Mr. Lyndo Tippett  
Secretary  
North Carolina Department of Transportation  
1501 Mail Service Center  
Raleigh, North Carolina

Dear Secretary Tippett:

The North Carolina Department of Transportation (NCDOT) requested the Federal Highway Administration’s (FHWA) independent review and evaluation of the premature pavement failure that occurred on I-795 near Goldsboro. The FHWA review team, which included staff from our Office of Pavement Technology and our Pavement and Materials Technical Service Team, has completed its report that is enclosed for your information.

The FHWA is aware that the NCDOT has already begun repair of the high severity distressed areas in the pavement. We would like to meet further to discuss and agree upon additional repairs needed to obtain the service life of the original contract.

Sincerely,

John F. Sullivan, III, PE  
Division Administrator

Enclosure
North Carolina I-795/US 117 bypass (Goldsboro to Fremont)
Flexible Pavement and Hot Mix Asphalt Quality Review Report

December 24, 2008

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Andrew Mergenmeier, P.E., FHWA Resource Center
Executive Summary

This report documents an evaluation of premature pavement failure experienced on I-795 near Goldsboro, North Carolina. Originally constructed as US-117, the roadway is a new alignment, 4 lane divided highway with 12-ft lanes, and 10-ft outside and 4-ft inside paved shoulders (lane and shoulder pavement structures are same), approximately 18 miles long. The roadway was built utilizing separate contracts/projects for earthwork and paving. The pavement was opened to traffic in December 2005. By summer of 2007 (less than 2 years after opening to traffic), areas of high-severity distress including fatigue cracking and rutting were observed. NCDOT conducted a comprehensive evaluation of the pavement section and extensive testing to determine the cause of failure and assess the extent of the problem, ultimately to determine the best course of action for correcting the problem. NCDOT asked the FHWA to perform an independent review and evaluation of the pavement failure. The FHWA agreed to provide this service, which included a site visit of the project; review of construction documentation and information; and, a review of data, and materials samples and test results conducted as part of a forensic evaluation after the observation of the distress. As a part of the FHWA evaluation, comprehensive ground penetrating radar (GPR) testing was conducted, using state-of-the-art technology to assess the hot mix asphalt (HMA) layer interfaces bond conditions.

Based on the information and data available to date, the key findings of this evaluation include the following:

• The pavement design was according to NCDOT procedures. The inputs used in the pavement evaluation were reasonable and the structural design was predicted to be satisfactory for the assumed design traffic.

• Structurally, the pavement is reasonably uniform throughout the entire length of the project in both SB and NB directions. The foundation support (subgrade and aggregate base course) is uniformly good, and the pavement layer thicknesses are within expectations of the pavement design.

• The two hot-mix asphalt (HMA) surface layers have high air voids (up to 12 % or higher) throughout the project. The forensic HMA air voids are greater than expected in design. The high air voids are a contributing factor leading to the premature pavement distress.

• Both forensic core shear test results and GPR testing results show that the risk of HMA surface layer debonding is relatively high throughout the project.

• The high air voids in the surface HMA layers and relatively low shear strengths between the HMA surface layers indicated in the core testing results indicate a high risk of stripping and debonding between the existing HMA layers.

• In addition to the high severity cracking noted initially, low severity cracking is noted throughout the project in both SB and NB directions.

• The low severity cracking is a possible indication of structural deficiency. With the high air voids in the HMA and the relatively high risk of HMA layer debonding throughout the project, the risk of continued premature pavement failures may be high throughout the project. The low severity cracking already noted could rapidly develop into high-severity
fatigue cracking, or new cracks could develop with progressive failure, unless a structural overlay is provided.

- In light of the degree of damage in the existing pavement, the minimum recommended HMA structural overlay thickness is 2.5 in.; a 3-in. overlay is preferred. Prior to overlaying, the existing two HMA surface layers should be removed and replaced. The areas exhibiting high-severity cracking should include removal and replacement of the HMA full-depth.
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APPENDIX A: I-795 TRAFFIC DATA

APPENDIX B: PAVEMENT DESIGN

APPENDIX C: STATE EVALUATION REPORT

APPENDIX D: STATE EVALUATION REPORT (NB)

APPENDIX E: MTU INVESTIGATION OF ABC MATERIAL
**Introduction**

This report documents an evaluation of premature pavement failure experienced on I-795 near Goldsboro, North Carolina. Originally constructed as US-117, the roadway is a new alignment, 4 lane divided highway with 12-ft lanes, and 10-ft outside and 4-ft inside paved shoulders (lane and shoulder pavement structures are same), approximately 18 miles long. The roadway was built utilizing separate contracts/projects for earthwork and paving. The pavement was opened to traffic in December 2005. By summer of 2007 (less than 2 years after opening to traffic), areas of high-severity distress including fatigue cracking and rutting were observed. NC DOT has conducted a comprehensive evaluation of the pavement section and extensive testing to determine the cause of failure and assess the extent of the problem, ultimately to determine the best course of action for correcting the problem.

The FHWA conducted an independent analysis and evaluation of the I-795 premature pavement failures at the request of the NCDOT. The scope of this evaluation included a site visit of the project, meetings, discussions, and reviews of the project design, construction “as-built” information and data, forensic sampling and test results, and recent deflection and other materials layer testing. The scope of the evaluation also included use of FHWA’s contract ground penetrating radar (GPR) for testing and evaluation.

**Information/Data NC DOT provided the following information:**

- Project (test results from material tested before the project was accepted by NCDOT):
  - pavement information and structural design
  - materials acceptance test results
  - construction observations
- Forensic (test results from material in-place and tested after the project accepted):
  - traffic loading - weigh in motion (WIM)
  - individual material layer thicknesses
  - foundation support – dynamic cone penetrometer (DCP) and falling weight deflectometer (FWD)
  - debonding between HMA layers
  - HMA (in place density, asphalt content, gradation)
  - pavement surface distress
  - other factors

FHWA conducted the ground penetrating radar (GPR) data collection and analysis.
Project Information

Pavement Structural Design

The pavement was designed utilizing standard NCDOT methodology, which is an adaptation of 1972 AASHTO Interim Guide for Design of Pavement Structures. The July 15 and October 23, 2003 NC DOT memoranda from Judith Corley-Lay (Appendix B) document the final pavement designs for I-795. The design traffic was 8.5 million equivalent single axle loads (ESAL) over a 20-yr service life, based on 20,000 ADT with 10% trucks. The subgrade support value was 4.0, corresponding to a CBR of 10.8. A Regional Factor of 0.5 was used. As determined by NC DOT, the required structural number (SN) corresponding to the design inputs is about 4.3. The required SN is to be satisfied by the paving materials as follows:

\[
SN = a_1D_1 + a_2D_2 + a_3D_3
\]

Where

\( a_1, a_2, a_3 \) = layer coefficients of surface, base, and subbase layers

\( D_1, D_2, D_3 \) = thickness of surface, base, and subbase layers

The NCDOT uses the following layer coefficients for its pavement layers:

- HMA: 0.43
- Aggregate base: 0.14
- Lime treated soil: 1.0 for the whole 8-in. layer

The layer coefficients used by NC DOT are typical for the HMA and aggregate base layers. For the treated soil the layer coefficient suggested in the AASHTO 72 guide ranges from 0.15 to 0.30. The value used by NC DOT is equivalent to 0.125 (SN of 1.0 for the 8-in. layer).

A preliminary pavement design was estimated for 8 inches of HMA with 8 inches of aggregate base. The earthwork contracts resulted in prepared subgrade with this profile. The final pavement design resulted in 5.2 inches of HMA with 8 inches of aggregate base. The paving contract included 3 inches of additional aggregate base to the final pavement section, rather than regrading the subgrade to meet roadway profile grade. Inside and outside shoulders had same HMA thickness as mainline lanes. Longitudinal edge drains located at outside edge of shoulder.

The pavement section from beginning of project (mile mark [MM] 87.44, station 67+50) to MM 88.7 (station 87 + 00) consists of the following:

- Two layers of S9.5C HMA total depth of 70 mm (2.8 in)
- One layer of I19.0C HMA depth of 60 mm (2.4 in)
- One layer of B25.0C HMA depth of 80 mm (3.1 in)
- One lift of aggregate base course depth of 200 mm (7.9 in)

The SN provided by this pavement section is 4.68, which exceeds the SN required by the pavement design.
The pavement section from MM 88.7 (station 87 + 00) to end of project (MM 105.45, station 78+80) consists of the following:

- Two layers of S9.5C HMA total depth of 70 mm (2.8 in)
- One layer of I19.0C HMA depth of 60 mm (2.4 in)
- Two lifts of aggregate base course total depth of 280 mm (11 in)
- One lift of chemically stabilized soil (majority was lime, southern end had cement as a result of soil type) depth of 200 mm (7.1 in)

The SN provided by this pavement section is 4.78, which exceeds the SN required by the pavement design.

Based on above discussion, it is concluded that the design assumptions for the I-795 design are consistent with the guidelines provided in the 1972 AASHTO guide, and the pavement sections for this project satisfy the structural requirements determined using the NCDOT design procedure.

**Forensic Traffic Loading**

The NCDOT collected traffic and truck loadings on the roadway during February 2008 to estimate the number of ESAL that have been applied to the pavement structure. A 7-day vehicle class count collected by NCDOT in February 2008 and 7 day weight-in-motion (WIM) data collected in March 2008 are summarized in Appendix A. The data show that design traffic loading and actual traffic loading are similar. While the overall vehicle count is about half that assumed for design, the percentage of truck traffic is significantly higher (13.7% vs. 9% assumed in design). Although majority of 5-axle tractor-trailers were running either empty or partially loaded, significant portion (about 10%) were over the gross vehicle weight (GVE) limit.

**Forensic Individual Material Layer Thickness**

The individual material layer thicknesses of the aggregate base course and HMA were uniform and within expected ranges, as documented in the NC state reports (Appendices C and D).

**Forensic Foundation Support – DCP and FWD**

FWD testing was conducted at 500 ft interval over the entire project in both SB and NB directions. Below the HMA layers the pavement structure consists of an 11-in aggregate base (ABC) placed over 8 in of stabilized soil. The FWD testing results showed that both the ABC and the stabilized soil appear to be in reasonable conformance to design requirements. The FWD testing results showed that the foundation support is very good and uniform throughout the project.

Normalized deflections and backcalculated subgrade resilient modulus (MR) are shown in figures 1 and 2. Backcalculation was performed using Evercalc and using the procedure provided in AASHTO 93 Guide. In the backcalculation results, the ABC, stabilized soil, and subgrade layers were not distinguishable. An average MR of about 24,000 psi seems to be representative of the foundation support the HMA layers “see.”
Figure 1. Normalized deflections and foundation MR, SB I795.

