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Factors Affecting Compaction of Asphalt Pavements

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Preface

An all-day workshop at the 84th Annual Meeting of the Transportation Research Board (TRB) addressed asphalt practitioners' concerns related to specifying and achieving density during hot-mix asphalt (HMA) pavement construction. The workshop was divided into four mini-sessions with the following themes and topics:

- **Optimizing HMA Construction Temperatures.** It has been known for some time that polymer-modified asphalts have high kinematic viscosities, yet can be mixed and compacted at temperatures well below those predicted by standard HMA guidelines. This session focused on research and good-practices which could enable the contractor to lower construction temperatures, thus achieving concurrent economic and environmental benefits.
- **Recent Advances in Compaction Equipment, Including "Intelligent Compaction."** New technologies offer significant advantages for achieving targeted HMA density. Compactor add-ons such as Global Positioning Satellite (GPS) systems, IR measurement of surface temperature, and continuous density tools provide equipment operators with critical data for enhanced efficiency. New technologies such as vibratory pneumatic and oscillatory rollers offer unique alternatives for applying vibration to maximize benefits. There are proven techniques to identify and adequately compact problem mixes.
- **Longitudinal Joint Density.** Construction of quality longitudinal joints continues to be one of the most problematic areas for HMA pavement performance. Agencies are now setting stricter compliance standards for joint density, requiring best-practice construction and effective quality control to earn full pay.
- **Incentives–Disincentives for Construction Quality.** Over the past decade, construction specifications have evolved to include financial incentive–disincentive clauses for key elements thought to impact pavement performance, particularly smoothness and density. More recently, many of these financial adjustments have been based upon statistical criteria as defined in PWL (percent-within-limits) specifications. Do incentives work? How does one write and adapt construction practice to meet statistical specifications?

The papers in this document are invited papers for this workshop. The views expressed in the papers contained in this publication are those of the authors and do not necessarily reflect the views of TRB or the National Research Council. The papers have not been subjected to the formal TRB peer review process.

Appreciation is expressed to James A. Scherocman, Gayle N. King, and Dale S. Decker for their efforts in developing this circular.

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Optimizing Hot-Mix Asphalt Construction Temperatures

Optimizing Mix and Compaction Temperatures
Why and How

RONALD CORUM
CITGO Asphalt Refining Company

Note: A paper was not written for this presentation. Corum presented findings from a recently published document by the Asphalt Paving Environmental Council (APEC). This document, "Best Management Practices to Minimize Emissions During HMA Construction (EC-101)," outlines best construction practices to reduce construction temperatures, thereby minimizing construction costs and reducing worker exposure to asphalt fumes.

The publication is available from the National Asphalt Pavement Association (NAPA) at www.hotmix.org/catalog.

Prediction of Compaction Temperatures Using Binder Rheology

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Guidelines for determining mixing and compaction temperatures of conventional asphalt mixtures were published by the Asphalt Institute in 1962. Since the adoption of the performance grading (PG) system in North America, the use of modified asphalts has been growing continuously. It is well recognized that those 1962 guidelines recommend excessively high construction temperatures that can result in damage to modified asphalts. But why doesn't the current system work? Is kinematic viscosity the wrong physical parameter? Are target viscosities too low? Or are the test parameters of 135°C and 6.8 s^{-1} inappropriate? Many attempts have been made to develop more rational guidelines, but none have been universally adopted at this time.

This study reviews the findings of those alternate methods and proposes a solution that requires measuring viscosity at various shear rates. The results show that most modified asphalts are shear thinning. Thus low-shear viscosity (LSV) could be a major factor in resistance of modified mixtures to compaction. A procedure for measuring LSV is described, and a discussion of the relevance of low-shear rates to gyratory compaction is presented. The study concludes by presenting recommendations for using LSV levels of 3,000 cP to estimate reasonable compaction temperatures for mixtures with modified binders. The study includes only laboratory testing. Field verification of the proposed guidelines is still needed.

INTRODUCTION

Asphalt binders are thermoplastic materials and their rheology is highly sensitive to temperature. They exhibit semisolid behavior at ambient temperatures but can be made fluid by heating. High temperatures make them fluid enough to coat aggregates and they need to remain hot enough to minimize resistance as the asphalt-aggregate mixture is compacted in the lab or on the road. Compaction is a densification process during which air voids are reduced by packing aggregates closer to each other. It is affected by asphalt viscosity because moving of aggregates requires flow of the asphalt binder films connecting the aggregates. Higher viscosity (resistance to flow) results in more resistance to packing.

While it is clear that asphalts need to be heated to reduce viscosity and thus enable mixing and compaction, the fluidity required for each process is not a simple matter. For example, asphalt rheology is sensitive to heat-catalyzed oxidation and to volatilization of lighter fractions. If heated too much, asphalts oxidize at such a high rate that hardening can cause significant damage. Complicating the issue further, asphalt chemistry as defined by crude source has a strong influence on both thermal susceptibility and rates of oxidative age hardening. This

means that to achieve a desired level of fluidity behind the paver unaged asphalts with similar initial rheology may have to be heated to different temperatures. Moreover, volatile emissions emanating from overheated mixtures may not be friendly to the environment. Some asphalt modifiers are themselves heat-sensitive and may break down during improper use. And finally, one must consider production costs, which may increase due to excessive energy use and precautions needed to protect workers and nearby residents.

It is logical to conclude that there is an optimum temperature range for asphalt construction. Most importantly, asphalt construction temperatures should not exceed what is needed to achieve sufficient density and satisfy maximum moisture requirements. While there is agreement in the research community about the need for an "optimum fluidity temperature," there is no consensus regarding answers to the two main questions:

- What physical property controls the role of the binder for mixing and compaction of hot-mix asphalt (HMA)?
- What is the optimum fluidity (viscosity?) for best-practice pavement construction?

In 1962 the Asphalt Institute introduced recommendations for viscosity levels for mixing and for compaction that are still in use today (*1*,*2*). The levels of viscosity were determined based on practical factors and appeared to work well for conventional asphalts of grades commonly used in practice at that time. Since 1993, when the SHRP program ended and the PG system was introduced, it became clear that asphalts with higher viscosities will be used. Modified asphalts, particularly those containing high percentages of polymers, must be heated to unreasonably high temperatures to meet the low viscosity standards recommended by the Asphalt Institute procedures (*3*). For many years users of modified asphalts have relied on general recommendations introduced by binder suppliers or by trade organizations that have developed best practices from general experience with some of these materials. Unfortunately, there is still no standard protocol to determine critical physical properties of these new materials with respect to their impact on construction variables. What should be measured? How? What target specification limits will ensure proper mixing and compaction?

MIXTURE DESIGN AND COMPACTION TEMPERATURES

Selection of the limits for viscosity can have important consequences on materials performance as well as mixture design. As indicated earlier, achieving relatively low viscosities could require excessive heating, resulting in degraded asphalts/modifiers or unnecessary volatilization. The limits could also affect the relative degree of laboratory compaction, and hence the design asphalt content of mixtures. Mixture design procedures used today require a specified compaction effort, as defined by a number of gyrations to reach a specified density. Reducing asphalt viscosity could result in reduced design asphalt content for a given gradation while increasing asphalt viscosity could have the opposite effect. It is apparent from published literature that current mixture design procedures were developed for conventional asphalts, not for modified binders. It is thus necessary to ensure that new procedures for determining mixing and compaction temperatures do not cause significant deviations from the design asphalt contents on which current practice is based. It is also important to consider that there could be significant interaction between viscosity of asphalts and aggregate properties, such as gradation,

shape and surface texture. The need for a simple, rapid, and accurate procedure for determining mixing and compaction temperatures is important not only to protect against causing degrading of asphalts, but also to ensure proper proportioning that will not compromise mixture durability or resistance to traffic loading.

This paper documents ongoing efforts to use binder rheology to estimate reasonable mixing and compaction temperatures for modified and conventional high grade binders. The paper also includes test results for a large number of binders to show temperature differences when the LSV concept is used and how such changes affect the densification of mixtures.

PROCEDURES CURRENTLY RECOMMENDED

While the Asphalt Institute recommendations for mixing temperature at a viscosity of 0.170 ± 20 Pa-s and a compaction temperature at a viscosity of 0.280 ± 30 Pa-s worked well in laboratory for neat binders, it is well recognized that they result in excessive temperatures for modified binders. To solve this problem the Asphalt Institute and the National Asphalt Paving Association (*4*,*5*) recommended a reduction of each of those temperatures by 14°C to 25°C as an arbitrary target to avoid overheating of asphalt binders. This recommendation at best is based on experience and has no basis in scientific testing, and it is known to result in compaction over a broad range of viscosities for different binders.

An attempt was made in NCHRP Project 9-10: Applicability of Superpave Protocols to Mixtures Produced with Modified Asphalts, in the period of 1996–2000, to introduce a revised procedure for establishing mixing and compaction temperatures in the laboratory (*6*,*7*). It was shown in the study that the vast majority of asphalts are shear thinning and that modified asphalts are more difficult to compact due to their relatively high viscosity as measured at low-shear conditions. Based on the assumption that compaction of mixtures in the laboratory is dominated by the high viscosity at low-shear rates, a procedure to determine LSV using the rotational viscometer was introduced, and target limits of LSV were defined. The procedure was presented to the Transportation Research Board (TRB)–FHWA expert task group and was used on a trial basis by a number of state highway agencies including New Mexico, Wyoming, Florida, and Indiana. Mixed results were reported. The procedure was found to be somewhat impractical because it is time consuming and requires curve fitting that is not simple. Also the target limits (LSV of 3.0–6.0 Pa-s) were found to be too high, which could result in difficulty in mixing and coating of aggregates. Other studies found these limits to be useful for field paving, and the method was considered a reasonable alternative to the Asphalt Institute procedure. Attempts were made by the expert task group to adjust limits and to simplify protocol, but resources were limited and no consensus was reached. A continuing need for more data appears to have resulted in a new NCHRP request for proposals.

The LSV concept is not the only procedure proposed to address this problem. A few alternatives followed the work introduced by NCHRP Project 9-10. A University of Texas at Austin (UT) research team introduced the concept of using high-shear viscosity (HSV) measures to determine proper temperatures (*8*,*9*). The team postulated that due to the thin films of asphalt, shearing during compaction is dominated by very high shear rates rather than low-shear rates. In their study, they calculated the shear rate on the binder during the compaction process in the Superpave gyratory compactor (SGC) to be close to $500s^{-1}$. Previously for viscosity measurements in general, a 6.8 s^{-1} shear rate value was utilized. Given these findings, UT

researchers recommended the use of high-shear values during viscosity measurements for the calculation of mixing and compaction temperatures. The team however acknowledged recently that only considering this high-shear rate cannot solve the problem. They subsequently introduced higher limits for viscosity (still based on HSV measurements) that resulted in lower, more reasonable construction temperatures that were verified for a variety of modified asphalts used in Texas.

The importance of shear rate was also corroborated by the asphalt research team at the FHWA. Shenoy and coworkers showed the importance of shear rate in relation to modified binder behavior, though he did not recommended specific mixing and compaction criteria for modified binders (*10*). He further concluded that variation of viscosity with shear rate at the relevant temperatures is very important for specifying conditions for mixing and proposed a method to unify the viscosity versus shear rate at different temperatures for a number of asphalt grades. The authors attempted to represent the change in viscosity from a shear rate as low as $10^{-5}s^{-1}$ to as high as $10^{7}s^{-1}$.

Other researchers have pursued approaches not based upon shear rate dependency. Saloman and Idaho Asphalt Supply research staff introduced activation energy as the binder physical property best related to effective HMA mixing and compaction (*11*). When mixture compaction was investigated, the results indicated that the higher the activation energy for flow, the higher the compactive effort needed to achieve the same density. For example, when the activation energy increased from 70 to 80 kJ/mol, the number of gyrations needed to achieve the same density, increased from 30 gyrations to more than 100 gyrations for the same gradation. Although limited research was done on this concept, Saloman's ideas imply that the rate of change of viscosity with temperature or shear rate could be more important than a single value of viscosity selected at a specific temperature or shear rate.

Another proposal was introduced by Gerald Reinke of Mathy Construction Company in collaboration with the research staff of rheometer manufacturer, TA Instruments (*12*). The proposed method is based on a stress concept. The authors propose a stress threshold, beyond which asphalts can readily coat aggregates and mixtures can be compacted with reasonable effort. The stress level is temperature dependent. Using a simple creep test in the Dynamic Shear Rheometer (DSR), a plot of stress versus temperature can be constructed, and optimum mix and compaction temperatures can be derived.

Researchers also have used mixture testing to determine workability and resistance to compaction at various temperatures. The National Center for Asphalt Technology introduced an instrumented mixer to measure the resistance of loose mixtures to mixing (*13*). Workability was defined as the inverse of the torque required to rotate the paddle within the sample of HMA. A preliminary attempt was made to utilize workability data to determine realistic compaction temperatures. Unfortunately, resources were limited and results were inconclusive.

Measuring compactibility of mixtures was the subject of an extensive study led by researchers at the University of Wisconsin (UW) at Madison. In this approach, the SGCis used to determine a mixture's resistance to densification. A special device called the Gyratory Plate Load Assembly (GLPA), and later called the Pressure Distribution Analyzer (PDA), is employed in combination with evolving specimen height to determine the Compaction Energy Index (CEI) (*14,15*). CEI was defined as the area under the densification curve between the relative density corresponding to the 8th gyration and the density at 92%G_{mm}. The area under the curve showing the variation of the shear resistance effort measured by the PDA and the number of gyrations to 92 % G_{mm} was defined as the Compaction Force Index (CFI). Both measures were used to

evaluate the impact of binder type, temperature, aggregate characteristics and SGC vertical pressure on evolving density (*15*).

A similar study was conducted by Voller to define parameters that affect compaction temperatures for modified asphalt binders (*16*). His primary hypothesis: optimum compaction occurs at the temperature where the shear stress in lowest. This study used the Intensive Compaction Tester (ICT) gyratory compactor to densify lab and field-mixed specimens and monitor the power required during compaction. Results for a limited number of binders indicated that shear stress generally increases as temperature decreases, but the power required to achieve a specific density depends largely on aggregate type and gradation. Surprisingly, temperature and asphalt grade were found to have minimal effects. The results generally confirmed UW findings that most mixtures exhibit a minimum in shear stress when observed over a range of temperatures. This provides further evidence that an optimum compaction temperature exists, but that this temperature varies with mixture type and aggregate properties.

