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# Evaluation of the Terminal Boulevard (SR 406) Concrete Rubblization Project

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16. Abstract:


In 2016, the Virginia Department of Transportation (VDOT) executed a contract to reconstruct portions of Terminal Boulevard (SR 406) in Norfolk. The work consisted of rubblizing the existing continuously reinforced concrete pavement and placing 6 in of asphalt mixture on top of the compacted rubblized concrete material. The project limits were from the I-564 loop ramp to the intersection with Hampton Boulevard in the westbound direction and from the intersection with Hampton Boulevard to the bridge over the railroad tracks in the eastbound direction.

The purpose of this study was to document the current condition of the rubblized and reconstructed pavement on Terminal Boulevard (SR 406) and to generate baseline pavement performance information. Laboratory tests indicated that the asphalt mixtures used were expected to be rut resistant and resistant to non-load related cracking. Field tests using a falling weight deflectometer showed that the pavement section is structurally strong with a low deflection. AASHTOWare Pavement ME software showed a design life of 18 to 19 years for the pavement in terms of bottom-up fatigue cracking. The analysis also predicted higher rutting than expected. VDOT pavement management data showed an increase in roughness and rutting between 2017 and 2018, but the difference could be attributable to random variation from year-to-year testing.

The study recommends that the Virginia Transportation Research Council continue to monitor the performance of the rubblized pavement on SR 406 and report on its condition in approximately 3 and 5 years. Any lessons learned during this time should be submitted to VDOT's Hampton Roads District and VDOT's Materials Division for review.


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**FINAL REPORT**

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## ABSTRACT

In 2016, the Virginia Department of Transportation (VDOT) executed a contract to reconstruct portions of Terminal Boulevard (SR 406) in Norfolk. The work consisted of rubblizing the existing continuously reinforced concrete pavement and placing 6 in of asphalt mixture on top of the compacted rubblized concrete material. The project limits were from the I-564 loop ramp to the intersection with Hampton Boulevard in the westbound direction and from the intersection with Hampton Boulevard to the bridge over the railroad tracks in the eastbound direction.

The purpose of this study was to document the current condition of the rubblized and reconstructed pavement on Terminal Boulevard (SR 406) and to generate baseline pavement performance information. Laboratory tests indicated that the asphalt mixtures used were expected to be rut resistant and resistant to non-load related cracking. Field tests using a falling weight deflectometer showed that the pavement section is structurally strong with a low deflection. AASHTOWare Pavement ME software showed a design life of 18 to 19 years for the pavement in terms of bottom-up fatigue cracking. The analysis also predicted higher rutting than expected. VDOT pavement management data showed an increase in roughness and rutting between 2017 and 2018, but the difference could be attributable to random variation from year-to-year testing.

The study recommends that the Virginia Transportation Research Council continue to monitor the performance of the rubblized pavement on SR 406 and report on its condition in approximately 3 and 5 years. Any lessons learned during this time should be submitted to VDOT's Hampton Roads District and VDOT's Materials Division for review.

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## INTRODUCTION

In 2016, the Virginia Department of Transportation (VDOT) executed a contract to reconstruct portions of Terminal Boulevard (SR 406) in Norfolk. The work consisted of rubblizing the existing continuously reinforced concrete pavement (CRCP) and placing 6 in of asphalt mixture on top of the compacted rubblized concrete material. The project limits were from the I-564 loop ramp to the intersection with Hampton Boulevard in the westbound direction and from the intersection with Hampton Boulevard to the bridge over the railroad tracks in the eastbound direction.

The total project length was about 1.8 miles, and the project limits are shown in Figure 1. A portion of the project was maintained by the City of Norfolk. The city and VDOT portions were rubblized at the same time. The VDOT section was from I-564 to Ruthven Road, and the City of Norfolk portion was from Ruthven Road to Hampton Boulevard. The annual average daily traffic for this section was about 17,000 vehicles, with 12% trucks.

Terminal Boulevard (SR 406) is a four-lane divided roadway that connects I-564 with Hampton Boulevard to the east and with Norfolk International Terminals to the west. The existing pavement structure was 8 in (nominally) of CRCP on a cement-stabilized subgrade, which was constructed circa 1967. The existing CRCP was judged to be in poor condition (Figures 2 and 3) and in need of replacement based on observed asphalt patches, open/spalled transverse cracks, open/spalled longitudinal cracks, and other distresses. A 2015 pavement evaluation found 8 to 12 in of CRCP underlain by 6 to 28 in of stabilized sub-base (Applied Research Associates, unpublished data, 2015).

In 2016, the existing CRCP was rubblized and covered with 6 in of asphalt mixture (4 in of an intermediate 19 mm nominal maximum aggregate size [NMAS] mixture, VDOT designation IM 19.0E, and 2 in of a surface 12.5 mm NMAS mixture, VDOT designation SM 12.5E). At two intersections, and at the beginning and end of the project, the CRCP was removed and a full-depth asphalt section was placed to tie into the existing grade (4 in base 25.0 mm NMAS mixture, VDOT designation BM 25.0D), 4 in IM 19.0E, and 2 in SM 12.5E).

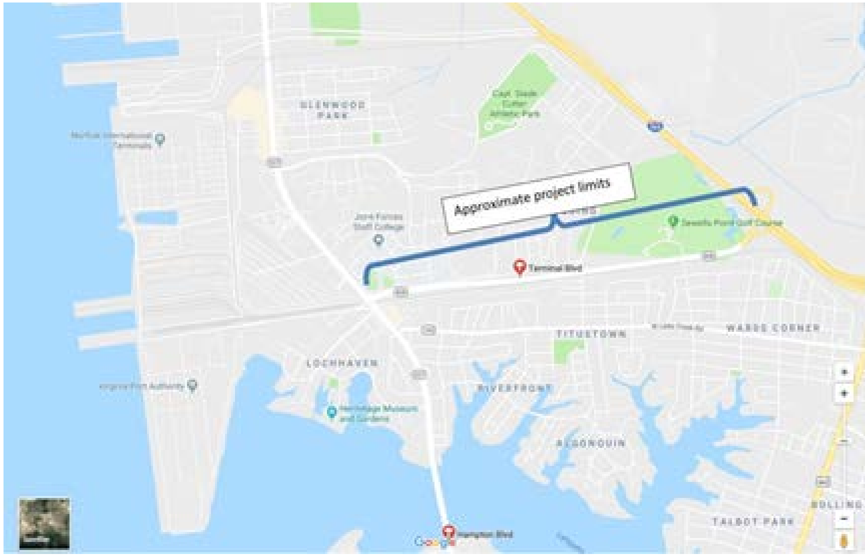



**Figure 1. Approximate Project Limits of SR 406**

**Figure 2. Existing SR 406: CRCP in Poor Condition. CRCP = continuously reinforced concrete pavement.**

**Figure 3. Existing SR 406: CRCP in Poor Condition.  CRCP = continuously reinforced concrete pavement.**

VDOT considered several options before selecting the rubblization as a central reconstruction technique.  Alternative methods included replacement with a jointed plain concrete pavement or the addition of a thick unbonded concrete overlay.  These other methods were not selected because of the higher costs associated with repairing existing distressed locations, limited flexibility in adjusting the finished pavement elevation, limited detours, and a short work window (weekends, Friday night 8 PM to Monday morning 5 AM).

Rubblizing distressed concrete pavement is performed to eliminate reflection cracking in a hot mix asphalt overlay by neutralizing the slab action of the existing pavement while retaining good interlock between the fractured particles.  Although there are other documented uses of this technique in Virginia, experience remains limited.  This project represented the first opportunity to explore performance modeling and analysis using mechanistic-empirical (ME) concepts.

## PURPOSE AND SCOPE

The purpose of this study was to document the current condition of the recently completed rubblization project on Terminal Boulevard (SR 406), to generate baseline pavement performance information for a future study, and to assess the structural adequacy of the rubblized design considering the higher truck traffic on this section.

The scope of the study included the section of Terminal Boulevard that was rehabilitated in 2016 having project limits from the I-564 loop ramp to the intersection with Hampton Boulevard in the westbound direction and from the intersection with Hampton Boulevard to the bridge over the railroad tracks in the eastbound direction.

The performance was assessed by analyzing the results of ride quality, rut depth, automated distress data, and falling weight deflectometer (FWD) testing.  In addition, cores were collected and dynamic modulus and repeated load-permanent deformation tests were performed on the new asphalt overlay materials to provide inputs for ME modeling.

The information will be used to assess the adequacy of the design and for possible future calibration for AASHTOWare Pavement ME Design (hereinafter "Pavement ME"), Version 2.2.

# METHODS

## Literature Review

Literature related to rubblization of existing concrete pavements was identified by searching various databases related to transportation engineering such as the Transport Research International Documentation (TRID) database. The identified literature was then reviewed to summarize the findings from the relevant previous work.

## Design and Construction Summary

The rubblization process and construction details were documented with help from VDOT's Hampton Roads District Materials Division and Virginia Paving Company (the prime contractor for this project). Table 1 shows the mix design used for the surface and intermediate mixtures. Polymer modified binder and 15% reclaimed asphalt pavement (RAP) were used in both mixtures. The design asphalt contents of the surface and intermediate mixtures were 5.8% and 4.8%, respectively. Table 2 shows the pavement design based on the AASHTO 1993 method (American Association of Highway and Transportation Officials [AASHTO], 1993) used for the project.

