product
Michigan Department of Transportation
8885 Ricks Rd.
Lansing, MI
USA

Flexible Structural Design Module

CS 16091/16092, JN 75001 I-75 from Topinabee Rd. to Riggsville Rd.
40 year Perp Pavement over Rubblize

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	9,000,000
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	95 %
Overall Standard Deviation	0.49
Roadbed Soil Resilient Modulus	3,800 psi
Stage Construction	1
Calculated Design Structural Number	6.20 in

Layered Thickness Design

Thickness precision Actual

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Spec Thickness (Di)(in)	Min Thickness (Di)(in)	Elastic Modulus (psi)	Width (ft)	Calculated Thickness (in)	Calculated SN (in)
1	5E10 Top Course	0.42	1	1.5	-	390,000	-	1.50	0.63
2	4E10 Leveling Course	0.42	1	2.2	-	390,000	-	2.20	0.92
3	3E10 Base Course	0.36	1	3.75	-	275,000	-	3.75	1.35
4	Rubblized Concrete	0.18	1	-	-	50,000	-	8.95	1.61
5	Agg. base/sand subbase	0.12	1	14	-	20,000	-	14.00	1.68
Total	-	-	-	-	-	-	-	30.40	6.20

1.5"
2.25"
3.75"
7.5"

Figure 30. 1993 AASHTO pavement design result, 40 years

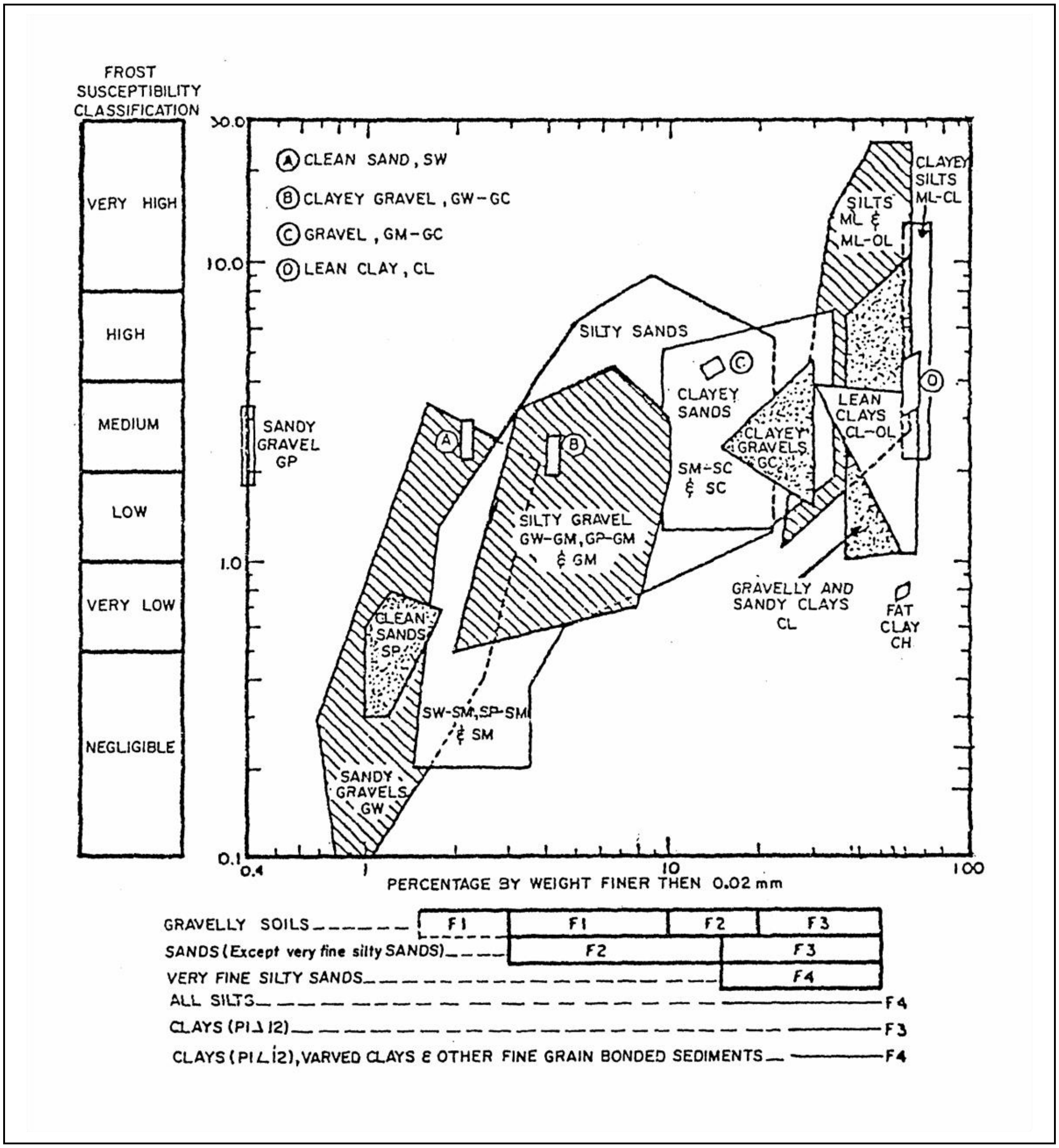


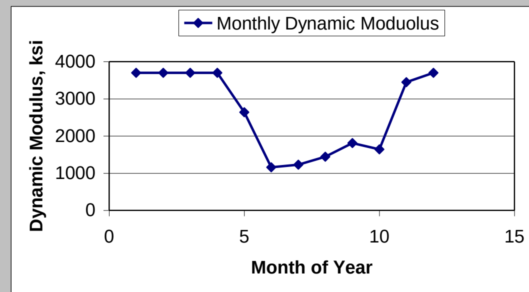
Figure 31. Average rate of heave versus percentage finer than 0.02 mm for natural soil gradations

DETERMINATION OF HMA EQUIVALENT ANNUAL MODULUS FOR FATIGUE CRACKING ANALYSIS