TEMPERATURE PREDICTIONS USING EXISTING PROCEDURES

As indicated earlier, the current Superpave procedure requires mixing at a viscosity of 170 cP and compacting at 280 cP. To show why these limits are not practical for use with modified binders, a data base maintained at UW was used to prepare the plot shown as Figure 1. The data for 40 binders tested at multiple temperatures and shear rates were used to estimate temperatures at which these limits could be achieved. Binder grades ranged from PG 64 to PG 82, and included various polymer additives, acid modification, and oxidized asphalt. As seen in the plot

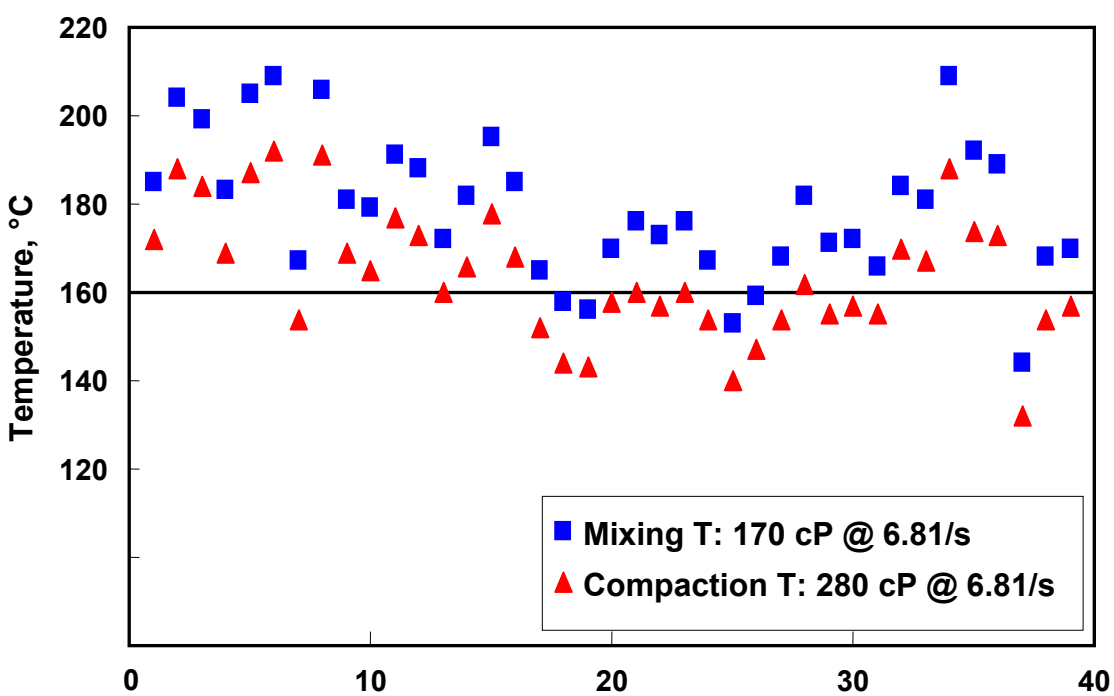

FIGURE 1 A sample of temperatures at which the current requirements of 170 cP and 280 cP could be achieved for a number of modified binders.

all but four binders required a mixing temperature above 160°C. The importance of this issue is magnified considerably when one observes that 20 of 39 binders required compaction temperatures above 180°C, and five required compaction temperatures of 200°C or more. Given the ever-present need to maintain HMA product quality while minimizing environmental impact during construction, these recommendations are unacceptable.

This is not a new finding. In fact suppliers of modified asphalts typically recommend construction temperatures significantly below those at which the 170 cP and 280 cP criteria are satisfied, yet mixes are consistently compacted to target densities.

EFFECT OF TEMPERATURES ON DENSITY IN THE GYRATORY COMPACTOR

It is generally accepted that temperature affects asphalt viscosity which in turn impacts density. However, the issue is complicated by the fact that at the same viscosity level, as measured at a given shear rate, mixtures produced with the same aggregates and volumetric properties do not achieve the same density when binders of different modification types are used. Figure 2 plots data collected by NCHRP Project 9-10. Here, the same mixture was produced with a control unmodified asphalt and four different modified binders. The air voids achieved at a given number of gyrations (Ndes) at various temperatures are plotted as a function of kinematic viscosity as measured at 6.8 s^{-1} (20 rpm) for these same temperatures.

Interestingly, the SGC compacted the unmodified control mixture to the target 4% air voids at a temperature where the binder's viscosity was 100,000 cP, which is over 300 times the

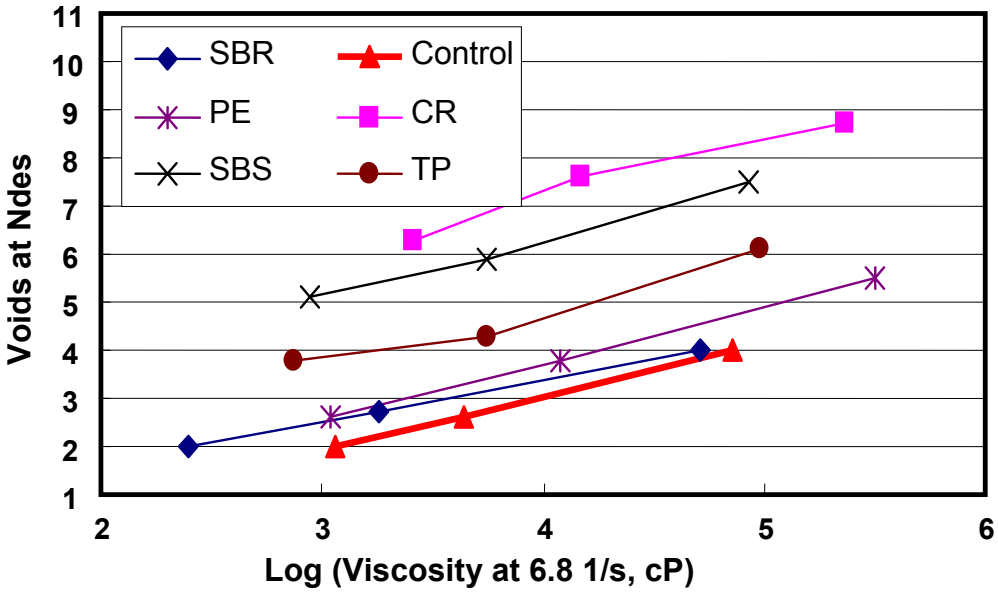

FIGURE 2 Comparison of effect of viscosity on air voids achieved at Ndesign for a single mixture produced with different binders.

recommended 280 cP. However, at a given viscosity as measured at 20 rpm, all modified mixtures exhibited higher air voids at the same number of gyrations. The differences in density are not small, reaching as high as 4% air voids. This observation is critical, because it indicates that choosing a higher limiting viscosity at the proposed shear rate cannot guarantee the same density for modified mixtures. Since changing the target viscosity does not appear to offer a solution to the problem, one must decide whether to change the applied shear rate away from 6.8 s^{-1}, or reject viscosity as the limiting parameter.

Before speculating about other solutions, it is important to notice the trend that all modified binders result in higher voids, which means that their viscosity must be under-estimated. One plausible explanation is that the shear rate as defined by 20 rpm in the Brookfield underestimates the viscosity at which modified binders resist densification. Now one must ask, "What is the effect of shear rate?" And more importantly, "Can other shear rates estimate compaction temperatures for these modified mixtures that lead to equal air voids for a given compactive effort?

EFFECTS OF SHEAR RATE ON VISCOSITY OF MODIFIED BINDERS

It is well recognized that modified binders are sensitive to shear rate. Most of these binders are shear thinning, a term used to indicate that higher shear rates result in lower viscosity. This can be easily demonstrated with the rotational viscometer by measuring viscosity at the same temperature over a range of shear rates. Figure 3 shows data for one of the modified binders as tested at two temperatures.

These results are typical of shear thinning as observed with most polymer-modified binders. However, conventional binders exhibit rheological behavior that is much less dependent upon shear rate, as would be expected for Newtonian fluids. Now recall from Figure 2 that modified binders do not compact as well as their conventional counterparts when viscosities at a shear rate of 6.8 s^{-1} are equal. If the applied shear rate during compaction is well below 6.8 s^{-1}, the elevated viscosities of modified binders at those low-shear rates could explain the observed high air void levels. Is it then possible to find a lower experimental shear rate at which compaction depends only upon viscosity, and is independent of the type of modifier used? Furthermore, can this change be theoretically justified, perhaps by arguing that very little relative movement of aggregate particles occurs as the mix approaches the design density?

SHEAR RATES OF MIXTURES IN THE GYRATORY COMPACTOR

It is not intuitive that asphalts are subjected to low-shear rates during compaction. A program was therefore developed to estimate the evolution of shear rates encountered by the binder during compaction of a typical gyratory specimen. The key hypothesis to be tested: the shear rate in the binder film is very low as the gyratory specimen approaches design density.

Figure 4 includes a schematic of the gyratory mold and two plots. The first graph shows the change in relative density (expressed as percent of maximum specific gravity, % Gmm) as a function of the number of gyrations. Since the mold diameter is not changing, the % Gmm can be used to calculate the change in height of the specimen as a function of the number of gyrations. Since the initial height is known, an approximate vertical strain (change in height

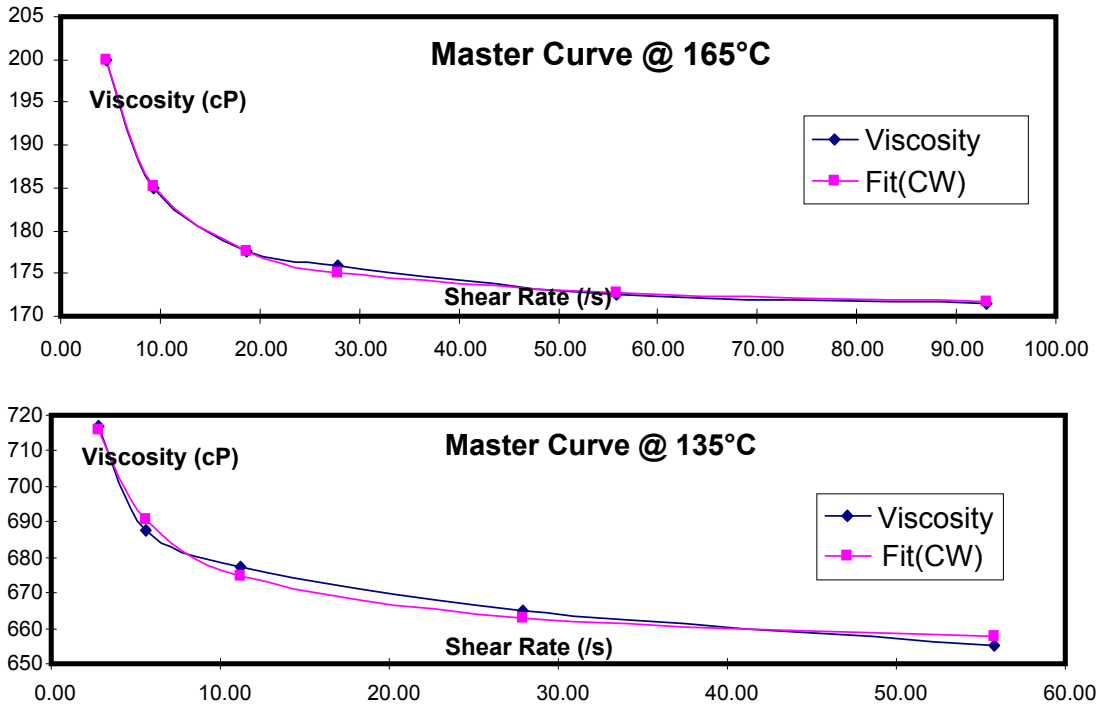

FIGURE 3 Example of shear rate dependency of viscosity of a modified binder.

divided by initial height) of the specimen can be calculated and plotted as a function of number of gyrations, as shown in the lower part of Figure 4.

Notice in the lower plot how fast the linear strain rate changes with number of gyrations. The rate drops from 6.0 s^{-1} to less than 0.25 s^{-1} in the first 25 gyrations. At 40 gyrations the shear rate is close to 0.0. Certainly this result can vary from one mixture to the next, and it may depend upon temperature, binder content and other factors. However, after studying a large number of quality Superpave mixtures, it seems clear that the shear rate is near zero for more than 50 % of the gyrations needed to reach the design target of 4% air voids. As further evidence to support the hypothesis, Figure 4 plots log gyrations versus the rate of increasing density. The rate of density increase falls very rapidly following a power law model.

SHEAR DEFORMATIONS IN THE GYRATORY COMPACTOR

The vertical linear strain is not the only type of strain that asphalt mixtures experience in a gyratory compactor. In fact, many would argue that the shear multidimensional strain is the dominant strain type in mixtures. To evaluate shear deformation, a device called the GLPA was used. As shown in Figure 5, the device has 3 load cells placed at 120-degree angles between two rigid plates. When it is placed on top of the mixture specimen in the SGC, the loads within sample can be measured in real time as sensed by each load cell. The lower plot in Figure 5 shows the signals recorded from each load cell. The signals can be used to calculate the moment perpendicular to the plane of compaction. That moment is a measure of mixture shear resistance.