**Table 1. Mix Designs**

| Material | SM 12.5E (50 Gyration) | IM 19.0E (65 Gyration) |
|---|---|---|
| No. 78 (Granite) | 38% | 40% (No. 67) |
| No. 8 (Granite) | 10% | 20% |
| Sand | 15% | 10% |
| No. 10 washed | 22% | 15% |
| Recycled asphalt pavement (top size 3/8 in) | 15% | 15% |
| Asphalt binder | 5.8% (PG 64E-22) | 4.8% (PG 64E-22) |

RAP = reclaimed asphalt pavement; PG = performance grade.

**Table 2. Pavement Design**

| Flexible | |
|---|---|
| **Mainline** | **Intersections** |
| 2.0 in SM-12.5E | 2.0 in SM-12.5E |
| 4.0 in IM-19.0E (2 lifts of 2 in each) | 4.0 in IM-19.0E (2 lifts of 2 in each) |
| 8 in rubblized existing concrete | 4.0 in BM-25.0D |
| 12.0 in existing cement treated sub-base | 12.0 in existing cement treated sub-base |
| Structural No. = 6.24<br>Layer coefficient<br>Asphalt: 0.44<br>Rubblized layer: 0.18<br>Soil cement: 0.18 | Structural No. = 5.68 |

# Laboratory Performance Evaluation

## Dynamic Modulus

The laboratory evaluation included dynamic modulus testing that was conducted on small-scale specimens extracted from field cores in accordance with the procedure outlined by Diefenderfer et al. (2015). The dynamic modulus describes the stress-strain relationship for a linear viscoelastic material. The testing was conducted in accordance with AASHTO TP 79, Standard Method of Test for Determining the Dynamic Modulus and Flow Number for Hot Mix Asphalt (HMA) Using the Asphalt Mixture Performance Tester (AMPT) (AASHTO, 2013). The test is conducted by subjecting a cylindrical specimen to an axial compressive sinusoidal load at a range of temperatures and loading frequencies. Testing was conducted at temperatures of 4.4, 21.1, 37.8, and 54.4°C. At each temperature, testing was conducted at loading frequencies of 25, 10, 5, 1, 0.5, and 0.1 Hz. Tests were conducted from the coldest to the warmest temperatures. At each temperature, tests were performed from the highest to the lowest frequency.

## Repeated Load Permanent Deformation

The repeated load permanent deformation (RLPD) test was used to evaluate rutting resistance in accordance with AASHTO TP 79 (AASHTO, 2013). Testing was conducted on specimens previously tested for dynamic modulus. Tests were conducted at 54°C based on LTPPBind software that represents the 50% reliability maximum high pavement temperature at sites in central Virginia. A repeated haversine axial compressive load pulse of 0.1 s every 1.0 s was applied to the specimens. The tests were performed in the confined mode using a confining stress of 10 psi (68.9 kPa) and a deviator stress of 70 psi (483 kPa). The tests were continued for 10,000 cycles or a permanent strain of 10%, whichever came first. During the test, permanent strain ($\varepsilon_p$) versus the number of loading cycles was recorded automatically, and the results were used to estimate the flow number, which was determined numerically as the cycle number at which the strain rate is at a minimum based on the Francken model.

## Ideal Cracking Test (IDEAL-CT)

Cracking resistance of the asphalt materials was explored using the indirect tensile asphalt cracking test (IDEAL-CT), which has been proposed by researchers at the Texas

Transportation Institute (Zhou et al., 2017). According to Zhou et al. (2017), this test shows promise in relating laboratory to field performance. Further, it has reasonable repeatability and simplicity, requiring no cutting, drilling, gluing, or notching of the test specimen. The IDEAL-CT is typically performed at room temperature with cylindrical specimens 150 mm in diameter and 62 mm in thickness with a loading rate of 50 mm/min. This test uses a specimen compacted in a gyratory compactor to 7% air voids that is placed in a Marshall load frame (or similar load frame) and loaded to failure in the indirect tensile mode. The load displacement curve is used to determine the CTIndex, a crack susceptibility indicator.

### Binder Extraction and Recovery

Extraction of binder was performed in accordance with AASHTO T 164, Quantitative Extraction of Asphalt Binder from Hot Mix Asphalt (HMA), Method A (AASHTO, 2013), using n-propyl bromide as the solvent. Binder was recovered from the solvent using the Rotavap recovery procedure in accordance with AASHTO T 319, Quantitative Extraction and Recovery of Asphalt Binder from Asphalt Mixtures (AASHTO, 2013).

### Binder Testing

Binder grading was performed in accordance with AASHTO M 320, Performance-Graded Asphalt Binder (AASHTO, 2014). The multiple stress and creep recovery (MSCR) test was also performed. Studies show that non-recoverable creep compliance (Jnr) based on the MSCR test is well correlated with pavement rutting (Anderson et al., 2011).

## Pavement ME Analysis

The pavement section was analyzed using Pavement ME. Pavement ME predicts rutting and cracking of the section based on the input properties (dynamic modulus of asphalt concrete layer; modulus values for rubblized layer, soil cement, and subgrade) and the thickness of the section.

## Field Performance Evaluation

The performance of the sections was evaluated using data from VDOT’s Pavement Management System (PMS). The PMS summarizes detailed distress data for each 0.1 mi of the right lane at the pavement surface. The condition is reported on a scale from 0 to 100, ranging from completely failed to new or like new, respectively. The Critical Condition Index is the lesser of two ratings that summarize the load related and non-load related distresses for a pavement. PMS data also address rutting performance and include the International Roughness Index (IRI) of the sections.

### Deflection Testing

Deflection testing to assess structural capacity was performed in accordance with ASTM D4694-09, Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load

Device (ASTM International [ASTM], 2013). The FWD was equipped with nine sensors at radial distances of 0, 8, 12, 18, 24, 36, 48, 60, and 72 in from the center of a load plate. Deflection testing was conducted at three load levels (6,000; 9,000; and 12,000 lbf) using 150-ft spacing. Following two unrecorded seating drops, four deflection basins were recorded at each load level.

## RESULTS AND DISCUSSION

### Literature Review

#### Slab Fracturing

One way to reconstruct an existing concrete pavement is fracturing the original concrete slab into smaller pieces. The primary benefits of this include the reduction of any slab-like features of the concrete, thus reducing the chances for future reflection cracking of any asphalt overlay. There are three processes in which this can be accomplished: crack and seat, break and seat, and rubblization. These processes are further described here.

The crack and seat method is generally suited for unreinforced plain jointed concrete pavements. In this method, the existing concrete pavement is typically fractured into 18- to 24-in pieces through the depth of the concrete layer using a variety of modified hammers (i.e., pile drivers, guillotine hammers, and drop hammers). Fracturing maintains load transfer through aggregate interlock with minimum loss to the structural value of the concrete layer. Once the concrete is effectively cracked into the appropriate sizes, the pavement is seated by using a 35- to 50-ton pneumatic-tired roller (Timm and Warren, 2004).

The break and seat method follows the same procedure as the crack and seat method but is mainly used on jointed reinforced concrete pavements. In this method, the concrete around the distributed steel is fractured in a way that all the reinforcing steel within the slab is completely debonded. Ensuring debonding of the reinforcing steel from the broken concrete is vital. This is so that the concrete will not behave as one unit and subsequently the effective length of the original slab is reduced or eliminated. Reducing the effective length of the slab minimizes its movement from thermal stresses. Seating after the fracturing is achieved using a 35- to 50-ton pneumatic-tired roller (Timm and Warren, 2004).

Rubblization can be used for any type of deteriorated concrete pavements. In the rubblization process, the existing concrete is cracked into pieces such that the textural and gradation characteristics resemble an aggregate base. The sizes of the cracked pieces typically range from sand sizes to 3 in at the surface of the pavement and from 12 to 15 in at the bottom of the rubblized layer (Von Quintus et al., 2007). Cracking the existing concrete is achieved through heavy-duty pavement breakers such as a resonant pavement breaker (RPB) or a multiple-head breaker (MHB). The seating requirements for rubblization projects involving MHB and RPB are slightly different. A 10-ton tandem steel drum vibratory roller is used in low amplitude and high frequency settings for rubblization projects with RPB. Seating of rubblized projects with MHB is usually achieved using a vibratory roller that is fitted with a “Z” or Elliott grid. The Z-grid roller helps further pulverize and stabilize the broken concrete particles at the surface (Decker and Hansen, 2006).

**Performance With Asphalt Overlays**

Several studies have assessed the performance of asphalt overlays placed on fractured concrete. These studies found that the overlays placed on the rubblized concrete showed better performance compared to the overlays placed on fractured concrete through the crack and seat and break and seat methods (Gulen et al., 2004; Owusu-Abadio and Nelson, 1999; Witzack and Rada, 1992). In a recent survey by Dhakal et al. (2016), responses from 35 highway agencies in the United States and Canada indicated that, among many other treatment options, the most effective treatment delaying reflective cracking for composite pavements was rubblization. States that have reported good to excellent performance results with rubblization are Alabama, Arkansas, Illinois, Iowa, Kansas, Louisiana, Michigan, Nevada, New Jersey, New York, Ohio, Oregon, Pennsylvania, South Carolina, West Virginia, and Wisconsin (Rajagopal, 2011).