Figure 2. Normalized deflections and foundation MR, NB I795.
The NC DOT also conducted DCP testing, which showed that all material underneath the HMA layers are in good condition, except for the top 4 in. of the ABC, which may or may not be an indication of poor compaction. During the DCP testing it was noted there was no evidence of high moisture content in the ABC (recent weather had been dry). Based on visual observations of the ABC in-place and test results performed on representative samples of ABC both during construction and in the forensic investigation, the NC DOT concluded that the ABC layers did not significantly contribute to the early pavement failure on I-795. The details of DCP testing results are provided in a NC DOT report (Appendix E). The FWD testing results also did not indicate any problems with foundation support, at least in the areas that are not severely deteriorated.

As-Built Project Materials Acceptance Test Results

During production contractor HMA mix and field density test (nuclear gauge, lot average > 92% Gmm [theoretical maximum specific gravity of HMA]) results were in reasonably close conformance with specifications. During production NC DOT HMA mix test results were in reasonably close conformance with specifications.

The NC DOT HMA Gmm verification test results are compared to contractor results on an individual test tolerance basis. Gmm is the reference (datum) against which HMA density is compared to determine acceptance. Incorrect Gmm values may result in inadequate or excessive, density in the field. The verification program should include an analysis to assess bias and/or non-comparable trends of NC DOT and contractor test results.

It is not expected that many of the NC DOT HMA Gmb (bulk specific gravity of compacted HMA) verification tests were performed on samples that the NC DOT ensured were randomly sampled and NC DOT took possession and storage of the sample through the testing process. NC DOT uses a modified AASHTO T166 test method to determine Gmb. Currently there is discussion that AASHTO T166 be limited to compacted HMA mixes that have 1% or less water absorption. Studies have indicated that in compacted HMA mixes with greater than 1% water absorption, AASHTO T166 may result in higher Gmb (test results that do not accurately characterize the material), a test result that indicates the percent density obtained is greater than actual in place density. Correct Gmm and Gmb are critical to understanding characteristics related to HMA pavement performance and justify a high level of effort to ensure correct.

NC DOT has a separate pay item for liquid asphalt cement. Pay is based on job mix formula.

Per mix design, both HMA surface courses were fine graded 9.5 mixes with PG 70-22 asphalt cement (Citgo). Both mixes utilized 20% natural sand (different sources). Coarse aggregate quarries had no clay/soil contamination. HMA was produced at two plants.

Project Construction Observations

NCDOT staff indicated that the surface course thicknesses used for this project are no longer specified on current contracts. The NC DOT minimum HMA lift thickness is now 1.5 in for 9.5-mm (0.375-in) nominal-maximum-aggregate-size (NMAS) mixes, which is four times the NMAS (4 x 9.5 mm = 38 mm [1.5 in]). For the I-795 project, the minimum HMA lift thickness was 3.7 times NMAS.
HMA construction sequence for the two surface courses layers: SBL paved first, paving started at inside shoulder/lane, then outside lane, then outside shoulder. Expected that aggregate/soil placed at pavement shoulder drop off to fore slope (pulling of fore slope to shoulders) before last HMA layer placed. Time between first layer of HMA 9.5 mm surface course and final layer of HMA 9.5 mm surface course was about a month, thus it is possible that the construction traffic and pulling of the shoulders could have contaminated the surface of the HMA before the last layer was placed.

Forensic In-Place Hot-Mix Asphalt (HMA) Properties

Although NC DOT QA tests conducted during construction did not indicate any problems with HMA densities, forensic cores showed high air voids in HMA surface layers (top and bottom HMA surface courses). Figures 3 through 6 show the in-place air void results determined by coring versus the air void results shown in the construction records. Figures 3 and 4 show the top two layers of surface course. Figures 5 and 6 represent the intermediate layer.

Table 1 shows a summary of statistical evaluation of the density testing results from project acceptance test results and forensic cores. The forensic cores were taken in close proximity to the contractor project density acceptance testing locations (correlation by NC DOT field Division – data is closest test data available to the core, always in same lane on same day of paving). The forensic cores were taken in 2008 and tested using the CoreLok method.

<table>
<thead>
<tr>
<th>HMA Layer</th>
<th>Mean density, %</th>
<th>Std. deviation, %</th>
<th>F-test of variances</th>
<th>t-test of means</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Project</td>
<td>Forensic</td>
<td>Project</td>
<td>Forensic</td>
</tr>
<tr>
<td>Surface, top</td>
<td>93.1</td>
<td>90.7</td>
<td>0.35</td>
<td>1.75</td>
</tr>
<tr>
<td>Surface, bottom</td>
<td>92.7</td>
<td>90.3</td>
<td>0.31</td>
<td>1.50</td>
</tr>
<tr>
<td>Intermediate</td>
<td>93.2</td>
<td>92.6</td>
<td>0.42</td>
<td>1.68</td>
</tr>
<tr>
<td>Surface, top</td>
<td>93.3</td>
<td>90.3</td>
<td>0.89</td>
<td>1.48</td>
</tr>
<tr>
<td>Surface, bottom</td>
<td>93.2</td>
<td>91.4</td>
<td>0.55</td>
<td>1.42</td>
</tr>
<tr>
<td>Intermediate</td>
<td>93.1</td>
<td>92.7</td>
<td>0.51</td>
<td>1.04</td>
</tr>
</tbody>
</table>

The statistical analysis of the density data shows the following:

- The two lifts of HMA surface course forensic testing air voids were higher than project testing results.
- The forensic HMA air voids were greater than expected in design.
- Both the results of analysis of variance (F-test results) and t-test of means indicate that the project acceptance and forensic cores are different populations, meaning different
material properties were measured. One possible reason for the difference may be the use of different testing methods. For core samples with high air voids, the method of testing is a significant variable. The CoreLok method is expected to provide more accurate and consistent results (WSDOT 2004) for core samples with high air voids.

Figure 3. SB I-795 air voids from forensic cores and construction QA testing for the surface HMA layers.
Figure 4. NB I-795 air voids from forensic cores and construction QA testing for the surface HMA layers.

Figure 5. SB I-795 air voids from forensic cores and construction QA testing for the intermediate HMA layer.
Figure 6. NB I-795 air voids from forensic cores and construction QA testing for the intermediate HMA layer.

It should be noted that the low HMA density in the surface courses are not confined to a single area but are located throughout the length of the project in both SB and NB sides, as shown in figures 20 and 21.

The HMA forensic asphalt content was in reasonable conformance with design requirements. A random few of the HMA forensic aggregate gradations were on the coarse side of design (1.16 mm sieve [No.16]) has greater than or equal to 7% less material passing the sieve than the job mix formula).

Forensic Debonding Between HMA Layers

Observation of cores and slabs at the NCDOT Central Laboratory showed debonding between the surface layers. The NCDOT also performed shear strength testing on intact cores (Florida shear strength test method) to estimate bonding capability between layers. The core testing results are summarized in the NC DOT reports (Appendix C and D). The core shear strength test results are shown in Figures 7 and 8, along with visual distress. A shear strength value of 0 indicates debonding. A significant portion of the cores exhibited less than desirable shear strength. As shown in figure 9, only about 40% of the cores tested had shear strength of 100 psi or higher. More important, 25 to 30% of the cores (depending on direction) exhibited shear strength less than 60 psi, which is considered minimum needed to hold the pavement layers together.

The FHWA used its GPR to quantify the risk of debonding between pavement layers. This can be evaluated using the step-frequency ground-penetrating radar (SF-GPR) results. The GPR
testing results are shown in figures 10 and 11, along with the shear strength data. The GPR results should be considered supporting information, rather than something more conclusive, because the evaluation techniques utilized are still developing technology, and further validation is needed. On the other hand, after the follow-up testing, the GPR analysis contractor was very confident that the results indicated are reliable, and the results show a relatively high risk of debonding throughout the project.

The GPR results are for the critical wheelpath, the inside wheelpath of the outer lane on this project. At the intermediate speed, SF-GPR provides 1-ft by 1-ft resolution. Each area is rated for possible debonding. The rating ranges from 0 to 10, depending on the intensity of the response signal, but only a rating higher than 7 was considered possible indication of debonding. An average rating of the 6 areas in the wheelpath (6 1-ft by 1-ft areas per wheelpath) are shown in figures 10 and 11. A rating of 10 indicates that a high debonding potential was detected in all 6 areas. A rating of 5 indicates that possible debonding in half of the areas. We might consider a rating of 3 or higher an indicator of the extent of debonding that can affect pavement response (i.e., high deflections and strains).

Figures 10 and 11 show that the GPR results are somewhat correlated to shear strength. In general, the GPR testing results indicate a relatively high risk of debonding throughout the project.

![Graph showing shear strength and cracking](image)

Figure 7. SB I-795 pavement condition and HMA surface layer air-voids.
Figure 8. NB I-795 pavement condition and HMA surface layer air-voids.

Figure 9. Distribution of shear strength.
Figure 10. SB I-795 GPR debonding evaluation results and forensic-core shear strength.

Figure 11. NB I-795 GPR debonding evaluation results and forensic-core shear strength.
Pavement Surface Distress

Initially, the distress appeared on the SB side on I-795. The NC DOT report prepared in July 25, 2007 (Appendix C) reports that majority of the distresses are located between MM 92 and 89. This section contains the only area that has developed high-severity distresses on the project. Subsequently, additional distresses were identified, including some on the NB side, as detailed in the NC DOT report dated September 19, 2007 (Appendix D). The NC DOT performed a detailed visual distress survey of the project in April 2008. Although no additional high-severity distresses were found, low-severity fatigue cracking was found in several areas, both in SB and NB.

SF-GPR data were also analyzed to identify distresses, but as mentioned earlier (p9, “Forensic Debonding Between HMA Layers”) the techniques used in this analysis are developing technology, which require further validation, and the GPR results should be viewed as supporting information. The distresses identified from the April 2008 visual survey and GPR-identified distresses are shown in figures 12 and 13. The GPR cracking results shown in these figures are simple yes/no indicators of possible presence of cracking. These results suggest that cracking may be present throughout the project.
Other Relevant Information

The following information was obtained during the February 15, 2008 meeting with NC DOT at the Materials Laboratory and I-795 site visit:

- NC DOT had performed some trenching and coring. Several of the larger cores (12 in diameter) the top 1 ¼ in. HMA layer was debonded from the remainder of the HMA and this was the case in several of the trenched samples (2 ½ ft x 2 ½ ft). An one inch diameter clay ball was observed in a 6 in. core. It is expected the trench samples and larger cores came from the most distressed pavement sections. Some of the 6” cores from the moderately distressed areas had what appeared to be bottom up cracking.

