Evaluation of WisDOT Quality Management Program (QMP) Activities and Impacts on Pavement Performance

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16. Abstract

This project aims to evaluate the effectiveness of the quality measures employed by the Wisconsin Department of Transportation to influence the in-service performance of flexible pavements. The study involved creating a relational geo-referenced database connecting production, placement, and routine in-service performance data collected over the years for Wisconsin Projects. The database relied on geo-referencing all the data such that each individual data point is assigned a location. This approach allows for tracking the quality of the material and construction of specific segments of the roadway to the in-service performance. A subset of the projects was also included in an in-depth study using on-site distress survey and non-destructive testing using Falling Wheel Deflectometer (FWD). In general, the thirty (30) highway projects studied in this project show that transverse and longitudinal cracking are the most common distresses. Construction joint longitudinal cracking appears to be highly common in all the on-site visits. Rutting is localized but not common. Alligator cracking takes place in multiple locations. This distress appears to relate to either soft foundation or some of the quality measures.

Deviations from the quality indicators' targets show correlations with the distresses. Mix production air voids (Va), mix voids in mineral aggregate (VMA), and placement density (%Gmm) are correlated with in-service performance. Deviation from target Va by reduction of 0.25% or more, correlates with a reduction in performance. Accomplishing VMA higher than the target is associated with lower levels of distresses. For %Gmm, about 2% increase in the compacted density show improved performance for thicker than 2" lifts. Thin lifts are more sensitive to increase in compaction effort than thicker pavements.

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Executive Summary:

Summary

A majority of state highway agencies have been engaged in Quality Management Programs (QMP) for a minimum of 10 years with current mix design methodologies (i.e., Superpave mix designs and use of the gyratory compactor). Most QMP testing protocols and resultant data are being applied primarily towards materials acceptance programs and, as a single/independent parameter, expected to predict or explain future performance.

There are multiple factors contributing to pavement performance and any ensuing distress. The factors can be material related, construction related, traffic-related, or even involving interdependency between distresses. Studies have not been conducted to evaluate or confirm the significance of QMP acceptance limits and their possible impact on pavement performance. Current tolerance limits were established based on individual AASHTO test procedures' related precision and bias statements, and local materials round-robin results. In order to prepare for migration to performance-based/related specifications, the current state of practice needs to be captured and enhanced. This will allow the department to consider performance-based/related factors in future studies leading to quantifiable and beneficial specification improvements.

In addition, the amount of time and effort spent in attaining and storing the QMP data is not proportional to their subsequent use either by the department or the paving industry. There is a need to expand the use of this data and directly relate it to other measurable components contributing to the eventual pavement service life. Subsequently, there is an equal need to track and connect existing pavement conditions with production Quality Control data submitted during construction, and ensuing field in-situ performance data collected by highway agencies over given service life. Given the complexity of this study, it should focus on a small set of roadways to establish trends and "proof of concept."

This report documents the work completed in Wisconsin Highway Research Program (WHRP) project 0092-15-05, "Evaluation of WisDOT Quality Management Program (QMP) Activities and Impacts on Pavement Performance. The objective of this research is to evaluate how quality control criteria influence long-term pavement performance of Wisconsin flexible pavements. Work conducted during this project includes:

1- Conducted a national survey of state highway agencies to document the quality indicators used in the different states. This survey identified the properties controlled by the 44 states and their different approaches in conducting their quality programs.
2- Surveyed the Wisconsin DOT databases for flexible pavements constructed within the past 10 years such that enough data can be extracted to conduct the needed analysis. This led to the identification of 30 paving projects.
3- Conducted analysis of the database containing information for the 30 pavement projects compiled for this study. This is followed by identifying 7 projects for field distress survey and structural testing using Falling Wheel Deflectometer (FWD).
4- Conducted field distress survey to document observed distresses. Analyze FWD results for identifying whether the observed distresses are caused by structural deficiencies or due to material related quality.
5- Executed in-depth review and analysis of the 7 projects selected for the field inspection for validation of findings.
6- Provided recommendations based on analysis interpretation and validation for the influence of quality indicators on in-service performance.

Research Approach

The research approach focused on creating a relational database where project quality data records and in-service performance are linked through geo-referencing. Accordingly, for a given location on the pavements included in this study, complete information can be extracted about the quality indicators (material and construction), and in-service performance. The indicators included the amount of air in the mix (Va), compaction density of the pavement during placement, mixture volumetric, and binder content. The literature review identified quality parameters historically being regarded as critically influential. The critical influence defined in-terms of reported impact on the in-service performance of pavements. The survey collected the parameters and specifications of quality control programs in 44 states. The survey shows that while different states apply different techniques to control the quality of their pavements, they still rely on similar quality indicators.

A number of constraints and criteria were applied in project selection to help assure that the conclusions of the study are useful to the state of practice. In total, 30 projects were selected for this study. Their related information was collected from different resources and databases. Criteria used in selecting these projects include: (1) pavement age of more than 5 years, (2) HMA used is more than 10,000 tons, (3) traffic level of more than 3 million ESALs, and (4) project length is more than one mile. Given the limited availability of the needed data, the final list of the 30 projects eventually included projects with traffic volume of less than 3 million ESALs.

Quality and performance data records are employed in a framework for developing the targeted relational database. The aim is to relate pavement performance at a given location to the individual data points recorded at the mix production and construction stages. The source records rely on different formats for geo-referencing their data (stations, landmarks, GPS...). This required that the geographical referencing systems be converted to a single common system. Therefore, quality indicators and performance are mapped throughout the project length. The conversion took place through the digitization of the source records and matching the project plans with GPS maps.

The research focused on four main distresses in the analysis as representatives of pavement performance. These distresses are Rutting, Alligator Cracking, Transverse Cracking, and Longitudinal Cracking. According to ASTM D6433: "Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys," distresses are typically categorized at different severity. For conducting the analysis, converting the categorical data into continuous data type was necessary to calculate an index value corresponding to the distresses impact on the roadway. Such index is calculated in accordance with the ASTM standard to quantify the distresses in a manner that can be incorporated in the relational database. At this stage, the data analysis was conducted since all data was digitized, quantified, and geo-referenced.

In the global analysis phase, the quality and performance information of the selected 30 WisDOT projects were extracted to investigate the potential correlation between the quality and performance. The global statistical regression analysis was conducted to quantify the influence of quality indicators on the deterioration indices. These models were developed primarily to conduct a sensitivity analysis of the distresses to deviations in the quality indicators from current specification targets. The results showed that the greatest reduction in rutting deterioration is associated with increased placement density followed by mix air voids. Alligator cracking shows no sensitivity to field compaction but rather highly dependent on mix volumetrics.

The project by project analysis focused on utilization of data of 7 projects such that analysis is conducted using data with minimal averaging. On-site distress evaluation was conducted along with FWD testing in order to isolate the potential failures due to the structurally compromised base and/or subbase layers. In this stage of the analysis, the findings from the global analysis are validated. It also helped to document the interaction between localized quality data points and the performance measures.

In general, this study documents the state of performance of pavements within the State of Wisconsin as represented by the sampled 30 projects. The abundance of longitudinal cracking requires significant evaluation within the state's network. In terms of quality measures, meeting current targets appear to lead to adequate quality. However, deviating from these targets is associated with variation in performance. For example, when a mix deviates from the target 4% compaction air voids (Va), the performance shows a drop when Va drops below target and within acceptance limit. When Va increases beyond target (and within limits) performance does not show change. Therefore, the limits for Va are recommended to be more stringent on the lower bound deviation than the upper bound. Allowing a wider upper bound limit for the VMA is correlated with improved performance. Finally, it is shown that placement compaction is a critical measure. While increasing compaction density shows improvement in durability, over-compaction is highly risky.

Summary of Findings

- State of practice survey showed that the majority of the state agencies rely on the same group of quality indicators. These properties are primarily related to asphalt mixture volumetrics.
- The work completed in this study provides the DOT a path for evaluating the effectiveness of the quality data, and a systematic approach for extracting valuable trends and correlations to manage the department resources better while producing durable pavements.
- Distress survey and recording method requires some revision. The current method of sampling of the pavement and distresses capturing may produce misleading conclusions.
- The holistic relational database created for this research was able to capture the potential influence of the some of the quality indicators on pavement performance. The analysis also showed that the Asphalt Cement (AC) content within the majority of mixes produced for the studied projects showed minimal variability. This made AC content statistically inadequate for studying its effect on the pavement performance. In addition, the actual AC used for each produced lot is not reported. Therefore, the effect of the deviation from the design AC content could not be studied.
- The current specification limits for the quality indicator appear to correlate with adequate performance when these limits are met.

- Deviation from these limits can influence the performance. Deviation in Va to lower value below 3.75% show association with poor performance. That is when deviation in the production of a mix designed to achieve 4% air after N_{des} compaction level leads to less air, rutting and cracking are more likely. But deviation in production leading to the mix resisting compaction and yield higher Va at N_{des}, the performance shows minimal change.
- Deviation in the VMA and placement %Gmm show that above target levels are associated with improved performance. The placement is %Gmm should have an upper bound limit to avoid over-compaction, especially for thinner lifts less than 2” thick.
- For pavements of ages 7-8 years, it is found that alligator cracking is recorded in 15%, rutting is recorded in 33%, longitudinal cracking in 62%, and transverse cracking in 71% of the database. With respect to the impact of these distresses, the maximum values of DI are recorded for longitudinal cracking followed by alligator cracking, rutting, and then transverse cracking.
- Rutting: this distress appears to be sensitive to field compaction followed by mix lot production air content at design number of gyrations.
- Cracking: while the statistical analysis showed good correlations for alligator cracking only, the project by project analysis revealed that transverse and in-line longitudinal cracking are also sensitive to the same quality indicators. Mix Va and VMA appear to play a significant role in improving quality, with minimal sensitivity to the field compaction.
- The effective asphalt content is not tracked by the department.

Conclusions and Recommendations

- The results of the analysis indicate that increasing field compaction density by about 2% is desirable for pavement thicker than 2”. With the Va target being 4%, the results confirm that this target is adequate. However, the global and by-project analysis show that deviation from that target to lower values are not desirable. The opposite is not true, where deviation to higher Va values does not correlate with deterioration in performance.
- The field over-compaction of pavement for thin lifts under 2” thick, especially over the stiff base can lead to the presence of all four types of distresses.
- The current trend in adopting performance related mix design appears to be the right path. Relying on the volumetrics alone does not yield clear recommendations as to how to improve pavement mixes. Consequently, the influence of the effective AC content is necessary for

establishing such a design protocol. It is also, necessary to understand the interactive effect of the volumetric properties including the AC content and the pavement performance.

- It is highly recommended that the DOT adopt the framework developed in this project to create its own complete relational database for the entire pavement network.

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1. Background and Survey Results

1.1 Introduction

Departments of Transportation nationwide have developed different programs to enhance the highway performance through inspection of asphalt paving productions and operations. The quality control programs follow asphalt mixes production, testing for consistency and quality from the stockpiles all the way to the finished road surfaces. In these programs, a series of tests are used to determine the engineering properties, and quality of the asphalt mixes being produced with the aim of increasing service life. For example, a study in Illinois DOT showed 15% improvement of fatigue life as a result of the quality control program [1]. Specifications are principally designed to make sure that all the critical raw ingredients used in the finished pavement, as well as the processes, are tested for consistent quality. However, defining critical parameters and their impacts on the final product is complicated and sometimes controversial. Therefore, quality control specifications may vary from state to state. In addition to the type of the tests, frequency, sampling and acceptance methods for materials, and processes can also vary.

1.2 Production and Construction Quality Indicators for WisDOT

The Wisconsin Department of Transportation (WisDOT) developed its Hot Mix Asphalt (HMA) Quality Management Program (QMP) in the early 1990's. Endorsed by the Federal Highway Administration (FHWA), QMP is considered a best construction practice to help ensure that the agency is receiving quality construction materials that are being produced by a contractor for an agency project. The development of the QMP specification involved identification of key asphalt mixture parameters and how they relate to long-term pavement performance, selection and potential modification of nationally standardized material testing methods and procedures, establishing testing frequencies and test method sample evaluation thresholds, corrective action guidance and pay adjustment factors. Additionally, the QMP specification required the development of the agency's quality control (QC) oversight program consisting of quality assurance (QA) and quality verification (QV) as well as a WisDOT sponsored QMP technician training program which became known as the Highway Technician Certification Program (HTCP). Full implementation of the HMA QMP specification, defined as being normally applied to all projects that utilized HMA pavement material, was accelerated and in place by the mid-1990's.

Since the implementation of the HMA QMP, the specification has seen multiple revisions in response to interpretation and application requests; but, the key HMA mixture parameters (aggregate gradation, asphalt content, air voids, and voids in the mineral aggregate) and in place pavement density have remained as the HMA QMP quality measures. As the specification approaches its twentieth anniversary, WisDOT is able to evaluate the effectiveness of the HMA QMP specification quantifiably. The effects and importance of these parameters on the long-term performance were investigated by many researchers in the literature. This chapter discusses the significance of the quality indicators used by the State of Wisconsin and their reported impact on in-service performance or pavements.

Air Content (Va)

Controlling the volume of the air in the mix is a critical parameter that maintains the mix quality during production in the asphalt mixing plants. In fact, the presence of air in the mixture significantly affects the mechanical response of the pavement to load. Zhao [2] showed that reducing air voids content can improve the resistance of the asphalt concrete against rutting and moisture damage. A study at Purdue University by Vivar et al. [3] was conducted to investigate the pavement performance relationship with the initial pavement density. The study attempted to quantify inter-relationship of moisture induced damage and mixture Va. Four hot-mix asphalt mixtures at four different air void contents were evaluated with the dynamic modulus and beam fatigue apparatus. The mixtures differed in both aggregate size and gradation. To assess durability effects, performance tests were performed on unconditioned, moisture conditioned and oven aged samples. The results indicated that reduction in Va is a significant factor in the improved performance and durability of the tested specimens against rutting and moisture damage respectively.

In-Place Air Voids (Density)

Construction methods and compaction efforts are influencing the behavior of the pavement in the field. Different methods result in producing different HMA densities, which ultimately affect the mechanical properties of the pavement. Therefore, due to the importance of the density, many agencies are trying to control it in order to protect pavements from further density related issues in the performance. Figure 1-1 shows the density levels required by different states as part of the QC/QA programs (Based on AASHTO Subcommittee on Materials 2007 Survey).

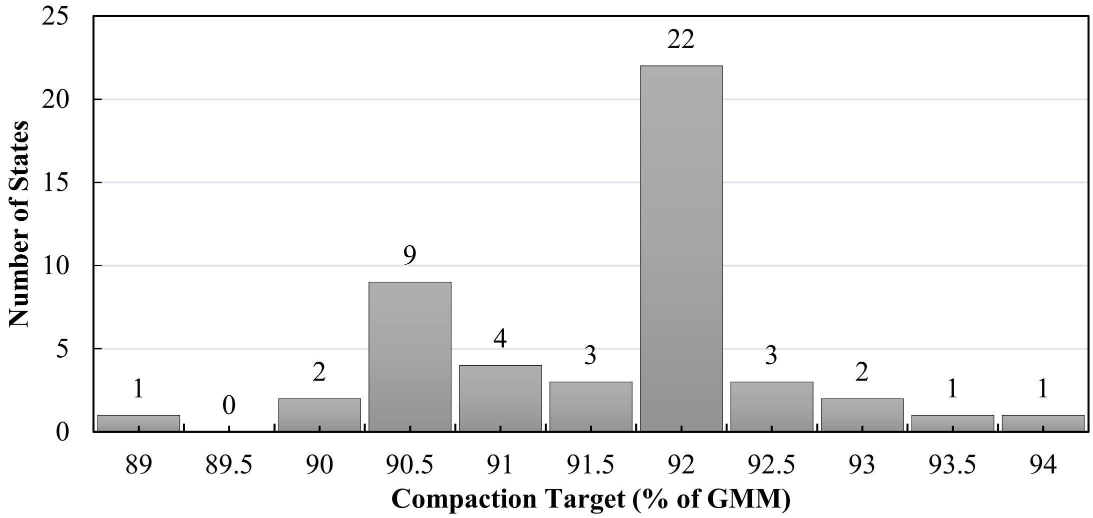


Figure 1-1 Compaction target density for different states, (after AASHTO 2007 SOM survey)

Conventional quality control programs are carried out on the basis of spot tests, typically using a nuclear density gauge or recently using modulus-based devices [4, 5]. Several studies have attempted to evaluate if a correlation could be developed between stiffness measurements of the placed mat and in-place density measurements, thereby eliminating the need for other quality control/quality assurance (QC/QA) density tests [6, 7, 8, 9]. The results of these studies show that the relationship between these measurements and in-place density are inconsistent.

Some transportation agencies control moisture damage by limiting the pavement density, which is believed to limit the access of water to the bulk of the asphalt mixture. For example, in Canada, the Ministry of Transportation of Ontario specifies that density in surface asphalt courses must be as low as possible to control moisture damage [10]. It is believed that density works as an important factor along with the other factors to control the rate of moisture transport in HMA [11,12,13].

Epps and Monismith [14] conducted a significant laboratory study on the effect of mixture density on fatigue performance of asphalt mixtures at the University of California at Berkeley. In this research, three asphalt mixtures with different binder content and aggregate sizes. Based on the test results reported, the effect of 1% decrease in density on fatigue performance was estimated to be a reduction of 20.6%, 43.8%, and 33.8% for the high binder content, fine mix, and coarse mix, respectively.

Harvey and Tsai [15] conducted another laboratory fatigue experiment using the flexural bending beam test. In this experiment, dense-graded asphalt with three levels of air voids and five levels of binder contents. The three density levels, including 99% to 97%, 96% to 94%, and 93% to 91%. A relationship between cycles to fatigue failure, air voids, binder content, and the applied strain was determined based on 97 fatigue test results. Based on the reported results, a 1% decrease in density was estimated to result in a 15.1% reduction in fatigue performance. Fisher et al. [16] showed that as the density increased, fatigue life increased. Lee et al. [17] Observed that for every 1% increase in density, the fatigue cracking is reduced by 19% and rutting by 10%. The similar trend is also shown by other studies [16, 18]. As can be noticed from these studies, mixed results are observed where the effect of density on performance is inconsistent. The authors of this report stipulate that this because the relationship between density and performance is not a linear, but rather quadratic where there is an optimum density value at peak performance level.

It is also important to mention that factors related to construction such as mixture production, placement, and compaction also affect the void structure of asphalt mixtures in the field. For example, Mohamed et al. [10] found that conventional compaction techniques in asphalt layers generate cracks, commonly known as checks. These cracks are typically 1 – 4 in length and are 1 – 3 in apart. These checks usually are not visible and are generated during the first or second pass of conventional steel-wheel rollers. Because of these cracks and due to their propagation, water migrates in spots that were supposed to be isolated. Furthermore, they proposed a new compaction technique that reduces the formation of cracks, preventing the general deterioration of the material due to moisture and oxidation. Chen et al. [13] also analyzed the effect of crack width on moisture damage. They found that crack growth and higher air void content had a similar influence on the moisture resistance of asphalt mixtures. Findings of their works were matching with the other studies since cracks act as links between air voids, generating new connected paths. There are other studies that analyzed the effect of lift thickness as well as other construction properties in the void structure and moisture susceptibility of the pavement [19, 20].

According to Beainy et al. [21], asphalt pavement density does not increase linearly with additional compaction efforts. It changes randomly “due to the continuous reorientation of aggregates and the randomness of aggregate shapes and textures.” Generally, compaction uniformity and overall compaction of pavements are increased through additional roller passes, although over-compaction is always possible. There have been many recent advances in compaction equipment, and construction practices that enhance the quality of the compaction. The literature and specifications are stating that the use of vibratory rollers, oscillatory rollers, or vibratory pneumatic tire rollers can help achieve optimized in-place density [22].

Starry [23] emphasized the importance of the monitoring of the surface and using real-time instruments like infrared sensors to achieve the ideal compaction times. He reported that significant improvement could be achieved by utilizing the new technologies. According to Scherocman [24]: “Whether asphalt mixtures are stiff or tender, breakdown or first rollers should be used immediately following the paver to ensure that the mixture is compacted while hot.” The effects of using GPS systems, color-coded maps, compaction meters, accelerometers, and other technologies that can improve achieving better final density values are all important and crucial to the pavement [22]. In fact, in addition to improving the quality, Intelligent Compaction can assist in collecting more detailed information of the pavement that can further analysis using the available software like Veta package. These technologies can be used as an advancement in density QC process.

The effect of changes in mixture properties, including in-place density, binder content, and aggregate gradation, on rutting performance, was investigated in a field study as part of the WesTrack project [25]. This investigation was only conducted based on rutting data measured on WesTrack test sections through the first two million Equivalent Single Axle Loads (ESALs). In a part of the study [25], three separate rutting models were developed. Based on the first series of models, a 1% increase in in-place air voids was resulted increasing the rutting of fine and coarse and coarse replacement mixes by 11.5%, 9.6%, and 6.3%, respectively. Based on the second model, a 1% increase in air voids corresponds to a 7.3% increase in rutting for the fine, fine-plus, and coarse mixes. Also, a 1% increase in air voids was estimated to cause a 10.9% increase in rutting for the coarse replacement mixes.

Blankenship [26] showed the results of a series of experiments on the Accelerated Mixture Performance Test (AMPT). The results showed that the “rutting resistance” measured by the flow number increased as the mixture density increase. It was shown that an increase in 1.5 percent density could increase the fatigue by 4 to 10 percent and the Flow Number by 34 percent. The research concluded that adding asphalt and increasing pavement density have a positive, compounding effect on overall performance if higher field density is achieved with the added asphalt cement content.

In a study conducted by [27], the performance of the pavement was evaluated for three different densities. Rutting depth as an average reported being 7.27, 6.49 and 5.12 mm for densities of 87%, 90%, and 93% respectively. The result showed the effectiveness of increasing density in controlling the rutting problem of pavement. They concluded that asphalt pavement density below 90% negatively impacts rutting performance. The result showed that 87% density resulted in 29.5% higher rutting than the 93% density test section. It also showed that 87% density resulted in 31.2% and 65.9% lower average tensile strength than the 90% and 93% density test sections, respectively.

In a study conducted by NJDOT, it was observed that for every 1% lower in-place air voids the fatigue cracking reduces 19% and rutting by 10% [17]. It has been reported in another study that 1% decrease in in-place air void results in about a 10% decrease in the field fatigue [31].

Mixture Properties

Asphalt Mixture properties like VMA, the maximum theoretical density, the bulk specific gravity of the mix, aggregate gradation, aggregate textures are shown to be effective in the long-term performance of the pavement. These are shown to influence the resistance of the pavement to the rutting, fatigue and thermal cracking initiation and propagation. Besides the mechanical failures of the pavement, the quality of the ride on the pavement is correlated with other parameters such as skid resistance and the smoothness of the paved asphalt. The asphalt mix should be designed in a way to provide mechanically durable asphalt with the satisfactory level of rideability.

Mrawira and Luca [32] studied the effects of mix design factors on the thermal properties of Superpave HMA. Their findings showed that the conductivity of the material, specific heat capacity, and thermal diffusivity are dependent on the mix design. The experimental results showed that the aggregate type has the most significant effect on the thermal properties. On the other hand, the compaction level showed to be not as statistically significant as the other factors. There are other studies that are also shown the sensitivity of the mixture thermal behavior to mix properties [33, 34, 35, 36]. They reported the high dependency of shrinking and hardening behavior of the asphalt due to the low temperature on its ingredients properties.

The skid resistance is a factor that shows the resistance of pavement surfaces, aggregates, and binder to sliding or skidding of the vehicle. Microtexture and Macrotexture of the mix that describes the aggregate component and aggregate arrangements are thoroughly investigated in the literature [37, 38, 39, 40]. Asi [37] investigated the effects of using the higher asphalt content, the Marshall mix design, aggregate gradation and aggregate quality on the skid resistance. The influencing factors on skid resistance at the Macrotexture level reported by Fwa and Choo [38] to be shape, size, gap width, layout, and gradation of the coarse aggregates. They reported that the area of the contact surface, as well as the number of gaps within the test area, has a significant effect on the measured frictional resistance. Based on this observation they recommended considering these factors for designing the mix to achieve the adequate skid resistance in actual construction.

The effect of mix properties on IRI value is thoroughly investigated in the literature [41, 42], and [43]. Choi et al. [44] researched the relationship of mix properties with the IRI performance of the pavements. Their developed models showed the significant effects of these properties on the field performance of the pavements. Anderson et al. [45] and Choi et al. [44] stated that the aggregate gradation (passing #200), percent of air void, viscosity, asphalt content, and total asphalt layer thickness are influencing factors on the IRI performance of the pavement. There are efforts in the literature on establishing the prediction models of pavement performance through the relationship of mix properties and International Roughness Index known as IRI [46, 47, 48, 49]. Recent studies focused on the relation of mixture properties with in-service performance [50, 51, 52, 53, 54, 55, 56]. However, these efforts were limited to developing correlation models based on the performance test results, rather than establishing true mechanical cause and effect relationships.

Binder Properties

Importance of the binder properties on the long-term performance of the pavement is undoubted. The flexural behavior of asphaltic pavements to some extent is dependent on the amount and type of the used binder. In fact, viscoelastic properties of the binder such as relaxation and stiffness strongly influence the cracking performance of a mixture [57].

The effect of binder properties on thermal cracking is shown by [36]. They showed a significant correlation between rheological properties of the binder and its resistance against the thermal cracking by using through thermal stress restrained specimen test (TSRST). A semi-empirical mechanistic model was also developed by Bouldin et al. [58] to determine the critical cracking temperature of the asphalt mixture by conducting tests on binder by using Bending Beam Rheometer. They reported a good agreement between performance models with the actual measurements in the field.

The relationships between the asphalt binder elastic recovery properties and the asphalt concrete fatigue performance were investigated by [59]. Elastic recovery (ER) and multiple stress creep recovery (MSCR) tests were conducted, and their results were then correlated to the predicted fatigue life. The results indicated that HMA mixes with high asphalt binder elastic recovery properties (⩾59%) exhibited better cracking resistance potential with long predicted fatigue life (>150 months).

Morea et al., [62], stated that within certain limits considering well design mixtures; asphalt binder type, climate temperature, and the service load appear as the main factors affecting the final rutting performance of a mixture. Similar work in the literature shows that the bitumen rheological properties reflect the binder contribution in the mixture rutting performance [63]. Many asphalt binders were studied including, three different conventional, multigrade, oxidized and three polymer modified bitumen. The rheological properties of them were measured at different temperatures and related to the performance test results. The research showed the correlation of binder properties to the performance. The result showed the binder properties must be taken into account during the pavement design process.

Wang et al. [64] investigated the performance of 8 different modified binders using mechanical tests such as rutting, fatigue, yield, and recovery tests. They developed a paving performance index for quantifying the cost-effectiveness of binders. However, the model was not able to provide a good correlation of cost and effectiveness for crumb rubber modified binders. Many recent studies focused on the effect of binder's properties on in-service performance [51, 53, 65, 66, 67, 69]. The common assumption in these studies is that the influence of binder on performance is independent of other factors. Thus, developed prediction models and recommendations that are based on the correlations developed in the study, but not establish true causality to support these correlations.

Many of the studies listed in this section show a significant influence of certain properties on pavement performance. However, in controlled experiments, the design focuses on targeted properties for examination. Therefore, it is unknown how the different quality-related properties combined can influence the performance. For example, would high asphalt content negate the effect of low in place density? To conduct an experiment that evaluates this question, significant effort must be exerted. On the other hand, Wisconsin DOT has been implementing QMP for 20 years. Therefore, it possesses data on enough projects concerning their construction quality and in-service performance. This data provides an excellent opportunity to conduct a comprehensive evaluation of the influence of these properties, combined, on the long-term performance of the Wisconsin pavement.

1.3 Developing a Holistic Framework for Evaluation of Quality Control Indicators

AASHTO pavement management guide stated the significance of assembling and preparing the best practices frameworks for the better functional management of pavements. The better management of the pavement is highly dependent on the proper evaluation of the current methods and practices. Therefore, many efforts have been made on developing a conceptual structure for the relation of performance with production and construction parameters [70, 25].

Explorations of a localized performance specification for HMA construction were examined in studies for Wisconsin DOT [68], Illinois DOT [71], for Indiana DOT [72], and for Arizona DOT [73]. Assessment of the effectiveness of construction quality parameters such as density or asphalt content has been offered by developing localized models like a study for Rhode Island DOT by Mensching et al. [74]. The localized quality or production related issues of flexible pavements have been mildly addressed in the literature [75, 76, 77]. However, the literature stressed the importance of a framework to allow for the utilization of advanced tools to understand construction-related factors influencing long-term performance [78, 79, 80].

In a technical report by Epps et al. [25], the above-mentioned need was addressed by developing prediction models. Using mechanistic-empirical theory the software application entitled HMASpec was developed, the software helped pay adjustments' calculation by analyzing the lifecycle costs of the project. Mathematical models were also introduced as another tool to enhance the effectiveness of performance-related specifications for production and construction procedures [81]. Deacon et al. [82] developed an approach to develop performance models from the results of the Westrack accelerated pavement test program for use in a performance-related specification for flexible pavement mixes. Laboratory tests such as flexural fatigue tests and shear tests were also conducted to determine fatigue response and rutting characteristics. The results were then used to develop performance models. Based on the performance models, it was determined that aggregate gradation, asphalt content, and air-void content should be considered as key construction factors for rutting. Witczak [73] in a study that was conducted for Arizona DOT characterized HMA performance factors to be used in models for the evaluation of HMA construction jobs in the state for rutting, fatigue cracking and thermal fracture. The derived models through the database yielded localized calibration, and it produced relevant results. Mogawer et al. [83] investigated the effective production and construction parameters on rutting and fatigue performance issues. These studies provide a fundamental understanding of the mechanical behavior of mixtures. However, most of them were done on laboratory samples.

Leverett [84] explored the efficiency of the construction quality control program of the Michigan Department of Transportation that pays incentives for contractors' conformance with specifications. He examined 77 projects which were constructed between 1994 and 2002, to see whether the QMP was successful to improve the long-term performance of the pavement or not. Different level of analysis was conducted on the collected data pool to assess the costs and benefits of the incentive/disincentive program of Michigan DOT for a variety of different pavement types and different pay items. However, due to the missing data elements, the analysis was limited to generalized evaluation of the program, as opposed to the project by project investigation. They have recommended using a systematic approach to store all the project data from inception to the end of construction and throughout the lifetime of the project that would allow all potential users to conduct the necessary analysis. The researchers also recommended converting the existing location reference system to a geographical information based system, linking the collected information to the common reference location system, and expanding the current analysis to the lower pavement structural layers to add more certainty to the results.

Buddhavarapu et al. [85] developed a database framework to incorporate the Texas DOT recorded information of construction quality to the performance records of projects with the age of 3 to 10 years. The statistical modeling of construction parameters relating to the performance was provided to use for revising the pay adjustment for the ride quality of HMA and concrete pavement. The research showed the abilities of such a framework for better pavement management. The operational tolerance of key HMA mix factors of asphalt content, gradation and density have also investigated in another research for Texas DOT to assess their effects on the performance [86]. Since this study was also conducted by using the laboratory samples, it was hard to generalize its findings to all different types of pavement conditions in the entire state. Smit et al. [87] demonstrated how it is doable to track the network-level performance of surface mixtures using Texas DOT databases. The authors claimed that the traffic load, environmental conditions, properties of the mix, design and construction properties, as well as the underlying structure of which the surface was paved on are all contributing factors to the life of the pavement and its performance. They concluded that the rougher the road, the more expensive it is for Texas DOT to maintain. The authors expressed that it was also necessary to expand their analysis to include factors such as binder performance and grade, and some HMA mixture-related properties such as asphalt content, voids in the mineral aggregates (VMA), density, and lift thickness.

In a study by Monismith et al. [88], performance simulations were conducted using common volumetric variations in California State. The types of addressed distresses were rutting and fatigue cracking, using Westrack and state department of transportation data. The study stressed the need for a systematic data management that allows analysis of the wide range of possible scenarios on the database. Comparing the procedure and findings of this study with those for Texas DOT [85], and [90] and Michigan DOT [84], the importance of geo-location of the available information as a mean to optimize the efforts of linking separate databases felt to be a missing link for all these studies.

In a study for Illinois DOT, in order to develop a performance-related specification, an attempt to facilitate the linkage between quality characteristics, engineering properties, and field’s distresses by connecting the points of each sublot of material to its related location as station and pavement lift was done through a properties map. However, since the mapping was done on a small scale, the research was unable to offer a systematic approach for such data collection [71].

Long-term Pavement Performance program (LTPP) provided a comprehensive database for several studies to investigate the effect of different parameters on the performance of the pavement. Choi et al. [44] investigated the contribution of material and construction variables of asphalt concrete on pavement performance by using a back-propagation neural network. Their results showed that the neural network models could efficiently be used to develop pavement performance models by quantifying the relationship of variables of asphalt content, void, asphalt layer thickness, and some other factors. Regarding the pavement performance indicator of IRI, Von Quintus et al. [89] used LTPP information to study the relationship between changes in pavement surfaces distresses to incremental changes in IRI. Their results showed that the selected distresses having a significant effect on abrupt changes in IRI with time and traffic. Perera and Kohn [90] researched the LTPP database to evaluate the factors affecting pavement smoothness. The factors such as design and rehabilitation parameters, climatic conditions, traffic levels, material properties, and extend and severity of distress were investigated.

Dong and Huang [91] evaluated the influencing factors on crack initiation of LTPP asphalt pavements and they concluded that the mixture and total thickness are not significant factors comparing to the high traffic levels and severe freeze-thaw cycles. Their parametric survival analysis showed that the on LTPP monitored sections the undesirable environments accelerated the initiation and progress of transverse cracking.

From the above, it is noticed that the literature contains several attempts to relate the construction quality and mix properties to long-term performance. The several studies presented prove the fact that this quest is yet to be completed. Reported research efforts show that developing an integrated framework to study the relation between the different components of a pavement life is challenging. This research presents the finding of Wisconsin DOT QMP program to navigate through these challenges.

1.4 QC/QA Program National Survey

To understand the components of different quality control specifications, Hughes [92] led a National cooperative highway research program (NCHRP) research project to collect information on quality assurance programs of construction works in the US states including HMA construction works. However, the survey did not report the specific details on the frequency of the tests and their acceptable limits and tolerance. A survey was conducted to collect information regarding the practices and protocols of quality control and quality assurance programs for the transportation authorities. There are 52 transportation authorities exist in the United States. 50 of them are state agencies, and there is one federal district agency and one territorial agency. The authors contacted all authorities through phone calls and emails. However, among all only 44 state agencies responded or had their information available on their websites. In this report, in addition to the description of these programs, an investigation is undertaken to show the key quality control and assurance measures. The results of the survey are categorized on different subjects and briefly discussed here.

Figure 1-2 is shown that all 44 surveyed states are using the quality control tests from contractors. Out of these, 36 agencies are combining quality testing with in-house tests. Moreover, there are 6 states that require an independent party to perform validation quality tests. It is observed that 14 agencies are using the same frequency of the test with contractors for QA programs, while the other agencies have a different frequency for test assurance. There are also 14 agencies that have the similar acceptable tolerance for their measurements compared to the contractors.

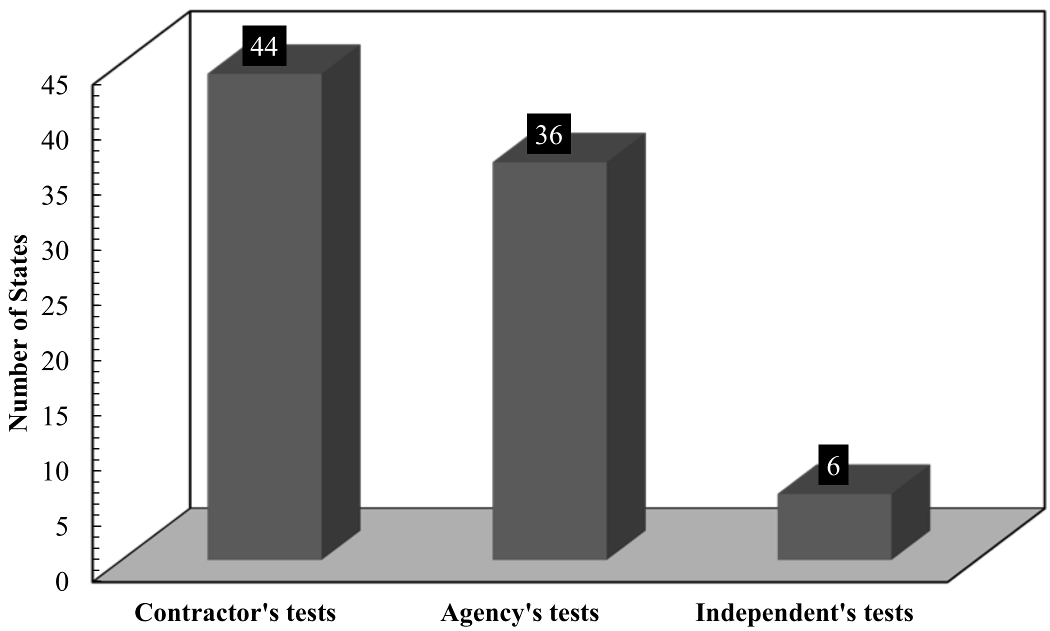


Figure 1-2 Type of quality control and assurance programs

Due to the wide range of QC/QA items in the survey, the test specifications related to the production and construction are categorized and briefly discussed here in the following sections.

Mix Volumetrics

Mix volumetrics such as air voids, voids in mineral aggregate, voids filled with asphalt, etc. are the factors that play important roles in the quality of the pavement. Therefore, proper inspection of them for quality control purposes is a necessity.

- ***Air Void (Va)***

Air voids as the vacant pores that are existed among the coated aggregate particles have significant effects on the asphalt properties. It is necessary to have a certain percentage of air voids in the mix to allow for a better flexible response of the asphalt to the traffic load and allow it to compact under traffic load without ruining the structure of the pavement. On the other hand, it must be limited to a threshold to avoid providing the passageways for the entrance of water and air, which can endanger the long-term performance of the pavement. The ultimate goal of quality control tests is to keep the air voids on the desirable level.

The amount of air voids in the mix is controlled both on the production stage (as a volumetric of the mix) and during the construction by measuring the density of the laid pavement. The majority of the surveyed states are controlling these properties by different methods (41 authorities). Controlling the air void in the lab and during production for (30) states and monitoring the compaction by taking cores and nuclear density tests in (27) states and (16) states are controlling both. The frequency of QC and QA control measurements for these items are varied from 1 sample per 500 tons (1), 700 tons (1), 750 tons (4), 1000 tons (3), 4000 tons (1), sublot (3) half a sublot (1), day (2) and half a day (2), and other standard frequencies for air void measurements. Also, the frequency of the test for construction stage measurements are 1 sample per 200 tons (1), 500 tons (2), 750 tons (3), 800 tons (1), 1000 tons (1), 10000 (1), lot (1), day (1), half a day (1), and other frequencies. QC control limit for the tolerance of measurement is also varied from 1% to up to the 2 %. Distribution of this data by State is shown in Figure 1-3.

Wisconsin DOT is controlling the mix air void through the quality control and quality verification programs by calculation in accordance to the AASHTO T269 (Appendix I). The number of contractor samples is determined on the amount of produced mix; generally, 1 in 1000 tons of mixture produced. The department verifies the mixture air voids by testing 1 in 5000 tons of mixture produced. The air void target has recently been changed to 3.0% for dense graded mixtures with a tolerance of +/- 1.3% for department verification tests and 4 point running average for contractor tests. The compaction of the mix and density of the asphalt layer is controlled during construction by the contractor’s certified nuclear density technician to assure its compliance with the specifications. The department verifies 10% of the number of contractor’s density tests.

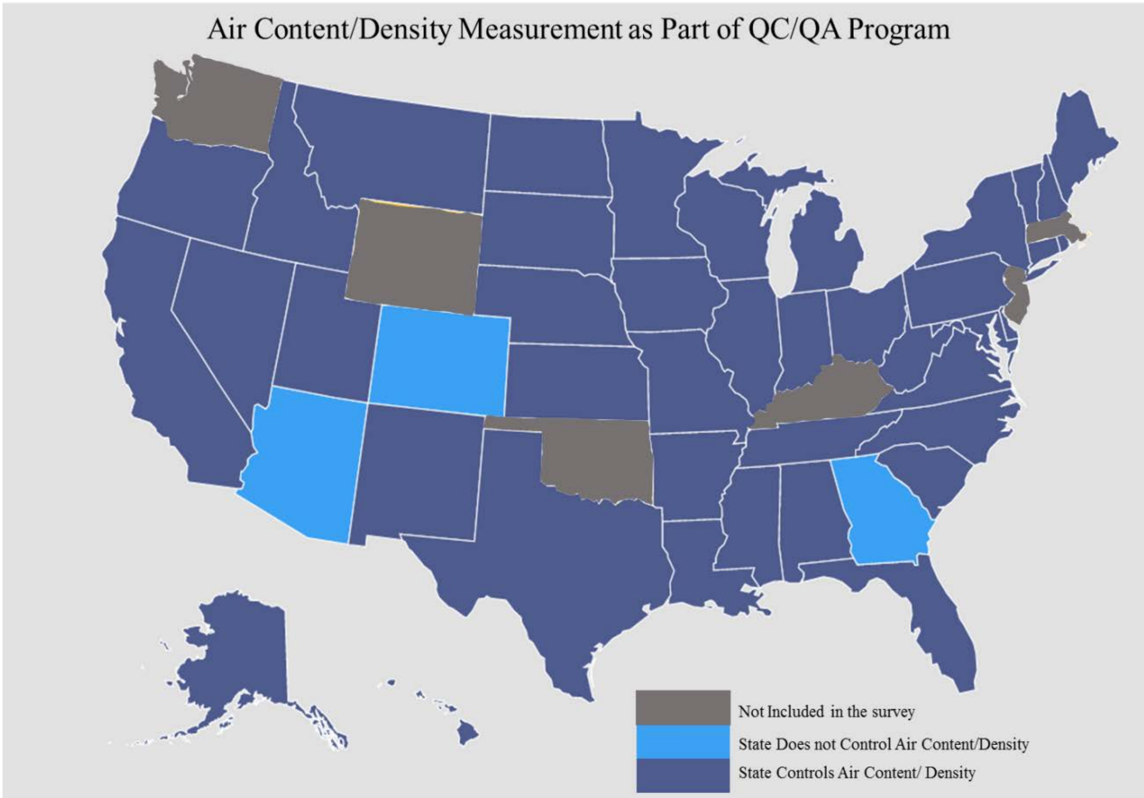


Figure 1-3 Air content/ Density measurement as part of QC/QA program

- ***Voids in Mineral Aggregate (VMA)***

Voids in the mineral aggregate are the spaces in the aggregates in the compacted mix including the filled spaces with asphalt. VMA is a parameter that correlates the necessary air void of the mix as well as the available spaces for the asphalt binder. This parameter is being tracked for (31) states. The measurement for QA and QC is varied from one sample per 500 tons (2), 750 tons (5), 1000 (4), sublot (2), day (2), half a day (2). The tolerance of the acceptance is also varied from 0.5% to up to the 2% variation from the state's required level. VMA control measurement as part of QC/QA programs is shown in Figure 1-4.

Wisconsin DOT controls VMA of the HMA mix throughout the QC/QA programs by calculating it in accordance to the AASHTO 35 test (Appendix I). The sampling frequency is the same as air voids for both contractor and department. The tolerance of acceptance is 0.5% of the required level

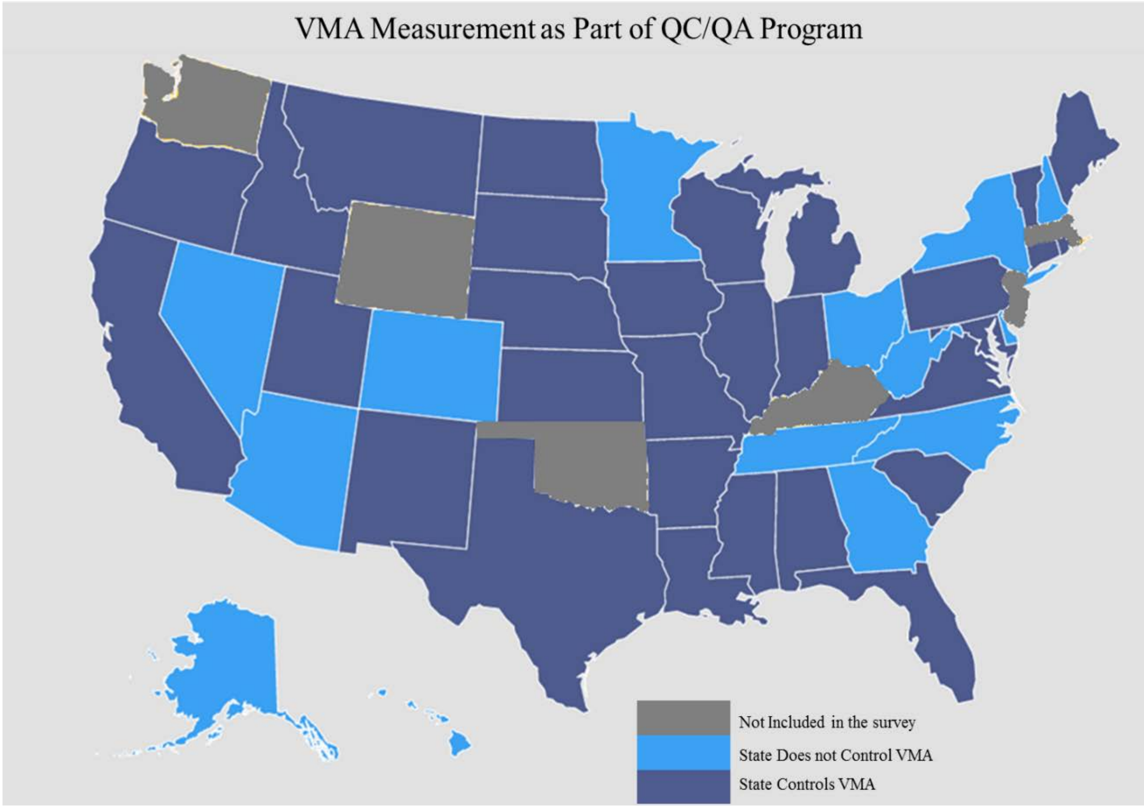


Figure 1-4 VMA measurement as part of QC/QA program

- ***Voids Filled in Aggregate (VFA)***

The VFA is an important factor for durability of asphalt as well as it is a relation to the density. Controlling the VFA helps to avoid mixes that are susceptible to rutting especially for highways with higher traffic levels. However, the majority of state agencies do not track this property (32 states). The ones that track it in (12) states have different frequency of QC and QA measurements, which is commonly similar to the other volumetric measurements' requirements. The accepted tolerance for the VFA values is different for each state with the maximum of 5% for two states (Vermont and Idaho).

Wisconsin DOT does not directly control this property as an independent item in QC/QA programs. However, the contractors are required to meet the state's target range for the VFA in the mix design (Appendix I).

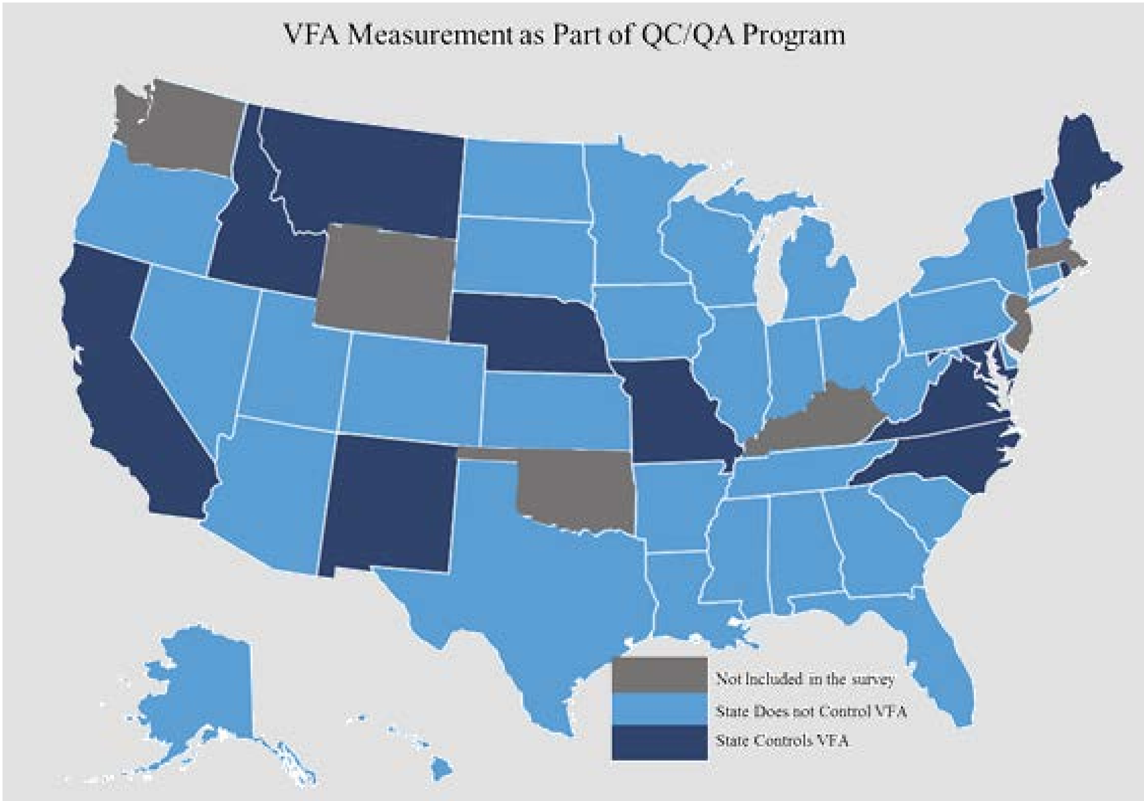


Figure 1-5 VFA measurement as part of QC/QA program

- ***Theoretical Maximum Density (Gmm) and Compacted Bulk Density (Gmb)***

Maximum density, as well as the bulk density, are the other Volumetrics that is being tracked for (27) states for Gmm, and (20) states for Gmb. The common frequencies of testing for them are one per 1000 tons (3), or one per 750 tons (2) one per 500 tons (1), one per half day (3) and one per day (1), or one per each time aggregate sample is taken for sieve analysis (1) or on a sublot basis (1) . The acceptance tolerance is varied as low as %0.019 for (2) states to as high as 2% for (1) state. Some information about QC/QA specifications is shown in Figure 1-6.

The state of Wisconsin is controlling both of these properties in QC/QA programs. The bulk specific gravity of the compacted mixture is determined in accordance to the AASHTO T166. Also, the maximum specific gravity is measured according to the AASHTO T209. The frequency of the test is the same as the air void frequency. (Appendix I).

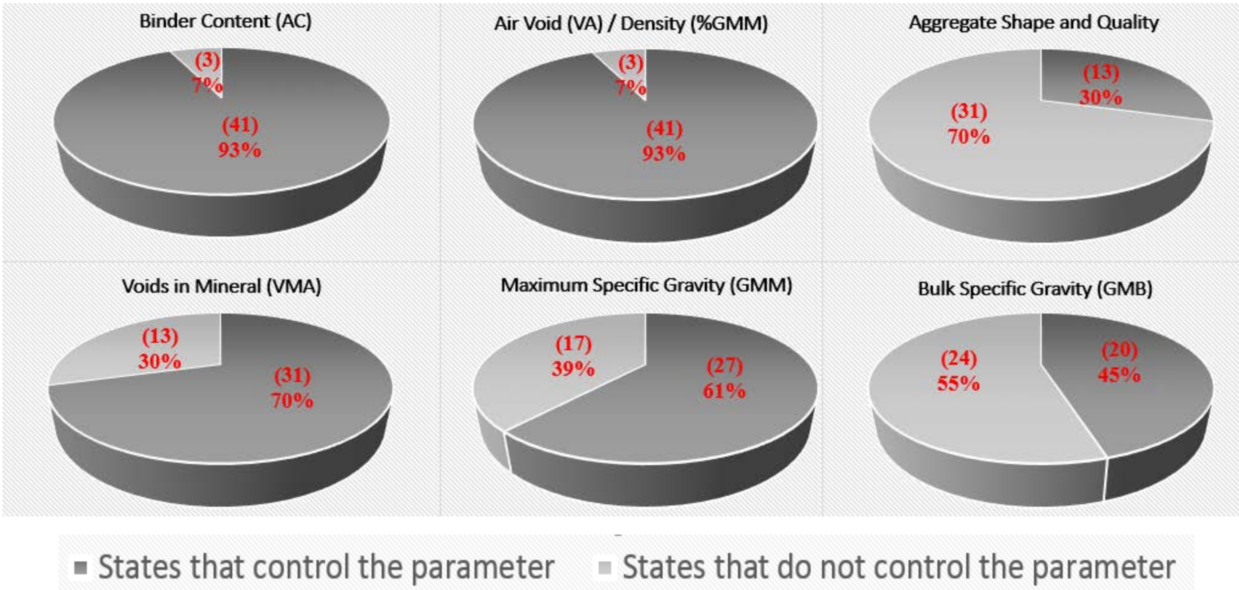


Figure 1-6 Implementation of different QC/QA items for surveyed states

Asphalt Binder

The properties and amount of binder in the mix play a vital role in the overall performance of the pavement. This characteristic of the mix is one of the main influencing factors on the long-term performance of the roadways. The constructability, cost, mechanical behavior of the pavement are the factors that are being influenced by the content of the binder and its quality. The performance of the pavement in terms of resistance against rutting and fatigue cracking are the other factors that make binder quality control tests critical.