Layer: **SMA Wearing Surface; 12.5mm, PG70-28**

Thickness: 2 inches

Month	Dynamic Modulus, ksi	Damage Factor	E X DF	EQUIVALENT ANNUAL MODULUS, ksi
Jan	3700	1.16E+04	4.30E+07	
Feb	3700	1.16E+04	4.30E+07	
Mar	3700	1.16E+04	4.30E+07	
April	3700	1.16E+04	4.30E+07	
May	2640	2.21E+04	5.85E+07	
June	1159	1.07E+05	1.23E+08	
July	1229	9.52E+04	1.17E+08	
Aug	1443	7.01E+04	1.01E+08	
Sept	1811	4.54E+04	8.23E+07	
Oct	1640	5.49E+04	9.01E+07	
Nov	3447	1.33E+04	4.59E+07	
Dec	3699	1.16E+04	4.30E+07	
Totals		4.66E+05	8.33E+08	**1.79E+03**



$$DamageFactor = 7.4754x10^{10}\left[E(T_i)\right]^{-1.908}$$

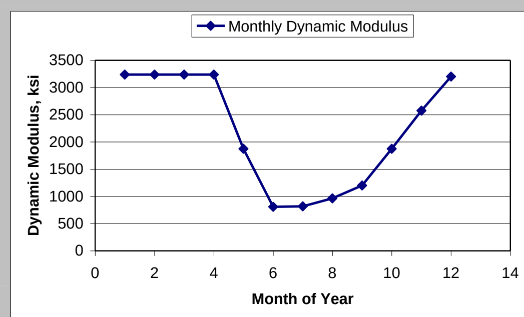
$$E_{equivalent} = \frac{\sum[E(T_i)](DF_i)}{\sum(DF_i)}$$

DETERMINATION OF HMA EQUIVALENT ANNUAL MODULUS FOR FATIGUE CRACKING ANALYSIS

Layer: **Dense-Graded HMA Binder Layer; 12.5 mm, PG64-28**

Thickness: 2.5 inches

Month	Dynamic Modulus, ksi	Damage Factor	E x DF	Equivalent Annual Modulus, ksi
Jan	3237	1.50E+04	4.86E+07	
Feb	3237	1.50E+04	4.86E+07	
Mar	3237	1.50E+04	4.86E+07	
April	3237	1.50E+04	4.86E+07	
May	1874	4.26E+04	7.98E+07	
June	808	2.12E+05	1.71E+08	
July	817	2.08E+05	1.70E+08	
Aug	964	1.51E+05	1.46E+08	
Sept	1200	9.97E+04	1.20E+08	
Oct	1874	4.26E+04	7.98E+07	
Nov	2577	2.32E+04	5.98E+07	
Dec	3202	1.53E+04	4.91E+07	
Totals		8.54E+05	1.07E+09	**1.25E+03**