- NC DOT personnel stated during a field review within the past week that moisture was coming up along the centerline longitudinal construction joint on SB I-795.

- From the cores at the Materials Laboratory and pavement in the field, evidence of clay balls were present.

- Mile Point 92.3 SB, the more questionable mix (NC DOT personnel indicated that it appears the following mix was one of more uncertainty than other mixes used: RS 9.5 C mix, # MD 04-171, JMF asphalt content of 5.1%, 100 gyration, 56% passing #8 sieve): observed clay balls, small pin hole and ¼” pop outs prevalent on surface.

- Figures 14-19 are photographs of the MD 04-171 HMA surface mix.
Figure 14: Photo I-795 MP 92.3 SBL pinholes and open surface texture (toothpick)

Figure 15: Photo I-795 MP 92.3 SBL clay ball (toothpick), open surface texture, pin holes.
Figure 16: Photo I-795 MP 91.5 SBL highly distressed area – patches are for cores and trenching.

Figure 17: Photo I-795 MP 91.5 SBL highly distressed area – surface cracking.
Figure 18:  Photo I-795 MP 91 SBL highly distressed area – fatigue cracking and rutting in outside lane inside wheel path (fill height 7-ft; water in ditch).

Figure 19:  Photo I-795 MP 90.5 SBL highly distressed area – fatigue cracking and rutting in outside lane inside wheel path, and spalling evident.
Conclusions

It is expected that a combination of factors have lead to the premature distresses observed on the I-795 pavement. Significant factors include the following:

- Throughout the project, the air voids of the two layers of HMA surface courses are greater than expected in design
- Debonding between HMA layers is associated with medium- and high-severity pavement surface distresses. The exact cause of debonding is difficult to identify, but high air voids in the HMA surface courses (with consequent high permeability) were likely a significant contributing factor.
- Low-severity pavement surface distresses occurred in some of the well bonded HMA layer areas. This is an indication of structural deficiency (even for the intact section), possibly due to poor quality HMA surface courses.
- Presence of heavy, over-weight trucks may have played a role. The design traffic for this pavement is not very high, and the WIM data, as well as the weight enforcement data did not show unusually high number of heavy axles; however, on a relatively light pavement structure, even a few heavy trucks can initiate cracking, especially if the quality of HMA material is marginal.

Based on the information reviewed, the structural deficiencies that lead to premature cracking on I-795 is a general, project-wide problem affecting both SB and NB. As shown in figures 20 and 21, the high air voids in the surface HMA layers and low shear strength at the layer interface are project-wide. The SF-GPR results (figures 10 and 11 for debonding; figures 12 and 13 for cracking) also indicate the uniformity of pavement condition throughout the project. The results of material property analysis and observations indicate that the weaknesses that resulted in premature pavement distress are present throughout the project.

The following observations point to the urgency of the problem:

- Low-severity pavement surface distresses occurred in some of the well bonded HMA layer areas. This is likely an indication of the initiation of fatigue cracking, which means that additional cracks may develop at an accelerating rate, unless a structural enhancement is made.
- Some areas with debonding between HMA layers have not yet developed pavement surface distress. Those areas are likely to develop distresses in the near future and rapidly deteriorate.
- SF-GPR results indicate possible presence of cracking that are not yet visible on the surface. These distresses may be a result of further material deterioration due to the high HMA air voids, aggravated by the repeated application of heavy axle loads.
Figure 20. SB I-795 pavement condition and HMA surface layer air-voids.

Figure 21. NB I-795 pavement condition and HMA surface layer air-voids.
A fatigue analysis conducted using the FWD backcalculated material properties and Asphalt Institute (AI) fatigue model showed a possibility of complete pavement failure (extensive, high-severity cracking and rutting throughout the project) within the next 5 years or less, unless a structural enhancement is made. The calculated fatigue damage to date (2 years of service) is 0.72. This damage number is only meaningful when calibrated to the field condition. The calculated fatigue damage of 0.72 for I-795 corresponds to the initiation of fatigue cracking in the field (which is the observed pavement condition). Figure 22 shows that the fatigue damage at failure (e.g., 10% cracking of the total lane area) is about 3 times the damage at the initiation of cracking. In figure 22, the crack initiated at fatigue damage of about 0.4 and the failure level (fatigue damage at 10% cracking) is about 1.2. For I-795, a fatigue damage of 2.2 may be considered the failure level (3 times 0.72, the calculated damage after 2 years of service for I-795). The failure level of fatigue damage would be exceeded in 4 years, unless something is done.

Other Factors

Other factors not fully assessed include the following:

- Clay in HMA, pin holes and/or pop outs (prevalent)
- Tack coat/HMA surface before next HMA lift
- Stripping of HMA
- History of similar pavement designs
- HMA performance history from HMA plants

Figure 22. Example calibration curve for fatigue cracking.
**Recommended Rehabilitation Strategy**

To prevent rapid pavement deterioration a structural overlay is needed. A fatigue analysis showed that a 3-in structural overlay is needed to achieve the desired performance life of 20 years, assuming the pavement will be overlaid at year 12. Good bond between HMA layers is essential to performance of the overlaid pavement. To ensure good bonding, all poor-quality material from the existing HMA needs to be removed. In the case of I-795, this means removing and replacing both existing HMA surface layers (top and bottom HMA surface layers) prior to overlaying with HMA.

In addition to the issue of relatively low shear strength at the existing HMA layer boundaries and the indication of possible debonding in SF-GPR results (shown in figures 10 and 11), there is the issue of high air voids, with accompanying high permeability. Because the underlying intermediate HMA layer has lower air voids (expected to be impermeable) than the HMA surface layers, there is a high likelihood of trapping water between the HMA layers, if the existing surface layers were left in place. Water trapped in between HMA layers can cause/facilitate stripping and lead to premature failure of the pavement structure. Therefore, placing an HMA overlay directly over the existing HMA pavement surface (without removing all high air-void HMA material) is not recommended.

A modest reduction in HMA overlay thickness may be possible, if the planned overlay is moved up to year 10 (rather than year 12), especially if a thicker overlay is placed at the time of planned overlay (e.g., mill 2 in. and place 3 in. overlay). The risk of failure increases with decreasing overlay thickness. Based on the analysis results, a HMA structural overlay thickness less than 2.5 in. is not recommended. A 3-in. structural overlay thickness is desirable for a higher reliability, but if a structural improvement of 1 in. additional HMA can be planned at year 10 (i.e., mill 2 in. and replace 3 in.), a 2.5-in. overlay may be an acceptable treatment at this time. The sensitivity of fatigue damage to the HMA overlay thickness is summarized in table 2.

<table>
<thead>
<tr>
<th>Rehabilitation Description</th>
<th>Damage</th>
<th>Mill 2 in. and Overlay @ yr 10 (Total Damage @ yr 20)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Yr 10</td>
<td>Yr 20</td>
</tr>
<tr>
<td>Mill &amp; replace top two surface courses then, overlay 2.0 in. HMA</td>
<td>Future Damage</td>
<td>1.12</td>
</tr>
<tr>
<td></td>
<td>Total Damage</td>
<td>1.85</td>
</tr>
<tr>
<td>Mill &amp; replace top two surface courses then, overlay 2.5 in. HMA</td>
<td>Future Damage</td>
<td>0.86</td>
</tr>
<tr>
<td></td>
<td>Total Damage</td>
<td>1.59</td>
</tr>
<tr>
<td>Mill &amp; replace top two surface courses then, overlay 3.0 in. HMA</td>
<td>Future Damage</td>
<td>0.66</td>
</tr>
<tr>
<td></td>
<td>Total Damage</td>
<td>1.39</td>
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<tr>
<td>Mill &amp; replace top two surface courses then, overlay 3.5 in. HMA</td>
<td>Future Damage</td>
<td>0.52</td>
</tr>
<tr>
<td></td>
<td>Total Damage</td>
<td>1.24</td>
</tr>
</tbody>
</table>

Table 2. Sensitivity of fatigue damage to overlay thickness.
In addition to milling and replacing the two surface HMA layers prior to overlaying, all areas exhibiting high-severity cracking (e.g., MP 90 to MP 92, approximately, on the SB side) should include removal and replacement of the HMA full-depth.

Summary

The key findings and recommendations of this evaluation are as follows:

- The structural weakness that resulted in premature cracking on I-795 is not a localized problem. The conditions are relatively uniform throughout the project, and the risk of additional deterioration is high, unless a corrective action is taken.
- Throughout the project, the two HMA surface course layers have air voids greater than expected in design. The high air voids are a contributing factor leading to the premature pavement distress.
- Shear strength and GPR test results indicate potential of debonding throughout the project.
- The NC DOT visual distress assessments have indicated pavement distress deterioration rates greater than similar pavements.
- The removal and replacement of the existing HMA surface layers is highly recommended to avoid premature failure of the overlaid pavement. Besides the high potential for debonding, the risk of trapping water between the HMA layers is high, resulting in stripping and rapid failure of the pavement structure.
- In light of the degree of damage in the existing pavement, the minimum recommended HMA structural overlay thickness is 2.5 in.; a 3-in. overlay is preferred. Prior to overlaying, the existing two HMA surface layers should be removed and replaced. The areas exhibiting high-severity cracking should include removal and replacement of the HMA full-depth.
- It is recommended that the NC DOT in-place HMA field density program be reviewed. For in-place HMA field density determination, an adequate HMA Gmm and Gmb verification testing program needs to be provided. The program needs to include NC DOT ensuring the samples are randomly taken and NC DOT taking possession and storage of the samples through the testing process. Alternative Gmb test methods should be explored that adequately characterize the HMA quality.

