ABSTRACT

Premature pavement failures and distresses continue to happen despite significant advancements in pavement technology. This paper reports an integrated approach used to evaluate subsurface conditions of 72 centerline kilometers of a highway pavement, as part of an investigation prompted by premature pavement failures of a recent mill and asphalt overlay project.

The integrated approach involved ground penetrating radar (GPR), coring, and laboratory testing, in conjunction with a review of existing falling weight deflectometer (FWD) data, design plans, construction and post-construction documents, and other available data.

A visit to the site was arranged to assess the conditions of the overlay and identify the type(s) of distresses which were visible on the pavement surface.

GPR data was used to identify contributing factors to the premature surface failures, such as large areas of stripped pavement beneath the new asphalt overlay, and localized areas of wet or moist old asphalt base.

Pavement cores provided data to verify the results of GPR Tests.

FWD test data was used to determine the resilient moduli of the existing pavement layers.

The premature pavement failures in this study were believed to be caused by infiltration of water into the milled surface prior to overlaying, questionable structural integrity of the milled surfaces of old asphalt base material, and de-bonding due to tack coat placement issues.

Recommendations based on the findings from this study are included in the paper.

Keywords: Premature failures, Asphalt overlay, Pavement Evaluation, NDT, GPR, Core data, Laboratory testing, FWD.
INTRODUCTION

Premature pavement failures and distresses continue to happen despite significant advancements in pavement technology. The root causes of these failures need to be identified to prevent them from occurring and to find corrective actions. This paper reports an integrated approach used to evaluate subsurface conditions of 72 centerline kilometers of a highway pavement, as part of an investigation for the owner/agency prompted by the premature pavement failures within 2 months of a mill and asphalt overlay project. The integrated approach involved ground penetrating radar (GPR) (1), coring, and laboratory testing, in conjunction with a thorough review of existing falling weight deflectometer (FWD) (2) data, design plans, construction and post-construction documents, and other available data.

During the investigation a site visit revealed a generalized pattern of potholes as the primary distress in the new asphalt overlay material. The overlay appeared to be delaminated from the milled surface and there was loose material in the associated potholes which was apparent as the separation of the overlay from the existing asphalt material. Areas of surface staining, that appeared to be consistent with the deposition of fines from the asphalt mix, were also observed. Shoving of the new overlay was noticed on some ramps. The distresses appeared consistent with stripping, either a cohesion failure between the aggregate surface and asphalt binder or an adhesion failure within the asphalt binder itself. It was also noted that there were areas where asphalt binder material was visible on the surface of the new overlay as “fat spots” (fat spots were isolated areas where the asphalt cement came to the surface of the mix versus bleeding which is characterized by flushed longitudinal streaks in the wheel path).

GPR was extremely useful in identifying contributing factors to the premature surface failures, such as large areas of stripped pavement beneath the new asphalt overlay, and localized areas of wet or moist old asphalt base (3), (4), (5), (6). Based on the results of the GPR survey, the causes of premature failures were further evaluated with a targeted coring and testing plan. Visual observations of pavement cores and testing indicated that the old asphalt base course mix was susceptible to moisture induced damage. Stripping detected by GPR was confirmed in some of the extracted cores.

The premature pavement failures in this study were believed to be caused by infiltration of water into the milled surface prior to overlaying, questionable structural integrity of the milled surfaces of old asphalt base material, and de-bonding due to tack coat placement issues. This paper will share the recommendations based on the findings from this study as well as review of previous studies.

PROJECT UNDERSTANDING

A pavement rehabilitation was designed and constructed (by others) for a section of the tested 72 centerline kilometers (6.61 km) toll road with an AADT of 257,000. The roadway generally consists of 4 to 6 lanes in each direction. The project is located in a tropical climate zone with an average year round temperature of 27ºC and an average of 1,550 mm of rainfall per year.
The pavement rehabilitation was performed between August 2012 and March 2013. Premature pavement failures appeared within two to three months following the final overlay placement. Premature pavement failure for this project is defined as the occurrence of defects and distresses in the completed asphalt courses appearing within 2 months to 1 year of placement creating a loss of expected performance and requiring ongoing maintenance. Distresses were indicated by potholes and the appearance of fines on the surface; usually fines appeared on the pavement surface first, followed by cracking, and finally potholes.

The pavement investigation occurred in two stages, first at the project level in May 2013 and second by an outside independent investigation in October 2013. The first investigation initiated by the owner/agency in May 2013 asked the original construction inspection/firm for the project to investigate and perform testing on a pavement section that exhibited premature failure, and on two pavement sections that did not. Pavement cores were taken and tested for moisture susceptibility, permeability, and pavement saturation. The construction inspection firm in conjunction with the contractor performed additional core testing postulated that water was moving upward from the subgrade and through compromised existing asphalt material to the new overlay causing stripping. Additionally, since the new asphalt overlay was essentially impermeable, the presence of water under the overlay, coupled with temperature changes and traffic loading, caused pore water pressure to build up and fractured the asphalt-aggregate bond.

