DEPARTMENT OF TRANSPORTATION

MnDOT Thin Whitetopping Selection Procedures

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16. Abstract (Limit: 250 words)

This report provides an integrated selection procedure for evaluating whether an existing hot-mix asphalt (HMA) pavement is an appropriate candidate for a bonded concrete overlay of asphalt (BCOA). The selection procedure includes (1) a desk review, (2) coring, (3) visual examination (site visit), (4) additional coring and/or laboratory testing (optional), (5) preparation of preliminary estimates (optional), and (6) a final report with design recommendations. This project also included an analysis of material testing performed by the Minnesota Department of Transportation (MnDOT) on numerous HMA cores to determine if the performance of existing BCOAs could be correlated to HMA material properties. The results of the laboratory testing were inconclusive. None of the existing HMA material properties tested could be correlated to long-term BCOA performance due to the high variability among the measured parameters within a section, the small number of cores per section, and the fact that the sections investigated exhibited little or no distress that could be attributed to the asphalt layer's properties. Although no conclusions could be made from the limited laboratory testing, it is fair to say that the BCOAs from these projects were performing as designed.

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MnDOT Thin Whitetopping Selection Procedures

FINAL REPORT

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EXECUTIVE SUMMARY

The purpose of this report is to provide an integrated selection procedure for evaluating whether an existing hot-mix asphalt (HMA) pavement is an appropriate candidate for a bonded concrete overlay. A bonded concrete overlay of asphalt (BCOA) is defined as a concrete surface bonded to the top of an existing HMA pavement. When an effective bond is achieved, the concrete surface and underlying HMA act as one monolithic structural unit. The thickness of BCOAs range from 2 in. to 6 in.

The step-by-step integrated selection procedure includes the following:

- Desk review
- Coring
- Visual examination (site visit)
- Additional coring and/or laboratory testing (optional)
- Preparation of preliminary estimates (optional)
- Final report with design recommendations

Also included in the scope of this project was the analysis of material testing performed by the Minnesota Department of Transportation (MnDOT) on numerous HMA cores to determine if the performance of existing BCOAs could be correlated to HMA material properties.

The results of this laboratory testing were inconclusive. None of the existing HMA material properties tested could be correlated to the long-term performance of BCOA projects. This was because the variability among the measured parameters within a section was high, the number of cores tested per section was small, and the sections investigated exhibited little, if any, distress that could be attributed to the properties of the asphalt layer. Although no conclusions could be made from the limited laboratory testing performed, which included HMA cores from BCOA projects that had widely varying material properties and that were in varying degrees of condition (good to fair), it is fair to say that the BCOAs from these projects were performing as designed.

CHAPTER 1: SCOPE OF THIS DOCUMENT

The purpose of this document is to provide a comprehensive process to assist Minnesota Department of Transportation (MnDOT) personnel in determining whether a bonded concrete overlay on asphalt (BCOA) is an appropriate design alternative for an existing pavement with a hot-mix asphalt (HMA) surface in fair to good condition. There are many factors that need to be considered in determining whether a BCOA is feasible for a specific project. Often, there are subtle differences between the feasibility of a given design and whether it is actually the optimal design. In other words, even with a comprehensive process in place, exceptions to the rule are common.

Where applicable, this document provides insights into the underlying concepts that are integral to the performance of BCOA pavements in an effort to encourage sound engineering judgement rather than reliance on a static process lacking flexibility. Therefore, proper application of the process included in this document is dependent upon balancing quantitative and qualitative information to determine correctly whether a BCOA design is appropriate for a specific project.

This document is not a design guide; details such as geometrics, structural design, joint design, etc. are not covered. The following resources should be consulted along with MnDOT policies for specific design guidance:

- *Guide to Concrete Overlays, 3rd Edition* (Harrington and Fick 2014)
- *BCOA-ME Design Guide*
- *Guide Specifications for Concrete Overlays* (Fick and Harrington 2015)

Likewise, this document does not provide guidance for life-cycle cost analysis (LCCA) or alternate design/alternate bid (AD/AB) procedures. Where appropriate, existing MnDOT procedures should be followed for these analyses. If further guidance is needed, consult the following resources:

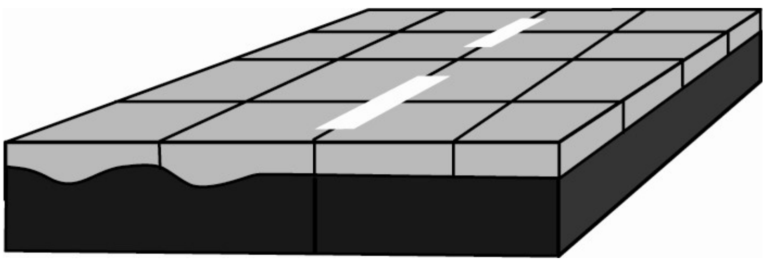
- Life-Cycle Cost Analysis website (FHWA 2017a)
- Life-Cycle Cost Analysis Software (FHWA 2017b)
- *NCHRP Report 703: Guide for Pavement-Type Selection* (Hallin et al. 2011)
- Alternate Pavement Type Bidding (AD/AB) website (FHWA 2017c)

CHAPTER 2: INTRODUCTION

Sometimes referred to as ultra-thin whitetopping (UTW) but now more correctly called BCOA, these designs began to gain momentum in the mid-1990s as many states and local agencies tried them for the first time. Currently there are 108 BCOA and UTW projects documented in the National Concrete Overlay Explorer, which is a database of all types of overlay projects in North America maintained by the American Concrete Pavement Association (ACPA 2017). BCOAs have been constructed on all functional classifications of roadways.

2.1 DEFINITION OF BCOA

A BCOA can be defined as a concrete surface bonded to the top of an existing HMA pavement (Figure 1). When an effective bond is achieved, the concrete surface and underlying HMA act as one monolithic structural unit. The thickness of BCOAs range from 2 in. to 6 in., but historically 4 in. has been the most common design thickness.



Harrington and Fick 2014

Figure 1. BCOA on HMA.

2.1.1 3 in. Minimum Asphalt Must Remain for BCOA

Because the concrete surface is so much stiffer than the underlying HMA (approximately 10 times stiffer), it is critical that at least 3 in. of HMA in fair to good condition remain in place to obtain any appreciable structural benefit from the HMA layer.

2.1.2 Why Are BCOAs Limited to ≤ 6 in. Thickness?

This question is related to the relative stiffness of the two materials that are bonded together. Once the thickness of the concrete layer exceeds approximately 6 in., the structural contribution of the HMA layer is negligible. Therefore, thicker overlays on HMA are commonly designed as unbonded overlays, where the existing HMA is considered as a uniform base layer.

2.1.3 Is There Any Benefit to Having a Thicker HMA Layer (>3 in.) Remaining?

For a given BCOA thickness, the flexural stress in the concrete decreases as the thickness of the remaining asphalt increases. For this reason milling good-condition HMA is not recommended unless there are vertical restraints or rutting is present. This is because as the thickness of the asphalt increases, the neutral axis is pushed further down in the structure, meaning that more of the concrete layer is in compression than tension. Figure 2 provides an illustration of the impact of increased asphalt thickness for a 3 in. thickness of BCOA, where the neutral axis moves down from a depth of 1.8 in. below the surface of the concrete to a depth of 2.4 in.

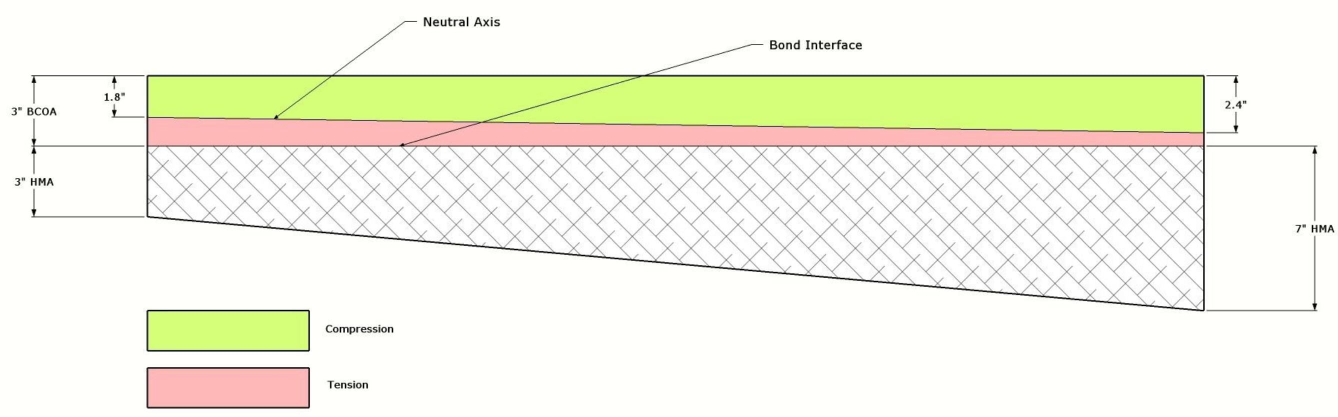


Figure 2. Decrease in tension of the BCOA as the depth of HMA increases (based on E_{pcc} = 3,600,000 psi and E_{hma} = 350,000 psi).

2.1.4 Applicability of BCOA Designs

When properly constructed, BCOA pavements have performed as designed for all functional classifications of roadways. A recent study of BCOA projects in Iowa by the National Concrete Pavement Technology Center found 175 existing BCOA projects in good or better condition with service lives between 25 and 35 years. Because BCOAs have such a wide range of applications that carry vastly different volumes of heavy truck traffic (e.g., from municipal streets to Interstate highways) (Figure 3), the design life of a BCOA may be 5 years to more than 30 years but is typically on the order of 20 years.

Figure 3. BCOA on I-70 in western Kansas.

2.1.5 Keys to Achieving Success for a BCOA Project

In cases where the performance of a BCOA has not been satisfactory, the most common reasons for unsatisfactory performance have been determined to be the following:

- Existing HMA pavement not a good candidate for a BCOA.
- Poor design details (e.g., joints, drainage, etc.).
- Inferior construction practices.
- Material-related distresses.

Because the most common reason for inadequate performance has been the application of BCOA designs to existing HMA pavements that were not good candidates, the primary objective of this document is to provide guidance on how to determine whether an existing roadway is a viable candidate for a BCOA. A few factors are typically predominant in determining whether an existing HMA pavement is a viable candidate for a BCOA design:

1. The thickness and condition of the existing HMA.
2. The estimated extent and cost of pre-overlay repairs that would be necessary to restore an existing HMA roadway to fair or good condition.

Once it has been determined that the existing HMA will cost-effectively accommodate a BCOA design, there are a number of other factors that are critical to the success of a BCOA. These are summarized in the *Guide to Concrete Overlays, 3rd Edition* (Harrington and Fick 2014) as follows:

- Milling of existing asphalt may be required to eliminate or reduce surface distortions of 2 in. (50 mm) or more and to help provide a good bond.
- Minimal spot repairs may be required to provide a uniform and stable base.
- A minimum of 3 in. (75 mm) of asphalt in fair to good condition should remain after milling.
- The asphalt surface should be sprinkled with water when the surface temperature is greater than 120°F (49°C) during overlay placement.
- A clean surface is critical to achieving an adequate bond between the overlay and the underlying asphalt.
- An appropriate panel size should be established with respect to the thickness of the concrete overlay and should preferably be sawed in small square panels. It is recommended that the length and width of individual slab squares in feet be limited to 1.5 times the overlay thickness in inches.
- Transverse joints must be sawed T/3 (with special attention to thickened overlay over asphalt ruts and other nonuniform areas).
- When feasible, the longitudinal joints should be designed to be outside of the normal wheel paths.
- No notable stripping or delamination at tack lines exists in asphalt pavement should remain after milling.
- Thinner overlays may shorten the sawing window; additional saws are likely to be required.

- Application of curing compound or other curing methods must be timely and thorough, especially at the edges.
- Sealing joints improves performance.
- When practical, heavy equipment should be kept off of the remaining 3 in. to 4 in. of HMA thickness before the overlay is placed. This minimizes damages to the remaining HMA from construction traffic.
- Adequate drainage of the HMA section should be provided to prevent stripping from heavy truck loading.

When a bonded concrete overlay is placed in cooler weather, the day/night temperature differential will cause movement in the existing HMA pavement; it will expand during the day and contract at night. To prevent cracking in the overlay, the concrete mixture must reach a strength adequate for sawing before nighttime contraction of the underlying HMA pavement. Specifying a concrete overlay mixture temperature of 65°F has proven to be helpful in mitigating this set-time issue.

CHAPTER 3: INTEGRATED SELECTION PROCEDURES

The following step-by-step process provides a framework for determining whether an existing pavement with a HMA surface is an appropriate candidate for a BCOA design. There are many subtleties and exceptions to the rule when determining the appropriateness of a BCOA design. Even though BCOA designs are robust, there may be cases where the physical properties of the existing HMA fall below the recommended quantitative criteria provided in this document but a BCOA design may still be appropriate. Likewise, there may be cases where the structural characteristics of the existing HMA are well suited for a BCOA, yet other circumstances such as geometrics dictate that a BCOA design is not optimal. Therefore, sound engineering judgment should always complement the quantitative analyses and, in some cases, supersede those analyses in determining whether a given existing HMA is a good candidate for a BCOA design.

3.1 SUMMARY OF HMA DISTRESSES TO BE CONSIDERED DURING THE BCOA DESIGN PROCESS

It is helpful to have a basic understanding of distresses in HMA pavements that may affect whether a BCOA design is appropriate. The following summaries are provided for general information on the cause(s) of these distresses and how they may impact whether a BCOA design is appropriate for a specific roadway.

3.1.1 Stripping

Stripping of asphalt is a loss of bond between the asphalt binder and aggregate. The susceptibility to stripping is a function of the asphalt film thickness (AFT), surface charge of the aggregate, and the presence of any anti-stripping agents, such as lime, in the binder. If stripping occurs at the surface of the asphalt layer, it will affect the bond with the overlay concrete (Vandenbossche and Fagerness 2002).

After the overlay is constructed, any water that enters the joint in the overlay may become trapped in the joint until the water slowly drains out of the structure and into the adjacent drainage system. This can leave the asphalt directly below the overlay near the joint more susceptible to stripping, which can potentially result in a loss of bond between the asphalt and the overlay in this region. Debonding between the layers can lead to a large stress increase in the overlay (Barman et al. 2016). If the stripping is restricted only to the surface, then milling may be an effective means to improve the initial durability of the bond. However, if the mixture is the same throughout the asphalt layer, then there may be future stripping concerns with the asphalt, which may lead to debonding between the portland cement concrete (PCC) and asphalt layers.

Although stripping on the surface can be identified by a visual inspection, it is important to pull cores prior to placing a BCOA to determine whether stripping has developed within or at the bottom of the asphalt as well. This check is necessary because the existing asphalt layer often consists of several overlays with different mixture designs that have been placed at different times throughout the life of the pavement, and some of these mixes might be more susceptible to stripping than others.

3.1.2 Rutting

Rutting can occur as a result of one of two mechanisms, shear flow or consolidation. Shear flow is a constant volume deformation process. Material flows from the wheel path to an area adjacent to the wheel path, resulting in an upward heave in the regions adjacent to the wheel path. This is the dominant mode of permanent deformation in asphalt-surfaced pavements (Epps et al. 2002).

When a BCOA is constructed, the PCC layer constrains the surface of the asphalt layer and prevents this heave from developing. Without space for material to flow, constant-volume shear flow cannot occur beneath a BCOA. This constraint against flow may not occur at the longitudinal joints. Material may flow across the joint, leading to "nonparallel slabs," where adjacent panels no longer lie on parallel planes but are divergent. Although structurally sound, this increases the roughness of the BCOA. Using tie bars at the longitudinal joints, which is common practice in Colorado and Iowa, or synthetic fibers in the mix, which is common practice in Illinois, has been shown to deter shear flow.

Rutting can also develop as a result of consolidation deformation. In this case, there is a decrease in the volume of asphalt due to a reduction in air voids as traffic loading compacts the mix beyond the initial construction compaction. This deformation is not accompanied by a heave adjacent to the wheel path and only manifests as consolidation within the wheel path. It is unlikely for consolidation deformation to occur in the asphalt layer of BCOA. Traffic on the asphalt surface prior to the overlay being constructed will result in any consolidation that might occur in the asphalt. Additionally, after the overlay is constructed the overlay will distribute the wheel load over a larger area, thereby greatly reducing the vertical stresses on the top of the asphalt layer.

3.1.3 Cracking

Fatigue cracking, longitudinal cracking, and transverse thermal cracking are common in existing HMA pavements and have multiple mechanisms. The primary concern with respect to BCOA design and performance is reflective cracking in the BCOA.

Further details, trigger values, and mitigation strategies for dealing with cracking and rutting distresses in the existing HMA are provided in the following step-by-step process.

3.2 STEP 1. DESK REVIEW

The purpose of a desk review is to gather as much pertinent information as is practical to provide a general idea of what the existing pavement section(s) consist of and the performance history of the pavement to date. The desk review does not determine whether the existing HMA surface is adequate for a BCOA design; rather, it provides information that assists in the successful completion of the latter steps of the process (i.e., visual inspection and coring). It is also important to acknowledge that the accuracy of historical records is sometimes lacking. As-built drawings do not always provide a true picture of what was constructed, and maintenance activities are sometimes not recorded or filed. This does not mean that a desk review is unnecessary, but that an appropriate level of effort should be dedicated to this task to enhance the subsequent steps in the process.

3.2.1 Determine the Project Limits

This may be by station, milepost, or other reference.

3.2.2 Summarize Current Traffic Data

Annual average daily traffic (AADT) with percent trucks is adequate.