References


WSDOT 2004. tech notes, “The CoreLok® Device.” Materials Laboratory, Washington State Department of Transportation, Olympia, WA.
Appendix A: I-795 Traffic Data
I-795 Traffic Data

- Annual Average Daily Traffic Volumes
- 7 Day Vehicle Class Count Collected in February
- 7 Days of Weigh In Motion Data Collected in March

I-795 AADT Data

<table>
<thead>
<tr>
<th>Station</th>
<th>2006</th>
<th>2007</th>
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</thead>
<tbody>
<tr>
<td>3497</td>
<td>6,600</td>
<td>10,000</td>
</tr>
<tr>
<td>3499</td>
<td>6,400</td>
<td>NA</td>
</tr>
<tr>
<td>3500</td>
<td>6,700</td>
<td>NA</td>
</tr>
<tr>
<td>3501</td>
<td>6,900</td>
<td>NA</td>
</tr>
</tbody>
</table>

- Volumes well below capacity
- Free Flow conditions
I-795 Vehicle Class Data

- Location: I-795 North of US 70 in Wayne County
- 7 consecutive days of class data
- Data collected by lane in 4 lanes
- Peek ADR 1000 classifier with 2 road tubes at 16’ spacings
- FHWA 13 Vehicle Class Scheme used
- Classified by the number of axles and axle spacings
- Class 1 – 3 are passenger vehicles
- Class 4 – 7 are single unit trucks (Duals)
- Class 8 – 13 are multi unit trucks (TTST)

About 1500 Trucks/Day on Weekdays and 400 Trucks/Day on Weekends
I-795 Vehicle Class Data

2 Axle SU (Class 5) and 5 Axle ST (Class 9) are the dominant types

I-795 Vehicle Class Data

About 89% of all trucks use the shoulder lanes
I-795 Vehicle Class Data

I-795 Vehicle Class Traffic Statistics

Weekly ADT = 8,649
Duals = 404 (4.7%)
TTST = 781 (9.0%)
50% Directional Split on Trucks
89% of Directional Trucks in Design Lane

In general, a typical volume of single unit trucks and a moderate volume of multi unit trucks for an interstate.

I-795 WIM Data

- Portable WIM considered but determined to be unreliable
- Temporary continuous WIM installed
- Location: I-795 North of US 70 in Wayne County
- 7 consecutive days of class and truck weight data collected
- WIM installed in shoulder lanes only (89% of all trucks)
- Peek ADR 3000 WIM - piezo/loop/piezo array - 2 BL Class 1 Piezos at 12’ spacing - 1 inductive loop - each lane
- FHWA 13 Vehicle Class Scheme used
- Same algorithm as portable class
- Calibrated using Class 9 loaded to 95% legal weight limits
- SB lane calibrated normally
- NB lane pavement appears to deflect more than typical
- NB lane calibrated but WIM equipment is near limits
Class 9 was evaluated separately because they apply most of the loads.

Variation by direction may indicate dominant truck combinations – local trips.
I-795 WIM Data

Most trucks in all the other classes have low GVW weights

I-795 WIM Data

Interstate Vehicle Class Distribution Comparison
7 Day Vehicle Class Counts - All Lanes

All 3 dominated by Class 5 and 9 – I-95 significantly higher Class 9
I-795 WIM Data

Interstate Class 9 GVW Frequency Distribution Comparison
7 Day Total - Shoulder Lanes Only

I-795 has lower peaks than I-40 – Both are significantly lower than I-95

I-795 WIM Data

Interstate Class 9 GVW Normalized Frequency Distribution Comparison
7 Day Total - Shoulder Lanes Only

I-795 has a similar pattern to I-40 – High unloaded and lower loaded (local trips)
I-95 has distinctly different pattern – Low unloaded, higher partial loads, and high loaded (through trips)
I-795 WIM Data

- I-795 has few trucks with heavy loads
- Truck loading indicates primarily local travel
- On an average day in a shoulder lane there are about:
  285 trucks classified as a 5 Axle Tractor Trailer
    3 are misclassified
    112 have empty trailers
    75 have partially loaded trailers
    73 are loaded at or near the GVW legal limit
    22 are loaded over the GVW limit
  250 trucks of all other types
    124 are 2 Axle Single Unit trucks
    4 are loaded over the GVW limit
### DAILY COUNT TOTALS

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<tr>
<th>Hours</th>
<th>Day</th>
<th>Date</th>
<th>Direction</th>
<th>Lane</th>
<th>Class 1</th>
<th>Cycle</th>
<th>Class 2</th>
<th>Cycle</th>
<th>Class 3</th>
<th>Cycle</th>
<th>Class 4</th>
<th>Cycle</th>
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<th>Cycle</th>
<th>Class 12</th>
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<td>24</td>
<td>1</td>
<td>2/6/2008</td>
<td>North Shoulder</td>
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<td>2,251</td>
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<td>32</td>
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<td>14</td>
<td>4</td>
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<td>24</td>
<td>1</td>
<td>2/6/2008</td>
<td>North Median</td>
<td>7</td>
<td>600</td>
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<td>3</td>
<td>31</td>
<td>3</td>
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<td>24</td>
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<td>South Median</td>
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<td>4</td>
<td>1</td>
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</tr>
<tr>
<td>24</td>
<td>1</td>
<td>2/6/2008</td>
<td>West Median</td>
<td>6</td>
<td>2,234</td>
<td>635</td>
<td>42</td>
<td>136</td>
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<td>1</td>
<td>66</td>
<td>364</td>
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<td></td>
</tr>
</tbody>
</table>

#### Weekly ADT Counts

- **PV (Passenger Vehicles)**
  - 19,694
- **SU (Single Unit Trucks)**
  - 4,461
- **MU (Multi Unit Trucks)**
  - 1,491

#### Percentages

- **Week ADT**
  - PV: 86.3%
  - SU: 4.7%
  - MU: 9.0%

- **Week PHP**
  - 28/08: 107.0%
  - 21/11: 52.0%
  - 11/08: 91.6%

#### Vehicle Class Summary

<table>
<thead>
<tr>
<th></th>
<th></th>
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<th></th>
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<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>PV</td>
<td>7,404</td>
<td>4,494</td>
<td>701</td>
<td>4,140</td>
<td>717</td>
<td>8,540</td>
<td>AM</td>
<td>21/11/2008</td>
<td>700</td>
<td>526</td>
<td>9.6%</td>
<td>53%</td>
<td>South</td>
</tr>
<tr>
<td>SU (DUALS)</td>
<td>4,461</td>
<td>1,491</td>
<td>1,501</td>
<td>1,491</td>
<td>1,151</td>
<td>1,115</td>
<td>626</td>
<td>11</td>
<td>21</td>
<td>5</td>
<td>1</td>
<td>8,649</td>
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<tr>
<td>MU (TTST)</td>
<td>1,491</td>
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<td>2</td>
<td>1</td>
<td>2</td>
<td>4</td>
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<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1,365</td>
<td></td>
</tr>
</tbody>
</table>

#### FHWA Class Summary

- **Week ADT**
  - PV: 19,694
  - SU: 4,461
  - MU: 1,491

#### NCDOT Class Summary

- **Week ADT**
  - PV: 7,404
  - SU: 4,494
  - MU: 701

#### Peak Hour Summary

- **Week PHP**
  - 28/08: 1700
  - 21/11: 526
  - 11/08: 923

#### County/Dir

- **Peebles**
  - 1000
- **Goldboro**
  - 1000

---

**Traffic Survey Group - NC Dept of Transportation**

**Vehicle Class Count Summary**

- **I-795 North of US 70**

---

**Station**: Project 100 Station 0001

**Start Date**: February 06, 2008

**County**: Wayne

**City/Town**: Goldboro
Appendix B: Pavement Design
MEMO TO: Roger Thomas, PE
FROM: Judith Corley-Lay, Ph.D., PE
DATE: October 3, 2003
SUBJECT: REVISED FINAL PAVEMENT DESIGN

R-1030E, 8.1330502, F.A. Project: NHF-DPR-0073(002)
US 117 Bypass from north of the US 70 Interchange at Goldsboro
to south of SR 1342 (Memorial Church Rd.)
Wayne County – Division 4

For addressing the difference between the pavement thickness assumed for the grading project and the thicknesses in the final pavement design, the main line pavement designs for the above project are revised as follows:

1) From Sta. 67+50 to Sta. 87+00 ±

   70 mm S9.5C
   60 mm I19.0C
   80mm B25.0C
   200mm ABC

2) From Sta. 87+00 ± to Sta. 21+620

   70 mm S9.5C
   60 mm I19.0C
   280mm ABC
   Cement/Lime Stabilization

All other pavement designs remained the same as described in the final design letter dated July 15, 2003.
Shoulder drains are recommended for this project.

If any additional information is needed, please contact Dr. Clark Morrison, PE, State Pavement Design Engineer at 250-4094.

JCL/dcc

cc:               Mr. C. W. Leggett, PE  Mr. N. W. Wainaina, PE
Mr. A. W. Roper , PE  Mr. J. S. Bourne, PE
Mr. J. A. Bennett, PE  Mr. Jim Phillips, PE - FHWA
Mr. S. D. DeWitt, PE
MEMO TO: Roger Thomas, PE
FROM: Judith Corley-Lay, Ph.D., PE
DATE: July 15, 2003
SUBJECT: FINAL PAVEMENT DESIGN
R-1030E, 8.1330502, F.A. Project: NHF-DPR-0073(002) 343631.1
US 117 Bypass from north of the US 70 Interchange at Goldsboro
to south of SR 1342 (Memorial Church Rd.)
Wayne County – Division 4

The main line pavement designs for the above project are as follows:

1) From Sta. 67+50 to Sta. 87+00 ±

<table>
<thead>
<tr>
<th>Material</th>
<th>Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>70 mm S9.5C</td>
<td>410 mm</td>
</tr>
<tr>
<td>60 mm I19.0C</td>
<td></td>
</tr>
<tr>
<td>80mm B25.0C</td>
<td></td>
</tr>
<tr>
<td>200mm ABC</td>
<td></td>
</tr>
</tbody>
</table>

Total Pavement Depth: 410 mm
Structural Number: 4.27
Relative Cost Factor: 1.065
(Initial Construction)

2) From Sta. 87+00 ± to Sta. 21+620

<table>
<thead>
<tr>
<th>Material</th>
<th>Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>70 mm S9.5C</td>
<td>330 mm</td>
</tr>
<tr>
<td>60 mm I19.0C</td>
<td></td>
</tr>
<tr>
<td>200mm ABC</td>
<td></td>
</tr>
</tbody>
</table>

Cement/Lime Stabilization

Total Pavement Depth: 330 mm
Structural Number: 4.31
Relative Cost Factor: 1.00
(Initial Construction)
The pavement designs for all other lines are as follows:

<table>
<thead>
<tr>
<th>LINE</th>
<th>Surface</th>
<th>Intermediate</th>
<th>ABC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ramp A @ YREV (SR 1002)</td>
<td>70mm S9.5B</td>
<td>60mm II9.0B</td>
<td>150mm</td>
</tr>
<tr>
<td>Ramp B @ YREV (SR 1002)</td>
<td>70mm S9.5B</td>
<td>60mm II9.0B</td>
<td>150mm</td>
</tr>
<tr>
<td>Ramp C @ YREV (SR 1002)</td>
<td>70mm S9.5B</td>
<td>60mm II9.0B</td>
<td>150mm</td>
</tr>
<tr>
<td>Ramp D @ YREV (SR 1002)</td>
<td>70mm S9.5B</td>
<td>60mm II9.0B</td>
<td>150mm</td>
</tr>
</tbody>
</table>

Shoulder drains are recommended for this project. Detail for construction of the shoulder drains is attached and should be included in the typical section sheets. Also, the locations of shoulder drain installations for the project are attached and should be included in the summary sheets.