In a survey of Iowa's pavement management system, Williams et al. (2015) evaluated the effectiveness of four widely used reflective cracking mitigation strategies for composite pavements. Mill and fill, overlay, heater scarification, and rubblization were included in the study for evaluation, and transverse (reflective) cracking, IRI, and Pavement Condition Index were analyzed as performance indicators for a total of 154 projects, with a service life of 14 years. The study found that rubblization significantly reduced reflective cracking development compared to the other three treatments.

Gaspard et al. (2013) reported the pavement distress and structural performance evaluation for 15 rubblized pavements with a service life of 15 years. The data were extracted from Louisiana's pavement management system. It was found that the overall performance of the rubblized pavements was superior, with minimal and practically negligible pavement distresses such as transverse, longitudinal, and alligator cracking. Based on the analysis performed, the pavements were projected to have an IRI value of approximately 60 in/mi at 15 years of service with acceptable levels of rutting resulting from normal pavement densification. In addition, an increase in the layer moduli for the rubblized pavement layers was observed as the pavement aged, leading the authors to designate rubblizing pavements as a superior option for concrete pavement rehabilitation.

A total of 13 rubblized projects in Ohio were evaluated with regard to the Ohio Department of Transportation's pavement condition rating threshold value of 65 (Rajagopal, 2011). Although there was a variation in performance trends, the rubblized projects were rated as performing well, with 11.7 years of an average performance period for the surface layers of the rubblized pavements. The traffic levels and existing pavement conditions are likely higher as the rubblized sections were constructed on Ohio's priority system highways.

Von Quintus et al. (2007) analyzed historical information and data on rubblization projects built in Wisconsin from 1990 to 2003 to determine the performance characteristics and expected service life of such treatments. Three performance indicators obtained from Wisconsin's pavement management databases were included in the analysis: Pavement Distress

Index, average rut depth, and IRI. It was found that the rubblization projects had a low severity of distresses. The analysis also revealed that the average service life expected for the rubblized pavements built between 1990 and 1997 was around 17 years whereas the service life was projected to reach or exceed the design life of 22 years for the projects completed after 1997. The study reported that the average service life for rubblized pavements was about 15 to 25 years for high and low volume roadways, respectively, citing an earlier study in Wisconsin.

LaForce (2006) evaluated the 6-year performance of the first rubblization project in Colorado. The field investigation showed that the asphalt overlay neither exhibited any reflective cracking from the rubblized concrete pavement nor demonstrated any settlement, permanent deformation, or other distress as a result of the rubblization process. It was also reported that the overlay performed similarly to other newly constructed asphalt pavements.

Sebesta et al. (2006) reported the performance of three existing rubblization projects in Texas to be good to excellent. Timm and Warren (2004) evaluated the performance of nine rubblized sections located on the interstate system in Alabama. The study included transverse crack density, average IRI, average rutting, percent longitudinal cracks per pavement section, and average alligator cracking as performance criteria. The authors compared the rubblization performance data to the critical values established by AASHTO and other state departments of transportation and reported that the rubblization projects either met or exceeded all of the criteria. The authors cited a memorandum from an engineer in the Alabama Department of Transportation and reported that the performance life of the rubblized pavement wearing surface was 8 to 10 years.

Rubblizing with an asphalt mixture overlay was reported as a successful rehabilitation method on both interstate and non-interstate projects in Illinois (Heckel, 2002). It was reported that over the course of 10 years, the asphalt mixture overlays of rubblized concrete pavements had none or minimal reflective cracking from D-cracking, as well as joints, cracks, and patches. It was also indicated that the rubblized pavement performed better than patching and overlaying with an asphalt mixture.

The Asphalt Institute performed an evaluation of 43 rubblization projects that ranged in age from 1 to 13 years across different regions of the United States (Fitts, 2001). The sections did not have any signs of reflective cracking but exhibited minimal rutting and fatigue cracking. Based on extrapolations of performance data from pavement condition ratings, the sections were projected to have a service life of 22 years.

Poor performance of rubblized projects was also noted in the literature. For example, it was reported that some rubblized sections constructed in Michigan since 1986 had very good performance whereas others showed various levels of distress (cracking, rutting, and raveling). The average service life of these poorer performing sections was reported as 14 years, although the design life was 20 years. A forensic investigation identified that construction and material issues were the underlying reasons for the observed premature failures. A set of recommendations for rubblization projects was developed from this study (Baladi and Niederquell et al., 1999; Niederquell et al., 2000). Some projects constructed in the early 1990s in Colorado also showed poor performance, attributed to low base modulus resulting in a structurally inadequate section to carry the traffic loads (Sebesta et al., 2006).

## Design and Construction Summary: Terminal Boulevard

The Terminal Boulevard resurfacing project included the following:

- rubblization of existing concrete pavement, with an overlay of 6 in of asphalt pavement
- adjustments to affected drop inlets, bringing them up to the grade of the new roadway, and construction of new curbs and gutters
- removal and replacement of existing guardrail
- tie-in to existing grade at the Ruthven Road and Diven Street intersections to avoid impacts to the railroad
- removal and replacement of damaged stormwater pipes.

Figures 4 and 5 show the equipment used and the rubblization process. One of the rubblization machines had 16 hammers (each 1,500 lb), and the second had 12 hammers. A rubblization machine breaks up the old concrete pavement into aggregate, which serves as a base for the new roadway. Rubblization involved breaking up the concrete pavement into maximum-size pieces while also separating reinforcing steel from the concrete. The specification required at least 75% of the broken particles to be less than 4 in in size; at the surface of the rubblized layer, all pieces were required to be less than 6 in in size. In the lower portion of the slabs (below the reinforcing steel), the maximum particle size was required to be 12 in. Figure 6 shows the particle size of aggregate immediately after rubblization (before rolling with compactors).


**Figure 4. Rubblization Machine**


**Figure 5. Rubblization Process**

**Figure 6. Particle Size After Rubblization (Before Rolling)**

Three different rollers follow the rubblization machine. First, a Z-pattern drum (Figure 7), then a rubber tire roller (Figure 8), and finally a steel drum roller (Figure 9) were used.

**Figure 7. Z-Pattern Drum**


**Figure 8. Rubber Tire Roller**


**Figure 9. Steel Drum Roller**

Figure 10 shows the aggregate particle size after Z-roller compaction, and Figure 11 shows the final rubblization surface. During rolling, reinforcing bars in old CRCP that came to the surface were removed before paving with asphalt.

**Figure 10. Aggregate Particle Size After Z-Roller Compaction**

**Figure 11. Final Rubblization Surface**

As mentioned earlier, at two intersections and at the beginning and end of the project, the CRCP was removed. Transverse saw cuts at a 10-ft spacing were used for excavating the concrete since the length of the reinforcing bar was manageable for loading in trucks. A guillotine drop hammer (varying in weight from 13,000 to 16,000 lb) was also used in part of the full-depth concrete removal areas. Most of the full-depth removal areas had good hard-stabilized soil cement with a few exceptions where water was present on the surface of the soil cement. Total concrete rubblization included 42,000 $yd^2$, and concrete removal was 11,200 $yd^2$.

After rubblization, 4 in of IM 19.0E mixture (two 2-in lifts) and 2 in of SM 12.5E mixture were used for paving (Figures 12 and 13). A total of 10,000 tons of IM 19.0E mixture and 5,600 tons of SM 12.5E mixture were used. In areas with full-depth removal, 4 in of base mixture (BM 25.0 D) was placed below the intermediate and surface mixtures. Figure 14 shows a part of the section before and after paving.

**Figure 12. Placement of IM 19.0E Mixture**

**Figure 13. Placement of SM 12.5E Mixture**

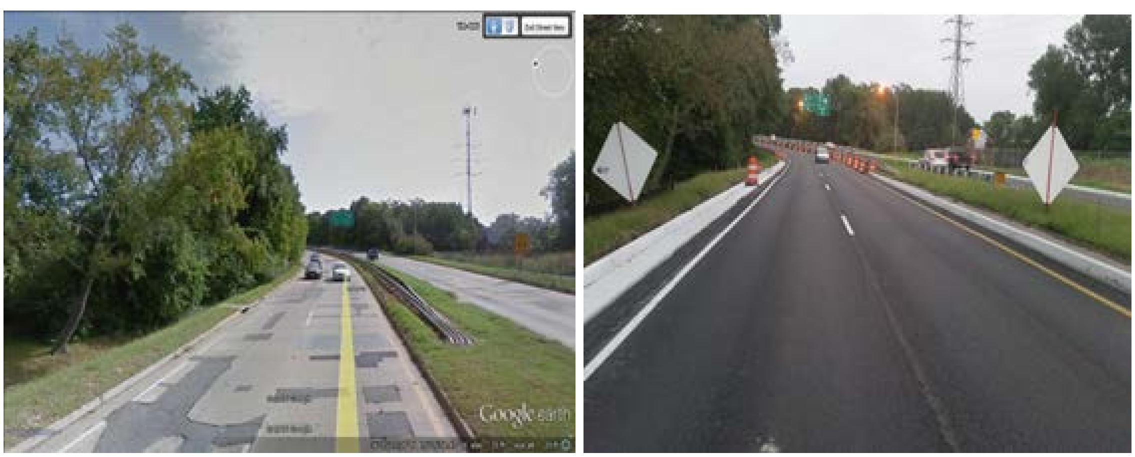


**Figure 14. Project Site Before and After Paving**

Most of the rubblization and paving work was done on weekends (Friday 8 PM through Monday 5 AM). In addition to the rubblized pavement, the project involved a total of 10 concrete removal areas. Removal included anchor lugs in 3 of the 10 areas, and less than desirable subgrade conditions were encountered in 3 others. The Terminal Boulevard resurfacing project was completed on November 18, 2016, nearly 2 months ahead of the original contract completion date.