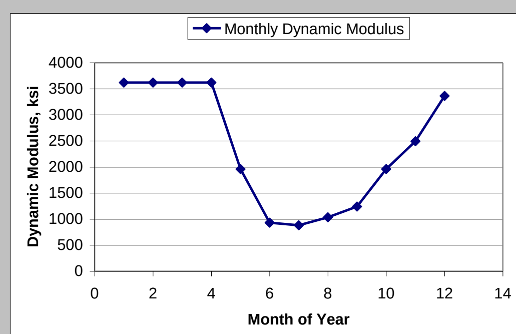


DETERMINATION OF HMA EQUIVALENT ANNUAL MODULUS FOR FATIGUE CRACKING ANALYSIS

Layer: **Dense-Graded HMA Base; 25 mm, PG70-28**

Thickness: 4 inches

Month	Dynamic Modulus, ksi	Damage Factor	E x DF	Equivalent Annual Modulus, ksi
Jan	3621	1.21E+04	4.39E+07	
Feb	3621	1.21E+04	4.39E+07	
Mar	3621	1.21E+04	4.39E+07	
April	3621	1.21E+04	4.39E+07	
May	1960	3.91E+04	7.66E+07	
June	930	1.62E+05	1.51E+08	
July	882	1.79E+05	1.58E+08	
Aug	1035	1.32E+05	1.37E+08	
Sept	1240	9.36E+04	1.16E+08	
Oct	1960	3.91E+04	7.66E+07	
Nov	2493	2.47E+04	6.16E+07	
Dec	3364	1.39E+04	4.69E+07	
Totals		7.33E+05	9.99E+08	**1.36E+03**



E-Combined;
Equivalent Annual HMA Modulus for Pavement **1411.5**

$$E_{Combined} = \left[\frac{h_i(E_i)^{0.333} + h_j(E_j)^{0.333} + h_k(E_k)^{0.333}}{h_i + h_j + h_k}\right]^3$$

Figure 32. Example of the spreadsheet used to determine the equivalent annual modulus values for the HMA layers

Figure 33. Photo of rubblized PCC for which the elastic modulus back-calculated from deflection basins is low, ranging from 25,000 to 50,000 psi

Appendix D: Construction Data and Notable Inspector's Daily Reports

MDOT
Michigan Department of Transportation

Inspector's Daily Report

9/14/2008 1:02 PM
FieldManager 4.3a

Contract: 16091-90279, Rubblizing and hot mix asphalt resurfacing

IDR Date	Day of Week	Sequence No.	Import Date	Project / Resident Engineer
9/11/2008	Thursday	1	N/A	Bill Wahl, Grayling TSC

Inspector's Initials-Name	Federal Project Number	Elec. Attachments
tt thomas taylor	IM 0816(014)	None

Prime Contractor
Six-S Inc/C&G Excavating, Inc.

Entered By	Revised By	Revision Date	Revision No.
tt, thomas taylor			

Temperatures	Weather
Low: 43 ° F High: 74 ° F	Pt. Cloudy

Comments

C&G, Exc. cont. placing more of the 4G, stone on grade sta. 651+00 thru sta. 714+00 on the rt, also working on the re-con section on ramp B, Riggsville road,
Reith-Riley working on prepping rubblize section sta. 530+00 thru sta. 651+00 on the rt, Antigo cont. with the Rubblizing.
NOTE: traffic control set in place per sections 103.05, 103.06, 812 of the standard specs of 2003.

Contractors

Contractor's Name	Personnel	No.	Hrs.	Equipment	No.	Hrs.
Antigo Construction, Inc.	Laborer	1	12.00	work truck	1	12.00
	Mike, foreman	1	12.00			
Rieth-Riley Construction Co., Inc.	Blaine, foreman	1	14.00	backhoe	1	14.00
	laborers	5	14.00	bobcat	1	14.00
				grader	1	14.00
				loader	1	14.00
				steel drum rollers	1	14.00
				truck	2	14.00
				water truck	1	14.00
				work trucks	2	14.00
Six-S Inc/C&G Excavating, Inc.	Frank, foreman	1	14.00	broom truck	1	14.00
	laborers	5	14.00	dozer	3	14.00
	operators	6	14.00	excavator	2	14.00
				grader	1	14.00
				loader	2	14.00
				steel roller	1	14.00
				trucks	8	14.00
				water truck	1	14.00
				work truck	2	14.00