If any additional information is needed, please contact Dr. Clark Morrison, PE, State Pavement Design Engineer at 250-4094.

JCL/dcc

cc:  Mr. C. W. Leggett, PE  
Mr. A. W. Roper, PE  
Mr. J. A. Bennett, PE  
Mr. S. D. DeWitt, PE  
Mr. N. W. Wainaina, PE  
Mr. J. S. Bourne, PE  
Mr. Jim Phillips, PE - FHWA
Appendix C: State Evaluation Report
July 25, 2007

Memorandum To: Richard E. Greene, Jr., PE
Division Engineer, Division 4

From: Clark Morrison, Ph.D., PE
State Pavement Design Engineer

Subject: Pavement Evaluation
US 117 southbound lanes from SR 1342 (Memorial Church Rd.) to US 70
Wayne County, Division 4

The Pavement Analysis/Design Group has evaluated the subject section of roadway at the request of Mr. Mike McKeel, Resident Engineer in Smithfield and Wendi Johnson, Division Construction Engineer, Division 4. This section of roadway has been open to traffic for less than two years and already areas of high severity fatigue cracking with low to moderate severity rutting are showing up. The project runs from Mile Marker (MM) 96.8 to MM 87.4. The vast majority of the distressed areas are located between MM 92.0 and MM 89.0 in the southbound, outside lane only. Trackless tack was used on this project in the inside lane from the bridge over SR 1316 (MM 90.0) to the southern limit of the project. The inside lane is not visibly experiencing problems.

Tests conducted on the roadway include Falling Weight Deflectometer (FWD), coring and Dynamic Cone Penetrometer (DCP). Initially, the Pavement Management Unit (PMU) and the Geotechnical Unit thoroughly tested a heavily distressed area in the southbound, outside lane just north of MM 90.5 on May 2, 2007. Also on this date, the inside wheelpath of the southbound lane from MM 91.8 to MM 90.7 was tested since there was already a lane closure in place. It was later determined that the entire southbound outside lane needed to be tested. The PMU tested the center of the southbound outside lane from MM 96.8 to MM 87.4 on May 22, 2007. Then, to find out if both southbound lanes were faulty, the center of the southbound inside lane was tested by PMU on June 19, 2007 in the same locations as the center of the outside lane. This report summarizes our findings from these three days of testing and then discusses possible rehabilitation procedures.

Overview of Facility and Pavement Conditions
US 117 is a 4-lane, divided facility with 12-foot lanes, 10-foot outside paved shoulders, 4-foot inside paved shoulders with milled rumble strips. On site traffic counts provided by the Division yielded the following traffic data: 6,654 ADT (year 2007) with 6% duals and 18% TTSTs. The traffic forecast data used for the pavement design of this project was as follows: 23,100 ADT (year 2004), 40,800 ADT (year 2024), 4% duals and 5% TTSTs. The traffic count data yielded 5.93 million Equivalent Single Axle Loads (ESALs) over a 20-year design period while the traffic forecast data yielded 8.54 million ESALs over a 20-year design period. Deflection analyses of this project were performed using the traffic count data for a 10-year pavement design period.
Visual inspection of US 117 indicated that the vast majority of this section shows very little distresses and is in good riding condition. However, there are many short areas of moderate to high severity fatigue cracking and low to moderate severity rutting located between MM 92.0 and MM 89.0 in the outside lane only. About 90% of these areas are in the inside wheel path (IWP) and the remainder are located in the outside wheel path (OWP).

**Testing of heavily distressed pavement in the southbound, outside lane north of MM 90.5**

**FWD**
This short section of distressed roadway was tested in the hope of discovering an obvious reason for the failures. FWD testing was performed in the IWP and the OWP at ten locations. The calculated deflection limit for the project is 11.59 mils using a 10-year design life.

**IWP**
Temperature and load corrected deflections ranged from 25.84 mils to 9.85 mils. The average deflection was 17.55 mils. The FWD analysis indicated the pavement is currently structurally deficient at nine of the ten (90%) tested locations. Our analysis indicates that a four-inch overlay would address the structural deficiencies for this section at 80% of the tested locations. (See attached FWD charts)

**OWP**
Temperature and load corrected deflections ranged from 15.88 mils to 9.98 mils. The average deflection was 12.60 mils. The FWD analysis indicated the pavement is currently structurally deficient at eight of the ten (80%) tested locations. Our analysis indicates that a two-inch overlay is needed to address the structural deficiencies at 80% of the tested locations. (See attached FWD charts)

**Coring**
Six cores were taken in the heavily distressed area. Three cores were taken from the OWP and three cores were taken from the IWP. Aggregate Base Course (ABC) was underneath all cores. The average thickness of the cores was 5.1 inches. The design pavement thickness was 5.25 inches. The table below summarizes the location, structure and condition of the cores.

<table>
<thead>
<tr>
<th>FWD Location #</th>
<th>Location (MM/Wheelpath)</th>
<th>Structure</th>
<th>Total Thickness</th>
<th>Distresses</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>90.477 (IWP)</td>
<td>1.25” Surface, 1.5” Surface, 2.25” Intermediate</td>
<td>5.0”</td>
<td>Top surface layer delaminated from the rest of the core</td>
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<td>7</td>
<td>90.485 (IWP)</td>
<td>1.25” Surface, 1.25” Surface, 2.5” Intermediate</td>
<td>5.0”</td>
<td>Top surface layer delaminated from the rest of the core</td>
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<tr>
<td>9</td>
<td>90.492 (IWP)</td>
<td>1.25” Surface, 1.5” Surface, 2.5” Intermediate</td>
<td>5.25”</td>
<td>Core was in good condition</td>
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<tr>
<td>12</td>
<td>90.4659 (OWP)</td>
<td>1.25” Surface, 2.0” Surface, 2.0” Intermediate</td>
<td>5.25”</td>
<td>Small voids were present in the intermediate layer</td>
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<tr>
<td>15</td>
<td>90.4773 (OWP)</td>
<td>1.25” Surface, 1.75” Surface, 2.0” Intermediate</td>
<td>5.0”</td>
<td>Top surface layer delaminated from the rest of the core</td>
</tr>
<tr>
<td>18</td>
<td>90.4886 (OWP)</td>
<td>1.25” Surface, 1.75” Surface, 2.125” Intermediate</td>
<td>5.125”</td>
<td>Small voids were present in the top surface layer and the intermediate layer</td>
</tr>
</tbody>
</table>

**Subgrade**
DCP tests were conducted at all six coring locations. DCP tests allow us to calculate incremental California Bearing Ratios (CBR) vs. penetration depths. A CBR value of 100 or greater is
considered adequate for ABC, with CBR values of greater than 150 indicating the presence of excellent ABC. A CBR value of 25 or greater is considered adequate for stabilized subgrade and CBR values of 10 or greater is considered adequate for subgrade.

The pavement design for the pavement at all six DCP locations is as follows:

2.75” S9.5C
2.5” I19.0C
11.0” ABC
8.0” Soil Stabilization

From studying the attached DCP graphs, one can see that the quality of all materials underneath the asphalt look good except for possibly the first four inches of ABC. The CBR values are below 100 in this region on all tests. It is hard to say if this is due to poorly compacted ABC or if it is because of sample disturbance caused by coring. (See attached DCP graphs)

All subgrade modulus values calculated from the FWD data are above 10,000 psi., considered the cutoff for adequate subgrade modulus values. (See attached subgrade modulus graphs)

**Testing of heavily distressed pavement in the outside, southbound lane from MM 91.8 to MM 90.7**

The inside wheelpath of the southbound, outside lane was tested on this section since there was already a lane closure. Only FWD testing was performed. Twenty locations were tested. The calculated deflection limit for the project is 11.59 mils using a 10-year design life.

**FWD**

Temperature and load corrected deflections ranged from 8.19 mils to 23.58 mils. The average deflection was 12.60 mils. The FWD analysis indicated the pavement is currently structurally deficient at eight of the twenty (40%) tested locations. Our analysis indicates that a three-inch overlay would address the structural deficiencies for this section at 85% of the tested locations. (See attached FWD charts)

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**Testing of US 117 southbound outside lane and inside lane from SR 1342 to US 70**

Eighty-nine FWD tests, seven cores and seven DCP tests were performed in the center of the southbound, outside lane on May 22, 2007. On June 19, 2007 matching locations in the center of
the inside lane were tested. The FWD analyses were then performed separating the heavily distressed area from the low distressed areas. The limits for the heavily distressed area are from MM 92.0 to MM 89.0. The limits for the low distressed areas are from MM 96.813 to MM 92.0 and from MM 89.0 to MM 87.359. Thirty-one locations were tested in the heavily distressed area and fifty-eight locations were tested in the low distressed area. The data is as follows:

**FWD**
The calculated deflection limit for the project is 11.59 mils using a 10-year design life.

**Outside Lane-Low Distressed Area**
Temperature and load corrected deflections ranged from 17.39 mils to 5.58 mils. The average deflection was 8.38 mils. The FWD analysis indicated the pavement is currently structurally deficient at three of the 58 (5%) test locations. Our analysis indicates that a structural overlay is not needed at this time. (See attached FWD charts)

**Inside Lane-Low Distressed Area**
Temperature and load corrected deflections ranged from 11.49 mils to 5.52 mils. The average deflection was 7.79 mils. The FWD analysis indicated the pavement is currently structurally sufficient at all 58 of the test locations. Our analysis indicates that a structural overlay is not needed at this time. (See attached FWD charts)

**Outside Lane-Heavily Distressed Area**
Temperature and load corrected deflections ranged from 22.34 mils to 7.15 mils. The average deflection was 10.95 mils. The FWD analysis indicated the pavement is currently structurally deficient at eight of the 31 (26%) test locations. Our analysis indicates that a two-inch would address the structural deficiencies for this section at 84% of the tested locations. (See attached FWD charts)

**Inside Lane-Heavily Distressed Area**
Temperature and load corrected deflections ranged from 26.06 mils to 11.31 mils. The average deflection was 15.72 mils. The FWD analysis indicated the pavement is currently structurally deficient at 28 of the 31 (90%) test locations. Our analysis indicates that a four-inch would address the structural deficiencies for this section at 90% of the tested locations. (See attached FWD charts)

**Coring**
Seven cores were taken in the southbound outside lane and seven cores were taken in the southbound inside lane at matching locations. All cores were taken in the center of their respective lane. Aggregate Base Course (ABC) was underneath all cores. The design pavement thickness for the
cores taken at MM 88.153 was 8.5 inches while the pavement design thickness for all other cores was 5.25 inches. The tables below summarize the location, structure and condition of the cores.