In October 2013 the owner/agency asked for an independent review and investigation to evaluate possible causes of the premature pavement failures. This paper addresses the methodology and findings of October - December 2013 investigation.

GROUND PENETRATING RADAR (GPR) TESTING

In October 2013 GPR testing was performed in the right wheel path of each lane of the toll road. The GPR utilized a non-contact horn antenna suspended 475 mm over the pavement surface executed at speeds of up to 110 km/hr. Large areas of low density asphalt zones were determined in the existing asphalt material indicating the material had prior defects (7). These defects were subsequently overlaid during the rehabilitation of the roadway.

Additionally based on the estimated dielectric constants, the new and existing asphalt material had low moisture content and existing aggregate base and subgrade were generally dry with some pockets of moisture in the existing asphalt.

INDEPENDENT REVIEW AND INVESTIGATION

In December 2013 as part of the independent review and investigation a site visit and project documentation review of the rehabilitation design and construction inspection was completed.
Rehabilitation Design Review

A review of the design procedure for the pavement rehabilitation indicated that the design was based on the evaluation of the FWD data, pavement cores, and visual examination of the project. The FWD data provided the resilient moduli of the existing pavement layers and was used to determine the pavement strength and required rehabilitation. The design was intended to address areas with poor moduli, areas of known pavement damage, and restore the International Ride Index (IRI) to acceptable limits.

The design team reported that based on the FWD data, the existing pavement structure was generally good, and the pavement surface exhibited typical signs of fatigue including alligator cracking and some areas of bleeding. The team further indicated that in general there was nothing unusual about existing pavement condition or pavement cores. They recommended drainage improvements in areas that were showing premature failures. The design process was standard and applicable based on the data obtained during the pre-design phase.

Construction Inspection Documents Review

Based on the review of the project documents the pavement rehabilitation project consisted of three specific construction activities;

- Type A (2 inches of cold milling and 2 inches surface asphalt course);
- Type B (3 inches cold milling, 1 inch leveling asphalt course, interlayer reinforcement system (1” x 1” geogrid), 2 inches surface asphalt course) ; and
- Type C (full depth removal of existing asphalt pavement, 8 inches of lean cement concrete pavement, 4 inches of base asphalt course, 2 inches of asphalt surface course).

The Type A rehabilitation was used widely throughout this project. The Type B rehabilitation, utilizing the geogrid, was also used throughout the project; however, the Type C rehabilitation was used in very limited locations.

The surface course was an owner/agency approved mix design for Warm Mix Asphalt (WMA). Based on the mix design information provided, the WMA was produced with the addition of Rediset WMX. Wetfix 312 was added as anti-strip chemical. Both of these chemicals are proprietary and were added to the PG 64-22 binder at the terminal. The mixed binder was then shipped to the asphalt plant by tanker trucks. Contractor-produced limestone and “quarry sand” was used for the coarse and fine aggregates with the addition of coarse sand from another source. There were no data available regarding the physical or chemical properties of the fine or coarse aggregates due to possible future litigation of this project.

It was reported that prior to January 2012, cold milling was typically performed three to five days ahead of the paving operation but there were many sections that were open substantially longer, up to 25 days. It was also reported that the cold milled surface appeared to be sound even after being exposed to traffic. There were some instances where additional milling was required as well as the placement of a leveling course to correct unsound existing asphalt surface.
However, these areas were reported as limited. It was noted that no moisture was observed on the
milled surface immediately after milling or prior to paving.

Per the owner/agency’s project records the cold milled surface was power-broomed prior to the
application of a tack coat. The tack coat application was reported as “good” with complete
coverage obtained. The tack was allowed to break prior to paving, but tack pick up by the paver
tires and asphalt trucks was noted. The project records did not include tack application rate or
cure time. The project specifications required application at a rate of 0.30 to 0.45 liter per square
meter. The project records did not provide detailed information regarding actual application
rates.

The mix production temperature was reported at 127°C to 138°C with the mix placement
temperature reported at 113°C to 135°C. The coarse aggregate was reported to be kept under
cover at the asphalt plant, but the fine aggregate was not. The asphalt plant was a standard
counter flow drum mix plant with a cyclone-type dust control unit and baghouse to capture fines.
Fines were introduced into the mix from the baghouse and the asphalt was stored in silos during
production. Records for the operation of the asphalt plant were not available for review due to
possible litigation, nor was it possible to visit the production facility. Release agents in truck
beds were reported to be a mineral oil or paraffin type coating and there were no reports of its
overuse.