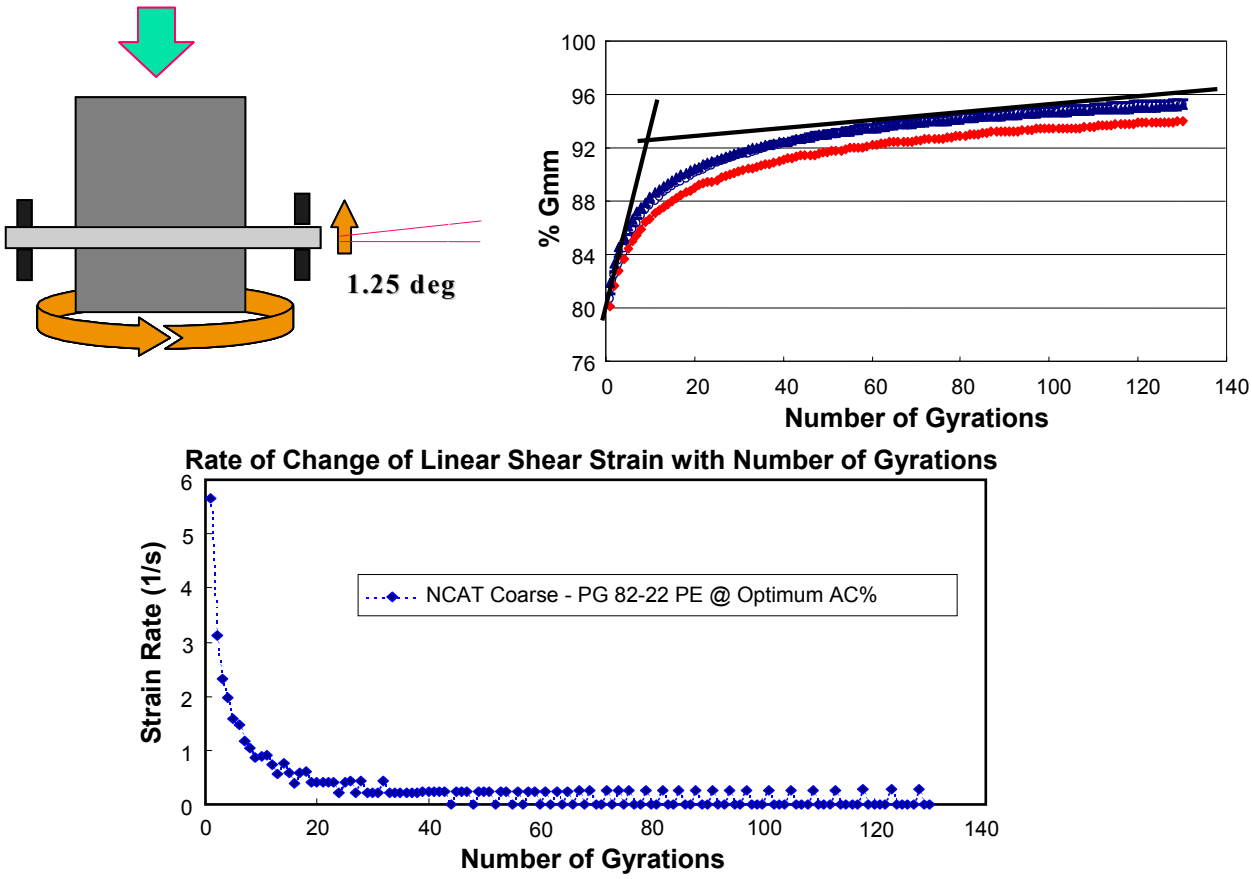

FIGURE 4 Change in vertical densification rate of mixture sample in the gyratory compactor with number of gyrations.

The details of estimating shear resistance are explained in another publication (*15*).

Notice the shape of the curves shown in the lower plot of Figure 5. Each load cell shows a sinusoidal variation indicating that the mixture immediately under the load cell undergoes varying shear rate as the mold is gyrating. The shear rate varies from zero at the highest and lowest load and goes through a maximum a+t the middle point of the force scale. During each gyration the mixture will pass by the zero-shear rate twice while the mold is rotating. Hence, low-shear rates can impact compaction and possibly very significantly. Although somewhat speculative, this analysis is consistent with the observation that modified asphalts show reduced viscosity at high-shear rates, and thus their high-shear behavior should not be the problem. It is their behavior under low-shear conditions where viscosities are high that should be responsible for reduced densities during compaction.

If this analysis is valid, then LSV might be an appropriate physical parameter for selecting compaction temperatures.

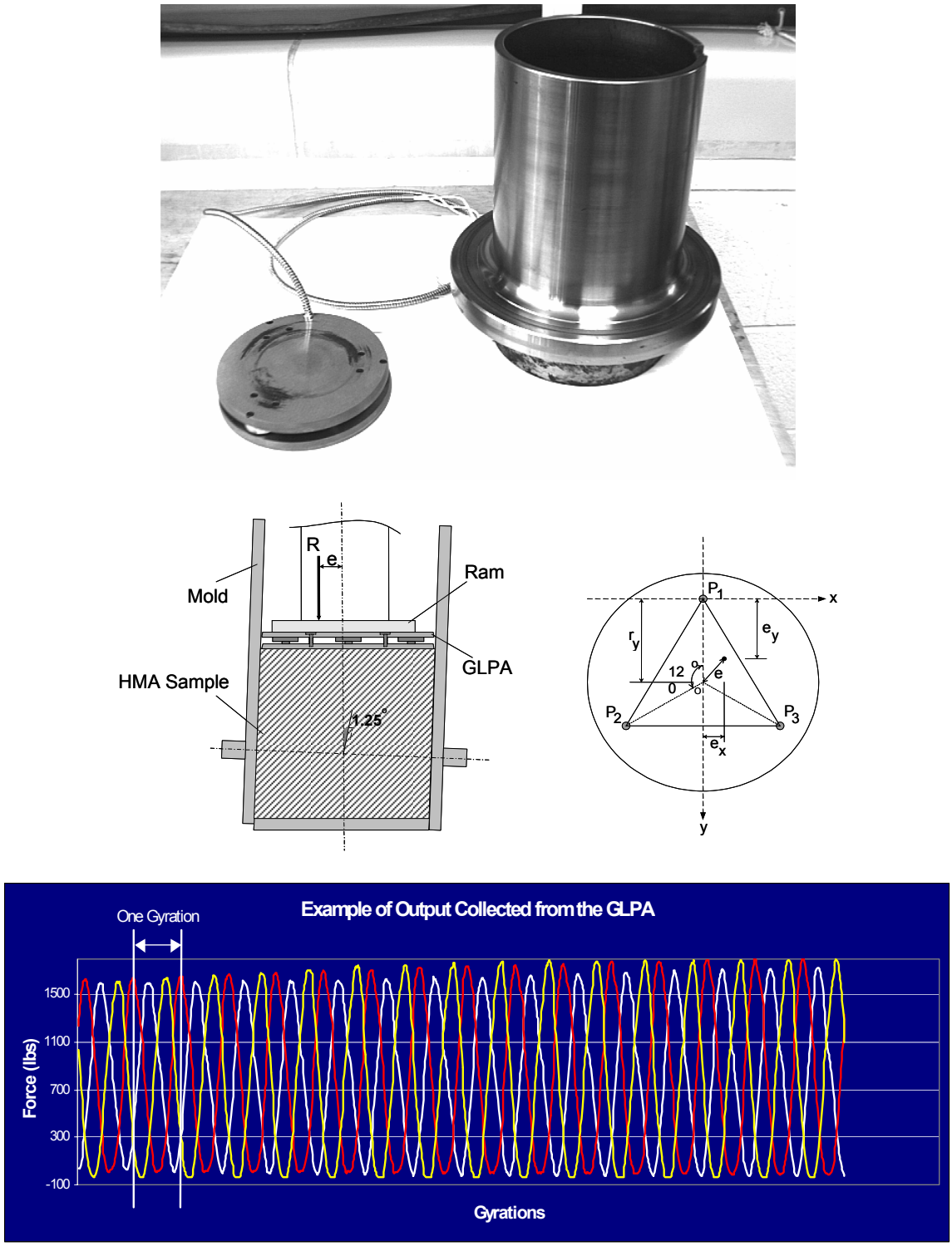

FIGURE 5 Measuring the variation in shear rates in the gyratory compactor using the GLPA.

USING LSV TO DETERMINE COMPACTION TEMPERATURES

Many experiments have been conducted to determine if the LSV concept is viable. Data is available in *NCHRP Report 459* and elsewhere (*6,7*). As one example, Table 1 shows that similar air voids can be achieved at Ndesign and Nmax when various modified mixes are compacted at temperatures where the estimated Brookfield LSV is 3000 cP.

The low-shear viscosities were plotted on a conventional viscosity–temperature chart (log–log viscosity versus log temperature) and the temperature at which each binder reaches 3,000 cP was estimated. Two plastomeric modifiers (EVA) and two elastomers (SBS) were compared to a control mix using the same mix design. Results are promising, since air voids achieved at Design and Maximum gyrations for all five mixes vary within an acceptable range. This result is in stark contrast to results shown previously in Figure 2, where compaction at equiviscous temperatures using the standard shear rate of 6.8 s^{-1} did not result in similar air voids.

CONSEQUENCES OF CHANGES TO LSV-BASED TEMPERATURES

Assuming the LSV concept is useful, there is still a need to select the target viscosity at which compaction temperatures will be chosen. Figure 6 compares the temperatures estimated from a viscosity of 280 cP @ 6.8 s^{-1} (called in the figure HSV), with the temperatures estimated from a LSV of 6,000 cP (solid line). Temperatures based on a LSV of 2,500 cP are shown as the dashed line.

From the top solid line it can be seen that temperatures of HSV=280 cP do not correlate well with temperatures for LSV= 6,000 cP. In fact, there is a wide scatter of approximately 30°C. This is not surprising, since it is known that shear thinning behavior is modifier-specific, and thus the relationship between HSV and LSV is not uniform for all binders. This line also shows that, for many of the binders, HSV will not fall below 280 cP until temperatures reach 160°C and higher. On the other hand, temperatures for which LSV = 6,000 cP are significantly lower. Recommended compaction temperatures would decrease almost 40°C for many modified binders. Although directionally correct, the magnitude of this change seems extreme, particularly with respect to achieving adequate compaction in the field.

To bring lab recommendations more in-line with current best-practice, field engineers were polled. They collectively suggested that field compaction temperatures in the range of 150°C to 160°C are appropriate and achievable for most highly modified asphalt mixes. As can be seen in Figure 6, a limiting LSV of 2,500 cP should result in compaction temperatures below 160°C for all but three of the binders studied here.

TABLE 1 SGC Air Voids at Ndesign and Nmax (LSV = 3,000 cP)

Binder	Gyrations (N)	
	Design	Max
Control	4.1	2.5
43 mi 33E VA	4.4	3.1
2.5 mi 19.3 EVA	4.7	3.7
SBS Linear	4.7	3.1
SBS Radial	4.9	3.5

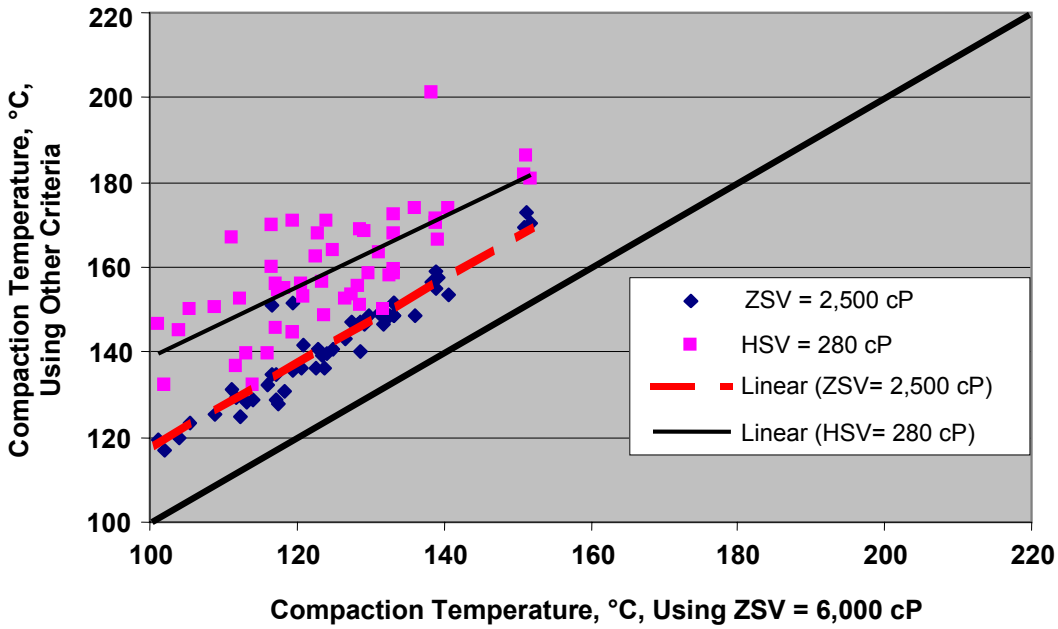

FIGURE 6 Comparison of compaction temperatures using LSV and HSV approaches.

EXAMPLES FOR BINDERS USED IN PRACTICE

As with any new specification limit, it is important to verify that materials successfully used in current practice will not be excluded. A number of binders commonly used in Wisconsin were selected for further testing.

The Brookfield Rotational Viscometer was used to conduct the standard 20 rpm testing at three temperatures for each of the binders. The viscosity-temperature plots were used to estimate the temperatures at which each binder reaches 170 cP and 280 cP. Figure 7 shows the estimated mixing and compaction temperature for each binder using these guidelines. The recommended compaction temperature for most of these binders exceeds the preferred 150°C. One PG 70-28 would require a totally unrealistic compaction temperature of 215°C.

The LSV was also determined by using the Brookfield Viscometer. The same binders were tested at three temperatures: 105°C, 135°C, and 165°C, and at a series of different shear rates for each temperature. The testing was always done at increasing temperatures and shear rates. The shear rates range from 0.47 s^{-1} to 93 s^{-1}. The data are entered in an Excel spreadsheet. The solver program uses a best-fit program to determine the LSV at each of the tested temperatures. The $\log^2$ of the LSV is plotted against the log of the temperature in degrees Kelvin. The temperatures corresponding to a zero-shear viscosity (ZSV) of 3,000 cP and 1,500 cP were estimated for the binders and are shown in Figure 8. All but three of the binders meet the 3,000 cPs limiting viscosity at temperatures below 150°C. 1,500 cP data represents a possible limit for mixing temperatures.

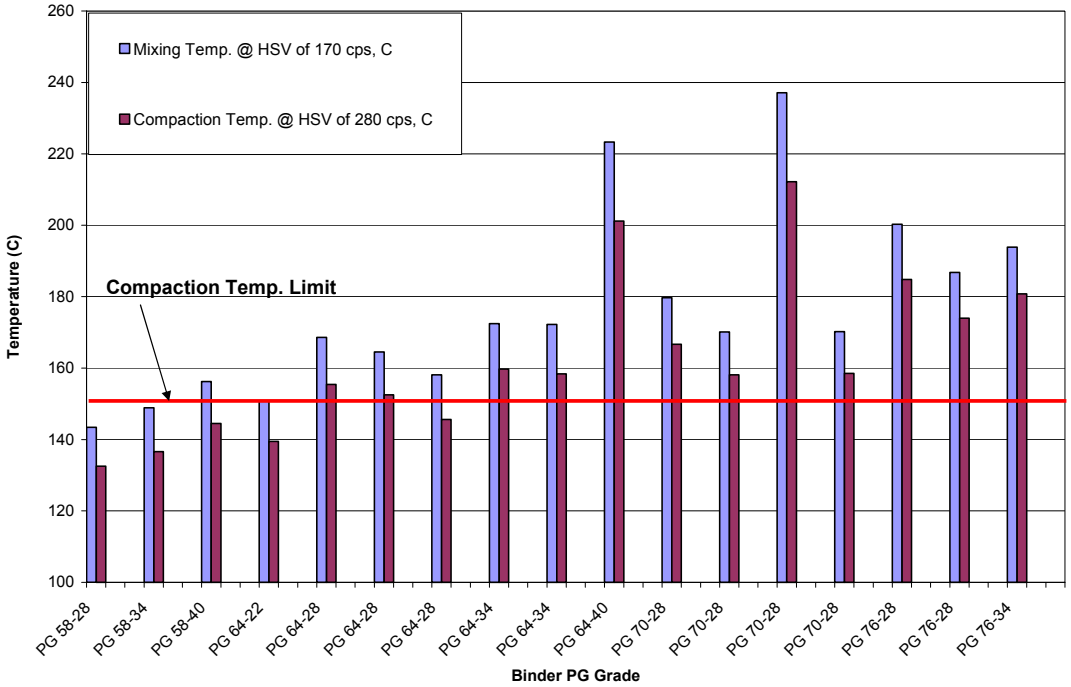

FIGURE 7 Estimated compaction temperatures using an HSV of 280 cP.