### Table 1 – Southbound Outside Lane Core Data Summary

<table>
<thead>
<tr>
<th>FWD Location #</th>
<th>Location (MM)</th>
<th>Structure</th>
<th>Total Thickness</th>
<th>Distresses</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>96.527</td>
<td>1.5” Surface, 1.5” Surface, 3.25” Intermediate</td>
<td>6.25”</td>
<td>Small voids were present in the top surface layer</td>
</tr>
<tr>
<td>14</td>
<td>95.506</td>
<td>1.5” Surface, 1.5” Surface, 2.75” Intermediate</td>
<td>5.75”</td>
<td>Small voids were present in the top surface layer</td>
</tr>
<tr>
<td>26</td>
<td>94.12</td>
<td>1.5” Surface, 1.5” Surface, 2.5” Intermediate</td>
<td>5.5”</td>
<td>Small voids were present in the top surface layer</td>
</tr>
<tr>
<td>39</td>
<td>92.614</td>
<td>1.5” Surface, 1.75” Surface, 2.75” Intermediate</td>
<td>6.0”</td>
<td>Core was in good condition</td>
</tr>
<tr>
<td>45</td>
<td>91.999</td>
<td>1.25” Surface, 2.0” Surface, 2.5” Intermediate</td>
<td>5.75”</td>
<td>Small voids were present in the top surface layer</td>
</tr>
<tr>
<td>63</td>
<td>90.201</td>
<td>1.25” Surface, 1.5” Surface, 2.5” Intermediate</td>
<td>5.25</td>
<td>Small voids were present in all layers</td>
</tr>
<tr>
<td>82</td>
<td>88.153</td>
<td>1.5” Surface, 1.25” Surface, 2.25” Intermediate, 3.0” Base</td>
<td>8.0”</td>
<td>Small voids were present at the intermediate/base interface</td>
</tr>
</tbody>
</table>

### Table 2 – Southbound Inside Lane Core Data Summary

<table>
<thead>
<tr>
<th>FWD Location #</th>
<th>Location (MM)</th>
<th>Structure</th>
<th>Total Thickness</th>
<th>Distresses</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>96.527</td>
<td>1.5” Surface, 1.5” Surface, 3.0” Intermediate</td>
<td>6.0”</td>
<td>Core was in good condition</td>
</tr>
<tr>
<td>14</td>
<td>95.506</td>
<td>1.5” Surface, 1.25” Surface, 3.25” Intermediate</td>
<td>6.0”</td>
<td>Small voids were present in the top surface layer</td>
</tr>
<tr>
<td>26</td>
<td>94.12</td>
<td>1.5” Surface, 1.25” Surface, 2.5” Intermediate</td>
<td>5.25”</td>
<td>Small voids were present in the top surface layer</td>
</tr>
<tr>
<td>39</td>
<td>92.614</td>
<td>1.25” Surface, 1.75” Surface, 2.5” Intermediate</td>
<td>5.5”</td>
<td>Core was in good condition</td>
</tr>
<tr>
<td>45</td>
<td>91.999</td>
<td>1.5” Surface, 1.75” Surface, 2.5” Intermediate</td>
<td>5.75”</td>
<td>Small voids were present in all layers</td>
</tr>
<tr>
<td>63</td>
<td>90.201</td>
<td>1.25” Surface, 1.75” Surface, 2.75” Intermediate</td>
<td>5.75</td>
<td>Small voids were present in all layers</td>
</tr>
<tr>
<td>82</td>
<td>88.153</td>
<td>1.75” Surface, 1.5” Surface, 2.5” Intermediate, 3.0” Base</td>
<td>8.75”</td>
<td>Small voids were present in the base layer</td>
</tr>
</tbody>
</table>

### Subgrade
DCP tests were conducted at all coring locations. The pavement design for the pavement at DCP locations 1-6 is as follows for both lanes:
- 2.75” S9.5C
- 2.5” I19.0C
- 11.0” ABC
- 8.0” Soil Stabilization

The pavement design for the pavement at DCP location 7 is as follows for both lanes:
- 2.75” S9.5C
- 2.5” I19.0C
- 3.25” B25.0C
8.0" ABC

Outside Lane
From studying the attached DCP graphs, one can see that the CBR values on DCP 1 drop below 100 in the first 11 inches of penetration, this indicates low quality ABC. On DCP 4 and DCP 5 the CBR values drop below 25 from 11 to 19 inches of penetration. This indicates poor quality stabilized subgrade. On DCP 7, CBR values drop below 10 after about 19 inches of penetration. This indicates poor subgrade. The other DCP tests indicate good materials under the pavement.

Inside Lane
From studying the attached DCP graphs, one can see that the CBR values on DCP 5 drop below 25 from 17 to 19 inches of penetration. This indicates poor quality stabilized subgrade. On DCP 7, CBR values drop below 10 after about 21 inches of penetration. This indicates poor subgrade. When conducting DCP 3, an impenetrable object was struck after about 9.5 inches of penetration and the test had to be discontinued. (See attached DCP graphs)

All subgrade modulus values for both lanes calculated from the FWD data are above 10,000 psi., considered the cutoff for adequate subgrade modulus values. (See attached subgrade modulus graphs)

Discussion and Recommendations

Our data shows that the heavily distressed pavement from MM 92.0 to MM 89.0 correlates with high deflection areas from our FWD testing. Also, only cores taken from the distressed area show delamination of the top surface layer. Trackless tack was not used in the area exhibiting delamination of the top surface layer. The high amount of fatigue cracking in this area may be attributed to the delamination of the top surface layer although it is still unclear what the exact cause is.

The pavement from MM 96.813 to MM 92.0 and from MM 89.0 to MM 87.359 is currently not showing any distresses and is structurally sound. We recommend no rehabilitation at this time.

From MM 92.0 to MM 89.0 we recommend milling 2.5 inches* and replacing with 2.5 inches of I19.0C on both southbound lanes. Then overlay both lanes with 1.5 inches of S9.5C. Also, full depth repair is needed in areas of visible fatigue cracking.

* Note that 2.5 inches is almost half of the pavement structure. Milling more than half of a pavement structure can result in removal of more material than was intended. In localized areas, it may be necessary to reduce the depth of milling slightly. Where this is done, 2.5 inches of I19.0C should still be placed after milling.

Please contact Clark S. Morrison, Ph.D., PE at (919) 250-4094 if you have any questions or would like to discuss this project further.
CSM/jhm

CC: Wendi Johnson, PE Division Construction Engineer, Division 4
    Mike McKeel, PE, Resident Engineer - Smithfield
    Tom Hearne, PE, Geopavement Supervisor – Harrisburg
    Kevin Bowan, PE, Resident Engineer - Wilson
Appendix D: State Evaluation Report (NB)
September 19, 2007

Memorandum To: Richard E. Greene, Jr., PE  
Division Engineer, Division 4

From: Clark Morrison, Ph.D., PE  
State Pavement Design Engineer

Subject: Pavement Evaluation  
US 117 northbound from SR 1342 (Memorial Church Rd.) to US 70  
Wayne County, Division 4

The Pavement Analysis/Design Group has evaluated the subject section of roadway at the request of Wendi Johnson, Division 4 Construction Engineer. On August 28, 2007, the northbound-outside lane was tested in order to find out if it is experiencing structural failure similar to the southbound couplet as described in our report dated July 25, 2007. Pavement testing began at Mile Marker (MM) 87.50 and extended north to MM 97.00. Trackless tack was used in the northbound-outside lane from north of the bridge over SR 1316 (MM 90.00) to south of the bridge over SR 1336 (MM 91.90). Trackless tack was also used in several untested locations, such as shoulders, deceleration lanes, and ramps. Tests conducted on the roadway include Falling Weight Deflectometer (FWD), coring and Dynamic Cone Penetrometer (DCP).

Overview of Facility and Pavement Conditions

US 117 is a 4-lane, divided facility with 12-foot lanes, 10-foot outside paved shoulders, 4-foot inside paved shoulders with milled rumble strips. On site traffic counts provided by the Division and used for the analysis of the project yielded the following traffic data: 6,654 ADT (year 2007) with 6% duals and 18% TTSTs. This is the same traffic data used for the analysis of the southbound lanes. Deflection analyses of this project were performed using the traffic count data for a 10-year pavement design period, this was the same procedure used for the analysis of the southbound lanes.

Visual inspection of US 117 northbound indicated that the vast majority of this section shows very little distresses and is in good riding condition. However, there are two short areas of low severity fatigue cracking located at MM 88.93 and MM 94.90 in the outside lane only. The fatigue cracking at MM 88.93 is located in the outside wheel path (OWP) and the fatigue cracking at MM 94.90 is located in the outside edge of the lane. These locations match up
closely with areas of fatigue cracking in the southbound-outside lane. There are also two potholes on the northbound-outside lane. One is located at MM 87.90 and the other at MM 92.16. In both potholes the top surface layer is missing. The potholes seem to be caused by localized construction problems, not widespread pavement failure. They are both about one square foot in area.

**FWD**
The calculated deflection limit for the project is 11.59 mils using a 10-year design life. Temperature and load corrected deflections ranged from 18.24 mils to 4.75 mils. The average deflection was 8.04 mils. The FWD analysis indicated the pavement is currently structurally deficient at one of the 92 (1%) tested locations. This location is at MM 88.93, an area experiencing fatigue cracking in the outside wheelpath. Our analysis indicates that a structural overlay is not needed at this time. (See attached FWD charts)

**Coring**
Nine cores were taken in the northbound-outside lane. All cores were taken in the outside wheelpath. Aggregate Base Course (ABC) was underneath all cores. The design pavement thickness for the core taken at MM 87.80 was 8.5 inches while the pavement design thickness for all other cores was 5.25 inches. The tables below summarize the location, structure and condition of the cores.