All the surveyed states are controlling the binder content/quality (44/44). The frequency of test is varied from 1 per project (Texas), per 1000 tons (Indiana) and as required (Alabama). The viscosity of the binder is another type of test that is needed by only two agencies (West Virginia and California). While California only requires this test for asphalt rubber binders on every hour frequency, West Virginia requires flow number test results as 1 per 3000 tons as QC frequency and 1 per 6-hour production as QA frequency. Percentage of the asphalt by ignition oven or extraction method (3 states), fine particles to asphalt ratio (14 states) are the other common tests. The typical frequency of asphalt content measurements for QC is 1 per 500(4), 750(5), 1000 (5), 3000 (1), day (4), half a day (1), sublot (1). Wisconsin DOT is controlling the AC of the mix in the QC program by requiring the contractors to measure AC at the frequency similar to the air void content measurements (Appendix I). The method used is called a bucket extraction where the P200 and AC are removed from the rest of the mixture. A combined P200 and AC content is achieved. The P200 is calculated by an assumed AC content from plant settings. WisDOT does not currently verify AC content or require extraction or ignition oven tests as part of the quality control program but plans to start verifying in the near future.

Aggregate

Aggregate is a collective term for all mineral materials of different sizes and properties that are used with the asphalt binder to form the asphalt mixture. The structure of the asphalt is based on the aggregates since it occupies the most volume of the asphalt mixture. Therefore, the durability and performance, as well as the constructability, are highly dependent on it. This is the reason why most of the surveyed states defined a way to control this item (42/44). Gradation of the aggregate and controlling it based on the state requirements is the most common one among them (42). There are also states that doing the control gradation test on the extracted aggregates after the mix (9). The typical frequencies of QC testing are 1 sample per 500 (2), 700 (1), 800 (1), day (1) and sublot (2).

In addition to the overall grading measurements, there are also states that require specific details for the size of used aggregates in the mix. It was observed that (36) states have their own specifications that control the presence of certain aggregate sizes in the mix. The frequency of the test is the same as the frequency of the aggregate gradation. However, the acceptable tolerance of the measurement is varied from 1% usually for finer particle sizes to up to 10% for the coarser materials. These kinds of measurements also widely being used by many states (25) for assurance purposes.

Aggregate shape and quality is another item that (13) states are controlling. Fracture face of the aggregate, elongation of particles, and the mineral composition are the main properties monitored. As it was mentioned, the ratio of asphalt to dust or fine particles is another controlling factor for DOTs (14).

Specific gravity test for the aggregate is a QC/QA item for (1), the Moisture content of the aggregate (18), Aggregate plasticity test (1), Permeability of the aggregates (1), and Sand equivalent test (8) states.

Wisconsin DOT includes aggregates as part of QC and QA programs. The contractor runs gradation analysis at the same frequency as the air void frequency mentioned above. The tolerances are listed in Appendix 1. The department does not verify gradations. Contractors supply specific gravity values for their mix designs. The department spot checks some of the values. Quality tests are run by the department on contractor supplied material for pits every three years and on quarries each year.

Asphalt Moisture Related Tests

In order to control the susceptibility of pavement to the moisture, some state agencies are requiring the quality control and assurance moisture related tests to ensure that the performance of the pavement meets the requirements. The most common type of the control test is moisture measurement. The US map for the distribution of the moisture content tests is shown in Figure 1-7. the past experiences of dealing with the moisture susceptible materials convinced some DOTs to not only measure the moisture content but also implement moisture susceptibility test as a routine QC/QA item for (9) states. Similarly, moisture susceptibility test based on AASHTO T 283 plays an important role in QC/QA program of the state of Wisconsin. Currently, the state of Wisconsin does not require moisture content measurement of the mix.

There are (10) states that are only measuring the moisture content of the aggregate and (5) states only measuring the moisture content of the mixture whereas there are (8) states measuring both. The measurement is varied from 1 sample per 1000 tons (2), 2500 tons (2), day (11), half a day (3), lot (1), week (1), and project (1).

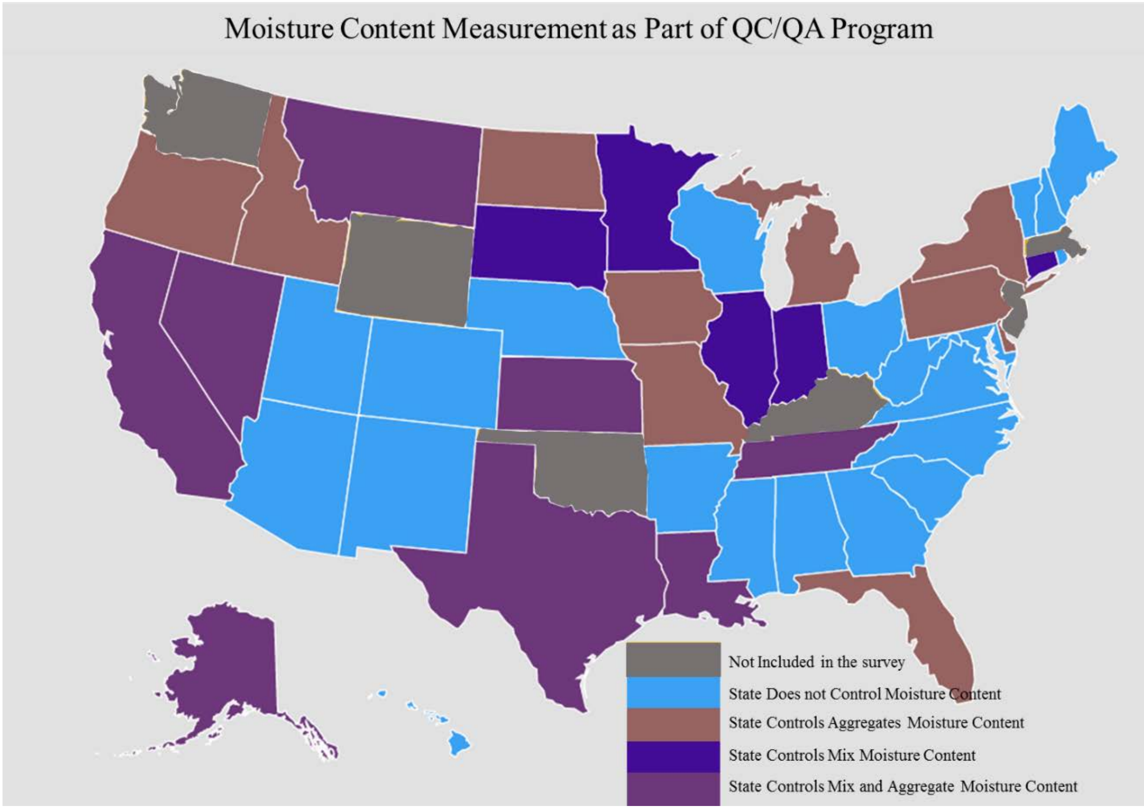


Figure 1-7 Moisture content tests for QC/QA program

Other Quality Control Tests

Importance of the quality control program for the DOTs has led them to adopt a variety of other material and performance tests. These tests are often designed to address state's localized known problems with materials and performance. For example, the stripping test for (3) states, presence of contaminants (1) state, emulsion residual test (2) states, Cantabro test (2) states, Wheel tracking tests (2), stability of the mix (4) states, controlling the temperature of the mix during production and placement (12) states are some of the other common tests. A summary of the QC/QA program items for the surveyed states is shown in Table 1-1.

Table 1-1 Survey of QC/QA items

State / QA/QC Item	Alabama (AL)	Alaska (AK)	Arizona (AZ)	Arkansas (AR)	California (CA)	Colorado (CO)	Connecticut (CT)	Delaware (DE)	Florida (FL)	Georgia (GA)	Hawaii (HI)	Idaho (ID)	Illinois (IL)	Indiana (IN)	Iowa (IA)	Kansas (KS)	Louisiana (LA)	Maine (ME)	Maryland (MD)	Michigan (MI)	Minnesota (MN)	Mississippi (MS)	Missouri (MO)	Montana (MT)	Nebraska (NE)	Nevada (NV)	New Hampshire (NH)	New Mexico (NM)	New York (NY)	North Carolina (NC)	North Dakota (ND)	Ohio (OH)	Oregon (OR)	Pennsylvania (PA)	Rhode Island (RI)	South Carolina (SC)	South Dakota (SD)	Tennessee (TN)	Texas (TX)	Utah (UT)	Vermont (VT)	Virginia (VA)	West Virginia (WV)	Wisconsin (WI)
Binder Content/ Binder Grade	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X		X	X	X	X	X	x	X	X	X	X	X	X	X
Binder Viscosity					X																																						X	
Gradation (Mix/Aggregate)	X	X	X	X	X	X	X	X	X	X	X		X	X	X	X	X	X	X	X	X	X		X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
Aggregate Gradation Detailed Requirements	X	X	X		X	X		X	X	X	X	X	X	X	X	X	X	X	X	X	X	X		X	X		X	X		X	X	X	X	X			X	X	X	X	X	X		X
Aggregate Shape and Quality					X									X		X		X			X	X	X					X	X		X		X				X		X					
Air Void (Production/Construction)	X			X	X		X	X	X			X	X	X	X	X	X	X	X	X	X		X	X	X	X		X	X	X	X	X	X	X	X	X	X	X	X		X	X	X	X
VMA (Mix)	X			X	X		X		X			X	X	X	X	X	X	X	X	X		X	X	X	X			X		X			X	X	X	X	X		X	X	X	X	X	X
VFA					X							X						X	X				X	X	X			X		X					X						X	X		
Maximum Theoretical Density	X			X	X		X						X	X			X	X	X	X	X		X	X	X	X			X	X	X	X	X	X	X	X	X		X	X				X
Bulk Specific Gravity								X			X	X	X	X	X				X	X	X		X	X	X			X	X	X	X			X			X		X					X
Density (Lab/Placed)		X					X				X	X			X	X	X		X	X		X	X	X	X	X	X	X		X	X		X		X	X	X	X	X	X	X			
Aggregate Specific Gravity																					X																							
Moisture Content of Aggregate		X			X			X	X			X			X	X	X			X			X	X		X			X		X		X	X				X	X					
Aggregates Plasticity Test																X																												
Permeability																																							X					
Sand Equivalent					X						X					X	X														X		X				X		X					
Fine Particles to Asphalt Ratio	X				X							X	X		X			X	X	X	X				X			X		X							X					X		
Moisture Content of The Mix		X			X		X						X	X		X	X				X			X		X											X	X	X					
Temperature (Mix/Production/Placement)		X						X			X			X			X	X	X									X	X							X					X	X		
Stability				X																						X										X							X	
Moisture Susceptibility Tests				X											X	X					X		X			X				X									X					X
Wheel Tracking Test				X																																			X					
Los Angeles Test					X						X																																	
Modified Asphalt Special Tests					X																																							
Asphalt Recycling Agent						X																																						
RAP Gradation								X	X							X													X									X						
RAP AC								X								X	X						X						X								X	X			X			
Emulsion Residue															X													X																
Moisture In RAP					X											X													X												X			
Binder Content In RAS																X																												
RAS Gradation																X																												
Moisture In RAS																X																												
Stripping Test																	X					X																	X					
Contaminates																									X																			

2. Data Collection And Database Development

2.1 Project Selection Criteria

The objectives of the study require the creation of a framework that allows evaluating the effectiveness of flexible pavement quality control program of Wisconsin DOT with respect to long-term pavement performance. The framework needs to relate the pavement performance at a given location to individual data points recorded in the mix production, and construction (placement) stages. These data points are documented per the requirements of the QMP. To achieve this goal, a holistic, relational database is developed based on the available information of Wisconsin DOT projects and other sources. This means that data residing on different databases within the DOT are connected based on their geolocation to allow for geospatial tracking of the performance of the roadway network within the jurisdiction of the DOT.

Due to the complexity of connecting the available databases and sources of information to field performance, the research utilized a layered approach in order to meet the research objectives. The current HMA quality practice as stated in Standard Specification 460.2.8 (Appendix I) focuses on four mix properties: (1) Gradation, (2) Asphalt Content, (3) Air Content (lab and field), and (4) VMA. These parameters are commonly used to indicate the quality of the construction and production of the pavement. To understand the relationship between these properties and pavement performance, search in the available databases was conducted to characterize projects with respect to:

- Variability degree of each property of the HMA mixtures.
- Pavement structural design (Number of HMA layers, Layer thicknesses, NMAS, Base thickness, etc.)
- The rate of deterioration.

To fulfill the objectives of this study, it is necessary to obtain sufficient information for HMA construction projects that can be used for comprehensive relational database development. Therefore, some filtration is needed to assure that the conclusions of this study are useful to the state of practice and focused on quality rather than branching to other parameters given the scale of information available in the Wisconsin state databases. The following criteria are followed in selecting projects to incorporate in the study.

- Construction date ➔ More than five years
- HMA production ➔ more than 10,000 tons
- Traffic level ➔ greater than 3 million ESALs
- Project Length ➔ greater than one mile

The Wisconsin DOT stores data for thousands of construction projects over the years. Therefore, the above criteria are crucial to focus on the study of projects that would provide enough data points per project. In the collection of data, efforts were made to avoid overlay projects. This has proven to be difficult to achieve given the way data is currently stored.

Applying the selection criteria listed above, 158 projects are identified. While collecting data for all Wisconsin DOT inventory is a significant undertaking, the true challenge was to obtain mix production data. This is because projects of this age have their production data archived as paper copies (not digital files), some of the mix production data needed to be obtained from the individual contractors. Due to these challenges and other ones related to data continuity, the available complete data for 30 projects is obtained. The details of these projects are listed below:

- Range of construction year: 2008-2013
- Mix production range: 18,000 to 177,000 tons
- Length range: 2 to 21 miles
- Pavement Condition Index: 9-100
- Projects distributed over 20 counties and cover projects in all regions of Wisconsin DOT.
- Asphalt Content range from 4.0 – 6.1 %

Data sources related to these projects are housed in different locations. The project plans and structural designs are obtained from the Wisconsin DOT. The data for mix production, construction quality, and long-term performance are discussed below:

Production Quality Data: This data is mostly obtained from individual contractors. Two contractors constructed these projects such that 11 projects were constructed by one, and 19 by the other. Production Data is recorded in lots and sub-lots, and the HMA mixes production is measured in tons. After digitization of the obtained documents, production dates were matched with placement dates (found in quality control reports) to identify the starting location of each lot. Placement yield rates are used to convert tonnage of production into areas of pavement constructed. The conversion used is 115lb/yd^2 per one inch of thickness. Therefore, assuming a constant thickness of each layer throughout the pavement mat, the conversion factor is used to estimate distances of pavement covered by the production. The original plans for the pavement are used to convert the distances to stations numbering on the plans. It is important to note that many of the pavements were constructed on multiple lifts at different dates and thicknesses. This was accounted for during the conversion by obtaining construction details that reflect the lifts construction dates and thicknesses. Majority of the selected projects are found to have two lifts with the NMAS of 12.5 mm for the upper lift and 19mm for the lower lift. The recorded NMAS for the mixes is used to differentiate between layers. The attributes like theoretical maximum specific density (Gmm), percent air voids in the mix (Va), percent air voids in the mineral aggregate (VMA), and asphalt content (AC) are extracted from the product information and used further for the analysis. All the extracted information stored on Microsoft Access® software as an independent dataset, which further incorporated with others to form a comprehensive relational database. The level of collaboration from the contractors was exemplary in the collection of the raw production information. The distribution of the projects in terms of the counties with at least one project is shown in Figure 2-1. Table 2-1 lists all the projects extracted from the WisDOT database.

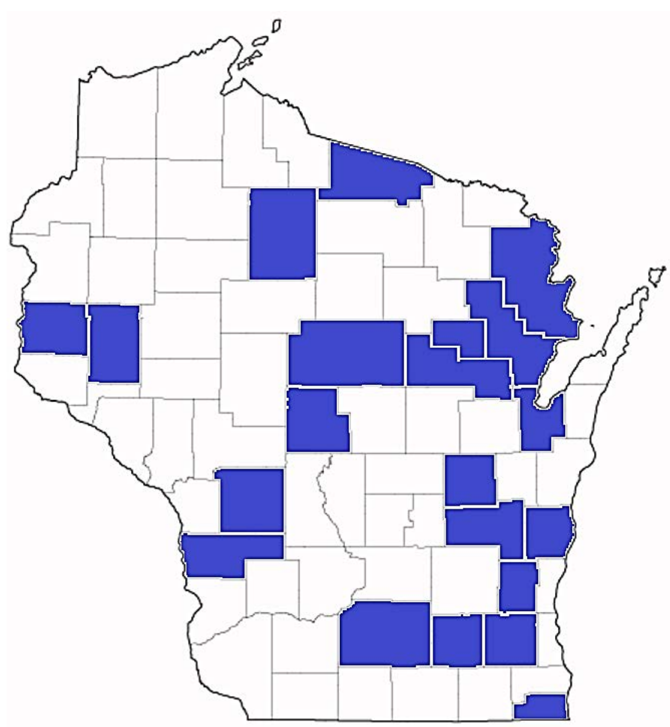


Figure 2-1 Distribution of the Wisconsin counties containing extracted projects data

Table 2-1 General Information on the Selected Projects

Project ID	Title	County	Route	Location in The State	Length (Mile)	Construction Date (Year)	Number of Surveys*	Number of SNs‡	HMA Usage (Tons)	Traffic Level (ESALs)
1017-01-70	Tomah-Camp Douglas WB	Monroe	IH 90	SW	10.61	2010	4	8	176,737	12,469,900
1020-03-74	Hudson - Menomonie	St. Croix	IH 94	NW	6.39	2008	4	14	33,117	N/A**
1053-02-73	Marathon City - Wausau	Marathon	STH 29	NC	3.72	2010	4	6	61,240	9,000,000¥
1053-02-78	Abbotsford - Wausau	Marathon	STH 29	NC	5.10	2010	4	4	91,780	N/A**
1060-31-70	East-West Freeway	Waukesha	IH 94	SE	2.25	2011	4	4	38,405	19,637,000
1066-00-71	Madison - Milwaukee	Jefferson	IH 94	SW	5.33	2011	3	10	84,000	12,285,900
1104-10-70	Milwaukee - Green Bay Road	Fond Du Lac	USH 41	NE	6.19	2008	5	9	140,000	25,433,200
1120-10-80	STH 26 - Breezewood Lane	Winnebago	USH 41	NE	2.89	2013	3	4	56,320	22,549,700
1120-11-83	STH 26 - Breezewood Lane	Winnebago	USH 41	NE	1.61	2013	2	2	42,000	22,549,700
1211-17-71	CTH X/Webster Ave - IH 43	Brown	STH 172	NE	3.34	2010	4	6	61,938	6,044,400
1211-18-71	USH 41 - CTH X/Webster Avenue	Brown	STH 172	NE	2.09	2010	4	6	30,340	8,030,000
1221-14-60	Port Washington - Sheboygan	Sheboygan	IH 43	NE	7.44	2011	4	7	52,000	9,271,000
1370-15-70	STH 16 Waukesha - Oconomowoc	Waukesha	STH 16	SE	3.60	2010	4	7	129,400	5,504,000¥
1590-13-71	CTH R - N Jct USH 141	Marinette	USH 8	NE	8.10	2009	5	8	53,000	2,248,400
1600-21-71	Wittenberg - NCL	Shawano	USH 45	NC	15.11	2008	5	14	114,140	2,306,800
1610-41-60	Westboro - Prentice	Price	STH 13	NC	14.04	2011	4	13	32,606	2,109,000
1620-00-79	Marshfield To Spencer	Marathon	STH 13	NC	3.26	2011	3	3	30,355	2,469,500
1646-07-74	Westby-Viroqua Bypass/4-Lane Sec	Vernon	USH 14	SW	4.90	2011	3	5	36,762	3,774,100
2210-07-70	Bluemound Road	Waukesha	USH 18	SE	1.82	2011	4	2	37,912	2,117,000
2420-02-70	256th Avenue	Kenosha	STH 75	SE	8.40	2011	4	8	50,000	620,500
4010-05-70	STH 144 R & U	Washington	STH 144	SE	5.57	2011	3	7	62,300	1,025,650¥
5420-03-70	Packers & Northport, C of Madison	Dane	STH 113	SW	2.52	2011	3	5	36,000	3,358,000
5420-03-71	Packers & Northport, C of Madison	Dane	STH 113	SW	2.14	2011	3	7	41,000	3,358,000
6140-00-75	Friendship - Wisconsin Rapids	Wood	STH 13	NC	5.77	2010	4	6	32,844	2,012,400
6200-15-71	Oshkosh - USH 10	Winnebago	USH 45	NE	4.93	2009	4	7	32,800	N/A**
6430-10-71	USH 41 - USH 10	Winnebago	STH 76	NE	15.31	2009	5	7	64,250	34,360
8110-06-61	Connorsville - Bloomer	Dunn	STH 64	NW	6.45	2010	4	7	38,900	503,700
9093-03-70	Price County Ln. - Silver Beach Dr.	Vilas	STH 70	NC	8.62	2009	5	6	36,545	788,400
9150-04-71	STH 32 - East County Line	Oconto	STH 64	NE	12.23	2010	4	9	75,761	759,200
9660-01-60	Shawano - Langlade	Menominee	STH 55	NC	18.56	2010	4	14	33,858	503,700

* Refers to the number of distress surveys conducted during the service life until 2017.

** ESALS information was not available.

¥ Average of the ESALs for the project.

‡ Refers to the number of segments a given project is divided into by WisDOT's Pavement Management System. Usually, each segment is about 1mile long.

Construction Quality Data: Controlling the quality of construction is an essential part of each QMP. Since the way in which pavement is constructed significantly affects its performance. In the state of Wisconsin, the relative density of the compacted mat is collected to serve as an indicator of construction quality for QMP. These densities are taken by nuclear gauge devices in accordance with the state established procedures. Density tests may be performed by the contractor if the contract contains QMP density testing obligations. Then verification tests will additionally perform by the WisDOT to validate the contractor reported results. Otherwise, WisDOT performs all density tests. Considering that once the testing method selected for a project, it should be used on the entire length of the project (WisDOT CMM 8-15) [93]. The recorded density is typically referenced by the station and offset from the right edge of the mainline according to the project plans. For this research study, construction data is obtained from Wisconsin DOT highway data management portal named "Atwood." Required density for each measurement was used to determine whether the measurement was related to the top or the bottom lift. The collected information then stored separately for each project along with the pertinent information. The attributes of this dataset including the pavement date, test date, required density, and measured density, are used for further analysis.

Performance Data*:* According to the Wisconsin DOT performance improvement report known as MAPSS [97] the performance data is recorded by the DOT for the nearly 12,000 miles of state highway in Wisconsin, which supports 60% of the vehicle miles traveled across the state. Every two years, for a given highway section, the DOT conducts a performance survey and record the performance based on the visual signs of pavement distress such as cracks, ruts, and potholes. The first year the department had complete statewide coverage using the Pavement Condition Index (PCI) rating method was 2011. Before 2011, the department assessed pavement condition using a different methodology known as the Pavement Distress Index (PDI). The highway inventory within the Wisconsin DOT is divided into segments called Sequence Numbers (SN). Each SN is typically about one-mile long. The performance is recorded through distress surveying of one-tenth a mile of a given SN such that the third tenth is the section surveyed. Distresses are recorded by type, level of severity, and extent. The beginning and end of the SN are referenced in the DOT database (PIF) by landmarks such as intersections, county borders, rivers, etc. Using Google Earth, these landmarks were identified. The original plans for the paving projects were overlapped on the Google Map to identify the SNs belonging to a given project. The beginning and end stations of each SN then identified. The Global Position System (GPS) coordinates were also manually recorded based on the location of each SN using Google Earth software. The performance survey reports contain several distresses including Rutting, Transverse Cracking, Alligator Cracking, and Longitudinal. These attributes are recorded along with their different severity levels.

Figure 2-2 shows the data fragments used to connect pavement history into one relational database. The figure illustrates the details mentioned in the previous points.

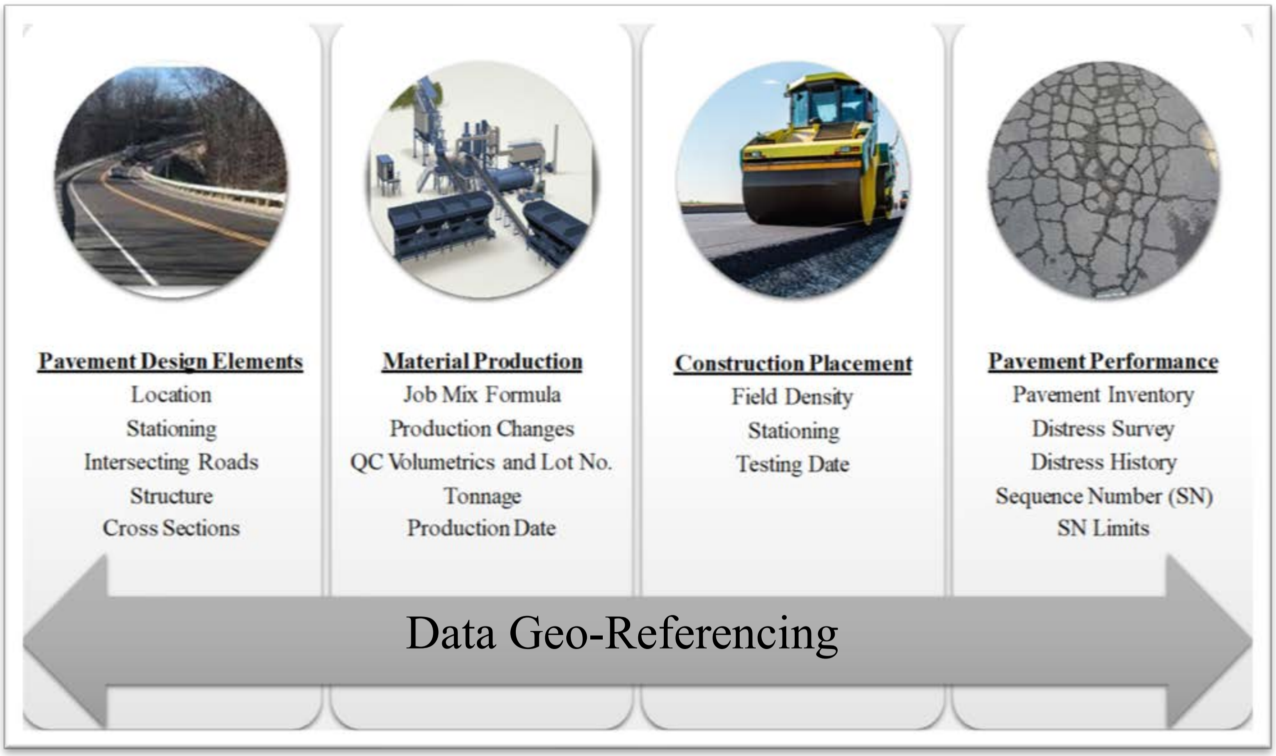


Figure 2-2 Data Fragments used to Create Relational Database

2.2 Developing a Holistic Database

In this study, the challenge was to reference and coordinate the different referencing systems to create a cohesive geo-reference system for the investigated pavement projects. Therefore, the research approach started with connecting the different data sources into geo-referenced locations and compiling them into a holistic relational database. Data pertaining to three stages of the pavement life; namely material production, construction, and in-service performance, are collected initially for compliance in the case of production/construction, and for maintenance/liability in case of performance. Therefore, the data housing, labeling, and level of details are not designed to be interconnected or meant for further analysis beyond these objectives. Example of such databases available in the WisDOT is shown below in Table 2-2.

Table 2-2 Databases available for the Study

Source Database	Description
Pavement Inventory Files (PIF)	Descriptions and pavement distress data for each sequence number (SN) are provided in the PIF database, including International Roughness Index (IRI), Pavement Condition Index (PCI), rutting depth, and individual pavement distress measurements (Alligator, Transverse, and Longitudinal Cracking). This database also includes highway number, surface year, and segment termini description, a directional lane of measurement, date of measurement, region number, and county. Data from the PIF provide a direct measure of flexible pavement performance over a flexible base.
Construction Reports/Plans	Attributes of projects constructed in each year are detailed, including such fields as prime contractor, base type and/or preparation (DGBC, OGBC, milled, pulverized, rubblized, etc.), thickness asphalt layer placed, mixture design (SMA, Superpave ESAL series, etc.), lane-miles of paving, and project identification number. The paving year and highway number in this database merged with the SN in the Meta Manager and PIF databases to develop a holistic database.
Highway Quality Management System	This database developed by Atwood Systems contains important data for QMP material properties. This database cannot be electronically linked to the databases above and requires manual extraction. The research team obtained electronic mix designs and QMP quality control data charts and moving averages to supplement this database.

Different pavement related agencies or even different divisions within an agency often have data collection methods that are not necessarily compatible with the others. Therefore, it is common to see a variety of different methods of referencing the pavement sections locations within the network of roadways. For example, in this study contractors are using a construction project numbering scheme of station numbering, while the DOT performance division use highway sequence number (SN) method for referencing the performance surveyed locations. The stations are length-dependent numbers which are based on the construction plans and length of the roadway. On the other hand, Sequence Numbers (SNs) are DOT defined segments of the highways that have referenced locations. These references are based on either landmarks or distances. NCHRP Synthesis 335 [98] reported a survey in which 96% of highway agencies indicated using the milepost/logpoint method for referencing, while 15% additionally use landmarks in referencing. The milepost referencing method requires each roadway to be given a unique name and/or number, and a distance along the route from a given origin to define points along the route. The research team decided to use a linear referencing system, which consists of a set of procedures and a method for specifying a location as distance, or offset, along with a linear feature, from a point with a known location. Thus, this method includes three components; network of highways, location referencing method, and datum. The location reference method refers to how to identify a single location in the field. The primary domains of location referencing methods include administrative (e.g., county), linear, geodetic/geographic, and public lands survey. Common linear location referencing methods include route/milepost, link node, reference point/offset, and street address [41].

Based on the available information in PIF database, the number of SNs for each project is first determined. Locations of start and end of SN are identified using the landmark references in the PIF. For some of the SNs, due to the unavailability of a detectable landmark, the length of consecutive SNs is used to determine the start and end location of each SN. Finally, GPS location of each SN was determined and used to match with the station locations in the plans. These steps were repeated for all the selected projects to form a database network level positioning system that facilitates the connection of data points in the database. Based on the points above, data belonging to the selected projects are collected. All the construction and production data points are given stationing location following the numbering in the plans sheets. The data is then grouped by the SNs. At this stage, production, construction, and performance data can be connected. The data points for the three main components of the pavement history are then overlapped based on their geo-location. This allows for investigating localized variability in construction, the relation between production variability and construction variability, and the effect of variability and compliance on long-term performance. The following schematic (Figure 2-3) presents the process followed in creating the relational database.

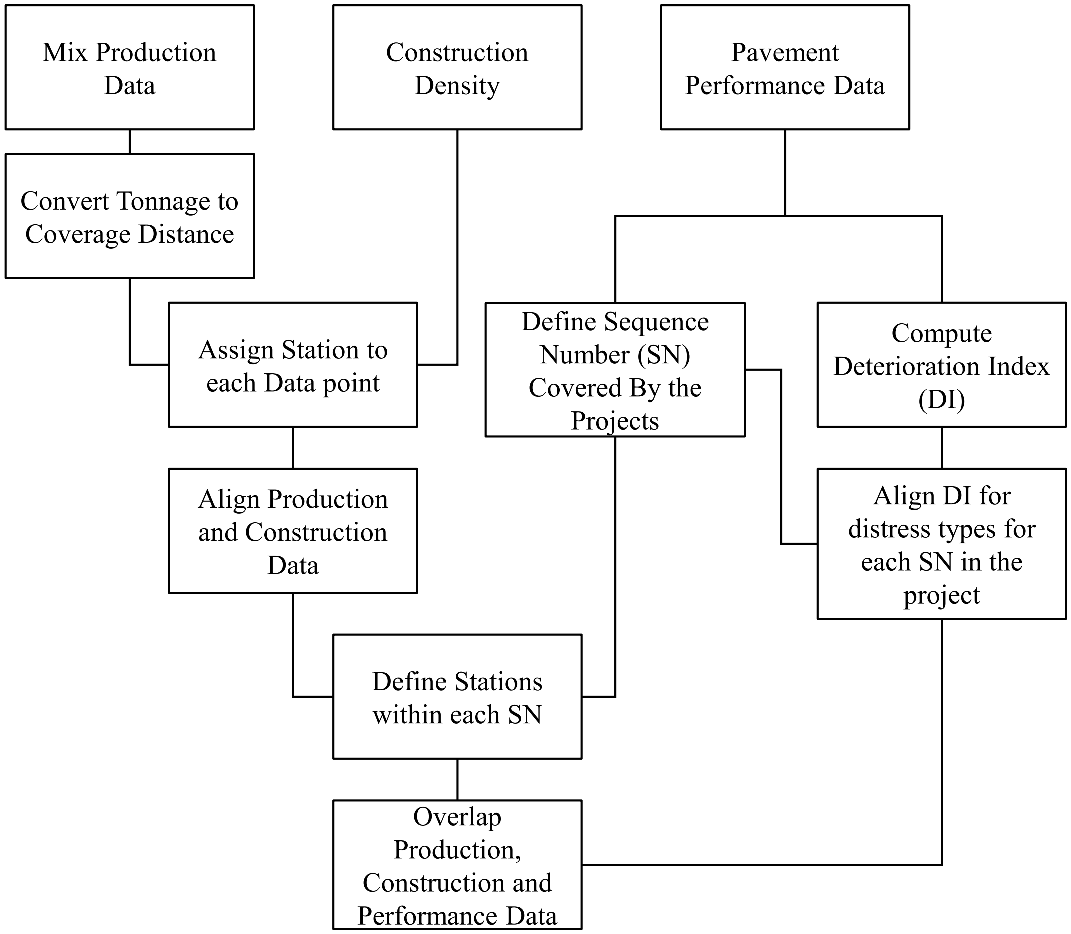


Figure 2-3 Process to Connect Data Sources to Obtain Relational Database

This method of conversion selected due to its convenience as well as the increasing use of geo-referencing methods in pavement industry. In fact, nowadays, Geographic Information System (GIS) and Global Positioning System (GPS) technologies are propelling the use of coordinate-based referencing systems to identify points along routes. NCHRP Synthesis 335 [95] stated the importance of using such systems for different pavement authorities nationwide. It identified that 35% of surveyed agencies using longitude and latitude, and 13% using state plane coordinate or related systems to reference the location of their performance measurements. Several works have been done in the field of pavement management that tried to improve the quality of decision making, analyzing, and reporting by use of geographic information. [96, 97, 98, 99, 100]. Geospatially referencing the data has a wide application in airport management [101] Transportation Management [102] Environmental Resource Management [103] Earthquake

Management [104, 105]. The fundamental aspect of all these systems is linking the separate databases by using their geographical location [106].

Figure 2-3 does not include activities such as digitization of hard copies and some other details. The figure is intended to present the outline for creating relational database assuming all data is digitally available in a format that allows manipulation using commercial database software. For this study, Microsoft Access software is used to execute and process the large data points and integrate the different databases into one relational database. The Access-based relational database is built on three individual core databases, namely, Production, Construction, and Performance. The design of database relationships is shown in Figure 2-4. The structure of the database is such that it allows for extracting queries related to a level of quality under study, yet all information with respect to other components maintain their connection. For example, in order to investigate the effects of HMA density on rutting of pavements within the database, an Access-based query was used with rutting as the search object. The query will then include all pavement sections with rutting as reported distress, in addition, it will pull the construction and production data of these sections. All the data retrieved is then presented in one table that includes the different fragments shown in Figure 2-2. Therefore, the extracted information is used for further analysis for optimized investigation process, while the structure of the global relational database remains to be separate and independent. By using this feature, many queries regarding the effects of binder content, Va, VMA, In-Place Density on the individual distresses such as Rutting, Alligator, Transverse and Longitudinal cracking were extracted and analyzed, which the results are shown in next chapters.

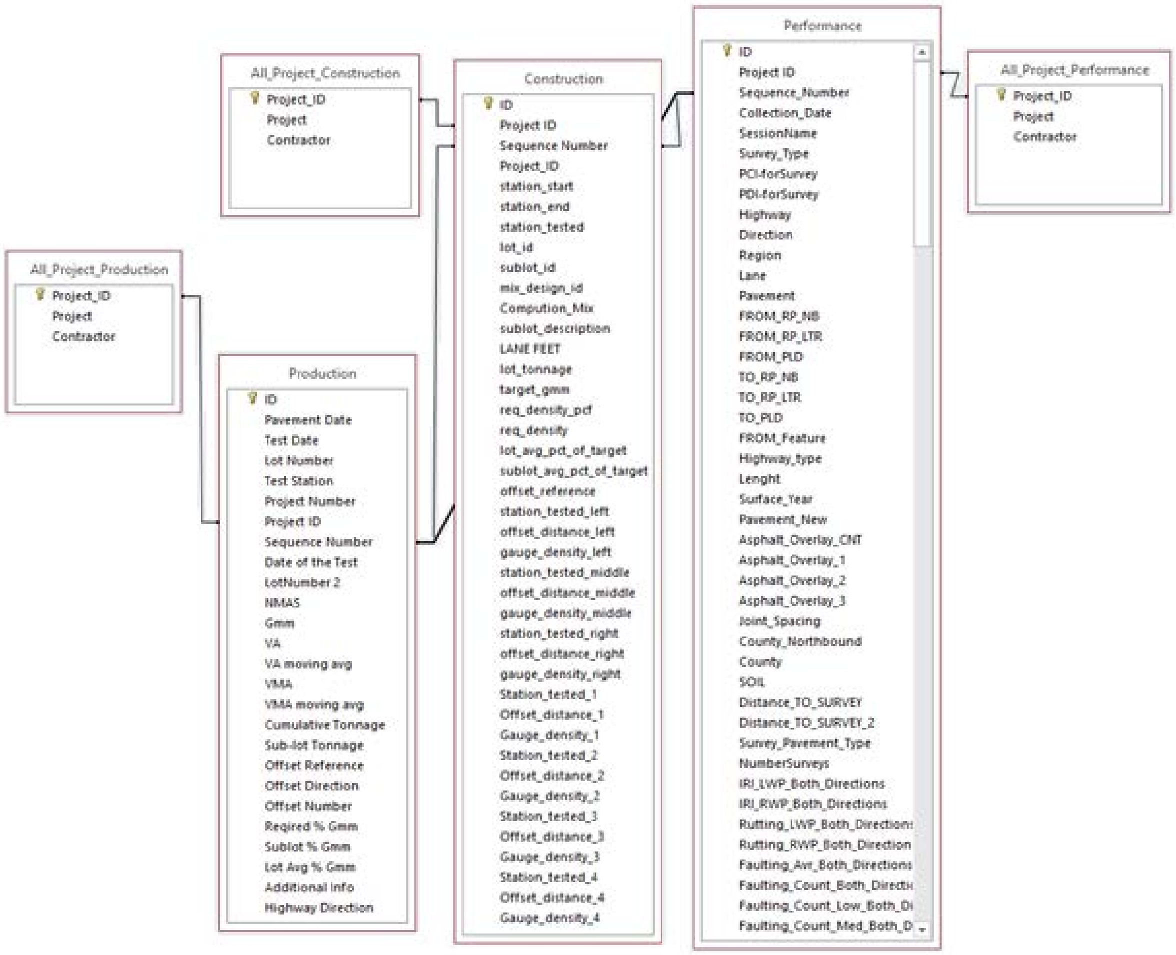


Figure 2-4 Database structure created using Microsoft Access®

2.3 Calculation of Deterioration Index (DI)

The pavement performance data is recorded by the DOT for different distresses. The distresses are surveyed every two years for a given SN. For flexible pavements, the distress survey is conducted to calculate Pavement Condition Index (PCI) according to the ASTM D6433 [107]. However, for this study, individual distresses are more valuable for tracking the potential correlation of quality control indicators to pavement durability. Based on the PCI method, each distress is recorded in terms of severity level and extent. The recorded data obtained from the DOT underwent multiple steps in order to integrate it into the relational database for analysis. Therefore, four types of distresses are studied as representatives of long-term performance of the pavement were selected to use for analysis. These types of distresses are as following:

1. **Rutting;** which can be the result of a permanent reduction in the HMA volume due to consolidation, traffic densification, or permanent movement with the constant volume due to the plastic deformation or shear. It can also be a combination of them [108]. This type of the distress was selected to investigate the effects of construction and production parameters like density and asphalt content on it.
2. **Alligator Cracking;** also called Alligator Cracking or Fatigue Cracking, happens due to the maximum tensile strain at the bottom of the asphaltic layer of a flexible pavement after repetitions of enough number of vehicular load [109].
3. **Longitudinal Cracking;** is an extension of top-down cracking that begins from the road surface and gradually extends to the depth of pavement, and it occurs along the vehicle driving direction of the road. This type of crack is also a point of interest since it is widespread and has detrimental effects on the serviceability of the pavement.
4. **Transverse Cracking;** is happening roughly perpendicular to the pavement's centerline. Given that the majority of the studied projects do not include any overlays, most of the transverse cracks are correlated with the thermal shrinkage of the pavement. Although in a few overlay projects reflection cracking are also recorded as transverse cracking data. The literature reports a high dependency of transverse cracking on the pavement production and construction parameters [110].

According to Shahin [111], there is a well-established procedure of essential steps in developing the Pavement Condition Index (PCI) value, which is a widely accepted parameter for describing the pavement distress state (ASTM D6433-09) [107]. However, this study focuses on the four types of distresses mentioned above. Based on the PCI method, the first step is to define each pavement distress types, the level of severity, and extent of distress. The next step is calculating the deduct value by using the deduct curves developed by Shahin [111], the deduction value curves are shown in Figure 2-5, Figure 2-6 and Figure 2-7 for Rutting, Alligator cracking, and Longitudinal/Transverse cracking respectively.

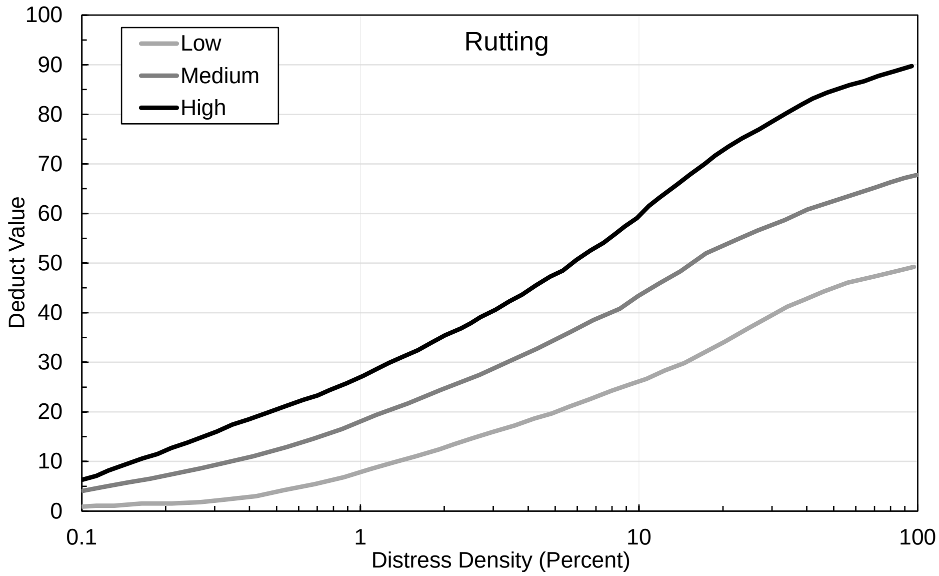


Figure 2-5 Deduct Value curves for different severities of Rutting (after Shahin 2005)

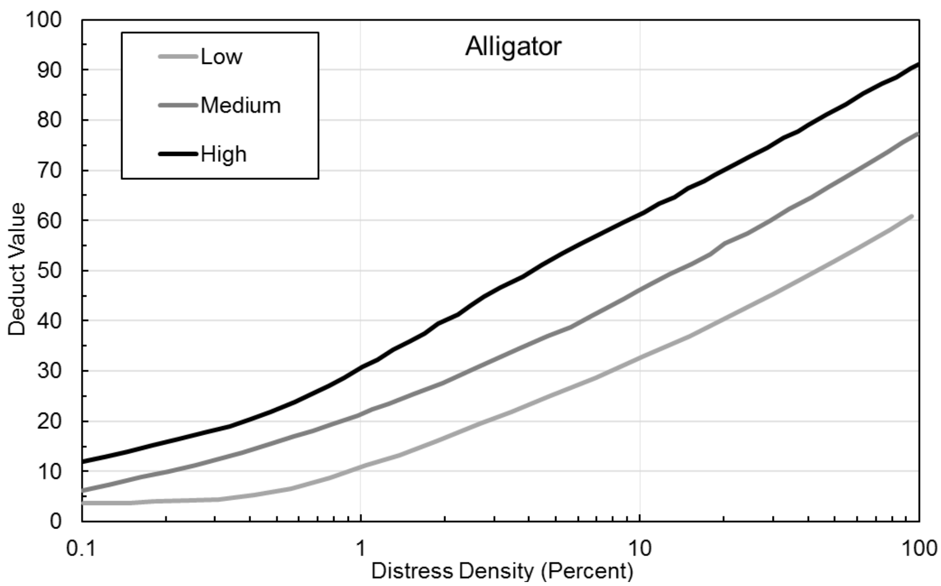


Figure 2-6 Deduct Value curves for different severities of Alligator Cracking (after Shahin 2005)

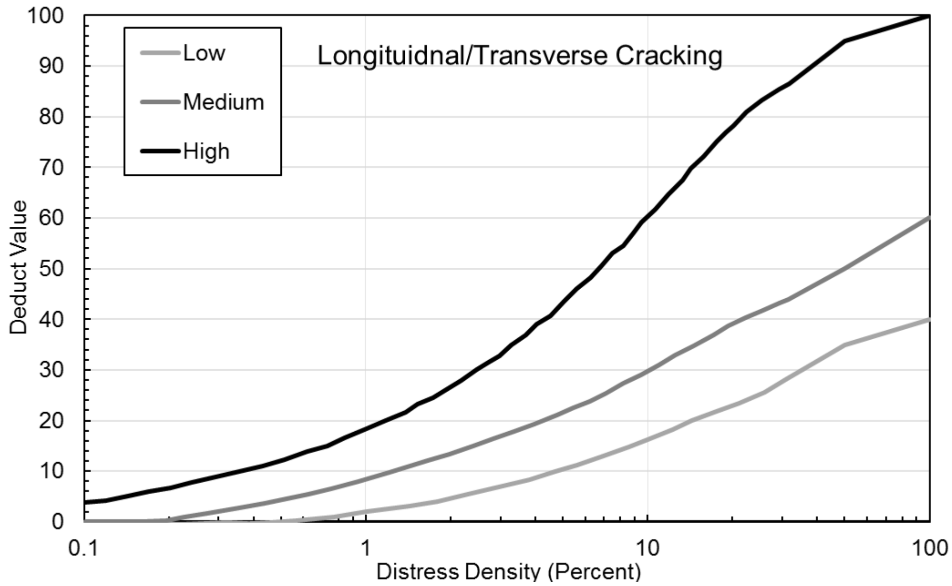


Figure 2-7 Deduct Value curves for different severities of Longitudinal/Transverse Cracking (after Shahin 2005)

In order to calculate the "Deterioration Index" values, the data related to the four selected distresses of Rutting, Alligator, Longitudinal and Transverse Cracking were identified and isolated from the state DOT performance survey in PIF database. This data contains information regarding the numbers, area, and severity of each distress. Based on the length and area of surveyed sections, the density of each severity level calculated for individual distresses. Finally, the equations describing the curves shown above are used to convert the density at a given severity level into a deduct value, then the total deduct values for all three levels of severities are added together representing a value for the overall deterioration in the pavement due to this given distress. This value is called Deterioration Index or DI, and each distress type is now receiving a single value reflecting its level of deterioration as a score out of 100. Therefore, the high DI represents a high deterioration of the pavement.

This process has been employed by one of the authors in a previous study [65]. This step is conducted to isolate the degree of deterioration per distress type rather than the generalized PCI. As a result, each SN possesses five distinct deterioration indices; one for each of the four distresses, and one represents the summation of all distresses deterioration indices. The DI values were calculated for all conducted performance surveys. By comparing the survey time and construction time of the pavement, the age of pavement at the time of the survey was determined. By matching the DI values versus the age of the SN for each performance survey, the deterioration rate also can be easily calculated. During the process, it was noticed that in some cases, the pavement type for several SN was recorded differently than the other data sources like the construction maps. Also, due to the contradiction of PIF information of construction year with the construction information, the reported construction years in the contractors' documents are used.

Separate field surveys were conducted by the research group in accordance with the Wisconsin Pavement Distress Survey Procedures to evaluate the pavement performance independently. Based on the results, DI values calculated with the same procedure. However, since research team reported distresses for every 25 ft and at three different locations of pavement including right wheel path, left wheel path and centerline, the calculated DI values have higher resolution compared to the WisDOT performed survey. This higher resolution enabled identification of the localized problems and construction-related problems. Based on the field surveys, it is observed that there is a high level of construction-related longitudinal cracks on the selected pavements. The research team decided to separate the construction-join longitudinal cracks from the in-lane ones in order to help better understanding of the sources of the performance problems. The DI values then calculated based on for four selected distress plus the one for construction-joint Longitudinal cracks.

It is important to note that the DI is calculated based on the curves provided in ASTM D6433. The authors of this report did not validate the relationship between the distresses and their corresponding DI. This is beyond the scope of this project. The DI method provides a useful tool to quantify each distress independently for further analysis.

In addition to the field surveys, by using the already identified GPS coordinates of the SNs ending and starting points, the Google Earth satellite and aerial photos were used to verify the extent of reported distresses in the field. The problem with this approach was the date of the available photo images in the Google database. Having the clear and up-to-date photos of the selected projects was a challenge. However, field surveys and taken photos helped to overcome this issue.

2.4 Data Analysis

After calculating the DI values and establishing a network level geo-referencing system, the relational database is completed and can be applied on a global level to study all projects' data for generalized trends. The relational database can also be used on a project by project basis to study the influence of quality compliance and variability of long-term performance. For the purpose of this research, the project level analysis as well as the network level analysis was conducted to illustrate the capability of this approach to capture the potential relation between quality indicators at the time of construction and long-term performance.