3.2.3 Consult Historical Records

The following historical records may be consulted:

- Design reports from original construction:
    - Geotechnical information (soil type, CBR, k-value, modulus, etc.)
    - Other materials properties
    - Environmentally sensitive areas
- Construction and/or as-built plans:
    - Typical section(s)
    - Material types associated with typical section layers (e.g., granular base and asphalt binder grade)
    - Presence of longitudinal edge drains
    - Bridges and overpasses
- Pavement management system:
    - Maintenance and repairs
    - Preservation activities
    - Pavement condition ratings
    - Distress types

3.2.4 Consult Design, Maintenance, and Construction Personnel

Often it can be beneficial to verify historical records or fill in gaps in the historical records by interviewing staff that were involved in the original construction and/or maintenance of the roadway.

At a minimum, this desk review should result in a sketch of all of the typical section(s) presumed to be within the proposed project limits. Ideally, the desk review should produce a summary report similar to the one shown in Appendix A.

Additionally, the information gathered during the desk review should be used to develop and/or modify a checklist for the visual examination to be performed during Step 2 of this process (see Appendix B).

3.3 STEP 2. CORING

Coring the existing HMA is mandatory; it is beneficial to obtain some cores from the pavement prior to or in conjunction with the visual survey. There are two primary purposes for obtaining cores from the existing HMA:

1. To verify that a minimum of 3 in. of HMA is present or will be present after milling, if necessary.
2. To determine, through visual examination of the core, the overall condition of the HMA that is to remain. For example, remaining HMA that is highly oxidized, has known or suspected stripping of asphalt binder, or has considerable delamination between remaining lifts may not be a good candidate for a BCOA. Further material testing as outlined in Step 4 and Appendix D should be considered when these conditions are noted.

A minimum of two cores per lane-mile should be taken from the travel lanes. One core per mile from surfaced shoulders should be obtained.

These cores should be measured for length and visually inspected for evidence of stripping. While still early in the investigation process, some determination of whether the existing HMA will be milled prior to the construction of a BCOA must take place. If no milling is necessary for profile grade concerns, the cores of the existing HMA should be a minimum of 3 in. thick and have no indication of stripping in any of the HMA layers (Figure 4). If milling is necessary to minimize the impacts of raising the profile grade or if evidence of stripping is present, verify that the stripped layer(s) can be removed by milling and that a minimum thickness of 3 in. of sound HMA will remain (Figure 5).

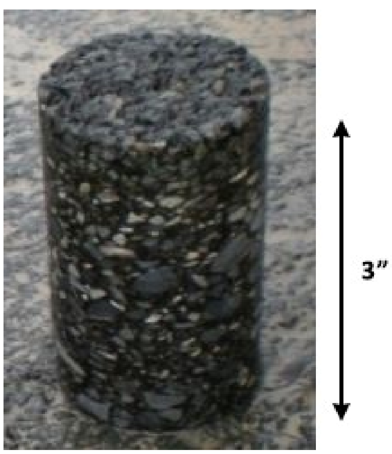


Figure 4. HMA core showing 3 in. of HMA in good condition.

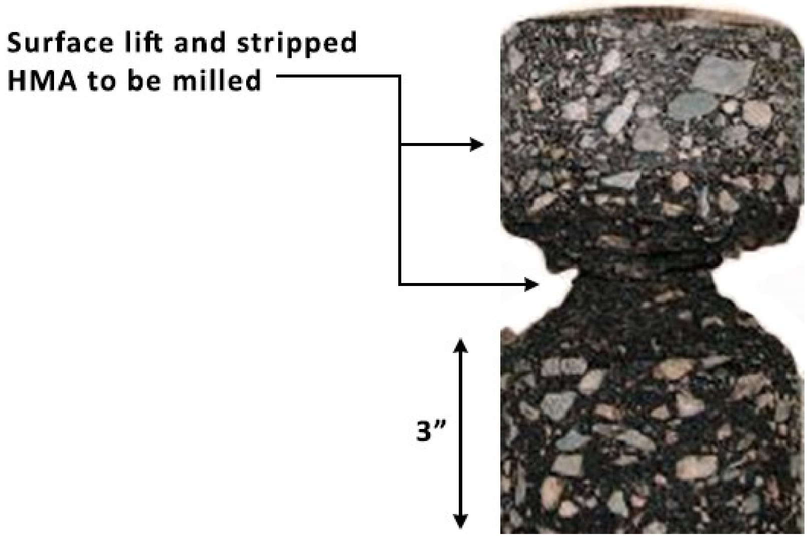


Figure 5. HMA core showing surface lift and stripped intermediate layer to be milled, with 3 in. of HMA in good condition remaining after milling.

3.4 STEP 3. VISUAL EXAMINATION (SITE VISIT)

The purpose of the visual examination is to verify as much as possible the condition of the existing HMA, identify inadequate support conditions, and quantify certain project features that may impact the applicability and/or cost of a BCOA design. An example checklist for the visual survey, as referenced in the previous section, is provided in Appendix B. Keeping in mind that BCOA designs require a minimum of 3 in. of HMA in fair to good condition, the initial on-site review of a project should consist of an overview of the entire project and focus on identifying whether the majority of the HMA appears to be in at least fair condition and identifying/quantifying any other features (e.g., guard rail, barrier rail, overhead structures, etc.) that may prove too costly to mitigate.

3.4.1 Distress Survey

A comprehensive visual survey includes quantifying the existing distresses in the HMA. Recommended maximum limits for each type of distress are shown in Table 1 (adapted from Harrington and Fick 2014, p. 110). Note that these are surface distresses and may not necessarily prohibit a BCOA design. That determination is dependent upon the condition and thickness of underlying HMA layers revealed during the coring operation and whether milling will be performed. Milling may be desirable either to minimize the impacts of raising the profile grade or to intentionally remove the distresses in the HMA surface layers.

Table 1. Recommended maximum threshold levels for distresses in the existing HMA

Distress (unit)	Roadway Functional Classification	Recommended Maximum Distress Level
Fatigue cracking (% of wheel path area)	Interstate/Freeway	10
	Primary	20
	Secondary	20
Longitudinal cracking in the wheel path (ft/mi)	Interstate/Freeway	550
	Primary	1250
	Secondary	1250
Transverse thermal crack spacing (ft)	Interstate/Freeway	130
	Primary	50
	Secondary	50
Mean depth of rutting in both wheel paths1 (in.)	Interstate/Freeway	2
	Primary	2
	Secondary	2
Shoving (% of wheel path area)	Interstate/Freeway	4
	Primary	15
	Secondary	30

[1] Rutting greater than 2 in. should be removed by milling to conserve the volume of concrete required in the BCOA.

3.4.1.1 Fatigue Cracking

In terms of the structural properties of the existing HMA, fatigue cracking is the primary distress that can be identified visually and may render BCOA designs inappropriate. Unless the surface lift of the HMA will be milled prior to the construction of a BCOA, extensive fatigue cracking is not conducive to good performance of a BCOA pavement. According to the *Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2003), levels of distress for fatigue cracking are defined as follows:

- Low (Figure 6). An area of cracks with no or only a few connecting cracks; cracks are not spalled or sealed; pumping is not evident.
- Moderate (Figure 7). An area of interconnected cracks forming a complete pattern; cracks may be slightly spalled; cracks may be sealed; pumping is not evident.
- High (Figure 8). An area of moderately or severely spalled interconnected cracks forming a complete pattern; pieces may move when subjected to traffic; cracks may be sealed; pumping may be evident.

Figure 6. Low-severity fatigue cracking in HMA that is not an issue for BCOA designs.

Figure 7. Moderate-severity fatigue cracking in HMA that is not an issue for BCOA designs.

Figure 8. High-severity fatigue cracking in HMA that is not suitable for BCOA designs unless milling will expose HMA in fair to good condition.

Isolated areas of fatigue cracking may be an indicator of underlying subgrade support and/or drainage issues that need to be repaired prior to construction of any pavement rehabilitation, including BCOA. The cost of pre-overlay repairs necessary to mitigate areas with fatigue cracking should be estimated and balanced against other design options. A rule of thumb is that when pre-overlay repairs exceed 20% of the total area to be overlaid with concrete, an unbonded concrete overlay may be more appropriate than a BCOA.

3.4.1.2 Longitudinal Cracking in the Wheel Path

Longitudinal cracking in the wheel path (Figure 9) is typically not an issue for BCOA designs unless there is vertical displacement between the adjacent sides of the crack. Differential deflections between the adjacent sides of the crack exceeding 1/4 in. indicate problems with subgrade support and/or drainage issues. Both of these problems need to be mitigated regardless of the final pavement design.

Figure 9. Longitudinal cracking in the wheel path of an existing HMA pavement.

Cores should be taken to establish the depth the crack has propagated into the asphalt layer as well as the degree of deterioration with depth.

If the longitudinal crack width is less than the maximum coarse aggregate size used in the concrete overlay mixture, then no pre-overlay action needs to be taken. A flowable fill should be used prior to the placement of the overlay if the longitudinal crack width is greater than the maximum coarse aggregate size used in the concrete overlay mixture (Harrington and Fick 2014).

3.4.1.3 Transverse (Thermal) Cracking

Transverse cracks develop in asphalt pavements due to the restraint of thermal contraction caused by the friction between the bottom of the asphalt and the base and the continuous nature of an unjointed asphalt pavement. A transverse crack is shown in Figure 10. These cracks can propagate up into the bonded concrete overlay when the asphalt is significantly stiff relative to the overlay, as shown in Figure 11.

Figure 10. Transverse (thermal) cracking in an existing HMA pavement.

Vandenbossche and Barman 2010, Vandenbossche et al. 2016

Figure 11. Reflective cracking observed at the Minnesota Road Research Facility (MnROAD).

The thickness of the overlay should not be increased to address thermal cracks in the existing asphalt. However, it may be necessary to perform pre-overlay repairs to ensure that the thermal cracks do not influence the performance of the overlay. If the transverse crack width is greater than that of the maximum coarse aggregate size used in the concrete overlay mixture, a flowable fill should be used. This will prevent interlocking between the overlay and the asphalt layer. A flowable fill is not required if the transverse crack width is less than the maximum coarse aggregate size used in the concrete overlay mixture (Harrington and Fick 2014).

Reflection cracks are anticipated to develop in the overlay if the flexural stiffness ratio, $D_{PCC/AC}$, falls below 1 at a temperature measured on site. Equation 1 can be used to determine $D_{PCC/AC}$. If the transverse crack width is less than the maximum coarse aggregate size used in the overlay mixture and $D_{PCC/AC}$ is less than 1, preventive measures can be taken to ensure that the transverse crack does not propagate up into the overlay.

$$D_{PCC/HMA} = \frac{E_{PCC} \times h_{PCC}^3}{E_{HMA} \times h_{HMA}^3} \left(\frac{1 - \mu_{HMA}^2}{1 - \mu_{PCC}^2} \right) \tag{1}$$

where

$D_{PCC/AC}$ = relative stiffness of the concrete with respect to the asphalt layer
E_{PCC} = elastic modulus of the concrete, psi
E_{HMA} = resilient modulus of the asphalt, psi
h_{PCC} = thickness of the concrete overlay, in.
h_{HMA} = thickness of the asphalt, in.
μ_{PCC} = Poisson's ratio of the concrete
μ_{HMA} = Poisson's ratio of the asphalt

Debonding the overlay from the asphalt layer in the vicinity of the thermal crack will prevent the crack from reflecting up into the overlay. Tar paper stapled to the surface or duct tape placed over the crack, as shown in Figure 12, has been demonstrated to effectively deter reflective cracking. Some departments of transportation consider the development of a few reflective cracks in the concrete overlay to be acceptable, but the development of reflective cracking can be prevented as described above, if desired.

Vandenbossche and Fagerness 2002

Figure 12. Prevention of reflective cracking by localized debonding.

A full-depth patch should be performed if the transverse crack is severely deteriorated throughout the depth of the asphalt layer and the pavement is unstable. If this type of heavily deteriorated cracking is extensive throughout the section, then an unbonded concrete overlay may be a more viable option than a thin bonded concrete or thin asphalt overlay.

3.4.1.4 Rutting and Shoving

Deformations in the HMA due to rutting and shoving (Figures 13 and 14) are not a structural concern for BCOA designs. However, it is recommended that surface deformations greater than 2 in. be removed by milling in order to conserve the volume of concrete required for the BCOA. Quantify the area of milling in square yards required to remove these distortions during the visual survey.

Figure 13. Rutting in HMA evidenced by standing water in the wheel paths.

Figure 14. Shoving in HMA pavement at an intersection approach.

3.4.2 Subgrade Support and Drainage Conditions

Some of the distresses (isolated fatigue cracking and longitudinal cracking in the wheel path) observed in an existing HMA pavement may indicate issues related to subgrade support and/or drainage of the pavement system. This should be noted during the visual survey. Variability in the roadway profile (long-wavelength bumps and dips) are also indicative of inadequate support/drainage. Regardless of the pavement design (BCOA, unbonded concrete overlay, HMA overlay, or full-depth reconstruction), these issues need to be mitigated. The most common mitigation method is undercut and backfill with select material. In some cases, retrofitting of longitudinal edge drains may be effective.

Because these areas should be addressed prior to any pavement rehabilitation strategy, an estimate should be prepared to determine if the mitigation method(s) required before any type of overlay are cost prohibitive. When widespread or numerous areas need extensive and costly mitigation, full-depth reconstruction may be the preferred design approach.

In cases where falling weight deflectometer (FWD) data are available for the existing roadway, Appendix C provides a method for graphically reviewing the underlying support conditions. This process is not necessary for most BCOA projects. However, when FWD data have already been collected by pavement management systems, this method can be helpful in determining whether a BCOA design is appropriate for a specific section of roadway and may also provide useful insights for optimizing BCOA designs.

3.4.3 Profile Grades and Cross-Slope

As stated above, variations in the longitudinal profile grade may indicate support and/or drainage issues. Other items to note during the visual examination relating to profile grade are areas of limited sight distance and short vertical curves that may be desirable to correct as part of the proposed project. Given an adequate thickness of existing HMA, these issues may be wholly or partially corrected by profile milling prior to construction of the BCOA.

Cross-slope(s) of the existing HMA surface should be measured, especially in the case of older roadways, where the cross-slope may have been reduced due to multiple HMA overlays or where the curves may not be super-elevated. In either case, it may be desirable to adjust the cross-slope for safety reasons; existing MnDOT policies should govern whether the proposed cross-slope of the BCOA needs to differ from the existing cross-slope. Profile milling may be considered in these areas if an adequate thickness of HMA exists that allows a minimum of 3 in. of HMA in fair to good condition after milling is accomplished. Otherwise, adjusting the cross-slope in the BCOA is necessary. An estimate of the volume of additional concrete necessary to make cross-slope adjustments should be made as shown in Figure 15.

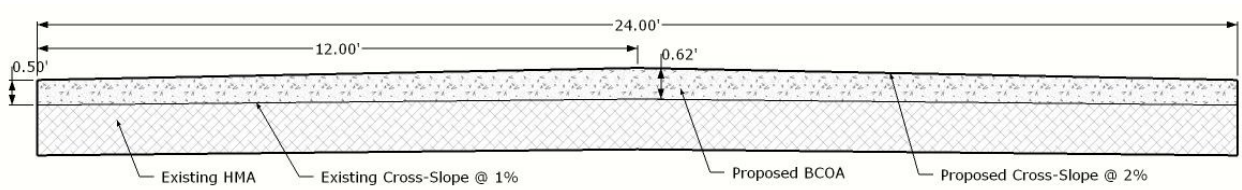


Figure 15. Adjusting cross-slope in the BCOA.

The following is an example of estimating the volume of additional concrete necessary for a cross-slope adjustment:

In Figure 15, where cross-slope is adjusted from an existing 1% to 2%, the additional quantity of concrete required due to the cross-slope adjustment for 1 mile of surfacing equals approximately 282 cubic yards (0.5 x 0.12 ft x 12 ft x 5280 ft x 2 ÷ 27).

3.4.4 Quantify Vertical Constraints

3.4.4.1 Effects of Grade Changes

Depending upon the type of roadway, raising the profile grade of an existing roadway can present challenges. For a BCOA design, there is a two-step process for determining the best approach:

1. A determination needs to be made as to whether the profile grade can be raised without incurring prohibitive costs due to adverse effects on adjoining features, including but not limited to the following:
    a. Driveways.
    b. Intersections.
    c. Curbs and gutters.
    d. Storm sewer inlets.
    e. Drainage structures – pipes and culverts.
    f. Guard rail, barrier rail, cable barrier, parapets, etc.
    g. Overhead clearances.
    h. Bridges.

   If so, then the BCOA design can be placed directly upon the existing HMA with no milling or minimal milling.
2. If there are too many vertical constraints to cost-effectively raise the profile grade by the full thickness of the BCOA, then a determination needs to be made as to whether full or partial milling can mitigate the vertical constraints. The maximum milling depth allowable is a function of the depth and condition of the existing HMA and is governed by the need to maintain a minimum of 3 in. of existing HMA in fair to good condition after milling is completed.
   For example (Figure 16), if the existing HMA is found to be 7 in. thick and in good condition and the BCOA thickness is estimated to be 5 in. thick, it is feasible to mill a nominal 4 in. of the existing HMA, thus minimizing the effect of raising the profile grade.

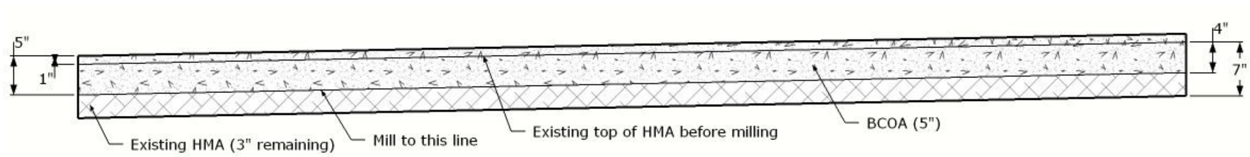


Figure 16. Mitigating the effect of grade changes through milling.