## Laboratory Performance Evaluation

Volumetric properties and gradation analyses were collected from VDOT's Materials Information Tracking System / Producer Lab Analysis and Information Detail (MITS/PLAID) System, and the results are shown in Table 3 (VDOT-reported results). In general, the mixtures met the VDOT volumetric and gradation specification requirements.

**Table 3. Volumetric Properties for SM 12.5E and IM 19.0E Mixtures From VDOT's MITS/PLAID System**

| Property | SM 12.5E | | | IM 19.0E | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | **Sample 1** | **Sample 2** | **Sample 3** | **Sample 1** | **Sample 2** | **Sample 3** | **Sample 4** | **Sample 5** | **Sample 6** |
| % AC | 5.53 | 5.62 | 5.98 | 4.67 | 4.86 | 5.02 | 4.4 | 4.49 | 4.78 |
| Rice Specific Gravity, $G_{mm}$ | 2.437 | 2.437 | 2.423 | 2.474 | 2.457 | 2.468 | 2.472 | 2.468 | 2.465 |
| % Air Voids, $V_a$ | 4.1 | 2.7 | 2.6 | 4.1 | 3.2 | 3 | 3 | 3.5 | 3.7 |
| % VMA | 16.3 | 16.3 | 15.9 | 14.6 | 14.2 | 14.5 | 13 | 13.6 | 14.5 |
| % VFA | 75 | 82 | 84 | 72 | 77 | 79 | 77 | 74 | 74 |
| Dust/Asphalt Ratio | 0.8 | 0.8 | 0.7 | 0.9 | 0.8 | 0.9 | 1.2 | 1 | 0.9 |
| Bulk Specific Gravity, $G_{mb}$ | 2.336 | 2.371 | 2.361 | 2.373 | 2.379 | 2.394 | 2.398 | 2.382 | 2.375 |
| Effective Specific Gravity, $G_{se}$ | 2.649 | 2.653 | 2.651 | 2.656 | 2.644 | 2.665 | 2.642 | 2.641 | 2.65 |
| Aggregate Specific Gravity, $G_{sb}$ | 2.638 | 2.642 | 2.64 | 2.65 | 2.638 | 2.659 | 2.636 | 2.635 | 2.644 |
| % Binder Absorbed, $P_{ba}$ | 0.16 | 0.16 | 0.16 | 0.09 | 0.09 | 0.09 | 0.09 | 0.09 | 0.09 |
| Effective % Binder, $P_{be}$ | 5.38 | 5.47 | 5.83 | 4.58 | 4.77 | 4.93 | 4.31 | 4.4 | 4.69 |
| **Sieve Size** | **Average Percent Passing** | | | | | | | | |
| 3/4 in (19.0 mm) | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 |
| 1/2 in (12.5 mm) | 93 | 93 | 93 | 84 | 86 | 85 | 82 | 84 | 82 |
| 3/8 in (9.5 mm) | 86 | 85 | 82 | 71 | 73 | 74 | 66 | 68 | 67 |
| No. 4 (4.75 mm) | 59 | 57 | 58 | 45 | 45 | 47 | 42 | 46 | 43 |
| No. 8 (2.36 mm) | 43 | 42 | 43 | 32 | 31 | 34 | 30 | 33 | 32 |
| No. 16 (1.18 mm) | 32 | 31 | 33 | 24 | 24 | 26 | 23 | 25 | 24 |
| No. 30 (600 µm) | 20 | 21 | 21 | 16 | 16 | 18 | 16 | 18 | 16 |
| No. 50 (300 µm) | 12 | 12 | 12 | 10 | 10 | 11 | 11 | 11 | 10 |
| No. 100 (150 µm) | 7 | 7 | 7 | 7 | 6 | 7 | 8 | 7 | 6 |
| No. 200 (75 µm) | 4.2 | 4.2 | 4.2 | 4.3 | 3.9 | 4.2 | 5.2 | 4.6 | 4.1 |

AC = asphalt content; VMA = voids in mineral aggregate; VFA = voids filled with asphalt.

Six full-depth asphalt cores were obtained from the project for performance testing. Cores were taken approximately 2 years after construction (from the VDOT-maintained section of Terminal Boulevard). All of the cores were taken from the center of the lane. Core locations are shown in Figure 15; Figure 16 shows one of the cores. All six cores were intact and there was no delamination, indicating a good bond between the layers.

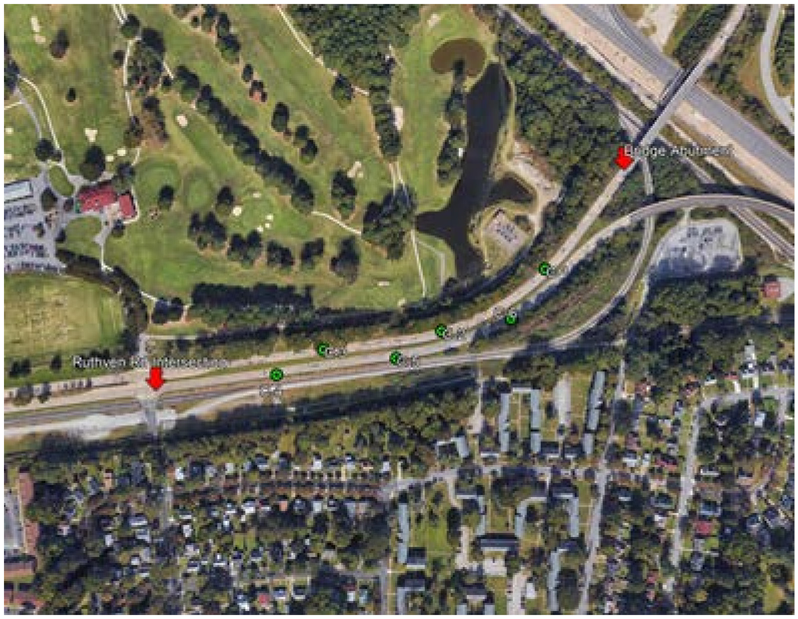



**Figure 15. Core Locations (Green Dots)**

**Figure 16. Full-Depth Core**

Table 4 shows core thickness and percent air voids. Cores for the SM 12.5E and IM 19.0E mixtures had average air voids of 4.6% and 6.7%, respectively, indicating that excellent compaction was achieved in the field. Since the cores were taken from the center of the lane, it was assumed that these density results reflected the initial field density.

**Table 4. Core Thickness and Air Voids**

| Core No. | Average Layer Thickness, in | Average Layer Thickness, in (mm) | | Air Void Content, % | |
|---|---|---|---|---|---|
| | | 12.5 mm Surface Mixture | 19.0 mm Intermediate Mixture | 12.5 mm Surface Mixture | 19.0 mm Intermediate Mixture |
| C-1 | 7.5 | 2.0 (51.2) | 5.3 (134.5) | 4.9 | 6.7 |
| C-2 | 7.5 | 1.5 (39.3) | 5.5 (139.1) | 2.9 | 8.9 |
| C-3 | 7.5 | 2 (51.6) | 5 (129.2) | 5.1 | 6.2 |
| C-4 | 5.5 | 1.6 (42.0) | 3.6 (93.1) | 5.6 | 5.1 |
| C-5 | 7 | 2.3 (57.7) | 4.2 (106.5) | 4.4 | 6.5 |
| C-6 | 5.5 | 1.6 (42.2) | 3.5 (89.4) | 4.9 | 6.8 |

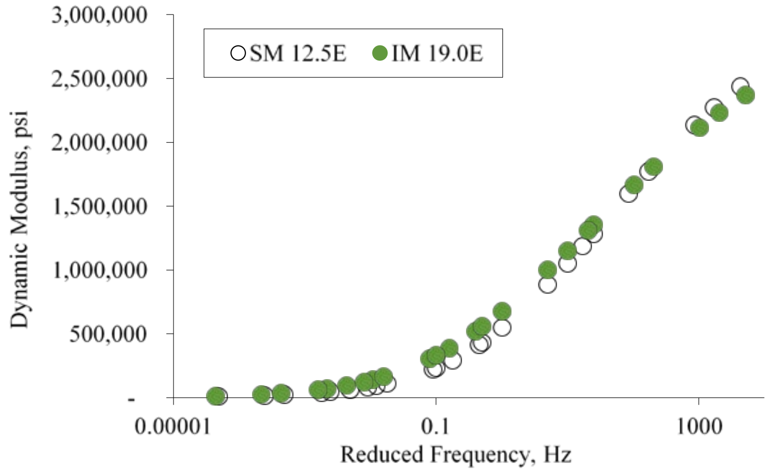
## Dynamic Modulus

As mentioned earlier, small-scale specimens extracted from cores were used for dynamic modulus testing. Dynamic modulus is one of the major inputs for asphalt mixtures in Pavement ME. Specimen thickness, diameter, and air-void details are shown in Table 5.