The contractor used a Material Transfer Vehicle (MTV) between the end dump asphalt delivery
trucks and the paver. A standard wheeled paver was used for placement. The mat during
placement was reported to be uniform. However it was reported that the mix was tender as
evidenced by a bow wave in front of the rollers. There were times that the rollers lagged far
behind the paver; this can indicate that the roller operators felt the material was tender. Rain was
reported during the paving of 10 of the 41 lots. The contractor power-broomed the surface to
remove any moisture and paving was resumed when possible.

During production quality control cores were extracted from the completed pavement lots and
tested to determine in-place compaction, gradation, and asphalt binder content for payment.
Based on the provided data, the average lot in-place compaction met the requirement of 92 to
97% in all but two lots. However, a review of the sub-lots showed there were twenty-two
locations where the in-place compaction was not met; this occurred in fifteen of the forty-one
lots.

Site Visit Observations

During the December 2013 site visit, all visual observations were performed from a vehicle or
from the pavement shoulder because of the very high average daily traffic. Potholes were the
primary distress in the new overlay and were not located along the longitudinal construction
joint. There seemed to be a generalized pattern of potholes forming approximately 1 to 2 feet
offset from longitudinal construction joints. Areas of surface staining were observed in locations
with and without potholes and appeared to be consistent with the deposition of fines from the
asphalt mix to the surface through a “pumping” action. On some ramps, shoving of the new
Amer-Yahia, Miller, Majidzadeh, Saraf, Majidzadeh

overlay was observed. The depth of potholes appeared to be the same as the thickness of the new overlay. The new overlay material appeared to be delaminated from the milled surface and there was loose material in the associated pothole. The distresses appeared consistent with stripping; there was either a cohesion failure between the aggregate surface and asphalt binder or an adhesion failure within the asphalt binder itself. It was also noted that there were areas where asphalt binder material was visible on the surface of the new overlay as fat spots (isolated areas where the asphalt cement came to the surface of the mix versus bleeding which is characterized by flushed longitudinal streaks in the wheel path.)

Potholes were observed in lower elevation areas and at higher elevations such as a ramp on embankments. The project area is part of a coastal plane and a portion is adjacent to a canal widening project that is to be used for flood control. During a rainfall event, the lower elevation areas of the project showed evidence of ponding water in ditches. In some areas, water remained ponded in ditches, while other areas appeared to drain.

**Discussion of Testing and Data Review**

Based on the design, construction, and post-construction documents and the available data, the distressed areas in the new asphalt surface overlay appeared consistent with a pavement that has stripped; where the bond between the aggregate and the binder, or the bond within the binder itself, has failed in the presence of water.

The contractor and the construction inspection firm claimed that the infiltration of surface water through the new asphalt overlay was not a cause of the stripping and pointed to the testing performed on two control sections, one distressed section, and the laboratory produced samples (“pills”) from the mix design. This testing included permeability using Florida Department of Transportation Test Procedure FM 5-565; Tensile Strength Ratio (TSR) Test Procedure AASHTO T-283 to determine potential for stripping reported as tensile strength ratio (TSR); and AASHTO T-324 (Hamburg Wheel-Track Testing of Compacted Hot Mix, HMA) to determine susceptibility for premature damage reported as Stripping Inflection Point (SIP). The first control section was labeled as “initial” and the second as “final”. The “final” control section was selected as representative due to the presence of cracks in the existing asphalt pavement layer in the “initial” control section. Table 1 summarizes the test results.
### TABLE 1  Section 1 Summary of Test Results by Contractor and Owner/Agency

<table>
<thead>
<tr>
<th>Test</th>
<th>Section. ID</th>
<th>Location</th>
<th>Layer</th>
<th>Acceptance Criteria</th>
<th>Results</th>
<th>Dry Strength Criteria, psi</th>
<th>Dry Strength, psi</th>
<th>Wet Strength, psi</th>
<th>TSR Criteria, %</th>
<th>TSR, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permeability per FM5-565</td>
<td>Final Control</td>
<td>KM 4.4 Lane 4 WB</td>
<td>Surface - New</td>
<td>125 cm/s x10^5</td>
<td>0 cm/s x10^5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Base AC - New</td>
<td></td>
<td>0 cm/s x10^5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Distressed</td>
<td>KM 3.23 Lane 3 EB</td>
<td>Surface - New</td>
<td>NA</td>
<td>58%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Base AC existing</td>
<td></td>
<td>75%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Initial Control</td>
<td>KM 2.68 Lane 3 EB</td>
<td>Surface - New</td>
<td>NA</td>
<td>27%</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Base AC existing</td>
<td></td>
<td>Core crumbled - no test</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*% Saturation = [(% Moisture by Wt. x Bulk SG)/ % Air Voids] x 100