Appendix C provides a method for determining maximum milling depths and whether multiple BCOA designs may be necessary due to differing support conditions. It should be noted that coring the existing HMA pavement is mandatory to determine the existing thickness and whether any stripping or delamination is present. Appendix C also provides a method for using ground penetrating radar (GPR) data to better determine maximum milling depth. This step is not necessary for most BCOA projects. However, when GPR data have already been collected by pavement management systems, this optional step may provide a more complete picture of the variability of existing HMA thickness between core locations.

3.4.4.2 Quantify Vertical Constraints

As a part of the visual examination, all vertical constraints listed above should be quantified for the proposed project. Depending upon the feature, this may be a count of each occurrence or the length of the feature.

Additional information and design details related to changes in profile grade can be found in the *Guide to Concrete Overlays, 3rd Edition* (Harrington and Fick 2014, pp. 65 and 66).

3.4.5 Shoulder Areas and Widened Lanes

3.4.5.1 Surfaced Shoulders

In many cases, the typical section of existing HMA-surfaced shoulders may be different than that of the main lanes. This is one reason for obtaining cores from the shoulders (see Step 2). The condition and width of the surfaced shoulders should be documented.

3.4.5.2 Existing Widened Lanes

Longitudinal cracking between the shoulder and main lanes may indicate differing support conditions, heaving soils, and/or the presence of a widened lane. None of these conditions affect the applicability of a BCOA design, but they do impact the design of the BCOA.

If it is determined that an existing widened lane is present, strong consideration should be given to removing the widened section prior to construction of the BCOA. Experience has shown that longitudinal cracking can occur in the BCOA due to nonuniform support from the widened section, even when a longitudinal sawcut joint is placed over the widened lane. Design details for the treatment of existing widened sections can be found in the *Guide to Concrete Overlays, 3rd Edition* (Harrington and Fick 2014, pp. 71 and 72).

3.4.5.3 Granular Shoulders

Document the width and depth of any granular material that is present. Consult the Guide to Concrete Overlays*, 3rd Edition* (Harrington and Fick 2014, p. 71, Figure 81) if widening of the roadway is part of the proposed BCOA design.

3.4.6 Summary Report of Visual Examination and Coring

Upon completion of the visual examination of the roadway, a summary report should be prepared. This report can be in any format but at a minimum should provide the following:

1. A summary of all observations made:
    a. Project description and limits.
    b. Existing cross-slopes.
    c. Existing safety slopes.
    d. HMA thickness if coring was done prior to or concurrent with the visual examination.
    e. Pavement width.
    f. Shoulder width.
    g. General description of the pavement condition.
2. Estimated quantities and locations of the following:

    a. Distresses in the existing HMA.
    b. Subgrade failures requiring pre-overlay repair.
    c. Potential profile grade corrections.
    d. Potential cross-slope adjustments.
    e. Vertical constraints.
3. Recommendations for additional coring and/or testing if necessary.
4. Action items necessary to make a final determination of the applicability of a BCOA design.

3.5 STEP 4. ADDITIONAL CORING AND/OR LABORATORY TESTING (OPTIONAL)

Subsequent to the summary report of the visual examination, any additional coring deemed necessary to further determine the thickness and/or condition of the existing HMA should be performed. Findings from any additional coring of the existing HMA should be attached to the summary report.

The original scope of this project included laboratory testing of the HMA as a means to assist in determining whether a BCOA design is appropriate for a specific section of roadway. The details of this task are provided in Appendix D.

The following distresses that can develop in the BCOA as a result of the existing asphalt were identified and are listed below. Each of the three distresses is followed by parameters evaluated as potential indicators of the development of that specific distress.

1. Reflective cracking or reflected distress (ratio of the flexural stiffness of the concrete layer with respect to that of the asphalt layer).
2. Uneven slabs (or migration) due to asphalt deformation (rutting in the existing HMA, inappropriate binder grade, and/or voids filled with asphalt [VFA]).
3. Premature fatigue cracking as a result of debonding or support loss due to stripping (VFA, adjusted AFT, and/or inappropriate aggregate type).

The following sections summarize the results of the laboratory study with respect to each of these three overlay distresses.

3.5.1 Reflective Cracking or Reflected Distress

Of the six sections cored, only four of the sections had developed transverse cracks in the existing HMA pavement. Of these sections, only one exhibited reflective cracks. This pavement was also the only one of the four that had a flexural stiffness ratio greater than 1.

3.5.2 Uneven Slabs (or Migration) due to Asphalt Deformation

Rutting was evident in three of the six sections cored. This can be an indication of an unstable asphalt mixture that could contribute to slab migration if a thin concrete overlay is used. The softest binder measured for the cores tested was PG 64-28. The VFA values measured for these sections were highly variable. The core with the highest VFA value (and therefore the most susceptible to slab migration) was the core with the PG 64-28 binder. Despite the fact that this section could potentially be the most susceptible to the development of uneven slabs, this distress was not reported to have developed in this section or any of the sections cored.

3.5.3 Premature Fatigue Cracking as a Result of Debonding or Support Loss due to Stripping

Stripping was found in two of the six BCOAs cored. The stripping in all cores was near the asphalt-concrete interface. Both sections are still performing well, and the stripping does not appear to be contributing to the premature deterioration of either roadway. Material characterization was performed in the laboratory on one of the six cores that exhibited stripping. The adjusted AFT was determined to be 9.1 microns, and the VFA value was found to be 50.1%. Although the VFA value would be considered low for an asphalt surface layer, the adjusted AFT is sufficient. It is difficult to determine the suitability of these values for an asphalt layer under a BCOA because insufficient data are available for characterizing asphalt layers that did exhibit stripping and because the stripping is not affecting the performance of the pavement. What can be concluded is that the stripping that developed near the asphalt-concrete interface did not cause premature deterioration in the BCOAs cored. These sections see limited traffic, which indicates that more leniency may be appropriate when evaluating allowable stripping criteria for roadways with lower volumes of traffic.

The results of the laboratory testing were inconclusive. None of the existing HMA material properties tested could be correlated to the long-term performance of the BCOA projects because the variability among measured parameters within a section was high, the number of cores tested per section was small, and the sections investigated exhibited little, if any, distress that could attribute to the properties of the asphalt layer. Although no conclusions could be made from the limited the laboratory testing performed, which included HMA cores from BCOA projects that had widely varying material properties and whose conditions varied from good to fair, it is fair to say that the BCOAs from these projects were considered to be performing as designed.

Therefore, until a comprehensive study including further laboratory testing of HMA from BCOA projects can be performed, existing HMA pavements should continue to be evaluated through visual examination of cores for thickness and identification of layers exhibiting stripping. The performance record across the nation for BCOA projects on HMA pavements in fair to good condition has been well documented. Designs based on sound engineering judgment and visual examination of HMA cores have proven to be adequate in the past.

3.6 STEP 5. PREPARE PRELIMINARY ESTIMATES (OPTIONAL)

Preliminary cost estimates should be prepared for the following:

- Pre-overlay repairs
- Volume of concrete needed for adjustments to profile grade and/or cross-slope
- Mitigation of vertical constraints:

- Adjustment/replacement of all safety barriers (if necessary)
- Modification of driveways and intersections
- Treatment of safety slopes and extension of drainage structures (if necessary)
- Adjustment of storm sewer inlets, manholes, utility structures, etc.
- Preparation of transitions at bridges
- Preparation of transitions at underpasses
- Preparation of transitions at project limits

Many of these vertical constraints are common to alternative designs. Therefore, the cost estimates should not be viewed as additional costs associated with a BCOA design but should be used in evaluating a BCOA against alternative designs.

3.7 STEP 6 FINAL REPORT WITH DESIGN RECOMMENDATIONS

This procedure is specific to evaluating whether an existing HMA is an appropriate candidate for a BCOA design. With experience, this process becomes straightforward. The primary considerations for determining whether an existing BCOA is appropriate are summarized as follows:

- Determine whether a minimum of 3 in. of HMA in fair to good condition will remain in place prior to construction of the BCOA
- Verify by coring that no remaining HMA layers are stripped or susceptible to stripping

BCOA designs have been placed on existing HMA pavements with highly variable physical and material properties and have exhibited good performance (see Appendix D for details). Therefore, until the physical properties of existing HMA pavements have been correlated to BCOA performance, there is no need for in-depth laboratory testing. A thorough visual examination, coring, and sound engineering judgement are all that is required to determine whether an existing HMA pavement is a candidate for a BCOA design.

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APPENDIX A: EXAMPLE SUMMARY REPORT FROM DESK REVIEW OF HISTORICAL RECORDS

Historical Records Review

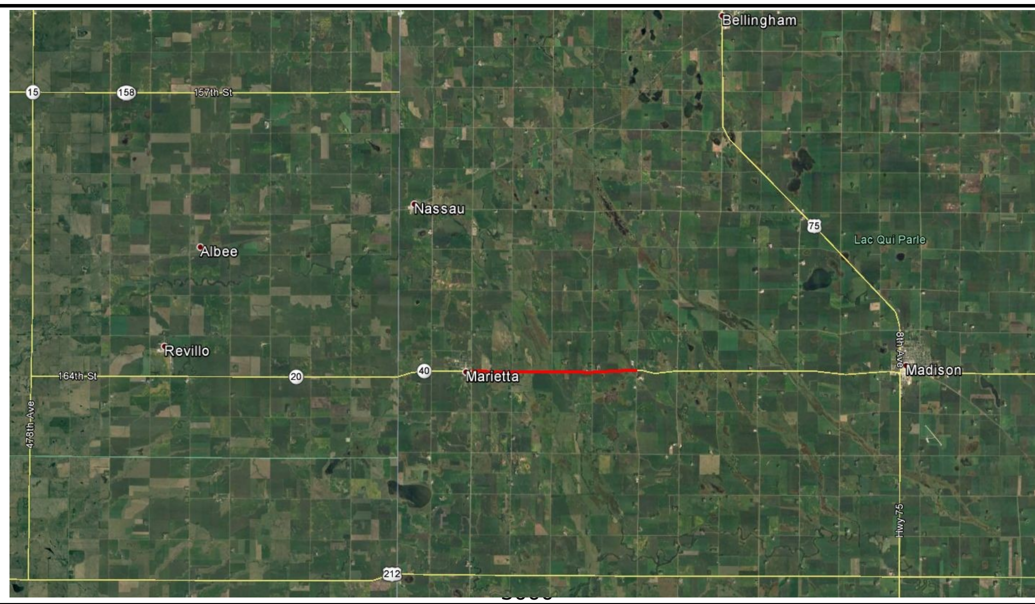
MnDOT BCOA Selection Procedure

Route: Example TH-40

Proposed Project Limits: From the intersection 161st Ave west to 121st Ave

Traffic: 4,000 AADT (2015)(both directions)(15% trucks)

Project Location:



Soils: A-3, fine sand

Other Materials Properties: Granular base circa 1952 comprised of pit run gravel

Environmentally Sensitive Areas: None noted in historical records

Pavement Quality Index (PQI):

PQI	Year
2.6	2015
3.2	2010
3.8	2005

Pavement Section:

Year	Commentary	Typical Section
1963	Original Construction - 22' wide; compacted subgrade, 6" gravel base & 4" HMA	4" HMA 6" Gravel Base Compacted Subgrade
1975	HMA overlay 1.5"	1.5" HMA (1975) 4" HMA (1963) 6" Gravel Base (1963) Compacted Subgrade
1983	Crack seal	

Historical Records Review

MnDOT BCOA Selection Procedure

1994	Mill 1.5" and HMA overlay 4"; widen to 26' shoulders consist of 4" HMA on 6" granular base	4" HMA (1994)
		4" HMA (1963)
		6" Gravel Base (1963)
		Compacted Subgrade

2001	HMA overlay 2"	2" HMA (2001)
		4" HMA (1994)
		4" HMA (1963)
		6" Gravel Base (1963)
		Compacted Subgrade

2009	Crack seal	

2012	HMA overlay 3"	3" HMA (2012)
		2" HMA (2001)
		4" HMA (1994)
		4" HMA (1963)
		6" Gravel Base (1963)
		Compacted Subgrade

APPENDIX B:

EXAMPLE CHECKLIST FOR VISUAL SURVEY

Visual Examination Checklist

MnDOT BCOA Selection Procedure

Route: Example TH-40
Proposed Project Limits: From the intersection 161st Ave west to 121st Ave
Traffic: 4,000 AADT (2015)(both directions)(15% trucks)

Distress Survey: Quantify and note locations
- ☑ Fatigue cracking (% of wheel path area)
- ☑ Longitudinal cracking in the wheel path (ft/mi)
- ☑ Reflective cracking in composite pavements (width of crack)(in)
- ☑ Transverse crack spacing (ft)
- ☑ Mean depth of rutting in both wheel paths (in)
- ☑ Shoving (% of wheel path area)

Subgrade Support and Drainage Conditions: Quantify and note locations
- ☑ Determine if longitudinal edge drains are present
- ☑ If present, are lateral drains open or clogged
- ☑ Note all areas of long wavelength profile variability
- ☑ Is FWD data available?
- ☑ Note all extents of pre-overlay repair needed

Profile Grades and Cross-Slope: Quantify and note locations
- ☑ Vertical curves needing correction for sight distance
- ☑ Existing cross-slope in tangent sections
- ☑ Existing cross-slope in curves

Vertical Constraints: Quantify and note locations
- ☑ Driveways
- ☑ Intersections
- ☑ Curb and gutter
- ☑ Storm sewer inlets
- ☑ Drainage structures – pipes and culverts
- ☑ Guard rail, barrier rail, cable barrier, parapets, etc.
- ☑ Overhead clearances
- ☑ Bridges

Shoulder Areas and Safety Slopes: Quantify and note locations
- ☑ Main lane and shoulder width(s)
- ☑ Shoulder support conditions
- ☑ Safety slopes - measure existing slope and horizontal distance from the edge of pavement to the ditch

APPENDIX C:

COMPARING PROPOSED PAVEMENT STRUCTURE AND EXISTING ROADWAY DATA

INTRODUCTION

A thorough understanding of the existing pavement conditions (hot-mix asphalt [HMA] thickness, base/subbase thickness, and support values) helps determine whether a bonded concrete overlay of existing asphalt (BCOA) is appropriate for a section of roadway and, when BCOA is appropriate, is valuable in making design decisions. Coring the existing pavement is mandatory (in each lane at 1/2-mile intervals). These cores are used to determine the thickness of the existing pavement, which can affect the maximum milling thickness, and are used to evaluate whether any HMA layers are stripped or have the potential for stripping. Because HMA is a good reflector of underlying support conditions, a visual examination of the pavement surface is almost always adequate for determining the following:

- Location and extent of pre-overlay repairs required
- Location and extent of subgrade support testing that may be necessary (e.g., dynamic cone penetrometer [DCP], California bearing ratio [CBR], resilient modulus) when indicated by spot locations that are distressed and merit further investigation to ensure that the existing pavement will serve as an acceptable candidate for a BCOA

In cases where data sets are available for ground penetrating radar (GPR) and falling weight deflectometer (FWD) testing, or when the visual inspection indicates that there are areas lacking subgrade support, GPR data can be used to estimate the HMA and subbase thicknesses and FWD data can be used to estimate subgrade support values. These data sets are not required to design a BCOA, but they may provide additional insight into the existing pavement conditions.

This appendix focuses on how to develop X-Y plots of the available data that have already been processed (i.e., data processing steps are not included). These plots can be used in conjunction with other observations and sound engineering judgment to assist in determining whether a section of pavement is a candidate for a BCOA.

FORMATTING EXISTING PAVEMENT DATA

Of interest are processed data sets representing the following:

1. Pavement thickness obtained through coring and/or GPR
2. Base and subbase thickness obtained through coring and/or GPR
3. Support conditions (materials test results and/or back-calculations from FWD testing)

All data should first be imported and/or entered into a computerized spreadsheet with graphing capabilities and then organized in tabular format sorted by a common longitudinal reference (e.g., milepost, reference point, or station). For the purposes of this report, all data presented are for one lane based on a hypothetical project two miles in length.

GRAPHING AND INTERPRETATION OF THICKNESS CORING DATA

Core data are entered into a computerized spreadsheet as shown in Table C-1.

Table C-1. Tabular thickness data from coring

Longitudinal Reference (milepost)	Lane 1 HMA Verification Core Thickness by 0.50-Mile Segments (in.)	Lane 1 Base & Subbase Verification Core Thickness by 0.50-Mile Segments (in.)	HMA Surface (in.)	Milling Depth (in.)	Remaining Asphalt ≥ 3 in
0.0000	-7.20	-13.50	0	-4.00	-7.00
0.5000	-11.50	-20.50	0	-4.00	-7.00
1.0000	-14.20	-23.00	0	-4.00	-7.00
1.5000	-10.10	-21.80	0	-4.00	-7.00
2.0000	-7.90	-14.50	0	-4.00	-7.00

Referring to Table C-1, the following should be noted:

- Cores should be taken at 1/2-mile intervals from each lane (with additional columns needed as lanes are added to the table)
- Thickness values are entered as negative values
- Columns D, E, and F represent the following:
    - HMA surface should always be set to 0 (zero)
    - Milling depth is a user input
    - Remaining Asphalt ≥ 3 is the sum of the milling depth minus 3 in.

Next, the coring data from Table C-1 are plotted on an X-Y graph (see Figures C-1 and C-2).