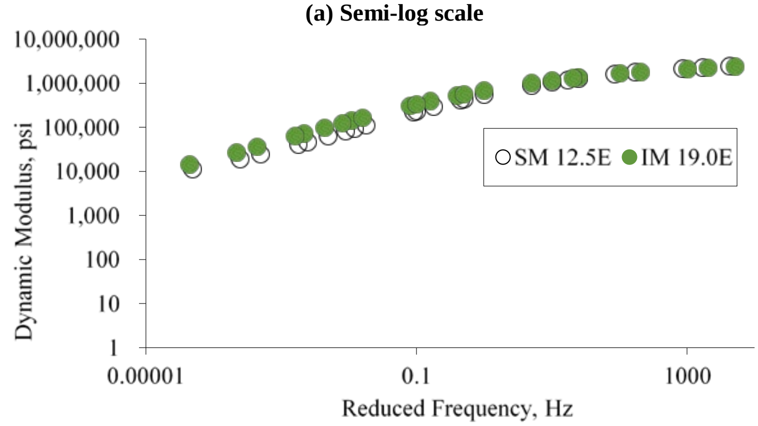
Dynamic modulus master curves for SM and IM mixtures are shown in Figure 17. It can be seen that the modulus of SM 12.5E mixture is slightly less than of the IM 19.0 mixture at higher (lower reduced frequency) and intermediate temperatures. This is because of the higher binder content in the SM 12.5E mixture compared to the IM 19.0E mixture.

**Table 5. Specimen Details for Cores Used in Dynamic Modulus Testing**

| Sample ID | Diameter, mm | Height, mm | Air Voids, % | Average % Air Voids |
|---|---|---|---|---|
| SM-C1 | 38.1 | 109.5 | 4.6 | 4.56 |
| SM-C3 | 38.3 | 109.5 | 4.7 | |
| SM -C6 | 38.2 | 109.6 | 4.4 | |
| IM-C1 | 48.7 | 109.0 | 7.1 | 6.87 |
| IM-C1 | 48.9 | 109.0 | 6.9 | |
| IM C5 | 49.2 | 109.6 | 6.0 | |
| IM-C6 | 49.2 | 109.1 | 7.5 | |




**(a) Semi-log scale**




**(b) log-log Scale**

**Figure 17. Dynamic Modulus Test Results for SM 12.5E and IM 19.0E Mixtures**

## Repeated Load Permanent Deformation (Flow Number) Test

The flow number represents the number of cycles required for loaded specimens to begin exhibiting tertiary creep, or flow. This condition is defined as the number of cycles that corresponds to the minimum rate of change in permanent axial strain of the specimen under a repeated load test. The test response for each mixture (three specimens each for the surface and intermediate mixture) is shown in Figure 18. Both mixtures had the maximum flow number value of 10,000, indicating excellent rutting resistance. Polymer modified binders in both mixtures contributed to the potential for increased rutting resistance.

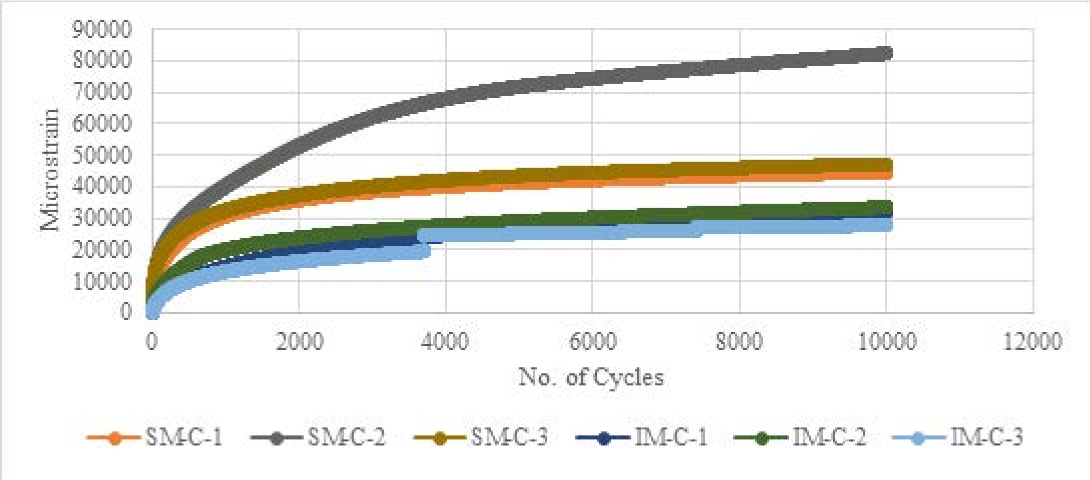



**Figure 18. RLPD Results (No. of Cycles vs. Microstrain). RLPD = Repeated load permanent deformation.**

## IDEAL-CT

Field cores were used for IDEAL-CT testing, and the results are shown in Table 6. As air voids and thickness values are different compared to the test requirements (62 mm thickness and 7% air voids), comparison of these results to test data from other studies is not possible. However, these values will be useful to compare with future core test values from this project to determine any changes in future cracking resistance.

**Table 6. IDEAL-CT Results**

| Core ID | Thickness, mm | Diameter, mm | Air Voids | IDEAL-CT Value |
|---|---|---|---|---|
| SM-1 | 39.3 | 149.9 | 2.9% | 127.7 |
| SM-2 | 41.9 | 149.9 | 5.6% | 68.4 |
| SM-3 | 57.7 | 149.9 | 4.4% | 41.6 |
| IM-1 | 50.7 | 152.1 | 7.4% | 43.1 |
| IM-2 | 51.9 | 152.1 | 8.8% | 35.6 |
| IM-3 | 49.9 | 151.9 | 3.9% | 43.1 |
| IM-4 | 48.8 | 152.4 | 7.4% | 173.6 |
| IM-5 | 47.9 | 151.9 | 4.9% | 69.0 |

## Binder Testing

Binder test results comprise one of the inputs required for Pavement ME analysis. Results from extracted binder are shown in Table 7. The rolling thin film oven (RTFO) failure temperature of the binder from the SM 12.5E mixture was slightly higher than that of the binder from the IM 19.0E mixture, indicating possible field aging (assuming similar RAP binder stiffness in both mixtures). However, the final performance grade for the SM 12.5E binder was PG 82-22 and for the IM 19.0E binder was PG 82-16 (but very close to PG 82-22).

**Table 7. Extracted Binder Test Results**

| Property | SM 12.5E | IM 19.0E |
|---|---|---|
| **Dynamic Shear, 10 rad/sec, specification: G*/sin delta > 2.20 kPa** | | |
| RTFO G*/sin delta, 76ºC | 4.997 | 4.185 |
| RTFO G*/sin delta, 82ºC | 2.729 | 2.257 |
| RTFO G*/sin delta, 88ºC | 1.543 | 1.232 |
| RTFO G*, 76ºC | 4.721 | 4.012 |
| RTFO G*, 82ºC | 2.612 | 2.188 |
| RTFO G*, 88ºC | 1.493 | 1.206 |
| RTFO phase angle, 76ºC | 73.15 | 73.48 |
| RTFO phase angle, 82ºC | 75.49 | 75.82 |
| RTFO phase angle, 88ºC | - | 78.2 |
| RTFO failure temperature | 84.31 | 82.33 |
| **Dynamic Shear, 10 rad/sec, specification: G* sin delta < 5000 kPa** | | |
| PAV G* sin delta, 25.0ºC | 5580 | - |
| PAV G* sin delta, 28.0ºC | 3977 | 5137 |
| PAV G* sin delta, 31.0ºC | 2801 | 3694 |
| PAV G*, 25.0ºC | 8.398E+06 | - |
| PAV G*, 28.0ºC | 5.714E+06 | 7.866E+06 |
| PAV G*, 31.0ºC | 3.863E+06 | 5.396E+06 |
| PAV phase angle, 25.0ºC | 41.64 | - |
| PAV phase angle, 28.0ºC | 44.12 | 40.78 |
| PAV phase angle, 31.0ºC | 46.47 | 43.2 |
| PAV failure temperature | 26 | 28.25 |
| **Creep Stiffness, 60 sec, specification: Stiffness < 300 MPa and m-value > 0.300** | | |
| Stiffness, -6ºC | - | 128 |
| M-value, -6ºC | - | 0.333 |
| Stiffness, -12ºC | 275 | 261 |
| M-value, -12ºC | 0.308 | 0.293 |
| Stiffness, -18ºC | 440 | - |
| M-value, -18ºC | 0.257 | - |
| $\Delta T_c$, ºC | 0 | - |
| Stiffness failure temperature | -22.9 | - |
| M-value failure temperature | -22.9 | - |
| Performance Grade | 82-22 | 82-16 |

RTFO = rolling thin film oven; PAV = pressure aging vessel; - = data not available/not relevant.

The parameter ΔTc, defined as the difference between the bending beam rheometer stiffness failure temperature and the m-value failure temperature, is an indicator of non-load related cracking susceptibility and has been proposed as a relatively simple method for measuring the loss of relaxation properties of asphalt binders. Minimum thresholds for $\Delta T_{cr}$ of -2.5 and -5.0 representing the cracking warning and cracking limit, respectively, have been recommended by previous work (Anderson et al., 2011). The ΔTc for the SM 12.5E mixture is shown in Table 7, and a value of zero indicates resistance to non-load related cracking.