<table>
<thead>
<tr>
<th>Test</th>
<th>Area</th>
<th>Location</th>
<th>Layer</th>
<th>Dry Strength Criteria, psi</th>
<th>Wet Strength, psi</th>
<th>TSR Criteria, %</th>
<th>TSR, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO T-283</td>
<td>Final Control</td>
<td>KM 4.5 Lane 4 WB</td>
<td>Surface - New</td>
<td>80</td>
<td></td>
<td>75</td>
<td>105</td>
</tr>
<tr>
<td></td>
<td>Initial Control</td>
<td>KM 2.68 Lane 3 EB</td>
<td>Surface - New</td>
<td>99</td>
<td>105</td>
<td>75</td>
<td>105</td>
</tr>
<tr>
<td></td>
<td>Distressed</td>
<td>KM 3.23 Lane 3 EB</td>
<td>Surface - New</td>
<td>79</td>
<td>40</td>
<td>51</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Base AC existing</td>
<td></td>
<td></td>
<td>93</td>
<td>30.8</td>
</tr>
<tr>
<td>AASHTO T-324</td>
<td>Final Control</td>
<td>KM 4.4 Lane 4 WB</td>
<td>Base AC - New</td>
<td>70</td>
<td>107</td>
<td>65</td>
<td>85</td>
</tr>
<tr>
<td></td>
<td>Initial Control</td>
<td>KM 2.68 Lane 3 EB</td>
<td>Base AC - New</td>
<td>179</td>
<td>135</td>
<td>NA</td>
<td>75</td>
</tr>
<tr>
<td></td>
<td>Distressed</td>
<td>KM 3.23 Lane 3 EB</td>
<td>Base AC - Existing</td>
<td></td>
<td></td>
<td>117</td>
<td>88</td>
</tr>
<tr>
<td></td>
<td>Design Lab</td>
<td>Surface - New</td>
<td>80</td>
<td>135</td>
<td>131</td>
<td>75</td>
<td>97</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test</th>
<th>Area</th>
<th>Location</th>
<th>Layer</th>
<th>Acceptance Criteria, TX</th>
<th>Rut Passes</th>
<th>SIP Passes</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO T-324</td>
<td>Design (AC Mix)</td>
<td>Lab. 12.5 mm</td>
<td></td>
<td>9,412</td>
<td>6,083</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Final Control</td>
<td>KM 4.4 Lane 4 WB</td>
<td>Surface - New</td>
<td>10,000</td>
<td>10,973</td>
<td>6,667</td>
</tr>
<tr>
<td></td>
<td>Design (AC mix)</td>
<td>Lab 19 mm</td>
<td></td>
<td>16,333</td>
<td>10,250</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Final Control</td>
<td>KM 4.4 Lane 4 WB</td>
<td>Base AC - New</td>
<td>20,000</td>
<td>No SIP</td>
<td></td>
</tr>
</tbody>
</table>
Evaluating these test data, the contractor and the construction inspection firm concluded that moisture could not be entering the pavement from the top and infiltrating into the base materials, and thus had to be a result of saturation of the existing asphalt material, the existing aggregate base, and the subgrade from the bottom or sides. This theory included water infiltration from the nearby canal, flooding of ditches and wetting of base material, and a substandard underdrain system that does not drain water away from the pavement. In all of these cases, it would be possible for water to enter the subgrade and potentially travel upward, particularly during flooding situations and due to the proximity to the canal dredging project. The data presented by the contractor and the construction inspection firm were not conclusive for the following reasons:

1. **Permeability of the surface course in the distressed section was not tested.** Permeability testing was performed on a control section with a corresponding higher average in-place compaction (Lot Production Number, LPN 33, IPC = 96%) than other lots that also have not shown signs of distress. For example, LPN 15 had an average in-place compaction of 91.1% and testing could reveal the pavement to be more permeable than reported from the testing of LPN 33. The contractor did not test the existing asphalt mix layer for permeability.

2. **The contractor felt that the TSR data from the distressed section was not relevant due to the presence of water and cracks in the pavement layers that would skew the test results.** However, since the location of the cores is near an area of potholes and stripping, it would follow that the low TSR results could indicate susceptibility to water damage or stripping.

3. **The in-situ moisture (% Saturation) is inconsistent with the results of the GPR testing performed in October 2013.** Overall, the GPR test data indicated that the asphalt material (new and existing) typically had low moisture content, and the aggregate base material was generally dry. Dielectric constants in two areas showing potholes were evaluated to assess the potential for trapped moisture in the asphalt layers; the distressed section (LPN 14) in lanes 3 and 4 between km 3.389 and km 3.445 that was used in the contractor’s testing and in another section (LPN 6). As can be seen in Table 2, the dielectric constants indicate some moisture in both the new surface course and existing asphalt base material. It is interesting that the higher dielectric constant in LPN 6 coincided with the greater number of days the cold milled surface was open to traffic (19 days) as compared to LPN 14 (2 to 5 days).