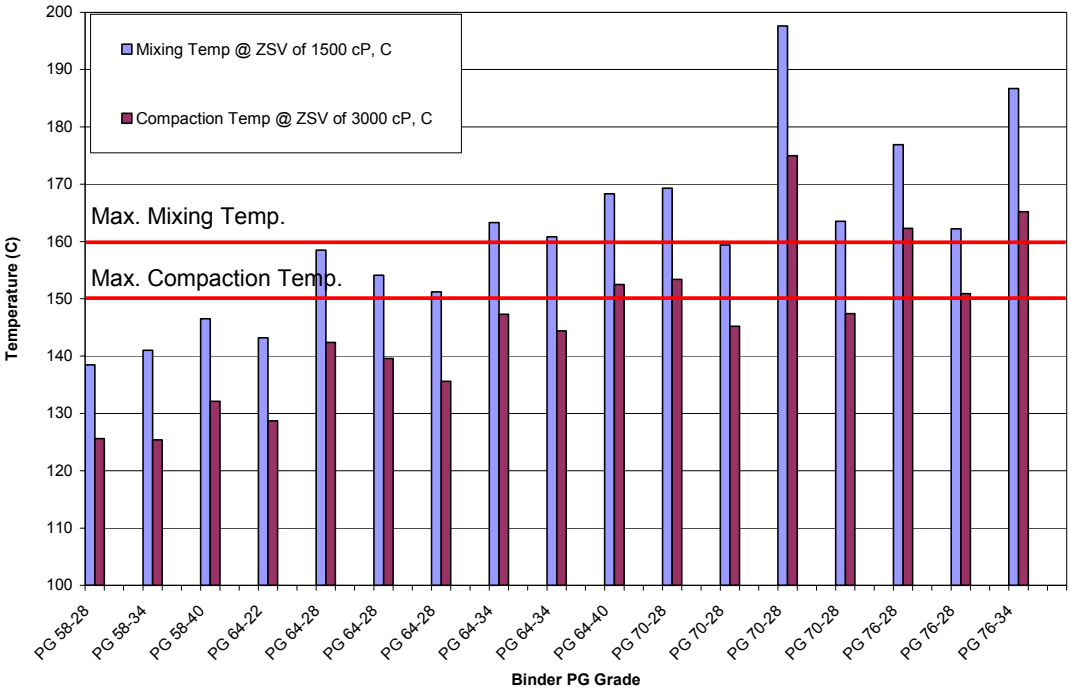

FIGURE 8 Estimated compaction and mix temperatures using LSV values of 3,000 and 1,500 cP.

Another important observation from this study relates to optimum temperatures prescribed for specific PG grades. One might intuitively predict that optimum construction temperatures would be grade specific. In fact, the National Asphalt Pavement Association (NAPA) includes such recommendations in published literature. However, data in Figure 8 suggests that optimum construction temperatures may vary greatly even with the same PG grade, depending upon the type and amount of modifier present in that specific formulation. For example, within the PG 70-28 grade, recommended mixing and compaction temperatures vary by as much as 30°C.

SUMMARY OF FINDINGS AND CONCLUDING REMARKS

Standard laboratory criteria used to recommend mixing and compaction temperatures were evaluated, with special emphasis on modified binders. This study confirms conclusions from numerous field surveys. Current Brookfield viscosity limits (170 cP and 280 cP @ 6.8 s^{-1}) used to select mix and compaction temperatures, as recommended in 1962 for standard Marshall mix designs, are not realistic for modified asphalt mixes. Construction temperature recommendations are much too high, resulting in accelerated binder aging, modifier degradation, excessive energy use, and possible negative environmental impact from volatile emissions.

A number of novel concepts have been proposed to resolve this issue, including activation energy, yield stress, and high- or low-shear rate viscosity. Further laboratory work and controlled field trials are needed to differentiate among these hypotheses.

This study confirmed that many modified binders are shear thinning. Hence, measuring viscosity at high-shear rates may under predict the binders' resistance to flow during compaction, resulting in unacceptably high air voids for resulting mixtures.

Because the viscosity of shear-thinning binders changes substantially with shear rate, one must pay more attention to shear rates as applied to the binder during the compaction process. To insure consistent compacted volumetric properties regardless of binder type, binder rheology must then be characterized within a comparable range of shear rates.

The LSV as estimated from measurements of viscosity as a function of shear rate with the Brookfield viscometer is shown to be a reliable procedure. Based upon results from this study, 3,000 cP is recommended as the limiting LSV for estimating compaction temperature. This target viscosity results in compaction temperatures below 160°C for most modified binders tested here.

ACKNOWLEDGMENTS

The authors wish to express their appreciation to the Wisconsin Highway Research Program for funding the project, to the suppliers of binders for providing the binders, and to the staff of the University of Wisconsin at Madison for assisting in performing the mixture tests. The authors wish also to thank Gayle King for his extensive review and editing of this paper.

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Field Testing of the Zero-Shear Viscosity Method

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It has been well recognized that the rheological properties of binders will affect the mixing and compaction temperatures of hot-mix asphalt. The Superpave mixture design method requires that mixing and compaction temperature be decided at equiviscous binder temperatures corresponding to viscosities of approximately 0.17 and 0.28 Pa·s, respectively. The established method is valid for neat binders, but since the viscosities of modified binders are sensitive to shear rate, the Superpave test protocol could result in unreasonably high temperatures for mixing and compaction. The objective of the research reported here is to verify the recent research based on zero-shear viscosity (ZSV) theory to determine appropriate mixing and compaction temperatures for construction projects in Indiana. Mixing and compaction temperature data as well as materials were taken from several field projects. The mixing and compaction temperatures for the collected binders were determined using the ZSV method. Laboratory samples were then compacted and the optimum binder contents determined and compared to the original mixture designs. The data shows that the ZSV method does appear to be a viable method for determining realistic mixing and compaction temperature for modified binders.

INTRODUCTION

Background and Problem Statement

Hot-mix asphalt (HMA) mixing and compaction temperatures are important to the performance of HMA pavements. Appropriate mixing and compaction temperatures are an aid in achieving complete aggregate coating and adequate field density. Current methods use volumetric criteria to control mixture properties in the field. This makes the issue of mixing and compaction temperatures even more pronounced.

To determine the mixing and compaction temperatures, a temperature–viscosity relationship must be established for a binder following the protocol outlined in ASTM D2493, Standard Viscosity–Temperature Chart for Asphalts (*1*). The Superpave mixture design method requires that gyratory specimens be mixed and compacted at equiviscous binder temperatures corresponding to viscosities of 0.17 ± 0.02 and 0.28 ± 0.03 Pa·s, respectively. The rotational viscometer (RV) is currently used to establish the relationship between temperature and viscosity. This approach is simple and provides reasonable temperatures for unmodified binders that are Newtonian fluids at high temperatures.

In construction practice, modifiers are widely used as additives to enhance the performance of binders when anticipated in-service conditions warrant it such as in high stress applications. However, modified binders are most often non-Newtonian in their behavior; their viscosity is dependent on shear rate. If the mixing and compaction temperatures of modified binders are

determined using the standard ASTM D2493 method, unreasonably high mixing and compaction temperatures can result. In some cases, mixing and compaction temperatures in excess of 190°C (375°F) have been reported.

There are potential dangers associated with elevated mixing and compaction temperatures, such as excessive fumes, worker safety, thermal separation of the modifier and binder, excessive binder oxidation, and binder drain down during production and placement. Currently, no acceptable test method has been found for establishing these temperatures for modified binders. Typically, modified binder mixing and compaction temperatures are recommended empirically. A more rigorous procedure for selecting reasonable mixing and compaction temperatures for modified binders are therefore necessary.

Objectives

The shear rate dependent properties of modified binders provide a basis for decreasing the mixing and compaction temperatures. The objective of this research is to collect field data to verify the method of specifying mixing and compaction temperatures for HMA mixtures using ZSV theory.

EXPERIMENTAL PROGRAM

Experimental Design

Initial pavement density is a critical factor in the construction of durable HMA pavements. It can affect a pavement's performance throughout its in-service life. Factors that can affect initial pavement density are mixing and compaction temperatures, binder type, compaction effort, and aggregate type and size. In an effort to investigate these factors an experimental matrix was designed to account for as many of the factors as possible. The matrix is show in Table 1 and includes the factors binder type (three levels), compaction effort (two levels), aggregate structure (three levels), and nominal maximum aggregate size (three levels).

The binder factor levels were selected so as to provide a range of binders currently used on Indiana Department of Transportation (INDOT) projects. PG 64-22 binders are unmodified and most typically used on less critical roadways. PG 76-22 binders are modified and used nearly exclusively on more heavily trafficked pavements. The PG 70-22 binders may or may not be modified. The two compaction factor levels were chosen in an attempt to represent different compaction efforts typically used on the various roadways in Indiana. The higher level of compaction is indicative of a more heavily trafficked roadway that requires a stiffer (perhaps modified) binder. The low compaction level represents the low to medium trafficked roadways.

The last two factors of the experiment, aggregate structure and nominal maximum aggregate size (NMAS) are inter-related and were chosen so as to provide a cross-section of mixtures. Coarse-graded mixtures are defined as those mixtures having a gradation that goes below the Superpave defined restricted zone while fine-graded mixtures have gradations going above the restricted zone. The NMASs are consistent with the Superpave definition. It should be noted that stone matrix asphalt (SMA) gradation was also selected for the experiment. SMA is a tough, stable, and rut-resistant HMA mixture that is used in areas of high traffic. It relies on stone-to-stone contact to provide strength and a rich mortar to provide durability.

TABLE 1 Experimental Design Matrix

NMAS	Binder Type	Compaction Effort	Mixture Gradation		
			Coarse	Fine	SMA
9.5 mm	64-22	High			
		Low			
	70-22	High			
		Low			
	76-22	High			
		Low			
12.5 mm	64-22	High			
		Low			
	70-22	High			
		Low			
	76-22	High			
		Low			
19 mm	64-22	High			
		Low			
	70-22	High			
		Low			
	76-22	High			
		Low			

Materials

As can be seen from Table 1, there are a total of 54 cells in the experimental design. However, not all of the combinations in the table are possible. For example, low-compaction effort for an SMA mixture is unlikely. SMA is used on high type roadways where a low-compaction effort would not be satisfactory. In Indiana it is also currently used with only a 9.5-mm NMAS. Additionally, fine-graded mixtures are not typically used by INDOT. When these limitations were taken into account, eight INDOT projects in the 2002 construction season were identified as suitable for the research. Table 2 provides the detailed project information.

Test Methods

According to the recommended test procedure developed by Khatri et al. (*2*), a rotational viscometer was used to measure the viscosities of each project binder at the temperatures of 105°C, 135°C, and 165°C (220°F, 275°F, and 330°F). The test data were then input into a spreadsheet to determine the mixing and compaction temperatures for each binder. The procedure for calculating the ZSV temperatures is based on the Cross–Williamson model. The ASTM D2493 test protocol is used to establish the viscosity–temperature profile. Mixing and compaction temperatures are calculated using this profile. The mixing temperature is determined as the temperature that yields a 3.0 Pa-s viscosity; the temperature at 6.0 Pa-s is chosen as the compaction temperature.

Rotational viscometers measure viscosity by the torque required to rotate a standard spindle at a constant rate while immersed in a binder. The torque is proportional to the viscosity of the

TABLE 2 Selected Projects

Project	Mixture Gradation	NMAS	PG Binder Grade	Compaction Effort
1	Coarse	19.0	64-22	Low
2	Coarse	12.5	64-22	Low
3	Coarse	12.5	64-22	Low
4	Coarse	12.5	70-22	High
5	Coarse	9.5	70-22	High
6	Coarse	9.5	76-22	High
7	Coarse	19.0	76-22	High
8	SMA	9.5	76-22	High

binder. Under the standard Superpave protocols, the RV is used to establish that a particular binder can be handled (pumped) at the hot-mix plant and to establish the mixing and compaction temperatures. The tests are performed at 20 rpms with the standard No. 27 spindle. This combination results in a test shear rate of 6.8 s^{-1}.

When determining mixing and compaction temperatures using the ZSV method, the viscosity of the binder is determined at three temperatures over a range of shear rates. If the binder in Newtonian, its viscosity will not vary with shear rate. This can be verified by plotting the viscosity as a function of shear rate at each of the three temperatures. However, if the data shows that the binder viscosity is dependent on shear rate, the binder is non-Newtonian. In this case, a spreadsheet can be used to determine the ZSV mixing and compaction temperatures.

For each project that used a non-Newtonian binder, the aggregates sampled from the hot-mix plant were dried and separated into standard size fractions in the laboratory. The appropriate proportions of each size were then batched to duplicate the job mix formula used in the field. These aggregate batches were then heated to the ZSV mixing temperatures, mixed with appropriate binder sampled from hot-mix plant, and oven-aged for 2 h at the ZSV compaction temperatures in accordance with Superpave mixture design guidelines. Once aging was completed, Superpave gyratory compactor (SGC) samples were compacted at the ZSV compaction temperature using the number of design gyrations specified for the given project. In each project, SGC specimens were produced at various binder contents in order to establish the optimum binder content based on the ZSV mixing and compaction temperatures. Optimum binder content was chosen so as to yield 4% air voids in the mixtures.