The project-level analysis intends to explore the role of factors not present in the DOT database on performance. Therefore, the on-site visits focused on conducted structure evaluation of the roadways to establish whether observed distresses are resulting from structural deficiencies. It also allows for evaluating the level of conformity and variability of the quality data and their possible correlation with observed performance. This report includes in-depth analysis of 7 projects out of the total 30. Detailed exploration of the QMP quality indicators, along with the field survey and non-destructive field test (FWD) was used in this analysis to allow better investigation of the long-term effects of production and construction issues on the performance. These 7 projects were selected based on the level of construction and production information availability, diversity and

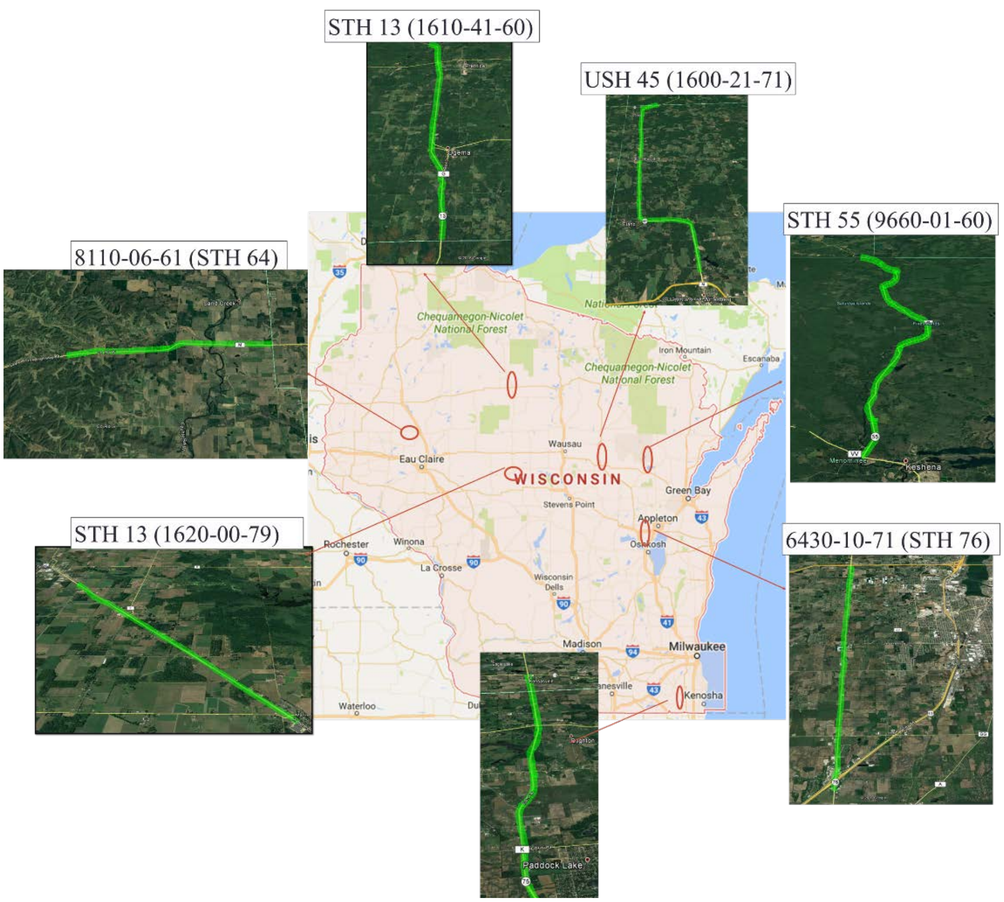


Figure 2-8 Location of the selected projects for project level analysis on the Wisconsin map

severity of the distresses, and feasibility of the field survey in terms of traffic control and accessibility. Moreover, the projects were selected from different location of the state to be representative of the Wisconsin highway network. The Location of these projects is shown in Figure 2-8.

3. Global Level Analysis

In this chapter, the quality indicators are studied for 30 projects extracted from the Wisconsin DOT roadway database. This study provides insight into the potential relationship between quality indicators measured during production and/or construction to the long-term performance recorded on the DOT's pavement information system database as known as PIF. This section starts by studying the distributions of the quality indicators as they are reported. This will help understand potential bias in the data and provide the needed information for further investigation on their influence on pavement performance.

The main challenge in conducting global level analysis is that the resolution of the data is not consistent. The production data is converted and recorded to the area, the placement data is measured by stations, and the performance data is recorded by Sequence Numbers (SN). In order to unify the resolution, the analysis at this level is conducted using the SN as the basis for comparison. Therefore, in the analysis phase, the quality indicators are averaged for each SN. For example, for a project that is comprised of 6 SNs, then the quality indicators for production and construction will be averaged for each of 6 SNs, to be compared with the performance.

The information provided in this chapter raises questions of what happens if the mix quality does not conform to the specifications. More importantly, what is conformity? The analysis of the data as well as employment of statistical modeling (Sections 3.2 and 3.3) attempt to answer these questions. Chapter 4 investigates the implication of these questions at the deeper level as the analysis is conducted at the project level.

3.1 Distribution of Quality Indicators

This section presents the distribution of the sampled data used in this research study. It is important to study the distribution to understand the boundaries of the analyses that follow. In addition, the distribution provides an overview of the variability and conformity of the quality indicators at the global level. In fact, these 30 projects can be considered a sample of the Wisconsin roadway network.

Production Mix Air Voids (Va)

The distribution of the production mix air void shows a wide range of values (1.8% to 5.4%) for the upper lift and much wider range for the lower lift (2.25% to 6.75). However, given that most designs are targeting 4% with the tolerance of +/- 1.3%, the average of the distribution overlaps with this value (3.91% for upper and 4.05 for the lower lift). Figure 3-1 shows the distribution of Va from 2442 individual data points produced by the contractors for 12.5mm mixes used for the upper lift of pavement. The values are the exact measurements, and no average value is included. Figure 3-2 shows the distribution of the 19mm lower lift mixes.

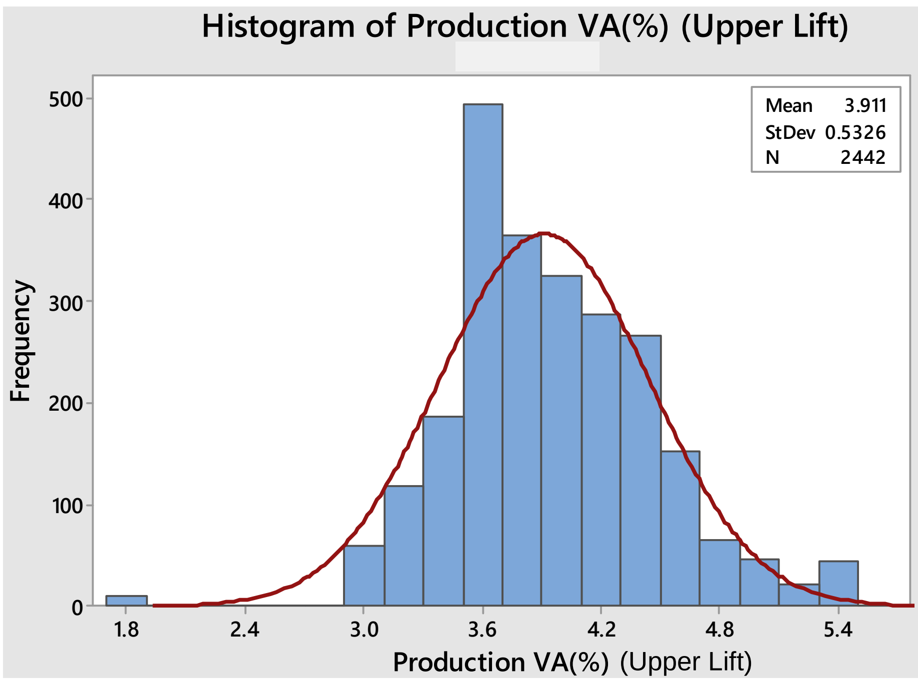


Figure 3-1 Distribution of Va for all Mixes Used for Upper Lift

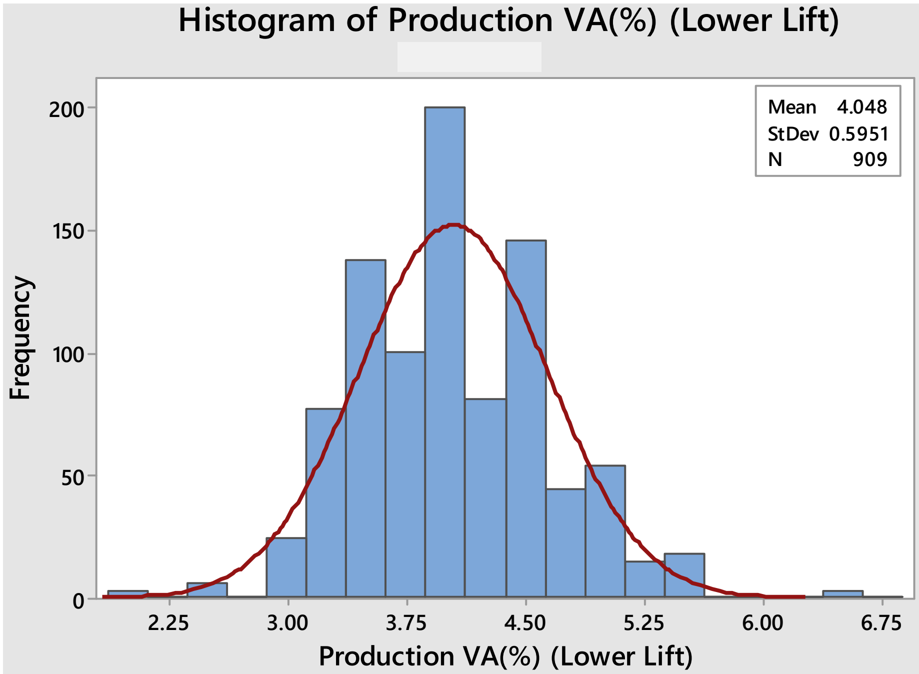


Figure 3-2 Distribution of Va for all Mixes Used for Lower Lift

The distributions in Figure 3-1 and Figure 3-2 conveys the intention of designers to conform to the DOT specifications. The distributions also confirm that for the surface layer the majority of the mixes try to avoid falling below 3 % Va as shown in Figure 3-1. The range of 3.6 to 4.8% contains the majority of the data points. For the 19mm mix, this trend is not as clear. By assuming a normal distribution, the data suggests that 64% of the 12.5mm mixes fall between 3.5% and 4.5% Va. For 19mm mixes the percentage is 60%.

Production VMA

For VMA, the mix design typically aims to exceed the minimum required value. The mixes included in this study are 12.5mm and 19mm for top and bottom lifts respectively. According to the Wisconsin DOT specifications (Appendix I), the minimum VMA for 12.5mm is 14% and for 19mm is 13% with the Job Mix Formula (JMF) limit of -0.5% and warning limit of 0.2%. By examining the distribution of the data, it can be noticed that the average value exceeds these minimums. The VMA for upper lift mixes ranges from 12.75% to 18%, for lower lift 19mm mixes the range is from 12.75% to 16.75%. This means that the bottom lift mixes are generally conforming to the specifications, while for the top lift, about 11% of all data did not meet the minimum level of specification.

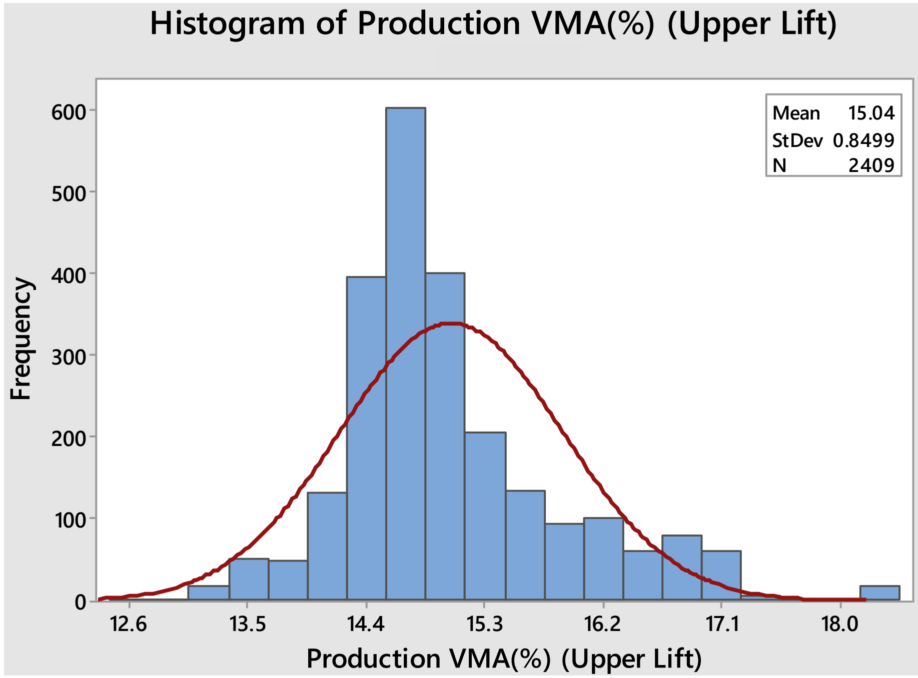


Figure 3-3 Distribution of VMA for Upper Lift Mixes

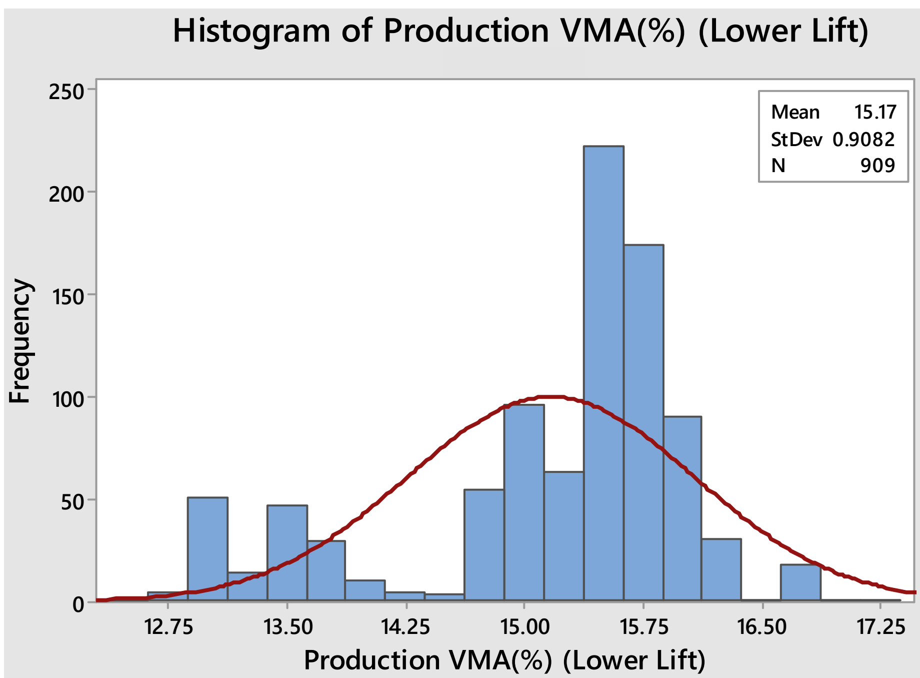


Figure 3-4 Distribution of VMA for Lower Lift Mixes

The distribution of the lower lift in Figure 3-4 shows two peaks. This observation indicates the possibility of two classes of mixes is used in the lower lift. The wide range is a natural product of the specifications as it calls for the minimum value of VMA. On the other hand, it is clear that there is a bias in production towards higher VMA for mixes.

Asphalt Content (AC)

The asphalt content ranged from 4% to 6% for the upper and lower lifts. The upper lift average AC value is slightly higher (5.32%) than the lower lift mixes (5.16%). The distribution of the AC% indicates a bias in the distribution towards values above 5%.

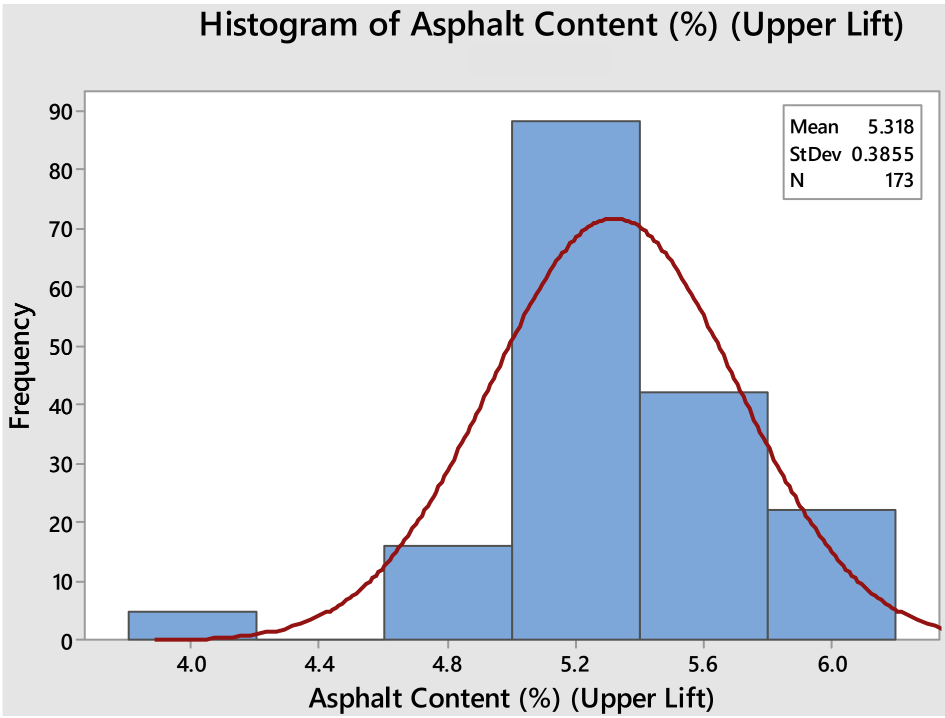


Figure 3-5 Distribution of Asphalt Binder Content Used in the Upper Lift 12.5mm Mixes

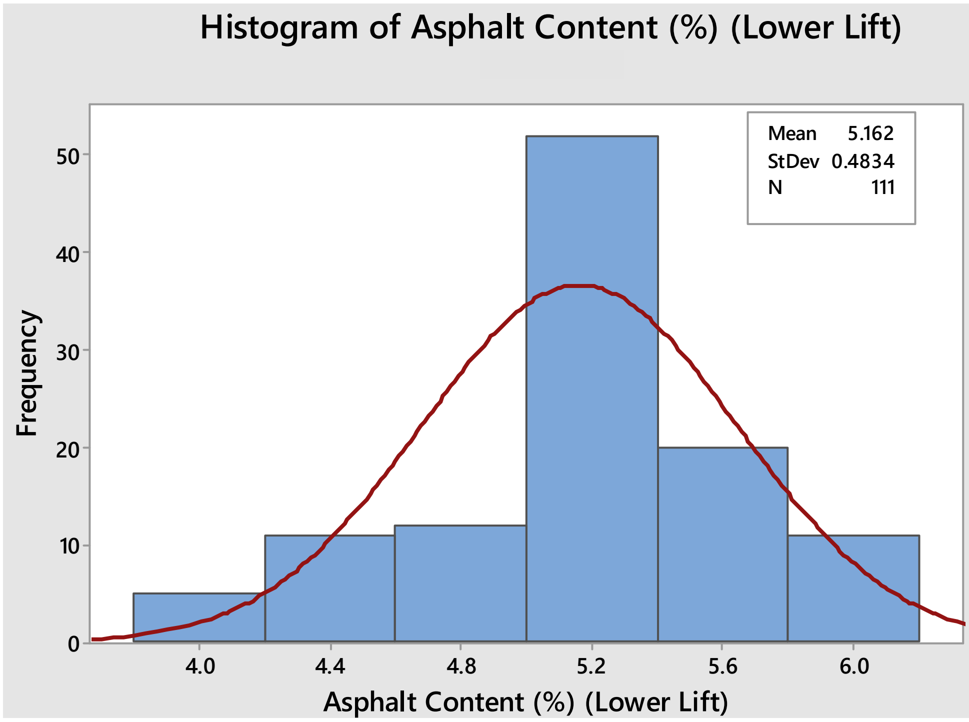


Figure 3-6 Distribution of Asphalt Binder Content Used in the Lower Lift 19mm Mixes

It is important to note that for both types of mixes, the content of 5.2% took place in more than 50% of all mixes produced and included in this study. This represents a challenge to establish a relationship between the AC and performance given the lack of variation in the AC content.

Construction In-Place Density (%Gmm)

The distribution of the In-place density can be used as an indicator of construction quality. The distributions of all the measurements are shown in Figure 3-7 and Figure 3-8. The data pool for this property is the largest of all the data collected.

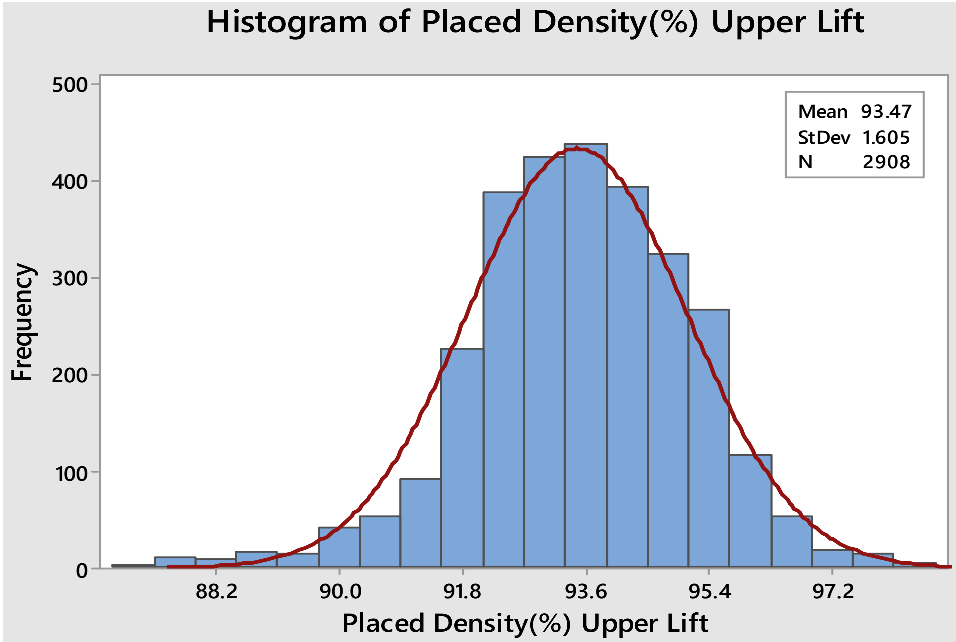


Figure 3-7 Distribution of Compaction Level as Measured by the Relative Density of the Upper Lifts

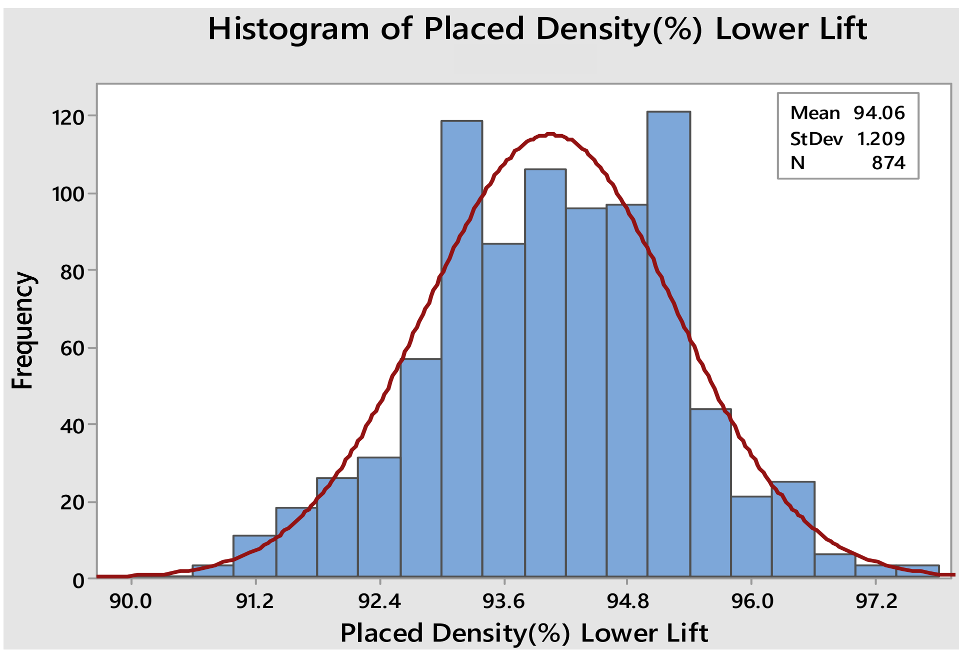


Figure 3-8 Distribution of Compaction Level as Measured by the Relative Density of the Lower Lifts

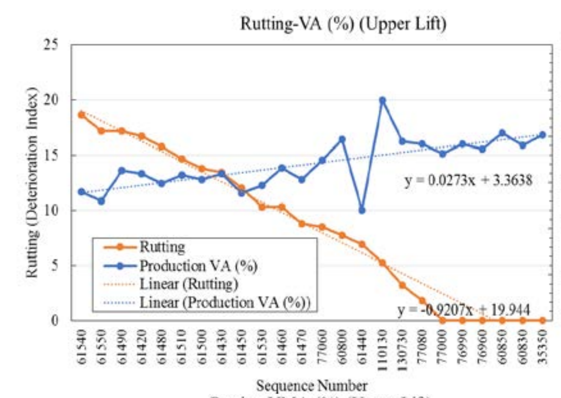
Assuming the data is normally distributed, the mean and standard deviation for the upper and lower lifts can be used to estimate the proportion of tests not meeting the target values for in-place density. For the upper lift, about 11% of the distribution falls below the 91.5% relative density. For the lower lift, the portion below the 90.5% is only 0.16%. Thus, virtually all lower lift compactions met the target density of WisDOT specifications at the time of construction. The statistical analysis studies whether the increase in compaction density is correlated with the better performance. This step helps to validate the currently used thresholds.

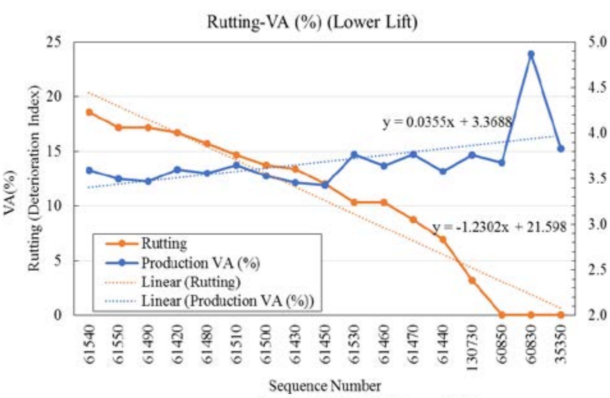
3.2 Distress General Correlations with Measured Quality Indicators

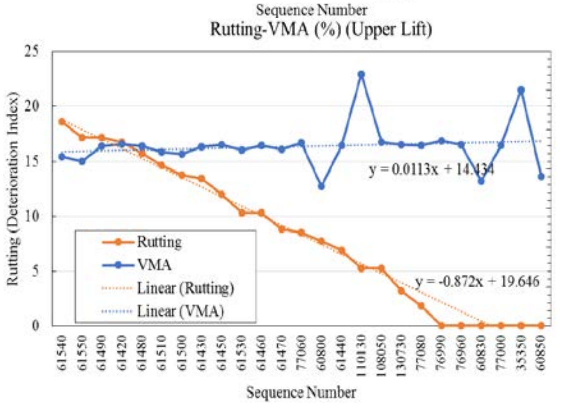
In this section, the SNs that are showing measurable distress values as included in the PIF database were compared to their corresponding average values of the quality indicators. These comparisons serve as screening for observation of general trends. The graphs are drawn based on non-zero value distresses, according to the fourth conducted performance survey which represents performance after 7-8 years of service. This also means that for all the 197 SN included will be evaluated at the same age.

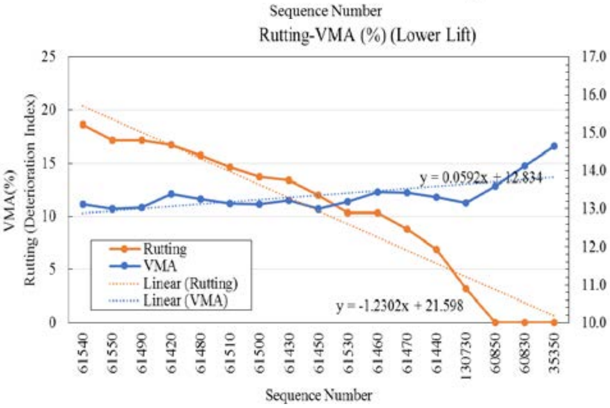
Quality Indicators' Correlations with Rutting

The following plots show the SN exhibiting non-zero values for Rutting and their corresponding quality indicator values. The SNs are plotted in descending order with respect to their rutting DI values; the higher DI means, the worse Rutting problem. Comparing the slopes of quality indicator property against rutting helps in isolating potential correlations for further analysis.









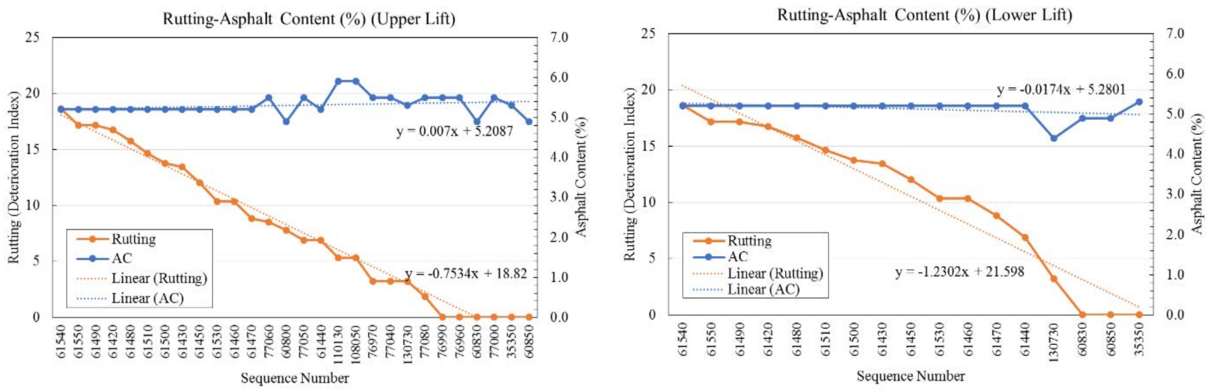


Figure 3-9 Rutting Against Production Quality Indicators

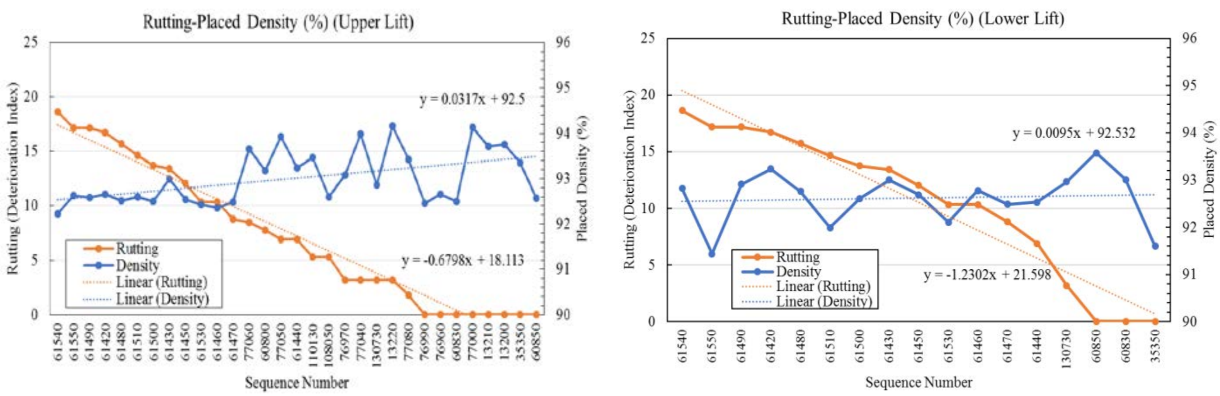


Figure 3-10 Rutting Against Construction Quality Indicators

Observations of Rutting Performance:

1- Potential correlation between Va and Rutting for 12.5mm and 19mm mixes; as mix Va goes below 4%, rutting increases.
2- Relationship with VMA is unclear at this point.
3- No apparent correlation between rutting and AC content. This is due to lack of diversity in the values of the AC content. The AC content is not tested during production like Va. The lack of this information does not allow for conducting a realistic analysis of the correlation between the AC content and performance.
4- Potential correlation between in-place density and rutting for 12.5mm mixes; as in-place density increases, rutting decreases. No apparent correlation with 19mm mixes.
5- Rutting DI is maxed at a value under 20/100. Rutting is recorded in 33 of the 101 SNs of the same age included in this project.

Quality Indicators' Correlations with Alligator Cracking

The plots show the alligator cracking as reported in the PIF. Fewer SNs are reported to experience alligator cracking compared to the rutting.

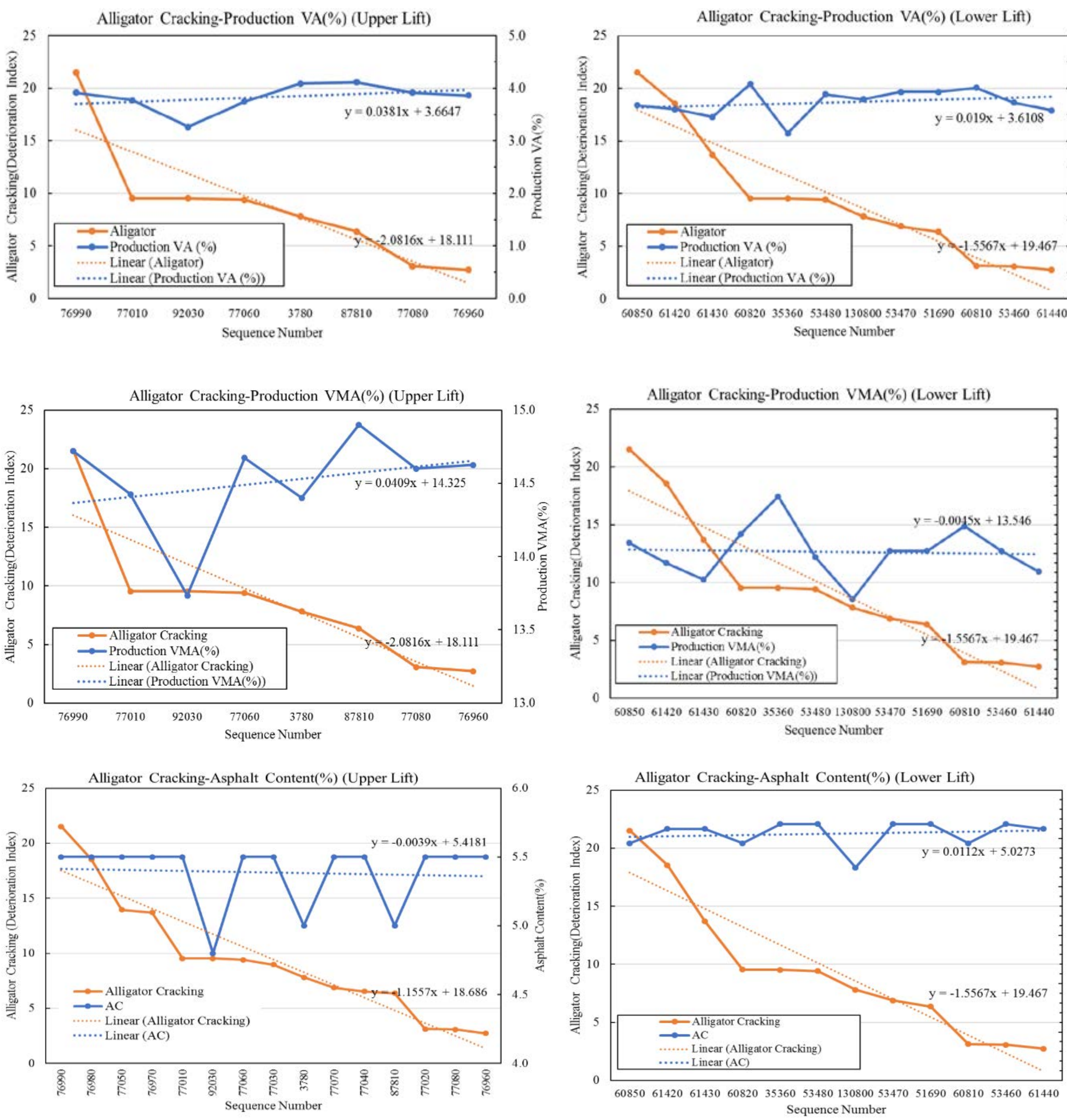


Figure 3-11 Alligator Cracking Against Production Quality Indicators

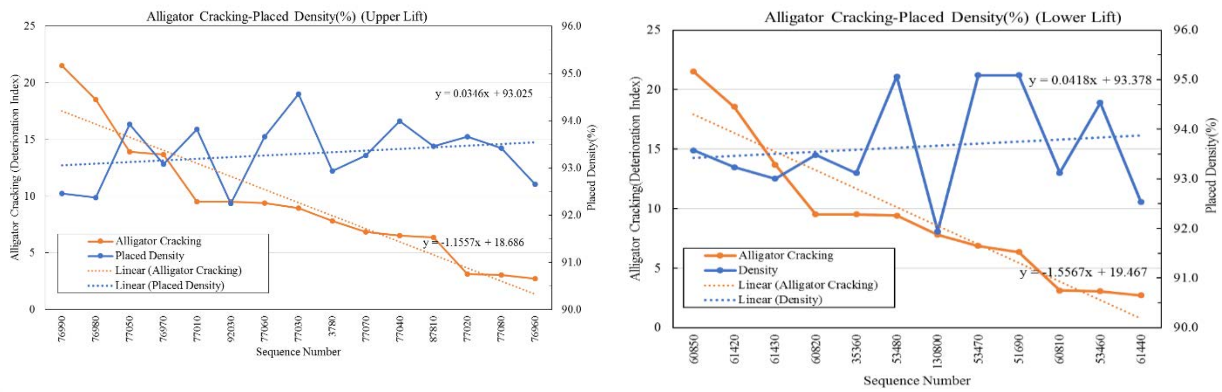


Figure 3-12 Alligator Cracking Against Construction Quality Indicator

Observations of Alligator Cracking:

1- There is a potential correlation between VMA and in-place density with the top lift HMA. By increasing these values, the pavement is showing better performance in terms of Alligator cracking.

2- Alligator cracking DI is maxed above 20/100 and is recorded in 15 out of the 101 SNs of the same age included in this project.

3- Some of the plots in Figure 3-11 and Figure 3-12 do not include all the SNs showing alligator cracking due to the unavailability of their corresponding quality indicator data.

Quality Indicators' Correlations with Longitudinal Cracking

Longitudinal cracking is the most common in the database. The SNs with non-zero DI for Longitudinal Cracking are plotted below.

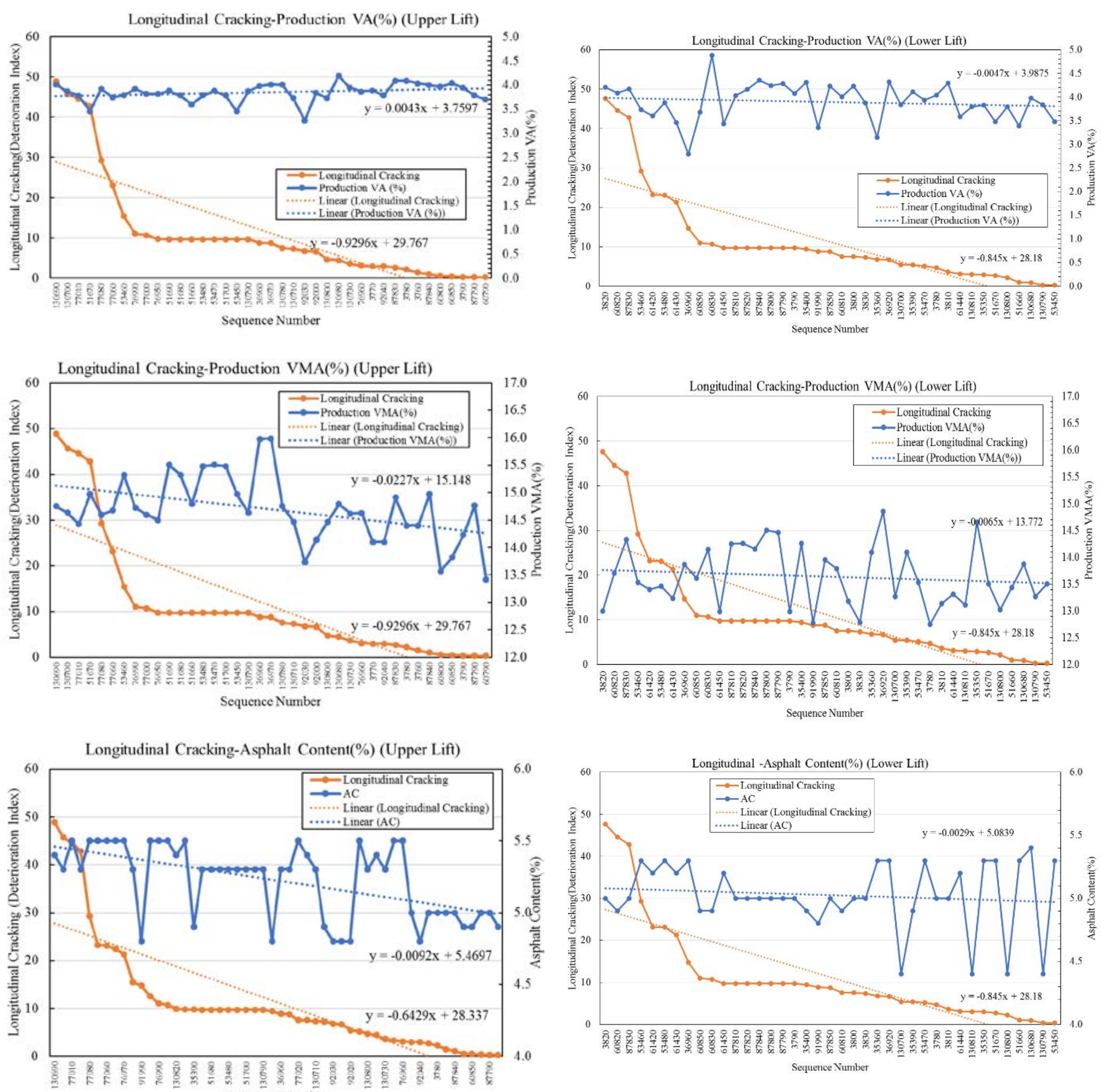


Figure 3-13 Longitudinal Cracking Against the Production Quality Indicators

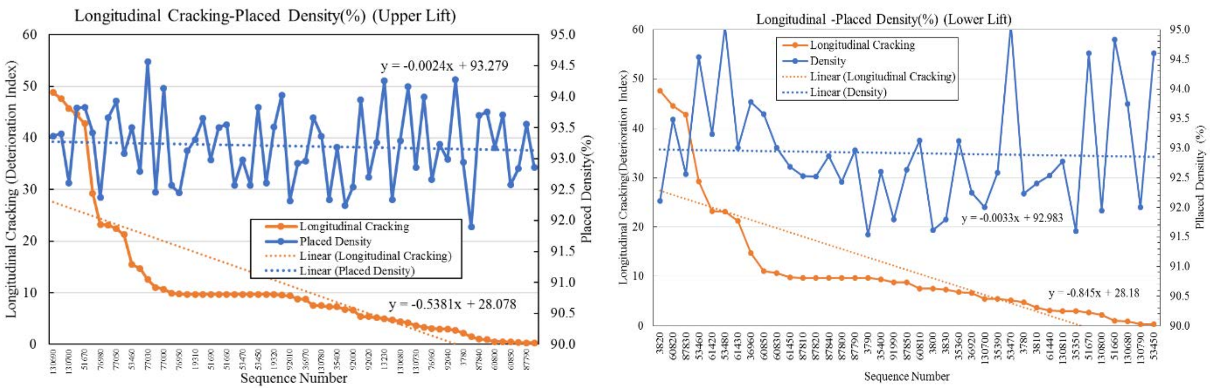


Figure 3-14 Longitudinal Cracking Against the Construction Quality Indicator

Observations for Longitudinal Cracking:

1- There is not an apparent association between the quality indicators and longitudinal cracking.

2- Lack of correlation could be due to the presence of a significant amount of joint longitudinal cracking included in this category. Therefore, it is difficult to separate the effect of the quality indicators, and construction practice on the presence of this cracking.

3- It will be helpful if separate fields are created in the PIF database to separate longitudinal construction joint cracks from in-lane performance cracks.

4- Longitudinal cracking DI is maxed at 50/100. It is recorded in 62 of the 101 SNs of the same age included in this project. Compared to the other distress and DI definition, this is a serious rate for pavements of this age.

Quality Indicators' Correlations with Transverse Cracking

Based on the distribution of Transverse Cracking at the global level, the graphs were prepared based on non-zero values of DI for Transverse Cracking.

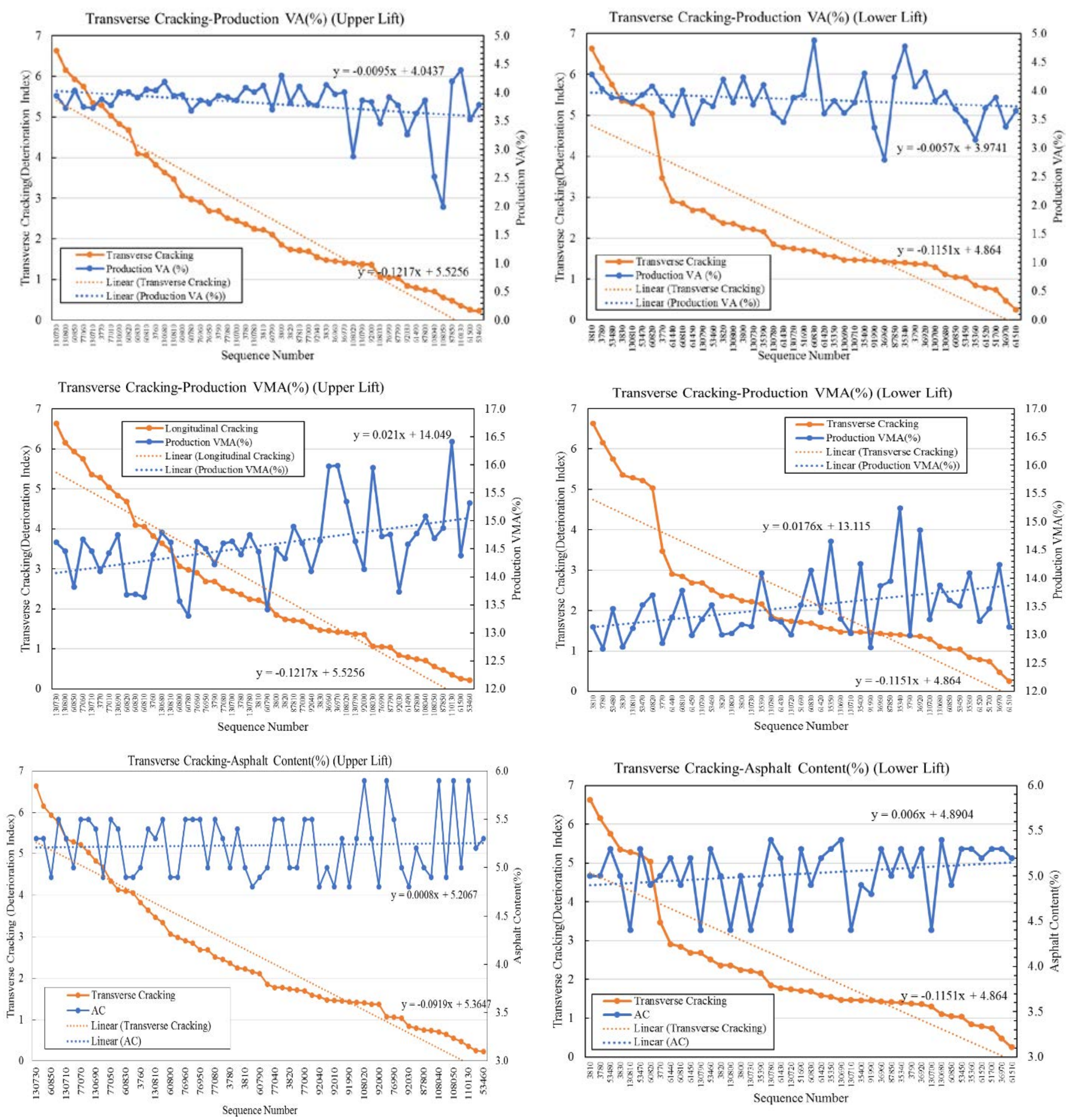


Figure 3-15 Transverse Cracking Against the Production Quality Indicators

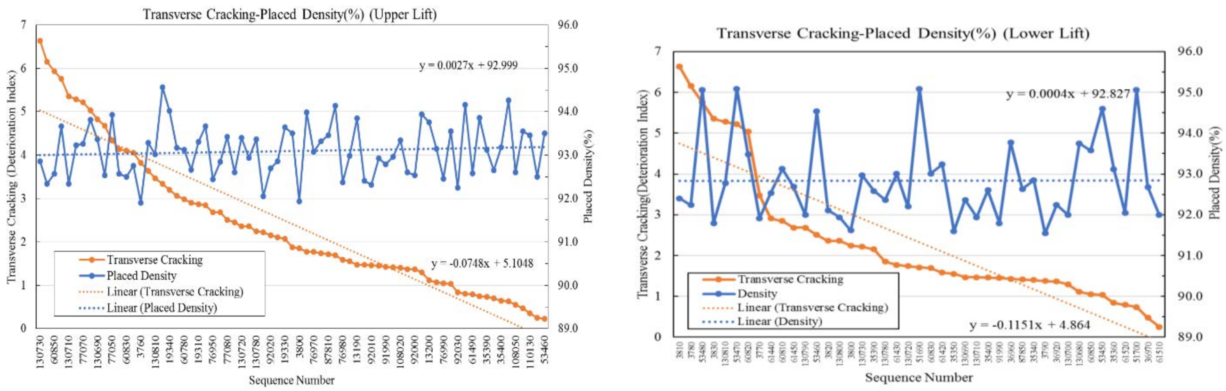


Figure 3-16 Transverse Cracking Against the Construction Quality Indicators

Observations for Transverse Cracking:

1- Potential correlation between VMA of 12.5mm & 19mm lift mixes and transverse cracking. The higher the VMA, the lower the DI of the transverse cracking.
2- No apparent association with any other quality indicator.
3- While transverse cracking is recorded for many SNs (71/101), its impact on the pavement is only maxed at a value less than 7/100. This indicates that most of the transverse cracks recorded are at low severity levels.
4- It is significantly important to note that the DOT distress recording method always assume a sealed crack as a low severity crack which might be misleading. The information shown in Figure 3-15 and Figure 3-16 indicate that transverse cracking is not a serious issue in Wisconsin while the reality could be different. There is no way to revise these numbers for the true level of severity of the transverse cracks. The on-site distress survey conducted by the authors show a significant discrepancy in attributing the level of severity of the distresses between the DOT records and the case on the ground.

3.3 Statistical Quantification of the Impact of Quality Indicators on DI of Reported Distresses

In this section, the observations made from sections 1 and 2 will be used to closely examine the association of the different distresses with the quality indicators. The following plots are not intended to evaluate the strength of correlation between distresses and the quality indicators. They are to serve in examining the association between the distress and the quality indicators identified in the previous sections. The examination focuses on evaluating if the trends are logical, the specification limits are reasonable and assign the independent factor for developing regression models. The statistical analysis conducted in this section is based on data collected on the fourth conducted distress survey by the DOT, which represents the long-term performance at the comparable ages for all pavements. This means the pavement ages for all the SN included is about eight years. The SNs with zero DI values for investigated distress were omitted from this analysis.

Rutting

DI values for Rutting at 12.5mm pavement lift is found to possibly correlate with multiple factors related to mix production quality and placement quality indicators. The following series of plots demonstrate these correlations leading to a regression model combining these factors into one equation.

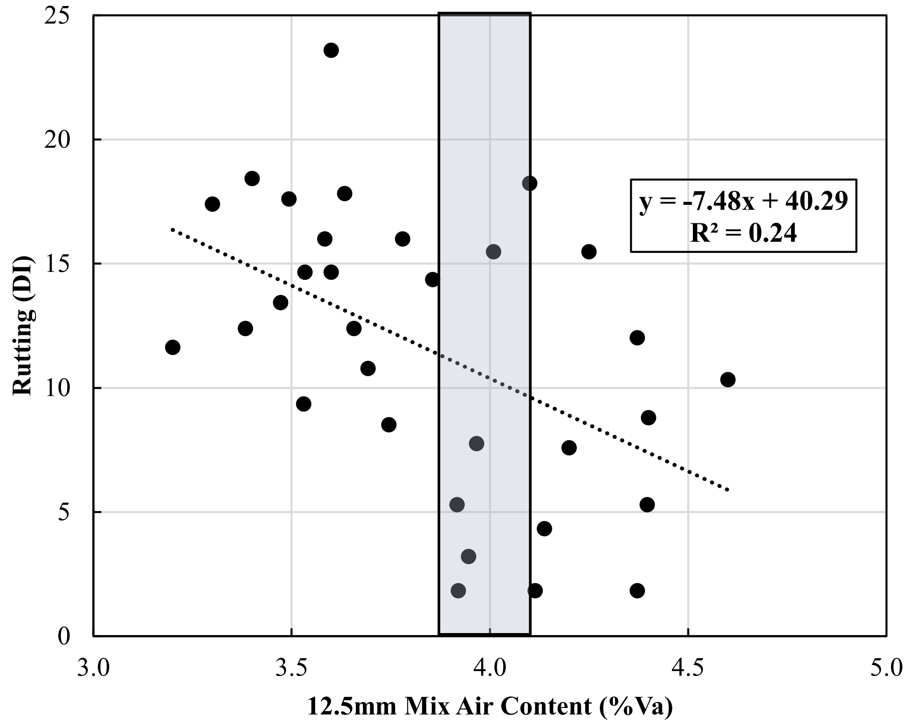


Figure 3-17 Rutting Against the Va(%) of the 12.5mm Mixes

Figure 3-17 shows a logical relationship between the mix production Va and the rutting DI. The trend shows that deviation in production leading to reduction in Va is correlated with increased rutting damage. This means that when mixes are designed to achieve 4% Va at N_{des}, changes in production leading air content to reduce further under the same level of compaction energy put this mix at risk. This observation is not about scrutinizing the Va limit of 4% but rather the effect of deviating from this limit for a given design during production. As to the cause of this shift, the data collected during this study did not include the asphalt cement content during production. This makes understanding why Va changed during production difficult.