### TABLE 2 Dielectric Constants from GPR Testing

<table>
<thead>
<tr>
<th>Location</th>
<th>Lane</th>
<th>LPN</th>
<th>Average Dielectric Constant - Surface</th>
<th>Average Dielectric Constant – Existing Asphalt Base</th>
</tr>
</thead>
<tbody>
<tr>
<td>Km 3.389 – 3.438 EB</td>
<td>3</td>
<td>14</td>
<td>6.02</td>
<td>6.53</td>
</tr>
<tr>
<td>Km 3.389 – 3.438 EB</td>
<td>5</td>
<td>6</td>
<td>6.83</td>
<td>7.31</td>
</tr>
<tr>
<td>Ramp Lane</td>
<td>6</td>
<td></td>
<td>6.70</td>
<td>7.10</td>
</tr>
</tbody>
</table>

A review of the pavement core data from 2012 (by others) shows that the aggregate base is mostly an AASHTO A-2-4 to A-2-6 material that could become saturated under flooded...
conditions but it is granular and should quickly drain as water recedes. A confining layer of clay, such as an AASHTO A-7-6, could trap water in the aggregate base material. A review of the pavement logs showed this could be possible in some areas where cores were taken, but these locations are inconsistent with the presence of moisture in the aggregate base material per the GPR testing. Also one area where a core was taken was not rehabilitated, indicating the pavement structure is still functioning adequately.

Additionally, the Hamburg Wheel testing was performed on cores extracted from the final control section (see Table 1); the new asphalt base material did not indicate stripping, based on the reported Stripping Inflection Point (SIP), the new surface asphalt did indicate stripping with an SIP of 6,667 passes (3,333 cycles). The industry standard and specification used by most state DOTs is 10,000 passes (5,000 cycles).

The asphalt production process for the WMA surface mixture was not available and there remain questions regarding the potential for dusty/dirty aggregates or residual aggregate moisture that could be a cause of the stripping. The mineralogical properties of the aggregate are not known and there are no data available. It was stated that the coarse aggregate was kept under cover but the fine aggregate was not. Fine aggregate moisture can range from 0 to 16% due to rain and dry cycles. The owner/agency could not provide access to the asphalt production plant or plant records due to possible litigation regarding this project.

**2013 CORE EVALUATION**

Additional cores were obtained by the owner/agency from LPN 15 and represent the following distressed and non-distressed areas. (Rehabilitation types were described on page 3.)

1. Area with visible distress at a relatively low pavement elevation area. This area was rehabilitated using Type B;
2. Area with visible distress at a relatively higher pavement elevation area. This area was rehabilitated using Type B; and
3. Area without visible distress. This area was rehabilitated using Type A.

All cores were taken from the eastbound left lane between approximate km 2.4 and 2.8. A total of seven cores were taken in each area. Six 4-inch diameter cores and one 6-inch diameter core were obtained. The 6-inch diameter core was a full-depth sample of the pavement from surface to base or to subgrade material. The cores were examined and the following tests were performed.

- AASHTO T 283 Moisture susceptibility testing, Tensile Strength Ratio (TSR)
- In-place air void content
- Gradation
- Percent binder (asphalt) content
- Percent passing sieve #200
Visual Examination of the Cores

The cores were visually examined. Cores in Areas 1 and 2 had at least three visible distinct layers of asphalt mix. These layers were: (1) top layer of new surface course mix, (2) second layer of new leveling course mix, and (3) third layer of old asphalt mix, which typically included more than one layer of asphalt mix. Cores in Area 3 had a visible new surface course mix and an old asphalt mix that included surface, intermediate, and base courses. All asphalt below a new surface course or leveling course was designated as “old asphalt base” mix.

Table 3 provides an overview of the visual observations of the cores.

<table>
<thead>
<tr>
<th>AREA</th>
<th>7 of 7 cores</th>
<th>2 of 7 cores</th>
<th>3 of 7 cores</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>7 of 7 cores</td>
<td>3 of 7 cores</td>
<td>1 of 7 cores</td>
</tr>
<tr>
<td>3</td>
<td>0 of 7 cores</td>
<td>3 of 7 cores</td>
<td>0 of 7 cores</td>
</tr>
</tbody>
</table>

2013 Laboratory Test Results

The cores were tested in the laboratory to determine the in-place characteristics of the new asphalt surface mix and old asphalt base mix. The results of these tests are summarized below.

Percent Air Void Content

Percent air void content of new surface mix samples and old asphalt base course mix samples were measured in the laboratory. The results are summarized in Table 4 for cores obtained from Areas 1, 2, and 3.