Contract: 16091-90279 **IDR:** 9/11/2008, tt, 1 Page 1 of 2

Figure 34. Inspector's daily report on 09-11-2008, page 1

MDOT
Michigan Department of Transportation

Inspector's Daily Report

9/14/2008 1:02 PM
FieldManager 4.3a

Item Postings

Item/Material Description	Item Code	Prop. Line	Project	Category	Quantity Unit	Location	Brkdwn ID	Attn
_ Filler Aggregate Contractor: Rieth-Riley Construction Co., Inc. Item Remarks: load tickets on file.	3067031	0410	90279A	0001	15.140 Ton	sta. 530+00 thru sta. 651+00 on the rt.	019	
_ Rubblized Pavt Contractor: Rieth-Riley Construction Co., Inc. Item Remarks: see field book.	3047011	0380	90279A	0001	18,934.000 Syd	sta. 530+00 thru sta. 651+00 on the rt.		
Geotextile Separator Contractor: Commerce Construction & Landscaping, Inc.	3030020	0360	90279A	0001	20,265.000 Syd	see field note attached to this IDR.		
Geotextile, Separator					20,265.00 Syd			
HMA Base Crushing and Shaping Contractor: Lois Kay Contracting Co.	3050002	0390	90279A	0001	12,069.000 Syd	see field book on locations and comps.		
Open-Graded Dr Cse, CIP, Modified Contractor: Six-S Inc/C&G Excavating, Inc.	3030006	0350	90279A	0001	4,903.000 Cyd	see field note attached to this IDR.		

Reviewed By: ______________________ (Signature) ______________ (Date)

Contract: 16091-90279 IDR: 9/11/2008, tt, 1 Page 2 of 2

Figure 35. Inspector's daily report on 09-11-2008, page 2

MDOT
Michigan Department of Transportation

Inspector's Daily Report

10/27/2008 7:59 AM
FieldManager 4.3a

Contract: 16091-90279, Rubblizing and hot mix asphalt resurfacing

IDR Date	Day of Week	Sequence No.	Import Date	Project / Resident Engineer
10/24/2008	Friday	1	N/A	Bill Wahl, Grayling TSC

Inspector's Initials-Name	Federal Project Number	Elec. Attachments
tt thomas taylor	IM 0816(014)	None

Prime Contractor
Six-S Inc/C&G Excavating, Inc.

Entered By	Revised By	Revision Date	Revision No.
tt, thomas taylor			

Temperatures	Weather
Low: 40 ° F High: 60 ° F	Clear to Cloudy

Comments

Reith-Riley cont. placing HMA 5E10 HS, see paving core sheets. NOTE: HMA paving is complete for this project.
Riet-Way Fence here to remove more guard rail @ Mullet-Burt bridge area, median side.
C&G, Exc. cont. working on placing more shoulder gravel on the median side.

Contractors

Contractor's Name	Personnel	No.	Hrs.	Equipment	No.	Hrs.
Marx Contracting, Inc.	Laborers	3	8.00	Post driver	1	8.00
				work trucks	1	8.00
Rieth-Riley Construction Co., Inc.	Jeff, foreman	1	12.00	bobcat	1	12.00
	laborers	5	12.00	flo-boys	8	12.00
	operators	8	12.00	paver	2	12.00
				steel drum rollers	4	12.00
				tac-truck	1	12.00
				transfer machine	1	12.00
				water truck	1	12.00
				work trucks	3	12.00
Six-S Inc/C&G Excavating, Inc.	Frank, foreman	1	10.00	backhoe	1	10.00
	laborers	2	10.00	broom truck	1	10.00
	operators	1	10.00	dozer	1	10.00
				excavator	1	10.00
				loader	1	10.00
				trucks	1	10.00
				work truck	1	10.00

Item Postings

Item/Material Description	Item Code	Prop. Line	Project	Category	Quantity	Unit	Location	Brkdwn ID	Attn
Guardrail, Rem Contractor: Six-S Inc/C&G Excavating, Inc.	2040008	0150	90279A	0001	400.000	Ft	Mullet-Burt Bridge bullnose in the median.		

Contract: 16091-90279 IDR: 10/24/2008, tt, 1 Page 1 of 2

Figure 36. Inspector's daily report on 10-24-2008, page 1

MDOT

Michigan Department of Transportation

Inspector's Daily Report

10/27/2008 7:59 AM

FieldManager 4.3a

Item Postings

Item/Material Description	Item Code	Prop. Line	Project	Category	Quantity	Unit	Location	Brkdwn ID	Attn
HMA, 5E10, High Stress Contractor: Rieth-Riley Construction Co., Inc.	5020516	0870	90279A	0001	825.350	Ton	sta. 595+75 thru sta. 651+00 on the rt. (see paving core sheets).		