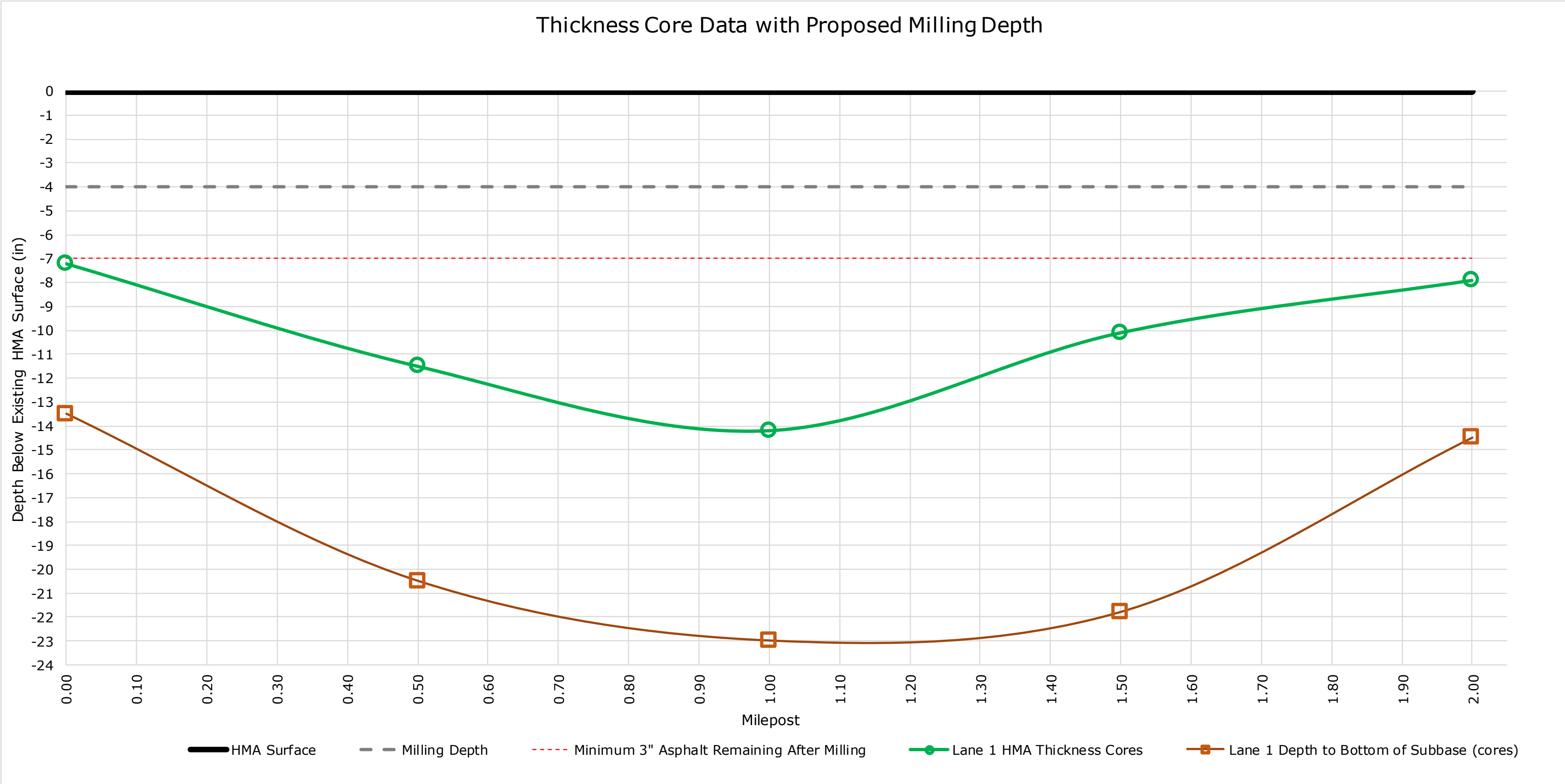


Figure C-1. HMA and subbase thickness data plotted with proposed milling depth (4 in.).

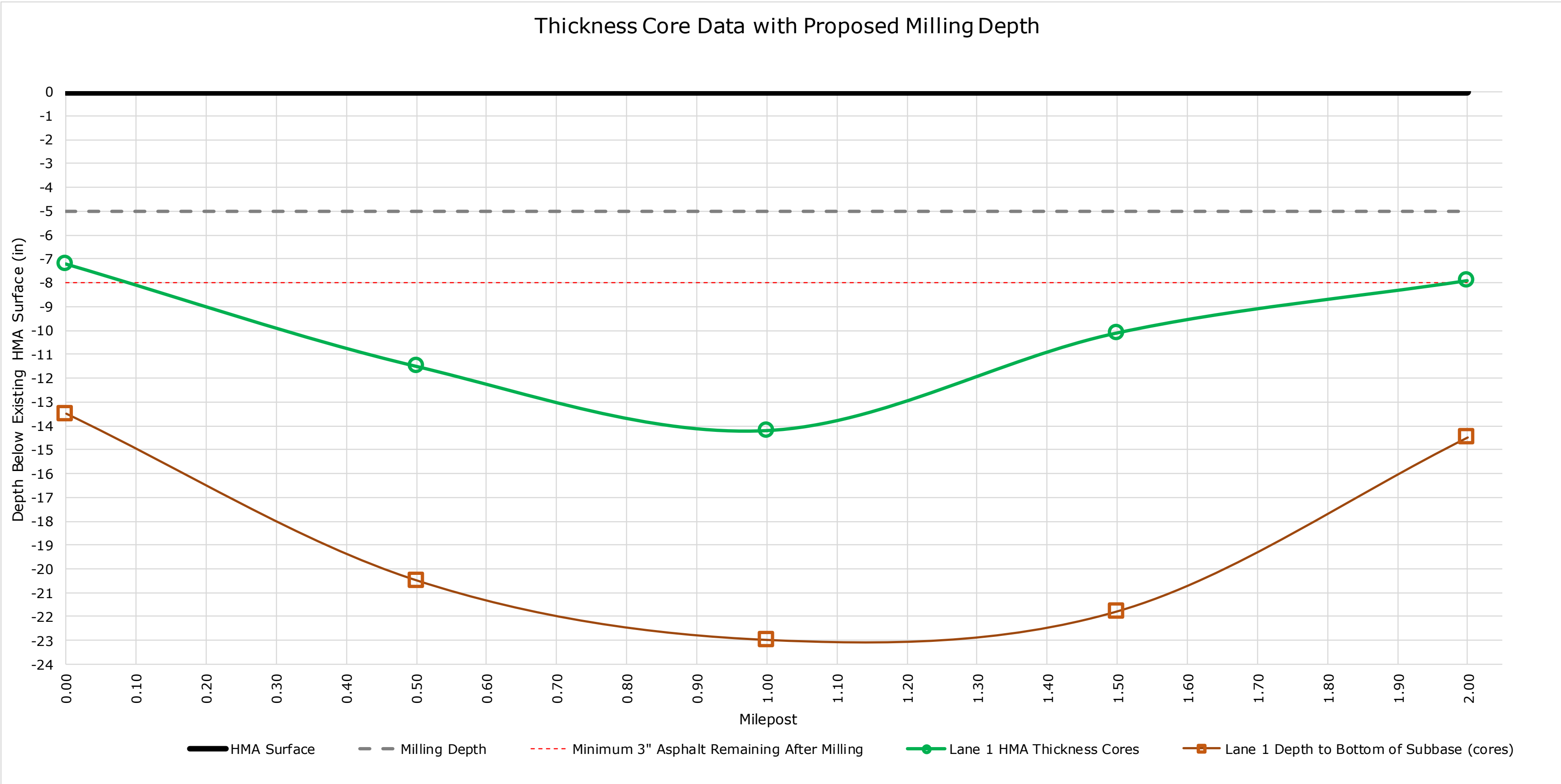


Figure C-2. HMA and subbase thickness data plotted with proposed milling depth (5 in.).

Maximum Milling Thickness (Ensuring that a Minimum of 3 in. of HMA Remains)

Determining whether milling the existing HMA is required for the construction of a BCOA is an important design detail. Milling may be necessary for numerous reasons:

- Minimize the change in profile grade
- Remove gross surface irregularities and cracking (rutting, shoving, and top-down longitudinal cracking) to optimize the volume of concrete required to construct the BCOA
- Remove surface distresses in the HMA leaving the remaining pavement in good or better condition
- Remove stripped layers within the HMA pavement structure
- Enhance the bond interface when the surface of the HMA contains excess asphalt binder

Note that milling is not required for all BCOA projects. When milling is necessary, it is imperative that a minimum of 3 in. of sound HMA remains after milling. This is a requirement for the proper design and performance of BCOAs. If it is determined that milling of the existing HMA is needed, the depth of milling should be based upon rut depth, visual examination of pavement cores, and/or project geometric constraints (raising the profile grade). The X-Y plot of existing pavement data can be used to evaluate whether the proposed milling depth will preserve a minimum thickness of 3 in. of HMA pavement.

Referring to Figure C-1, the dashed red line (minimum 3 in. asphalt remaining after milling) is compared to the HMA thickness cores; when the dashed red line plots above both data sets, as shown, the design can move forward at the proposed milling thickness. In contrast, the following can be inferred from Figure C-2, which shows a milling depth of 5 in.:

1. When compared to the core thickness at milepost 0.00, the milling depth of 5 in. leaves less than 3 in. of HMA remaining. For cases such as this, the milling depth should be adjusted in the design.
2. At milepost 2.00, both data sets plot very close to the dashed red line, which indicates that the minimum 3 in. of HMA remains after milling. Additional coring should be taken at 0.10-mile intervals from milepost 1.5 to milepost 2.0 in each lane to verify the actual HMA thickness, or the milling depth should be adjusted in the design.

Although GPR data are not necessary to design a BCOA, when available these data may add further insight into the existing pavement depth. An example of processed data from a GPR data set is shown in Table C-2.

Table C-2. Formatted tabular data for GPR thickness

GPR Reading Distance (ft)	Longitudinal Reference (milepost)	Lane 1 GPR HMA Thickness (in.)	Lane 1 GPR Base & Subbase Thickness (in.)	Lane 1 Moving Average of GPR HMA Thickness (in.)	Lane 1 Moving Average of GPR Base and Subbase Thickness (in.)
0.00	0.0000	7.41	5.53		
1.19	0.0002	6.72	4.97		
2.38	0.0005	7.31	5.71		
3.57	0.0007	7.03	5.31		
4.76	0.0009	6.81	5.15		
5.95	0.0011	6.79	5.07		
7.14	0.0014	6.94	5.42		
46.41	0.0088	8.28	5.95		
47.60	0.0090	7.90	5.56		
48.79	0.0092	9.03	5.81	-7.59	-13.15
49.98	0.0095	8.84	5.38	-7.62	-13.18
51.17	0.0097	8.81	4.97	-7.67	-13.23
52.36	0.0099	8.68	5.14	-7.70	-13.25
53.55	0.0101	7.99	5.27	-7.73	-13.27
54.74	0.0104	7.81	5.43	-7.75	-13.30
55.93	0.0106	7.94	5.69	-7.78	-13.35
57.12	0.0108	8.83	6.03	-7.82	-13.41
58.31	0.0110	7.73	5.70	-7.83	-13.41
59.50	0.0113	8.76	6.25	-7.86	-13.45
60.69	0.0115	8.34	5.70	-7.86	-13.45
61.88	0.0117	7.97	5.80	-7.86	-13.46
63.07	0.0119	7.81	5.39	-7.85	-13.44
64.26	0.0122	8.18	5.27	-7.85	-13.44
65.45	0.0124	8.10	5.65	-7.84	-13.43
66.64	0.0126	8.02	5.85	-7.85	-13.42
67.83	0.0128	7.57	5.21	-7.84	-13.39
69.02	0.0131	7.84	5.87	-7.81	-13.35
70.21	0.0133	7.60	5.12	-7.80	-13.36
71.40	0.0135	7.64	5.57	-7.80	-13.38
72.59	0.0137	7.95	6.09	-7.82	-13.40
73.78	0.0140	8.88	7.20	-7.85	-13.47
74.97	0.0142	8.64	6.94	-7.88	-13.54
76.16	0.0144	9.35	7.61	-7.94	-13.65
77.35	0.0146	9.93	7.83	-8.00	-13.78

Referring to Table C-2, the following should be noted:

- GPR data readings are at approximately 1.2 ft intervals.
- The entire table for 2 miles of project length consists of more than 8,000 rows of data.

- For clarity, only a subset of the data is presented. The rows from milepost 0.0014 through milepost 0.0088 have been hidden and only data for Lane 1 are shown.
- A moving average with a base length of approximately 50 ft has been calculated for the GPR data and converted to a negative value (two right-most columns). This results in a null value for the moving average for the first 42 rows of data.

The data from Table C-2 should be plotted on an X-Y graph with the data from Table C-1 (see Figure C-3).

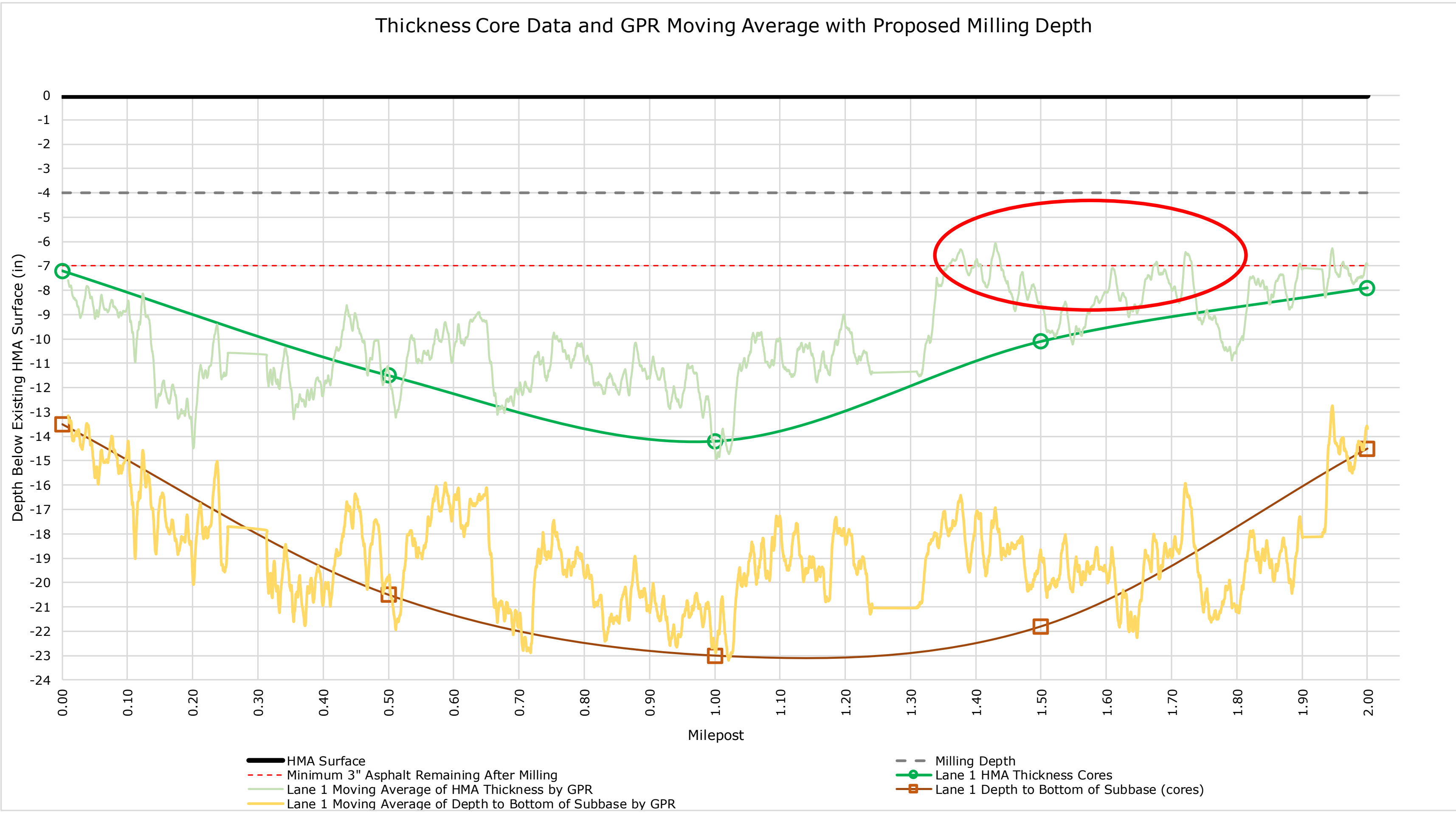


Figure C-3. Lane 1 GPR thickness data plotted with core thickness data.

Referring to Figure C-3, the following should be noted:

- The line for Lane 1 Moving Average of HMA Thickness by GPR crosses over the Minimum 3 in. Asphalt Remaining After Milling line from approximately milepost 1.35 to milepost 1.45 and around ±mileposts 1.70 and 1.90 (red oval).
- Additional coring should be taken at 0.10-mile intervals from milepost 1.3 to milepost 2.0 in each lane to verify the actual HMA thickness, or the milling depth should be adjusted in the design.

Design Adjustments for Isolated Areas Where a Minimum 3 in. of Remaining Asphalt Cannot Be Maintained

The examples shown in Figures C-1, C-2, and C-3 identify areas that require additional coring to verify the existing asphalt thickness. If, after coring, the existing asphalt in isolated areas (less than 500 ft in length) is found to be deficient for the proposed milling depth and the milling depth cannot be adjusted due to profile grade restrictions, there are design options to consider:

1. Increase the milling depth through the isolated areas by 2 in. (subbase/subgrade layers may be exposed), and thicken the concrete overlay thickness by 2 in. through these areas. The thicker concrete pavement section will compensate for the lack of support conditions.
2. Keep the milling depth and overlay thickness as designed through these isolated areas. This will increase the risk of early cracking from loading due to the inability of the remaining asphalt to provide any structural contribution to the pavement section.

Determining Whether Multiple Pavement Designs May Be Necessary

In some cases, existing pavement conditions may drive the need to consider more than one BCOA design. This is commonly due to the proposed project spanning different pavement sections, or in some cases there may be a distinct change in geotechnical conditions within the project. The X-Y plot shown in Figure C-4 indicates changes in the typical section near milepost 1.50.