DISCUSSION OF RESULTS

The results of the study are shown in Table 3. Four of the binders proved to be Newtonian while the remaining four were non-Newtonian. As expected, the PG 64-22 binders used in the projects are Newtonian and the three PG 76-22 binders are non-Newtonian. There were two PG 70-22 binders. One proved to be non-Newtonian (Project 5), while the other was Newtonian (Project 4). For Newtonian binders, there are no recommended ZSV temperatures since the standard RV method yields valid temperature ranges. For the projects with non-Newtonian binders (Projects

TABLE 3 Mixing and Compaction Temperatures

		1	2	3	4	5	6	7	8
NMAS		19.0	12.5	12.5	12.5	9.5	9.5	19.0	9.5
Gradation Type		Coarse	Coarse	Coarse	Coarse	Coarse	Coarse	Coarse	SMA
Binder Grade		64-22	64-22	64-22	70-22	70-22	76-22	76-22	76-22
Mixing Temperature (°C)	RV	159	158	157	165	172	198	196	233
	Design	160	152	154	149	160	163	156	168
	Field	—	149	—	177	152	164	155	158
	ZSV	116	115	118	120	154	157	153	173
Compaction Temperature (°C)	RV	147	144	146	153	160	181	181	210
	Design	143	141	143	138	149	152	150	160
	Field	—	—	—	138	146	143	—	158
	ZSV	104	103	106	112	145	144	141	152
Design Optimum Binder Content (%)		4.5	5.8	6.2	5.3	5.9	6.1	4.1	5.6
ZSV Optimum Binder Content (%)						—	6.1	4.1	5.6

— No data were available

5, 6, 7, and 8), the mixture designs were performed using the recommended ZSV mixing and compaction temperatures. Using the ZSV mixing and compaction temperatures does not change the optimum binder contents for the PG 76-22 mixtures. This seems to verify the hypothesis regarding the possibility of using lower mixing and compaction temperatures without affecting the mixture coating and density.

The optimum binder content for Project 5 could not be determined in the laboratory. This was due to equipment error. While the ZSV mixing and compaction temperatures for this project were very close to those specified and used during the original mixture design, when the design was completed using the ZSV temperatures, higher specimen densities and thus lower optimum binder content were obtained. A problem with the SGC is believed responsible, but due to lack of material, it was impossible to make additional specimens. Nevertheless, one can clearly see that the design mixing and compaction temperatures are close enough to the ZSV temperatures that little to no difference in the optimum binder content is to be expected.

TEMPERATURE RELATIONSHIPS

Data plots of the relationships between standard RV and ZSV temperatures are shown in Figures 1 and 2. Each plot shows a regression line through the data as well as the regression equation information. A line of equality is also shown in each plot. Note that for both the mixing and compaction temperatures, there appears to be a shift of approximately 40°C (77°F); the ZSV temperatures seem, on average, to be about 40°C (77°F) below the RV temperatures.

Figures 3 and 4 show the relationships between the design mixing and compaction temperatures and the ZSV temperatures, respectively. Each plot clearly shows non-Newtonian binder data points gathered around the line of equality. This is an indication that for non-Newtonian binders, reasonable design mixing and compaction temperatures are currently being

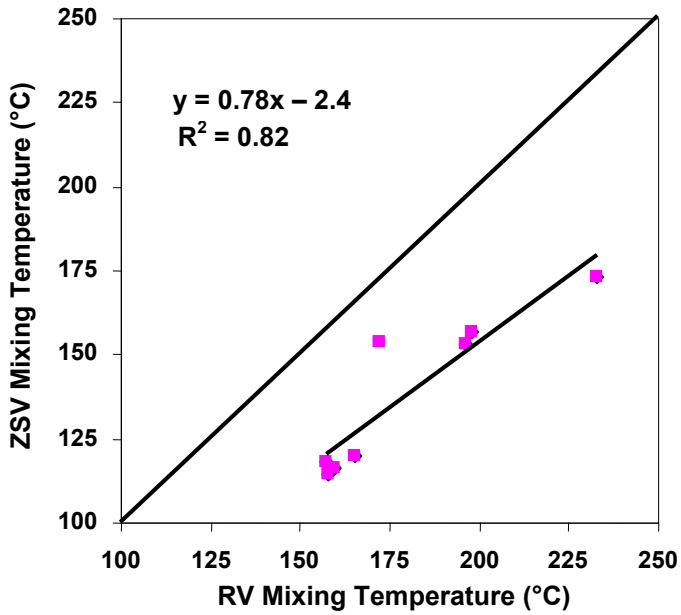

FIGURE 1 Mixing temperature.

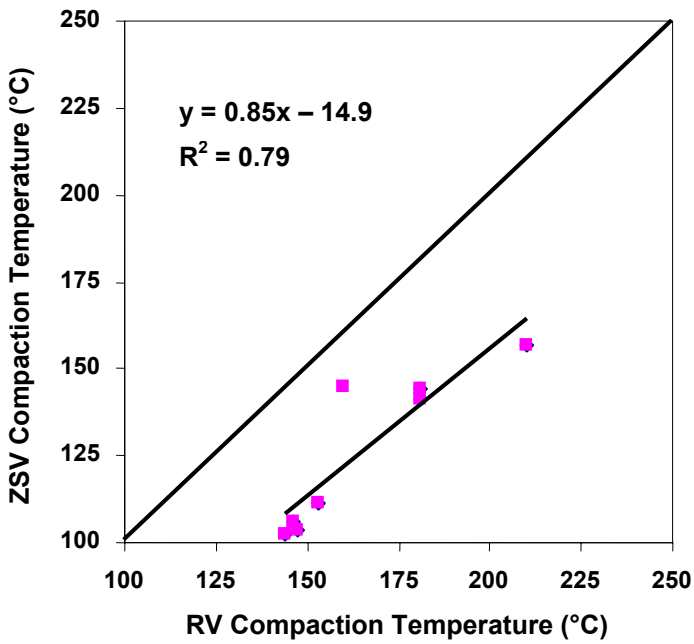

FIGURE 2 Compaction temperature.

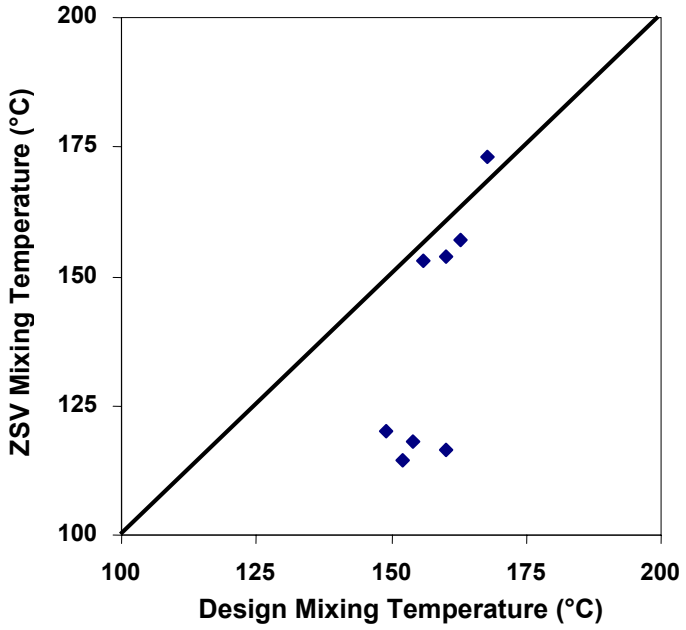

FIGURE 3 Design mixing temperature.

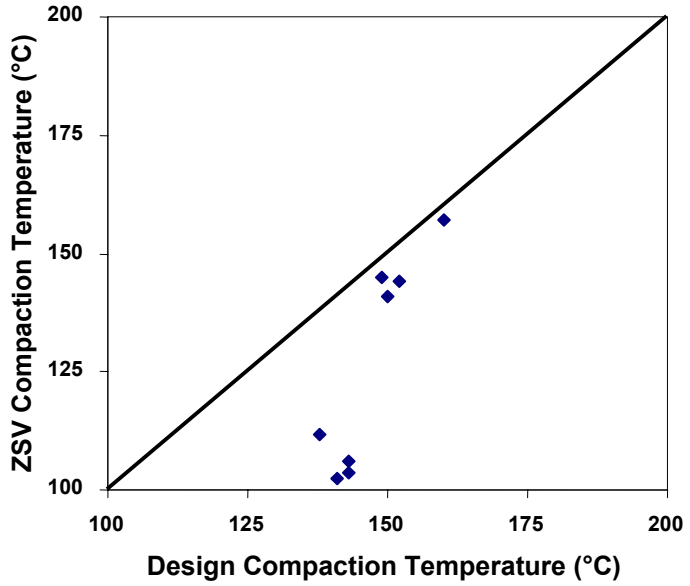

FIGURE 4 Design compaction temperature.

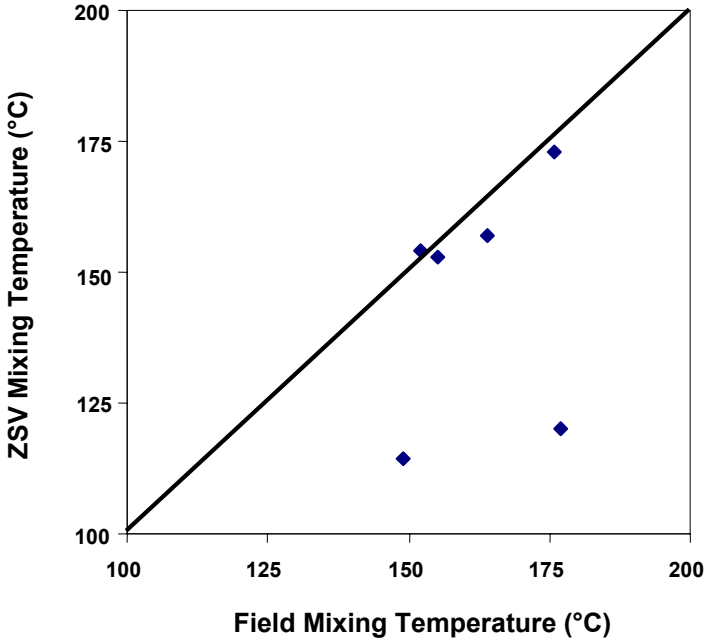

FIGURE 5 Field mixing temperature.

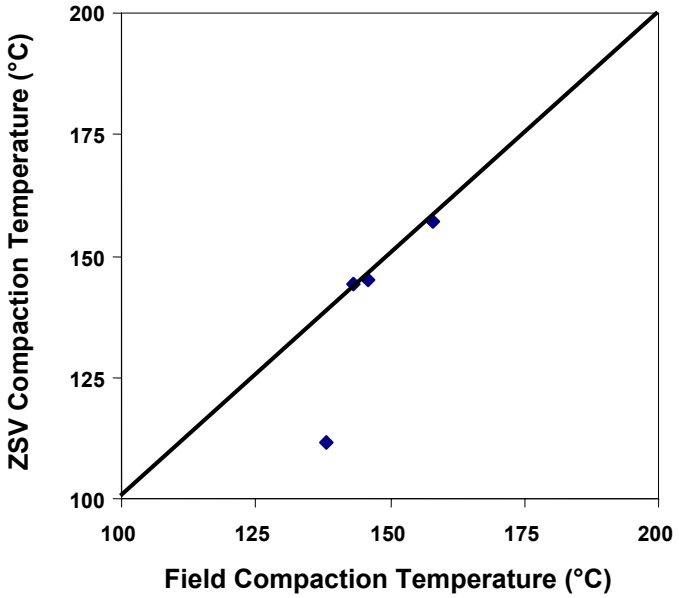

FIGURE 6 Field compaction temperature.

used in practice. The four points on each plot that are well below the line of equality are the Newtonian binders.

Figures 5 and 6 show the relationships between the ZSV temperatures and the mixing and compaction temperatures used in the field, respectively. The data points for the non-Newtonian binders are gathered near the line of equality. This again suggests that for non-Newtonian binders, the ZSV method yields temperatures consistent with those currently being used in practice. The Newtonian data is again well off the line of equality.

For the Newtonian binders, the ZSV method does not appear to work as well as for non-Newtonian binders. This is due to the Newtonian binders' lack of sensitivity to shear and the resulting difficulty in curve fitting the data to the model. With additional research and/or more sensitive test equipment, it is possible that the ZSV method could be used in establishing the mixing and compaction temperatures equally well for both Newtonian and non-Newtonian binders.

CONCLUSIONS

Perhaps the most encouraging finding of the study is that contractors on INDOT projects are currently using reasonable mixing and compaction temperatures even when non-Newtonian (modified) binders are being used. Extreme temperatures are being avoided. This phenomenon indicates that pavement engineers have recognized and corrected the problem. However, the use of these reasonable temperatures appears to be based on experience rather than having any basis in theory. While experience is always a good guide, the lack of a codified procedure for determining mixing and compaction temperatures for non-Newtonian (modified) binders could lead to misunderstandings and perhaps remedial actions in the field. The standardization of a method employing the ZSV method could alleviate these problems.

Establishing mixing and compaction temperatures for modified binders using the ZSV method described in this paper seems a viable method. The data clearly shows that using these temperatures does not change the optimum binder contents for HMA mixtures containing modified binders. These temperatures should also allow adequate densities to be achieved in the field as well.

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State-of-the-Practice for Cold-Weather Compaction of Hot-Mix Asphalt Pavements

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Asphalt technologists around the world understand that achieving proper density of hot-mix asphalt (HMA) pavements in the field is the single most important factor in determining the performance of the pavement. There are literally hundreds of references in the technical literature on the subject. Compaction of the mix at the time of placement is the process whereby density is achieved.

Many agencies have specification requirements which force the contractor to discontinue paving operations at some arbitrary temperature or calendar date. Yet pavements can be successfully placed at low temperatures. Done properly, pavements placed at low temperatures can perform well. Demands of construction scheduling, inclement weather and public safety often force the owner–agency and contractor into a position of needing to get the paving completed, regardless of the environmental circumstances. These are the issues to which this paper is addressed.

INTRODUCTION

Asphalt technologists around the world understand that achieving proper density of HMA pavements in the field is the single most important factor in determining the performance of the pavement. There are literally hundreds of references in the technical literature on the subject. Compaction of the mix at the time of placement is the process whereby density is achieved.

Many agencies have specification requirements which force the contractor to discontinue paving operations at some arbitrary temperature or calendar date. Yet, pavements can be successfully placed at low temperatures. Done properly, pavements placed at low temperatures can perform well. Demands of construction scheduling, inclement weather and public safety often force the owner–agency and contractor into a position of needing to get the paving completed, regardless of the environmental circumstances.

The National Asphalt Pavement Association (NAPA) defines cold-weather paving as placing and compacting HMA when either the base or air temperature is below 50°F (*1*). In many geographical areas, satisfying this requirement would severely limit the available paving season. In order to be able to place good performing mix below this temperature, the contractor must be aware of materials properties and environmental conditions impacting the densification of HMA. Fundamentally, the contractor is responsible for placing good materials with good construction techniques, regardless of the weather.