The highlighted band in Figure 3-17 encompasses mixes of Va of 4±0.1%. It can be observed that this band contains the highest variability of DI values. This is understandable given that the mix is designed targeting this value. Thus it influences the statistical independence of Va. The range of 4±0.1% is selected to be as narrow as possible. It is not meant to separate the data into passing vs. not passing groups. Its purpose is to isolate the highly diverse group of data due to the realities of mix design procedures. On the other hand, evaluating mixes deviating from the target Va by less than 0.1% their DI is clustered closely at higher Rutting DI values. These observations emphasize the effects of production variation on Va is critical for rutting resistance.

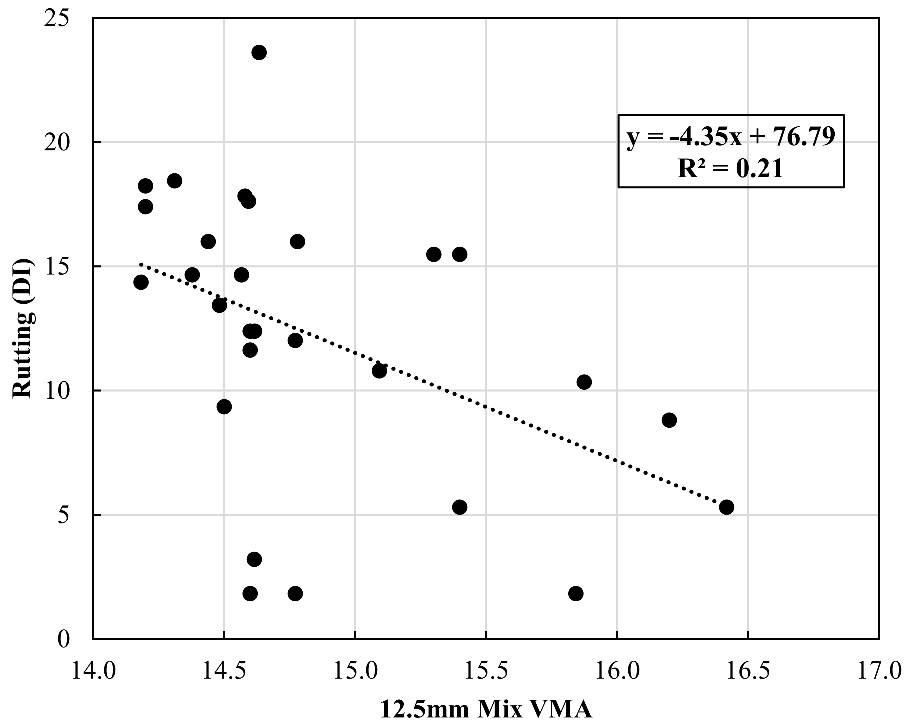


Figure 3-18 Rutting Against the VMA(%) of the 12.5mm Mixes

The aggregate structures as indicated by the VMA also shows a trend of influence on the rutting DI. While most of the mixes meet the required VMA, yet its higher level of values correlate with a reduction in rutting damage. (Figure 3-18)

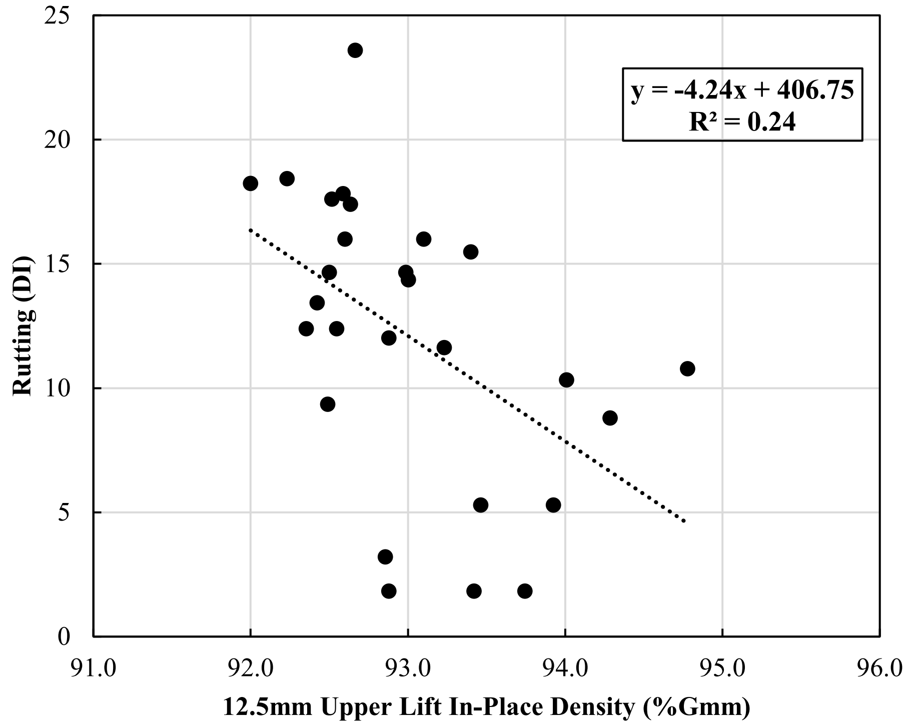


Figure 3-19 Rutting Against the In-Place Density of the 12.5mm Mixes

With respect to construction, Figure 3-19 shows a consistent trend in reduction of the rutting damage when the pavement is compacted well above the minimum required density. The trend shown in Figure 3-19 shows that increasing in-place density from 92% to 94% correlates with a 50% reduction in rutting deterioration.

A regression model is prepared to predict rutting DI based on the quality indicators measured during production and construction. The regression equation is shown below.

Rutting DI = 280.46 - (4.25) (Va) – (0.71) (VMA) – (2.60) (%Gmm) ***Equation 3-1***

The model maintains the trends observed earlier. Figure 3-20 shows the actual rutting DI plotted against the predicted and the 95% confidence interval. The figure shows that the model is able to predict all the data points with 95% confidence.

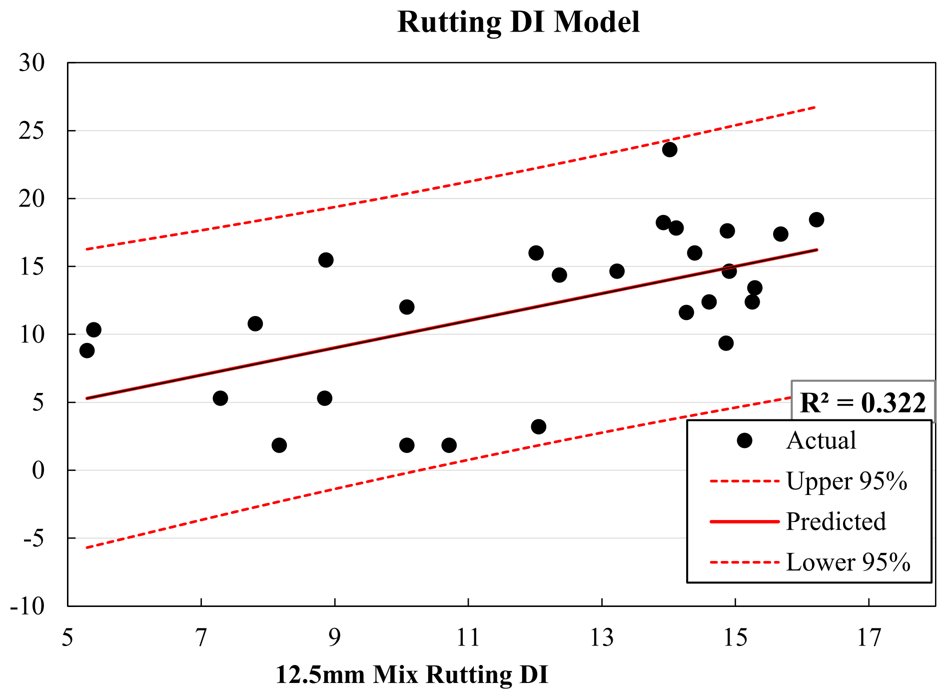


Figure 3-20 DI Model of Rutting for 12.5 mm Mixes

With respect to the 19mm pavement lifts, the rutting factor shows some correlation with the mix Va. Figure 3-21 shows that the trend is similar to that noticed for the 12.5mm mixes, whereas the Va increases, the rutting damage is decreased.

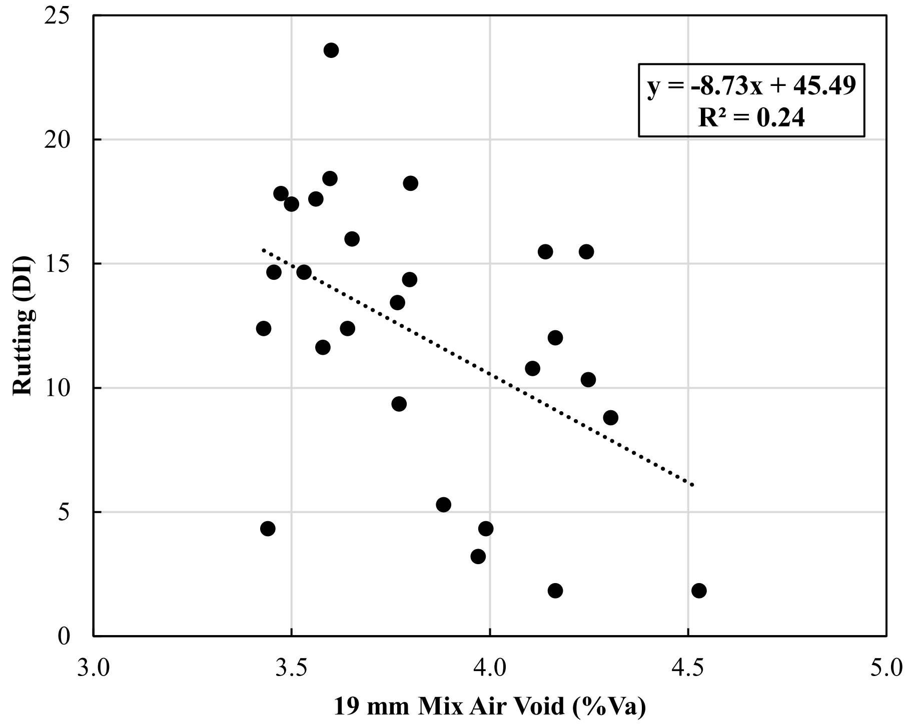


Figure 3-21 Rutting Against the Va(%) of the 19 mm Mixes

Again, the general trend follows the basic understanding of mix compaction. While the statistical correlation is not strong ($R^2 = 24\%$), the true value of the relationship is that it follows the logic. It is important to note that each data point represents the average value for one SN. In addition, the DI is calculated based on surveying only 10% of the length of the SN. These factors are expected to influence the strength of the correlation along with interaction with other hidden factors not shown in the data. More importantly, the clustering of the SNs with higher rutting damage are consistently made with mixes of lower Va. Combining this observation with that made for Figure 3-17 increases the confidence in the association between Va and in-service rutting.

Alligator Cracking

For alligator cracking for the 12.5mm mix also show a logical trend against the production and construction quality indicators. The following figures (Figure 3-22, Figure 3-23 and Figure 3-24) show that alligator cracking damage is reduced at higher Va, higher VMA, and higher in-place density.

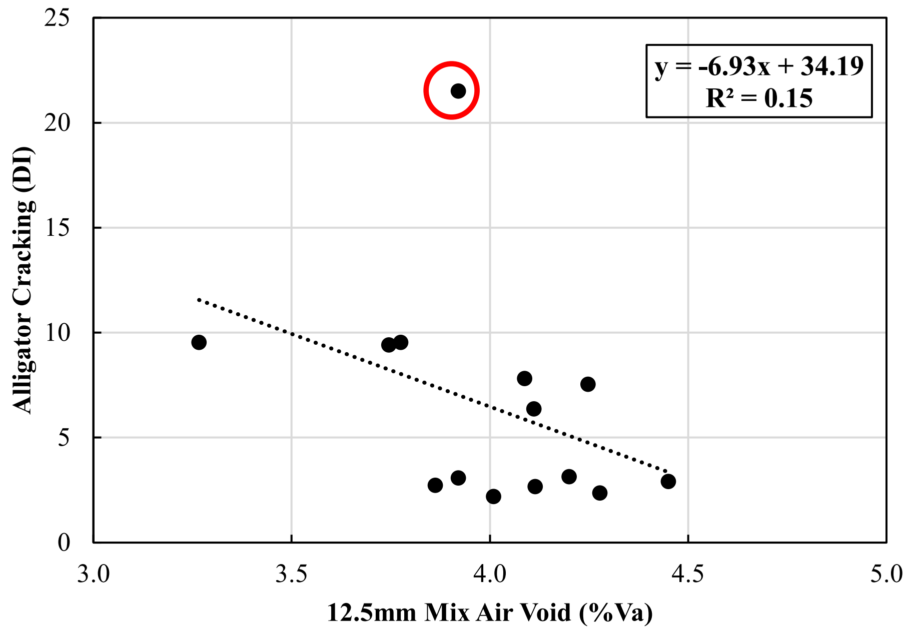


Figure 3-22 Alligator Cracking Against the Va(%) of 12.5mm mixes

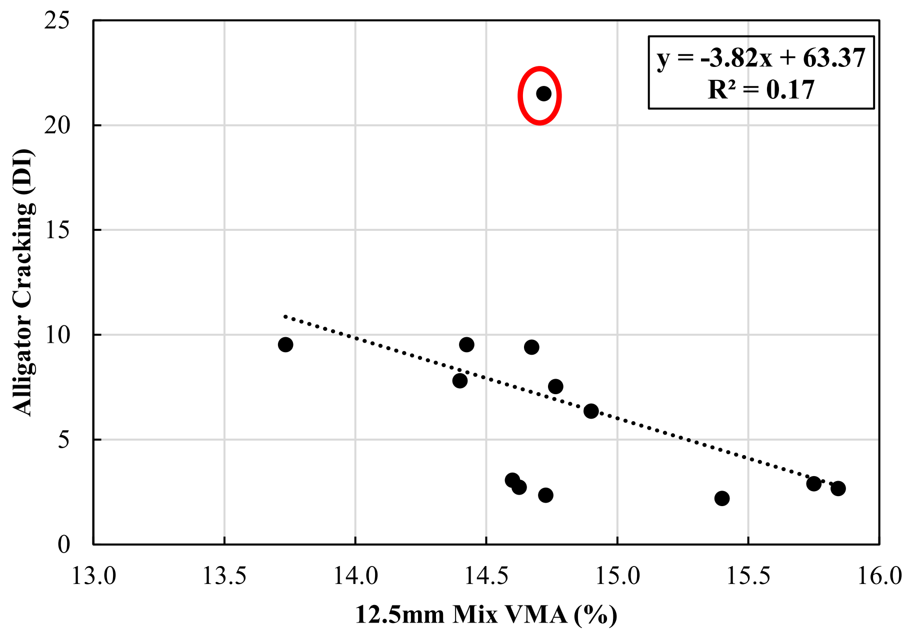


Figure 3-23 Alligator Cracking Against the VMA (%) of 12.5mm Mixes

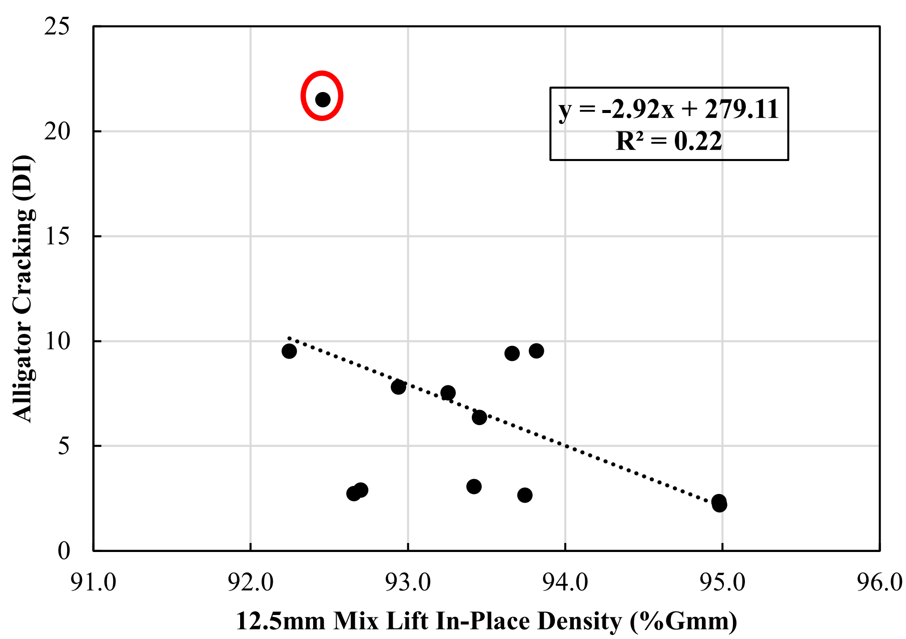


Figure 3-24 Alligator Cracking Against the In-Place Density of 12.5mm Mixes

The previous figures (Figure 3-22, Figure 3-23 and Figure 3-24) contains one SN showing higher alligator cracking damage (more than double that of the following data point). This circled SN is deviating the statistical analysis due to its value. This SN belongs to STH 55. This project has alligator and longitudinal cracking as the dominant type of distress. In addition, the FWD tests indicated a soft pavement structure. Due to these facts, the regression model is prepared without including this project. The regression model is shown below:

12.5mm Alligator DI = 101.878 – (1.509) (Va) – (2.629) (VMA) – (0.550) (%Gmm) ***Equation 3-2***

The comparison between the actual and predicted fatigue damage is shown in Figure 3-25. The comparison includes 95% confidence interval of the regression model.

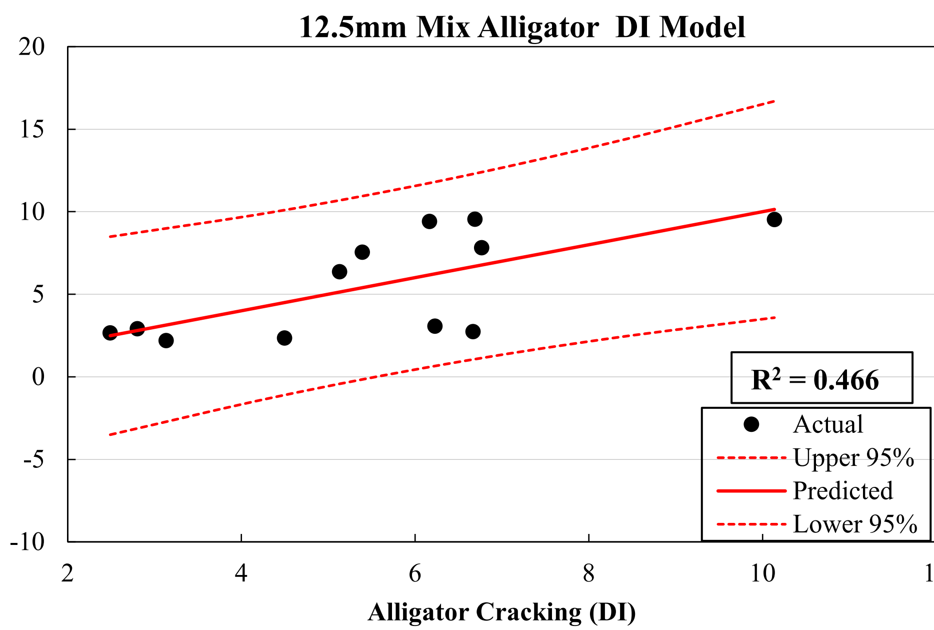


Figure 3-25 Alligator Cracking DI Model for 12.5mm Mixes

For the 19 mm mix, similar trends are observed for the production and construction quality indicators. The following set of figures (Figure 3-26, Figure 3-27, and Figure 3-28) establish the correlation between the quality indicators and the alligator DI. While the correlation coefficients are not strong, the trends follow expectations.

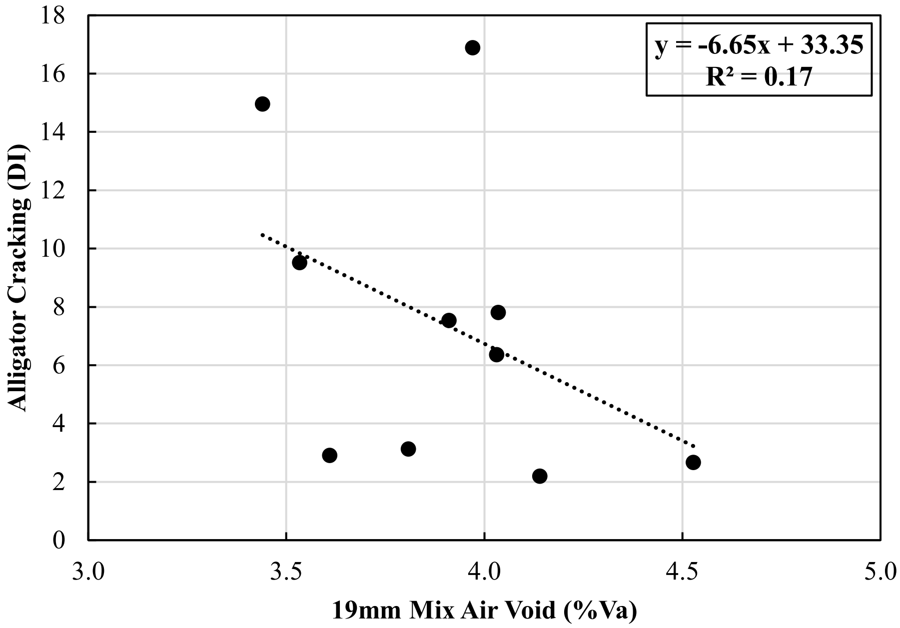


Figure 3-26 Alligator Cracking Against the Va(%) for 19mm Mixes

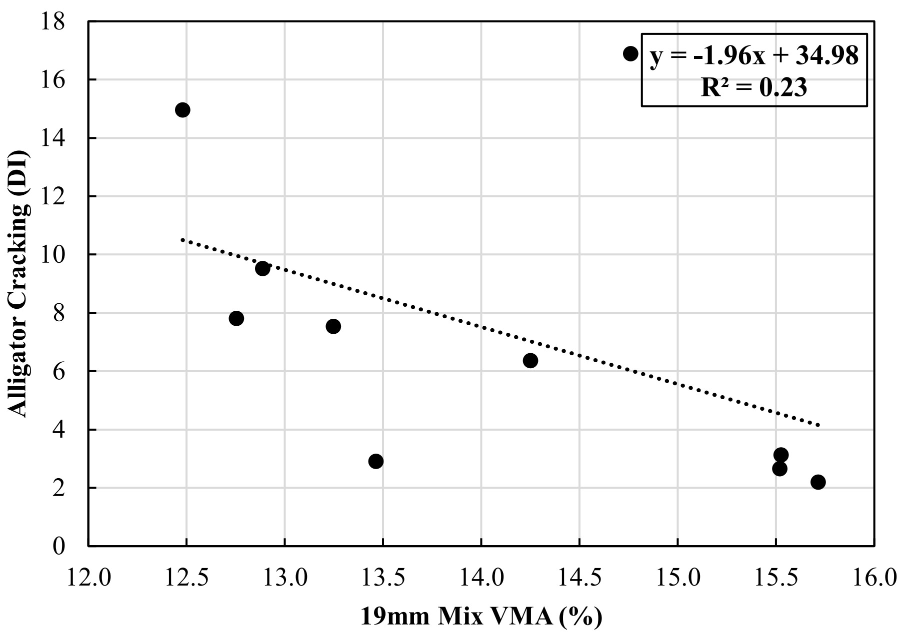


Figure 3-27 Alligator Cracking Against VMA(%) for 19 mm Mixes

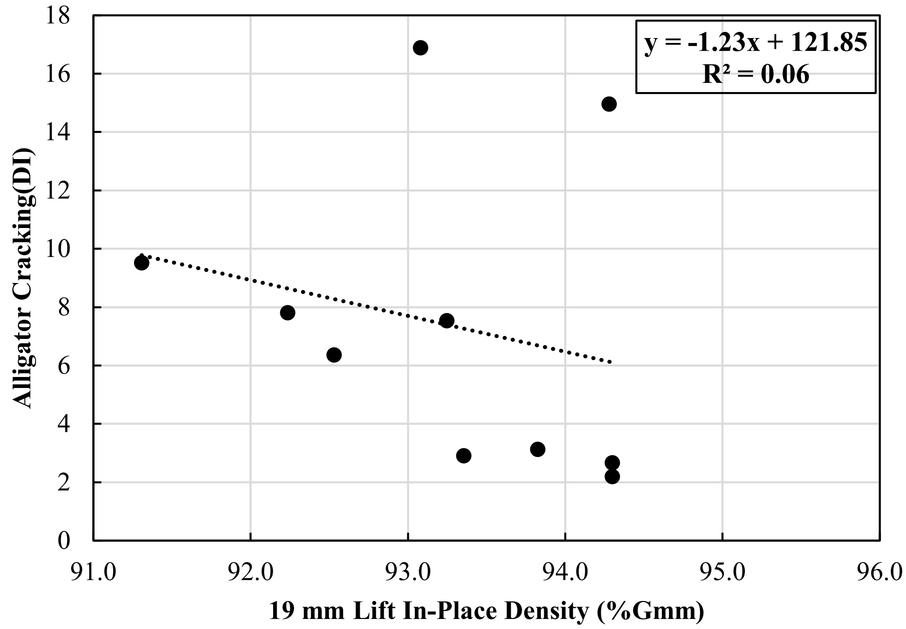


Figure 3-28 Alligator Cracking Against In-Place Density for 19 mm Mixes

The regression model prepared using the quality indicators, provide adequate prediction within 95% confidence interval. Also, the coefficients of the model follow the expected trends. This model could encompass the data points with 95% confidence as shown in Figure 3-29.

19 mm Alligator DI = 40.828 – (2.848) (Va) – (1.471) (VMA) – (0.018) (%Gmm) **Equation 3-3**

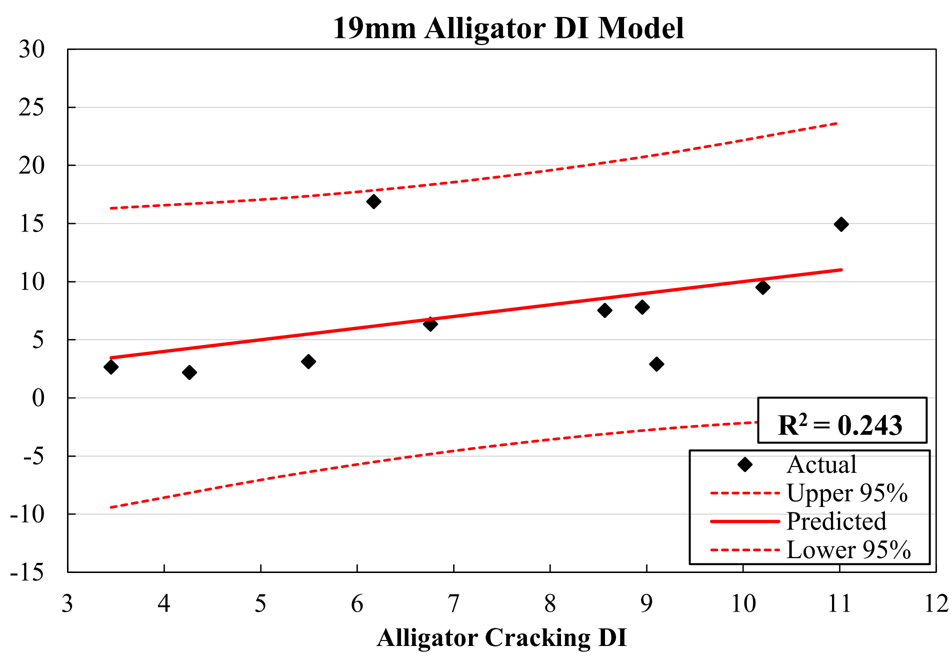
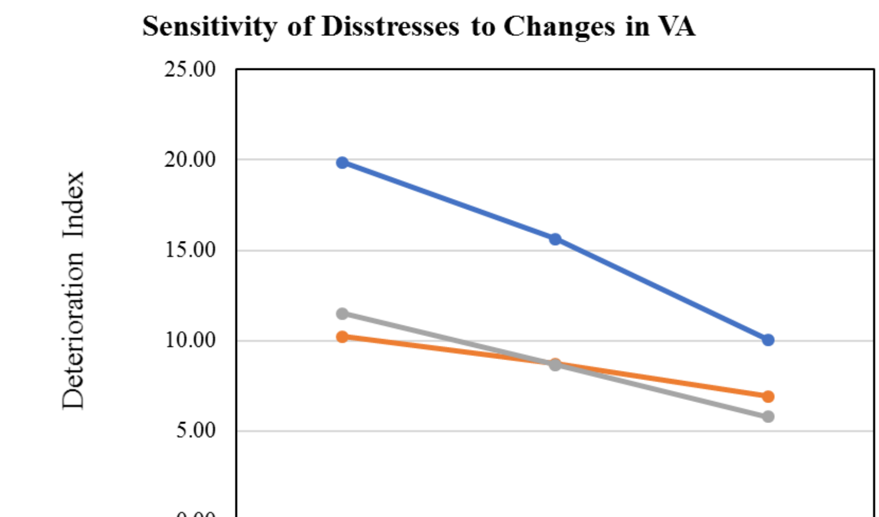


Figure 3-29 Alligator Cracking DI Model for 19 mm Mixes

3.4 Global Analysis Summary and Sensitivity Analysis

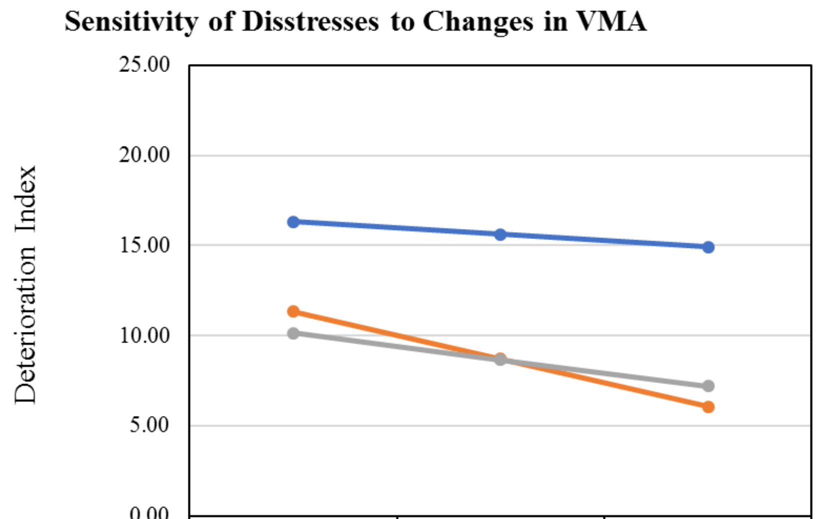
It is important to note that the purpose of the regression models included in section 3.3 do not serve for prediction of pavement distresses as a function of quality indicators. The basic use of regression is to analyze the data at hand not to predict future patterns. These models are prepared to establish the effect of deviating from the specification limits on the recorded performance.

Using the regression models, the influence of the quality indicators can be evaluated through a simple sensitivity analysis. The starting point should be what is the status of the distresses after 7-8 years of service if all the quality indicators are met? The sensitivity analysis examines the change in the distresses due to the ± 1% deviation in Va and VMA and -1% and +2% changes in in-place density while the remaining ones stay constant. The following plots (Figure 3-30, Figure 3-31 and Figure 3-32) show these changes.



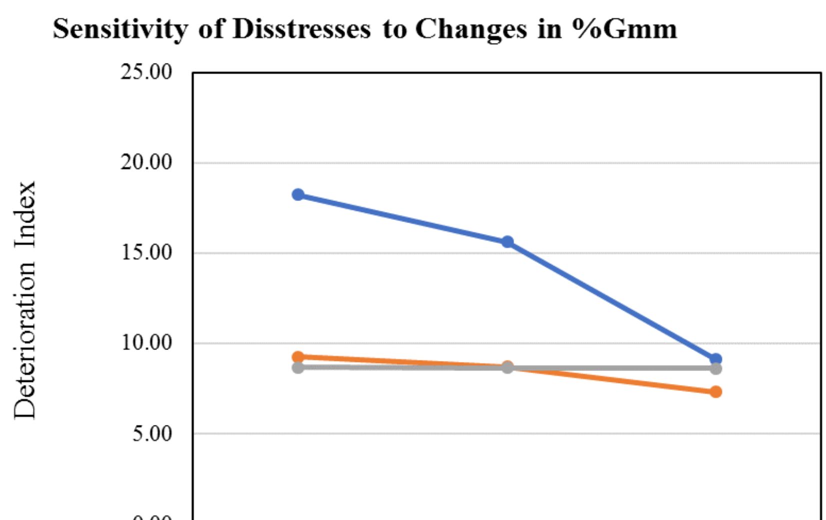
	-1.0	0.0	1.0
Rutting	19.87	15.62	10.07
12.5 mm Alligator	10.22	8.71	6.93
19 mm Alligator	11.51	8.67	5.81

Figure 3-30 Change in the Deterioration Indices due to Unit Change in %Va



	-1.0	0.0	1.0
Rutting	16.3	15.62	14.91
12.5 mm Alligator	11.3	8.71	6.08
19 mm Lift Alligator DI	10.1	8.67	7.20

Figure 3-31 Change in the Deterioration Indices due to Unit Change in %VMA



	-1.0	0.0	2.0
Rutting	18.22	15.62	9.12
12.5 mm Alligator	9.26	8.71	7.34
19 mm Alligator	8.68	8.67	8.62

Figure 3-32 Change in the Deterioration Indices due to Unit Change in %Gmm

Figure 3-32, Figure 3-31, and Figure 3-30 reveal that the most reduction in overall deteriorations is achieved when the %Gmm is increased by 2%. The analysis of the %Gmm included adding 2

%Gmm to the acceptance value due to the fact that the current state of practice already clusters around this value. On the other hand, if the pavements are constructed at the acceptance values as set by the specifications, the expected distresses after 7-8 years of service will be the noticeable levels of rutting and alligator cracking. The PCI value due to these distresses alone (not accounting for others such as longitudinal and transverse cracking) will be less than 80. Therefore, it may be beneficial to revise the current specifications limits for quality indicators. It is recommended to narrow the range of the Va lower limit, increase the minimum VMA, and increase the minimum %Gmm. However, the %Gmm should also implement a maximum value for %Gmm to avoid over compaction of the pavement lifts during construction.

4. Project Level Analysis

In this chapter, the selected field sections are discussed individually. This will allow for utilization of the high-resolution data collected on site for the in-service performance. As discussed in chapter 2, the quality data is used to characterize each SN within a given project. The regression models developed in the previous chapter will be tested on the project level in an attempt to validate them.

The in-service performance data is obtained from two different sources. The first source is supplied by Wisconsin DOT. This is the part of the DOT continuous effort to track the performance of its network. The data from this source is extracted from the Pavement Information System Database (PIF). The PIF tracks the performance through sending crews to evaluate each SN every two years. The SN is evaluated through the distress evaluation of the third tenth of the mile from starting point of the SN.

The second source of the in-service performance is through site visits. The distress of every SN within the project is evaluated at the same segment the PIF was used. The field survey observations were processed based on the Wisconsin pavement distress survey procedures. This allows for comparing the results of this study against that in the PIF. In addition, FWD is conducted for all these SNs to isolate possible failure due to subsurface failure and to quantify the in-place mechanical properties of the test sections. The results were analyzed using ERI's in-house developed Pavement Analysis Software.

4.1 Comparison of Collected Projects to Database

In order to validate the selected projects for field testing, their performances are compared to the database discussed in chapter 3 and the WisDOT database (PIF). This comparison is to demonstrate that the performance of the selected projects are comparable to the WisDOT data population as well as the collected ones. Figure 4-1 shows the comparison between the average Rutting performances for the different years of the survey. The average values included in the Figure 4-1 reflect the average of all the data points (SNs) included in the sources labeled in the figure. The comparisons are based on the different categories of projects in the PIF; the entire stored projects in the PIF, the 30 initially selected projects, and the 7 selected projects for the detailed analysis.

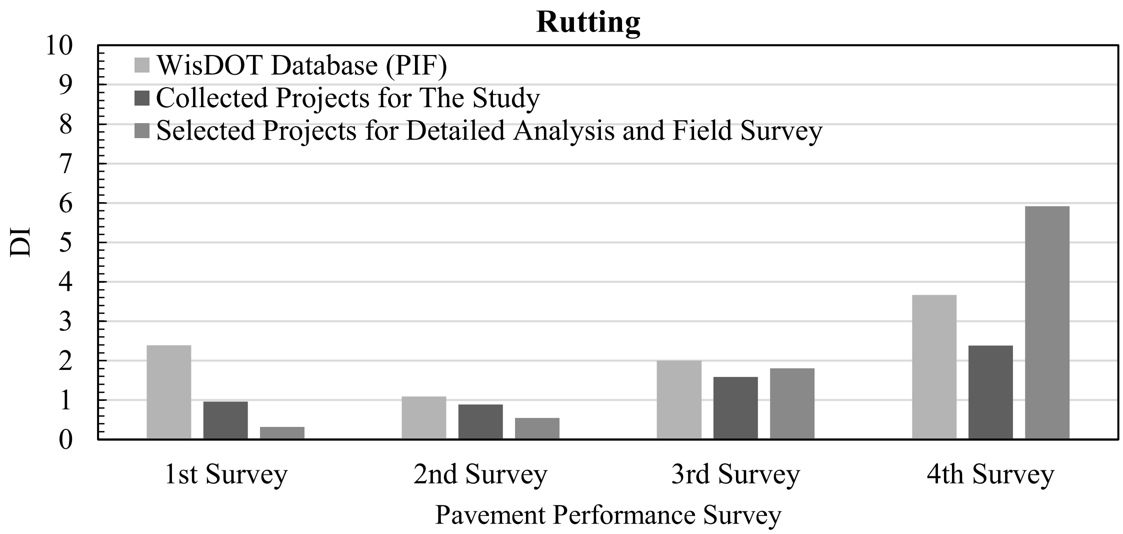


Figure 4-1 Comparison of Rutting DI for Selected Field Projects and the Created Database with PIF

On average, the rutting is not major distress in the projects included in the database. As seen in Figure 4-1, the average rutting for all projects scores below 10 out of a maximum of 100. The average rutting of the selected projects for field survey is slightly higher than the average of the collected database. This can help the analysis to look at more details and investigate the causes of this problem. However, considering that the score is out of 100, the averages are comparable. Figure 4-2 shows that most of the SN within the collected database (101 SNs) show no rutting on their fourth conducted survey. The distribution in Figure 4-2 shows that the first three quartiles of the SN experience no rutting.

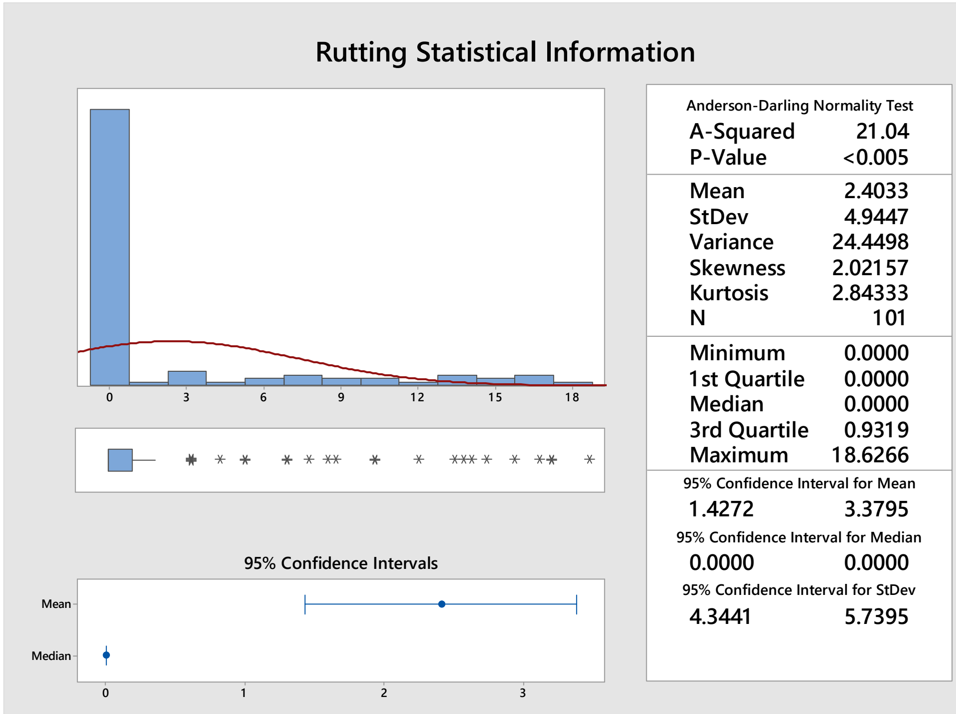


Figure 4-2 Distribution of Rutting DI for the Database

With respect to alligator cracking, Figure 4-3 shows that the 30 collected projects have a comparable level of alligator cracking to the network. Although, the seven selected projects for the field survey are showing a comparatively higher alligator cracking problem, which again, enables the more detailed research on its causes.

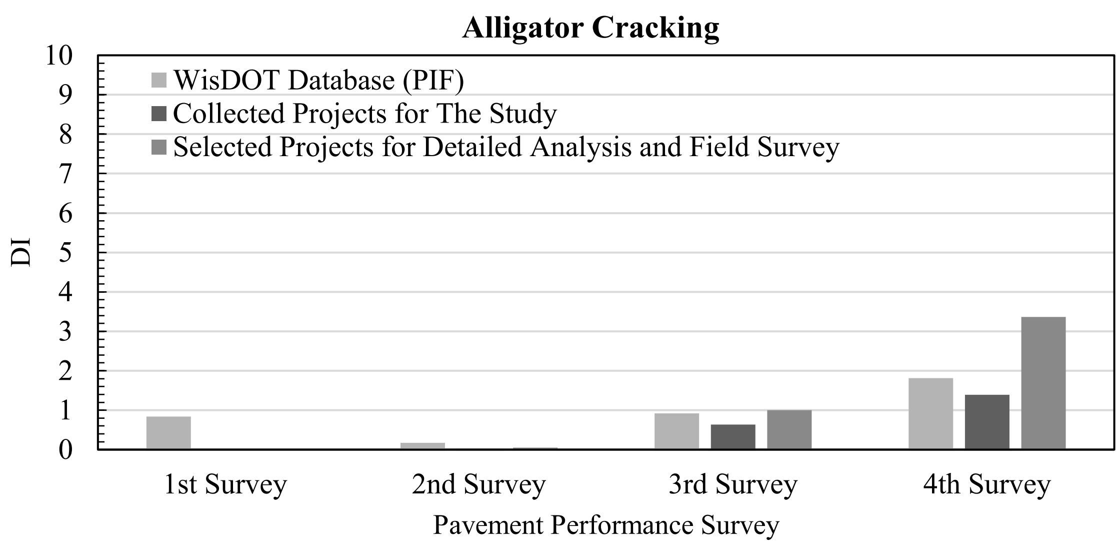


Figure 4-3 Comparison of Alligator Cracking Deterioration Index for Selected Field Projects and the Created Database with PIF

The distribution of the database of all the SNs based on the fourth conducted survey shows a similar distribution to that of the rutting. therefore, data used to generate the distribution reflect 7-8 years of service (Figure 4-4). The distribution shows that the 95% confidence interval for the alligator DI ranges from 0.623 to 2.181.

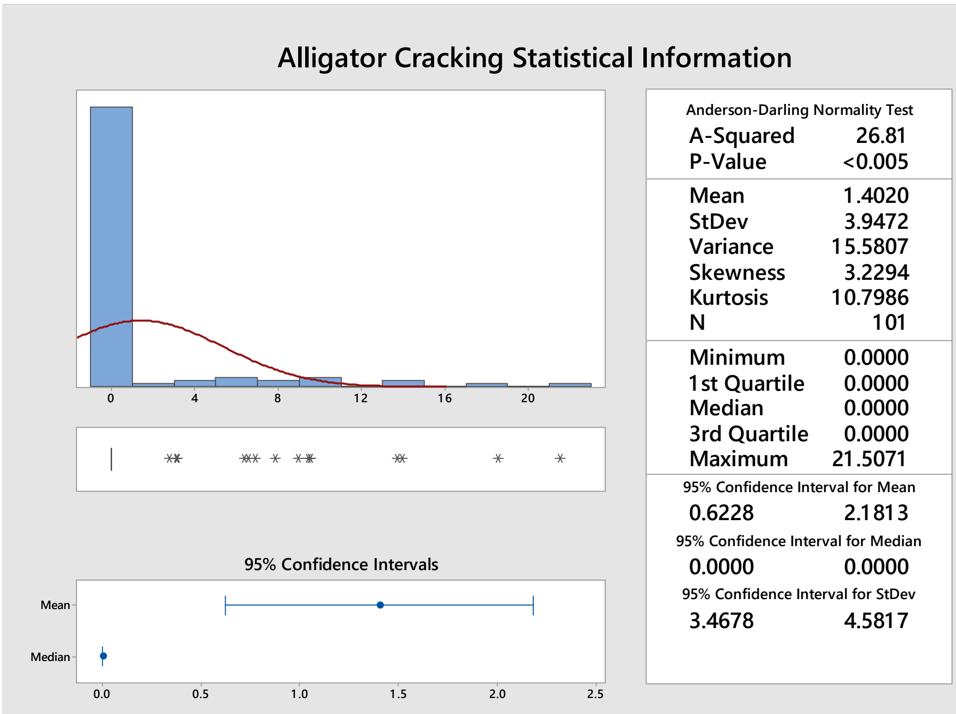


Figure 4-4 Distribution of Alligator Cracking DI for the Database

The Figure is showing that the majority of the sections are reporting a low level of alligator DI. Only few sections (within the fourth quartile of the distribution) are showing high level of DI at maximum of 21.5.

The selected SNs for the field survey are also comparable to the network with respect to the longitudinal cracking damage. This is shown in Figure 4-5. In fact, the selected projects are on average slightly higher than the database (according to the fourth survey).

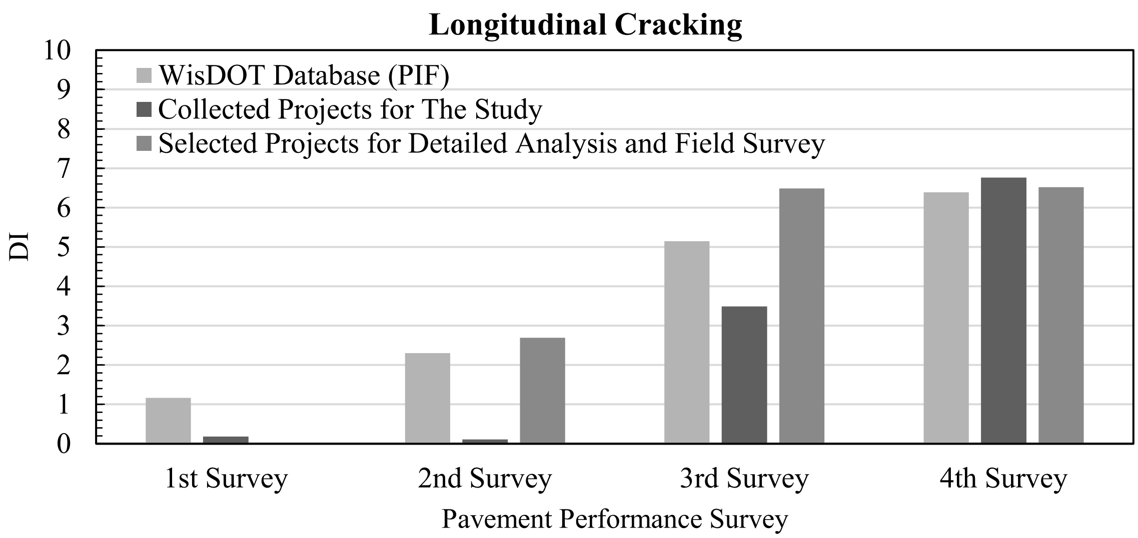


Figure 4-5 Comparison of Longitudinal Cracking DI for Selected Field Projects and the Created Database with PIF

The distribution of the longitudinal cracking is more prevalent over more SNs than the rutting and alligator cracking. In addition, the maximum value observed is much higher (48.8) than the previously discussed distresses of Alligator Cracking and Rutting (Figure 4-6). Analysis of the distribution show 95% confidence interval on the DI of longitudinal cracking ranges from 4.69 to 8.97.

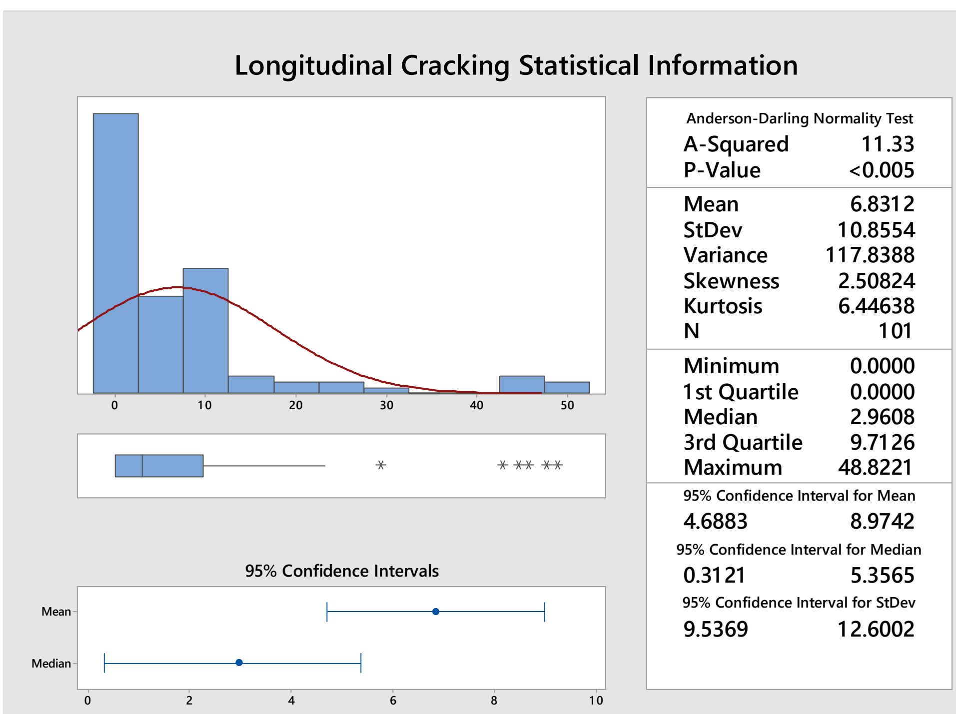


Figure 4-6 Distribution of Longitudinal Cracking DI for the Database

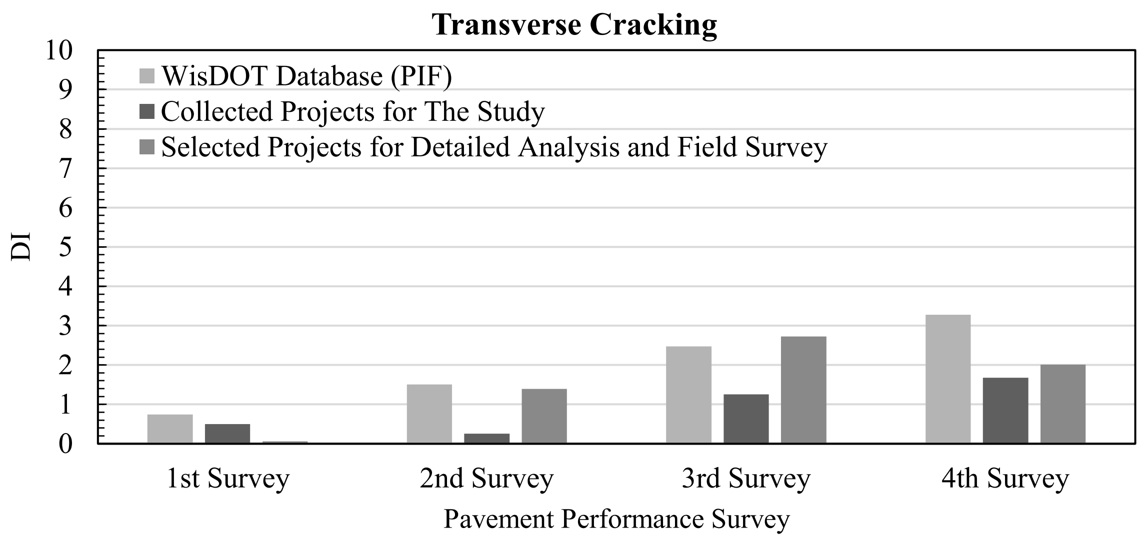


Figure 4-7 Comparison of Transverse Cracking DI for Selected Field Projects and the Created Database with PIF

Figure 4-7 shows the comparison of the selected projects for research and field evaluation against the WisDOT database in terms of transverse cracking. The selected projects are on average lower than the average of the database. However, due to the low level of damage index for this distress type, it is difficult to match the database.