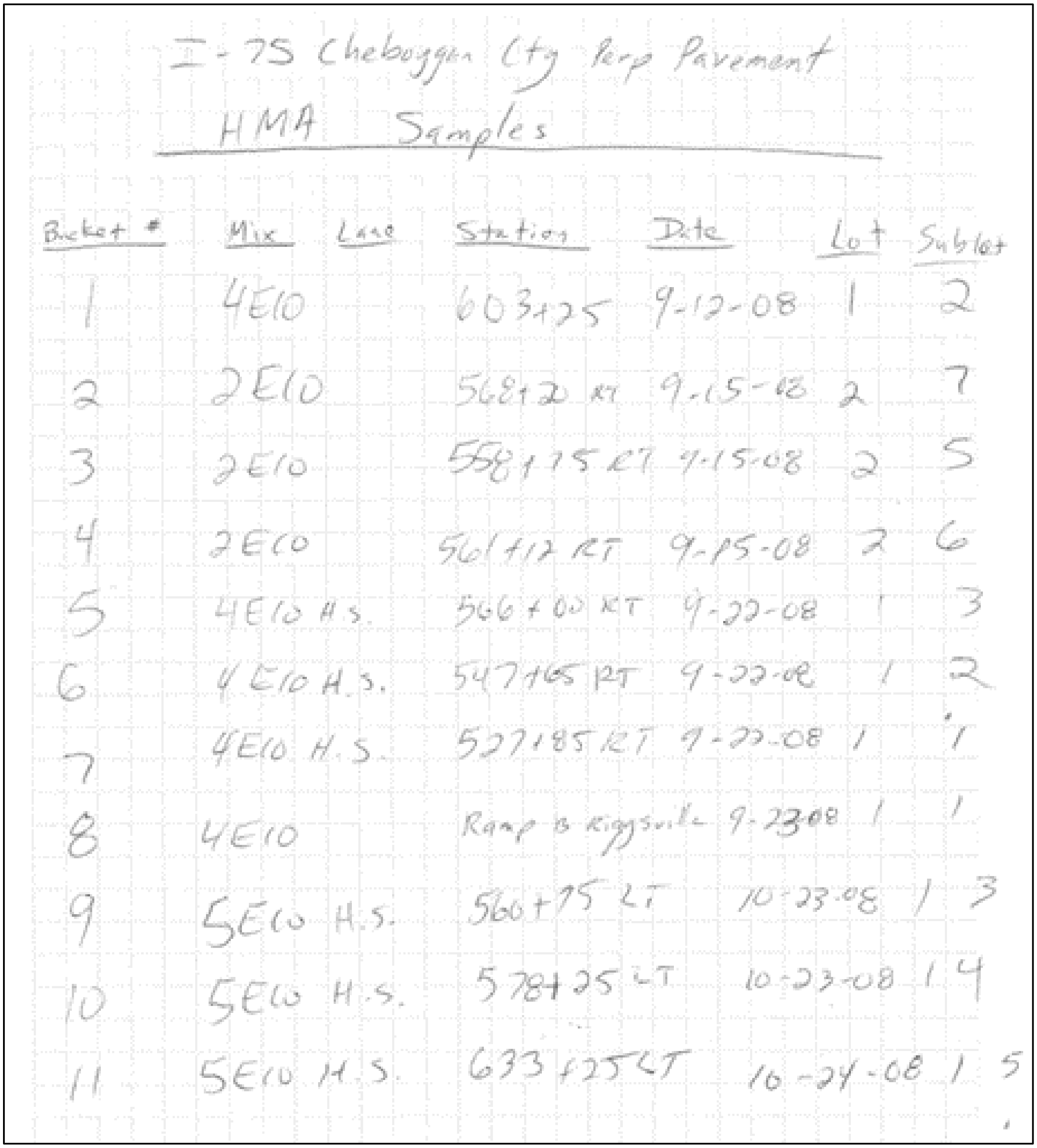
Item Remarks: Load tickets on file. Covered 10000 syds for a YIELD of 165.1 #/syd.