In this hypothetical case, some of the design options that may be considered include the following:

- Reduce the milling depth from milepost 1.50 to milepost 2.00 to ensure that a minimum of 3 in. of HMA is maintained after milling.
- Increase the concrete thickness from milepost 1.50 to milepost 2.00. This may be necessary because the BCOA design from milepost 0.00 to milepost 1.50 is based on approximately 6 in. of HMA remaining after milling.

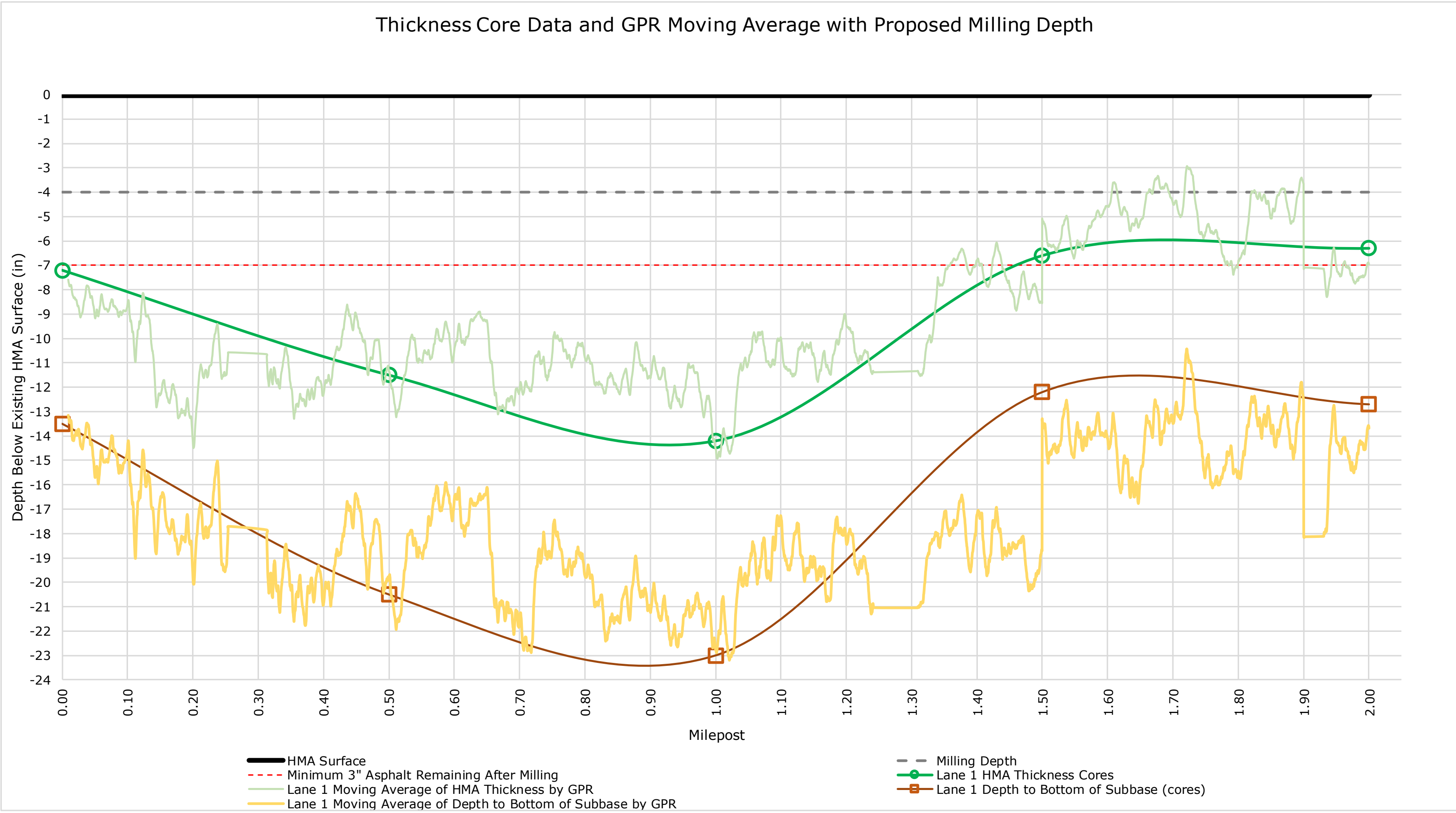


Figure C-4. X-Y plot showing a change in typical section at milepost 1.50.

Plotting and Interpreting Falling Weight Deflectometer Data

In general, concrete pavements, including BCOA designs, are insensitive to subgrade strength. In contrast, bituminous pavements are sensitive to subgrade support. Because a BCOA design depends upon the HMA as a structural component in good condition, it may be necessary during the design phase to investigate distressed areas identified in a visual inspection that need further confirmation to determine whether a BCOA design is appropriate and/or whether isolated areas of pre-overlay repair may be necessary. Often, distresses in the surface of HMA pavements reflect areas that are lacking in subgrade strength. These areas are typically clearly identified during a visual inspection of the pavement surface.

The pavement design should be based upon an estimate of support values that is representative of the overall project. Isolated areas of distress should be repaired prior to placement of the bonded concrete overlay. Borderline projects present a challenge when determining whether an asphalt pavement is a candidate for BCOA. Collection of additional FWD and/or DCP data can be justified when there is a good possibility that cost-effective repairs can be made to the asphalt pavement (including milling) that can restore the pavement to a good condition.

When subgrade support data are justified or otherwise available, these data can be plotted on a X-Y graph to help confirm visual observations and identify areas that may potentially be lacking in subgrade support. Tabular FWD data are shown in Table C-3.

Table C-3. Tabular FWD data by 0.10-mile segment

Longitudinal Reference (milepost)	FWD Subgrade Modulus by 0.10-Mile Segment (psi)
0.0000	7,827
0.1000	13,160
0.2000	2,815
0.3000	10,213
0.4000	8,806
0.5000	5,870
0.6000	7,215
0.7000	9,410
0.8000	6,516
0.9000	7,303
1.0000	9,094
1.1000	7,194
1.2000	6,478
1.3000	9,285
1.4000	5,820
1.5000	9,720
1.6000	11,724
1.7000	13,617
1.8000	7,440
1.9000	12,149
2.0000	8,790

Subgrade Stability Issues

The line for FWD Subgrade Modulus by 0.10 Mile Segment in Figure C-5 shows typical results derived from FWD testing. Variability in the subgrade modulus should be expected. The data in Figure C-5 show an area at milepost 0.20 that should be further investigated to determine if a pre-overlay repair is necessary and the extent of this pre-overlay repair (undercut and base/subbase repair).

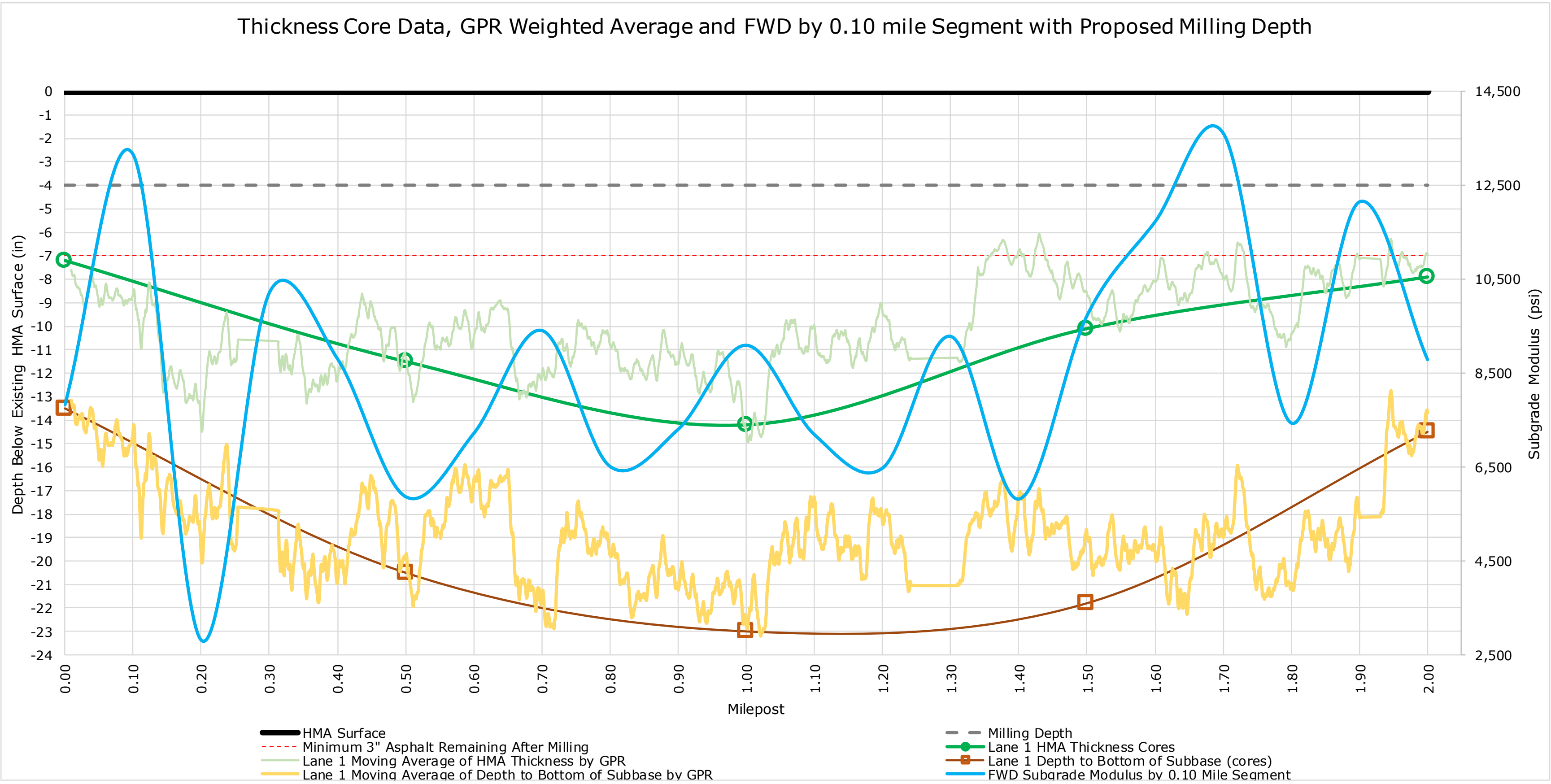


Figure C-5. X-Y Plot showing an area with potentially weak subgrade.

CONCLUSIONS

Graphical representations of existing pavement data on X-Y plots can be helpful in determining whether a BCOA design is appropriate for a specific section of roadway. They also provide useful insights for optimizing the BCOA designs. To be useful, these plots must be based upon good data that have been properly processed (i.e., proper sampling procedures followed and outliers removed). Additional sampling and/or on-site reviews should be conducted whenever the X-Y plots suggest that additional investigation is needed. The interpretation of these plots is not complex, but sound engineering judgment should be used before making any design decisions based upon these analyses.

APPENDIX D:

FIELD AND LABORATORY TESTING OF EXISTING HMA

TASK 3: ESTABLISH MINIMUM BCOA REQUIREMENTS OF THE REMAINING MIX PROPERTIES FOR THE HMA PAVEMENT

The goal of this task was to identify the hot-mix asphalt (HMA) mix volumetric properties that a HMA pavement should have for bonded concrete overlay of existing asphalt (BCOA). Appropriate HMA material property criteria and, when necessary, test procedures (modulus of elasticity, falling weight deflectometer [FWD], etc.) for the Minnesota Department of Transportation (MnDOT) to perform on existing asphalt pavements were to be identified as a means to better characterize the existing asphalt as a structural layer in a bonded concrete overlay application.

The data collected by MnDOT will be used to develop enhanced guidance regarding common design inputs for BCOA design procedures. Each section of the procedure will contain a number of graphs and photographs, as necessary, and dialog that fits the section subject matter. The format will follow MnDOT's Pavement Design Manual. Subject matter to be covered includes, but is not limited to, the following:

- Air voids
- Tensile stress ratio
- Modulus of elasticity (stiffness)
- Fatigue resistance
- Aging of asphalt mixtures
- Balance between high stiffness values versus high shear creep (bonded overlays)
- Moisture susceptibility (stripping potential)

TASK 3 MEMORANDUM

The objective is to relate the distress observed in the BCOA sections evaluated to the properties and the condition of the existing asphalt. The following distresses that can develop in the BCOA as a result of the existing asphalt were identified under Task 1:

1. Reflective cracking or reflected distress
2. Uneven slabs (or migration) due to asphalt deformation
3. Premature fatigue cracking as a result of debonding or support loss due to stripping

Under Task 2, a list of tests that could be performed on cores retrieved from the existing asphalt pavement to determine the susceptibility of the BCOA to developing the three distresses identified above was generated. It was determined that binder grade, voids filled with asphalt (VFA), and the adjusted asphalt film thickness (AFT) would initially be determined under the Phase I testing. It was determined that it would be easier to establish a relationship between these fundamental parameters and the three distresses defined above than if performance-based tests were used. If there proved to be sufficient data to establish these relationships, additional performance-based testing would then be performed under the Phase II plan developed as part of Task 2. The results from this work are summarized below.

Asphalt characterization was performed on cores pulled from the six different BCOA projects described in Table D-1. The performance data obtained from these BCOAs are summarized in Table D-2, while Table D-3 provides a summary of the condition of the asphalt pavement prior to the placement of the overlay. Tables D-3 through D-6 summarize the results for the visual inspection and laboratory testing performed on the cores.

Table D-1. Project description

State	Roadway	Project Label	Year Overlay Built	Design	Saw & Sealed or Sealed	Dowel Dia. (in.)	Traffic	Remaining HMA Layers	Pre-overlay distress		
									Rutting (in.)	Trans. Crack. (crk/mi)	Fatigue Cracking (%)
MN	CSAH (02-622-31) 22 from CSAH 5E to TH47 (Nowthen in Anoka County)	MN-CSAH22	2011	6” - 6’x6’ & 6’x7’ (TxL)	AC	0	6,124 ADT (0.9 Million 20-yr design ESALs)	2011: 2” mill 1988: 1.5” HMA Wear 1988: 1.5” HMA Binder 1988: 2” HMA Base 1988: 5” CL-5A Aggregate Base 1987: 8” CL-4A Aggregate Base		Yes	
MN	(1380-74) I-35 from TH95 to 0.4 mi south of CSAH 9 (North Branch)	MN-I35	2009	6” - 6’x6’	Sealed	0	27,700 ADT (9.1 Million 20-yr design ESALs)	2009: 4” mill 1987: 1” HMA wear; 2.25” HMA binder; 1” min. leveling 1969: 1.5” HMA wear; 2” HMA binder; 4.5” HMA base; 4” bituminous treated base; CL 5 1967: 3” stabilized SGM with asphalt emulsion SS-1; 9” select granular material (SGM)	0.24	Yes	
MI	Patterson Ave. 44th to 36th St. (near Ford Int. Airport)	MI-Pat	2006	4”-4.5” 4’x4’ micro fibers	Not Sealed	0	25,730 ADT	1987-88: 6-7” HMA	Approx. 1 in		
IA	IA 21, Section 43	IA-21(A)	1994	4” - 6’x6’ micro fibers	Not Sealed (tight)	0	1,300 ADT 15% Trucks	1964: 3” HMA base 1961: Chip seal; 7” cement treated sand; 6” granular material; 24” subgrade select soil treatment		Yes	
IA	IA 21, Section 41	IA-21(B)	1994	4”-4’x4’ micro fibers	Not Sealed (tight)	0	1,300 ADT 15% Trucks	1964: 3” HMA base 1961: Chip seal; 7” cement treated sand; 6” granular material; 24” subgrade select soil treatment		Yes	

State	Roadway	Project Label	Year Overlay Built	Design	Saw & Sealed or Sealed	Dowel Dia. (in.)	Traffic	Remaining HMA Layers	Pre-overlay distress		
									Rutting (in.)	Trans. Crack. (crk/mi)	Fatigue Cracking (%)
MO	US 60	MO-US60 (near Neosho)	1999	4”-4’x4’			13,000 ADT 15% Trucks	1999: 1.75”Milled; 2”-4” of HMA 1974: 3” Type B base; 4.5“ Type C 1960: 5” HMA	Heavy Rutting		Minimal (if any)

Table D-2. Overlay performance

Project Label	Year Survey	% Panel Cracked	Faulting (in.)	Avg. IRI (in./mile)	Initial IRI (in./mile)	Slab Shifting	Transverse Joints Mismatched	Trans. Reflect. Cracks (avg. distance apart, ft)	Trans. Fatigue Cracks (%)	Long. Cracks (%)	Diag. Cracks (%)	Corner Cracks (%)	Comments
MN-CSAH22	2015	<1	0.02	66.89	54.4		No						Smooth ride, no notable distress; Shoulder transverse cracking
	2015	<1											
	2011	0	-	54.4									
MN-I35	2015	2.5% (1-mile survey)	-	77.1	47.24		No	?		some			Mostly reflective trans. cracking with a few longitudinal (Direction D is 10 in/mile higher than I throughout the section)
	2014			66.9									
	2013			62.5									
	2012			58.8									
	2011			57.7									
	2010			55.9									
MI-Pat	2016 unofficial						No						Shattered slabs to be replaced mainly at headers and gaps; little room for expansion causing pressure, blowups, or spalling. Expansion joints are supposed to be installed
IA-21 Section 43	2004						No	See comment	2.3	0.3	0		Trans. reflective cracks developed every 60 to 100 ft but distress survey did not differentiate between reflective and trans. fatigue cracks.