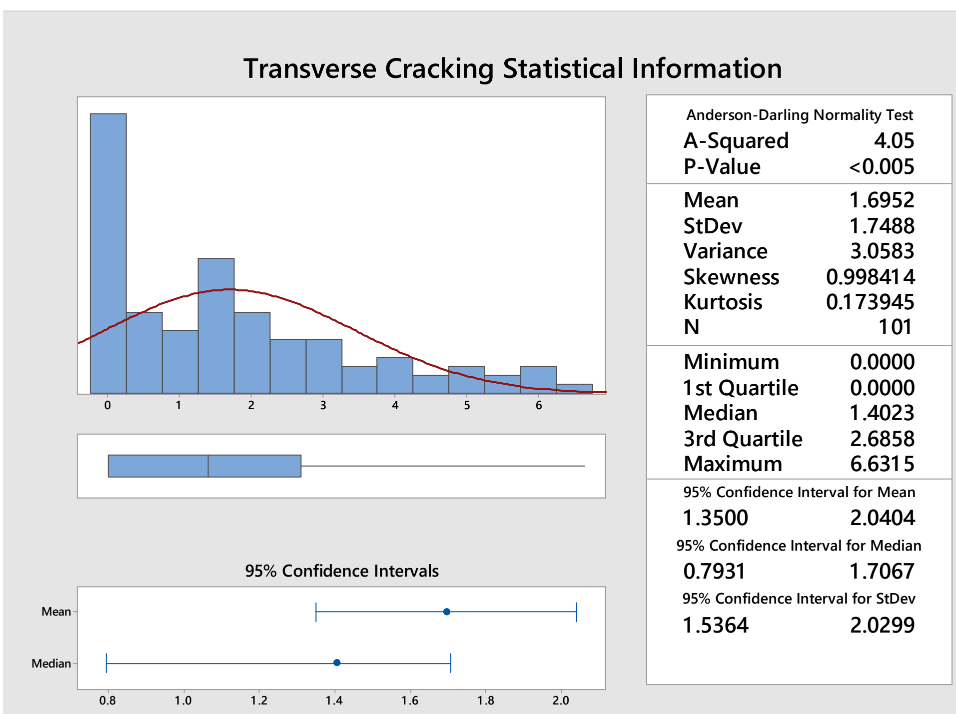


Figure 4-8 Distribution of Transverse Cracking DI for the Database

Transverse cracking is also prevalent in the database. However, the extent of the deterioration due to transverse cracking is minimal compared to the longitudinal cracking. The maximum value for the DI is only 6.6. (Figure 4-8)

4.2 Project USH 45 (#1600-21-70)

Construction year: 2008

Design Traffic: 2,306,800 ESALS

Asphalt Binder Content: 5.2%

Asphalt Layer Thickness: 6.25”

SN	61420	61430	61440	61450	61460	61470	61480	61490	61500	61510	61520	61530	61540	61550
PCI	78	84	90	91	89	91	85	81	85	86	100	86	77	81

USH 45 is a North-South United States Highway. The project is located Northeastern of the state of Wisconsin, in Shawano County. The total length of the project is 15.107 mile, and it covers 14 Sequence Numbers (SNs). The location of the project is shown in Figure 4-9. The project was constructed using 114,140 tons of HMA. This highway contains two lifts of 12.5 mm and 19 mm NMAS for upper and lower lift respectively.

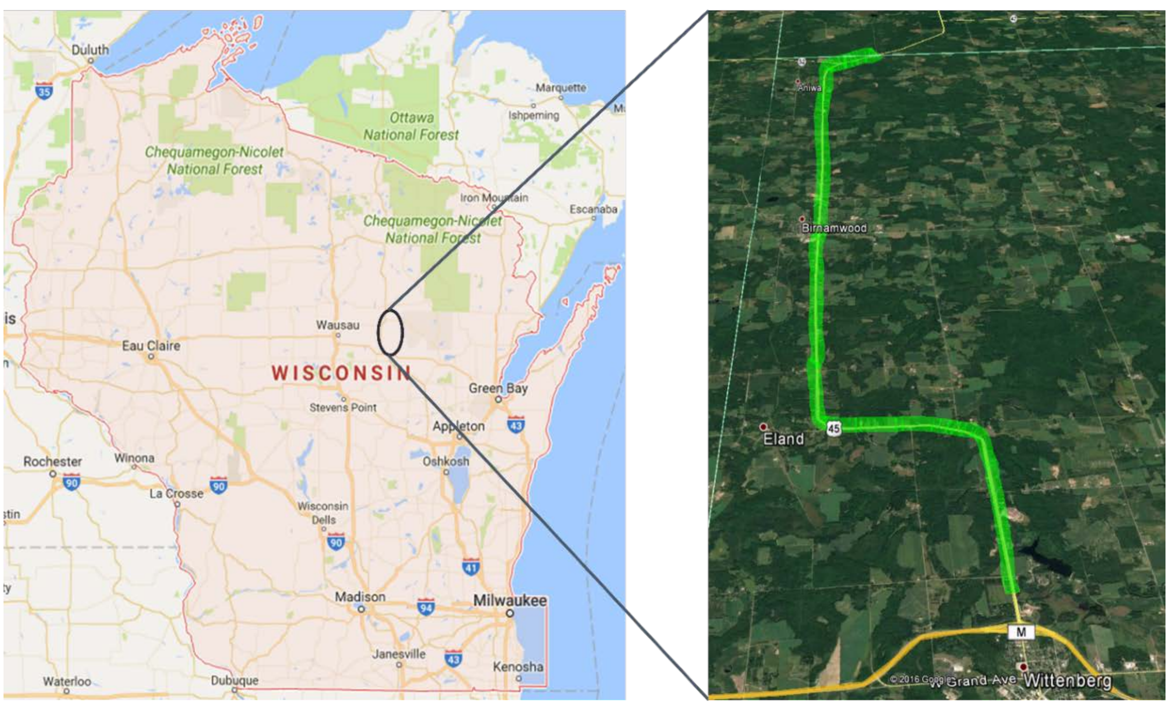


Figure 4-9 Highway USH 45, project 1600-21-70, Wittenberg-NCL, Shawano County

In-Service Performance

The last performance survey by DOT was conducted on 2016 and is illustrated in Figure 4-10. The deterioration indices for the recorded distresses are stacked for all the SNs encompassed within this project. As can be noted, rutting is the predominant distress throughout this project. Only one SN does not show rutting. Transverse cracking is consistently distributed throughout the project at minimal levels of impact; this could be weather-related.

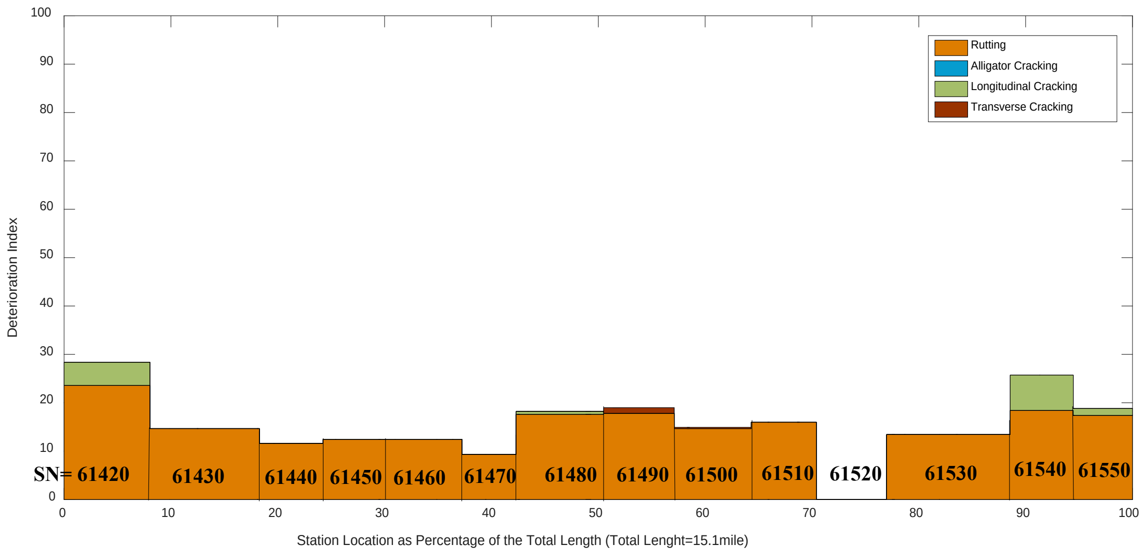


Figure 4-10 Performance of the project (based on the DOT information) (2016)

Data shown in Figure 4-10 represent performance after eight years of service for this highway. For an under 3 million ESALs pavement, alligator cracking is not expected at this age, which is consistent with this project performance. On the other hand, four SNs are showing the noticeable level of longitudinal cracking. Given the way the PIF records longitudinal cracking, it does not separate longitudinal due construction joints from performance-based longitudinal cracking. The authors confirmed that most of these cracks are due to construction joints. Figure 4-11 shows the results of the on-site distress survey. The results contradict with the reported performance on the PIF database.

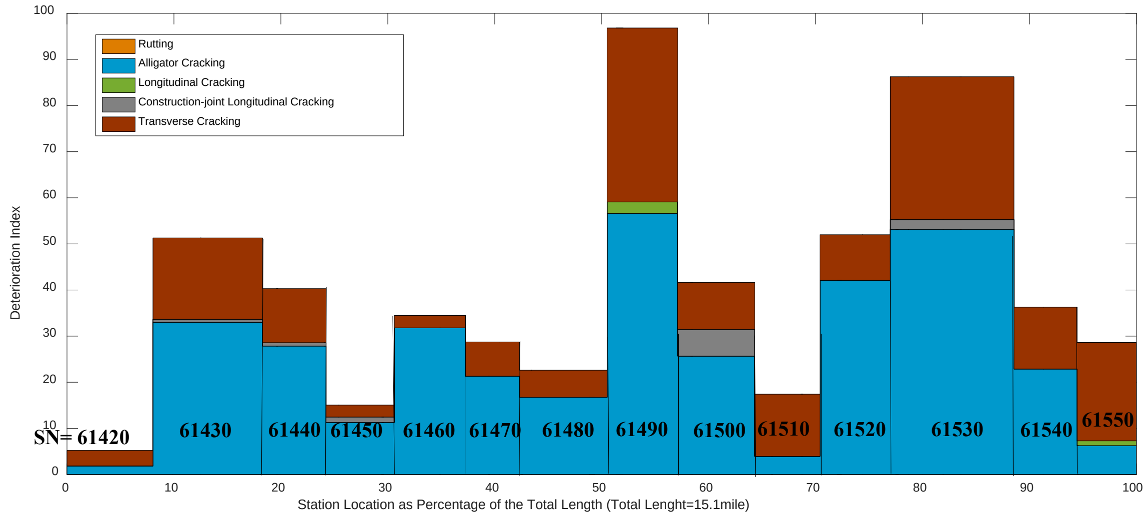


Figure 4-11 Performance of the project USH 45 (based on the on-site survey, 2017)

Figure 4-11 does not include rutting because it was not examined in the field due to the traffic control issues. In addition, alligator cracking is the most dominating followed by transverse cracking. No falling weight deflectometer (FWD) testing was conducted on this project, again due to the traffic-related issues. Therefore, it is impossible to verify if the cracking is because of the structural issues or related to the materials quality indicators.

Production Quality

Production quality data were obtained from the contractors. The production lots were converted to mat area based on the design thickness in the plans. As a result, the production quality data is associated with the appropriate SN. Figure 4-12 and Figure 4-13 show the average air content of the gyratory samples tested at the contractor's lab. The figures are for the mixes used for the upper and lower lifts. With the target being 4% air, both mixes are showing total averages of much lower values.

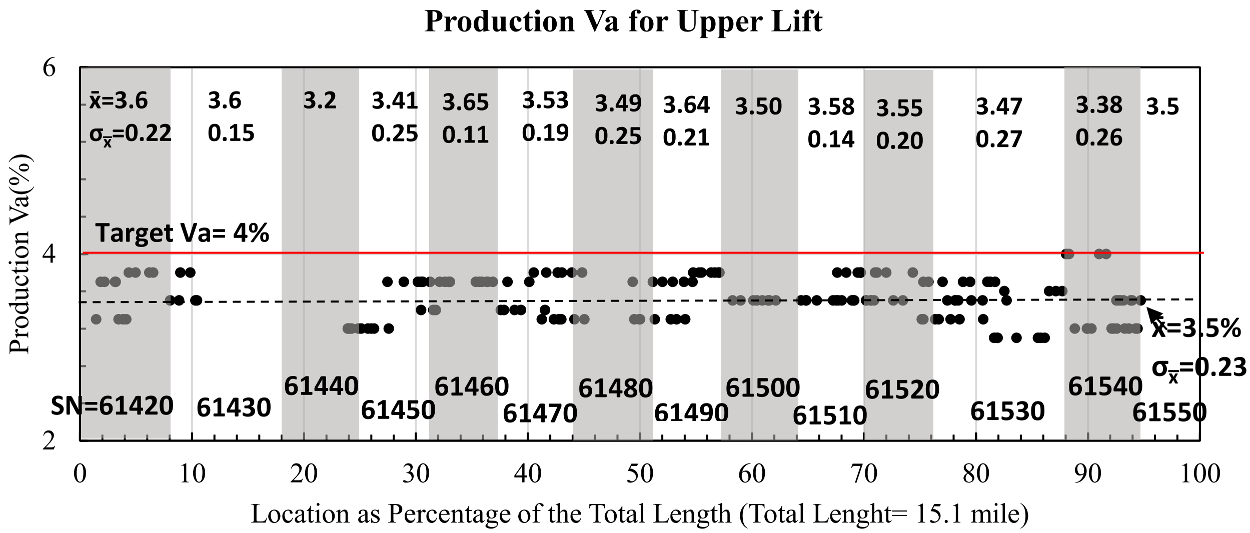


Figure 4-12 Production information air voids (Va) based on station locations (Project USH 45) (Upper Lift)

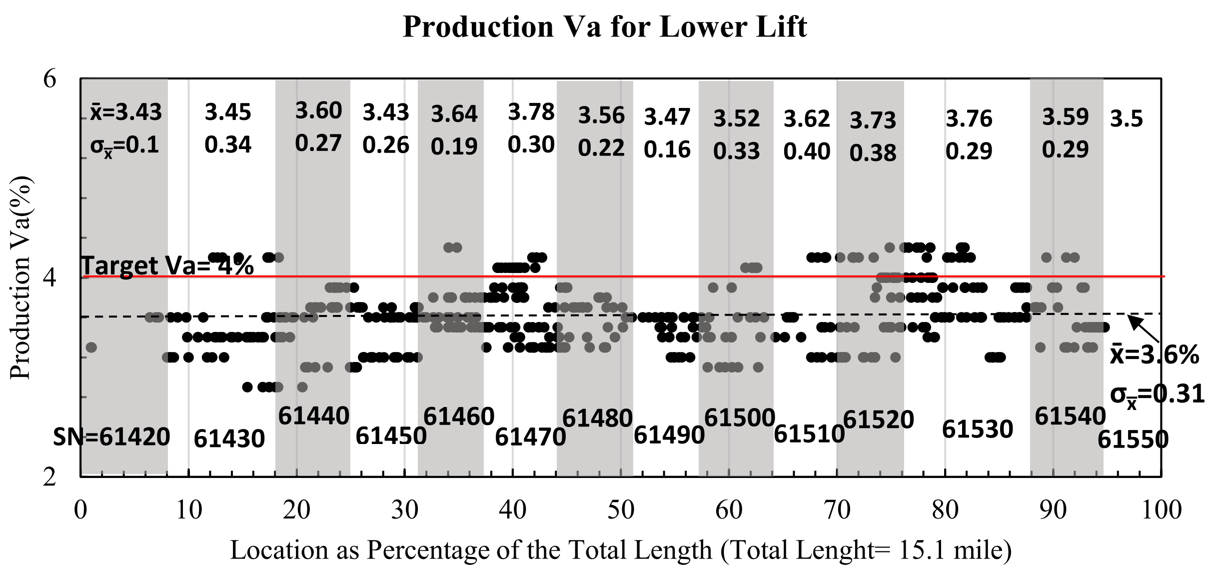


Figure 4-13 Production information air voids (Va) based on station locations (Project USH 45) (Lower Lift)

Based on the statistical analysis conducted in chapter 3, Va for the 12.5 mm and 19 mm mixes are consistently lower values (less than 4). This level of Va is associated with higher level of alligator cracking. Figure 4-12 and Figure 4-13 clearly show that the majority of the tests for both types of mixes are lower than 3.75%. In fact, only a few of the tests met the 4% Va target.

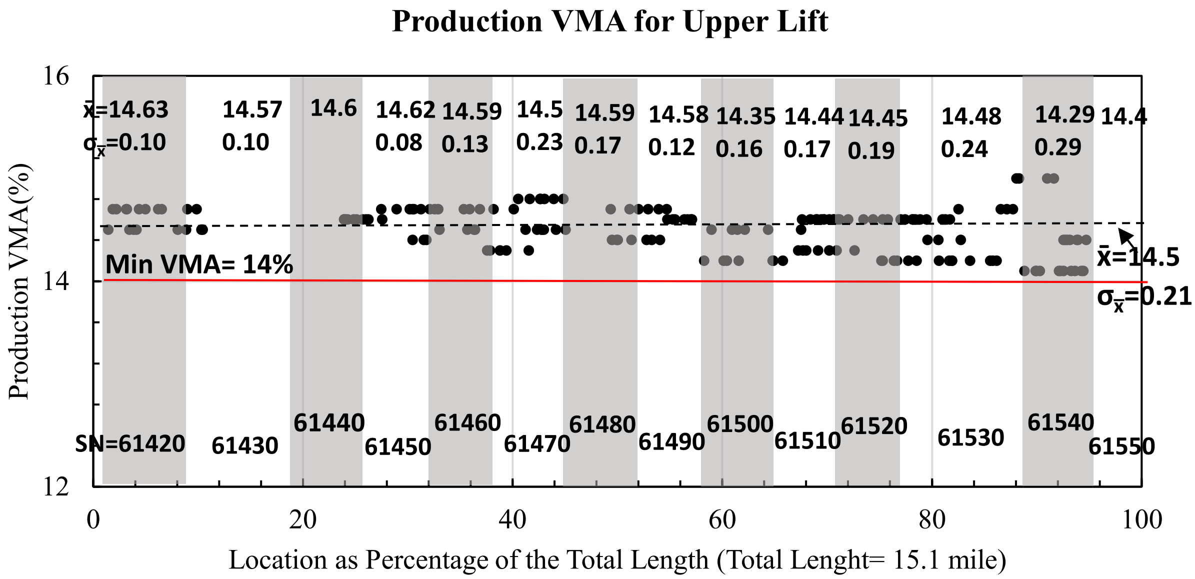


Figure 4-14 Production information of voids in mineral aggregates (VMA) based on station locations (Project USH 45) (Upper Lift)

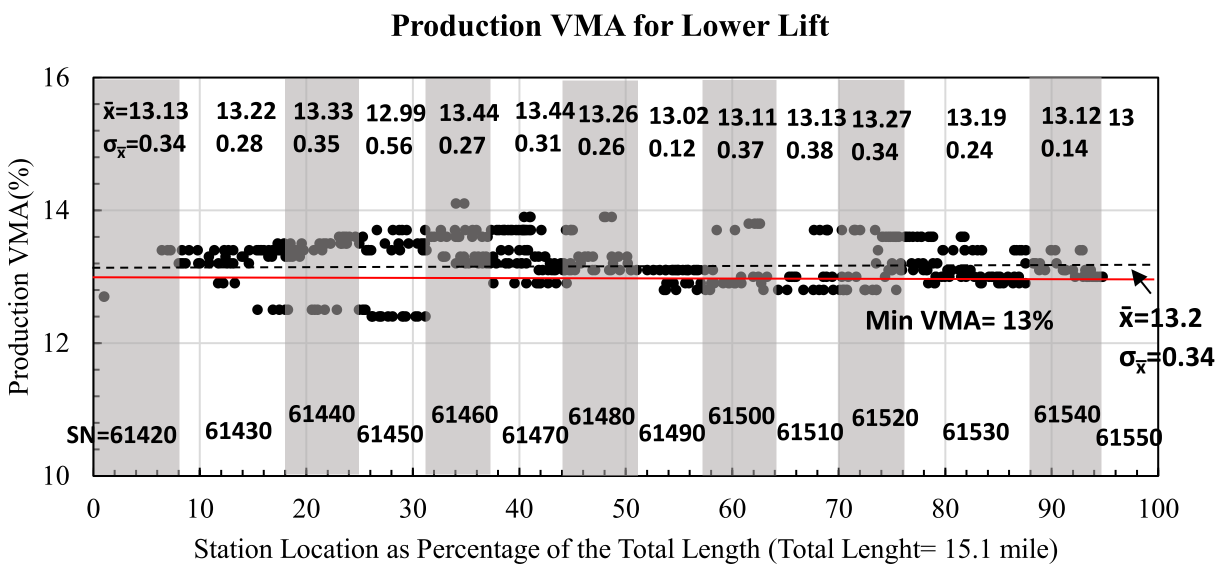


Figure 4-15 Production information of voids in mineral aggregates (VMA) based on station locations (Project USH 45) (Lower Lift)

The values of the VMA are also aligned with the trends noted in the global analysis. This project production data is highly consistent with minimal variability. This consistency did not prevent the pavement from showing a high level of cracking at early ages. (Figure 4-14 and Figure 4-15)

Placement Quality

Figure 4-16 and Figure 4-17 show the placement density for both lifts. The top lift is averaging 1.0% above the required density, while the lower lift is averaging 3.1% above the required density.

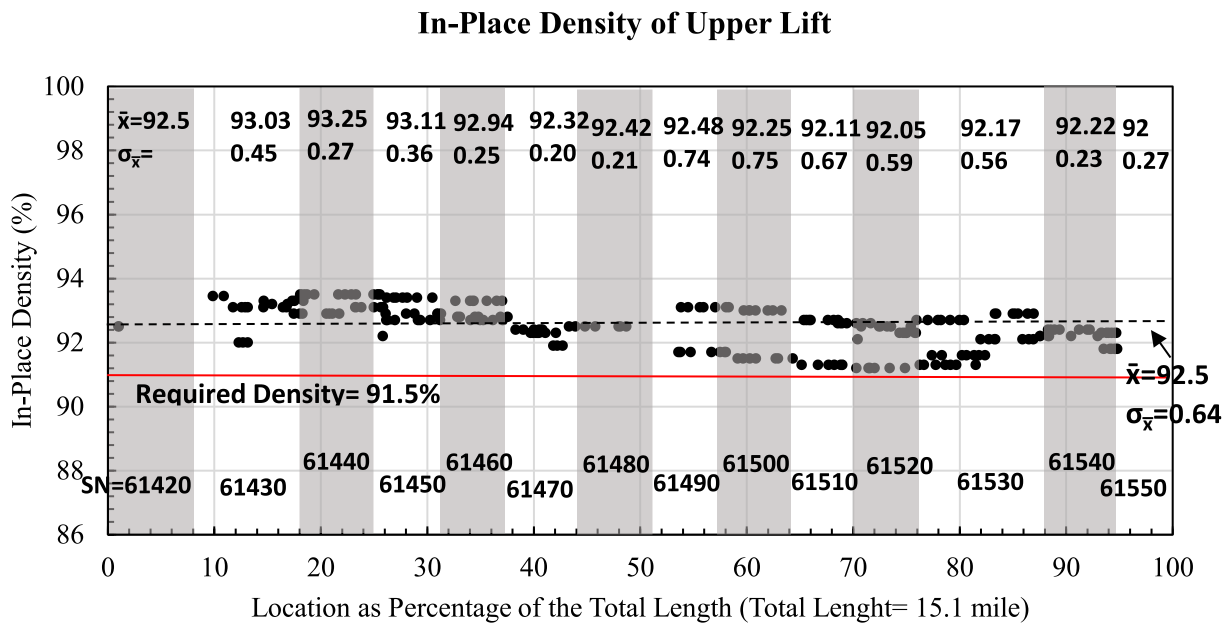


Figure 4-16 In-Place density (Gmm %) based on station locations (project USH 45) (Upper Lift)

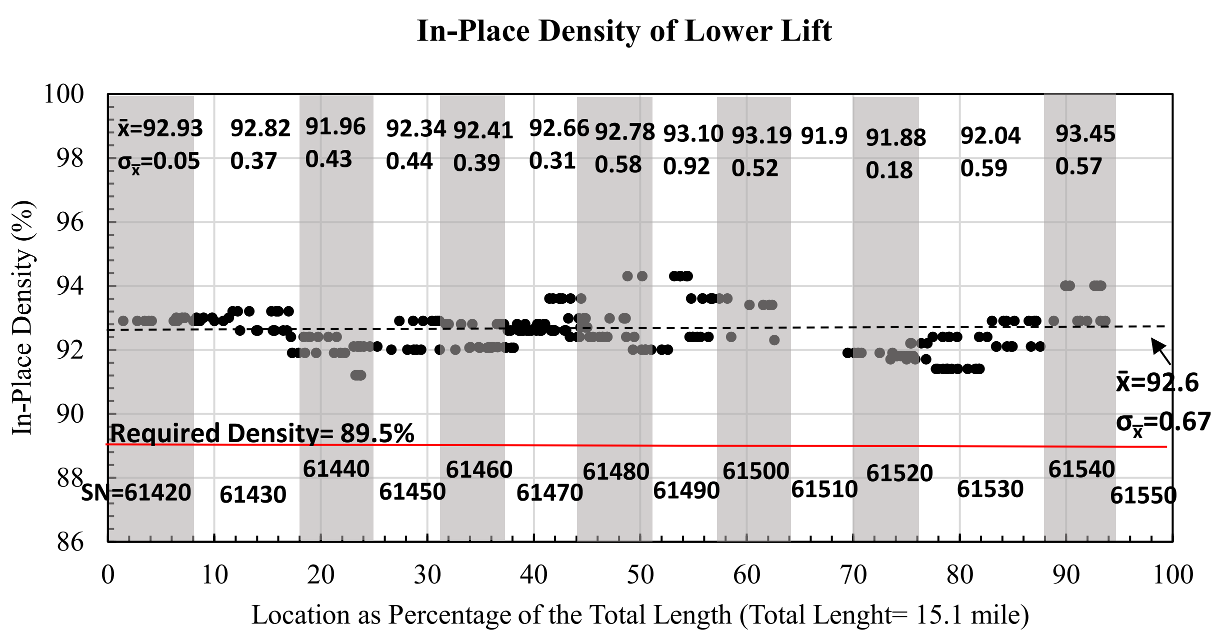


Figure 4-17 In-place density (Gmm %) based on station locations (project USH 45) (Lower Lift)

USH 45 Summary (#1600-21-70)

1. The in-place density for the SN presented in the figures show consistent values around the average. This is confirmed by the low value for the standard deviations within the individual SN and the pooled data. The results represent an attention-grabbing situation. While low Va is correlated with high alligator cracking, the high in-place density is correlated with low alligator cracking. Examining the regression coefficients, and correlation trends presented in the global analysis (chapter 3), the weight impact of changing one unit of Va on alligator cracking DI is about three times that of the in-place density. This could be the reason.
2. The regression model prepared based on the PIF database showed a significant level of rutting and no alligator cracking. Since rutting was not examined in the field along with the FWD, it is difficult to compare the global analysis for rutting to the on-site distress survey results.
3. The PCI values reported in the PIF database show high values that underestimate the effect of the PIF reported rutting damage or the on-site measured alligator cracking.
4. It is noteworthy that such soft mix did not experience rutting, which is the common belief. However, the data coincide very well with the global analysis. As the Va increase, the pavement is more prone to cracking than rutting. On the other hand, as %Gmm increases, the potential for rutting drops. This is the case for this pavement where the pavement was constructed with an average of 2% higher %Gmm.

4.3 Project STH 13 (#1610-41-60)

Construction year: 2011

Design Traffic: 2,109,000 ESALS

Asphalt Binder Content: 5.3%

Asphalt Layer Thickness: 2” + (existing 3.75” to 5”)

SN	14190	14200	14210	14220	14230	14240	14250	14260	14270	14280	14290	14300
PCI	79	82	82	79	79	81	81	77	81	82	73	72

This project is located in Price County in North Central Wisconsin. It is constructed in 2011 using 36,600 tons of HMA. The project covers 13 SNs. However, due to the lack of information about the last SN, only information of 12 SNs are discussed here. The most recent distress survey was conducted in 2016. The pavement was built with one lift of 12.5 mm NMAS. Figure 4-18 shows the location of the project. The surface lift was constructed on an existing layer of HMA and a concrete layer beneath it.

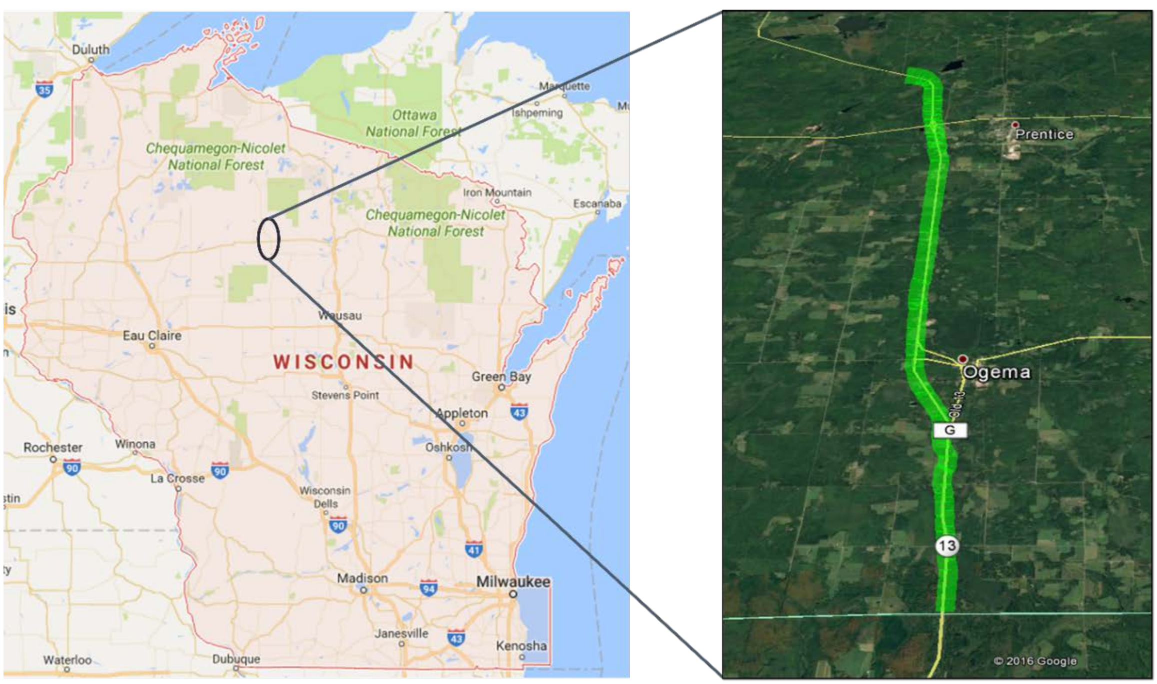


Figure 4-18 Highway STH 13, project 1610-41-60, Westboro-Prentice, Price County

In-Service Performance

The PIF database, show that longitudinal and transverse cracking are the predominant distresses within this project. Field visits confirmed that the longitudinal cracking is distributed in the centerline of the highway and the center of the lanes. Visual inspection indicates that the transverse cracks observed are most likely reflective cracks from the existing underlying HMA layer. Figure 4-19 shows the distribution of the DI of the different distresses recorded on this highway in PIF. It is surprising to notice that alligator cracking is recorded in two of the 13 SNs. At the time of recording the data in the PIF database, the pavement was only five years old.

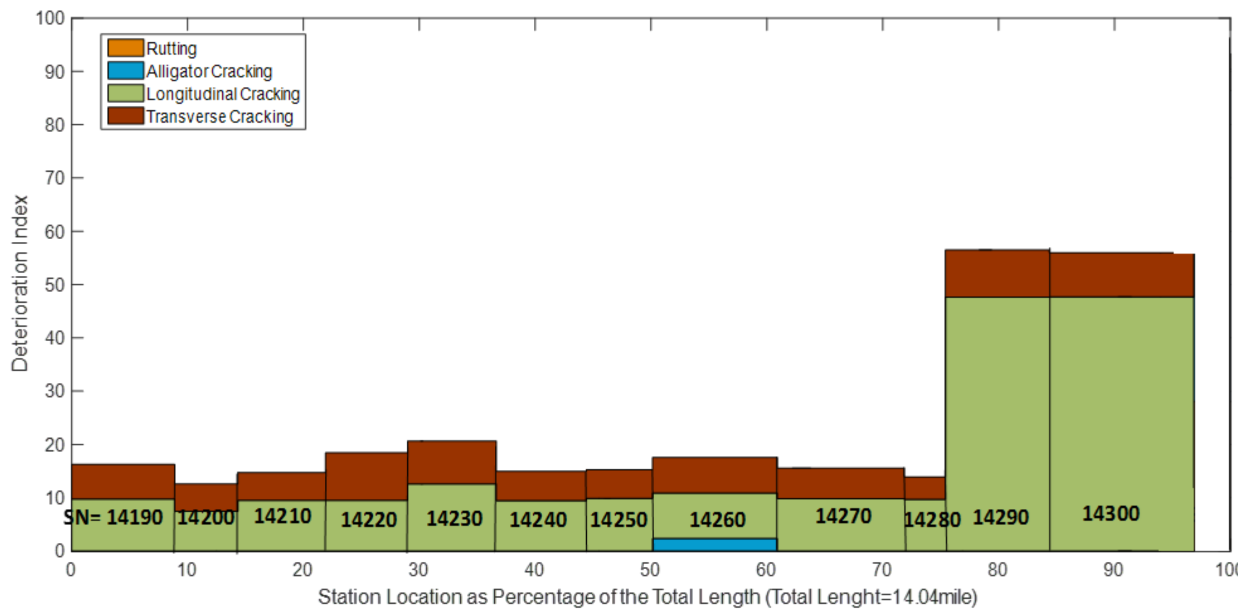


Figure 4-19 Performance of the project STH 13 (based on the DOT information) (2016)

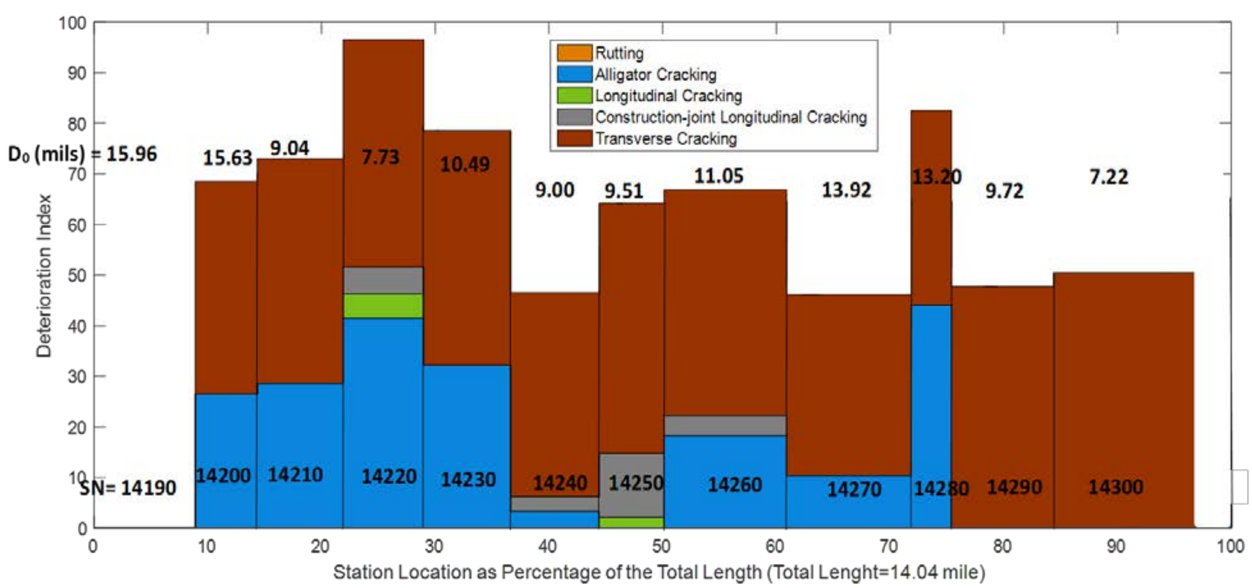


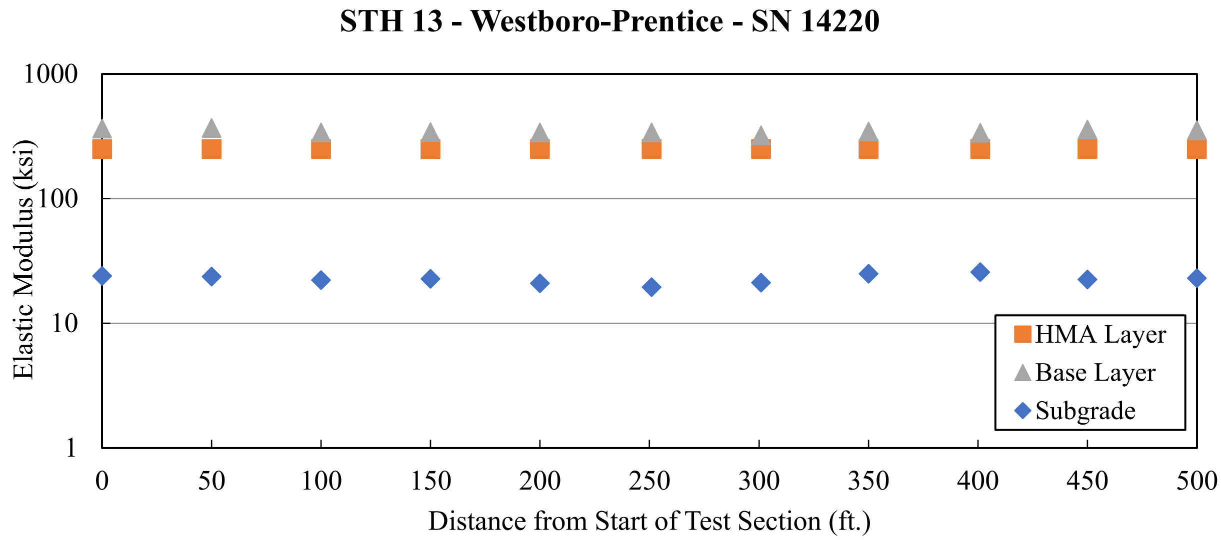
Figure 4-20 Performance of the project STH 13 (based on the research group survey) (2017)

Figure 4-20 shows the DI distribution for the different distresses based on the on-site distress survey. Again, the results are different compared to the PIF in terms of the extent of the distresses, although the survey was conducted only one year after the PIF. However, both sources agree on the types of distresses observed. The PIF data for SN 14310 reflects distresses not included in this project as it is at the edge of the project.

Since the longitudinal and transverse cracking observed, appear to be influenced by factors such as reflective cracking and construction joints. It is difficult to rely on these distresses for evaluating the role of the quality data. Therefore, the analysis will mainly focus on the early alligator cracking observed. The SNs showing alligator cracking are 14200, 14210, 14220, 14230, 14240, 14260, 14270, and 14280.

SN 14220 shows the most deterioration among these SNs for this highway. It is important to note the FWD test revealed that the pavement structure for this SN is one of the strongest as shown by the low value of deflection (D_0) under the load reported in Figure 4-20. For this report the D_0 values are used to screen potential dependency of the distresses on structural issues. Based on experience, the following levels of D_0 are used to evaluate structure stability: Good: $D_0 < 10$, Moderate: $10 < D_0 < 15$, Poor: $15 < D_0$.

In Figure 4-21, the back-calculated modulus at the 50ft spacing within the surveying section show the consistent value of high Moduli for the pavement structure. This means that the failures observed are not due to structural deficiency, but rather material related.



From (ft.)	0	50	100	150	200	250	300	350	400	450
To (ft.)	50	100	150	200	250	300	350	400	450	500
D_0 (mils)	7.45	10.37	5.63	8.35	6.21	9.14	10.1	8.32	6.45	6.36
Alligator Cracking (DI)	8.06	0.00	0.00	28.98	52.04	35.09	17.68	41.02	36.85	33.57
Longitudinal Cracking (DI)	0.00	0.00	0.00	0.00	0.00	0.00	24.88	33.60	0.00	0.00
Construction-joint Longitudinal Cracking (DI)	0.00	7.53	0.00	30.20	0.00	0.00	0.00	0.00	0.00	0.00
Transverse Cracking (DI)	37.54	37.54	37.54	65.73	42.98	43.50	35.55	42.61	49.32	31.23

Figure 4-21 Elastic moduli of layers compared with distresses for SN 14220

It is important to note that the back-calculated modulus for the base layer is higher than that of the surface layer. This is because the base layer is, in fact, a combination of an older HMA pavement layer and a concrete layer beneath it. This is a critical case since having base layer stiffer than the surface layer increases internal stresses within the surface layer. Combining this with the fact that the pavement thickness is only 2”, this is a typical scenario for early life cracking.

Production Quality

The distribution of the mix quality is shown in Figure 4-22 and Figure 4-23 below.

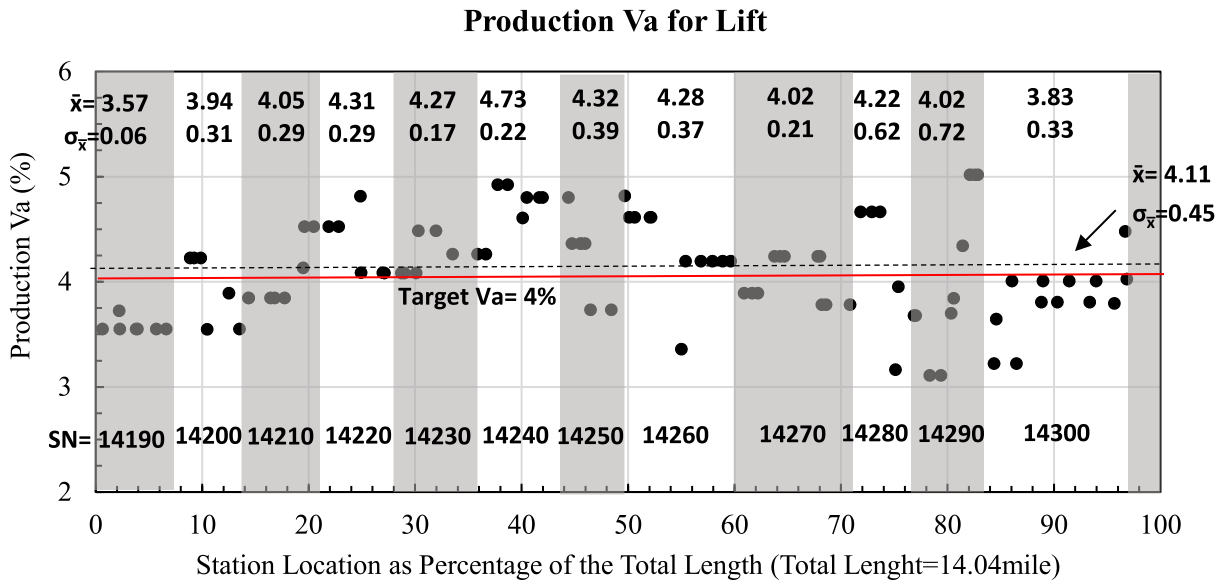


Figure 4-22 Production information of air voids (Va %) based on station locations (project STH 13)

The average air voids for the mix production is 4.11% which is indicative of on spec mix. Some SN show mixes compacted to low air levels close to 3% and others compacted to high levels close to 5%. The same trend of fluctuation is observed for the VMA as seen in Figure 4-23. The average VMA for the project is 14.54, which is on spec, with a lower level of variability compared to Va.

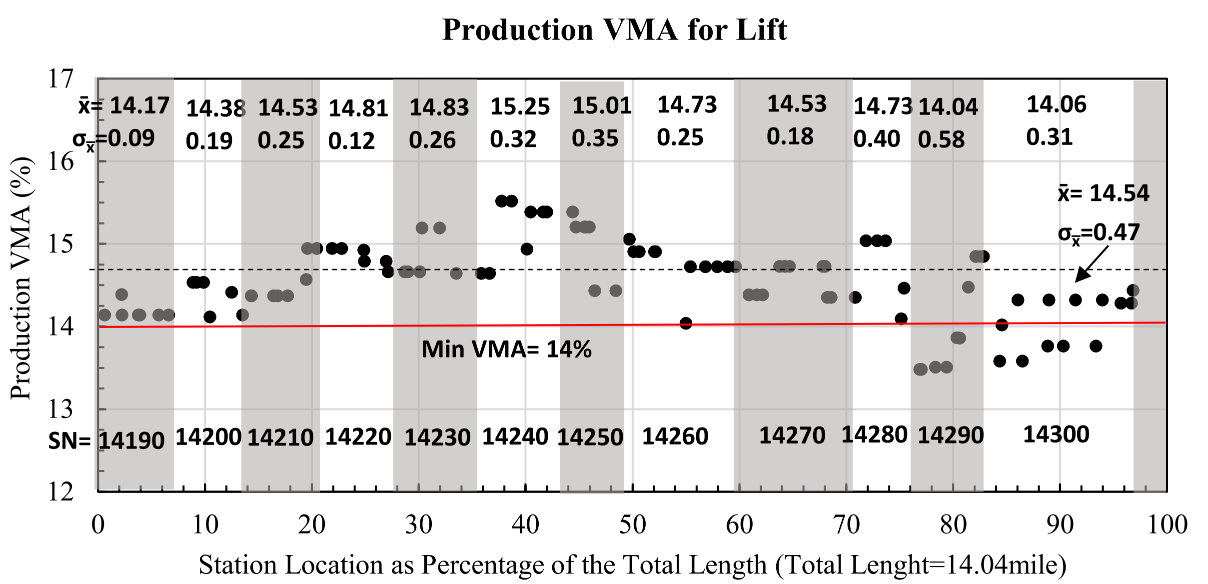


Figure 4-23 Production information voids in mineral aggregates (VMA %) based on station locations (project STH 13)

Figure 4-22 and Figure 4-23, some SN needs a closer look due to the noticeable trends in the mix data. For example, SN 14280 on average is conforming to Va of 4.22%, but multiple tests for the mix Va within this SN do not conform to the specs. In fact, this SN has one the lowest recorded Va in the project and the highest variability. This coincides with the highest alligator cracking noticed on site. The adjacent SNs (14290 and 14300) show no sign of alligator cracking while having some of the tests giving low levels of Va. It appears that the contractor was gaining control of the mix for these SNs and that there is interaction with placement density that will be discussed later.

Placement Quality

Field compaction on this project achieved to an average of high density at 3.16% above target required density. Figure 4-24 shows the distribution of the in-place density quality. The variability in the in-place density is also prevalent similar to the production variability. SN 14220, which is the one showing the most cracking, in general, has a 4% wide range of density measurements, but all the individual tests exceed specification requirements. In fact, only one data point was compacted below target dentistry in the entire project.

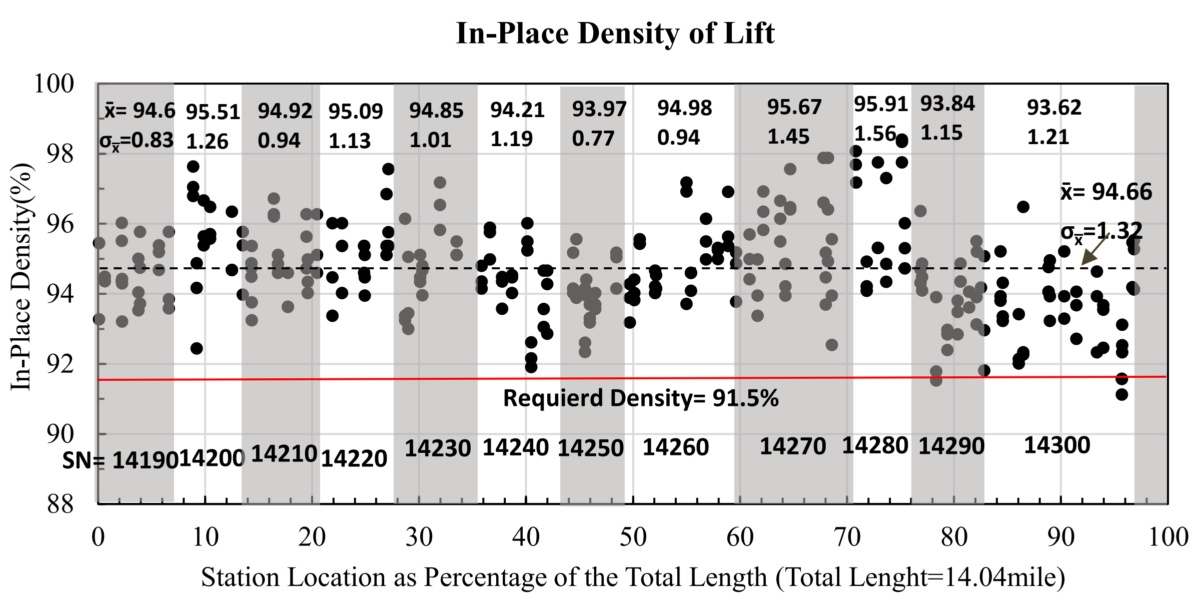


Figure 4-24 Construction information of in-place density (Gmm %) based on station locations (project STH 13)

STH 13 Summary (#1610-41-60):

1- Production data showed variability within specification limits.

2- Placement quality shows the significantly high level of compaction with a density more than 3% above required target density. However, it is highly variable.

3- Several types of distresses are observed. Causes for some could be due to reflective cracking. Figure 4-25 shows sample pictures from the project. This level of cracking after five years of service is highly likely to be due to reflective cracking, especially that the pavement is only 2” thick over old HMA. Thus, the analysis focuses on alligator cracking

4- Most of flagged SNs based on the production quality do not overlap with the alligator cracking SN. The following table lists the flagged SNs against the alligator cracking SNs.

5- It can be concluded that the placement quality data show association with the in-service performance for this project. The FWD results show that the pavement structure is adequate. The distinctions in the columns listed in the table above are not based on average values from the SN but rather individual data points within the SN.

6- The selection of 3.75% Va as the lower threshold is based on the results of the analysis of the global data.

7- The 96% Gmm cutoff in the placement density is driven by the data and engineering judgment. For a 2 in the thick pavement, compacting beyond this density could lead to damaging the aggregate structure in the mix.

8- Another factor that could have had a detrimental effect on the performance is the material selection. Binder type (modification) and grade are unknown for this project. Also, the job mix formula could be inadequate for such an overlay over the stiff base. It is expected if a soft JMF is placed in a thin layer (2 inches in this case) on stiff base, that cracking under traffic will take place rapidly.