The purpose of this paper is to describe the state-of-the art for cold-weather compaction, with consideration for project management issues, material property issues, and environmental conditions.

IMPORTANCE OF COMPACTION

Achieving good density of the in-place HMA optimizes all desirable mix properties. Hughes (*2*) defines the desirable mix properties as

- Strength,
- Durability/aging,
- Resistance to deformation,
- Resistance to moisture damage,
- Impermeability, and
- Skid resistance.

Getting the air voids at an acceptable level (and therefore the density to the proper level) will improve the performance of the pavement. Bell et al. concluded that air void content was the most significant factor affecting mix performance (*3*). Linden et al. showed that a 1% increase in air voids (above the base air void level of 7%) tends to produce about a 10% loss in pavement life (*4*). Clearly, the in-place air voids and therefore the in-place density have a significant impact on the pavement life.

FACTORS AFFECTING COMPACTION

Lift thickness, mix properties, and environmental conditions are the key factors that affect the ability of the contractor to achieve density in the HMA layer in any environmental condition. Placing HMA in cold weather may amplify the effect of the properties on the compactability.

If the lift thickness is greater than 2 in., the mix should be able to be placed at the proper density, due to the high retention of heat by the mass of material. However, thinner lifts will require significant effort and attention to detail to achieve proper density of the mix.

Mix properties can have an important impact on the compactability of the materials. Properties of the aggregate and asphalt binder (including use of modifiers) have an impact on the ability of the contractor to achieve density. It is well known that mixes made with coarse, angular aggregates may be more difficult to compact than mixes made with rounded materials. As a result, the coarser mixes may cool before density can be achieved.

The performance grade (PG) of the asphalt binder has a significant influence on the compactability of the mix. Mixes made with low viscosity asphalt binders are mixed, placed, and compacted at lower temperatures. Knowledge of this fact has led some contractors to use softer grades of asphalt during cold-weather mix placement.

To understand the importance of the asphalt binder to the compaction process, it is necessary to discuss the compaction cut-off temperature. NAPA defines the cut-off temperature as "that at which the mix becomes so stiff that additional rolling is ineffective. Total compaction time between placement and cut-off temperatures for asphalt binders of different grades, is roughly the same" (*1*). Figure 1 illustrates the concept of cut-off temperature. At high temperatures, compaction can occur while at low temperatures achieving density is impossible. Discussion of time available for compaction will be later in this paper.

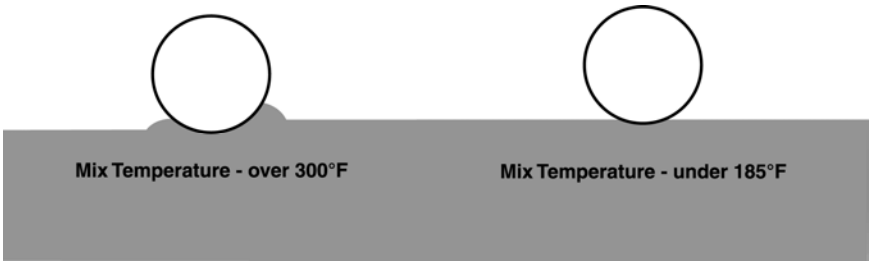

FIGURE 1 Mix cut-off temperature.

Determination of the proper mixing and compaction temperature has become an important issue with the implementation of Superpave PG binder specifications and particularly with the use of modified asphalts. In the past, temperature–viscosity charts were used to determine mixing and compaction temperature, based on the properties of the asphalt cement. Because of the variety of modifiers and the impact of a specific modifier on an asphalt binder made from a specific crude source, the temperature–viscosity charts do not always provide correct mixing and compaction temperatures. More viscous asphalt binders will probably have higher cut-off temperatures, but may not have increased time available for compaction. The contractor must work with the asphalt binder supplier to establish proper mixing and compaction temperatures.

Softer than normal grades of asphalt and slight increase in the asphalt content are approaches that have been used to assist the cold weather compaction process. Both of these approaches have serious potential drawbacks for the performance of the pavement. Depending on the climate at the specific location, asphalt that is too soft or an excess of asphalt content may result in mix instability during hot summer weather. Any such changes need to be carefully evaluated for potential problems.

MIXING AND PLACING HMA IN COLD WEATHER

NAPA (*1*) identifies the following key issues for mixing and placing HMA in cold weather:

- Aggregate drying and heating,
- Mixing and compaction temperature,
- Hauling mix,
- Base influence,
- Base preparation,
- Handwork, and
- Joint construction.

The NAPA publication details each of the items. A brief review is included here.

Drying and heating of the aggregate can become a significant problem for the HMA producer. Stockpiles may be frozen. Moisture content of the materials may be high. One percent additional moisture in the materials may increase the drying cost by as much as 13% (*5*). These factors will increase the cost of mix production during cold weather. Therefore if a decision is

made to place HMA in cold weather, all parties must understand the increase in cost to produce the mix.

The natural response to placing mix in cold weather is to increase the mix temperature. While this temperature increase can be a benefit to achieving density, great caution must be taken to ensure that damage does not occur to the asphalt binder. Excessive temperature can harden the asphalt binder and reduce the film thickness on the aggregate, thereby reducing the performance of the mix in the pavement.

The major issue for hauling mix in cold weather is to ensure that temperature is maintained in the mass of material. While there is some disagreement as to efficacy (*6*), tarping loads to maintain heat is usually recommended. In some areas, insulated truck beds have also been used. It may be necessary and appropriate to remix the HMA at the site for some situations. Each of these approaches should be analyzed for each specific project.

The type of base on which the HMA is being placed does not have a significant influence on compactability if the moisture content is low. However, the relationship between the base and mix temperature is very important. Dickson and Corlew state, "To achieve adequate compaction of hot-mix asphalt pavement, the temperature of the mat must be sufficiently high for the period of time necessary to complete rolling." Thus, with a cold base and a cool mix temperature, a thin lift of HMA would cool rapidly. Dickson and Corlew also report that frozen subgrades with high moisture content will decrease time available for compaction significantly compared to placement on an unfrozen subgrade. In addition to rapid cooling, placement of HMA on wet, frozen subgrade also presents the risk of pavement structural failure, due to thawing of the subgrade (*7*). Clearly, placing HMA on frozen subgrades with high moisture content should not be done.

As with any placement of HMA, preparation of the base materials is important in cold weather. In addition to the normal issues of finding and correcting soft spots in the base, the engineer must be aware that frozen areas may mislead the true load-carrying capability of the materials. Proof rolling is a very effective technique for identifying soft spots. In some cases, removal and replacement or stabilization may be the only solution. However, repair of deficient areas may be difficult in cold weather.

Handling mix in the middle of the summer presents significant compaction issues with which the contractor must contend. In cold weather, all the summer issues are magnified. Mix and placement tools cool quickly so it is desirable to keep handwork to a minimum. If used, HMA windrows must be monitored for maintenance of mix temperature. Feathering of mix during placement is not recommended due to the rapid loss in temperature.

Joint construction is also challenging in the best of times. The additional issue of cold weather forces the contractor to use a heightened sense of attention to detail. All conventional best practices for joint construction should be followed. If possible, preheating the joints prior to placement of an adjoining pass is desirable. The paver must not get too far ahead of the compaction operation. Loss of temperature in the mix can create significant compaction problems. The roller needs to get on the joint quickly. Paving in echelon works well if the site permits.

COMPACTION IN COLD WEATHER

Conventional static steel, vibratory, and pneumatic rollers can be used to compact HMA in cold-weather conditions. As in any rolling train, it will be necessary to establish the sequence of rolling with the specific equipment to achieve the required density. Roller checking may be a problem when using the steel wheel roller in cold weather. Required density may be achieved with fewer passes using the vibratory roller. Pneumatic rollers may prove to be a benefit in cold weather due to the kneading action imparted to the mix. The major challenge in cold weather compaction with a pneumatic roller however is keeping the tires hot. It is imperative that the tires are kept hot.

NAPA identifies six temperature loss factors that must be considered:

1. Thickness of lift,
2. Base temperature,
3. Initial mat temperature,
4. Air temperature,
5. Wind speed, and
6. Solar gain.

The thickness and temperature of the HMA lift being placed control heat dissipation and therefore largely determine the cooling rate. At low temperature and high wind speed, the surface cooling rate is significantly influenced. The primary influence of solar gain is on the temperature of the base (*1*).

The ability of the contractor to achieve densification in the HMA layer is closely tied to the mix temperature. The temperature loss factors must be considered in developing the paving plan. However, it is important to also realize that temperatures may not be consistent throughout the mat, particularly during cold-weather compaction. As the mix is placed on a cold subgrade, the temperature at both the top and bottom may vary from the temperature at the center of the mat. This variation will impact the compactability of the mix.

Balancing the production rates of the plant, paver, and roller is vital in cold-weather compaction situations. By establishing the speed at which the paver and roller should operate to handle the mix being delivered by the plant, the contractor can ensure that the paver and the roller are being operated in a manner whereby optimum density can be achieved. NAPA provides guidance in determining balance between plant, paver, and roller elements of the paving operation (*8*).

The concept of time available to compact (TAC) is the most important issue of this discussion. The concept highlights the relationship between mix temperature, base temperature, mat thickness, and the time available for the densification process to occur. The original thermal computations by Dickson and Corlew developed cooling curves that were published in "Thermal Computations Related to the Study of Pavement Compaction Cessation Requirements" (*7*) and again in "Cold Weather Compaction" (*1*). These cooling curves have been simplified as shown in Figure 2 (*9*).

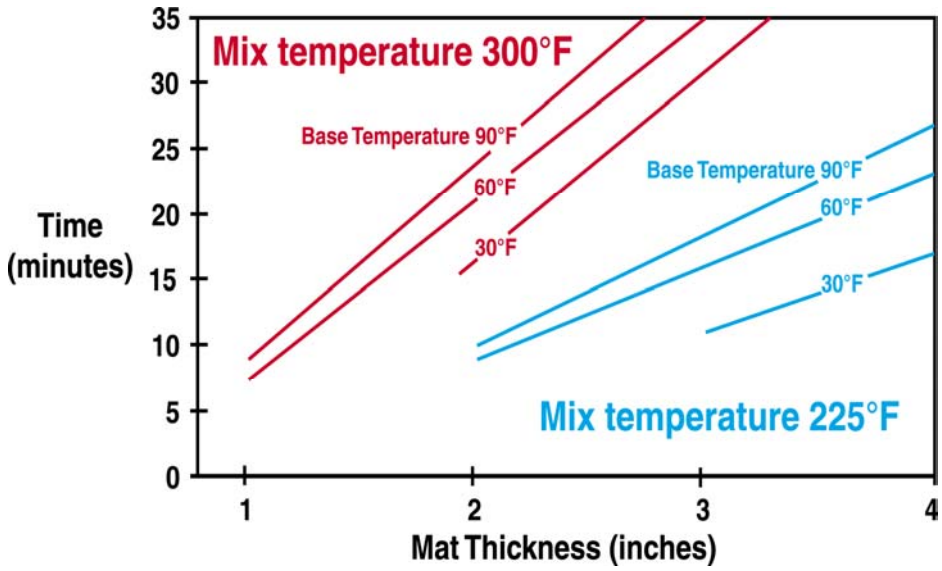

FIGURE 2 Time available to compact.

Referencing Figure 2, consider a mix temperature of 300°F, a base temperature of 90°F, and a mat thickness of 2 in. For these conditions, the roller operator would have approximately 23 min to complete densification before the mix became too cold, as shown in the curves of Figure 2. For the same conditions with a 30°F base temperature, the roller operator would have about 15 min. If however, the mix temperature was 225°F and a 90°F base temperature, the time available to compact drops to about 10 min for a 2-in. mat as shown in the Figure 2. Even at a base temperature of 60°F, the roller operator has only about 8 min to complete compaction. If the base temperature was 30°F, the lift thickness would need to increase to about 3 in. in order to have about 10 min to complete the densification process. The TAC is the most important issue for the achievement of proper densification in the field. Setting arbitrary dates or temperatures for cessation of paving operations does not acknowledge the fundamental temperature relationships necessary to properly achieve densification.

Table 1 presents a set of information developed from "Cold Weather Compaction" (*1*). The recommended Minimum Laydown Temperatures are based on the work of Dickson and Corlew (*7*). It is noted that placement of lifts less than 3 in. thick at temperatures below freezing is not recommended.

TABLE 1 Recommended Minimum Laydown Temperature (*1*)

	Lift Thickness			
Base Temp, °F	**1 in.**	**1.5 in.**	**2 in.**	**≥ 3 in.**
20–32	—	—	—	285*
33–40	—	305	295	280
41–50	310	300	285	275
51–60	300	295	280	270
61–70	290	285	275	265

* Only on treated base materials

CONCLUDING OBSERVATIONS

Proper density in the HMA pavement is the single most important determinant of the performance of the pavement. The three most important issues for achieving density are temperature, temperature, and temperature. In many instances, owners request or demand that mix be placed in cold-weather conditions.

While placement of HMA can be successfully accomplished in cold-weather conditions, there are risks involved. The owner and contractor need to understand the risks involved and understand that the risk may not be justified. In some cases, cold-weather paving simply may not be the right decision. In other situations, the owner may understand the risk and choose to pave for safety or business reasons, fully understanding that the performance life of the pavement may be compromised.

Placing HMA in cold-weather conditions is inevitable. Project planning and heightened attention to detail during the construction operation are vital in cold-weather operations. The owner and contractor must work together to ensure good quality materials and placement are achieved.

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Recent Advances in Compaction Equipment, Including Intelligent Compaction

RECENT ADVANCES IN COMPACTION EQUIPMENT,
INCLUDING INTELLIGENT COMPACTION

Vibratory Rollers

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This paper offers a brief history of evolution of vibratory rollers for use on hot-mix asphalt (HMA) compaction applications. Primary topics discussed include rolling patterns to achieve uniform pavement density and smoothness with attention to determining the ability of the vibratory breakdown roller to keep up with the paving spread. The growth of interest in HF (high frequency) vibratory rollers is analyzed, with explanation of the benefits of high vibration frequency to agency, contractor, and motoring public. Drum impact spacing is presented as one of the primary factors to achieve smoothness of HMA pavements, with mention of the relationship of drum dimensions and weight as related to roller design. Also presented is practical information on the effect of drum amplitude in the compaction process and how variable amplitude can add versatility to double drum vibratory roller performance. The importance of rolling within the proper temperature during the TAC (time available for compaction) is presented. Current innovations of machine automation are noted, as well as planned future developments that will provide value to agency and contractor alike. The importance of operator safety equipment is noted. The paper concludes with a look into the future of vibratory compactors.