Project Label	Year Survey	% Panel Cracked	Faulting (in.)	Avg. IRI (in./mile)	Initial IRI (in./mile)	Slab Shifting	Transverse Joints Mismatched	Trans. Reflect. Cracks (avg. distance apart, ft)	Trans. Fatigue Cracks (%)	Long. Cracks (%)	Diag. Cracks (%)	Corner Cracks (%)	Comments
	2002								2.3	0.9	0		
	1999								2.3	0.4	0		0.21% joints spalled
	1995								1.8	0	0		
IA-21 Section 41	2004							See comment	0.7	0.2	0		Trans. reflective cracks developed every 60 to 100 ft but distress survey did not differentiate between reflective and trans. fatigue cracks.
	2000								0.7	0.2	0		
	1999	<1							0.7	0	0		2.35% trans & 0.43% long provided but doesn’t match report.
	1996								0.7	0	0		
	1995								0.6	0	0		
MO-US60	2015	2.8					No						
	2009	2.2											

IRI = International Roughness Index

Table D-3. Core information

Project Label	Year Cored	# of Cores	Core #	Test Core Label	Asphalt Thickness (in.)	PCC Thickness (in.)	Depth of Stripping (in.)	Additional Information
MN-CSAH22	2015	3	1	Anoka 1	4.63	6.13		May have lost bond from coring between PCC/Asphalt
			2	Anoka 2	4.5	6.38		Bonded
			3	Anoka 3	1.88	6.5		Bonded
MN-I35	2015	3	1	MN-A1	9.5	6.18		PCC/Asphalt bond intact, separation between top and second layer of asphalt lifts
			2		3	6.38		May have lost bond from coring between PCC/Asphalt
			3	MN-A2	13	6.38		May have lost bond from coring between PCC/Asphalt
MI-Pat	2016	4	1		-	4.06		PCC/Asphalt unbonded
			2	MI-1	6.16	3.94		May have lost bond from coring
			3	MI-2	5.42	4.36		May have lost bond from coring between PCC/Asphalt
			4	MI-3	6.02	4.48		May have lost bond from coring between PCC/Asphalt
IA-21 Section 43	2016	3	1		1.16	4.56		Cored at Sta. 2570 (NB); 1.16” asphalt bonded to PCC; crumbled Asphalt beneath (Dry)
			2		0.5	4.5		Cored at Sta. 2570 (NB) at intersection of trans. and long. jts.; 0.5” of asphalt bonded to PCC; joint appears to have activated through Asphalt; crumbled asphalt beneath (Dry)
			3	IA-1	4.09	4.84		Cored at Sta. 2570 (NB); debonded between asphalt layers 1 and 2
IA-21 Section 41	2016	3	1		3.89	4.50		Cored at Sta. 2558 at intersection of long. crack (tight) and trans. jts. at mid lane. May have lost bond from coring; joint appears to have activated through top layer of asphalt; debonded between asphalt layers 1 and 2
			2		-	4.19		Cored at Sta. 2558; debonded between PCC/Asphalt; crumbled asphalt beneath (dry)
			3	IA-1	3.39	4.16		Cored at Sta. 2558; Unclear if debonding at PCC/Asphalt interface was caused by coring; debonded between asphalt layers 1 and 2
MO-US60	2015	6	1		5.625	4.625		Bonded, good condition; cored at Sta. 19+574
			2	MO-1	5.5	4.75		Bonded, good condition; cored at Sta. 19+574

Project Label	Year Cored	# of Cores	Core #	Test Core Label	Asphalt Thickness (in.)	PCC Thickness (in.)	Depth of Stripping (in.)	Additional Information
			3	MO-2	5.5	4.75		Bonded, good condition; cored at Sta. 19+574
			4		4.75	4.75	0.25	May have lost bond from stripping (not at PCC/Asphalt interface) Sta. 19+623
			5	MO-3	4.75	5	0.25	May have lost bond from stripping (not at PCC/Asphalt interface), coring process increased level of debonding; cored at Sta. 19+623
			6	MO-4	4.5	4.75		May have lost bond from coring between PCC/Asphalt; cored at Sta. 19+623

Table D-4. Core testing: binder

Project Label	Core #	Core Label	Grade	G*/sin (δ) (kPa)	BBR S (MPa)	BBR m-value	Gb (assumed)	Pb (%)	Pba (%)	Pbe (%)
MN-CSAH22	1	Anoka 1	PG 88-16	2.74	112	0.326	1.035	4.4	0.46	4.0
	2	Anoka 2	PG 82-16	3.08 [a]	100	0.327	1.035	4.6	0.16	4.4
	3	Anoka 3	PG 94-10	3.76	133	0.313	1.035	4.2	0.42	3.8
MN-I35	1	MN-A1	PG 70-22	2.60	121	0.361	1.035	6.2	1.20	5.1
	3	MN-A2	PG 64-28	2.20 [a]	143	0.345	1.035	4.9	0.54	4.4
MI-Pat	2	MI-1	PG 76-16	3.21 [a]	77	0.317	1.035	4.4	0.30	4.1
	3	MI-2	PG 76-16	2.66 [a]	62	0.336	1.035	4.5	0.35	4.2
	4	MI-3	PG 76-16	3.68 [a]	87	0.329	1.035	4.2	0.32	3.9
IA-21, Section 43	3	IA-1	PG 82-22	2.59	74	0.312	1.035	5.8	0.27	5.5
IA-21, Section 41	3	IA-2	PG 82-16	3.43	32	0.354	1.035	5.3	0.14	5.2
MO-US60	2	MO-1	PG 82-22	4.08	38	0.313	1.035	3.4	-1.46	4.8
	3	MO-2	PG 82-22	4.56	38	0.312	1.035	5.1	-0.49	5.6
	5	MO-3	PG 82-22	5.87	50	0.309	1.035	4.6	-0.53	5.1
	6	MO-4	PG 88-16	4.13	45	0.307	1.035	4.7	-0.38	5.1

[a] Grade temperature DSR performed on the same sample

G* = Dynamic shear rheometer (DSR) complex shear modulus

δ = DSR phase angle

BBR S = Bending Beam Rheometer (BBR) creep stiffness

BBR m-value = BBR slope

Gb = Specific gravity of binder

Pb = Percent binder

Pba = Percent binder absorbed by aggregate

Pbe = Effective asphalt content

Table D-5. Core testing: aggregate

Project Label	Core #	Core Label	Gradation: Percent Passing (%)												Gse	Ps (%)
			5/8”	1/2"	3/8"	#4	#8	#10	#16	Calc #30	#40	#50	#100	#200		
MN-CSAH22	1	Anoka 1	95	89	82	70	61	59	52	36	31	22	10	6.2	2.702	95.6
	2	Anoka 2	98	91	86	72	62	59	51	33	28	20	9	5.7	2.681	95.4
	3	Anoka 3	96	89	82	70	60	58	51	35	30	21	10	6.0	2.699	95.8
MN-I35	1	MN-A1	100	96	88	71	56	53	43	24	19	13	7	4.1	2.755	93.8
	3	MN-A2	97	91	83	69	57	54	47	31	26	18	9	5.6	2.708	95.1
MI-Pat	2	MI-1	97	94	88	67	52	49	42	31	28	20	8	5.1	2.691	32.1
	3	MI-2	95	93	88	67	52	49	43	31	28	20	8	5.0	2.694	32.0
	4	MI-3	94	90	84	64	49	46	41	30	27	19	7	4.8	2.692	30.3
IA-21, Section 43	3	IA-1	100	93	86	68	53	50	42	27	23	16	9	7.3	2.689	94.2
IA-21, Section 41	3	IA-2	100	96	89	69	55	52	44	28	23	16	9	6.9	2.680	94.7
MO-US60	2	MO-1	100	99	90	60	38	36	33	26	24	19	11	5.0	2.573	31.3
	3	MO-2	100	99	89	58	38	36	33	26	24	19	10	5.0	2.637	30.7
	5	MO-3	100	100	91	58	35	33	30	25	23	18	9	4.6	2.634	28.6
	6	MO-4	100	99	90	58	35	33	30	25	23	18	9	4.5	2.644	28.5

Ps= Percent stone

Gse = Apparent specific gravity

Table D-6. Core testing: HMA mixture

Project Label	Core #	Core Label	Air Voids (%)	Oven Dry Density (%)	Gmm	Gmbair	Fines/Pbe	Gmboven	VMA (%)	VFA (%)	AFT (microns)	Adj. AFT (microns)
MN-CSAH22	1	Anoka 1	12.4	87.6	2.523	2.222	1.6	2.211	19.7	37.1	5.4	6.0
	2	Anoka 2	10.4	89.6	2.498	2.248	1.3	2.238	18.3	43.1	6.4	6.8
	3	Anoka 3	12.4	87.6	2.528	2.226	1.6	2.215	19.3	35.8	5.3	5.8
MN-I35	1	MN-A1	1.8	98.3	2.452	2.459	0.8	2.452	14.3	87.4	9.8	9.7
	3	MN-A2	4.9	95.1	2.509	2.394	1.3	2.385	14.1	65.1	6.7	7.0
MI-Pat	2	MI-1	7.7	92.3	2.514	2.331	1.2	2.320	15.4	50.1	6.5	6.7
	3	MI-2	6.4	93.6	2.513	2.359	1.2	2.351	14.5	55.9	6.7	6.9
	4	MI-3	8.3	91.7	2.522	2.329	1.2	2.313	15.5	46.5	6.5	6.6
IA-21, Section 43	3	IA-1	6.9	93.1	2.461	2.310	1.3	2.291	17.7	60.9	8.2	8.6
IA-21, Section 41	3	IA-2	9.0	91.4	2.472	2.260	1.3	2.250	18.5	51.2	7.8	8.2
MO-US60	2	MO-1	8.1	91.9	2.449	2.259	1.0	2.252	13.4	49.0	7.7	7.9
	3	MO-2	7.5	92.5	2.444	2.272	0.9	2.261	16.6	49.0	9.4	9.6
	5	MO-3	8.3	91.7	2.459	2.260	0.9	2.255	16.3	50.1	9.1	9.1
	6	MO-4	8.5	91.5	2.464	2.260	0.9	2.255	16.7	55.9	9.1	9.1

Gmm = Maximum specific gravity

Gmb_{air} = Bulk specific gravity air dried

Gmb_{air} = Bulk specific gravity oven dried

VMA = Voids in mineral aggregate

VFA = Voids filled with asphalt

AFT = Asphalt film thickness

Adj. AFT = Adjusted asphalt film thickness

The following is a review of the three distresses observed in the sections cored and a review of the laboratory results with respect to the observed distresses.

Reflective Cracking or Distress

Deteriorated cracks in the asphalt can result in reflective distress in the concrete overlay. In order to prevent this, pre-overlay repairs should be performed. Medium- or high-severity fatigue cracking should be repaired with full-depth patching. A non-deteriorated working crack in the existing overlay can result in a reflective crack, which propagates up into the concrete overlay. A non-deteriorated reflection crack has the potential to develop in the overlay if the flexural stiffness of the concrete layer is less than the flexural stiffness of the asphalt layer (Vandenbossche and Barman 2010, Vandenbossche et al. 2016). The flexural stiffness, D, of a layer is defined using Equation D-1.

$$Flexural\ Stiffness = D = \frac{Eh^3}{12(1-\mu^2)} \quad \text{(D-1)}$$

where E is the elastic modulus of the material, h is the thickness of the layer, and μ is the Poisson's ratio. The BCOA-ME design guide indicates whether a BCOA will be susceptible to reflection cracking based on the ratio of the flexural stiffness of the asphalt layer with respect to that of the concrete (Vandenbossche et al. 2016). The stiffness of the asphalt during winter months should be used when determining the flexural stiffness for the asphalt layer.

Transverse cracks were reported to have developed in the Minnesota and Iowa sections. No reflective distress was reported as a result of deteriorated transverse cracks in any of the BCOAs cored. Reflective cracking was reported to occur on I-35 in Minnesota. The development of reflective cracking was also documented on IA-21 in Iowa, which contains sections with many different BCOA designs. It is not clear from the report if the transverse cracks specifically reported for Sections 41 and 43 (included in this study) are reflective cracks or just fatigue cracking.

The flexural stiffness ratio was determined for each of the pavements cored. An elastic modulus of 4.5 million psi and a Possion's ratio of 0.2 were used for all concrete overlays. These represent typical values for Portland cement concrete paving mixtures. Typical values should also be used for the stiffness of the asphalt. The stiffness of the asphalt during winter months should be used because this is when reflective cracks are most likely to develop. A maximum stiffness of 2 million psi is commonly assumed for asphalt during cold months in cold climate regions. Because volumetric data were available for the sections included in this study, a master curve was developed for each data set so that the stiffness of the asphalt could be established as a function of the temperature of the asphalt and the loading rate. It should be noted that it is not necessary to generate a master curve for the existing asphalt mixture and that using a typical winter asphalt stiffness of 2 million psi is acceptable.

A master curve was generated for each of the asphalt pavements cored using the data from the laboratory testing so that the stiffness of the asphalt (dynamic modulus, E*) during the winter months could be estimated. All master curves were developed using the Witczak-Andrei equation (Equations D-2 and D-3) (ARA, Inc., ERES Consultants Division 2004).

$$\log(E*) = \delta + \frac{\alpha}{1+e^{\beta+\gamma \log t_r}} \tag{D-2}$$

where

E^* = Dynamic modulus (psi)
t_r = Time of loading at the reference temperature
δ = Minimum value of E^*
$\delta+\alpha$ = Maximum value of E^*
β, γ = Parameters describing the shape of the sigmoidal function

$\delta = 3.750063+0.02932\rho_{200}-0.001767(\rho_{200})^2- 0.002841\rho_4-0.058097V_a-0.802208\left[\frac{Vb_{eff}}{Vb_{eff} + V_a}\right]$
$\alpha = 3.971977 + 0.0021\rho_4 + 0.003985\rho_{38} – 0.000017\rho_{38}^2 + 0.005470\rho_{34}$
$\beta = -0.603313 - .393532 \log(\eta_{T_r})$
$\log(t_r) = \log(t) – c(\log(\eta) – \log(\eta_{Tr}))$
$\gamma = 0.313351$
$c = 1.255882$
t = Time of loading (reciprocal of load frequency)
η = Viscosity at the test temperature (MP)
η_{Tr} = Viscosity at the reference temperature (MP)
V_a = Air void content, %
V_{beff} = Effective bitumen content, % by volume
ρ_{34} = Cumulative % retained on the 3/4 in. sieve
ρ_{38} = Cumulative % retained on the 3/8 in. sieve
ρ_4 = Cumulative % retained on the No. 4 sieve
ρ_{200} = Cumulative % retained on the No. 200 sieve

Asphalt binder viscosity-temperature relationship (ARA, Inc., ERES Consultants Division 2004):

$$\log \log \eta = A + VTS \log T_r \tag{D-3}$$

where

η = Viscosity, cP
T_r = Temperature, Rankine
A = Regression intercept
VTS = Regression slope of viscosity temperature susceptibility

The inputs used for these equations are summarized in Table D-7.

Table D-7. Inputs used for Witczak-Andrei equation

Section	Core	Binder Grade	A (typical value)	VTS (typical value)	Air Voids (%)	Volumetric Effective Binder Content (%)	Passing ¾ (assumed) (%)	Passing 3/8 (%)	Passing #4 (%)	Passing #200 (%)
MN-CSAH22	1	PG 88-16*	8.785	-2.857	12.4	8.6	100	82	70	6.2
MN-CSAH22	2	PG 82-16	9.475	-3.114	10.4	9.6	100	86	72	5.7
MN-CSAH22	3	PG 94-10*	8.256	-2.659	12.4	8.2	100	82	70	6
MN I-35	1	PG 70-22	10.299	-3.426	1.8	12.1	100	88	71	4.1
MN I-35	3	PG 64-28	10.312	-3.440	4.9	10.2	100	83	69	5.6
MI-Pat	2	PG 76-16	10.015	-3.315	7.7	9.2	100	88	67	5.1
MI-Pat	3	PG 76-16	10.015	-3.315	6.4	9.6	100	88	67	5
MI-Pat	4	PG 76-16	10.015	-3.315	8.3	8.8	100	84	64	4.8
IA-21, Section 43	3	PG 82-22	9.209	-3.019	6.9	12.3	100	86	68	7.3
IA-21, Section 41	3	PG 82-16	9.475	-3.114	9	11.4	100	89	69	6.9
MO-US60	2	PG 82-22	9.209	-3.019	8.1	10.5	100	90	60	5
MO-US60	3	PG 82-22	9.209	-3.019	7.5	12.3	100	89	58	5
MO-US60	5	PG 82-22	9.209	-3.019	8.3	11.1	100	91	58	4.6
MO-US60	6	PG 88-16*	8.785	-2.857	8.5	11.1	100	90	58	4.5

Note: Typical A-VTS values are from the 2004 MEPDG documentation (Part 2, Chapter 2) and represent rolling thin-film oven (RTFO) values. Starred (*) values are extrapolated from the typical values provided in this documentation.

The stiffness versus temperature relationships and the master curve parameters for each core are provided in Figures D-1 through D-5 and Tables D-8 through D-12.