Figure 4-25 Satellite and street view photos from Google Earth ® of the distresses (SN 14300)

Table 4-1 List of SN showing Alligator Cracking and Flagged Based on Production Quality

Alligator Cracking	Production Below 3.75%Va	Production Above 4.75%Va	Production High Variability	Placement Over 96%Gmm	Placement Under 91%Gmm
14200	×			×	
14210				×	
14220				×	
14230				×	
14260				×	
14270				×	
14280			×	×	

4.4 Project STH 13 (#1620-00-79)

Construction year: 2011

Design Traffic: 2,469,500 ESALS

Asphalt Binder Content: 6.1%

Asphalt Layer Thickness: 6”

SN	13690	13700	13710
PCI	100	99	96

This project is a section of highway 13 located in the Marathon County at the center of the state. The construction of the project was completed in 2011 with 30,355 tons of HMA. This highway has 3.26 miles length starting from Marshfield town, and it ends at Spencer County (Figure 4-26). Based on the production information, the top lift layer of HMA has an NMAS of 12.5 mm. The last conducted performance survey was done in 2016.

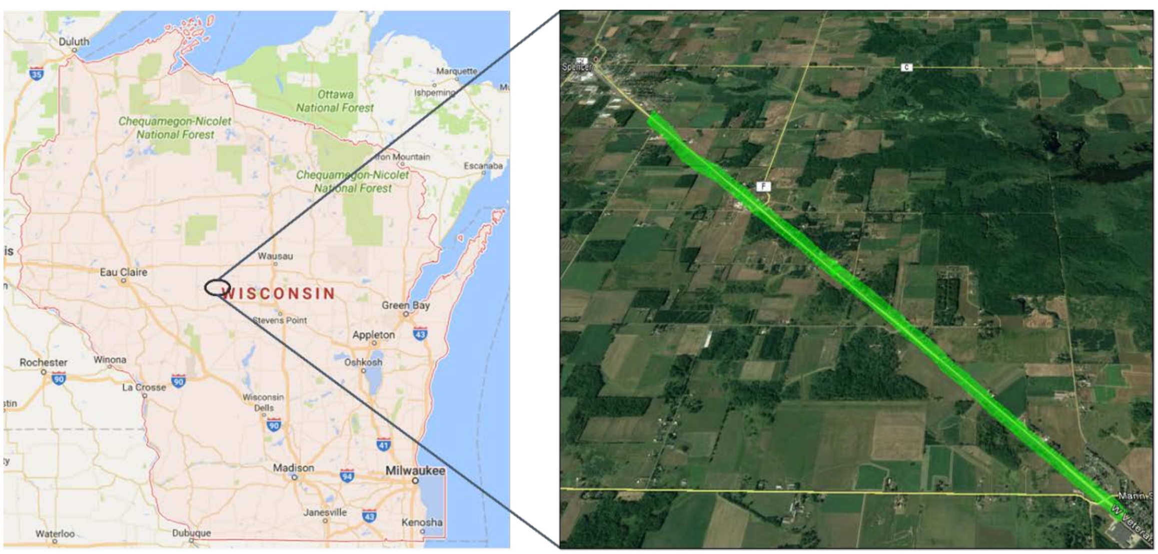


Figure 4-26 Highway STH 75, project 1620-00-79, Marshfield-Spencer, Marathon County

In-Service Performance

The project has three SNs in total. At the time of the PIF survey, the pavement was five years old. As the PCI numbers suggest, minimal distresses are expected. Figure 4-27 is showing the distribution of the distresses on the entire length of the project. According to the PIF, the two first SNs (13690 and 13700) are clear of sings of distresses. However, the last SN (13710) is showing the minor level of transverse cracking. The performance surveys of this project did not show any rutting problem on the entire length. Similar to the previous projects, the 2017 on-site distress survey shows more details than the PIF. The results are shown in Figure 4-28.

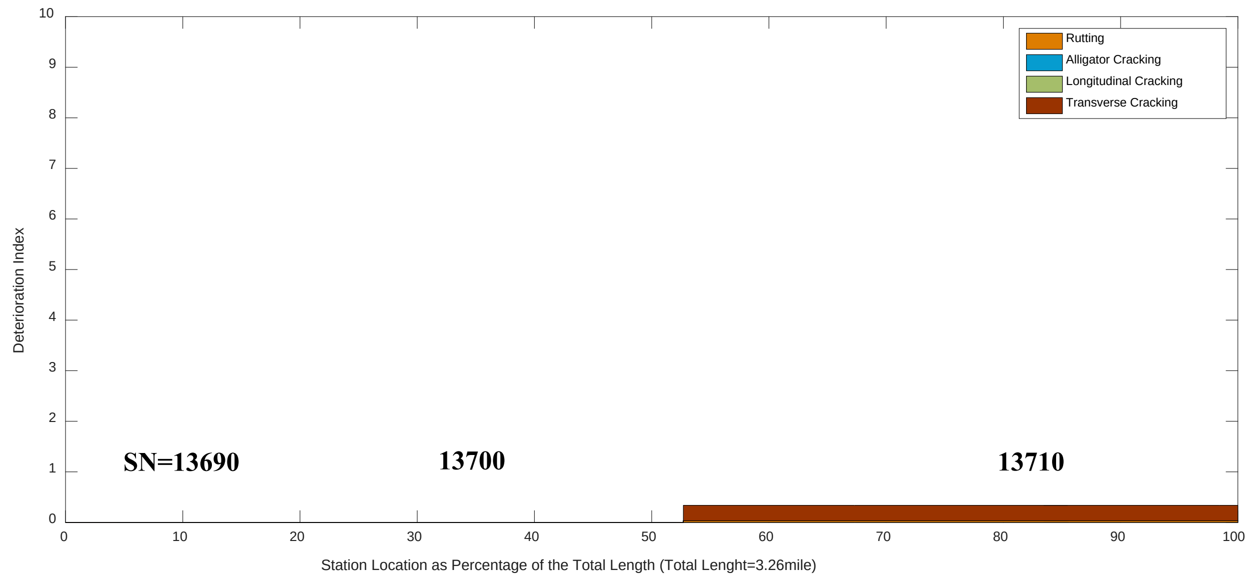


Figure 4-27 Performance of the project STH 13 (based on DOT information) (2016)

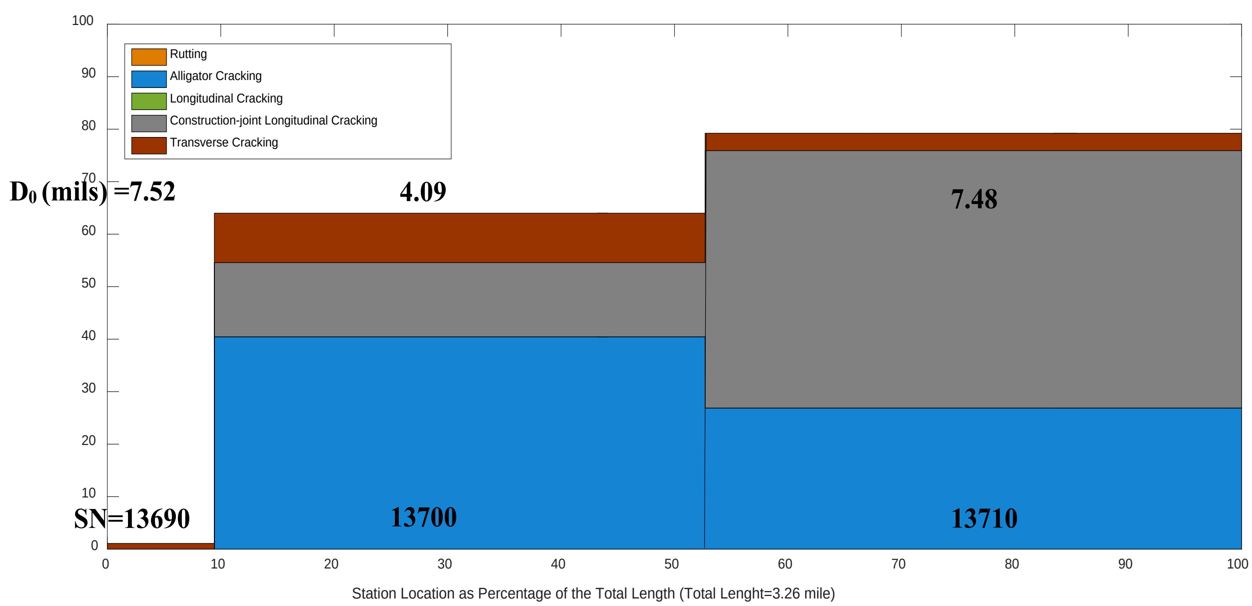
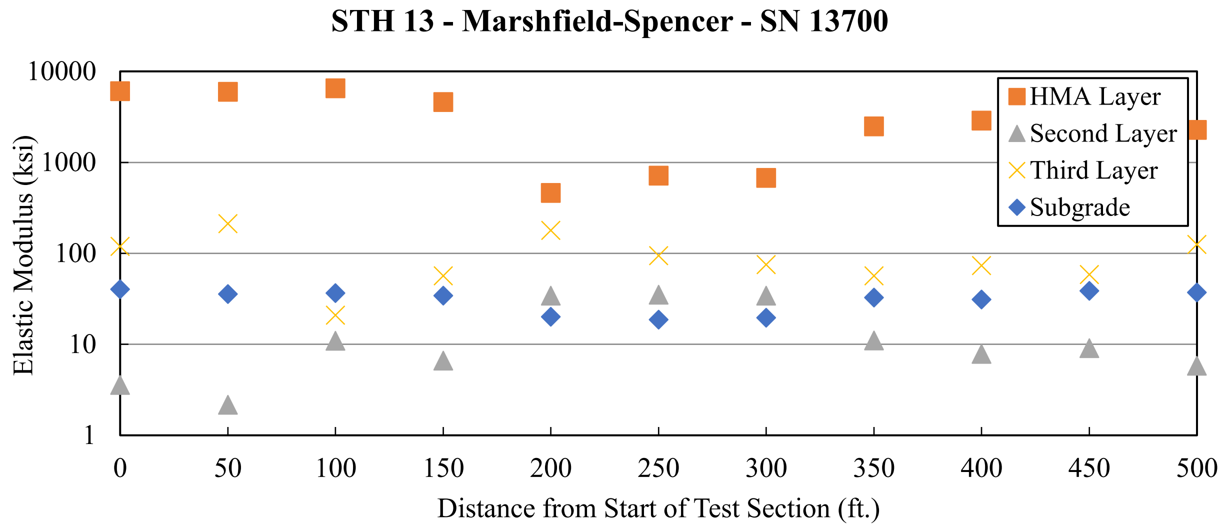


Figure 4-28 Performance of the project STH13 (based on research group survey) (2017)

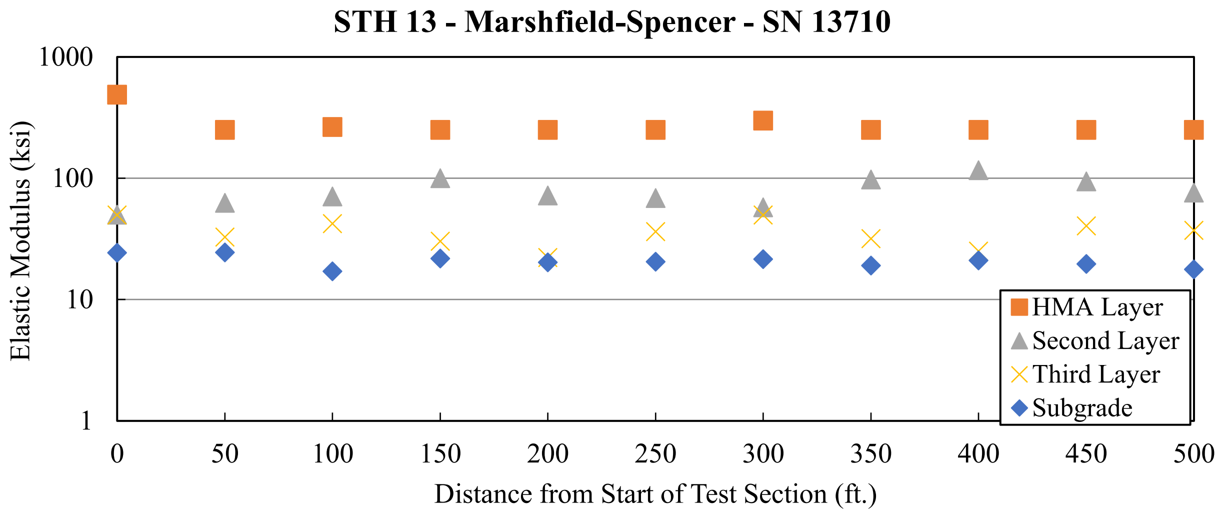
Based on the information presented in Figure 4-28 significant construction joint longitudinal cracking is observed. There is a consistent presence of the transverse cracking but at a lower level. The results show that two of the three SNs are having significant alligator cracking. The FWD deflection under the drop weight (D_0) is also shown in the figure. These values are consistently on the lower side indicating structurally stable pavement on average. To evaluate the structural stability of SN 13700 and 13710, the Figure 4-29 and 4-30 are shown detailed results of the FWD test and the special distribution of the distresses .



From (ft.)	0	50	100	150	200	250	300	350	400	450
To (ft.)	50	100	150	200	250	300	350	400	450	500
D0 (mils)	3.61	3.55	3.28	3.7	4.78	4.91	5.29	4.05	3.88	3.92
Alligator Cracking (DI)	35.14	26.35	48.15	46.26	46.19	48.29	34.33	21.28	14.29	20.42
Longitudinal Cracking(DI)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Construction-joint Longitudinal Cracking (DI)	15.31	5.59	18.73	28.88	9.87	20.26	4.73	13.30	0.00	10.14
Transverse Cracking (DI)	35.59	5.00	0.00	13.40	9.87	11.34	1.09	0.70	0.00	0.00

Figure 4-29 Elastic moduli of layers compared with distresses for SN 13700

As can be seen in Figure 4-29, the data suggests a significant drop in surface modulus in the middle of the 500ft test section. This drop coincides with the presence of higher severity alligator cracking. For SN 13710, the FWD showing a different picture of the structural capacity.



From (ft.)	0	50	100	150	200	250	300	350	400	450
To (ft.)	50	100	150	200	250	300	350	400	450	500
D0 (mils)	4.07	5.65	7.32	6.95	6.9	9.75	8.46	6.74	7.34	7.66
Alligator Cracking (DI)	23.53	29.99	11.77	36.85	30.09	11.77	26.81	27.22	11.77	0.00
Longitudinal Cracking(DI)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Construction-joint Longitudinal Cracking (DI)	57.01	57.01	57.01	57.01	27.44	28.88	28.88	57.01	46.69	8.19
Transverse Cracking (DI)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	24.25	0.00	0.00

Figure 4-30 Elastic modulus of layers compared with distresses for SN 13710

The results show that the entire section is structurally weak. On the other side, the nature of the FWD testing is to obtain a relative comparison between the different points on the same project and comparison of the relative stiffness of the different layers. The results suggest that the stiffness of the surface layer is higher than the underlying layers, but they are within the same order of magnitude, which suggests a soft structure.

At this point, the failure in the pavement sections appears to be structural. The following sections investigate the possible contribution of the production and placement quality measures.

Production Quality

With respect to data used for evaluating this quality, the only available data was for the 12.5 mm NMAS top lift mix. Data for the 19 mm bottom lift was not available. Figures 4-31 and 4-32 show the Va and VMA production data by location on the project and SN.

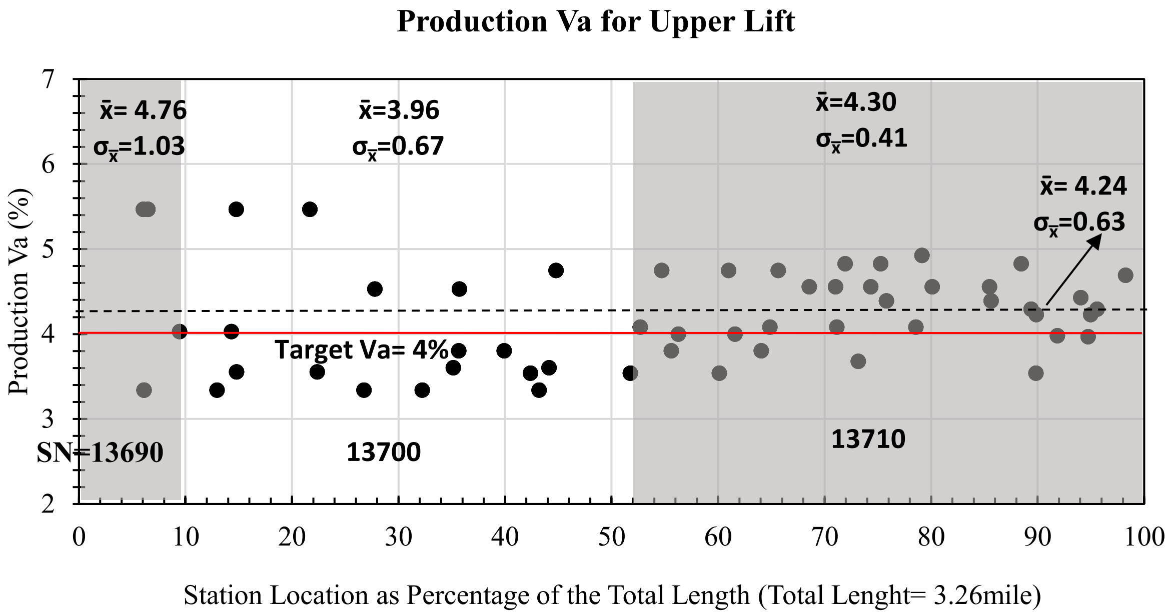


Figure 4-31 Production information air voids (Va %) based on station locations (project STH 13)

The Va data is on average meeting the target level of 4%. That is on spec. However, SN 13700 is showing high variability as demonstrated by the standard deviation (correspond to the coefficient of variability of 17% respectively). Assuming normal distribution, 53% of the area is below 4% Va, and 38% is below 3.75%. This is a low level of Va according to the global analysis conducted in the previous chapter and is associated with Alligator cracking. SN 13710 show ideal values for Va with minimal variability, yet it experienced alligator cracking as well. VMA data show the lower level of variability and 100% conformity. The distribution is shown below. There is no apparent association between the VMA and the observed performance.

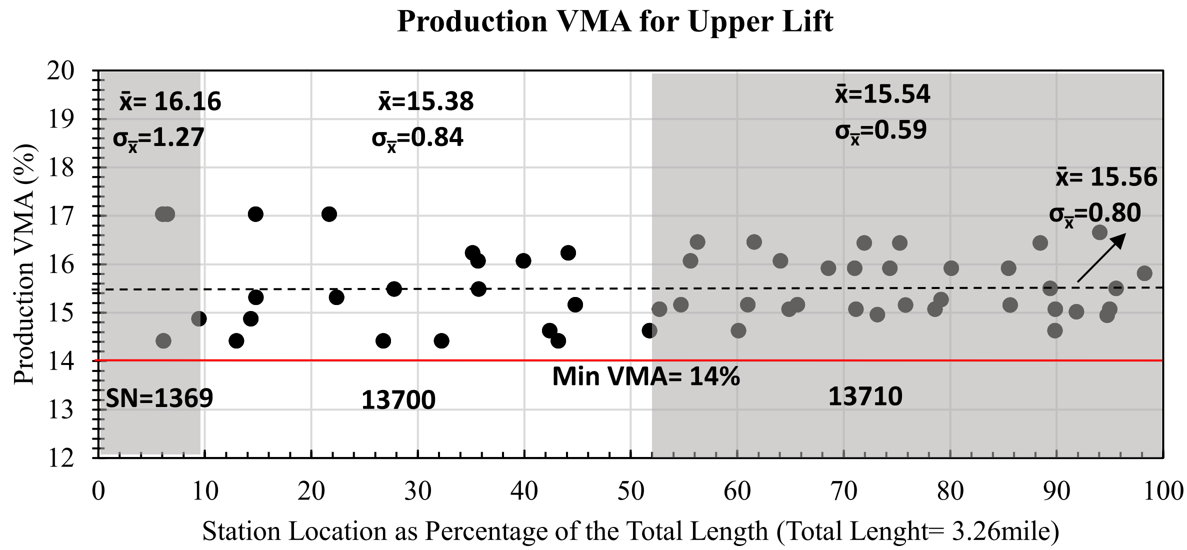


Figure 4-32 Production information of voids in mineral aggregates (VMA %) based on station locations (project STH 13)

Placement Quality

Placement quality data is available for both lifts. The distribution of the measurements is shown in Figures 4-33 and 4-34. The upper lift shows an average density of about 1% above requirement. The bottom lift is about 1.9% above requirement.

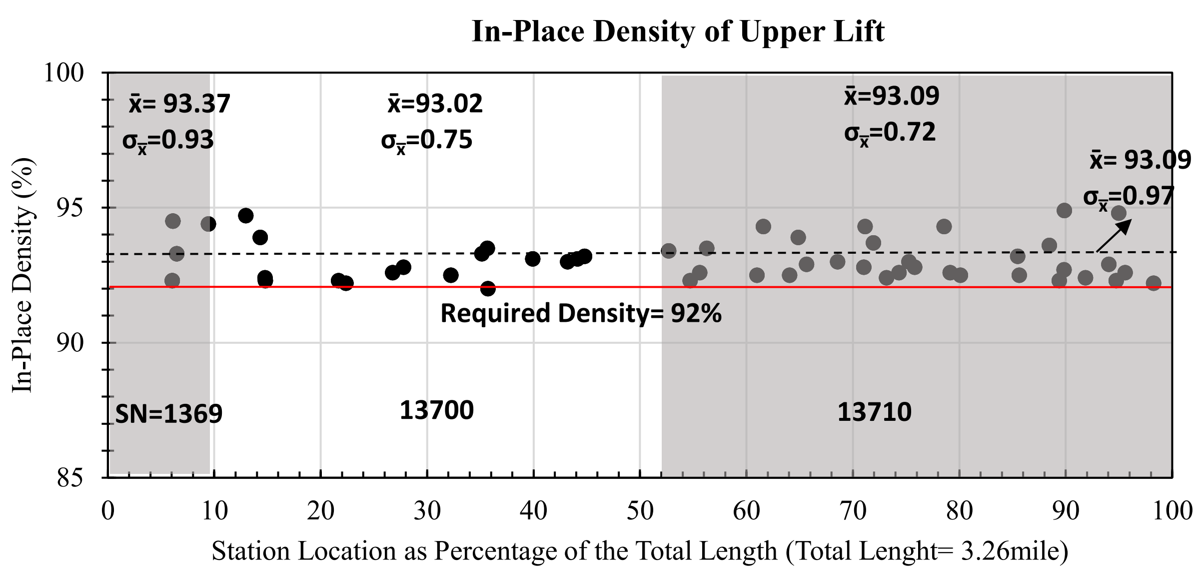


Figure 4-33 Construction information of in-place density (Gmm %) based on station locations (project STH 13) (Upper Lift)

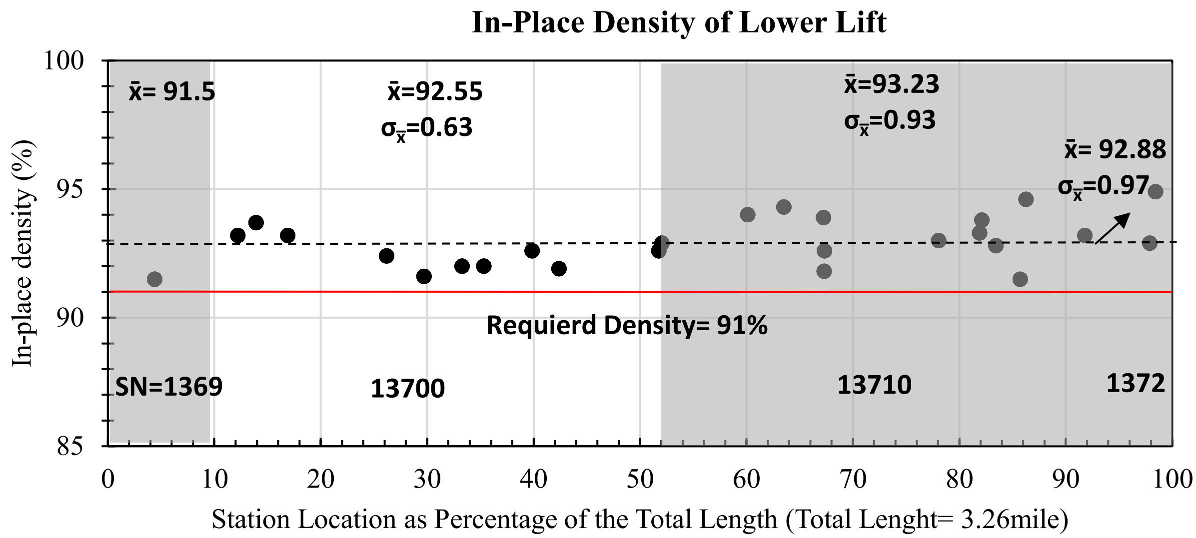


Figure 4-34 Construction information of in-place density (Gmm %) based on station locations (project STH 13) (Lower Lift)

The placement density is showing 100% conformity for both lifts. In addition, the density levels do not suggest any excessive levels of compaction was applied.

STH 13 Summary (#1620-00-79):

1- Production data show high variability in some locations. These locations also show non-conforming Va data points.
2- Placement density does not appear to be influenced by the variability in production. All measurements were either on spec or exceeding. Further, the variability for both lifts is much lower compared to production.
3- Structural nondestructive testing and mixture volumetric suggest that the distresses observed are due to the interaction of both structural and material deficiencies.
4- SN 13700 is characterized by the lowest level of Va as well as the fluctuation in structural stability. This section experiences the most alligator cracking in the project.
5- Comparing this project to the one before it; they are the same age, but the level of deterioration in project #1620-00-79 is much lower. This could be attributed to the adequate level of placement compaction.

4.5 Project STH 75 (#2420-02-70)

Construction year: 2011

Design Traffic: 620,500 ESALS

Asphalt Binder Content: 5.2%

Asphalt Layer Thickness: 2”

SN	95820	95830	95840	95850	95860	95870	95880	95890
PCI	82	79	80	77	80	81	80	88

State Trunk Highway 75 construction location is in the southeastern part of Wisconsin State. The highway runs from Wisconsin Highway 50 and 83 to the highway 11. This project construction is 8.36 mile long. It serves Kenosha and Racine counties. The construction of the project was completed in 2011 by using 50,000 tons of HMA. Location of the project is shown in Figure 4-35.

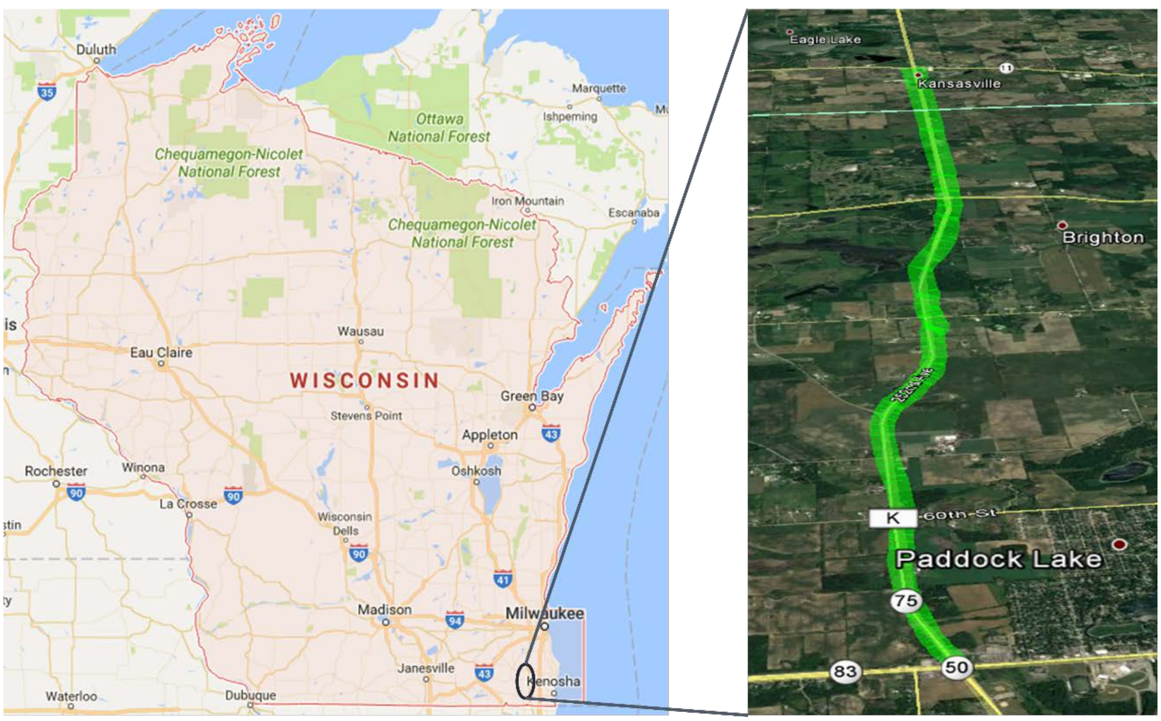


Figure 4-35 Highway STH 75, project 2420-02-70, Kenosha and Racine Counties

In-Service Performance

The performance of the project STH 75 as reported in the PIF is shown in Figure 4-36. Transverse and longitudinal cracking are the most common among all the SNs. Alligator cracking is reported in one SN only. In any case, all these distresses are reported to have low DI value. The on-site survey is again documenting higher levels of distresses. The survey shows that the longitudinal cracking distributed between centerline construction joint cracking, and in line longitudinal cracking. The on-site survey noted alligator cracking in all SN. The extent of the distress is greatest at the ends of the project limits as shown in Figure 4-36.

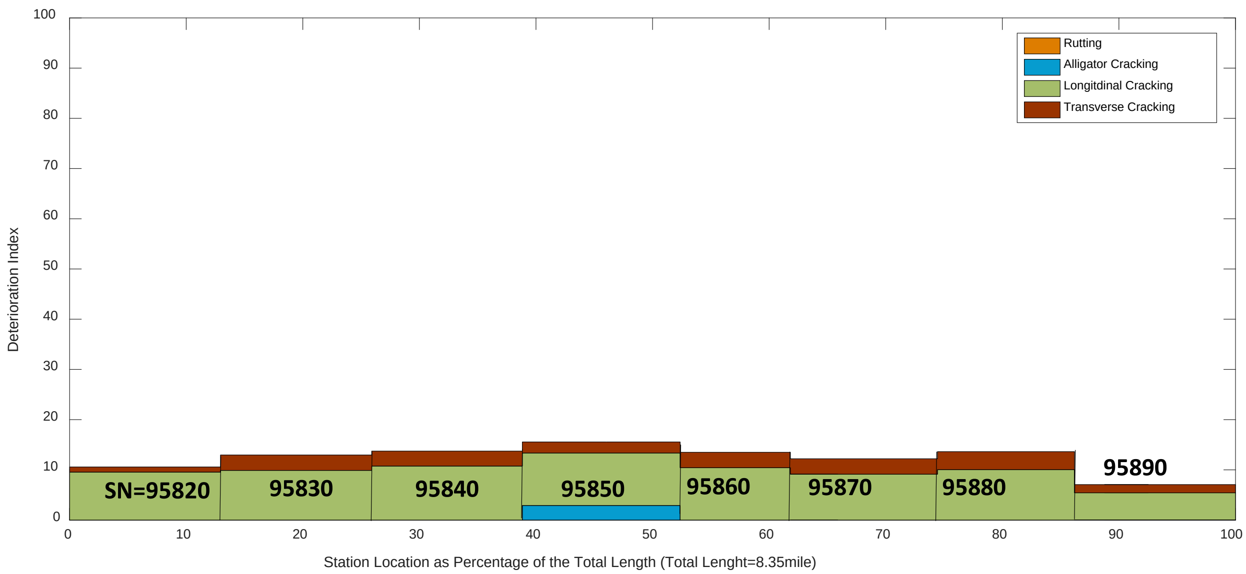


Figure 4-36 Performance of the project STH 75 (based on the DOT information) (2016)

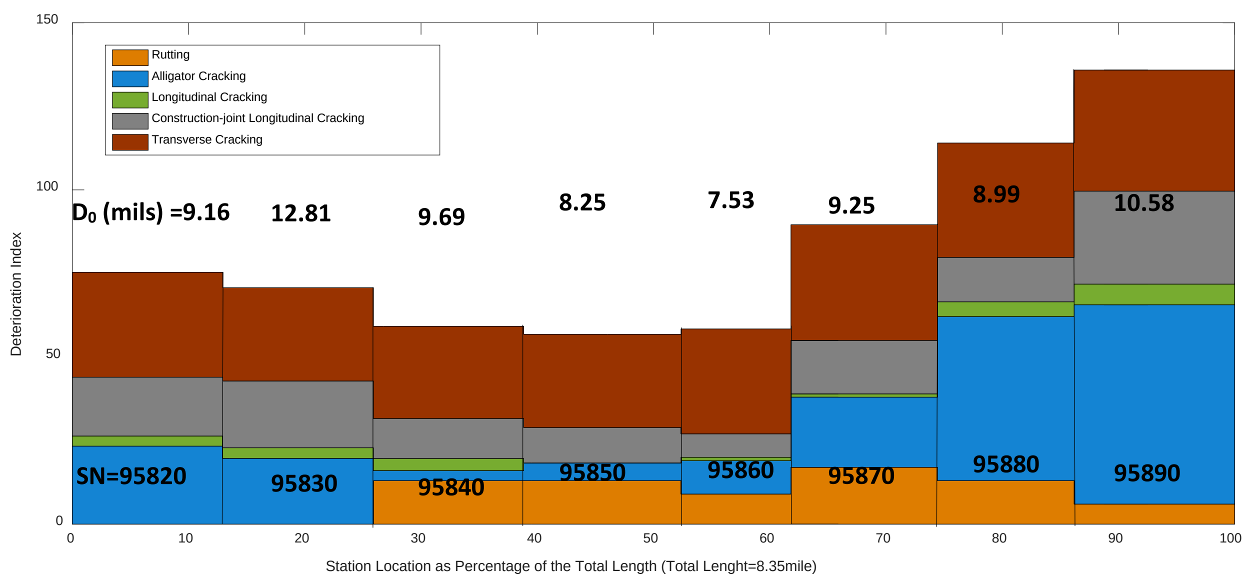


Figure 4-37 Performance of the project STH 75 (based on the research group survey) (2017)

As seen in Figure 4-37, transverse cracking is uniformly distributed over all the SNs of the project, same as the longitudinal cracking. While the middle portion of the project shows the least amount of distress overall, which contradicts with the PIF, minor rutting is also noted in this portion of the project. Examining the D_0 average values for these SNs, the structure appears to stable. Thus, the distresses are not expected to be due to structural failures.

Production Quality

Production quality shown in Figure 4-38 and Figure 4-39 reflects the Va and VMA of the 12.5 mm upper lift. With respect to Va, the average of the projects in on spec, with the individual SNs show on spec Va as well. However, SN 95860 show high standard deviation (COV = 19%) due to the variation of about 1.5% air within the SN. The VMA data reported show minimal variability, with all the data points exceeding the minimum required value.

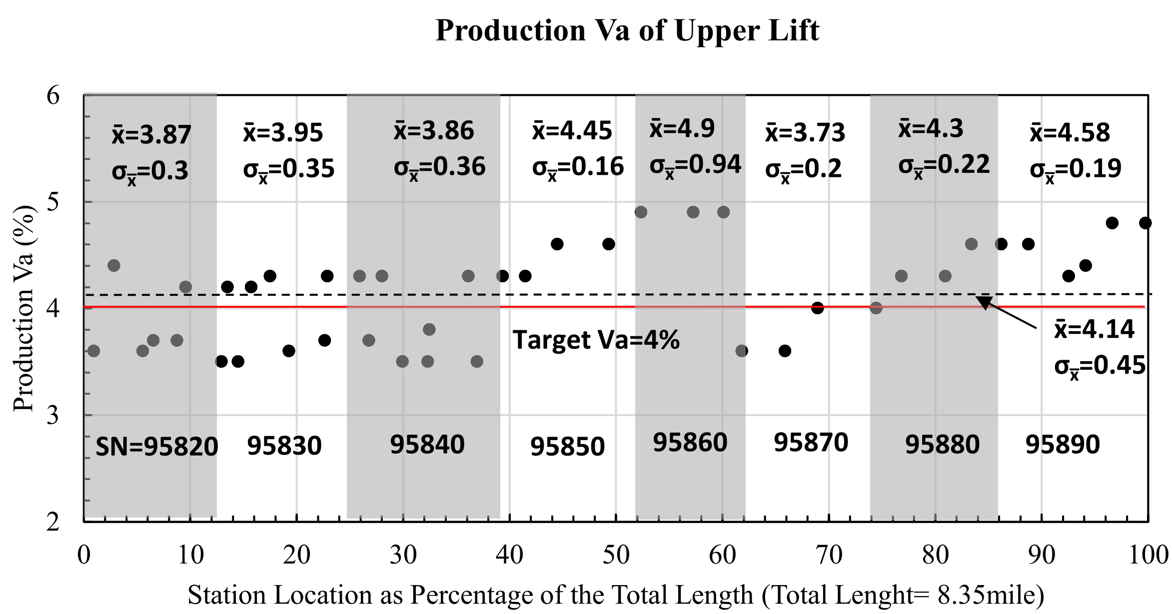


Figure 4-38 Production information of air voids (Va %) based on station locations (project STH 75) (Upper Lift)

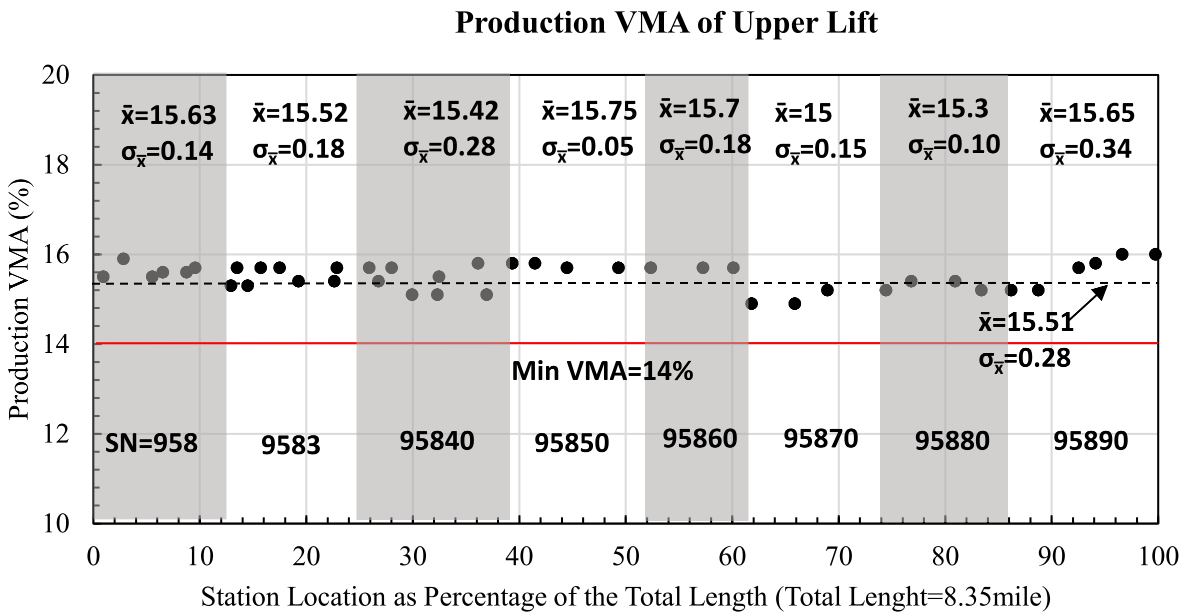


Figure 4-39 Production information of voids in mineral aggregates (VMA %) based on station locations (project STH 75) (Upper Lift)

The middle SNs (95850 and 95860) show the high value of Va while they have the least level of distresses. The three SN at the first 40% of the project length shows a significant number of sublots with low Va (below 3.75%). These SN show a gradual increase in distress DI.

The last 40% of the project length, show even more severe levels deterioration indices. Va for the three SN making the last 40% of the length shows an increasing trend. SN 95870 shows Va level as low as 3.6%, and it continues to increase until it reaches its highest for SN 95890 at Va value of 4.8%. While this increasing trend indicated continuous changes in the mix to achieve Va, the VMA shows a lower level of variability. However, this could relate to variations in the mix, but the data available does not provide information about these possible variations. Therefore, the sever distress levels at this portion of the project cannot be associated with production quality on a cause and effect relationship.

Placement Quality

Placement quality data show all data conforming to the required density value. SN 95890 show the incomparable average density of 94.83%; this is about 2% above all other SNs in the project.

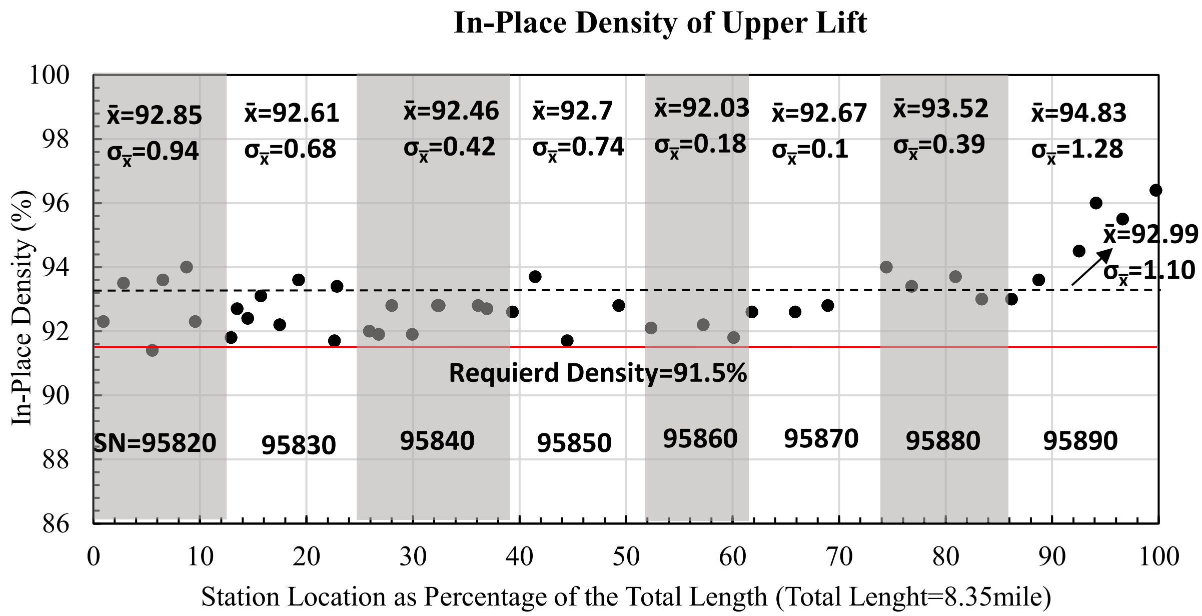


Figure 4-40 Construction information of In-Place Density (Gmm %) based on station locations (project STH 75) (12.5 mm Lift)

The placement density shows consistent compaction throughout the first 90% of the project length. SN 95890 is showing on average 1.5% more compaction than required at %Gmm of 92.99. This is due to few spots showing a very high level of densification as high as 96.4%. For a 2in lift, this level of compaction is dangerous. Combining this over compaction with the fact that the mix at this location shows high resistance to compaction in as demonstrated by the high level of mix Va. There is a good chance that the severe level of deterioration observed is due to crushing of the mix during construction.

STH 75 Summary (#24-20-02-70):

1. Low Va is associated with increased DI for the first 40% of the project length.
2. Variations in the mix may be related to the increased DI for the last 40% of the project length. However, the information available does not provide conclusive evidence.
3. The field over-compaction is associated with the highest level of distresses in this project.

Table 4-2 Summary of the project STH 75

SN	Alligator Cracking	Rutting	Production Below 3.75%Va	Production Above 4.75%Va	Production High Variability	Placement Over 96%Gmm
95820	×		×			
95830	×		×			
95840		×	×			
95850		×				
95860		×	×	×	×	
95870	×	×	×			
95880	×	×				
95890	×	×				×

4. Table 4-2 significantly helps in painting a picture that coincides with fundamentals of asphalt mixtures and the results of the global analysis.
    - Low Va in production and high %Gmm during placement correlate with alligator cracking.
    - Rutting performance does not follow a singular path. The data available suggest the rutting may be influenced by the mix non-conformity and the over compaction during compaction. However, the data available does not include binder properties that may contribute to this distress.

4.6 Project STH 76 (#6430-10-71)

Construction year: 2009

Design Traffic: 34,360 ESALS

Asphalt Binder Content: 4.9%

Asphalt Layer Thickness: 2.5” & 4” + (3” to 4.5” existing)

SN	60780	60790	60800	60810	60820	60830	60850
PCI	92	92	87	90	89	89	87

State trunk highway 76 is a state highway in the Wisconsin state. This highway runs north-south in the east-central part of the state. It starts from highway 45 near Bear Creek to a junction with US Highway 45 in the downtown of Oshkosh. The project number 6430-10-71 is a 7.84-mile project constructed in 2009 in Winnebago County as a part of highway 76. The construction of the project consumed 31,250 tons of HMA for two lifts of asphalt. The lower lift of 4inch thickness has 19 mm NMAS while the upper lift of 2.5inch thickness has 12.5 mm NMAS. The project consists of 7 SNs. The location of the project is shown in Figure 4-41.

In-Service Performance

Performance survey (Figure 4-42) of this project, is showing the distribution of the distresses on the entire length of the project. The degree of deterioration is minimal throughout the project length with minor transverse cracking distributed uniformly. SN 60800 is showing noticeable rutting and minimal longitudinal cracking.

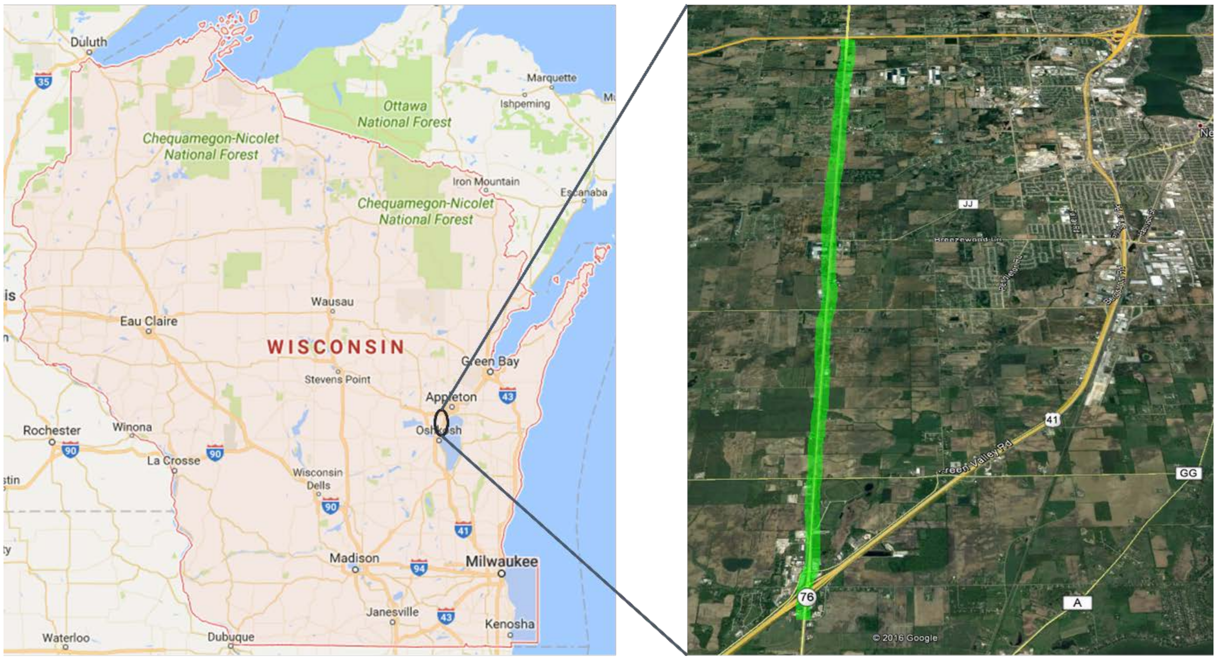


Figure 4-41 Highway STH 76, project 6430-10-71, Winnebago County

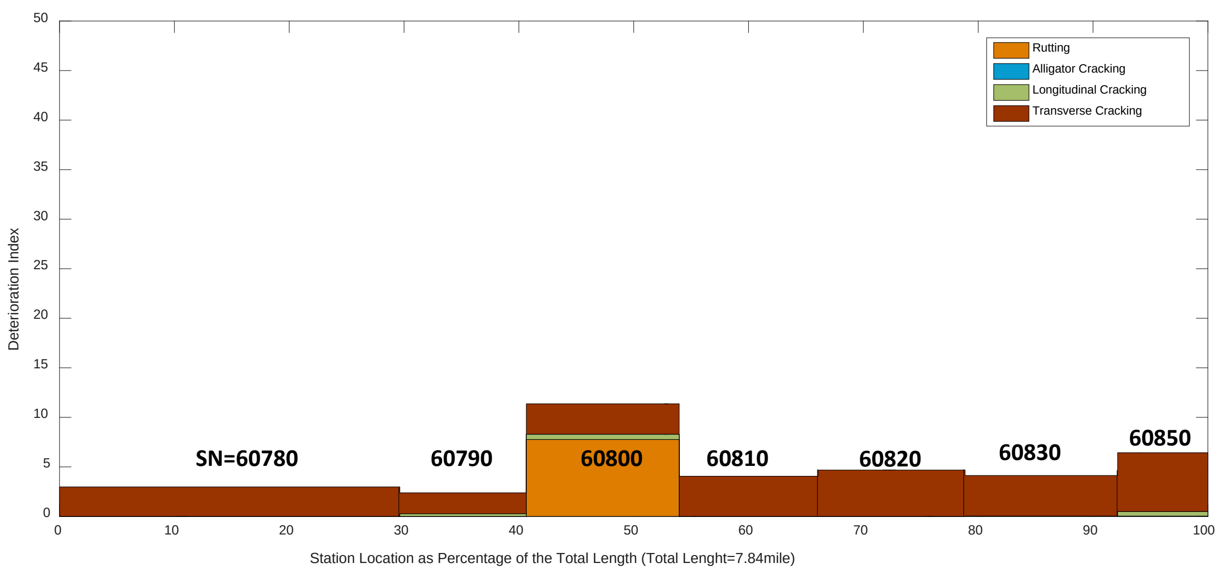


Figure 4-42 Performance of the project STH 76 (based on the DOT information) (2016)

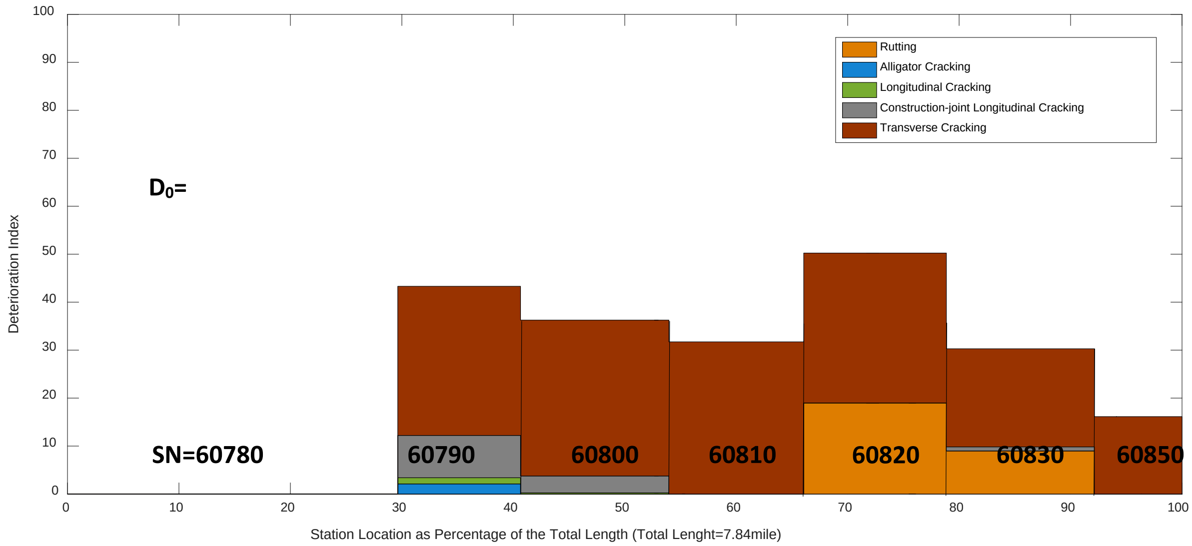


Figure 4-43 Performance of the project STH 76 (based on the research group survey) (2017)

Field distress survey was not conducted on SN 60780 due to traffic control issues. However, the rest of the SN conform with PIF with respect to the presence of transverse cracking, but the extent and severity of the distress are much more severe according to the on-site survey. FWD testing was conducted for this project, but the results are unreliable due to the presence of the transverse reflective cracking. This pavement is eight years old at the time of the onsite survey. Other than the transverse cracking (which is not a reflection of the properties measured in the quality program), the remaining distresses are adequate for the age.