HMA, 5E10, High Stress 825.35 Ton

Reviewed By: ______________________________ (Signature) ______________________ (Date)

Contract: 16091-90279 **IDR:** 10/24/2008, tt, 1 Page 2 of 2

Figure 37. Inspector's daily report on 10-24-2008, page 2


I-75 Cheboygan Cty Perp Pavement

HMA Samples

Bucket #	Mix	Lane	Station	Date	Lot	Sublot
1	4E10		603+25	9-12-08	1	2
2	2E10		568+20 RT	9-15-08	2	7
3	2E10		558+75 RT	9-15-08	2	5
4	2E10		561+12 RT	9-15-08	2	6
5	4E10 H.S.		566+00 RT	9-22-08	1	3
6	4E10 H.S.		547+65 RT	9-22-08	1	2
7	4E10 H.S.		527+85 RT	9-22-08	1	1
8	4E10		Ramp B Riggsville	9-23-08	1	1
9	5E10 H.S.		560+75 LT	10-23-08	1	3
10	5E10 H.S.		578+25 LT	10-23-08	1	4
11	5E10 H.S.		633+25 LT	10-24-08	1	5

Figure 38. HMA sample location recording

Appendix E: Field Evaluation Reports

Field Evaluation Report

Michigan Department of Transportation
Construction & Technology Division
Pavement Structures Group

Sheet 1

Of 1

Research Proj.:	**Date:** 9/11/08	**Weather:** Sunny, 70's	
Proj. Manager:	**Control Sec./Job No.:** 16091/90279		**Attendance:** M. Eacker
Item(s) Surveyed: Rubblization			
Location: I-75 NB from Topinabee Rd. north for 2 miles			
Contractor(s): Six-S (prime), Antigo (sub)			
Objective: Review the rubblization			

Observations:

- Arrived on site approximately 10:20 am
- Steve Purdy is on-site with the FWD.
- Antigo has one multi-head breaker (MHB). They are using a z-grid roller followed by a steel drum roller.
- The MHB is rubblizing 8 feet wide so a second pass is needed to get remainder of the lane plus at least 18 inches beyond centerline. The outside lane is being worked while traffic is maintained on the inside lane.
- Contractor made a dig with a backhoe at station 621+20 at EOM. The affected area was about three to four feet in each direction. A few larger pieces stuck to the mat, including one about 14 inches in length, but most of it was clean.
- I did a hand dig next to the contractor's hole. It was about 5 feet long starting at centerline and about 3.5' to 4' wide. A few high pieces of concrete were easily chipped away to reveal steel. Quite a bit of the mesh was visible. Area about a foot from centerline (hard to tell where centerline exactly is) is not debonded at all.
- I took a half bag sample of the rubblize material (pre-roller) from about station 621+15.
- MHB operator says the full-depth repairs are rubblizing just as well as the old concrete. He is seeing "moon shaped" cracks in front of his outside hammer (the one at EOM) due to lack of support at the shoulder (which is just gravel at this point). He says that sometimes subsequent blows on that piece will only drive it down instead of breaking it further.
- Another dig 4.5' by 2.5' at station 631+60. Hole was from centerline towards outside shoulder. What little steel was showing was painted and two pictures taken.
- Just for curiosity, I did a little bit of digging at a full-depth repair. I did not find any mesh, but I did find one of the dowel bars.
- 3rd hole at station 625+83, 6' by 3'. Hole started 1.5' from EOM. Steel painted orange with 3 pictures taken. Pretty good job of debonding.
- Got another half bag of rubblized material (pre-roller)at about station 626+30.
- Left the project approximately 2:30 pm.

Conclusions:

Future Work: Continue to visit the project

Notes taken by: M. Eacker

Figure 39. Field evaluation report in 2008

Field Evaluation Report

Michigan Department of Transportation
Construction Field Services Division
Pavement Management Section

Sheet 1 of 2

Research Proj.:	**Date:** 5/10/22	**Weather:** 63°F, sunny
Proj. Manager:	**Control Sec./Job No.:**	**Attendance:** J. Schenkel, F. Kaseer
Item(s) Surveyed: Perpetual HMA Pavement Demonstration Project		
Location: I-75 NB Topinabee Mail Rd north for 2.37 mi, Cheboygan County		
Contractor(s):		
Objective: Yearly visual review		

Observations:

NOTE: JN 204267 chip and fog seal project occurred in August 2020.