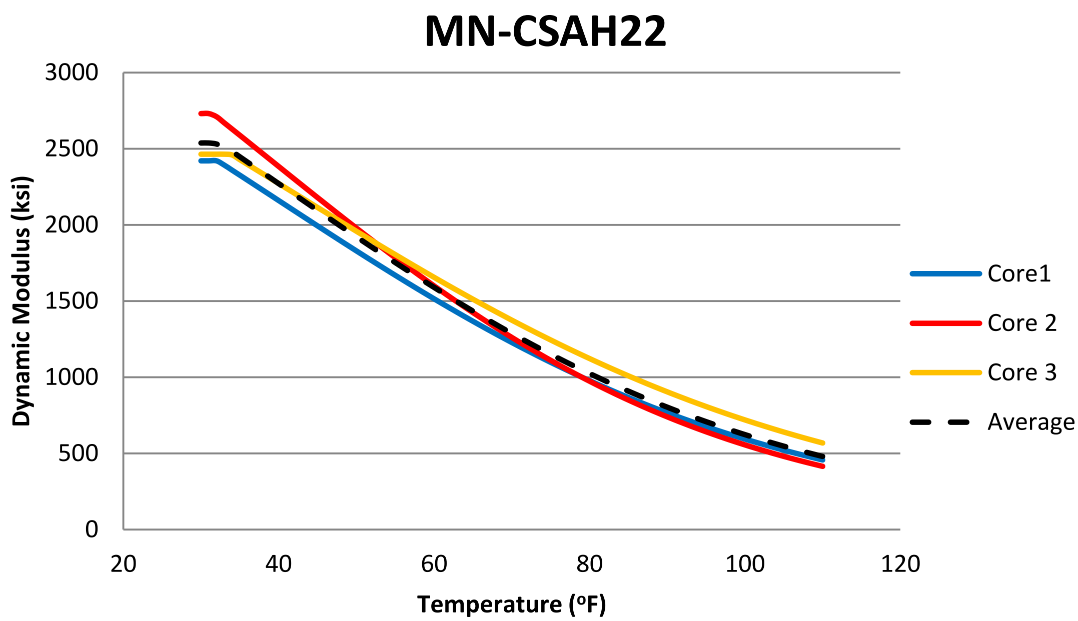


Figure D-1. Stiffness versus temperature curves for MN-CSAH

Table D-8. Master curve parameters for MN-CSAH

Core	Master Curve Parameter			
	Delta	Alpha	Gamma	Beta
1	2.73	3.87	0.3134	-1.4132
2	2.79	3.87	0.3134	-1.3225
3	2.74	3.87	0.3134	-1.5087
Average	2.75	3.87	0.3134	-1.4148

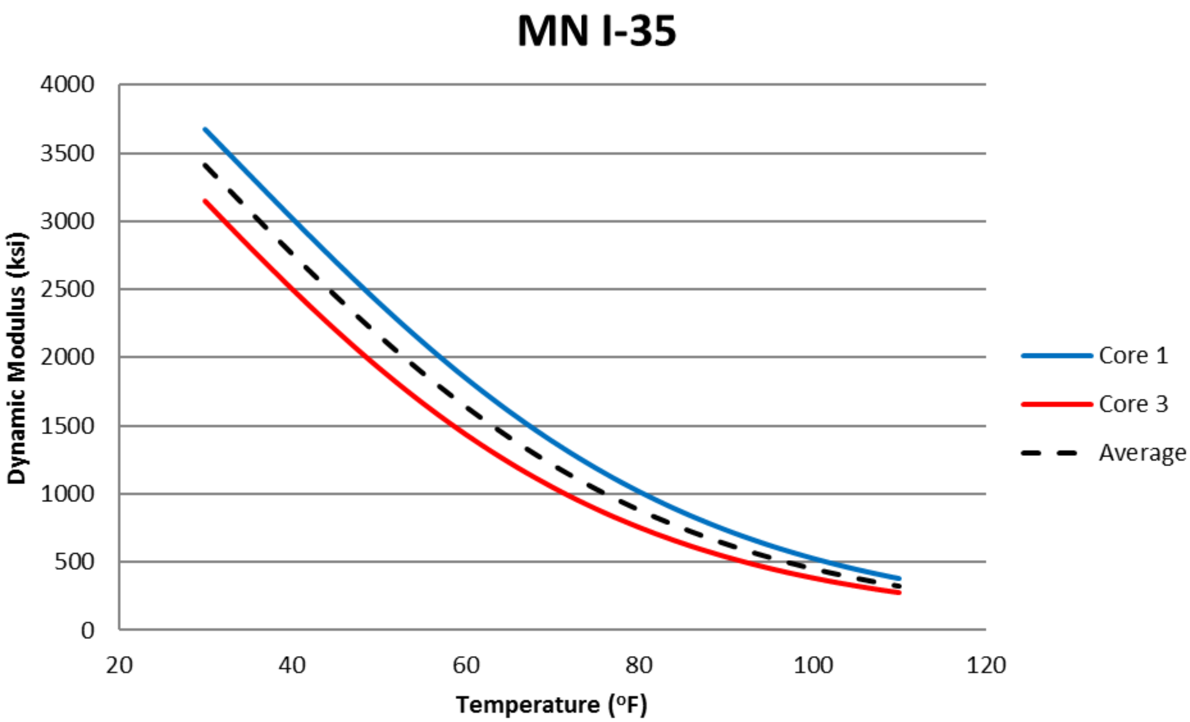


Figure D-2. Stiffness versus temperature curves for MN I-35

Table D-9. Master curve parameters for MN I-35

Core	Master Curve Parameter			
	Delta	**Alpha**	**Gamma**	**Beta**
1	2.95	3.86	0.3134	-1.0987
3	2.94	3.87	0.3134	-0.8938
Average	2.95	3.86	0.3134	-0.9963

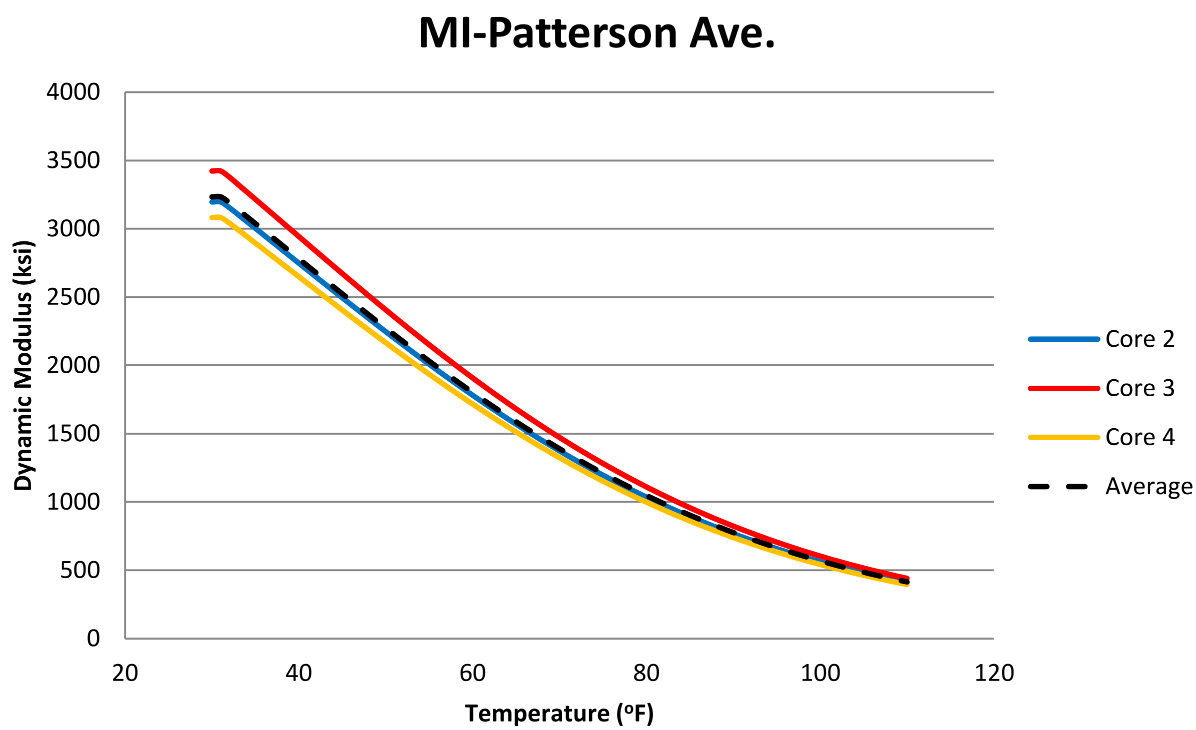


Figure D-3. Stiffness versus temperature curves for MI-Pat

Table D-10. Master curve parameters for MI-Pat

Core	Master Curve Parameter			
	Delta	Alpha	Gamma	Beta
2	2.88	3.85	0.3134	-1.2561
3	2.91	3.85	0.3134	-1.2561
4	2.85	3.86	0.3134	-1.2561
Average	2.88	3.85	0.3134	-1.2561

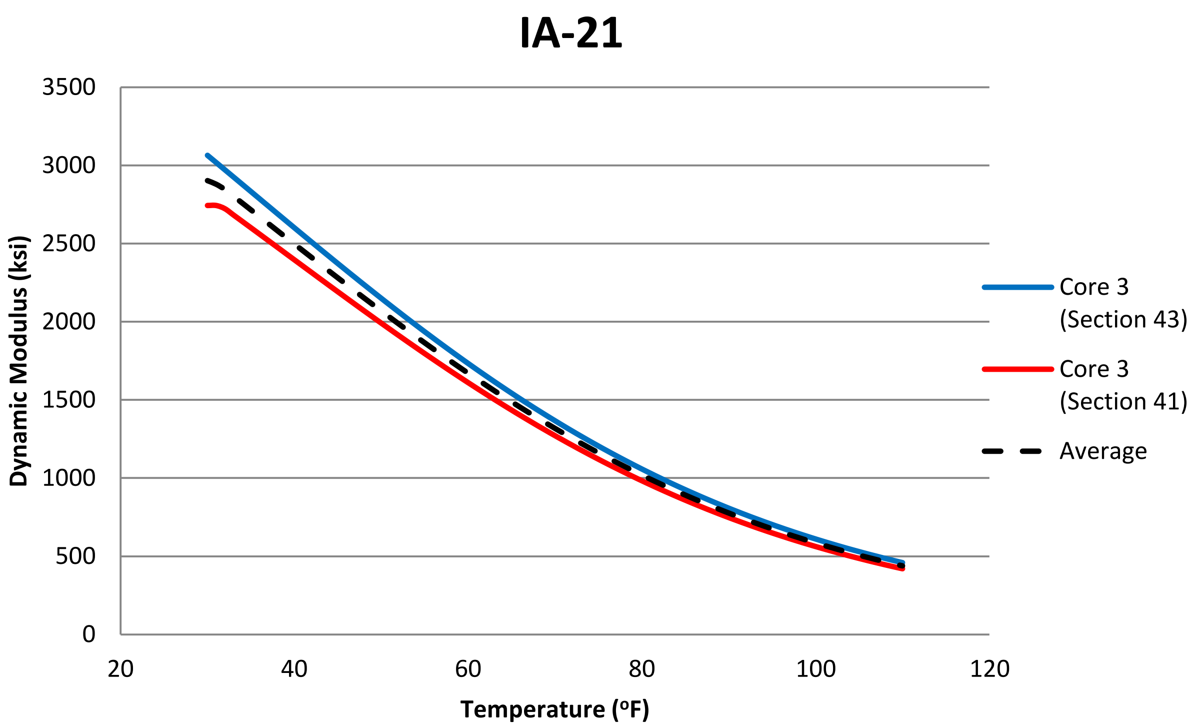


Figure D-4. Stiffness versus temperature curves for IA-21

Table D-11. Master curve parameters for IA-21

Core	Master Curve Parameter			
	Delta	**Alpha**	**Gamma**	**Beta**
3 (Section 43)	2.86	3.86	0.3134	-1.2588
3 (Section 41)	2.81	3.85	0.3134	-1.3225
Average	2.84	3.85	0.3134	-1.2907

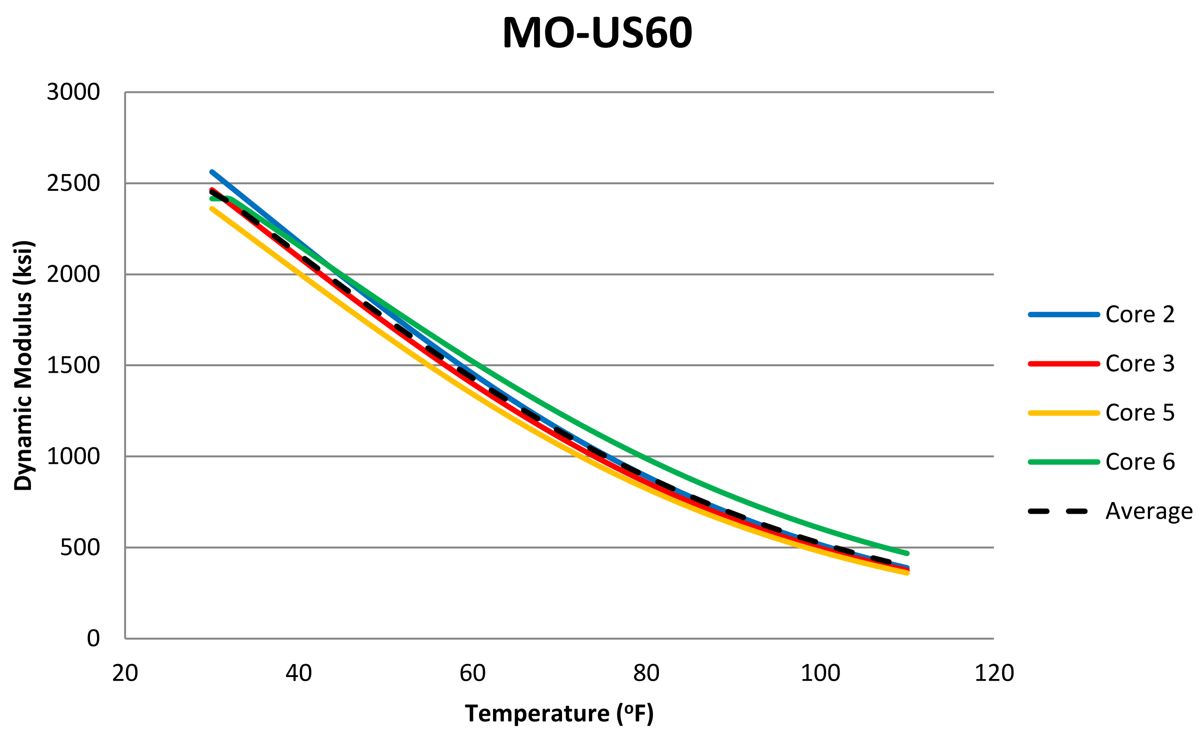


Figure D-5. Stiffness versus temperature curves for MO-US60

Table D-12. Master curve parameters for MO-US60

Core	Master Curve Parameter			
	Delta	Alpha	Gamma	Beta
2	2.82	3.83	0.3134	-1.2588
3	2.80	3.83	0.3134	-1.2588
5	2.79	3.82	0.3134	-1.2588
6	2.78	3.82	0.3134	-1.4132
Average	2.79	3.82	0.3134	-1.2974

A load frequency of 30 Hz is assumed for all curves. The average stiffness versus temperature relationships for each pavement section are shown in Figure D-6, with the master curve parameters used in Equation 2 summarized in Table D-13.

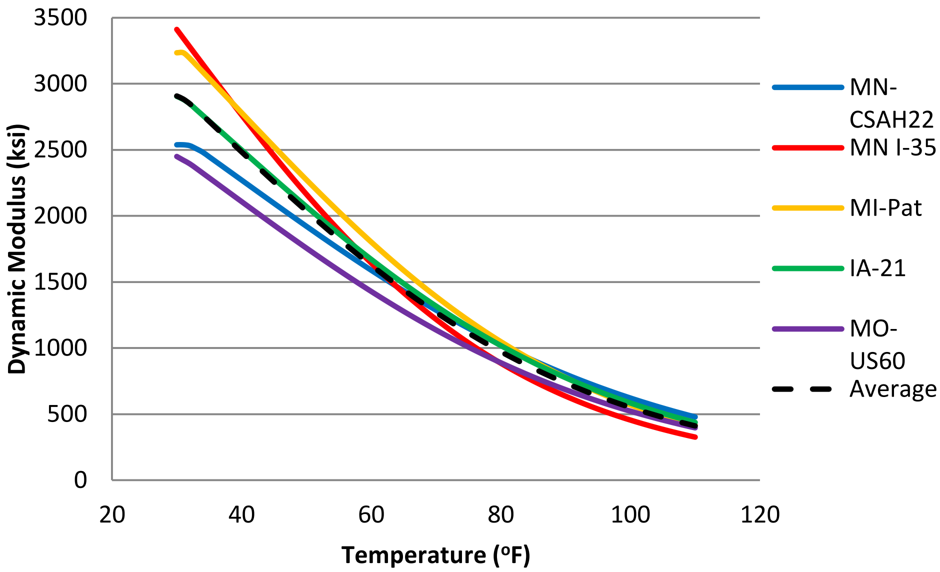


Figure D-6. Average stiffness versus temperature curves for all cored sections.

Table D-13. Average master curve parameters

Section	Master Curve Parameter			
	Delta	**Alpha**	**Gamma**	**Beta**
MN-CSAH22	2.75	3.87	0.3134	-1.4148
MN I-35	2.95	3.86	0.3134	-0.9963
MI-Pat	2.88	3.85	0.3134	-1.2561
IA-21	2.84	3.85	0.3134	-1.2907
MO-US60	2.79	3.82	0.3134	-1.2974
Average	2.84	3.85	0.3134	-1.2510

The average temperature for the coldest month was determined and is provided in Table D-14.

Table D-14. Summary of flexural layer stiffness analysis

Project Label	Ave. Asphalt Thickness (in.)	Ave. PCC Thickness (in.)	[1]Ave. Mthly Temp. In Coldest Mthly. (°F)	Asphalt Layer, [2]D_{HMA} (ksi)	Concrete Layer, [3]D_{pcc}(ksi)	Ratio D_{HMA}/D_{pcc}
MN-CSAH22	**3.67**	**6.34**	**15**	**9,389**	**99,390**	**0.09**
MN-I35	***8.50***	***6.31***	***8***	***116,643***	***98,296***	***1.19***
MI-Pat	5.87	4.21	18	38,351	29,148	1.32
IA-21 (Sect. 43)	**1.92**	**4.63**	**20**	**1,337**	**38,854**	**0.03**
IA-21 (Sect. 41)	**3.64**	**4.28**	**20**	**9,160**	**30,698**	**0.30**
MO-US60	5.10	4.77	32	25,257	42,417	0.60

[1] Weather data from RSSweather.com

[2] HMA: E = 2,000,000 psi; Poisson's Ratio = 0.35

[3] PCC: E = 4,500,000 psi; Poisson's Ratio = 0.2

Bold indicates sections with trans. cracking in existing HMA

Italics indicate sections with reflective cracking.