<table>
<thead>
<tr>
<th>FWD Location #</th>
<th>Location (MM)</th>
<th>Structure</th>
<th>Total Thickness</th>
<th>Distresses</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>87.80</td>
<td>2.0” Surface, 3.75” Intermediate, 3.0” Base</td>
<td>8.75”</td>
<td>Small voids were present in the intermediate and base layers</td>
</tr>
<tr>
<td>16</td>
<td>88.91</td>
<td>1.75” Surface, 1.0” Surface, 2.5” Intermediate</td>
<td>5.25”</td>
<td>Top surface layer debonded</td>
</tr>
<tr>
<td>17</td>
<td>88.93</td>
<td>1.25” Surface, 1.0” Surface, 2.75” Intermediate</td>
<td>5.0”</td>
<td>Top surface layer debonded. Full depth cracking was present</td>
</tr>
<tr>
<td>31</td>
<td>90.30</td>
<td>1.75” Surface, 1.5” Surface, 2.75” Intermediate</td>
<td>6.0”</td>
<td>Top surface layer debonded</td>
</tr>
<tr>
<td>45</td>
<td>91.69</td>
<td>1.5” Surface, 1.0” Surface, 2.75” Intermediate</td>
<td>5.25”</td>
<td>Core was in good condition</td>
</tr>
<tr>
<td>57</td>
<td>92.80</td>
<td>1.5” Surface, 1.75” Surface, 3.0” Intermediate</td>
<td>6.25</td>
<td>Core was in good condition</td>
</tr>
<tr>
<td>69</td>
<td>94.40</td>
<td>1.5” Surface, 1.0” Surface, 3.0” Intermediate</td>
<td>5.5”</td>
<td>Core was in good condition</td>
</tr>
<tr>
<td>75</td>
<td>94.90</td>
<td>1.5” Surface, 1.0” Surface, 2.0” Intermediate</td>
<td>4.5”</td>
<td>Top surface layer debonded and was cracked. Low severity aggregate stripping was present in other two layers</td>
</tr>
<tr>
<td>86</td>
<td>96.28</td>
<td>1.25” Surface, 1.5” Surface, 2.5” Intermediate</td>
<td>5.25”</td>
<td>Small voids were present at the intermediate/base interface</td>
</tr>
</tbody>
</table>
Subgrade
DCP tests were conducted at all coring locations except for core location #2. DCP tests allow us to calculate incremental California Bearing Ratios (CBR) vs. penetration depths. A CBR value of 100 or greater is considered adequate for ABC, with CBR values of greater than 150 indicating the presence of excellent ABC. A CBR value of 25 or greater is considered adequate for stabilized subgrade and CBR values of 10 or greater is considered adequate for subgrade. Usually, the top 24 inches of subgrade are analyzed.

The pavement design for the pavement at DCP location 1 is as follows for both lanes:
- 2.75” S9.5C
- 2.5” I19.0C
- 3.25” B25.0C
- 8.0” ABC

The pavement design for the pavement at all other DCP locations is as follows for both lanes:
- 2.75” S9.5C
- 2.5” I19.0C
- 11.0” ABC
- 8.0” Soil Stabilization

From studying the attached DCP graphs, the following comments can be made:
DCP graph 1 located at MM 87.80: The CBR values drop below 100 in the first 8 inches of penetration. This indicates poor quality ABC. The subgrade beneath the ABC is in excellent condition.
DCP graph 2 located at MM 88.93: The ABC is in good condition. The CBR values drop below 25 after about 16 inches of penetration. This indicates poor stabilized subgrade for the bottom 3 inches of stabilization.
DCP graph 3 located at MM 90.30: The ABC is in good condition. The CBR values drop below 25 after about 16 inches of penetration. This indicates poor stabilized subgrade for the bottom 3 inches of stabilization.
DCP graph 4 located at MM 91.69: All materials under the pavement appear to be of high quality.
DCP graph 5 located at MM 92.80: CBR values are below 100 in the top 4 inches of penetration, indicating poor quality ABC. Also CBR values drop below 25 after about 17 inches of penetration indicating poor stabilized subgrade for the bottom 2 inches of stabilization.
DCP graph 6 located at MM 94.404: All materials under the pavement appear to be of high quality.
DCP graph 7 located at MM 94.90: CBR values are below 100 in the top 5 inches of penetration, indicating poor quality ABC. Also CBR values drop below 25 after about 15 inches of penetration indicating poor stabilized subgrade for the bottom 4 inches of stabilization.
DCP graph 8 located at MM 96.28: The ABC is in good condition. The CBR values drop below 25 after about 17 inches of penetration. This indicates poor stabilized subgrade for the bottom 2 inches of stabilization.
It appears that the ABC, stabilized subgrade, and subgrade underneath the stabilized subgrade on the northbound lanes is in overall worse condition than the materials under the pavement on the southbound lanes.

All subgrade modulus values calculated from the FWD data are above 10,000 psi., considered the cutoff for adequate subgrade modulus values. (See attached subgrade modulus graphs)

**Discussion and Recommendations**

Locations of distressed areas on the northbound-outside lane appear to nearly match up with locations of distressed areas in the southbound-outside lane. Also similar to the southbound-outside lane, only cores taken near or on distressed areas on the northbound-outside lane exhibit delamination of the top surface layer. Furthermore, only cores with delamination have cracking. We recommend the following treatment (same as southbound treatment):

From MM 88.7 to MM 89.7 and from MM 94.6 to MM 95.1 we recommend milling 2.5 inches* and replacing with 2.5 inches of I19.0C on both southbound lanes. Then overlay both lanes with 1.5 inches of S9.5C. Also, full depth repair is needed in areas of visible fatigue cracking.

* Note that 2.5 inches is almost half of the pavement structure. Milling more than half of a pavement structure can result in removal of more material than was intended. In localized areas, it may be necessary to reduce the depth of milling slightly. Where this is done, 2.5 inches of I19.0C should still be placed after milling.

Please contact Clark S. Morrison, Ph.D., PE at (919) 250-4094 if you have any questions or would like to discuss this project further.

CSM/jhm

CC:  Wendi Johnson, PE Division Construction Engineer, Division 4  
Mike McKeel, PE, Resident Engineer - Smithfield  
Tom Hearne, PE, Geopavement Supervisor – Harrisburg  
Kevin Bowen, PE, Resident Engineer - Wilson
Appendix E: MTU Investigation of ABC Material
Memo to: Wendi Johnson, PE  
From: Jack Cowser, PE  
Subject: Results of Investigation by MTU of ABC material on US 117 between Wilson and Goldsboro

Per your request, an investigation was made to determine potential contribution of ABC materials to the pavement failure on US-117 between Wilson and Goldsboro. Three sections were identified and the asphalt pavement was carefully removed to avoid disturbing the underlying ABC (refer to Diagram 1). Based on measurements taken at the test sites the asphalt pavement was between 5 and 5 ½ inches. Visual observation of the asphalt indicated cracking at Test Site 1 while no cracking was observed at Test Sites 2 or 3. Visual observation of the ABC surface did not indicate moisture problems at any of the test sites, and no evidence of subgrade intrusion into the ABC was discovered. The ABC was placed in two layers during construction, so samples were taken from the individual layers and tested separately.

Diagram 1
Density and Moisture Results

According to nuclear test section reports obtained from Project records, the density of this material met the minimum specified requirement during construction of this area (refer to Tables 1 and 2). The data for Tables 1 and 2 was obtained from field density reports submitted by the Resident Engineer.

<table>
<thead>
<tr>
<th>Test Site</th>
<th>Station</th>
<th>ABC Percent Compaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>135+47</td>
<td>10.0 Lt 102.8</td>
</tr>
<tr>
<td>2</td>
<td>134+85</td>
<td>4.0 Lt 99.8</td>
</tr>
<tr>
<td>3</td>
<td>134+39</td>
<td>8.0 Lt 103.3</td>
</tr>
<tr>
<td>4</td>
<td>133+79</td>
<td>4.0 Lt 106.5</td>
</tr>
<tr>
<td>5</td>
<td>133+42</td>
<td>6.0 Lt 102.6</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>103.0</td>
</tr>
</tbody>
</table>

Table 1 – Nuclear Density Tests performed on Layer 1 during construction

<table>
<thead>
<tr>
<th>Test Site</th>
<th>Station</th>
<th>ABC Percent Compaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>135+34</td>
<td>10.0 Lt 102.8</td>
</tr>
<tr>
<td>2</td>
<td>134+80</td>
<td>2.0 Lt 102.6</td>
</tr>
<tr>
<td>3</td>
<td>134+44</td>
<td>2.0 Lt 102.1</td>
</tr>
<tr>
<td>4</td>
<td>133+90</td>
<td>3.0 Lt 103.6</td>
</tr>
<tr>
<td>5</td>
<td>133+36</td>
<td>11.0 Lt 99.4</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>102.1</td>
</tr>
</tbody>
</table>

Table 2 - Nuclear Density Tests performed on Layer 2 during construction

Prior to obtaining the ABC samples, nuclear density readings were taken at each test site. According to project records the ABC in this area was placed between May 13, 2005 and June 6, 2005. During this time period the target density for Princeton Quarry ABC was 141.8 lbs/ft³; therefore, the percent compaction was calculated based on that target density and is listed in Table 3 below.
<table>
<thead>
<tr>
<th>Depth</th>
<th>Test Site 1</th>
<th>Test Site 2</th>
<th>Test Site 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Percent Compacti</td>
<td>Percent Moisture</td>
<td>Percent Compactio</td>
</tr>
<tr>
<td>10”</td>
<td>104.2 4.5</td>
<td>101.6 4.3</td>
<td>103.2 4.5</td>
</tr>
<tr>
<td>8”</td>
<td>103.9 4.7</td>
<td>101.9 4.7</td>
<td>103.0 4.4</td>
</tr>
<tr>
<td>6”</td>
<td>103.7 4.5</td>
<td>102.5 4.3</td>
<td>103.5 4.2</td>
</tr>
<tr>
<td>4”</td>
<td>102.4 4.9</td>
<td>101.4 4.9</td>
<td>101.9 4.7</td>
</tr>
<tr>
<td>2”</td>
<td>100.8 4.7</td>
<td>101.3 4.6</td>
<td>100.5 4.6</td>
</tr>
<tr>
<td>Average</td>
<td>103.0 4.7</td>
<td>101.7 4.6</td>
<td>102.4 4.5</td>
</tr>
</tbody>
</table>

Table 3 – Nuclear Density Tests performed during investigation

An in-place moisture sample was also obtained from each test site and the results confirm visual observations and nuclear gauge results (refer to Table 4).