Northbound (counts per mainline lanes only):

- Cracking summary: 228′ total transverse (228′ unsealed); 0′ total longitudinal
    - Left Lane:
        - Transverse = 120′ total; 120′ unsealed (11 locations with 9 being full width)
        - Longitudinal = 0′ total
    - Right Lane:
        - Transverse = 108′ total; 108′ unsealed (9 locations with 9 being full width)
        - Longitudinal = 0′ total
- Despite the maintenance project, potholes and delamination of surface course at the beginning and ending transitions (mostly in the right lane) have improved but are still observable.
- Due to the maintenance project, transverse tears (1′ to 4′) in the on and off rest area ramps and shoulders are no longer visible.
- The right shoulder transverse cracks and longitudinal cracking are starting to become visible again, (with the transverse cracking being more visible).
- Almost all of the observable transverse cracks are very straight and full width. These may be the end of paving sequences and/or insufficiently rubblized locations that are reflecting through the surface of the chip seal.
- The longitudinal joints at the left shoulder and centerline are very tight. Prior to the maintenance project, the longitudinal joint between the rightmost lane and shoulder was noted as being somewhat wide in some locations, up to 2″. However, this was improved by the maintenance project, so now only very few locations have separation. Still, where present, separation appears to be up to 2″.
- Overall, this location looks good and continues to perform well. The chip and fog seal still looks good. It has sealed most of the cracks and is limiting water infiltration.

Conclusions:

Cracking increased from 9 to 11 transverse locations in the left lane but stayed the same in the right lane (9 locations). However, this is still minimal and the transverse cracking per mile is very low at ~48 ft/lane-mile. The longitudinal joint between the rightmost lane and shoulder has only a few locations showing separation (up to 2″). The past noted potholes and delamination of the surface at the north and south end transitions of this project remain but was improved due to the maintenance project.

Figure 40. Field evaluation report in 2022, page 1

Field Evaluation Report

Michigan Department of Transportation
Construction Field Services Division
Pavement Management Section

Sheet 2 of 2

Overall, the pavement is performing well.

Future Work:
Per the February 2022 Pavement Demonstration Program Project Evaluation technical report, it is recommended that monitoring of this demonstration project end with its final report because it has reached a reasonable age with enough condition data points for project close out.

In the interim, monitoring of this project will continue until this final report is officially approved by MDOT.