It can be seen that these temperatures are all at or below freezing. The corresponding dynamic moduli are unrealistically high at these low temperatures because these equations tend to have a higher error associated with the extreme high and extreme low temperatures with respect to the more moderate temperatures. This is another reason why the use of typical values for the winter asphalt stiffness is justifiable. A dynamic modulus of 2 million psi was used to establish the cold weather stiffness for all asphalt layers. This is a typical value commonly used to represent the maximum asphalt stiffness. A Possion's ratio of 0.35 was also assumed for the asphalt.

The flexural stiffness values of the concrete layer (D_{PCC}) and the asphalt layer (D_{HMA}) are provided in Table D-14, along with the flexural stiffness ratio. The sections that exhibited transverse cracking in the existing asphalt are highlighted in bold. Of the sections having transverse cracks in the existing asphalt, the BCOAs that exhibited reflective cracking are in italics. It can be seen that these sections had a flexural stiffness ratio greater than 1 and therefore reflective cracking would be anticipated to develop. Some states consider the development of a few transverse reflective cracks in the BCOA to be acceptable. If this is not considered acceptable, then a debonding material should be placed directly over the crack (such as duct tape) when the flexural stiffness of the asphalt layer is greater than that of the concrete layer. The duct tape should be wide enough to span sufficiently far across each side of the crack to ensure that the tape will remain bonded to the asphalt for the duration of the construction of the overlay.

Uneven Slabs (or Migration) due to Asphalt Deformation

Rutting in the asphalt pavement can occur as a result of one of two mechanisms, shear flow or consolidation. Shear flow is a constant volume deformation process. Material flows from the wheel path to an area adjacent to the wheel path, resulting in an upward heave in the regions adjacent to the wheel path. This is the dominant mode of permanent deformation in asphalt-surfaced pavements (ARA, Inc., ERES Consultants Division 2004).

When a BCOA is constructed, the PCC layer constrains the surface of the asphalt layer and prevents this heave from developing. Without space for material to flow, constant-volume shear flow cannot occur beneath a BCOA. This constraint against flow may not be present at the longitudinal joints. Material may flow across the joint, leading to "nonparallel slabs," where adjacent panels no longer lie on parallel planes but are divergent. Although structurally sound, this greatly increases the roughness of the BCOA.

Rutting was exhibited prior to the placement of the overlay in three of the six BCOAs cored. These sections would therefore be more susceptible to slab migration. Of these three BCOAs, however, slab migration was not reported to have occurred in any of them.

Using tie bars at the longitudinal joints, which is common practice in Colorado and Iowa, or synthetic structural fibers in the concrete mixture, which is common practice in Illinois, has been shown to deter shear flow. In terms of the slab migration susceptibility noted above, it cannot be known for sure if these preventive measures were effective in locking the slabs together or if there was just not a propensity for this type of distress to develop due to the characteristics of the underlying asphalt. Neither structural fibers nor tie bars were reported to have been used in any of the cored sections.

Because slab migration was not reported, the corresponding binder grade and/or mixture characteristics that lead to slab migration could not be identified. What could be established is the characteristics that do not lead to slab migration. The softer asphalt binders are more prone to shear deformation, as are mixtures with a large percentage of VFA. The binder grade and VFA percentages for each of the sections that exhibited rutting prior to the overlay are summarized in Table D-15.

Table D-15. Summary of binder and mixture design characteristics from core testing

Project Label	Rutting (in.)	Core #	Core Label	Grade	VFA (%)
MN-I35	0.24	1	MN-A1	PG 70-22	87.4
		3	MN-A2	PG 64-28	65.1
MI-Pat	approx. 1	2	MI-1	PG 76-16	50.1
		3	MI-2	PG 76-16	55.9
		4	MI-3	PG 76-16	46.5
MO-US60	Heavy rutting	2	MO-1	PG 82-22	49
		3	MO-2	PG 82-22	49
		5	MO-3	PG 82-22	50.1
		6	MO-4	PG 88-16	55.9

The higher the first number in the PG grading, the more resistant the pavement is to rutting and, therefore, the higher the likelihood that the binder is more resistant to slab migration. The VFA is typically designed to be between 65% and 80% for HMA surface layers, with higher values more acceptable for lower traffic volumes. It appears that the acceptable range might safely be higher for

HMA layers beneath a concrete overlay. This seems reasonable because the stresses in the HMA layer are substantially lower when it is beneath a concrete overlay. The softest grade binder of these three sections was for the same section of roadway (I-35) that had the highest VFA value. The binder grade for the asphalt from Cores 1 and 3 from I-35 had the lowest resistance to rutting according to the binder grade, PG70-22 and PG 64-28, respectively. With such a limited amount of data and such a large amount of variability, a statistical statement cannot be made regarding the effects of VFA or the grade of the asphalt on the potential for slab migration. It is evident that the criteria used when the HMA is a surface layer is too conservative to be applicable when the asphalt layer is under a concrete overlay.

Premature Fatigue Cracking as a Result of Debonding or Support Loss due to Stripping

Stripping involves the loss of binder from the asphalt matrix due to moisture. Stripping can lead to raveling and decreased tensile strength of the asphalt mix. Stripping of the asphalt adjacent to the concrete overlay is a primary concern because this is where water entering through the joints can become trapped. Stripping that develops deeper within the asphalt is unlikely to continue because water is less likely to find its way deep into the asphalt layer after the overlay is constructed; rather, water will likely collect at the asphalt-overlay interface.

Characterization of the volumetric properties was performed to determine the potential for stripping, as described in the following sections.

Adjusted Asphalt Film Thickness

The adjusted AFT represents the thickness of the asphalt coating around the aggregate. Increasing the film thickness decreases the potential for stripping. The adjusted AFT is estimated by dividing the volume of binder by the estimated surface area of the aggregate to be coated. Although it is assumed that the asphalt film thickness is the same for all aggregates, which is not the case, it is still a useful parameter for determining the susceptibility to stripping. The AASHTO R323, Standard Specification for SuperPave Volumetric Mix Design, does not consider AFT. MnDOT Specification 2360, Plant Mixed Asphalt Pavement, requires the adjusted AFT to be greater than 8.5 microns. This criterion is not applicable for BCOA, but appropriate criteria can be established by comparing the test results to the field performance of the BCOAs.

Voids Filled with Asphalt

The AASHTO R323, Standard Specification for SuperPave Volumetric Mix Design, requires a minimum VFA to prevent stripping and raveling. The standard criteria applied to asphalt pavements are not necessarily applicable for BCOA. Appropriate criteria can be established by comparing the VFA for the field cores from BCOAs that exhibited early fatigue cracking due to stripping to those cores that did not.

Characterization of the Aggregate

Some aggregates are more susceptible to stripping then others. The type of aggregate used in the asphalt mix should be characterized after the binder has been extracted.

Stripping was found in two of the six BCOAs cored. Cores 2 and 3 from Sections 43 and 41 on I-21 in Iowa and Cores 4 and 5 from US 60 in Missouri showed signs of stripping. The stripping in all cores was near the asphalt-concrete interface. Both sections are still performing well, and the stripping does not appear to be contributing to the premature deterioration of either roadway.

Material characterization was performed in the laboratory on one of the six cores that exhibited stripping (Core 5 [MO-3] from US-60 in Missouri). The adjusted AFT was determined to be 9.1 microns and the VFA was found to be 50.1%. Although the VFA would be considered low for an asphalt surface layer, the adjusted AFT is sufficient. It is difficult to determine the suitability of these values for an asphalt layer under a BCOA because insufficient data are available for characterizing the asphalt layers that did exhibit stripping and because the stripping is not affecting the performance of the pavement. The type of aggregate used in the asphalt was not noted for any of the cores, but this factor can contribute to the development of stripping. What can be concluded is that the stripping that developed near the asphalt-concrete interface did not cause premature deterioration in the BCOAs cored. These sections see limited traffic, which indicates that more leniency may be appropriate when evaluating allowable stripping criteria for roadways with lower volumes of traffic.

CONCLUSIONS

The lack of distress, specifically premature cracking due to stripping, slab migration, and reflective cracking, exhibited in the BCOA sections cored made it impossible to establish the volumetric material properties of the asphalt contributing to these distresses. It is recommended that further evaluation of these sections continue over time to determine if the life expectancy of the overlay is achieved or if any of these three specific distresses eventually result in a reduced life for the overlay. It would also be good to supplement the current database with additional pavement sections that do exhibit these specific distresses. Based on the lack of distress data exhibited in the BCOA, it is recommended that the additional testing defined in Phase II under Task 2 not be performed.

In the interim, existing HMA pavements should continue to be evaluated through visual examination of cores for thickness and identification of layers exhibiting stripping. The performance record across the nation for BCOA projects on HMA pavements in fair to good condition has been well documented. Designs based on sound engineering judgment and visual examination of HMA cores have proven to be adequate in the past.

APPENDIX D REFERENCES

ARA, Inc., ERES Consultants Division. 2004. *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures*. NCHRP Project 1-37A. National Cooperative Highway Research Program, Washington, DC.

Vandenbossche, J. M. and M. Barman. 2010. Bonded Whitetopping Overlay Design Considerations for Prevention of Reflection Cracking, Joint Sealing, and the Use of Dowel Bars. *Transportation Research Record,* 2155: 3–11.

Vandenbossche, J. M., N. Dufalla, and Z. Li. 2016. Bonded Concrete Overlay of Asphalt Mechanistic-Emperical Design Procedure. *International Journal of Pavement Engineering*, March 4, 2016.

APPENDIX E:

REFLECTIVE CRACKING

TASK 5: DETERMINE HOW EXISTING TRANSVERSE CRACKS IN THE HMA COULD AFFECT SELECTION OR DESIGN

The goal of this task was to develop a procedure to determine when a transverse or longitudinal crack needs to be repaired and makes the roadway a good candidate for thin Portland cement concrete (PCC) whitetopping. What are the condition and characteristics of the transverse or longitudinal crack? How do those determine what needs to be done? The primary issue with cracks in the existing HMA is reflective cracking in the bonded concrete overlay due to differential movement. For example, thermal cracks can reflect in the concrete overlay at an early age or develop later in the life of the overlay.

A component of this task was to develop guidance that assists the designer in characterizing the potential for reflective cracking and/or the need to perform some pre-overlay repair based on the width of the crack, the HMA material properties, placement conditions, and the proposed thickness of the concrete overlay.

The following provides a description of how transverse and longitudinal cracks should be addressed when considering bonded concrete overlays as a rehabilitation option. Each type of distress is addressed individually below.

Transverse Cracks

Transverse cracks develop in asphalt pavements due to the restraint of thermal contraction caused by the friction between the bottom of the asphalt and the base and the continuous nature of an unjointed asphalt pavement. These cracks can propagate up into the thinner bonded concrete overlay when the asphalt is significantly stiff relative to the overlay, as shown in Figure E-1.

Vandenbossche and Barman 2010

Figure E-1. Reflective cracking observed at MnROAD.

Figure E-1 shows a reflective crack that developed in a bonded concrete overlay constructed at the Minnesota Road Research Facility (MnROAD). The thermal crack depicted in the shoulder extended across the width of the pavement prior to the placement of the overlay. These lanes were then milled to the depth of the overlay to maintain the existing elevation, while existing shoulders were left unmilled.

As Figure E-1 shows, a reflective crack developed into the overlay. The pre-existing thermal crack in the shoulder designates the location of the thermal crack in the underlying asphalt layer that reflected up into the overlay.

Reflection cracking in the overlay is a function of both uniform temperature- and load-related stress. The thermal contraction of the asphalt in the winter creates a stress concentration at the bottom of the concrete in the region near the tip of the crack in the asphalt. The magnitude of the tensile stress at the bottom of the concrete then increases as a result of vehicle loads, thereby causing the crack in the underlying asphalt to propagate up through the concrete overlay. The fact that these cracks developed during the winter and early spring at MnROAD and developed at a faster rate in the driving lane than the passing lane supports the fact that reflection cracking is a function of both uniform temperature- and load-related stress.

The performance of the MnROAD test sections has shown that reflection cracks are a function of the relative stiffnesses of the concrete and the underlying asphalt layer as well as the accumulation of heavy traffic loads. The stiffness of the concrete overlay relative to that of the asphalt layer can be determined using the following equation (Vandenbossche and Barman 2010):

$$D_{PCC/HMA} = \frac{E_{PCC} \times h_{PCC}^3}{E_{HMA} \times h_{HMA}^3}\left(\frac{1-\mu_{HMA}^2}{1-\mu_{PCC}^2}\right) \tag{E-1}$$

where

$D_{PCC/HMA}$ = relative stiffness of the concrete with respect to the asphalt layer

E_{PCC} = elastic modulus of the concrete, psi

E_{HMA} = resilient modulus of the asphalt, psi

h_{PCC} = thickness of the concrete overlay, in.

h_{HMA} = thickness of the asphalt, in.

μ_{PCC} = Poisson's ratio of the concrete

μ_{HMA} = Poisson's ratio of the asphalt

Reflection cracks are anticipated to develop if the value of $D_{PCC/HMA}$ falls below 1 at a temperature measured on site.

A discussion is provided below regarding what pre-overlay repairs should be performed and when if transverse cracks are present in the existing asphalt pavement.

Pre-overlay Repairs

The thickness of the overlay should not be increased to address thermal cracks in the existing asphalt. However, it may be necessary to perform pre-overlay repairs to ensure that the thermal cracks do not influence the performance of the overlay. If the transverse crack width is greater than that of the maximum coarse aggregate size used in the concrete overlay mixture, a flowable fill should be used. This will prevent interlocking between the overlay and the asphalt layer. A flowable fill is not required if the transverse crack width is less than the maximum coarse aggregate size used in the concrete overlay mixture (Harrington and Fick 2014).

If the transverse crack width is less than the maximum coarse aggregate size used in the overlay mixture and the flexural stiffness ratio is less than 1, then preventive measures can be taken to ensure that the transverse crack does not propagate up into the overlay. Debonding the overlay from the asphalt layer in the vicinity of the crack will prevent the crack from reflecting up into the overlay. Tar paper stapled to the surface or duct tape placed over the crack, as shown in Figure E-2, has been shown to effectively deter reflective cracking. Some departments of transportation consider the development of a few reflective cracks in the concrete overlay to be acceptable, but the development of reflective cracking can be prevented as described above, if desired.

Vandenbossche and Fagerness 2010

Figure E-2. Prevention of reflective cracking by localized debonding.

A full-depth patch should be performed if the transverse crack is severely deteriorated throughout the depth of the asphalt layer and the pavement is unstable. If this type of heavily deteriorated cracking is extensive throughout the section, then an unbonded concrete overlay may be a more viable option than a thin bonded concrete or thin asphalt overlay.

Longitudinal Cracking

Longitudinal fatigue cracking generally develops in thicker asphalt pavements when the pavements are trafficked by trucks having high tire pressures. These high tire pressures result in high tensile stresses perpendicular to the direction of the tire at the pavement surface, which results in longitudinal fatigue cracks that initiate at the pavement surface and propagate downward. See Figure E-3.

Photo courtesy of Benjamin Worel, Minnesota Department of Transportation

Figure E-3. Top-down longitudinal fatigue cracking at MnROAD.

Pre-overlay Activities

Cores should be taken to establish the depth the crack has propagated into the asphalt layer as well as the degree of deterioration with depth.

If the longitudinal crack width is less than the maximum coarse aggregate size used in the concrete overlay mixture, then no pre-overlay action needs to be taken (Harrington and Fick 2014). The longitudinal crack shown in Figure E-3 is an example of a low-severity crack that would not affect the performance of the overlay if the overlay is constructed directly upon the distressed asphalt surface. A flowable fill should be used prior to the placement of the overlay if the longitudinal crack width is greater than the maximum coarse aggregate size used in the concrete overlay mixture (Harrington and Fick 2014).

If milling of the asphalt is performed prior to the placement of the overlay, then the newly exposed longitudinal crack width should be evaluated after milling. A flowable fill should be used if the newly exposed longitudinal crack width is greater than that of the maximum coarse aggregate size used in the concrete overlay mixture. Otherwise, the overlay can be placed directly on the asphalt. It is possible that the longitudinal crack would be completely removed during the milling process because longitudinal fatigue cracks develop from the top-down and may not have propagated below the planned depth of milling. Because longitudinal cracking is a top-down distress, the severity of the distress tends to decrease as its depth into the asphalt layer increases. Therefore, milling prior to placement of the overlay can also be beneficial because the most heavily distressed region can be removed. Cores can be taken to establish the depth of milling and to determine whether a flowable fill might be needed based on the severity of the deterioration of the crack at the depth to which the asphalt is to be milled.

Longitudinal cracking is typically not an issue for BCOA designs when the width of the crack is less than the maximum coarse aggregate size used in the concrete overlay mixture. Regardless of the crack width, the vertical differential displacement between the adjacent sides of the crack should not exceed 1/4 in. This would indicate problems with subgrade support and/or drainage. Both of these issues would need to be mitigated regardless of the final pavement design.

A full-depth patch is only needed if the longitudinal crack is severely deteriorated throughout the depth of the asphalt layer and the pavement is unstable. If this occurs extensively throughout the section, then an unbonded concrete overlay may be a more viable option than a thin bonded concrete or thin asphalt overlay.

APPENDIX E REFERENCES

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