Production Quality

Production information shown in Figure 4-44, Figure 4-45 are reflecting the Va information for the upper and lower lift of the project STH 76. The VMA information for the upper and lower lift are also shown in Figure 4-46 and Figure 4-47 respectively.

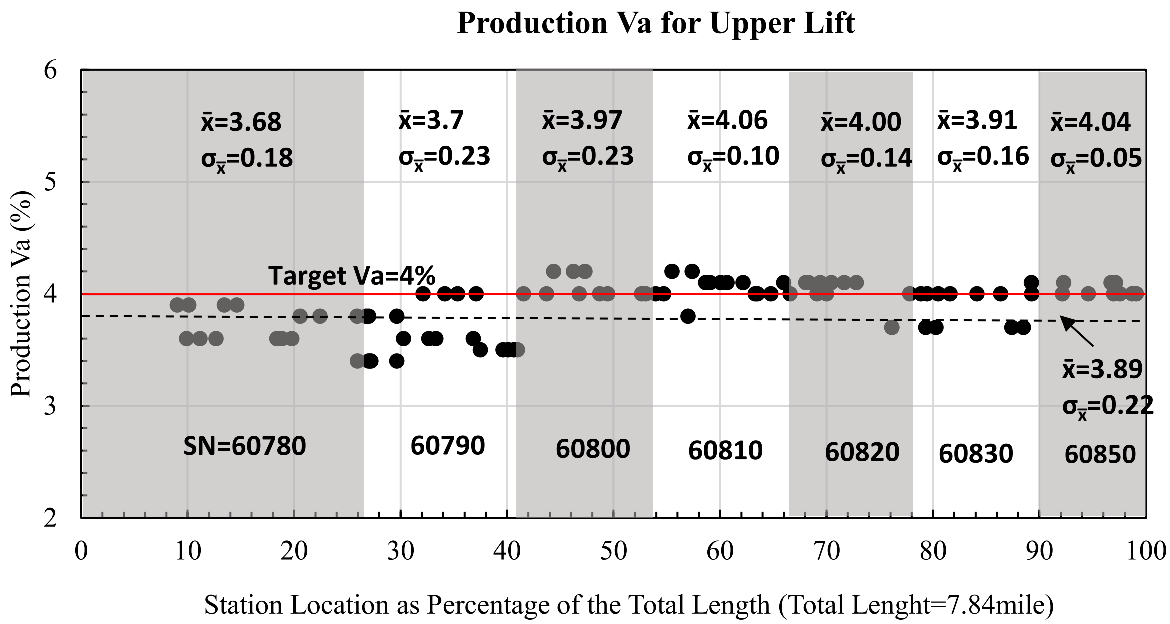


Figure 4-44 Production information of air voids (Va %) based on station locations (project STH 76) (Top lift, NMAS=12.5 mm)

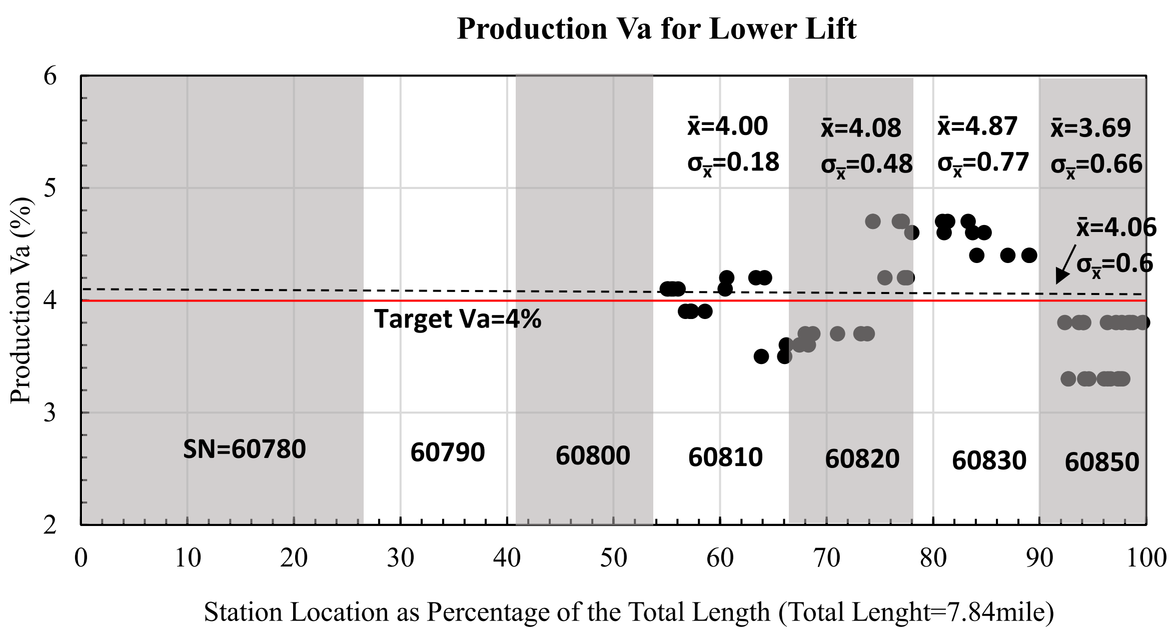


Figure 4-45 Production information of percent of air voids (Va %) based on station locations (project STH 76) (Lower lift, NMAS=19 mm)

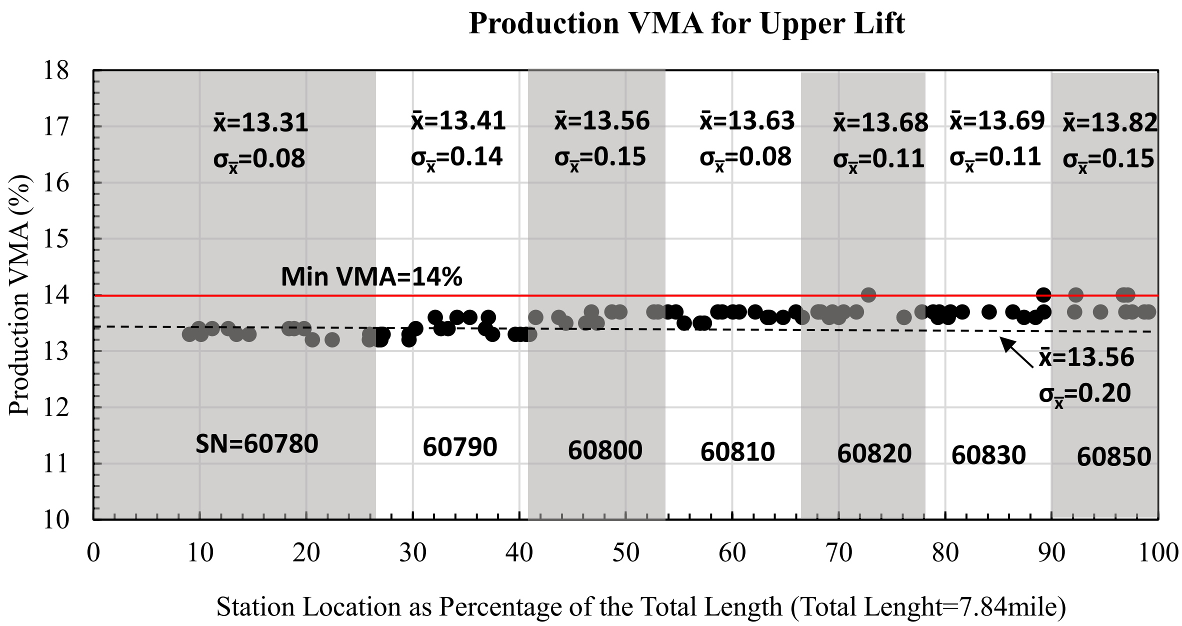


Figure 4-46 Production information of air voids (Va %) based on station locations (project STH 76) (Lower lift, NMAS=12.5 mm)

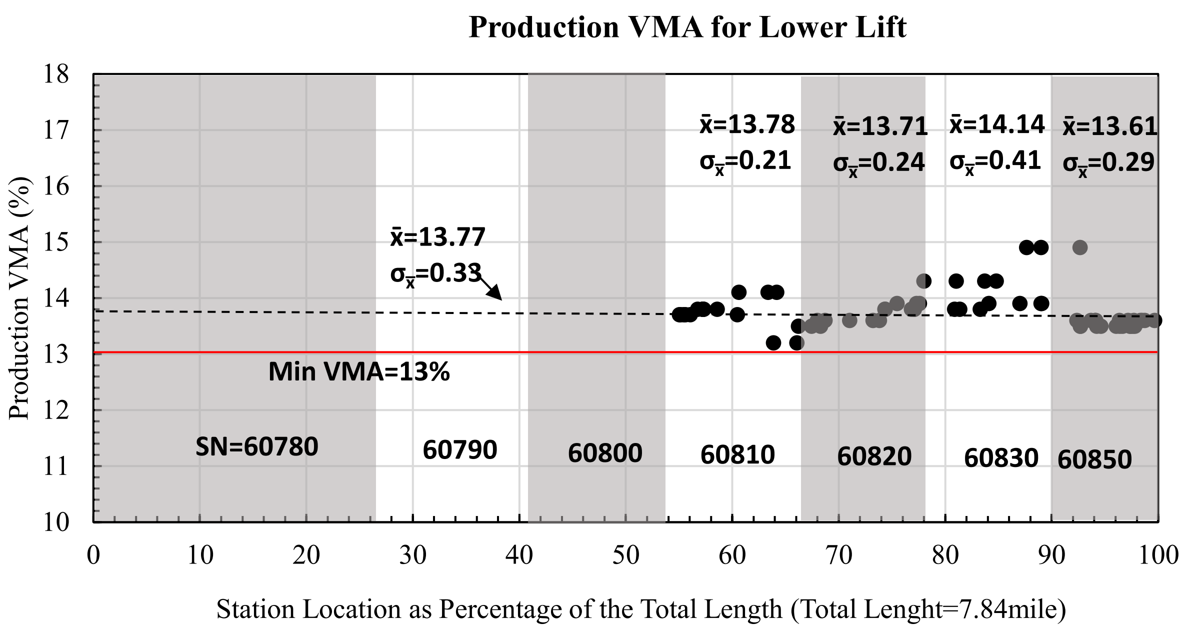


Figure 4-47 Production information of air voids (Va %) based on station locations (project STH 76) (Lower lift, NMAS=19 mm)

The 12.5 mm mix compaction resistance as measured by Va is consistent throughout the project except for SN 60790 which shows some tests below the 3.75%. This section is the only SN showing measurable levels of alligator cracking. The case is a bit different for the 19 mm mix. This is because a higher level of variability observed in the available data. The VMA of the 12.5 mm mix is consistently non-conforming, for all the SN.

Placement Quality

Production quality shown in Figure 4-48 and Figure 4-49 does not show over compaction. However, it shows higher variability with some test points falling below the minimum accepted density for both distresses.

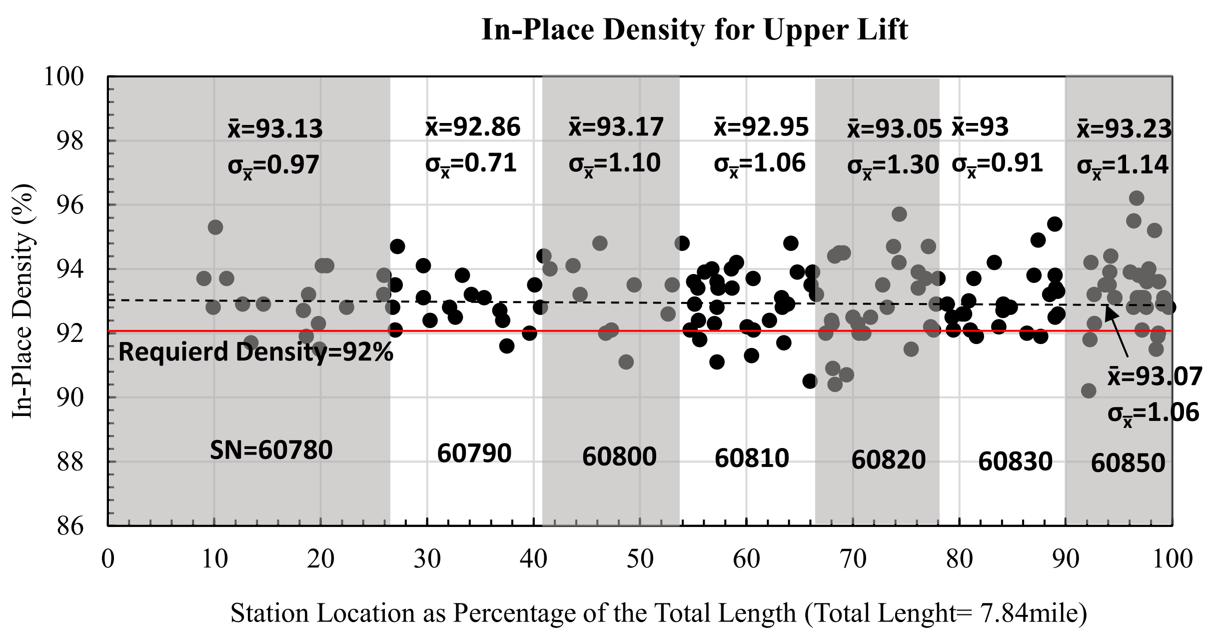


Figure 4-48 Construction information of in-place density (Gmm %) based on station locations (project STH 76) (Top lift)

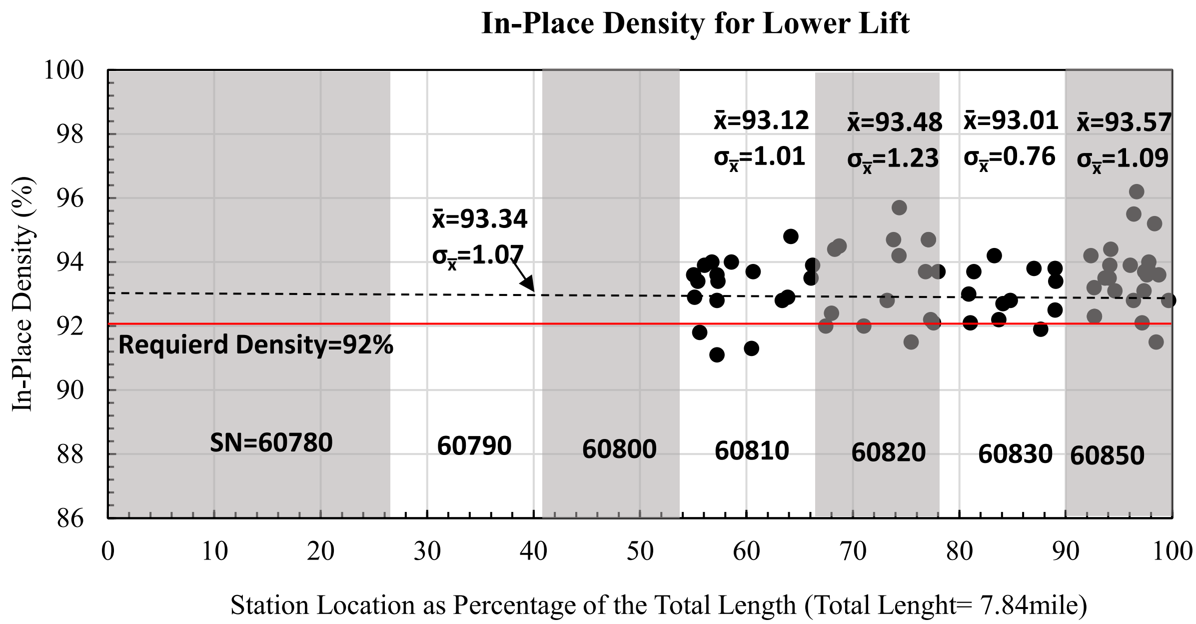


Figure 4-49 Construction information of in-place density (Gmm %) based on station locations (project STH 76) (Lower lift)

Figure 4-50 Pavement condition on the place with the highest compaction variation (SN=60850)

STH 76 Summary (#6430-10-71):

1. Performance of this pavement is dominated by transverse reflective cracking.
2. Other moderate levels of distresses are present in some of the SNs.
3. The production and placement quality data show some individual data points not conforming to the specifications limit. However, the majority of the data points are adequate.
4. Overall, the quality indicators during construction represent conforming pavement. Thus, the lack of significant and severe levels of distresses is attributed to the adequate production and placement compaction.

4.7 Project STH 64 (#8110-06-61)

Construction year: 2010

Design Traffic: 503,700 ESALS

Asphalt Binder Content: 5.2%

Asphalt Layer Thickness: 3.5”

SN	86130	86140	86150	86160	86170	86180	86190
PCI	75	85	84	90	84	84	85

State Trunk Highway 64 as a state highway in the state of Wisconsin, runs east-west across central Wisconsin. The total length of the highway is about 275 miles. However, this selected project is only 6.4 miles of it from Connorsville to Bloomer at Dunn County in the north-west of the state. The project was completed in 2010 by use of 38,900 tons of HMA. The project level is located at a higher elevation at the west side of the project. The height decreases towards the east as it reaches to the flat areas. The location of this project in Wisconsin State is shown in Figure 4-51. The project covers totally seven SNs.

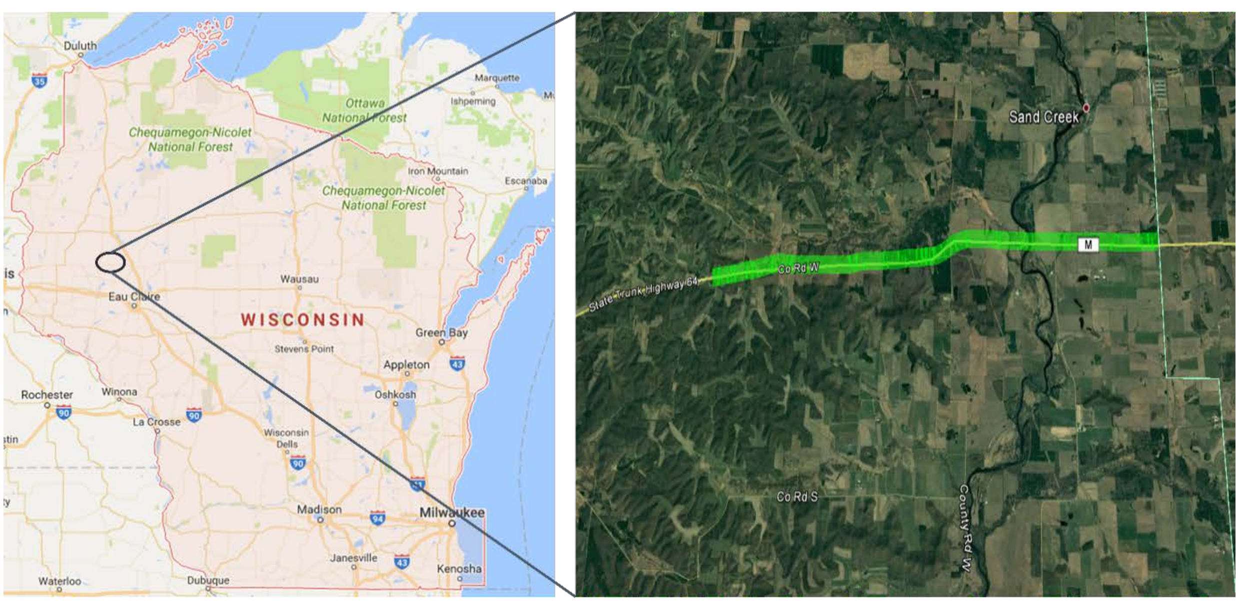


Figure 4-51 Highway STH 64, project 8110-06-61, Connorsville-Bloomer, Dunn County

In-Service Performance

The long-term performance can be investigated through the performance indicators that are shown in Figure 4-52. According to the PIF, the longitudinal and transverse cracking recorded are of the minimal level; only SN 86130 is showing noticeable rutting.

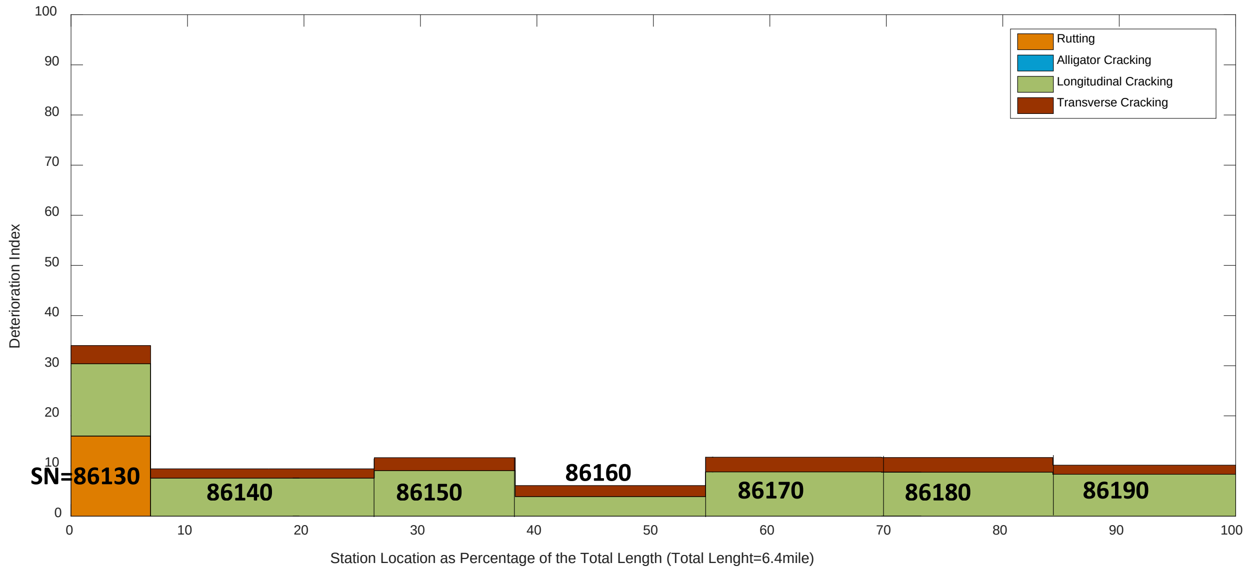


Figure 4-52 Performance of the project STH 64 (based on the DOT information) (2016)

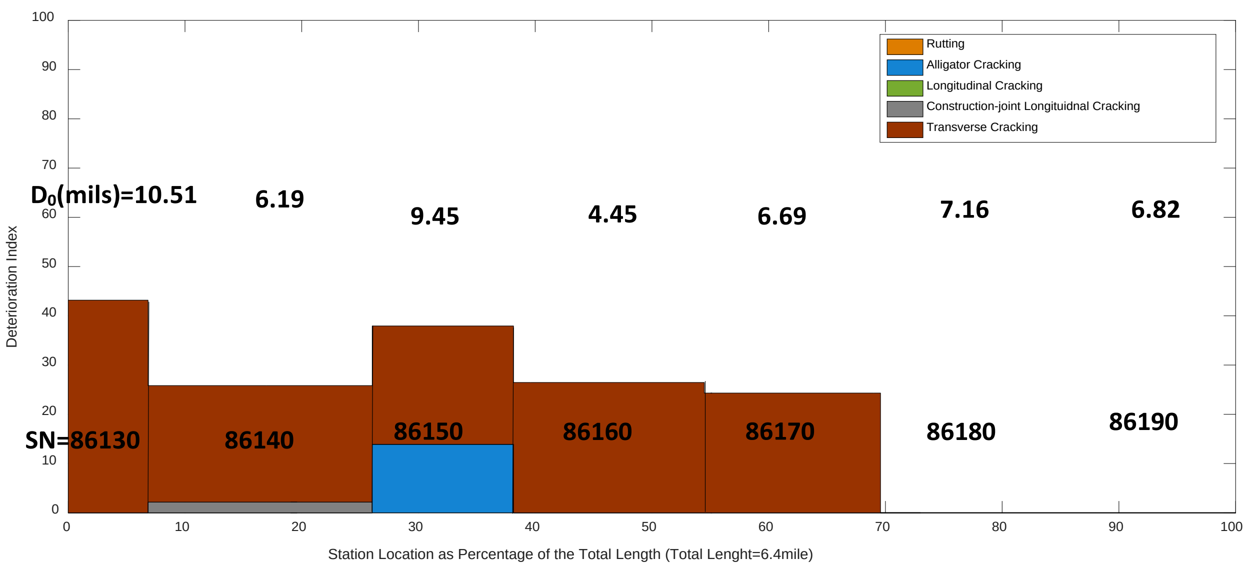
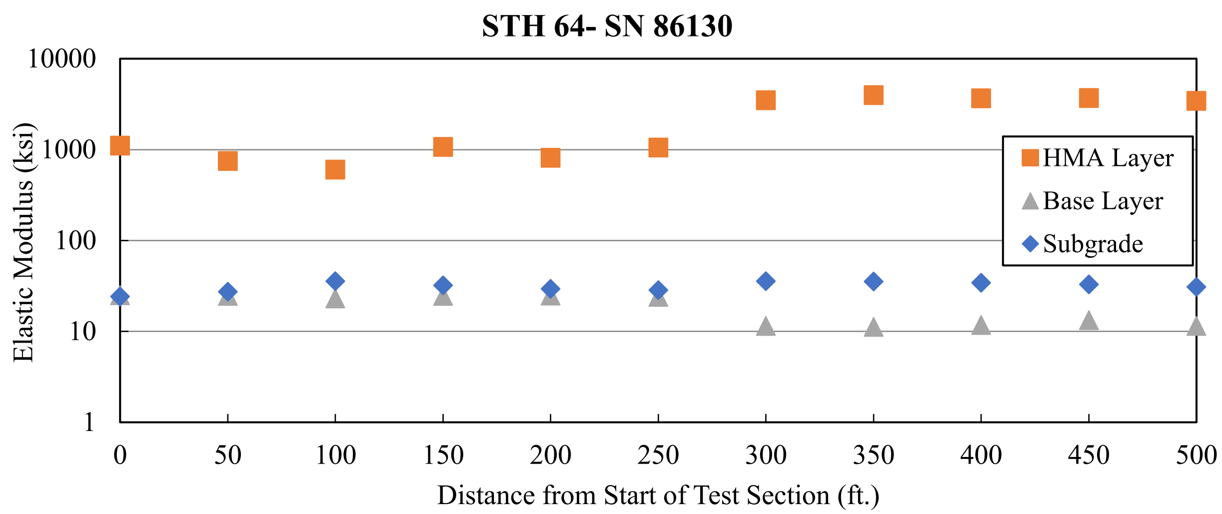


Figure 4-53 Performance of the project STH 64 (based on the research group survey) (2017)

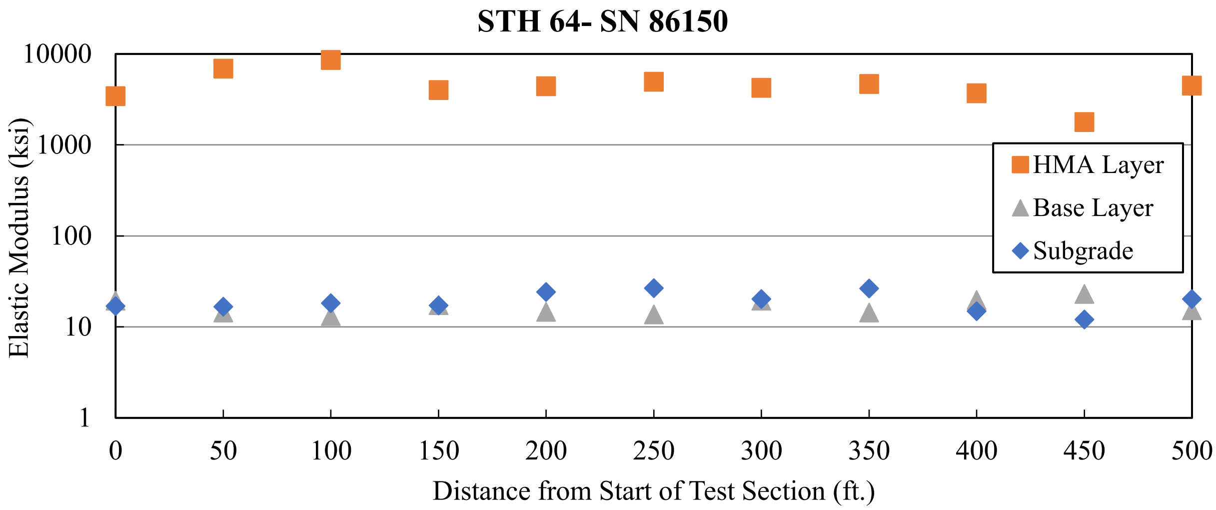
Figure 4-53 shows the on-site survey along with the D_0 values from the FWD. A significant level of transverse cracking is noticed for the first 70% of the project length. No rutting is observed, and alligator cracking is noted at SN 86150. SNs 86130 and 86150 demonstrate the highest level of deterioration and the highest value of D_0. SNs 86180 and 86190 did not show measurable distresses.

In general, the types of distresses observed in this pavement do not indicate poor production or construction quality. A closer look is paid to SN 86130, and 86150 due to their high comparable DI to SN 86180 and SN 86190 due to the lack of recordable any distresses.



From (ft.)	0	50	100	150	200	250	300	350	400	450
To (ft.)	50	100	150	200	250	300	350	400	450	500
D0 (mils)	13.91	13.44	12.69	11.12	12.67	11.85	8.03	7.67	7.91	7.8
Alligator Cracking (DI)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Longitudinal Cracking(DI)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Construction-joint Longitudinal Cracking (DI)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Transverse Cracking (DI)	41.31	59.93	42.98	35.59	55.19	28.29	42.98	47.41	35.59	35.59

Figure 4-54 Elastic modulus of layers compared with distresses for SN 86130



From (ft.)	0	50	100	150	200	250	300	350	400	450
To (ft.)	50	100	150	200	250	300	350	400	450	500
D0 (mils)	10.36	8.47	7.5	9.84	8.36	7.61	8.76	7.69	11.07	15.42
Alligator Cracking (DI)	0.00	0.00	11.77	0.00	0.00	13.28	0.00	0.00	36.85	44.72
Longitudinal Cracking(DI)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Construction-joint Longitudinal Cracking (DI)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Transverse Cracking (DI)	24.95	24.95	24.25	0.00	37.54	24.25	26.71	26.71	24.25	24.25

Figure 4-55 Elastic modulus of layers compared with distresses for SN 86150

Production Quality

The production mix Va is shown in Figure 4-56 provides a noticeable trend. For the first 70% of the project length, mix compaction Va is consistently below the 4.0 % level. Some mix tests had Va value of as low as 3%. For the remaining 30% of the project is constructed with more compaction resistant mix of average air above 4.2%. The same region of the project is the one with the least distresses.

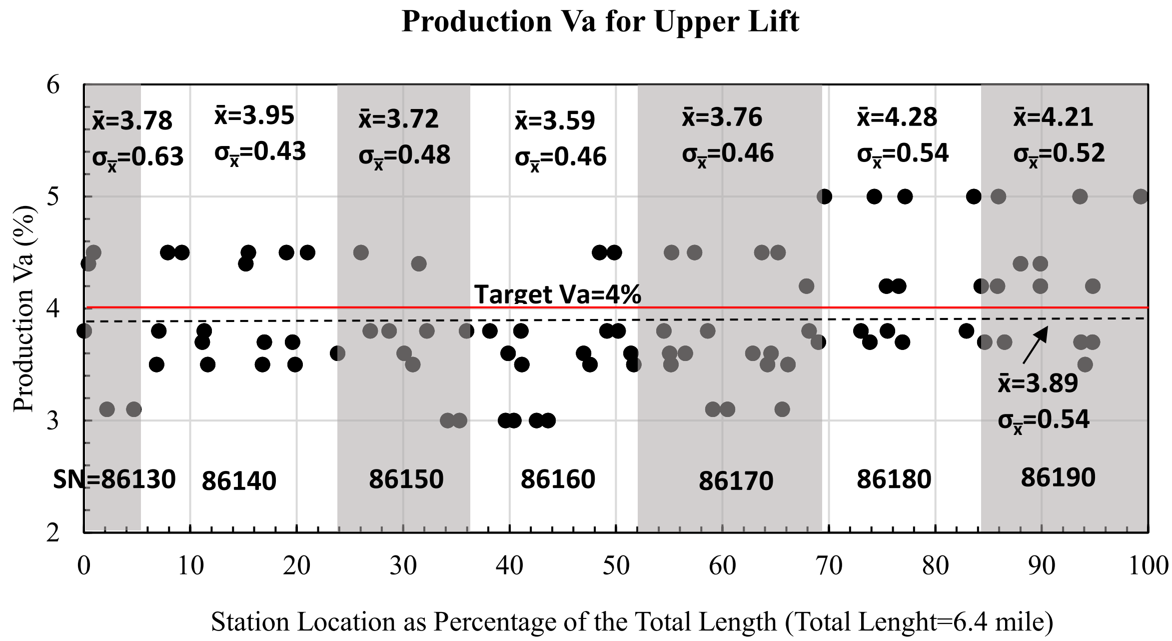


Figure 4-56 Production information of air voids (Va %) based on station locations (project STH 64)

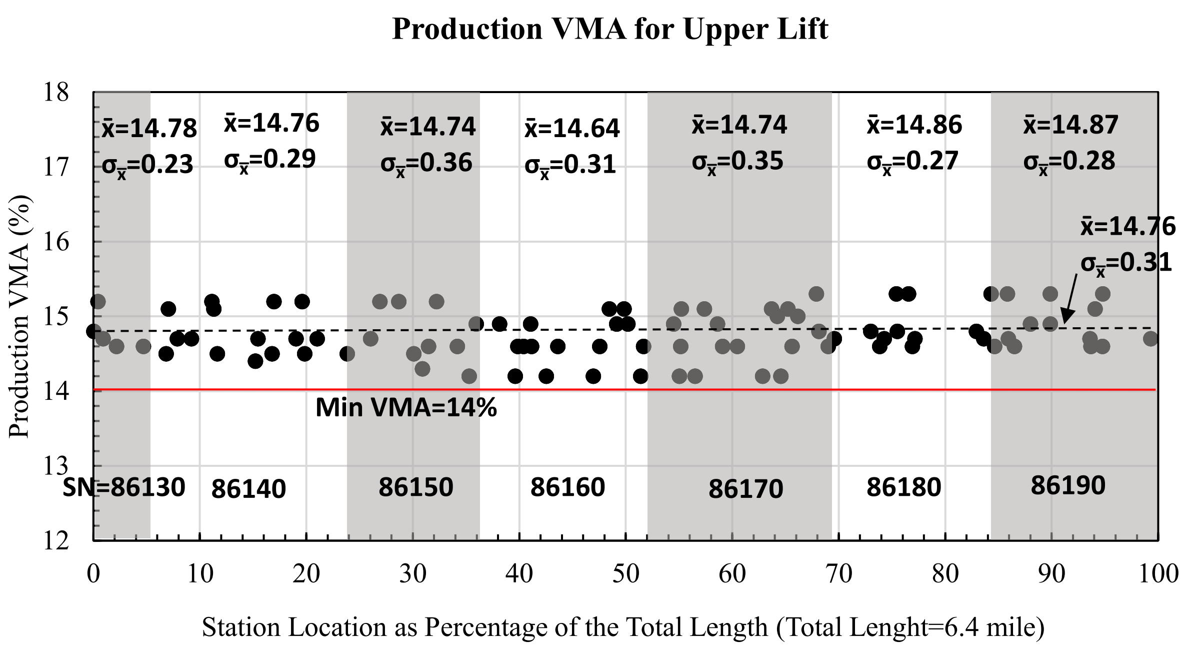


Figure 4-57 Production information of voids in mineral aggregates (VMA %) based on station locations (project STH 64)

The mix VMA is showing a lower level of variability and more conformity compared to Va (Figure 4-57). There is no apparent association between the VMA of this mix and the noted performance.

Placement Quality

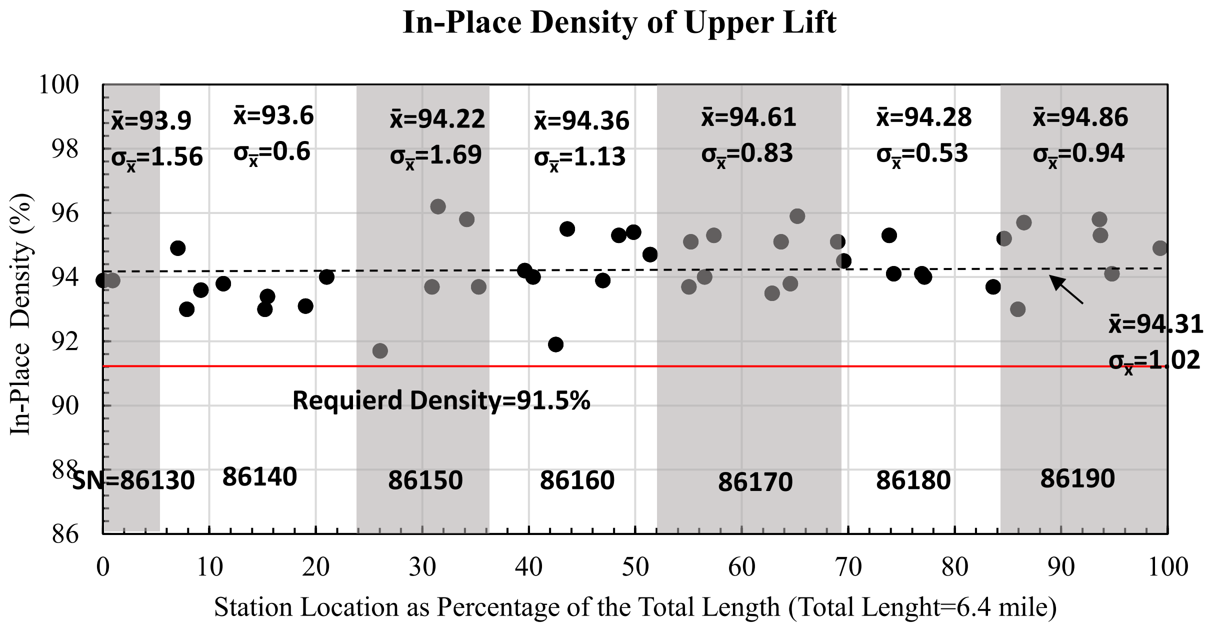


Figure 4-58 Construction information of in-place density (Gmm %) based on station locations (project STH 64)

The placement density is showing full conformity, SNs showing the higher level of distresses (SN 86130 & 86150) are compacted to more than 2% Gmm above the minimum required, yet they experienced significant levels of deterioration. This project continues a trend where field compaction cannot improve soft mixes. The data validates the continuous need to monitor quality during production and construction. However, the quality measures used currently still do not provide clear-cut thresholds for distinguishing good from poor quality.

STH 64 Summary:

1. Low Va are associated with high deterioration.
2. The only alligator cracking SN in the onsite survey is associated with the lowest level of Va in the entire project. The same SN show placement quality record including the most under compacted test point (less than 92%Gmm), and the most compacted test point (more than 96%Gmm).
3. FWD D_0 are highest at the top-distressed sections. Yet the values of the D_0 are within typical well-performing structures. The modulus values for the base and subgrade for these sections are also at adequate levels. This indicates that the surface layer is soft.

4. SN 86180 and SN 86190 consistently show that they were constructed with the stable mix as indicated by the production quality results. The placement quality indicates the adequate level of compaction.

4.8 Project STH 55 (#9660-01-60)

Construction year: 2010

Design Traffic: 503,700 ESALS

Asphalt Binder Content: 5.5%

Asphalt Layer Thickness: 2” + (4” existing)

SN	76950	76960	76970	76980	76990	77000	77010	77020	77030	77040	77050	77060	77070	77080
PCI	80	84	64	65	61	75	66	79	72	75	68	69	76	77

The state trunk highway 55 is located in the US state of Wisconsin. This highway travels south to north in the northeastern part of Wisconsin from the Route 151, located at the north of Bothertown, to the Michigan state line at the Brule River. The total length of the STH 55 is 175.5 miles. The selected project is a part of the entire highway and is located in the Menominee County, with the length of 18.56 miles. The project goes along with the Wolf River from Shawano to Langlade Counties. The project constructed in 2010 by using 33,858 tons of HMA. The project has totally 14 Sequence Numbers. The location of the project in Wisconsin State is shown in Figure 4-59.

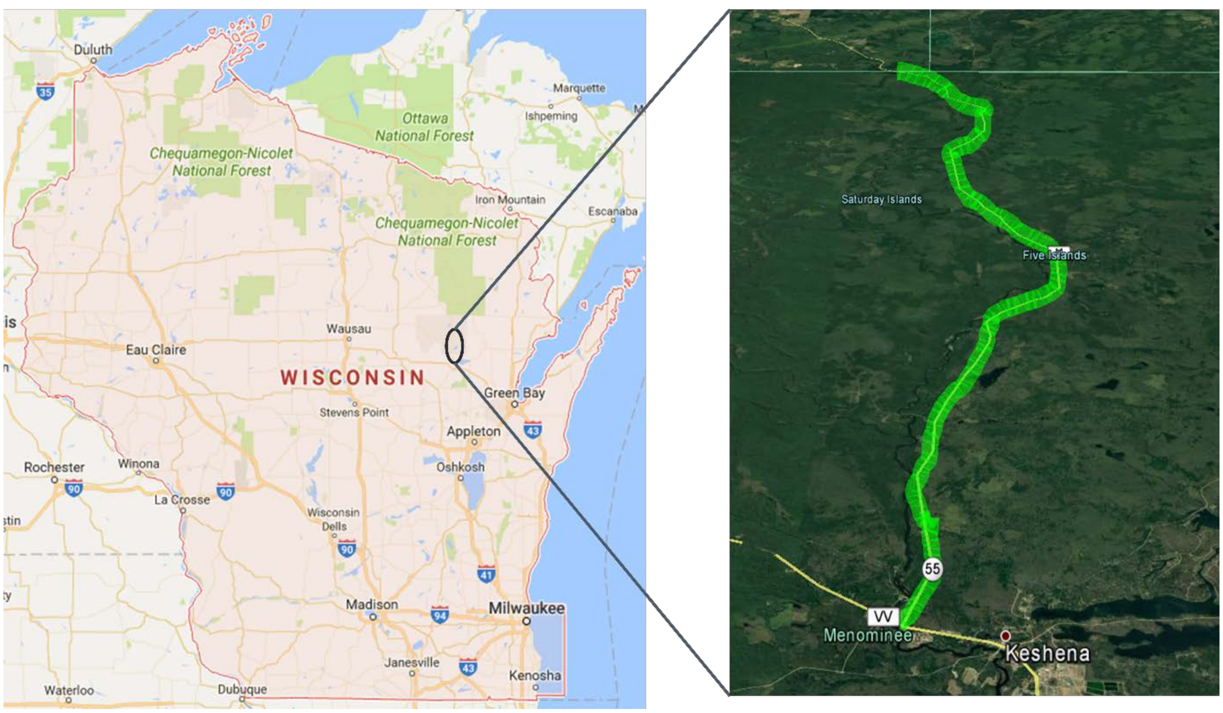


Figure 4-59 Highway STH 55, project 9660-01-60, Shawano-Langlade, Menominee County

In-Service Performance

The quality of the performance is shown in Figure 4-60. As is shown, the DI is dominated by longitudinal cracking. All SN experience minor transverse cracking. Alligator cracking is noticed in 12 out of the 14 SNs, and rutting is noticed in 5 SNs. SN 77010 experience the most level of deterioration caused mostly by longitudinal cracking.

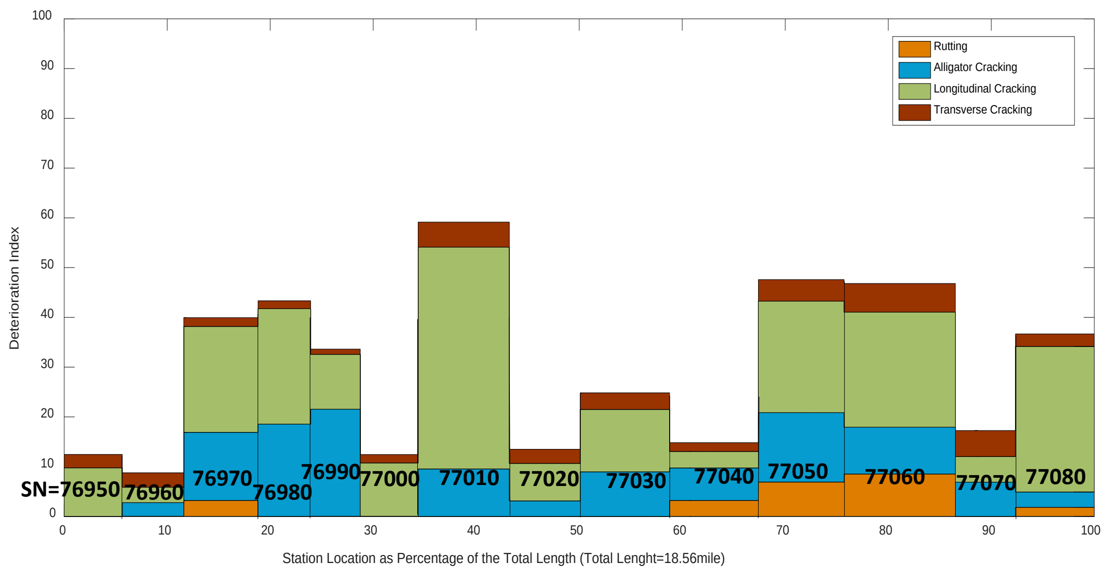


Figure 4-60 Performance of the project STH 55 (based on DOT information) (2016)

Figure 4-61 shows the results of the onsite distress survey and FWD testing. The dominating distress is alligator cracking. This contradicts the PIF results. The reason could be that the PIF is calculated mostly from videos of the surface, with the onsite survey is taken from actual measurement. Thus, what looks like longitudinal cracking on the screen, could be categorized as an alligator on site.

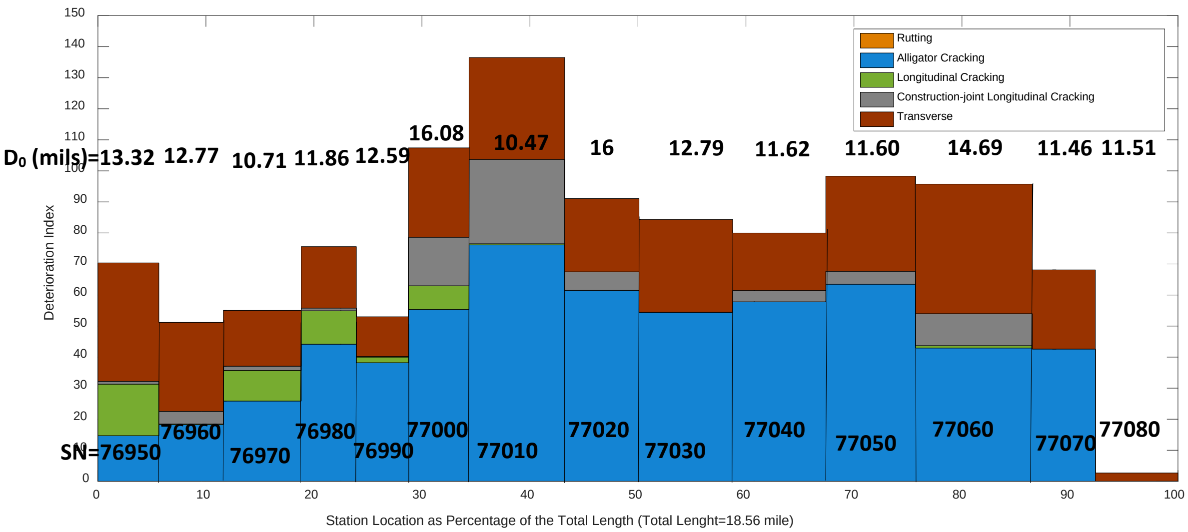


Figure 4-61 Performance of the project STH 55 (based on research group survey) (2017)

Figure 4-61 also shows the D_0 values. The values are showing are mostly within the moderate range for the D_0. This indicates a potential interaction between structural stability and materials. Therefore, the modulus for the pavement layers are calculated and shown for all SN in the following figure. With respect to SN 77010, the most distressed in the project, a plot of the individual FWD tests conducted is shown in Figure 4-63 with the distresses DI overlapped.

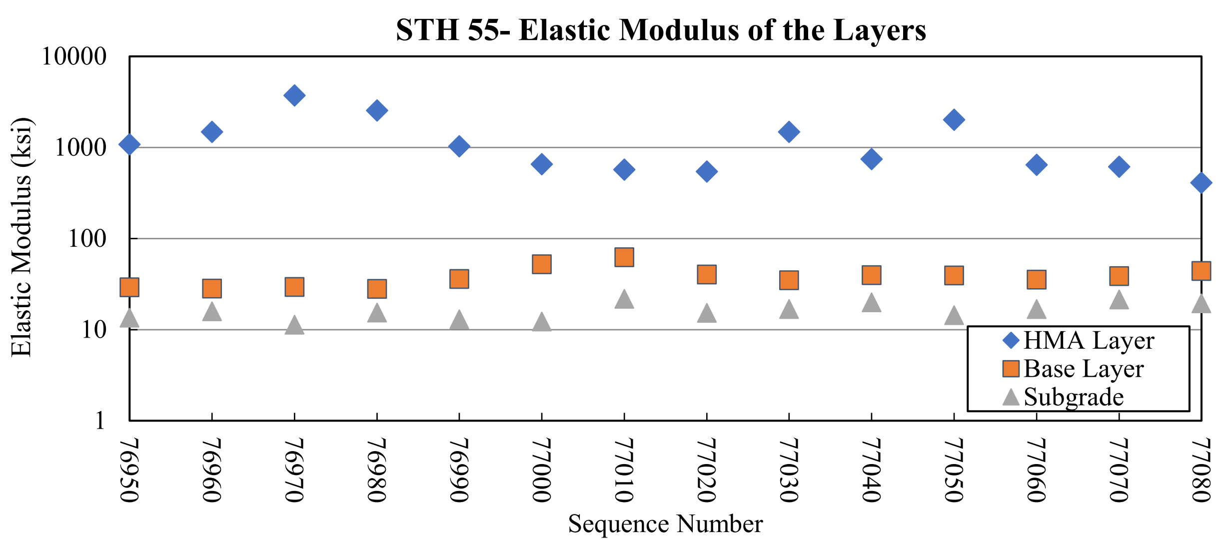
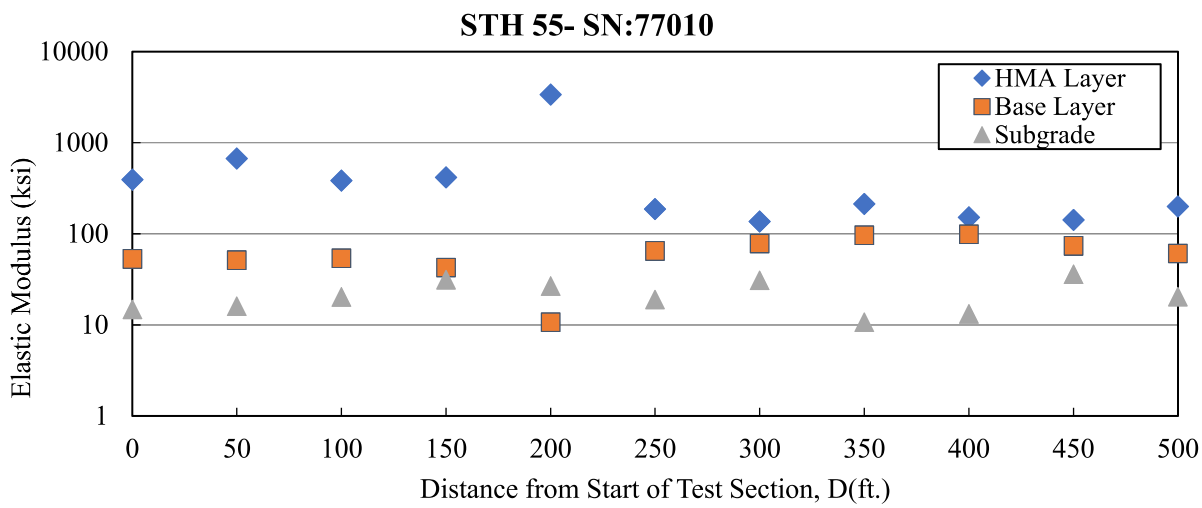


Figure 4-62 Average moduli of the layers for project STH 55

Figure 4-62 shows the distribution of the average moduli for all the SNs included in the project. For each of the SNs, a 500ft section was tested with FWD at 50 ft. spacing. It is clear that the sections with the lowest level of surface modulus are the ones with the highest level of DI.