<table>
<thead>
<tr>
<th>Area</th>
<th>Location</th>
<th>No. of Samples Tested</th>
<th>Avg. Bulk Sp. Gr.</th>
<th>Avg. Air Voids, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>New Surface Mix</td>
<td>6</td>
<td>2.421</td>
<td>4.823</td>
</tr>
<tr>
<td></td>
<td>Old Base Course Mix</td>
<td>6</td>
<td>2.275</td>
<td>10.125</td>
</tr>
<tr>
<td>2</td>
<td>New Surface Mix</td>
<td>6</td>
<td>2.413</td>
<td>5.429</td>
</tr>
<tr>
<td></td>
<td>Old Base Course Mix</td>
<td>6</td>
<td>2.349</td>
<td>7.186</td>
</tr>
<tr>
<td>3</td>
<td>New Surface Mix</td>
<td>6</td>
<td>2.384</td>
<td>6.421</td>
</tr>
<tr>
<td></td>
<td>Old Base Course Mix</td>
<td>6</td>
<td>2.372</td>
<td>6.284</td>
</tr>
</tbody>
</table>
Gradation, Percent Passing Sieve #200, and Percent Binder (Asphalt) Content

The gradations of asphalt mixes used in the new asphalt surface mix were determined from the cores of Areas 1, 2, and 3. The results of these tests are summarized in Figure 1. The values in the last column “JMF” (Job Mix Formula) refer to the values that were approved by the owner/agency for the Marshall design Warm Mix Asphalt (WMA).

![Summary of Surface Mix Gradation](image)

FIGURE 1 Surface Mix Gradations of Areas 1, 2, 3 and JMF.

The percent binder (asphalt) content and percent Passing Sieve # 200 (dust determination) were also determined. The results of these tests are listed in Table 5. The following results are from the quantitative extraction method. The ignition method was used and provides slightly different results as shown in parentheses. The extraction method is considered more accurate.

<table>
<thead>
<tr>
<th>Test Area</th>
<th>% Asphalt Content</th>
<th>% Passing Sieve #200 (Fine)</th>
<th>F/A (Fines/Asphalt) Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.7 (6.49)</td>
<td>2.7 (3.8)</td>
<td>0.40 (0.58)</td>
</tr>
<tr>
<td>2</td>
<td>5.9 (6.18)</td>
<td>2.4 (3.9)</td>
<td>0.41 (0.63)</td>
</tr>
<tr>
<td>3</td>
<td>6.0 (6.13)</td>
<td>2.4 (4.2)</td>
<td>0.40 (0.56)</td>
</tr>
<tr>
<td>JMF</td>
<td>5.52*</td>
<td>6.5</td>
<td>1.31**</td>
</tr>
</tbody>
</table>

*Does not include WMA Additive and Antistripping agent.

**Dust to Effective Asphalt Ratio.
Tensile Strength (Unconditioned and Conditioned)

Indirect Tensile Strength tests were performed on core samples obtained from all three areas (1, 2 and 3). For this purpose, three cores of each area were tested unconditioned (dry) and three cores were tested after conditioning the cores according to the procedure described in AASHTO T 283. The results of these tests are summarized in Table 6.

TABLE 6 Results of Indirect Tensile Strength Tests
New Surface Course (Top) and Old Asphalt Base Course (Bottom) Mixes

<table>
<thead>
<tr>
<th>Test Area</th>
<th>Average Strength, Unconditioned (Dry) Samples, psi</th>
<th>Average Strength, Conditioned Samples, psi</th>
<th>Strength Ratio = (Col. C/Col. B)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Column B</td>
<td>Column C</td>
<td>Column D</td>
</tr>
<tr>
<td>1-Top</td>
<td>120.5</td>
<td>105.1</td>
<td>0.87</td>
</tr>
<tr>
<td>2-Top</td>
<td>104.5</td>
<td>77.5</td>
<td>0.74</td>
</tr>
<tr>
<td>3-Top</td>
<td>104.2</td>
<td>87.5</td>
<td>0.84</td>
</tr>
<tr>
<td>JMF</td>
<td>130.9</td>
<td>130.7</td>
<td>0.969</td>
</tr>
<tr>
<td>1-Bottom</td>
<td>126.6</td>
<td>71.55</td>
<td>0.56</td>
</tr>
<tr>
<td>2-Bottom</td>
<td>121.3</td>
<td>77.44</td>
<td>0.64</td>
</tr>
<tr>
<td>3-Bottom</td>
<td>153.4</td>
<td>102.97</td>
<td>0.67</td>
</tr>
</tbody>
</table>

DISCUSSION OF 2013 LABORATORY TEST RESULTS

The results of tests performed on the cores obtained from three areas (1, 2 and 3) are as follows:

1. The gradation tests performed on the new surface course mix (see Table 4) indicated that the aggregates of the mixes in all three areas had similar gradation characteristics with little or no variability. The gradations of the tested cores met the JMF requirements with the exception of percent passing the #200 sieve. A low percent passing the #200 sieve can result in a tender mix that is highly permeable.