The MSCR test uses the well-established creep and recovery concept to evaluate the binder's potential for permanent deformation. MSCR tests in this study were performed at 64°C. For heavy, very heavy, and extremely heavy traffic, $J_{nr}$ is 2.0, 1.0, and 0.5 $kPa^{-1}$, respectively. The Jnr test results from the MSCR test (Table 8) indicated that both binders were rut resistant against extremely heavy traffic. Higher percentage recovery values confirm the presence of polymers in both binders.

**Table 8. MSCR Test Results for Extracted Binder**

| Sample Source | Property | Binder Data | |
|---|---|---|---|
| | | SM 12.5E | IM 19.0E |
| Extracted Binder | Performance grade | 82-22 | 82-16 |
| | Non-recoverable Jnr100Pa | 0.1612 | 0.2435 |
| | Non-recoverable Jnr3200Pa | 0.1888 | 0.2834 |
| | % Jnr | 17.15 | 16.36 |
| | Avg. % recovery R100Pa | 54.99 | 44.99 |
| | Avg. % recovery R3200Pa | 47.47 | 36.6 |
| | % difference | 13.68 | 18.65 |

MSCR = multiple stress and creep recovery.

## Deflection Testing

Deflection tests were conducted using a Dynatest Series 8000 FWD and were performed by a third party consultant. Figure 19 shows the results of this testing in terms of the deflection at the loading plate (D0) and the deflection at a distance of 72 in from the loading plate (D72). The D0 parameter is an indicator of the overall structural capacity of the pavement system, whereas D72 is an indicator of the quality of the pavement foundation. The results in Figure 19 show the pavement foundation to be stiff and uniform with a deflection of approximately 2 mils. The results in Figure 19 also show the deflection of D0 to be uniform from Station 0 to approximately Station 30, with an average of approximately 7 mils, and a bit higher from approximately Station 30 to Station 58, with an average of approximately 8 mils. Given the variation, the change is not statistically significant.

Table 9 gives the average deflection and standard deviation for each lane tested. It can be seen that all of the lanes had similar deflection values for the three different load levels, indicating the same structure strength in all lanes. Earlier studies showed that these deflection values indicate a strong structural pavement (Diefenderfer et al., 2019; Pierce et al., 2017).

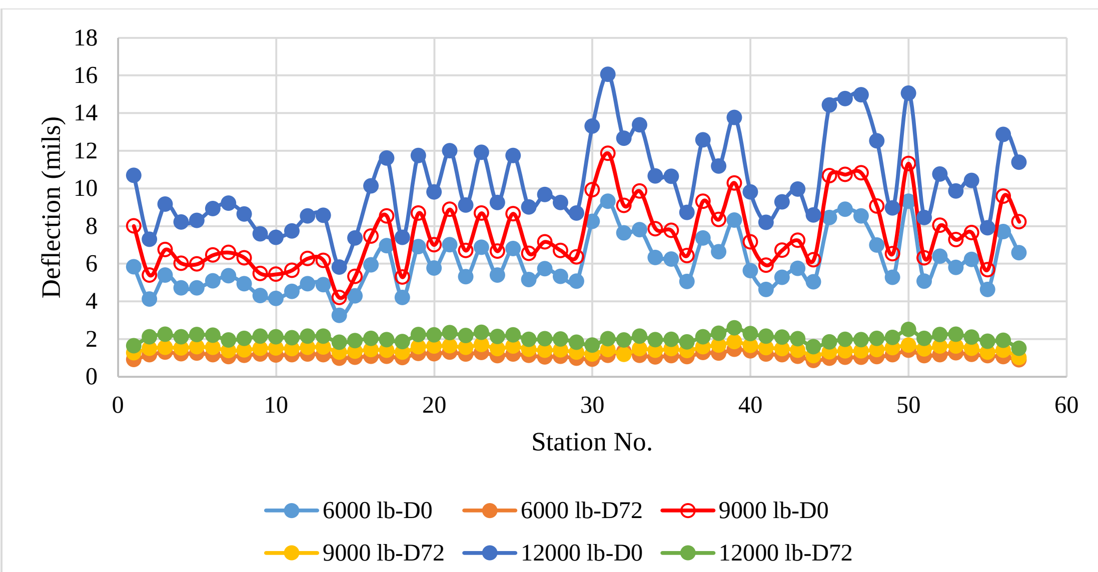



**Figure 19. Deflection Results From FWD Testing at Eastbound Lane 1. FWD =falling weight deflectometer; D0 = deflection at loading plate; D72 = deflection at 72 in from loading plate.**

**Table 9. Deflection Data for All Lanes (Average Pavement Temp 90°F)**

| Lane | Load, lb | Deflection, mils | |
|---|---|---|---|
| | | **D0, Average (SD )** | **D72, Average (SD)** |
| EB lane 1 | 6000 | 6 (1.4) | 1.1 (0.1) |
| | 9000 | 7.5 (1.7) | 1.5 (0.2) |
| | 12000 | 10.3 (2.3) | 2.1 (0.2) |
| EB lane 2 | 6000 | 5.6 (1.1) | 1.1 (0.2) |
| | 9000 | 7.2 (1.4) | 1.4 (0.2) |
| | 12000 | 9.9 (1.9) | 2 (0.3) |
| WB lane 1 | 6000 | 6.3 (1.5) | 1.2 (0.1) |
| | 9000 | 7.9 (1.9) | 1.5 (0.2) |
| | 12000 | 10.8 (2.5) | 2.1 (0.2) |
| WB lane 2 | 6000 | 6 (1.6) | 1.1 (0.1) |
| | 9000 | 7.6 (1.9) | 1.4 (0.2) |
| | 12000 | 10.5 (2.6) | 2 (0.2) |

EB = eastbound; WB = westbound; SD = standard deviation.

# Inputs for Pavement ME Analysis

## Traffic Inputs

Traffic details for SR 406 are shown in Table 10. VDOT's historical traffic data (VDOT, 2019) were used to determine the truck class distributions. Inputs for vehicle class distribution, axle load spectra, and axles per truck were used in accordance with VDOT's *Pavement ME User Manual—2017* (VDOT, 2017). Based on historical traffic data, a 2% traffic growth rate was used. Based on the data in Table 10, annual average daily truck traffic of 2110 (one direction) was used in the Pavement ME analysis.

**Table 10. Traffic Data for SR 406**

| Direction | 2015 Annual Average Daily Traffic | Percent Trucks |
|---|---|---|
| Eastbound | 17,600 | 2.3% single unit trucks, 9.7% tractor-trailer units |
| Westbound | 15,900 | 2.5% single unit trucks, 10.5% tractor-trailer units |

## Thickness and Modulus Inputs

A 2015 pavement evaluation of the pre-rubblization pavement (Applied Research Associates, unpublished data, 2015) encountered 8 to 12 in of CRCP underlain by 6 to 28 in of stabilized sub-base. The thickness of the cement-treated sub-base was estimated from changes in the standard penetration test values. The reported average resilient moduli of the subgrade, based on dynamic cone penetrometer testing, in the eastbound and westbound directions were 29,013 psi and 23,533 psi, respectively. Minimum subgrade resilient modulus ($M_r$) values were approximately 12,700 psi, and the subgrade type was A-2-4 soil. A subgrade resilient modulus (Mr) value of 16,500 psi was used in the ME analysis, which is the default value suggested in Pavement ME for A-2-4 soil.

During the 2015 field investigation, only one sample of the cement-treated base was obtained (2.25-in-long core sample). The core sample failed at 1,100 psi during the compressive strength testing. However, the failure to obtain a sample with adequate dimensions for testing at the other locations could be an indication of possibly weaker material elsewhere within the project. The thickness of the soil cement was variable, with readings from 6 to 28 in reported. The thickness of the soil cement used in the new pavement design calculations has an impact on the service life of the asphalt concrete pavement design; a soil cement thickness of 12 in was used for the ME analysis. VDOT's *Pavement ME User Manual—2017* (VDOT, 2017) suggests a resilient modulus of 500,000 psi for soil cement. Assuming a weaker material and possible damage caused by the rubblization process, separate analyses were performed using values of 200,000 and 300,000 psi in Pavement ME.

A single weather station from near the project location, Norfolk, was selected as the reference for climatic data. Asphalt material properties are shown in Tables 11 and 12. Since dynamic modulus is not tested at -10ºC (which is a required input in Pavement ME), modulus values were estimated from master curves using the time-temperature superposition principle.

The modulus of a rubblized pavement is an important parameter that is needed for determining the thickness of proposed asphalt overlays. In general, the greater extent of rubblization achieved during construction (i.e., smaller particle sizes), the lower the modulus of the rubblized layer. The representative elastic modulus calculated for the rubblized layer with much larger particles (6 to 12 in in size) was found in the literature to exceed 70,000 psi, suggesting good interlocking between the fractured particles (Von Quintus and Tam, 2000). A study by the National Asphalt Pavement Association (NAPA) recommended elastic modulus values of 100,000 to 150,000 psi for a rubblized layer (NAPA, 1994). The default value recommended for use in the *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures* is 100,000 psi (Applied Research Associates, 2004). Von Quintus et al. (2007) suggested an estimated average elastic modulus of the rubblized layer to be 65,000 psi based on matching the predicted to the observed pavement performance of rubblized projects in Wisconsin. The study by NAPA (1994) concluded that the expected strength of the rubblized layer is 1.5 to 3 times as effective in load distribution characteristics as a high-quality, dense-graded crushed stone base. Typical VDOT aggregate base (21A/B) modulus values range from 16,500 to 23,000 psi (Hossain, 2010).