Notes taken by: *Justin Schenkel*

Figure 41. Field evaluation report in 2022, page 2

Appendix F: Field Evaluation Figures

Figure 42. Field evaluation on 12-21-2010

Figure 43. Field evaluation on 12-10-2011

Figure 44. Field evaluation on 11-16-2012

Figure 45. Field evaluation on 12-23-2013

Figure 46. Field evaluation on 12-22-2014

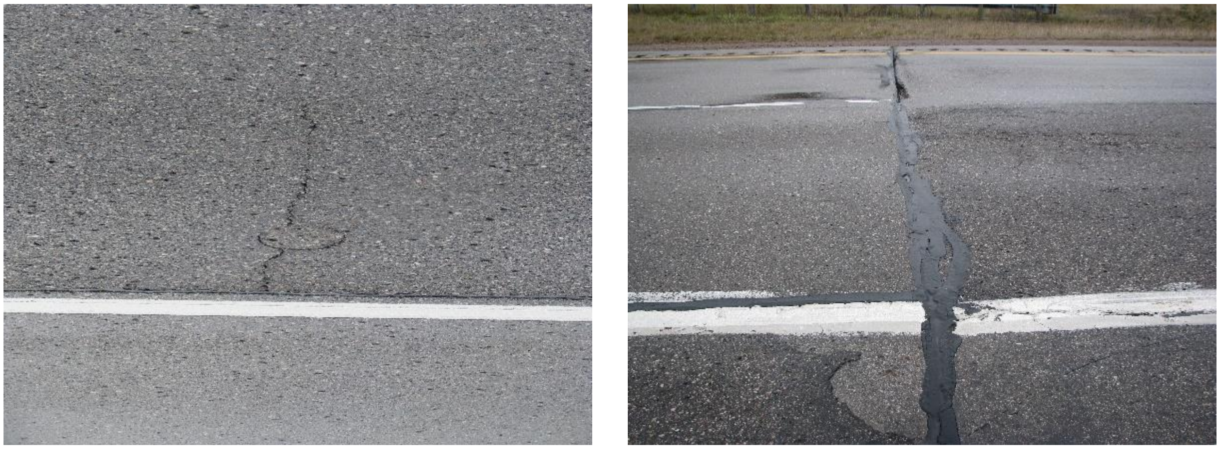


Figure 47. Field evaluation on 12-02-2015

Figure 48. Field evaluation on 11-30-2016

Figure 49. Field evaluation on 05-02-2018

Figure 50. Field evaluation on 04-03-2019

Figure 51. Field evaluation on 04-16-2020

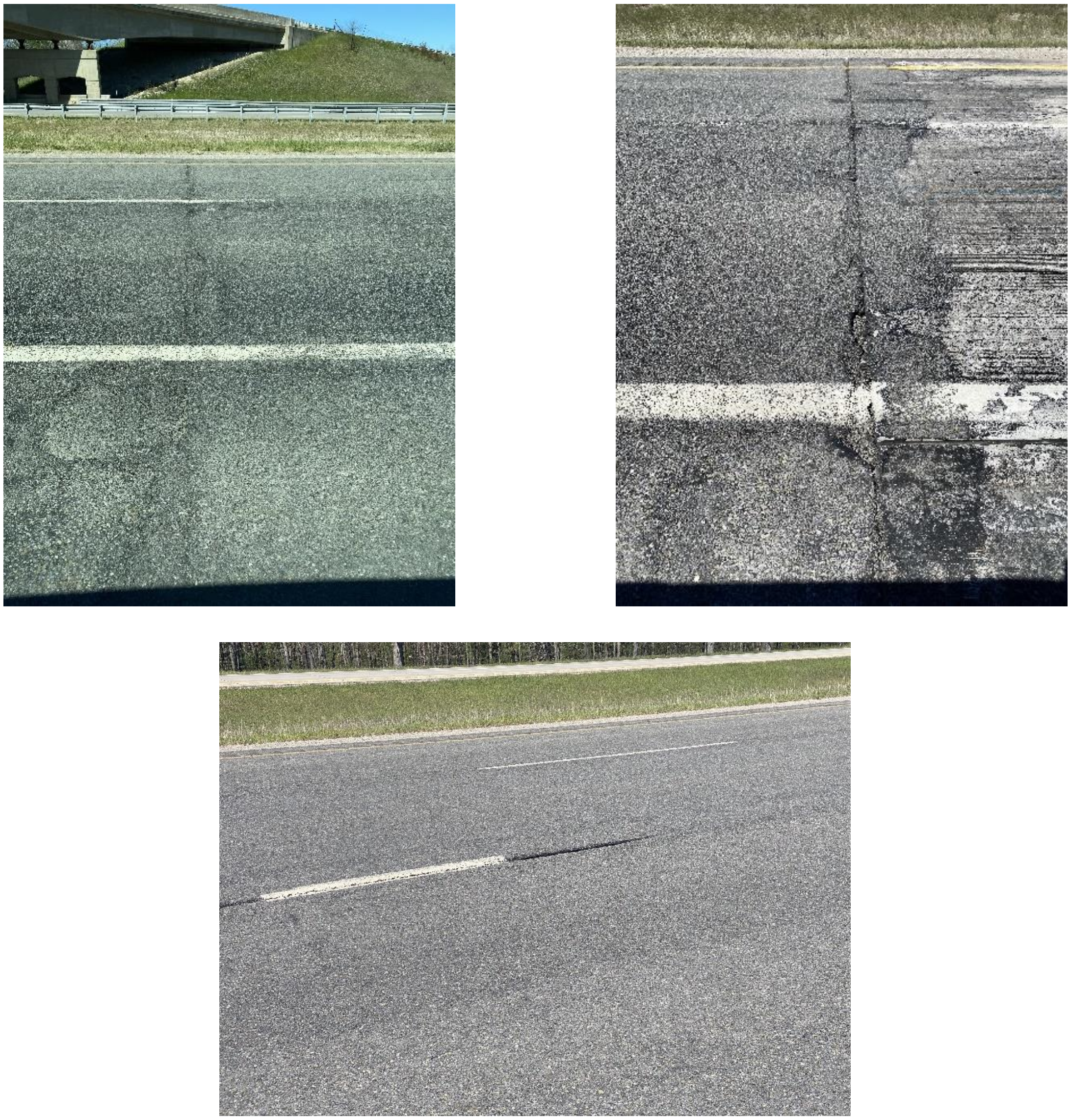


Figure 52. Field evaluation on 05-12-2021

Figure 53. Field evaluation on 05-10-2022