2. Average bulk specific gravity (BSG) of the surface mix ranged from 2.421 to 2.384 and average percent air voids ranged from 4.823 to 6.421 (see Table 3). These results indicated that Area 1 with visible failures and relatively low elevation had the maximum average BSG and lowest average air voids (highest average in-place compaction of 95.2%), whereas, Area 3 with no visible failure had the lowest average BSG and highest average air voids (lowest average in-place compaction of 93.6%). In all cases, the average in-place compaction of the surface mix in all areas ranged from 93.5% to 95.1%; all are within specification and not statistically different.

3. The lowest single in-place compaction result for the new surface course was 91.7% in Area 3 and was only slightly out of specification.

4. Average bulk specific gravity (BSG) of the old asphalt base mix ranged from 2.275 to 2.372 and average percent air voids ranged from 10.125 to 6.284 (89.9 % to 93.7% average in-place compaction) (see Table 3). These results indicate that Area 1 with
visible failures and relatively low elevation had the lowest average BSG and highest
average air voids with an average in-place compaction of 89.9%, whereas, area 3 with no
visible failure had highest average BSG and lowest average air voids with an average in-
place compaction of 93.7%.

On the basis of these observations, the development of premature failure may be related
to lower BSG and higher air voids (lower in-place compaction) in the old asphalt base
mix.

5. The discussion of Percent Binder (Asphalt) Content and the Percent Passing Sieve #200
is based on the quantitative extraction method. The Percent Binder (Asphalt) Content of
the new surface mix ranged from 5.9 to 6.7 as compared to the JMF of 5.52. The Percent
Binder (Asphalt) Content of 6.7 from Area 1 did not meet the specification tolerance of ±
0.52 (5.52 + 0.52 = 6.04). The Percent Passing Sieve #200 of the surface mix ranged
from 2.4 to 2.7 as compared to the JMF of 6.5 and did not meet the specification
tolerance of ± 3 (6.5 – 3 = 3.5). Note that these data are based on unwashed samples. It is
noted that LPN 15 that represents the placement quality control testing for Areas 1, 2, and
3 did not indicate that the F/A ratio was out of tolerance.

6. The average unconditioned (dry) Indirect Tensile Strength (ITS) of the new surface
course mix was 120.5 psi, 104.5 psi and 104.2 psi in Areas 1, 2, and 3 respectively. These
results are lower than the JMF value of 130.9 psi but met the specification requirement of
80 psi (see Table 6). Also, in Area 1, where the pavement elevation was relatively low
and premature failures were observed, the average unconditioned ITS was higher than the
average unconditioned ITS of Area 2 (higher elevation and visible failures) and Area 3
(no visible failures). The average conditioned (wet) ITS of the new surface course mix
was 105.1 psi, 77.5 psi and 87.5 psi in Areas 1, 2, and 3, respectively. These results are
lower than the JMF value of 130.7 psi. All but Area 2 met the specification requirement
of 80 psi (see Table 6). Also, in Area 1, where the pavement elevation is relatively low
and premature failures were observed, the average conditioned ITS was higher than Area
2 and Area 3. The ITS Ratio of new surface mix (see Table 6) ranges from 74% to 87%
as compared to the JMF value of 96.87%. All but Area 1 met the specification
requirement of 75%. These ITS test values indicate that the new surface course is not a
mix that is particularly susceptible to moisture induced damage. However some evidence
of stripping is visible in the cores.

7. The ITS test data for old asphalt base mix indicated that the average unconditioned ITS
values for all three areas were higher than the corresponding JMF values for the new
surface mix (see Table 6). The average conditioned ITS values were lower and therefore
resulted in a lower ITS Ratio of 56% to 67% indicating that the old asphalt base mix is
susceptible to moisture induced damage. Stripping in the old asphalt layer was confirmed
using GPR imaging and is visible in some of the extracted cores.
CONCLUSIONS

Based on the GPR test results; review of the project records and test data; visual observation of pavement cores; and testing performed on the cores of the new surface course mix and the old asphalt base course mix, our findings are as follows:

1. The existing asphalt base material had significant areas of stripping as determined using GPR. After milling existing asphalt base material may have absorbed significant amounts of moisture when it was exposed to surface water runoff and traffic for several days before overlaying:

   - A review of the available construction records for Areas 1, 2, and 3 (LPN 15) indicated that the areas were open to weather and traffic for approximately six to eleven days.
   - GPR and core testing showed that the existing asphalt base mix had areas of relatively high air void content and other damage related to stripping (as visible in some cores) that would allow surface water runoff to enter the pavement matrix prior to the overlay application.
   - The highest air voids in the existing asphalt base layer were measured in Area 1, which exhibited the greatest amount of surface failures; lesser air voids were measured in the existing asphalt base layer in Area 2 and corresponded to lesser amounts of surface failures; and the least air voids in the existing asphalt base layer measured in Area 3 corresponded to no visible surface failures.