**Table 11. Dynamic Modulus Values for SM 12.5E Mixtures**

| | Dynamic Modulus (E*) | | | | | | |
|---|---|---|---|---|---|---|---|
| | **Temp.** | **25 Hz** | **10 Hz** | **5 Hz** | **1 Hz** | **0.5 Hz** | **0.1 Hz** |
| **SM 12.5E** | -10ºC | 3159301 | 3095168 | 3037801 | 2869545 | 2779715 | 2524921 |
| | 4ºC | 2493395 | 2313563 | 2163080 | 1772725 | 1592168 | 1167998 |
| | 20ºC | 1291605 | 1055209 | 886402 | 509001 | 432738 | 233407 |
| | 38ºC | 413546 | 292788 | 221634 | 111683 | 82496 | 41318 |
| | 54.4ºC | 94142 | 63132 | 46908 | 24485 | 18960 | 11229 |

**Table 12. Dynamic Modulus Values for IM 19.0E Mixtures**

| | Dynamic Modulus (E*) | | | | | | |
|---|---|---|---|---|---|---|---|
| | **Temp.** | **25 Hz** | **10 Hz** | **5 Hz** | **1 Hz** | **0.5 Hz** | **0.1 Hz** |
| **IM 19.0E** | -10ºC | 2900670 | 2813845 | 2738859 | 2530505 | 2425036 | 2142814 |
| | 4ºC | 2423757 | 2269240 | 2141279 | 1811351 | 1658086 | 1290389 |
| | 20ºC | 1364707 | 1155699 | 1002188 | 677563 | 557012 | 331595 |
| | 38ºC | 518696 | 387330 | 304836 | 165471 | 124754 | 63030 |
| | 54.4ºC | 141353 | 96522 | 71820 | 35969 | 26845 | 14066 |

In general, no consistent elastic modulus values have been reported to represent rubblized concrete pavement layers, and the value is site specific and dependent on the rubblization process itself. Separate ME analyses were done using modulus values of 65,000 and 100,000 psi for the rubblized layer for this project, which the researchers expect to be conservative values.

The Pavement ME analyses are summarized in Table 13. A 30-year design life was used in the analysis. VDOT completed local calibration of the MEPDG distress models focusing on bottom-up fatigue cracking and rutting (Smith and Nair, 2015). Currently, VDOT considers bottom-up fatigue cracking and total rutting to be performance criteria to design flexible pavements.

It is acknowledged that rehabilitation work is likely to be performed on a pavement before the end of the design life to maintain its functional characteristics, whereas the objective of the design life is to prevent structural repairs from being required during the design life period. Based on VDOT's *Pavement ME User Manual—2017* (VDOT, 2017), a value of 6% is a recommended limit for bottom-up fatigue cracking and a limiting value of 0.26 in is recommended for predicted rutting distress in a 15-year period.

In general, Pavement ME analysis predicted a design life of 18 to 19 years with respect to bottom-up fatigue cracking criteria (<6% cracking). However, ME analysis predicted only 7.1% bottom-up cracking for a 30-year design period. The original design was based on the AASHTO 93 empirical method (AASHTO, 1993) using a 30-year design life. Pavement ME predicted that the rutting distress limit (0.26 in) will be reached in 5 to 6 years, which is sooner than expected since polymer modified binders were used in surface and intermediate mixtures.

**Table 13. Pavement ME Analysis Results**

| | **Distress at Specified Reliability** | | | |
|---|---|---|---|---|
| | **Distress Predicted at 30 Years (rutting at 15 years)** | **Year Predicted Distress Reaches Limit (6% cracking and 0.26 in rutting)** | **Distress Predicted at 30 Years (tutting at 15 years)** | **Year Predicted Distress Reaches Limit (6% cracking and 0.26 in rutting)** |
| **Distress** | **CTA modulus = 200,000 psi** | | | |
| | **Modulus of rubblized layer: 65,000 psi** | | **Modulus of rubblized layer: 100,000 psi** | |
| AC bottom-up fatigue cracking (% lane area) | 7.1 | 18 years | 7.1 | 18 years |
| Permanent deformation, total pavement (in) | 0.47 | 5 years | 0.46 | 6 years |
| **Distress** | **CTA modulus = 300,000 psi** | | | |
| | **Modulus of rubblized layer: 65,000 psi** | | **Modulus of rubblized layer: 100,000 psi** | |
| AC bottom-up fatigue cracking (% lane area) | 6.89 | 19 years | 6.89 | 19 years |
| Permanent deformation, total pavement (in) | 0.45 | 5 years | 0.45 | 6 years |

AC = asphalt concrete.

Since small-scale testing was used for dynamic modulus, modulus values at 54ºC were also predicted from the master curve using the time-temperature superposition principle. Earlier research work (Diefenderfer et al., 2015) did not recommend small-scale dynamic modulus testing at 54ºC. Dynamic modulus results are shown in the Appendix (Table A1 and Table A2). Pavement ME analysis was also done using those dynamic modulus values, and similar results as shown in Table 13 were obtained. Since this project used an IM 19.0 mixture, which is a polymer modified mixture and not commonly used, comparison to other available VDOT mixture properties was not possible. An example of Pavement ME analysis output is also shown in the Appendix.

## Early-Age In-Service Performance

Early-age distress data were extracted from VDOT's PMS and are shown in Table 14. There was no observed cracking in the pavement. Table 14 shows an increase in roughness and rutting in 2017 and 2018, but the difference is within the random variability encountered with year-to-year testing. More detailed data to include each 0.1-mi segment are shown in the Appendix. Additional years of data are required to determine if a trend is evident.

**Table 14. Distress Data From VDOT's PMS**

| Direction | Year | Mileposts | IRI Average, in/mi | Rut Depth Average, in | CCI Average |
|---|---|---|---|---|---|
| Eastbound | 2018 | 0-1.33 | 97 | 0.13 | 97 |
| | 2017 | 0-1.33 | 95 | 0.09 | 99 |
| Westbound | 2018 | 1.33-0.052 | 107 | 0.17 | - |
| | 2017 | 1.33-0.048 | 99 | 0.09 | - |

PMS = Pavement Management System; IRI = International Roughness Index; CCI = Critical Condition Index.

## Summary of Findings

- The asphalt mixtures placed on the SR 406 rubblization project met the VDOT volumetric and gradation specification requirements.
- Cores collected showed the asphalt mixtures used had good field density, and intact cores indicated adequate interlayer bonding.
- The flow number test results indicated that the IM 19.0E and SM 12.5E mixtures are rut resistant.
- The $\Delta T_c$ parameter for the SM 12.5E mixture indicated resistance to non-load related cracking.
- Jnr test results from the MSCR test indicated that binders used in the IM 19.0E and SM 12.5E mixtures were rut resistant against extremely heavy traffic.
- Deflection values from FWD testing indicated a structurally strong pavement section.

- Pavement ME analysis showed a design life of 18 to 19 years for the pavement. Analysis also predicted that the rutting distress limit (0.26 in) will be reached in 5 to 6 years, which is sooner than expected since polymer modified binders were used in surface and intermediate mixtures. Initial predicted rutting and observed rutting PMS were similar. However, more data are needed to confirm the trend.

- VDOT pavement management data showed an increase in roughness and rutting in 2017 and 2018.

## CONCLUSIONS

- *Continued performance monitoring of this section is needed to calibrate/validate the Pavement ME performance models.* Pavement ME analysis predicted a design life of 18 to 19 years based on VDOT bottom-up cracking design criteria (<6% cracking). However, VDOT has not calibrated pavement overlay design models for rubblized pavement sections.

## RECOMMENDATIONS

1. *VTRC should continue to monitor the performance of the rubblized pavement on SR 406 and report on its condition in approximately 3 and 5 years. Any lessons learned during this time should be submitted to VDOT's Hampton Roads District and VDOT's Materials Division for review.*

## IMPLEMENTATION AND BENEFITS

### Implementation

With regard to Recommendation 1, VTRC along with staff of VDOT's Hampton Roads District Materials Division will continue to monitor the performance of this section, resulting in a future technical assistance report.

### Benefits

The benefits of continued performance monitoring of this section include giving VDOT the opportunity to document the performance of a rubblized section within an ME framework. This is a rare opportunity since VDOT does not conduct this type of pavement rehabilitation often. This section can also be used for future calibration of ME design performance models.