From (ft.)	0	50	100	150	200	250	300	350	400	450
To (ft.)	50	100	150	200	250	300	350	400	450	500
D0 (mils)	14.04	11.59	9.42	6.83	7.35	14.1	7.87	15.09	12.61	6.45
Alligator Cracking (DI)	75.34	85.21	64.39	49.09	67.92	68.97	50.75	50.64	65.89	36.85
Longitudinal Cracking (DI)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	4.21	0.00	0.00
Construction-joint Longitudinal Cracking (DI)	12.62	12.15	5.06	3.33	0.00	28.88	56.44	26.41	14.05	51.90
Transverse Cracking (DI)	38.05	40.36	24.25	29.06	38.50	24.25	33.48	29.06	24.25	24.25

Figure 4-63 E modulus of subgrade for SN=77010

Based on the information above, pavement structure appears to be one of the influencing factors. The following sections discuss the quality measures for this project.

Production Quality

Production data for this project is not complete. The Va and VMA data is only available for a subset of the SNs.

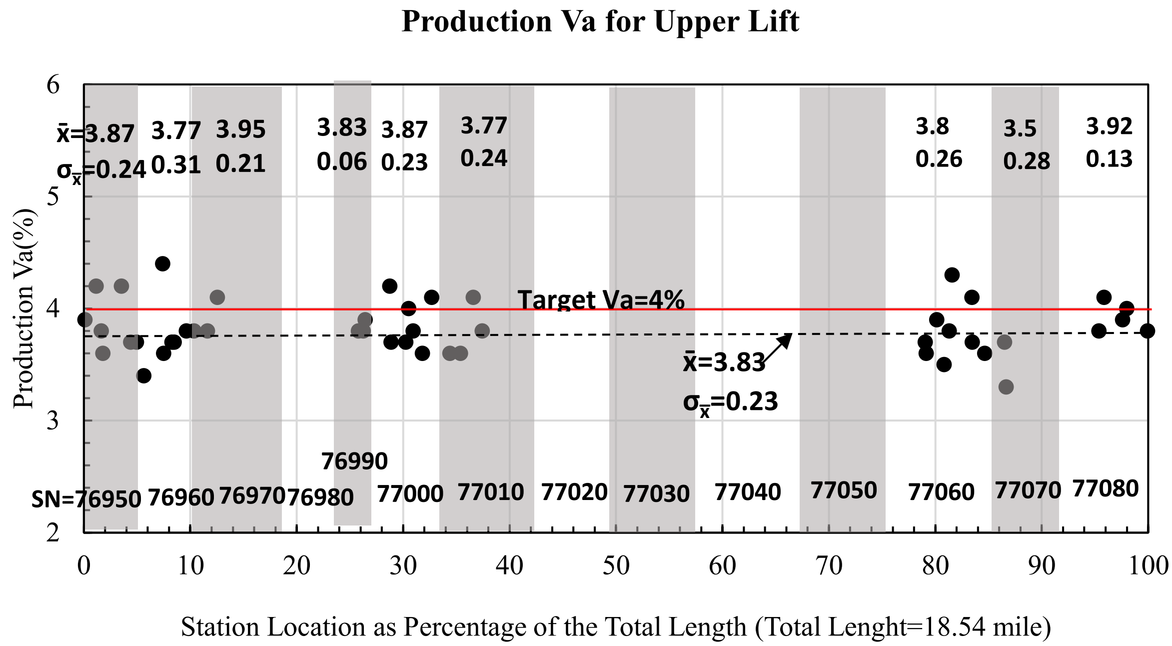


Figure 4-64 Production information of air voids (Va %) based on station locations (project STH 55)

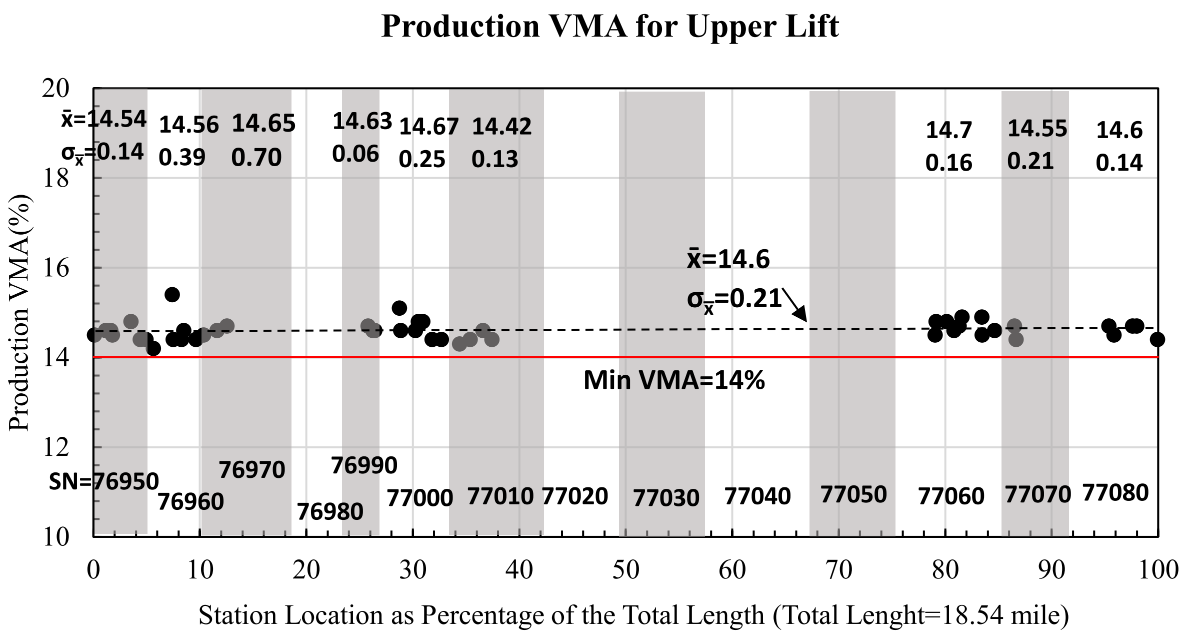


Figure 4-65 Production information of voids in mineral aggregates (VMA %) based on station locations (project STH 55)

The available results from the production quality data show that the Va values are not conforming on average, but the VMA values are on average above the minimum required. Low Va indicates a soft mix which is associated with increased distresses, especially that this mix is used at 2-inch thickness.

Placement Quality

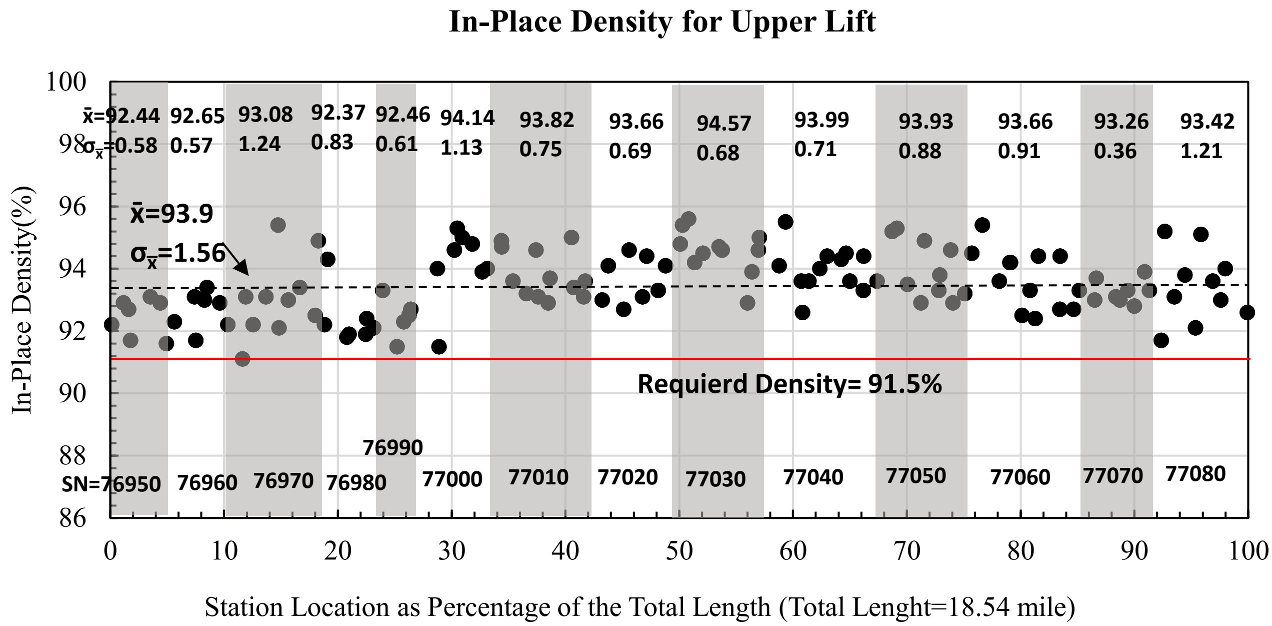


Figure 4-66 Construction information of in-place density (Gmm %) based on station locations (project STH 55)

Placement density shown in Figure 4-66 represents a full record of the %Gmm. The average density for the project is 93.9, which is 1.4% above the minimum required for all individual tests meeting or exceeding requirements. SN 77010 shows consistent densification values with the maximum value of about 95%Gmm (3.5% above the minimum required). For a soft mix (as shown by the Va and D_0) and thin lift of 2 inches over existing, this level of compaction may be excessive. This trend follows that noticed for STH 64, STH 75, and STH 13.

STH 55 Summary

1. The available production data is inconclusive.
2. FWD results for the modulus values show that the sections with the highest level of deterioration are the ones with the lowest modulus values.
3. Placement compaction is in some cases 4% above the minimum required at 95.5%. This is a 2” layer paved over an exititing compacted surface.
4. The same sections that are over compacted are showing the lower surface modulus. These sections have the highest DI.

4.9 Project Level Analysis Summary

The following represent summaries and recommendations based on the observations made during the project level analysis.

1- Thin lifts compacted at more than 95% are showing a higher level of deterioration.

2- Normal thickness lifts show no improvement of performance for compaction beyond 96%.

3- According to the observation, Low Va during projection in some cases lead to high early deterioration.

4- Combination of high in-field compaction with a soft mix of low Va is the most deteriorated.

5- There appears to be an optimum level of field compaction that correlates with minimal deterioration. For thin lifts, this range is from 92 to 94%, and for typical thickness, it is from 93 to 95%.

6- Performance does not show sensitivity to mixture production variability. However, placement variability shows some evidence.

7- Structural stability cannot be fixed without proper pavement mix quality.

5. Summary and Conclusions

Summary of Findings

This project involved handling a significant volume of data for a large number of projects. The goal of utilizing this data is to understand the relationship of the quality indicators with in-service performance. The analysis approach allows for developing a strategy to improve the durability of the Wisconsin roadway network and to upgrade data collection and storage methods. The following statements summarize the results of the work completed for this project:

- The analysis included in the project could not be conducted without the development of the relational database. Implementing this method on the network level for all DOT data will yield significant benefits such as defining effective tolerance limits for quality indicators or associate trends in in-service performance to appropriate design inputs or material quality indicators. The work completed in this study is intended to provide the DOT a path to not only evaluate the effectiveness of the quality data but also, a systematic approach for extracting valuable trends and correlations to manage the department resources better while producing durable pavements. This was demonstrated by the ability to connect deviations in construction quality to in-service performance. This approach was also able to extract trends in distress evolution and distribution within the database.
- It is recommended that distress survey and recording is revised. Given the advancement of technology, many of the distresses can be captured using advanced image analysis software, or a mix of human and software analysis. This will result in increasing the volume of sections reviewed. The current approach of surveying only about 10% of the total surface using video recording can result in misleading conclusions.
- As shown in chapter 4, the discrepancy in attributing distress levels of severity is observed between PIF and on-site distress surveys. It is understandable that on-site survey is not practical for the entire roadway network. However, there is room for improving the pavement condition assessment. Advancements in imaging and sensing could result in minimizing such discrepancy. In addition, the distress recording could benefit from revisiting the current definitions for some of the common distresses. For example, combining construction joint longitudinal cracks with in-line longitudinal cracks in one category limits the effectiveness of the PIF in tracking damage evolution as well as connecting this damage to potential causes.

- With respect to the evaluated distresses in this study, for pavements of ages 7-8 years, it is found that alligator cracking is recorded in 15%, rutting is recorded in 33%, longitudinal cracking in 62%, and transverse cracking in 71% of the sections collected to form the database. The corresponding deterioration index calculated for these distress distributions is at maximum for longitudinal cracking followed by alligator cracking, rutting, and then transverse cracking.
- The extent of damage is evidently not a function of the abundance. For example, transverse cracking is observed in the majority of roadway section, but the associated DI value is the least among the four distresses studied in this research.

Conclusions and Recommendations

- Given the focus of this research study, it is understandable that the findings are confined by the dataset available for analysis. This research was not intended to develop deterioration models of the Wisconsin roadways, but rather explore potential correlations between quality indicators and in-service performance. The quality in this analysis is defined in terms of deviation from the targeted specification limits. Therefore, no attempt was made to evaluate the effectiveness of the specification limits, but rather the association of deviation from the limits to performance.
- Further research is obviously needed to establish such deterioration models. The authors would like to draw attention to the need for upgrading the current specifications to encompass performance related QMP for better association with in-service performance.
- The following conclusions are developed to address the presence of distresses from a quality control point of view:
    1. Rutting: this distress appears to be sensitive to field compaction followed by mix air content at design number of gyrations (Va). Exceeding the current minimum required field compaction density, but up to 2% correlates with reduced rutting levels. The trend is only observed for lifts thicker than 2”. For thin lifts, over compaction is associated with increased cracking potential. This is demonstrated by the analysis of the database and confirmed by the on-site surveys.
        - Correlation between rutting and the target Va is showing sensitivity the direction of deviation from the target. Meeting the target Va or exceeding by up to 1% correlate with reduced rutting levels, on the other hand, when the

deviation from the target drops below 0.25% rutting rate increases. This is demonstrated by the analysis of the database and confirmed by the on-site surveys. This means that variation during production phase is critical when they lead to ease of densification under the same level of compaction energy.

  - The above points need to be further qualified after incorporating the asphalt content of the lots. The deviation in production may be due to fluctuation in the lot AC effective content. In this case, the trend and correlation between the Va and performance are logical. Therefore, not tracking the effective asphalt as a quality measure within Wisconsin DOT is highly detrimental to any further analysis.

2. Cracking: while the statistical analysis showed good correlations for alligator cracking only, the project by project analysis revealed that transverse and in-line longitudinal cracking are also sensitive to the same quality indicators. Mix deviation from the target Va and VMA appear to play a significant role in improving quality, with minimal sensitivity to field compaction. Similar to rutting, changes in production leading to drop in Va by more than 0.25% increased the risk of cracking. Exceeding the VMA limit by 1% correlated with the significant reduction in cracking. Fluctuation of both Va and VMA for the same mix design could be a function of variation in the effective asphalt binder or changes in the aggregate structure. The current system of quality management does not track AC or the aggregate structure. Therefore, the effectiveness of the quality program can be greatly enhanced by exploring the relation of these parameters to performance.

   - Field over-compaction of pavement, especially for thin lifts thinner than 2" over the stiff base can lead to the presence of all four types of distresses.

3. Variability in quality indicators values, as measured by standard deviation, does not appear to correspond to any of the distresses studied. Only when the variability of quality indicators in the test point's results in mixes to fall within the ranges discussed in the previous points, distresses may occur.

4. The current trend in adopting performance related mix design appears to be the right path. As can be seen from the above points, relying on the volumetrics alone does not yield clear recommendations as to how to improve pavement mixes. Consequently, the

influence of the effective AC content is necessary for establishing such design protocol. It is also, necessary to understand the interactive effect of the volumetric properties including the AC content and the pavement performance. The bottom line is that the volumetrics properties are used to represent the ability of the mix (AC and aggregate structure) to perform during the service life. However, the department does not track either AC or aggregate gradation during production.

- Given the analysis in this study and the findings of the literature and States survey, it appears that the most important components of a quality program are determining the frequency of testing to have representative sampling and setting the right quality limits. In addition, the current properties measured are adequate under the current mix design protocol. However, tolerance limits may need revisions.
- The research approach used in this study provides the foundation for the DOT to build its relational database of the existing pavement inventory. To build such database, the researchers recommend the following steps:
    1. Digitization of all mix production quality data.
    2. Require the contractors to identify the GPS coordinates (location) for the placement of lots produced.
    3. Integrate project plans with digital mapping system to easily convert project stations into GPS coordinates.
    4. Define sequence numbers using GPS coordinates to be integrated within the mapping system.
    5. House construction and production data in electronic storage in the form of georeferenced databases.
    6. Build user-friendly interface for quarrying data flexibly according to the needs of the DOT.
    7. Collect additional data with regards to mix effective asphalt content.
    8. Define distresses in more focused terms including splitting current categories into more subdivisions that can be tracked for future analysis. These subdivisions include separating longitudinal cracks from construction joints, to between wheel path, and on wheel path. For Transverse cracking, to be separated into reflective and load related (thermal loading for example) cracking.

9. Finally, it is highly recommended that the further data collection from the visited sites is conducted. The further data should include taking cores for calculating the asphalt binder content and evaluating aggregate gradation for each of the lots produced. This will allow for this study to close the loop on the relationship between quality measures and in-service performance.

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7. Appendix I

Section 460 Hot Mix Asphalt Pavement

460.1 Description

(1) This section describes HMA mixture design, providing and maintaining a quality management program for HMA mixtures, and constructing HMA pavement. Unless specifically indicated otherwise, references within 460 to HMA also apply to WMA.

460.2 Materials

460.2.1 General

(1) Furnish a homogeneous mixture of coarse aggregate, fine aggregate, mineral filler if required, SMA stabilizer if required, recycled material if used, warm mix asphalt additive or process if used, and asphaltic material.

460.2.2 Aggregates

460.2.2.1 General.

(1) Provide coarse aggregates from a department-approved source as specified under 106.3.4.2. Obtain the engineer's approval of the aggregates before producing HMA mixtures.

(2) Furnish an aggregate blend consisting of hard durable particles containing no more than a combined total of one percent, by weight, of lumps of clay, loam, shale, soft particles, organic matter, adherent coatings, and other deleterious material. Ensure that the aggregate blend conforms to the percent fractured faces and flat & elongated requirements of table 460-2. If the aggregate blend contains materials from different deposits or sources, ensure that material from each deposit or source has an LA wear percent loss meeting the requirements of table 460-2.

460.2.2.2 Freeze-Thaw Soundness

(1) If the aggregate blend contains materials from different deposits or sources, ensure that material from each deposit or source has a freeze-thaw loss percentage meeting the requirements of table 460-2 and 106.3.4.2.2.

460.2.2.3 Aggregate Gradation Master Range

Revise 460.2.2.3(1) table 460-1 to include gradation numbers and traffic levels used for the new combined bid items. This revision incorporates STSP 460-025 HMA Pavement (gradation)(traffic)(binder)(designation).

(1) Ensure that the aggregate blend, including recycled material and mineral filler, conforms to the gradation requirements in table 460-1. The values listed are design limits; production values may exceed those limits.

TABLE 460-1 AGGREGATE GRADATION MASTER RANGE AND VMA REQUIREMENTS

SIEVE	PERCENT PASSING DESIGNATED SIEVES						
	NOMINAL SIZE						
	No. 1 (37.5 mm)	No. 2 (25.0 mm)	No.3 (19.0 mm)	No. 4 (12.5 mm)	No. 5 (9.5 mm)	SMA No. 4 (12.5 mm)	SMA No. 5 (9.5 mm)
50.0-mm	100						
37.5-mm	90 –100	100					
25.0-mm	90 max	90 -100	100				
19.0-mm	___	90 max	90 -100	100		100	
12.5-mm	___	___	90 max	90 -100	100	90 - 97	100
9.5-mm	___	___	___	90 max	90 -100	58 - 72	90 - 100
4.75-mm	___	___	___	___	90 max	25 - 35	35 - 45
2.36-mm	15 – 41	19 - 45	23 - 49	28 - 58	32 - 67	15 - 25	18 - 28
75-µm	0 – 6.0	1.0 - 7.0	2.0 - 8.0	2.0 - 10.0	2.0 - 10.0	8.0 - 12.0	10.0 - 14.0
% MINIMUM VMA	11.0	12.0	13.0	14.0[1]	15.0[2]	16.0	17.0

[1] 14.5 for LT and MT mixes.

[2] 15.5 for LT and MT mixes.

460.2.3 Asphaltic Binders

(1) The department will designate the grade of asphaltic binder in the HMA Pavement bid item. Use the binder grade the bid item specifies. Do not change the PG binder grade without the engineer's written approval. The department will designate the grade of virgin asphaltic binder in the contract, however, the contractor may use virgin binder, modified binder, a blend of virgin and recovered binder, or a blend of

modified and recovered binder. When the percent asphalt binder replaced exceeds the allowable limits in 460.2.5, provide test results from extracted and recovered binder to ensure that the resultant asphaltic binder conforms to the contract specifications.

460.2.4 Additives

460.2.4.1 Hydrated Lime Antistripping Agent

(1) If used in HMA mixtures, furnish hydrated lime conforming to ASTM C977 and containing no more than 8 percent unhydrated oxides. Percent added is by weight of the total dry aggregate.

460.2.4.2 Liquid Antistripping Agent

(1) If used in HMA mixtures, add liquid antistripping agent to the asphaltic binder before introducing the binder into the mixture. Provide documentation indicating that addition of liquid antistripping agent will not alter the characteristics of the original asphaltic binder performance grade (PG).

460.2.4.3 Stone Matrix Asphalt Stabilizer

(1) Add an organic fiber, an inorganic fiber, a polymer-plastic, a polymer-elastomer, or approved alternate stabilizer to all SMA mixtures. If proposing an alternate, submit the proposed additive system, asphaltic binder, and stabilizer additive, along with samples of the other mixture materials to the department at least 14 days before the project let date. The department will approve or reject that proposed alternate additive system no later than 48 hours before the project let date.

(2) Use a single additive system for all SMA pavement in the contract.

460.2.4.4 Warm Mix Asphalt Additive or Process

(1) Use additives or processes from the department's approved products list. Follow supplier or manufacturer recommendations for additives and processes when producing WMA mixtures.

460.2.5 Recycled Asphaltic Materials

(1) The contractor may use recycled asphaltic materials from FRAP, RAP, and RAS in HMA mixtures. Stockpile recycled materials separately from virgin materials and list each as individual JMF components.

(2) Control recycled materials used in HMA by evaluating the percent binder replacement, the ratio of recovered binder to the total binder. Conform to the following:

MAXIMUM ALLOWABLE PERCENT BINDER REPLACEMENT

RECYCLED ASPHALTIC MATERIAL	LOWER LAYERS	UPPER LAYER
RAS if used alone	25	20
RAP and FRAP in any combination	40	25
RAS, RAP, and FRAP in combination [1]	35	25

[1] When used in combination the RAS component cannot exceed 5 percent of the total weight of the aggregate blend.

460.2.6 Recovered Asphaltic Binders

(1) Establish the percent of recovered asphaltic binder from FRAP, RAP, and RAS for the mixture design according to AASHTO T164 using the appropriate dust correction procedure. If production test results indicate a change in the percent of recovered asphaltic binder, the contractor or the engineer may request a change in the design recovered asphaltic binder. Provide the department with at least 2 recent extraction samples supporting that change. Ensure that those samples were prepared according to CMM 8-65 by a WisDOT qualified laboratory.

(2) The contractor may replace virgin binder with recovered binder up to the maximum percentage allowed under 460.2.5 without changing the asphaltic binder grade. If using more than the maximum allowed under 460.2.5, furnish test results indicating that the resultant binder meets the grade the contract originally specified.

460.2.7 HMA Mixture Design

(1) For each HMA mixture type used under the contract, develop and submit an asphaltic mixture design according to CMM 8-66 and conforming to the requirements of table 460-1 and table 460-2. The values listed are design limits; production values may exceed those limits. The department will review mixture designs and report the results of that review to the designer according to CMM 8-66.

Revise 460.2.7(1) table 460-2 to switch from E mixes to LT, MT, and HT mixes; and change tensile strength ratios. This revision incorporates STSP 460-025 HMA Pavement (gradation)(traffic)(binder)(designation).

TABLE 460-2 MIXTURE REQUIREMENTS

Mixture type	LT	MT	HT	SMA
ESALs x 10^6 (20 yr design life)	<2.0	2 - <8	>8	> 5 mil
LA Wear (AASHTO T96) 100 revolutions (max % loss) 500 revolutions (max % loss)	 13 50	 13 45	 13 45	 13 40
Soundness (AASHTO T104) (sodium sulfate, max % loss)	12	12	12	12
Freeze/Thaw (AASHTO T103) (specified counties, max % loss)	18	18	18	18
Fractured Faces (ASTM D5821) (one face/2 face, % by count)	65/	75 / 60	98 / 90	100/90
Flat & Elongated (ASTM D4791) (max %, by weight)	5 (5:1 ratio)	5 (5:1 ratio)	5 (5:1 ratio)	20 (3:1 ratio)
Fine Aggregate Angularity (AASHTO T304, method A, min)	40	43	45	45
Sand Equivalency (AASHTO T176, min)	40	40	45	50
Gyratory Compaction Gyrations for N_{ini} Gyrations for N_{des} Gyrations for N_{max}	 6 40 60	 7 75 115	 8 100 160	 8 65 160
Air Voids, $\%V_a$ ($\%G_{mm}$ N_{des})	4.0 (96.0)	4.0 (96.0)	4.0 (96.0)	4.0 (96.0)
% G_{mm} N_{ini}	<= 91.5[1]	<= 89.0[1]	<= 89.0	
% G_{mm} N_{max}	<= 98.0	<= 98.0	<= 98.0	
Dust to Binder Ratio [2] (% passing 0.075/P_{be})	0.6 - 1.2	0.6 - 1.2	0.6 - 1.2	1.2 - 2.0
Voids filled with Binder (VFB or VFA, %)	68 - 80[4] [5]	65 – 75[3] [5]	65 - 75[3] [5]	70 - 80
Tensile Strength Ratio (TSR) (AASHTO T283) no antistripping additive with antistripping additive	 0.75 0.80	 0.75 0.80	 0.75 0.80	 0.75 0.80
Draindown (AASHTO T305) (%)	____	____	____	0.30

[1] The percent maximum density at initial compaction is only a guideline.

[2] For a gradation that passes below the boundaries of the caution zone (ref. AASHTO M323), the dust to binder ratio limits are 0.6 - 1.6.

[3] For No. 5 (9.5mm) and No. 4 (12.5 mm) nominal maximum size mixtures, the specified VFB range is 70 - 76%.

[4] For No. 2 (25.0mm) nominal maximum size mixes, the specified VFB lower limit is 67%.

[5] For No. 1 (37.5mm) nominal maximum size mixes, the specified VFB lower limit is 67%.

460.2.8 Quality Management Program

460.2.8.1 General

(1) Provide and maintain a QC program defined as all activities, including mix design, process control inspection, sampling and testing, and process adjustments related to producing and placing HMA pavement conforming to the specifications.

(2) The department will provide product quality verification as follows:

1. By conducting verification testing of independent samples.
2. By periodically observing contractor sampling and testing.
3. By monitoring required control charts exhibiting test results and control parameters.
4. By the engineer directing the contractor to take additional samples at any time during production.

(3) Refer to CMM 8-36 for detailed guidance on sampling, testing, and documentation under the QMP.

460.2.8.2 Contractor Testing

460.2.8.2.1 Required Quality Control Program

460.2.8.2.1.1 Personnel Requirements

(1) Provide HTCP-certified sampling and testing personnel. Provide at least one full-time HMA technician certified at a level appropriate for sampling and production control testing at each plant site furnishing material to the project. Before mixture production begins, provide an organizational chart in the contractor's laboratory. Include the names, telephone numbers, and current certifications of personnel with QC responsibilities. Keep the chart updated.

(2) Ensure that sampling and testing personnel are minimally qualified as follows [1]:

- HMA technician certified at a level appropriate for sampling and production control testing.
- HMA ACT [2].

[1] After informing the engineer, a non-certified person under the direct observation of a certified HMA technician may sample for a period not to exceed 3 calendar days.

[2] A certified HMA technician must coordinate and take responsibility for the work an ACT performs. No more than one ACT can work under a single certified technician.

(3) Have a certified HMA technician ensure that sampling and testing is performed correctly, analyze test results, and post resulting data.

(4) Have an HMA technician certified at a level appropriate for process control and troubleshooting or mix design available to make necessary process adjustments.

460.2.8.2.1.2 Laboratory Requirements

(1) Conduct QC testing in a facility conforming to the department's laboratory qualification program.

(2) Ensure that testing equipment conforms to the equipment specifications applicable to the required testing methods.

460.2.8.2.1.3 Required Sampling and Testing

460.2.8.2.1.3.1 Contracts with 5000 Tons of Mixture or Greater

(1) Furnish and maintain a laboratory at the plant site fully equipped for performing contractor QC testing. Have the laboratory on-site and operational before beginning mixture production.

(2) Obtain random samples and perform tests according to CMM 8-36. Obtain HMA mixture samples from trucks at the plant. Perform tests the same day taking the sample.

(3) Retain the split portion of the contractor HMA mixture and blended aggregate samples for 14 calendar days at the laboratory site in a dry, protected area. The engineer may decrease this 14-day retention period. At project completion the contractor may dispose of remaining samples if the engineer approves.

(4) Use the test methods identified below, or other methods the engineer approves, to perform the following tests at a frequency greater than or equal to that indicated:

Blended aggregate gradations: Drum plants:

- Field extraction by CMM 8-36.
- Belt samples, optional for virgin mixtures, obtained from stopped belt or from the belt discharge using an engineer-approved sampling device and performed according to AASHTO T11 and T27.

Batch plants:

- Field extraction by CMM 8-36.

Asphalt content (AC) in percent: AC by

calculation.

AC by nuclear gauge reading, optional. AC by inventory,

optional.

Bulk specific gravity of the compacted mixture according to AASHTO T166. Maximum

specific gravity according to AASHTO T209.

Air voids (V_a) by calculation according to AASHTO T269. VMA by

calculation according to AASHTO R35.

(5) Test each design mixture at a frequency at or above the following:

TOTAL DAILY PLANT PRODUCTION

FOR DEPARTMENT CONTRACTS in tons	SAMPLES PER DAY [1]
50 to 600	1
601 to 1500	2
1501 to 2700	3
2701 to 4200	4
greater than 4200	see footnote [2]

[1] Frequencies are for planned production. If production is other than planned, conform to CMM 8-36.

[2] Add a random sample for each additional 1500 tons or fraction of 1500 tons.

(6) Also conduct field tensile strength ratio tests according to ASTM D4867 on mixtures requiring an antistripping additive. Test each full 50,000 ton production increment, or fraction of an increment, after the first 5000 tons of production. Perform required increment testing in the first week of production of that increment. If field tensile strength ratio values are either below the spec limit or less than the mixture design JMF percentage value by 20 or more, notify the engineer. The engineer and contractor will jointly determine a corrective action.

460.2.8.2.1.3.2 Contracts with Less Than 5000 Tons of Mixture

(1) Conform to 460.2.8.2.1.3.1 modified as follows:

- The contractor may conduct QC tests in an off-site laboratory.
- No field tensile strength ratio testing is required.

460.2.8.2.1.3.3 Contracts with Less Than 500 Tons of Mixture

(1) The engineer may waive QC testing on contracts with less than 500 tons of mixture. If testing is waived, acceptance will be by visual inspection unless defined otherwise by contract change order.

(2) If HMA density testing is waived under 460.3.3.3, QC testing is also waived.

460.2.8.2.1.3.4 Temporary Pavements

(1) The engineer may waive all testing for temporary pavements, defined for this purpose as pavements that will be placed and removed before contract completion.

460.2.8.2.1.4 Documentation

460.2.8.2.1.4.1 Records

(1) Document observations, inspection records, mixture adjustments, and test results daily. Note observations and inspection records in a permanent field record as they occur. Record process adjustments and JMF changes. Submit copies of the running average calculation sheets for blended aggregate, mixture properties, and asphalt content along with mixture adjustment records to the engineer each day. Submit testing records and control charts to the engineer in a neat and orderly manner within 10 days after paving is completed.

(2) Continue charts, records, and testing frequencies, for a mixture produced at one plant site, from contract to contract.

460.2.8.2.1.4.2 Control Charts

(1) Maintain standardized control charts at the laboratory. Record contractor test results on the charts the same day as testing. Record data on the standardized control charts as follows:

- Blended aggregate gradation tests in percent passing. Of the following, plot those sieves the design specifications require: 37.5-mm, 25.0-mm, 19.0-mm, 12.5-mm, 9.5-mm, 2.36-mm, and 75-µm.
- Asphalt material content in percent.
- Air voids in percent.
- VMA in percent.

(2) Plot both the individual test point and the running average of the last 4 data points on each chart. Show QC data in black with the running average in red. Draw the warning limits with a dashed green line and the JMF limits with a dashed red line. The contractor may use computer generated black-and-white printouts with a legend that clearly identifies the specified color coded components.

460.2.8.2.1.5 Control Limits

(1) Conform to the following control limits for the JMF and warning limits based on a running average of the last 4 data points:

ITEM	JMF LIMITS	WARNING LIMITS
Percent passing given sieve:		
37.5-mm	+/- 6.0	+/- 4.5

25.0-mm	+/- 6.0	+/- 4.5
19.0-mm	+/- 5.5	+/- 4.0
12.5-mm	+/- 5.5	+/- 4.0
9.5-mm	+/- 5.5	+/- 4.0
2.36-mm	+/- 5.0	+/- 4.0
75-µm	+/- 2.0	+/- 1.5
Asphaltic content in percent	- 0.3	- 0.2
Air voids in percent	+/- 1.3	+/- 1.0
VMA in percent [1]	- 0.5	- 0.2

[1] VMA limits based on minimum requirement for mix design nominal maximum aggregate size in table 460-1.

(2) Warning bands are defined as the area between the JMF limits and the warning limits.

460.2.8.2.1.6 Job Mix Formula Adjustment

(1) The contractor may request adjustment of the JMF according to CMM 8-66.2. Have an HMA technician certified at a level appropriate for process control and troubleshooting or mix design submit a written JMF adjustment request. Ensure that the resulting JMF is within specified master gradation bands. The department will have a certified Hot Mix Asphalt, Mix Design, Report Submittals technician review the proposed adjustment and, if acceptable, issue a revised JMF.

(2) The department will not allow adjustments that do the following:

- Exceed specified JMF tolerance limits.
- Reduce the JMF asphalt content unless the production VMA running average meets or exceeds the minimum VMA design requirement defined in table 460-1 for the mixture produced.

(3) Have a certified Hot Mix Asphalt, Troubleshooting, Process Control technician make related process adjustments. If mixture redesign is necessary, submit a new JMF, subject to the same specification requirements as the original JMF.

460.2.8.2.1.7 Corrective Action

(1) When running average values trend toward the warning limits, consider taking corrective action. Document corrective actions undertaken. Include test results in the contract files and in running average calculations.

(2) Notify the engineer if running average values exceed the warning limits. If two consecutive running average values exceed the warning limits, stop production and make adjustments. Do not restart production until after notifying the engineer of the adjustments made. Do not calculate a new running average until the fourth test after the required production stop.

(3) If the process adjustment improves the property in question so that the running average after 4 additional tests is within the warning limits, the contractor may continue production with no reduction in payment.

(4) If the adjustment does not improve the properties and the running average after 4 additional tests stays inside the warning bands, the mixture is nonconforming and subject to pay adjustment.

(5) If the contractor fails to stop production and make adjustments when required, all mixture produced from the stop point to the point when the running average is back inside the warning limits is nonconforming and subject to pay adjustment.

Delete 460.2.8.2.1.7(6) to move pay adjustment for nonconforming materials to the payment subsection.

(6) If the running average values exceed the JMF limits, stop production and make adjustments. Do not restart production until after notifying the engineer of the adjustments made. Continue calculating the running average after the production stop.

(7) If the air voids running average of 4 exceeds the JMF limits, the material is nonconforming. Remove and replace unacceptable material. The engineer will determine the quantity of material to replace based on the testing data using the methods in CMM 8-36 and an inspection of the completed pavement. If the engineer allows the mixture to remain in place, the department will pay for the mixture and asphaltic material as specified in 460.5.2.1.

(8) If the running average of 4 exceeds the JMF limits for other properties, and the engineer allows the mixture to remain in place, the department will pay for the mixture as specified in 460.5.2.1. The engineer will determine the quantity of material subject to pay reduction based on the testing data and an inspection of the completed pavement.

Vacate 460.2.8.2.2 to eliminate contractor assurance HMA testing from the contract. The department no longer recognizes these tests as a valid means to assure the accuracy of QC test results.

Chapter 1 460.2.8.2.2 (Vacant)

460.2.8.3 Department Testing 460.2.8.3.1 Quality Verification Program 460.2.8.3.1.1 General

(1) The engineer will conduct QV tests to determine the quality of the final product and measure characteristics that predict relative performance.

460.2.8.3.1.2 Personnel Requirements

(1) The department will provide at least one HTCP-certified HMA technician, certified at a level appropriate for sampling and mixture production control testing, to observe QV sampling of project mixtures.

(2) An HMA technician certified at a level appropriate for sampling and mixture production control testing, or an HMA ACT working under the HMA certified technician, will split samples and do the testing. An HMA technician certified at a level appropriate for sampling and mixture production control testing must coordinate and take responsibility for the work an ACT performs. No more than one ACT can work under a single certified technician.

(3) An HMA technician certified at a level appropriate for sampling and mixture production control testing will ensure that sampling and testing is performed correctly, analyze test results, and post resulting data.

(4) The department will make an organizational chart available at the testing laboratory and to the contractor before mixture production begins. The department's chart will include names, telephone numbers, and current certifications of QV testing personnel. The department will update the chart with appropriate changes, as they become effective.

460.2.8.3.1.3 Laboratory Requirements

(1) The department will furnish and maintain a facility for QV testing conforming to the department's laboratory qualification program requirements and fully equipped to perform QV testing. In all cases, the department will conduct testing in a separate laboratory from the contractor's laboratory.

460.2.8.3.1.4 Department Verification Testing Requirements

(1) HTCP-certified department personnel will obtain random samples by directly supervising HTCP-certified contractor personnel sampling from trucks at the plant. The department will sample according to CMM 8-36. Sample size must be adequate to run the appropriate required tests in addition to one set of duplicate tests that may be required for dispute resolution. The engineer will split the sample for testing and retain the remaining portion for additional testing if needed.

(2) The department will verify product quality using the test methods specified in 460.2.8.3.1.4(3), other engineer-approved methods, or other methods the industry and department HMA technical team recognizes. The department will identify test methods before construction starts and use only those methods during production of that material unless the engineer and contractor mutually agree otherwise.

(3) The department will perform testing conforming to the following standards:

Bulk specific gravity (G_{mb}) of the compacted mixture according to AASHTO T166. Maximum specific gravity (G_{mm}) according to AASHTO T209.

Air voids (V_a) by calculation according to AASHTO T269. VMA by calculation according to AASHTO R35.

(4) The department will randomly test each design mixture at the following minimum frequency:

FOR TONNAGES TOTALING:

Less than 501 tons .... no tests required

From 501 to 5,000 tons..one test

More than 5,000 tons.. add one test for each additional 5,000-ton increment

460.2.8.3.1.5 Documentation

(1) The engineer will document observations during QV sampling, and review QC mixture adjustments and QC test results daily. The engineer will note results of observations and inspection records in a permanent field record as they occur.

460.2.8.3.1.6 Acceptable Verification Parameters

(1) The engineer will provide test results to the contractor within 2 mixture-production days after obtaining the sample. The quality of the product is acceptably verified if it meets the following limits:

- Va is within a range of 2.7 to 5.3 percent.
- VMA is within minus 0.5 of the minimum requirement for the mix design nominal maximum aggregate size.

(2) If QV test results are outside the specified limits, the engineer will investigate immediately through dispute resolution procedures. The engineer may stop production while the investigation is in progress if the potential for a pavement failure is present.

(3) If production continues for that mixture design, the engineer will provide additional retained sample testing at the frequency provided for in CMM 8-36. This supplemental testing will continue until the material meets allowable differences or as the engineer and contractor mutually agree.

460.2.8.3.1.7 Dispute Resolution

(1) When QV test results do not meet the specified limits, the bureau's AASHTO accredited laboratory and certified personnel will referee test the retained portion of the QV sample and the retained portion of the nearest available previous QC sample.

(2) The department will notify the contractor of the referee test results within 3 business days after receipt of the samples.

(3) The department will determine mixture conformance and acceptability by analyzing referee test results, reviewing mixture project data, and inspecting the completed pavement all according to CMM 8-36.

460.2.8.3.1.8 Corrective Action

(1) Remove and replace unacceptable material at no additional expense to the department.

(2) The department will reduce pay for the tonnage of nonconforming mixture, as determined during QV dispute resolution, if the engineer allows that mixture to remain in place. If production of that mixture design continued during the investigation, the department will also adjust pay for that mixture forward to the next conforming QV or QC point. The department will pay for the affected mixture as specified in 460.5.2.1.

Chapter 2 460.2.8.3.2 Independent Assurance Testing

(1) The department will evaluate both the contractor and department testing personnel and equipment as specified in 106.3.4.3.4.

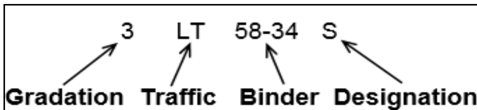
460.3 Construction

460.3.1 General

Revise 460.3.1 to specify the encoding for the combined bid item naming convention.

(1) Construct HMA pavement of the type the bid item indicates encoded as follows:

Combined Bid Item Encoding



GRADATIONS (NMAS) 137.5		TRAFFIC VOLUME		BINDER DESIGNATION LEVEL	
mm		LT	Low	S	Standard
2	25.0 mm	MT	Medium	H	Heavy
3	19.0 mm	HT	High	V	Very Heavy
4	12.5 mm			E	Extremely Heavy
5	9.5 mm				
6	4.75 mm				

(2) Construct HMA pavement conforming to the general provisions of 450.3.

460.3.2 Thickness

Revise 460.3.2(1) to include gradation numbers and update the minimum and maximum required thickness.

(1) Provide the plan thickness for lower and upper layers limited as follows:

NOMINAL SIZE	MINIMUM LAYER THICKNESS in inches	MAXIMUM LOWER LAYER THICKNESS in inches	MAXIMUM UPPER LAYER THICKNESS in inches	MAXIMUM SINGLE LAYER THICKNESS[3] in inches
No. 1 (37.5 mm)	4.5	6	4.5	6
No. 2 (25.0 mm)	3.0	5	4	6
No. 3 (19.0 mm	2.25	4	3	5
No. 4 (12.5 mm)[1] [4]	1.75	3[2]	2.5	4
No. 5 (9.5 mm)[1] [4]	1.5	3[2]	2	3

[1] SMA mixtures use nominal size No. 4 (12.5 mm) or No. 5 (9.5 mm).

[2] SMA mixtures with nominal sizes of No. 4 (12.5 mm) and No. 5 (9.5 mm) have no maximum lower layer thickness specified.

[3] For use on cross-overs and shoulders.

[4] Can be used for a leveling layer or scratch coat at a reduced thickness.

460.3.3 HMA Pavement Density Maximum Density Method

460.3.3.1 Minimum Required Density

Revise 460.3.3.1(1) table 460-3 to switch from E mixes to LT, MT, and HT mixes. This revision incorporates STSP460-025.

(1) Compact all layers of HMA mixture to the density table 460-3 shows for the applicable mixture, location, and layer.

TABLE 460-3 MINIMUM REQUIRED DENSITY [1]

LOCATION	LAYER	PERCENT OF TARGET MAXIMUM DENSITY		
		MIXTURE TYPE		
		LT and MT	HT	SMA [5]
TRAFFIC LANES [2]	LOWER	91.5[3]	92.0[4]	___
	UPPER	91.5	92.0	___
SIDE ROADS, CROSSOVERS, TURN LANES, & RAMPS	LOWER	91.5[3]	92.0[4]	___
	UPPER	91.5	92.0	___
SHOULDERS & APPURTENANCES	LOWER	89.5	89.5	___
	UPPER	90.5	90.5	___

[1] The table values are for average lot density. If any individual density test result falls more than 3.0 percent below the minimum required target maximum density, the engineer may investigate the acceptability of that material.

[2] Includes parking lanes as determined by the engineer.

[3] Minimum reduced by 2.0 percent for a lower layer constructed directly on crushed aggregate or recycled base courses.

[4] Minimum reduced by 1.0 percent for a lower layer constructed directly on crushed aggregate or recycled base courses.

[5] The minimum required densities for SMA mixtures are determined according to CMM 8-15.

460.3.3.2 Pavement Density Determination

(1) The engineer will determine the target maximum density using department procedures described in CMM 8-15. The engineer will determine density as soon as practicable after compaction and before placement of subsequent layers or before opening to traffic.

(2) Do not re-roll compacted mixtures with deficient density test results. Do not operate continuously below the specified minimum density. Stop production, identify the source of the problem, and make corrections to produce work meeting the specification requirements.

(3) A lot is defined in CMM 8-15 and placed within a single layer for each location and target maximum density category indicated in table 460-3. The lot density is the average of all samples taken for that lot. The department determines the number of tests per lot according to either the linear sublot system or the nominal tonnage system defined in CMM 8-15.

(4) A certified nuclear density technician, or a nuclear density ACT working under a certified nuclear density technician, will locate samples and perform the testing. A certified nuclear density technician must coordinate and take responsibility for the work an ACT performs. No more than one ACT can work under

a single certified technician. The responsible certified technician will ensure that sample location and testing is performed correctly, analyze test results, and provide density results to the contractor weekly.

460.3.3.3 Waiving Density Testing

(1) The engineer may waive density testing for one or more of the following reasons:

1. It is impracticable to determine density by the lot system.
2. The contract contains less than 750 tons of a given mixture type placed within the same layer and target maximum density category.

(2) If the department waives density testing notify the contractor before paving. The department will accept the mixture by the ordinary compaction procedure as specified in 450.3.2.6.2.

(3) If HMA QC testing is waived under 460.2.8.2.1.3.3, density testing is also waived.

460.4 Measurement

(1) The department will measure the HMA Pavement bid items acceptably completed by the ton as specified in 450.4.

460.5 Payment

460.5.1 General

Revise 460.5.1(1) to add combined bid items that specify the gradation, traffic, binder, and designation. This revision incorporates STSP460-025.

(1) The department will pay for measured quantities at the contract unit price under the following bid items:

ITEM NUMBER	DESCRIPTION	UNIT
460.5000 - 5999	HMA Pavement (gradation) LT (binder)(designation)	TON
460.6000 - 6999	HMA Pavement (gradation) MT (binder)(designation)	TON
460.7000 - 7999	HMA Pavement (gradation) HT (binder)(designation)	TON
460.8000 - 8999	HMA Pavement (gradation) SMA (binder)(designation)	TON
460.2000	Incentive Density HMA Pavement	DOL

460.5.2 HMA Pavement

460.5.2.1 General

Revise 460.5.2.1 to specify nonconforming combined bid item pay adjustments. This revision incorporates STSP460-025.

(1) The department will pay for the HMA Pavement bid items at the contract unit price subject to one or more of the following adjustments:

1. Disincentive for density of HMA pavement as specified in 460.5.2.2.
2. Incentive for density of HMA pavement as specified in 460.5.2.3.
3. Reduced payment for nonconforming smoothness as specified in 450.3.2.9.
4. Reduced payment for nonconforming QMP HMA mixtures as specified in 460.2.8.2.1.7.

(2) Payment for the HMA Pavement bid items is full compensation for providing HMA pavement including binder; for mixture design; for preparing the foundation; and for QMP and aggregate source testing.

(3) If provided for in the plan quantities, the department will pay for a leveling layer, placed to correct irregularities in an existing paved surface before overlaying, under the pertinent paving bid item. Absent a plan quantity, the department will pay for a leveling layer as extra work.

(4) The department will administer pay reduction for nonconforming QMP mixture under the Nonconforming QMP HMA Mixture administrative item. The department will reduce pay based on the contract unit price for the HMA Pavement bid item.

(5) The department will reduce pay for nonconforming QMP HMA mixtures as specified in 460.2.8.2.1.7, starting from the stop point to the point when the running average of 4 is back inside the warning limits. The engineer will determine the quantity of material subject to pay reduction based on the testing data and an inspection of the completed pavement. The department will reduce pay as follows:

PAYMENT FOR MIXTURE [1] [2]

ITEM	PRODUCED WITHIN WARNING BANDS	PRODUCED OUTSIDE JMF LIMITS
Gradation	90%	75%
Asphalt Content	85%	75%
Air Voids	70%	50%
VMA	90%	75%

[1] For projects or plants where the total production of each mixture design requires less than 4 tests refer to CMM 8-36.

[2] Payment is in percent of the contract unit price for the HMA Pavement bid item. The department will reduce pay based on the nonconforming property with lowest percent pay.

[3] In addition to any pay adjustment listed in the table above, the department will adjust pay for nonconforming binder under the Nonconforming QMP Asphaltic Material administrative item. The department will deduct 25 percent of the contract unit price of the HMA Pavement bid item per ton of pavement placed with nonconforming PG binder the engineer allows to remain in place.

(6) If the department discovers nonconforming mixture during a QV dispute resolution investigation, and the engineer allows that mixture to remain in place, the department will pay for the quantity of affected material as specified in 460.2.8.3.1.8 at 50 percent of the contract price.

(7) If the department waives density testing under 460.3.3.3, the department will not adjust pay under either 460.5.2.2 or 460.5.2.3.

(8) Restore the surface after cutting density samples as specified in 460.3.3.2(1) at no additional cost to the department.

460.5.2.2 Disincentive for HMA Pavement Density

(1) The department will administer density disincentives under the Disincentive Density HMA Pavement administrative item. If the lot density is less than the specified minimum in table 460-3, the department will reduce pay based on the contract unit price for the HMA Pavement bid item for that lot as follows:

DISINCENTIVE PAY REDUCTION FOR HMA PAVEMENT DENSITY

PERCENT LOT DENSITY BELOW SPECIFIED MINIMUM	PAYMENT FACTOR (percent of contract price)
From 0.5 to 1.0 inclusive	98
From 1.1 to 1.5 inclusive	95
From 1.6 to 2.0 inclusive	91
From 2.1 to 2.5 inclusive	85
From 2.6 to 3.0 inclusive	70
More than 3.0[1]	

[1] Remove and replace the lot with a mixture at the specified density. When acceptably replaced, the department will pay for the replaced work at the contract unit price. Alternatively the engineer may allow the nonconforming material to remain in place with a 50 percent payment factor.

(2) The department will not assess density disincentives for pavement placed in cold weather because of a department-caused delay as specified in 450.5.2(3).

460.5.2.3 Incentive for HMA Pavement Density

(1) If the lot density is greater than the minimum specified in table 460-3 and all individual air voids test results for that mixture placed during the same day are within +1.0 percent or - 0.5 percent of the design target in table 460-2, the department will adjust pay for that lot as follows:

INCENTIVE PAY ADJUSTMENT FOR HMA PAVEMENT DENSITY

PERCENT LOT DENSITY ABOVE SPECIFIED MINIMUM	PAY ADJUSTMENT PER TON [1]
From -0.4 to 1.0 inclusive	$0
From 1.1 to 1.8 inclusive	$0.40
More than 1.8	$0.80

[1] The department will prorate the pay adjustment for a partial lot.

(2) The department will adjust pay under the Incentive Density HMA Pavement bid item. Adjustment under this item is not limited, either up or down, to the bid amount the schedule of items shows.

(3) The department will restrict incentive payment for shoulders paved integrally with the traffic lane, if the traffic lane does not meet incentive requirements, the department will not pay incentive on the integrally paved shoulder.