HISTORY OF VIBRATORY ROLLERS

Vibratory rollers have been used for compaction of HMA pavements for nearly 40 years. The first units were single-drum vibratory soil compactors that were modified for use on HMA by changing tires from treaded to smooth and adding a water spray system for tires and drum. Double-drum (DD) vibratory rollers were introduced in the 1970s to provide greater efficiency and productivity on HMA laydown and compaction applications.

One of the author's first major projects with his current employer was to help introduce DD asphalt compactors in the United States. Our mentor was Victor Berhondo. Vic taught us practices that guided our recommendations to contractors on HMA applications. His first rule was logical: always operate any vibratory compactor at its highest frequency. The implications of his advice were not fully understood at that time, but the advice was remembered and has been passed on with the passion he taught. From a practical perspective, vibration frequency really controls productivity of any compactor. This is certainly true on HMA projects where a breakdown roller is required to keep up with the paver spread. In truth, the popularity of HF vibratory rollers beginning during the mid-1990s is a direct consequence of the need for higher production breakdown rollers. Vic's second rule has also passed the test of time: selecting the lowest amplitude consistent with achieving target density. He knew that creating excessive drum force, by selecting amplitude too high for the material layer thickness or mix properties risks aggregate damage

ROLLING PATTERNS AND DRUM-WIDTH-TO-PANEL-WIDTH RELATIONSHIP

The evolution of double drum vibratory compactors has followed industry needs. Consider this typical application. A paver is laying an HMA panel 12 ft (3.65 m) wide, 3 in. (75 mm) thick. It is being fed by a fleet of haul trucks, delivering sufficient hot mix to the paver to permit 40 ft (12.2 m) per minute continuous speed. In order for any breakdown roller to keep up with this paver, it must be capable of an average rolling speed of 250 ft (76.2 m) per minute when making a five-pass rolling pattern.

To understand how compaction equipment rolling widths have evolved, consider the following examples. For a breakdown roller, there are a number of currently manufactured double drum rollers with rolling widths of approximately 66 in. (1,675 mm), 78 in. (1,980 mm) or 84 in. (2,135 mm) to choose from. The breakdown roller might be any brand of appropriate size and weight. Remember that any compactor's effective rolling width on the panel may not be the same as its measured drum width.

If a breakdown roller has 66-in. (1,675-mm) wide drums, on most panels its effective drum width is about 10% narrower. Where supported edges require overlap, effective rolling width may be between 60 to 63 in. (1,525 to 1,600 mm) even under the control of the most experienced operators. Therefore, to uniformly cover a paved panel 12 ft (3.65 m) wide, this size roller requires more than two side-by-side effective machine passes to completely cover the paved panel. As depicted in Figures 1, 2, 3, and 4, the center section of the panel is not rolled by the return passes made along left and right sides of the panel. This may require additional rolling through the center of the panel. (By definition, a pass is a roller movement from point A to point B in one direction. Returning to the starting point is a second pass.) The rolling pattern for this compactor is often described as a seven-pass pattern.

To achieve higher project productivity, contractors often utilize compactors with wider drums. Rollers with 78.7 in. (2 m) and 84-in. (2,135-mm) wide drums are available from nearly all manufacturers. Effective rolling widths for the intermediate drum width models is normally 72 to 75 in. (1,830 to 1,905 mm); the widest drums have 78 to 81 in. (1,980 to 2,055 mm) effective drum width. Compactors with wider effective drum width are able to uniformly cover 12 ft (3.65 m) wide panels in two side-by-side passes. To achieve target density normally requires two coverages. In addition, a make-up pass is made to move forward on the panel to the next section to be rolled, as depicted in Figure 5. This creates what is termed a five-pass rolling pattern.

HIGH FREQUENCY

To determine appropriate rolling speed for our breakdown roller we use vibration frequency and drum diameter as control factors. The vibration frequency of ten-ton class DD vibratory rollers varies between 2,400 to 4,200 vibrations per minute (40 to 70 Hertz). With their intermediate drum diameter, these rollers need to produce drum impact spacing of 12 impacts per foot (~39 impacts per meter) to achieve spec-compliant smoothness. Dividing vibration frequency by drum impact spacing yields rolling speed. The vibratory roller with 2,400 vpm (40 Hertz) frequency can roll (with vibration) at a speed of 200 ft per minute (61 m per minute). If required to make seven passes to effectively cover the panel, the roller with 2,400 vpm (40 Hertz) vibration frequency cannot keep up with the paver moving 40 ft per minute.

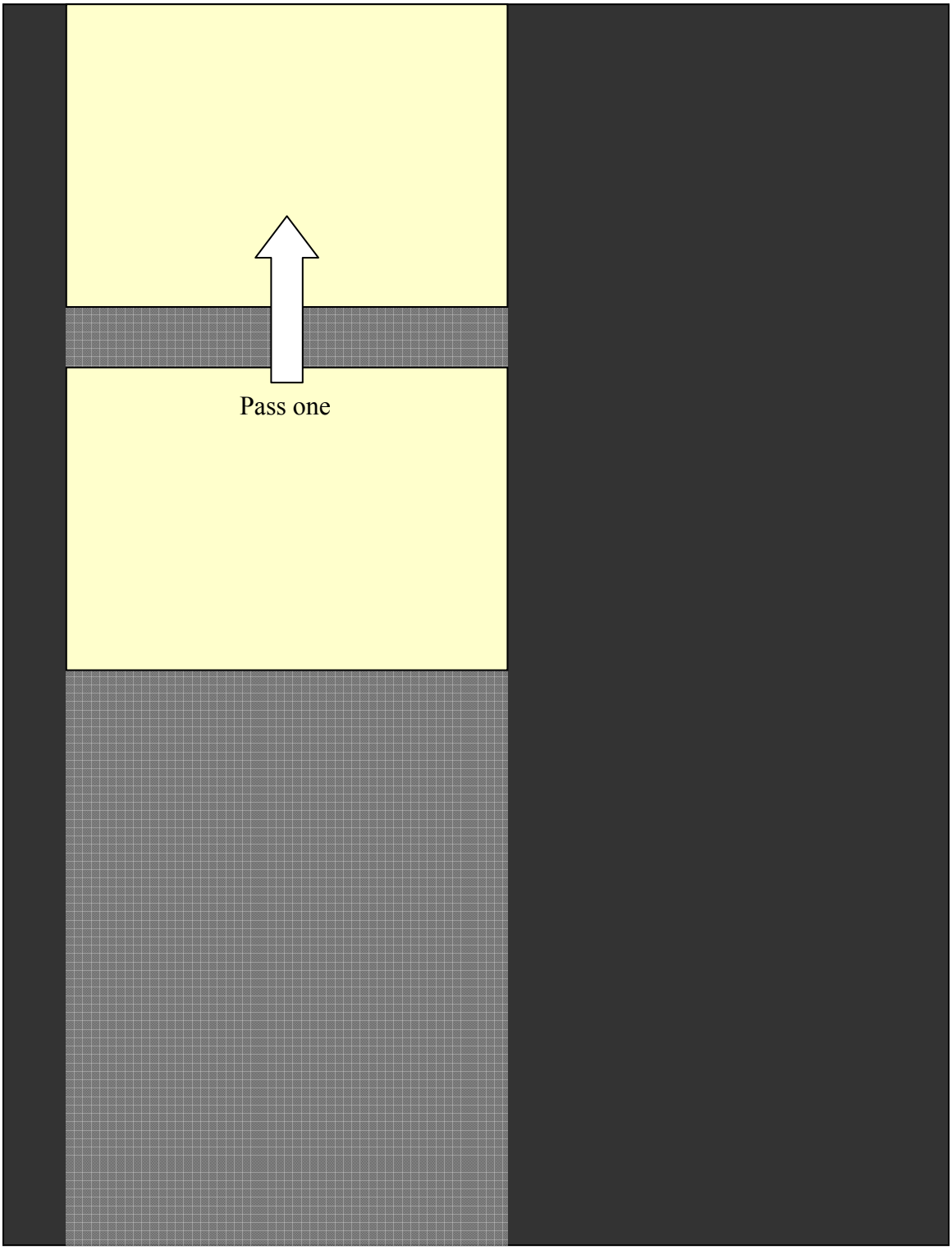

FIGURE 1 DD vibratory roller making first pass toward paver on left section of panel, staying ~6 in. (150 mm) away from unsupported left edge of paved panel. Panel width is 12 ft (3.65 m); drum width is 66 in. (1,675 mm).

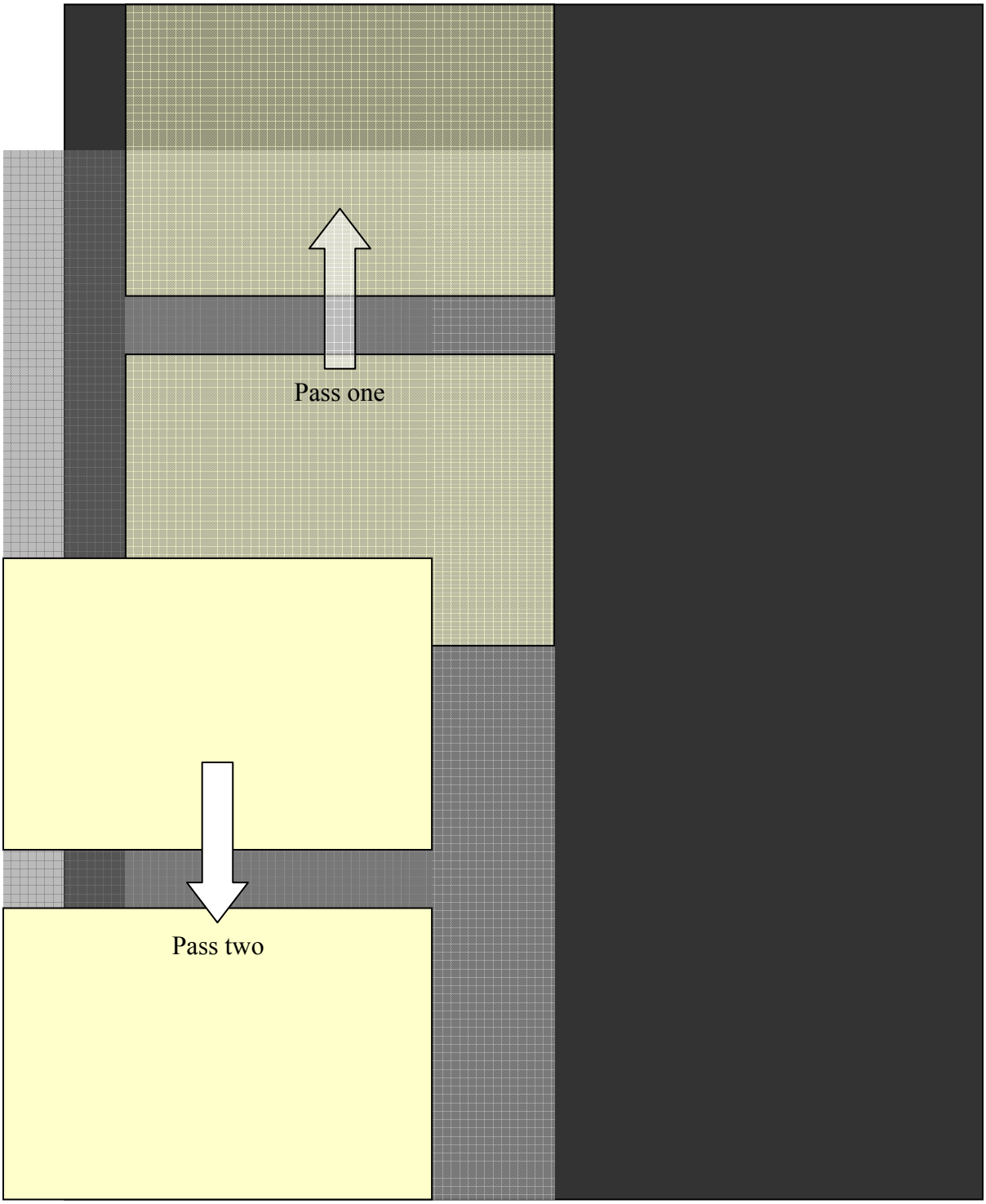

FIGURE 2 DD vibratory roller making second pass away from paver on left section of panel, overhanging left edge of paved panel ~6 in. (150 mm). Panel width is 12 ft (3.65 m); drum width is 66 in. (1,675 mm).

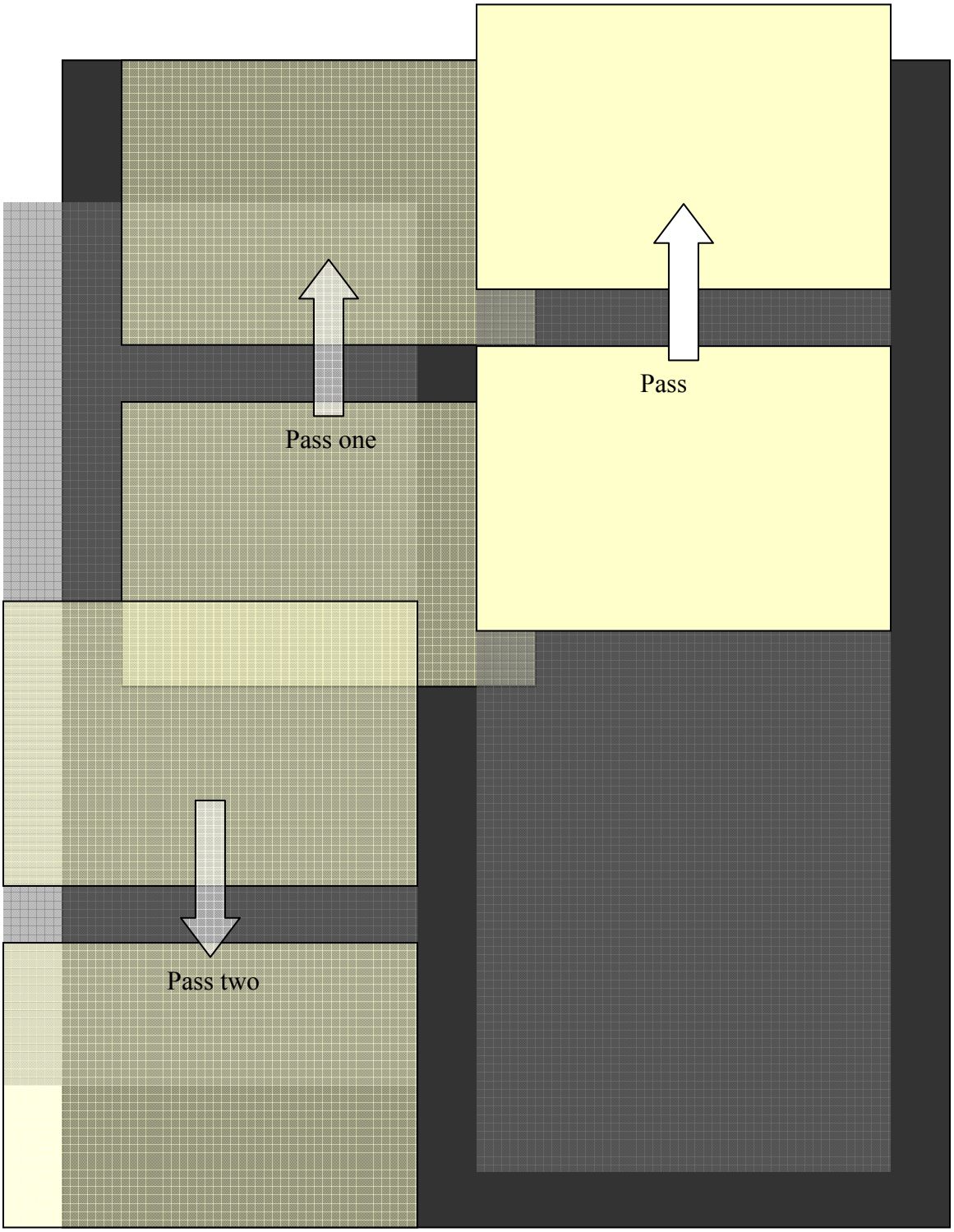

FIGURE 3 DD vibratory roller making third pass toward paver on right section of panel, staying away from unsupported right edge of paved panel ~6 in. (150 mm). Panel width is 12 ft (3.65 m); drum width is 66 in. (1,675 mm).