## ACKNOWLEDGMENTS

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## APPENDIX

## DYNAMIC MODULUS TEST RESULTS AND PAVEMENT MANAGEMENT SYSTEM DATA

### Dynamic Modulus Test Results

**Table A1. Dynamic Modulus Values for SM 12.5 E Mixtures**

| | Dynamic Modulus (E*) | | | | | | |
|---|---|---|---|---|---|---|---|
| | **Temp.** | **25 Hz** | **10 Hz** | **5 Hz** | **1 Hz** | **0.5 Hz** | **0.1 Hz** |
| **SM 12.5E** | -10ºC | 3180602 | 3128483 | 3076964 | 2902822 | 2797954 | 2567865 |
| | 4ºC | 2663075 | 2457617 | 2274366 | 1772884 | 1539363 | 1023885 |
| | 20ºC | 1250592 | 972690 | 790417 | 477212 | 386520 | 250988 |
| | 38ºC | 346940 | 271382 | 230589 | 171401 | 155897 | 133068 |
| | 54.4ºC | 156075 | 141249 | 133161 | 121075 | 117787 | 112800 |

**Table A2. Dynamic Modulus Values for IM 19.0E Mixtures**

| | Dynamic Modulus (E*) | | | | | | |
|---|---|---|---|---|---|---|---|
| | **Temp.** | **25 Hz** | **10 Hz** | **5 Hz** | **1 Hz** | **0.5 Hz** | **0.1 Hz** |
| **IM 19.0E** | -10ºC | 2948840 | 2853793 | 2771068 | 2539173 | 2421137 | 2105084 |
| | 4ºC | 2377138 | 2199171 | 2052626 | 1680602 | 1511682 | 1120158 |
| | 20ºC | 1419378 | 1195939 | 1032224 | 688992 | 563265 | 332243 |
| | 38ºC | 577710 | 433019 | 342193 | 188957 | 144200 | 76075 |
| | 54.4ºC | 173539 | 121002 | 91817 | 48813 | 37612 | 21489 |

### Pavement Management System Data

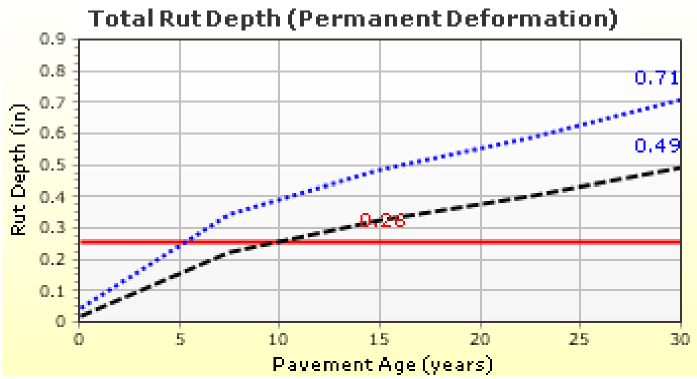

**Figure A1. Total Rut Depth**

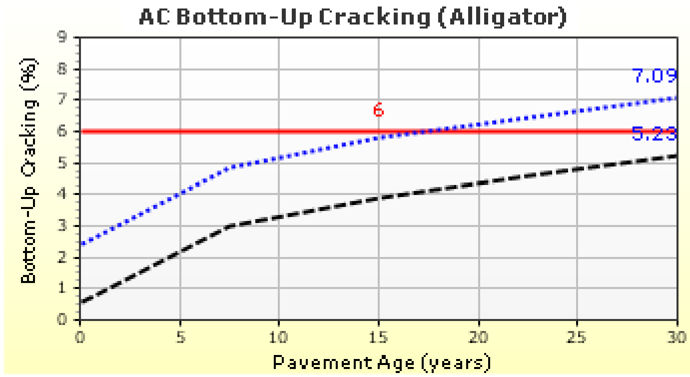


**Figure A2. AC Bottom-Up Cracking. AC =asphalt concrete.**

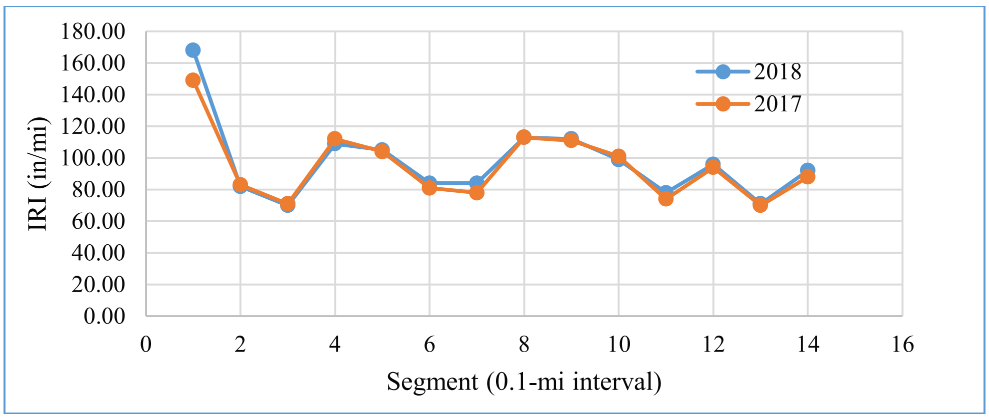


**Figure A3. IRI Data for SR 406 (Eastbound). IRI = International Roughness Index.**

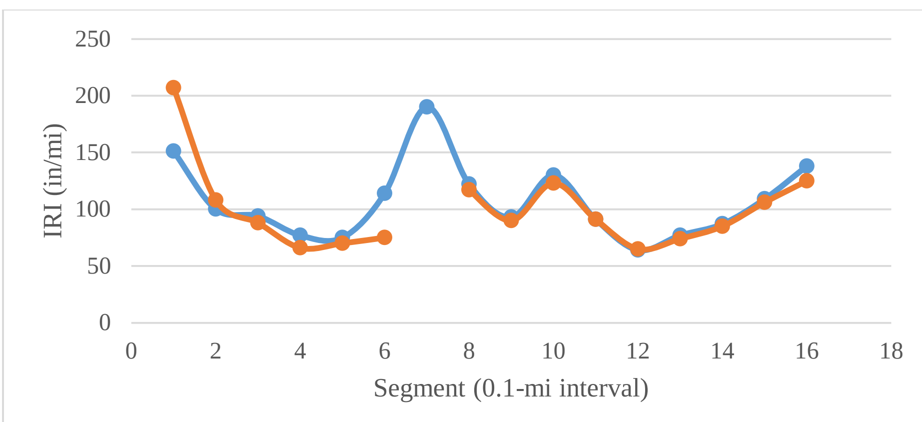


**Figure A4. IRI Data for SR 406 (Westbound). IRI = International Roughness Index.**

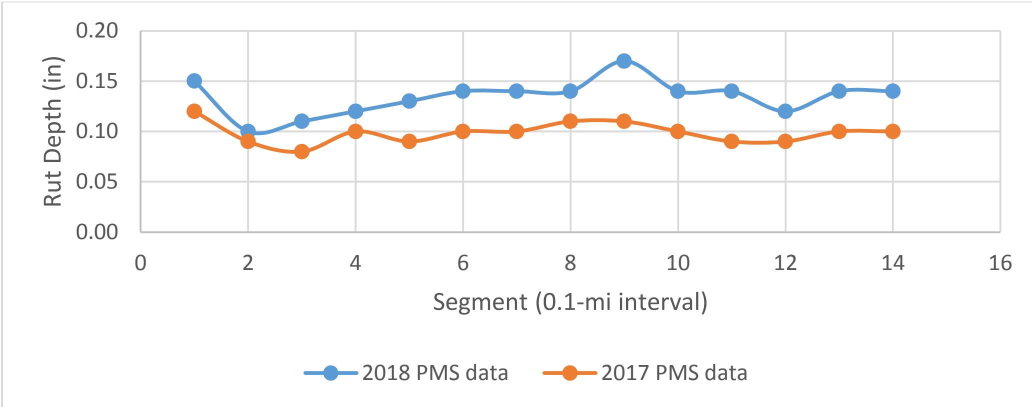

**Figure A5. Rut Data for SR 406 (Eastbound). PMS = Pavement Management System.**

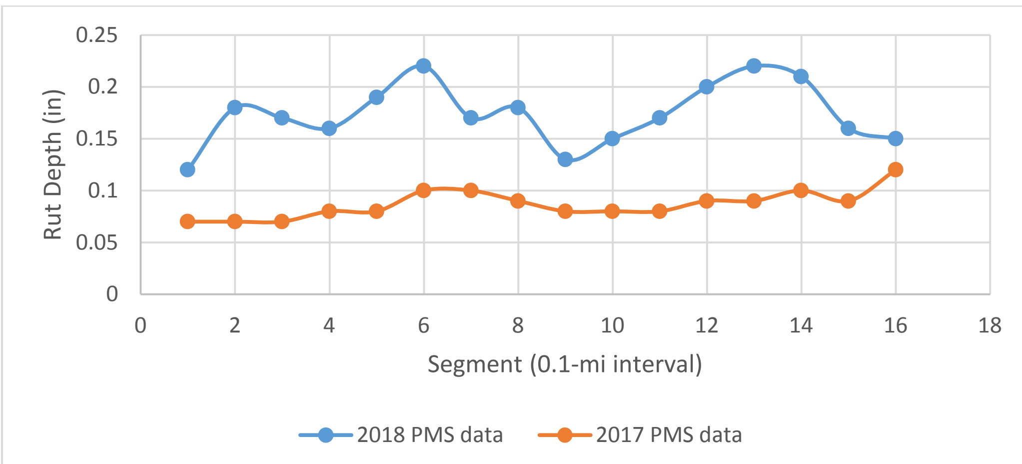

**Figure A6. Rut Data for SR 406 (Westbound). PMS = Pavement Management System.**