<table>
<thead>
<tr>
<th>In-place Moisture Content</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Site 1</td>
</tr>
<tr>
<td>4.2 %</td>
</tr>
</tbody>
</table>

Table 4 – Moisture Content tests performed during investigation

Gradation Results

ABC samples were taken from each of the layers to test for gradation and L. A. Abrasion. Gradation results were compared to Table 520-1 in Standard Specifications for Roads and Structures (2002). The Table 5 below provides the specification requirements for gradation, Liquid Limit, and Plasticity Index.
<table>
<thead>
<tr>
<th>Column A Sieve size</th>
<th>Column B % Passing</th>
<th>Column C % Passing</th>
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<tr>
<td>1 ½”</td>
<td>100</td>
<td>98-100</td>
</tr>
<tr>
<td>1”</td>
<td>75-97</td>
<td>72-100</td>
</tr>
<tr>
<td>½”</td>
<td>55-80</td>
<td>51-83</td>
</tr>
<tr>
<td># 4</td>
<td>35-55</td>
<td>35-60</td>
</tr>
<tr>
<td># 10</td>
<td>25-45</td>
<td>20-50</td>
</tr>
<tr>
<td># 40</td>
<td>14-30</td>
<td>10-34</td>
</tr>
<tr>
<td># 200</td>
<td>4-12</td>
<td>3-13</td>
</tr>
</tbody>
</table>

Material Passing No. 10 Sieve (Soil Mortar)

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
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<tbody>
<tr>
<td># 40</td>
<td>40-84</td>
<td>36-86</td>
</tr>
<tr>
<td># 200</td>
<td>11-35</td>
<td>10-36</td>
</tr>
</tbody>
</table>

Material Passing the No. 40 Sieve

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>L. L.</td>
<td>0-30</td>
<td>0-30</td>
</tr>
<tr>
<td>P. I.</td>
<td>0-6</td>
<td>0-6</td>
</tr>
</tbody>
</table>

**Table 5 – Gradation Specifications for ABC**

Test results for the Roadway Assurance samples taken during construction near the test site are listed in Table 6.

<table>
<thead>
<tr>
<th>Sieve No.</th>
<th>RA – 45B % Passing</th>
<th>RA – 46A % Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 ½”</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>1”</td>
<td>94</td>
<td>92</td>
</tr>
<tr>
<td>½”</td>
<td>69</td>
<td>68</td>
</tr>
<tr>
<td># 4</td>
<td>52</td>
<td>53</td>
</tr>
<tr>
<td># 10</td>
<td>36</td>
<td>39</td>
</tr>
<tr>
<td># 40</td>
<td>20</td>
<td>19</td>
</tr>
<tr>
<td># 200</td>
<td>12</td>
<td>10</td>
</tr>
</tbody>
</table>

Material Passing No. 10 Sieve (Soil Mortar)

<table>
<thead>
<tr>
<th></th>
<th>RA – 45B % Passing</th>
<th>RA – 46A % Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td># 40</td>
<td>54</td>
<td>49</td>
</tr>
<tr>
<td># 200</td>
<td>34</td>
<td>26</td>
</tr>
</tbody>
</table>

Material Passing the No. 40 Sieve

<table>
<thead>
<tr>
<th></th>
<th>RA – 45B % Passing</th>
<th>RA – 46A % Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>L. L.</td>
<td>23</td>
<td>24</td>
</tr>
<tr>
<td>P. I.</td>
<td>3</td>
<td>4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Station</th>
<th>RA – 45B % Passing</th>
<th>RA – 46A % Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>93+24 SBL</td>
<td></td>
<td>99+30 SBL</td>
</tr>
</tbody>
</table>

**Table 6 – Gradation tests performed during construction**
Data from tests performed for this investigation of the top and bottom layer of each test site is provided in Tables 7 and 8.

<table>
<thead>
<tr>
<th>Sieve No.</th>
<th>Test Site 1</th>
<th>Test Site 2</th>
<th>Test Site 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>R-A1 Top</td>
<td>R-B1 Top</td>
<td>R-A1 Top</td>
</tr>
<tr>
<td>1 ½”</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>1”</td>
<td>92</td>
<td>93</td>
<td>94</td>
</tr>
<tr>
<td>½”</td>
<td>66</td>
<td>66</td>
<td>73</td>
</tr>
<tr>
<td># 4</td>
<td>59</td>
<td>47</td>
<td>63*</td>
</tr>
<tr>
<td># 10</td>
<td>45</td>
<td>36</td>
<td>49</td>
</tr>
<tr>
<td># 40</td>
<td>26</td>
<td>22</td>
<td>29</td>
</tr>
<tr>
<td># 200</td>
<td>16*</td>
<td>15*</td>
<td>18*</td>
</tr>
</tbody>
</table>

Material Passing No. 10 Sieve (Soil Mortar)

<table>
<thead>
<tr>
<th></th>
<th>Test Site 1</th>
<th>Test Site 2</th>
<th>Test Site 3</th>
</tr>
</thead>
<tbody>
<tr>
<td># 40</td>
<td>58</td>
<td>62</td>
<td>59</td>
</tr>
<tr>
<td># 200</td>
<td>36</td>
<td>40*</td>
<td>37*</td>
</tr>
</tbody>
</table>

Material Passing the No. 40 Sieve

<table>
<thead>
<tr>
<th></th>
<th>Test Site 1</th>
<th>Test Site 2</th>
<th>Test Site 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>L.L.</td>
<td>24</td>
<td>24</td>
<td>22</td>
</tr>
<tr>
<td>P.I.</td>
<td>5</td>
<td>4</td>
<td>5</td>
</tr>
</tbody>
</table>

Visual Condition

* Out of Column C Specification

Table 7 – Gradation tests performed on Layer 2 (top) during investigation

<table>
<thead>
<tr>
<th>Sieve No.</th>
<th>Test Site 1</th>
<th>Test Site 2</th>
<th>Test Site 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 ½”</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>1”</td>
<td>96</td>
<td>95</td>
<td>90</td>
</tr>
<tr>
<td>½”</td>
<td>72</td>
<td>70</td>
<td>61</td>
</tr>
<tr>
<td># 4</td>
<td>52</td>
<td>52</td>
<td>46</td>
</tr>
<tr>
<td># 10</td>
<td>40</td>
<td>39</td>
<td>31</td>
</tr>
<tr>
<td># 40</td>
<td>26</td>
<td>25</td>
<td>14</td>
</tr>
<tr>
<td># 200</td>
<td>17*</td>
<td>17*</td>
<td>8</td>
</tr>
</tbody>
</table>

Material Passing No. 10 Sieve (Soil Mortar)

<table>
<thead>
<tr>
<th></th>
<th>Test Site 1</th>
<th>Test Site 2</th>
<th>Test Site 3</th>
</tr>
</thead>
<tbody>
<tr>
<td># 40</td>
<td>65</td>
<td>65</td>
<td>45</td>
</tr>
<tr>
<td># 200</td>
<td>43*</td>
<td>43*</td>
<td>27</td>
</tr>
</tbody>
</table>

Material Passing the No. 40 Sieve

<table>
<thead>
<tr>
<th></th>
<th>Test Site 1</th>
<th>Test Site 2</th>
<th>Test Site 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>L.L.</td>
<td>27</td>
<td>25</td>
<td>20</td>
</tr>
<tr>
<td>P.I.</td>
<td>7*</td>
<td>6</td>
<td>3</td>
</tr>
</tbody>
</table>

Visual Condition

* Out of Column C Specification

Table 8 - Gradation tests performed on Layer 1 (bottom) during investigation
L. A. Abrasion Results

L. A. Abrasion tests were performed to determine if the aggregate meets the minimum resistance to abrasion. As stated in section 1008-1 (A): “the percent wear shall not be greater than 55 percent”. Test results are summarized in Table 9.

In order to have enough material to perform the L.A Abrasion test, the samples described above were combined as shown in the table. The test procedure requires specific gradations of the material as follows (corresponding to the individual column headers in the table):

- The “A” gradation consists of stone passing the 1 1/2” sieve and retained on the 3/8” sieve. A charge of 12 steel balls weighing 5000 grams +/- 25 grams is added to the machine.

- The “B” gradation consists of stone passing the 3/4” sieve and retained on the 3/8” sieve. A charge of 11 steel balls weighing 4584 grams +/- 25 grams is added to the machine.

- The “C” gradation consists of stone passing the 3/8” sieve and retained on the No. 4 sieve. A charge of 8 steel balls weighing 3330 grams +/- 20 grams is added to the machine.

The test results for L. A. Abrasion indicate that all ABC material passes the specification requirement.

<table>
<thead>
<tr>
<th>Test Site 1 Top (A1 &amp; B1 Combined)</th>
<th>Test Site 2 Top (A1 &amp; B1 Combined)</th>
<th>Test Site 3 Top (A1 &amp; B1 Combined)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percent Wear</td>
<td>Percent Wear</td>
<td>Percent Wear</td>
</tr>
<tr>
<td>A 25</td>
<td>A 28</td>
<td>A 29</td>
</tr>
<tr>
<td>B 25</td>
<td>B 25</td>
<td>B 28</td>
</tr>
<tr>
<td>C 27</td>
<td>C 25</td>
<td>C 26</td>
</tr>
<tr>
<td>Test Site 1 Bottom (A2 &amp; B2 Combined)</td>
<td>Test Site 2 Bottom (A2 &amp; B2 Combined)</td>
<td>Test Site 3 Bottom (A2 &amp; B2 Combined)</td>
</tr>
<tr>
<td>Percent Wear</td>
<td>Percent Wear</td>
<td>Percent Wear</td>
</tr>
<tr>
<td>A 34</td>
<td>A 23</td>
<td>A 24</td>
</tr>
<tr>
<td>B 32</td>
<td>B 22</td>
<td>B 24</td>
</tr>
<tr>
<td>C 38</td>
<td>C 24</td>
<td>C 25</td>
</tr>
</tbody>
</table>

Table 9 – L. A. Abrasion tests performed during investigation
Conclusion

Though some of the gradation results and one Plasticity Index test exceeded specification requirements the results are reasonably close to conformity with specification limits. Additionally, ABC has a tendency to degrade while being compacted to the minimum required density. One must also consider there is a potential for further degradation when obtaining a representative sample from ABC material that has been compacted to over 100 percent of AASHTO T-180.

Therefore, based on visual observations of the ABC in-place and test results performed on representative samples of ABC both during construction and in the investigation, it is concluded that the ABC layers are not significantly contributing to the early pavement failure on US-117.