   Additionally infiltrated surface water that was trapped in the old asphalt base layer would condense with temperature variations and create pressure within the matrix that could cause distress in the new surface or leveling courses, eventually allowing potholes to form and thus allowing additional water migration into the existing pavement layer(s). Also, paving over an existing damaged asphalt surface can manifest in future pavement failures even if moisture is not an issue.

2. Considering the inconsistency of premature surface failures in the different coring areas, it is possible that Area 1 absorbed more moisture than area 2 due to the elevation difference. Area 2 is at a relatively higher elevation; therefore, rainfall on this area would run off faster than rainfall on Area 1. In both Areas 1 and 2, there were old asphalt base failures that would allow surface water infiltration which could result in damage to the new pavement courses. The old asphalt base of Area 3 was comparatively denser than the other two areas, having the lowest air voids. Additionally, the old asphalt base cores from this Area 3 were intact and did not have significant damage. Therefore, the infiltration of surface water runoff into the old asphalt base is less and the area had no visible failures.

3. Delamination without stripping was observed in cores from Areas 1, 2, and 3. The delamination appeared in Areas 1 and 2 at the surface/leveling course interface (at the
location of the geotextile), at the leveling course/old asphalt base course; and at the
surface/old asphalt base course in Area 3. Delamination without other apparent distress
mechanisms generally occurs due to the lack of an adequate bond between layers.
Without a secure bonding face, the pavement structure does not act as a unit and the
individual layers must resist shear stressed induced by traffic. Tack coat between the
layers at delaminated areas was difficult to see. It is possible that tracking and pick up of
the tack coat under construction traffic was a factor in these failures. Allowing the milled
surface to remain open to traffic and precipitation over a period of days can also create
problems with cleanliness and dampness that will inhibit a good tack bond. It is
additionally possible that the “tack coat” for the interlayer reinforcement system
(geogrid) was not as specified. The project specifications required the use of the “same
grade asphalt cement as used in the hot mix asphalt”. Records for the material used in
this application could not be found. The delamination issue could exacerbate potential
problems with surface water runoff (as postulated in Items 1 and 2 above). Delamination
or de-bonding would allow water to infiltrate into the pavement structure as the weak
pavement layer is peeled away by the action of traffic. Water would move along the
layer interface causing further delamination and the potential for stripping. Based on the
available records it is unlikely that the mix design itself, as produced, is a cause of
premature pavement failure. Field compaction of the mix appears consistent in Areas 1,
2, and 3. It is improbable that the source of water is from the subgrade, as was proposed
by the construction contractor and the construction inspection company. The geotechnical
firm hired to extract cores reported that they did not observe free water in any of the core
holes. This is consistent with GPR testing that found no excess moisture in the subgrade
or base materials but some moisture in the asphalt pavement layers.

In summary, and based on the mechanisms of premature failure outlined above, the causes of
premature failure are: (1) surface water infiltration into the milled surface prior to overlaying; (2)
questionable structural integrity of the milled surfaces of old asphalt base material (stripping);
(3) delamination/de-bonding due to tack placement issues.

RECOMMENDATIONS

Based on our findings from this study as well as review of previous studies, it was recommended
to the owner/agency that for the constructed project:

1. The correction of areas of pavement distress likely will require full depth repairs. The
majority of failures from Areas 1, 2, and 3 were located in the wheel path. In addition to
surface water infiltration during construction, there are areas of fatigue cracking and
other existing asphalt base deficiencies (including stripping) that need repair. These areas
will need to be located based on current observations and any other pavement records and
past pavement surveys. GPR and FWD can be used to assess the areas of significant
failures under the overlay.
2. The existence of underdrains should be determined and if they do not exist or are not operational at this time, installation of underdrains should be considered. Functional underdrains can help outlet water that is trapped in pavement layers.

Based on our findings from this study as well as review of previous studies, it was recommended to the owner/agency that for the future projects:

1. Because of the local weather conditions (frequent rainfall) and the known potential for stripping in the existing asphalt the use of GPR and targeted FWD testing can help locate low density areas for additional evaluation in the design phase.

2. Because of the local weather conditions (frequent rainfall) projects which require milling and overlay should be milled only in lengths that can be paved almost immediately so that the milled surface is not left open to rain and traffic for any significant length of time. Also, the damaged areas which may become visible after milling should be appropriately repaired (partial or full depth) before overlaying.

3. The use of trackless tack should be considered to avoid tracking and pickup of the tack coat to prevent potential delamination problems.

REFERENCES