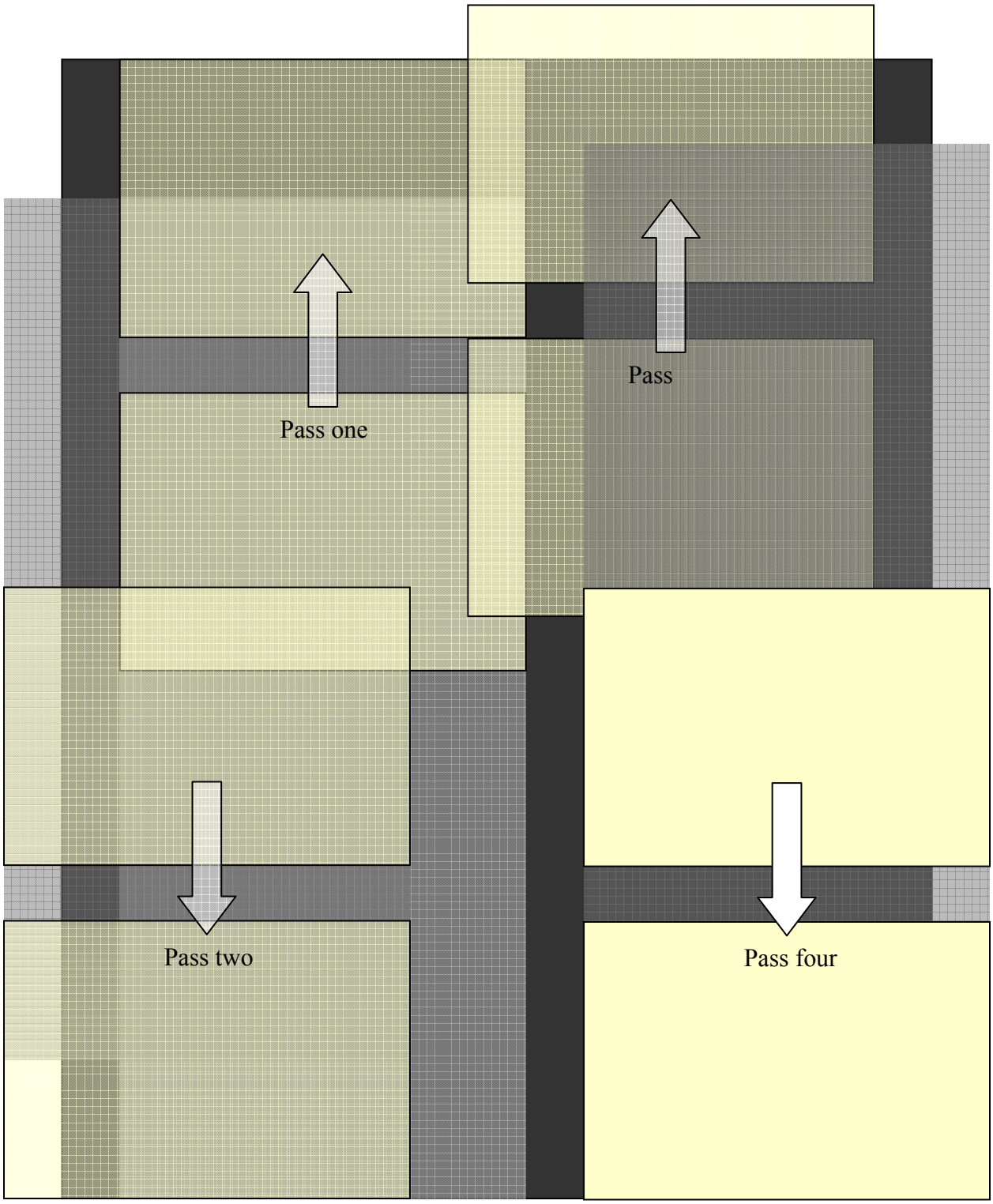

FIGURE 4 DD vibratory roller making fourth pass away from paver on right section of panel, overhanging right edge of paved panel ~6 in. (150 mm). Panel width is 12 ft (3.65 m); drum width is 66 in. (1,675 mm).

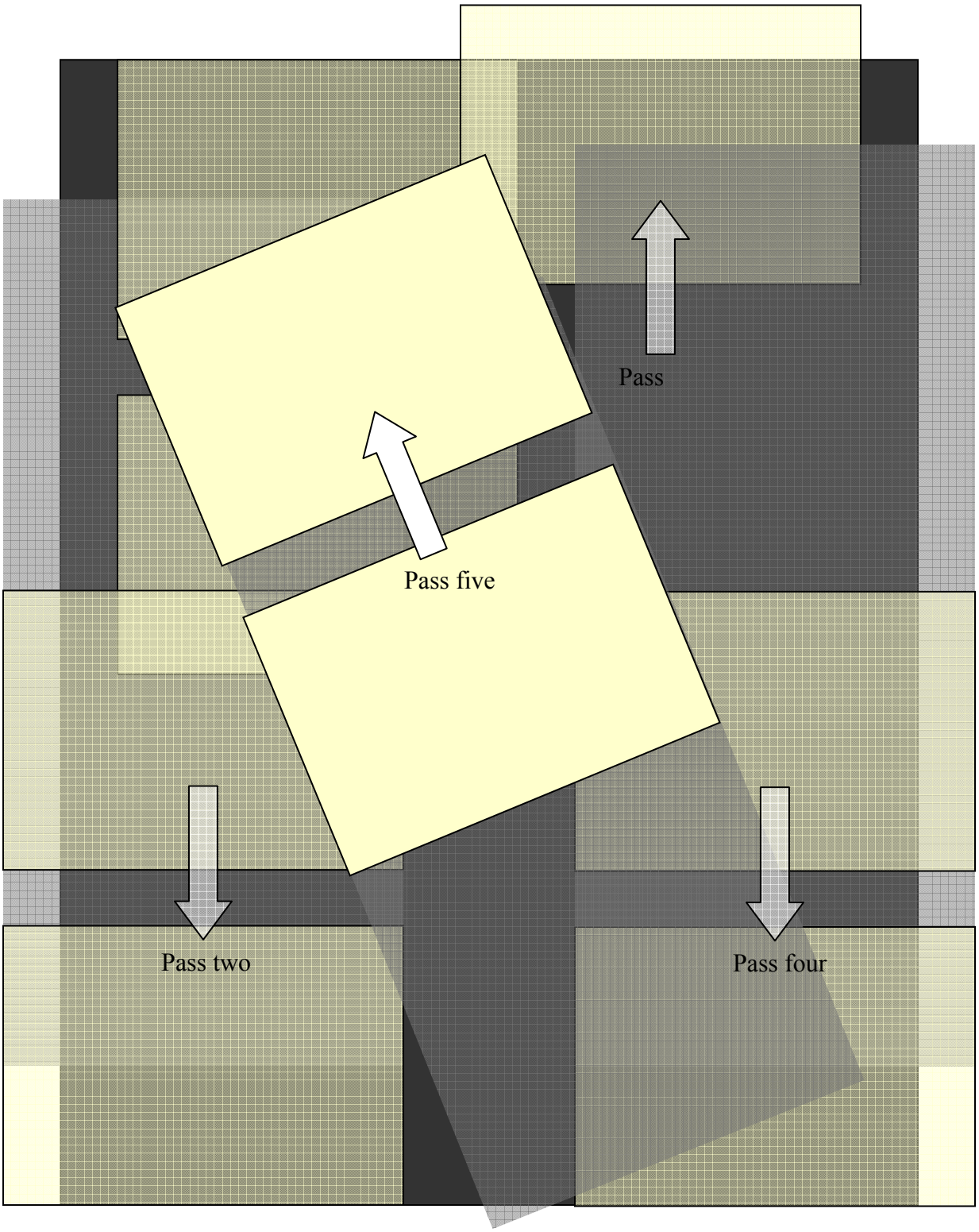

FIGURE 5 DD vibratory roller making fifth (make-up) pass toward paver, moving from right to left across panel, to resume original first pass in rolling pattern. Panel width is 12 ft (3.65 m); drum width is 66 in. (1,675 mm).

In the mid-to-late 1990s, many vibratory compactor manufacturers introduced units with 3,600 vpm (60 Hertz) or higher frequency. Consider a vibratory roller of the same physical size as the previous example, but with 3,600 vpm (60 Hertz) frequency. This higher frequency roller can complete its seven-pass pattern while maintaining average forward speed of nearly 43 ft per minute (13.1 m per minute) due to its 3,600 vpm (60 Hertz) frequency. High frequency enables this roller to keep pace with the paver with no sacrifice in pavement surface smoothness. All vibratory roller manufacturers now produce high-frequency DD rollers. Regardless of the panel width being paved, there are HF rollers of every preferred brand from which a contractor may choose. Refer to Table 1 to review the effect of vibration frequency on rolling speed with vibration.

DRUM IMPACT SPACING AND NIJBOER'S FACTOR

Drum impact spacing was mentioned briefly during earlier comments on rolling patterns—a very important factor for vibratory compactors. Every vibratory roller drum creates a depression or impression in the pavement surface with each cycle of vibration frequency. As the unbalanced weights inside the roller's drums spin they cause the drums to lift and drop through a dimension termed amplitude. (Amplitude will be explained in the next section of this paper.) Each impact of the drums creates a pavement surface depression with its dimension dependent upon measurable factors. A Dutch scientist named Nijboer developed a formula that predicts the relative tendency of a roller drum to build-up a bow wave in front of its drums. This formula represents the relationship

TABLE 1 Vibration Frequency Versus Rolling Speed with Vibration

Vibration Frequency	Rolling Speed with Vibration	
[basis 14 impacts/ft, 46/m]	**Miles per hour**	**Kilometers per hour**
4,200 vpm (70 Hertz)	3.4	5.5
3,600 vpm (60 Hertz)	2.9	4.7
3,000 vpm (50 Hertz)	2.4	3.9
2,400 vpm (40 Hertz)	1.9	3.1
[basis 12 impacts/ft, 39/m]		
4,200 vpm (70 Hertz)	4.0	6.4
3,600 vpm (60 Hertz)	3.4	5.5
3,000 vpm (50 Hertz)	2.8	4.5
2,400 vpm (40 Hertz)	2.3	3.7
[basis 10 impacts/ft, 33/m]		
4,200 vpm (70 Hertz)	4.8	7.7
3,600 vpm (60 Hertz)	4.1	6.6
3,000 vpm (50 Hertz)	3.4	5.5
2,400 vpm (40 Hertz)	2.7	4.3
[basis 8 impacts/ft, 26/m]		
4,200 vpm (70 Hertz)	6.0	9.7
3,600 vpm (60 Hertz)	5.1	8.2
3,000 vpm (50 Hertz)	4.3	6.9
2,400 vpm (40 Hertz)	3.4	5.5

between axle load, drum width, and drum diameter. These same factors also determine the roller's tendency to create severe pavement surface depressions that may contribute to roughness and inability to meet specified smoothness.

Practice shows that optimum vibratory drum impact spacing can be adjusted narrower or wider primarily based on compactor drum diameter. Drum diameter controls the length of arc of drum contact with the pavement surface. A rule-of-thumb is to recommend 14 drum impacts per foot [~46 impacts per meter] for drums up to 39.4 in. (1 m) in diameter and 12 impacts per foot [~39 impacts per meter] for drums between 39.4 and 47.2 in. (1.0 to 1.2 m) in diameter. Larger diameter drums permit wider drum impact spacing. Ten impacts per foot (~33 impacts per meter) are recommended for drums between 47.2 and 55.1 in. (1.2 to 1.4 m) in diameter. Drums greater than 55.1 in. (1.4 m) in diameter can roll with vibration at 8 to 9 impacts per foot (~26 to 30 impacts per meter) and provide pavement surface smoothness. Refer to Table 2 for impact spacing recommendations based on drum diameter.

DRUM AMPLITUDE SELECTION

All highway class DD vibratory compactors are designed to permit selection of drum amplitude. Remember that drum amplitude primarily controls the penetration depth of vibratory compaction forces. In general terms, roller manufacturers recommend lower amplitude settings for efficient compaction of thinner HMA layers or for vibratory compaction of sensitive materials. Higher amplitude settings are recommended for effective compaction of thicker pavement layers or for more difficult-to-compact materials. The majority of compaction equipment manufacturers produce rollers with two amplitude selections, low and high. A few manufacturers produce machines with three or more selections of amplitude. Industry practice suggests that vibratory compactors with two amplitudes can perform the majority of HMA compaction applications. This is certainly true when these rollers can breakdown roll in the effective compaction temperature zone and when ambient conditions are favorable. Compaction difficulties arise, however, when ambient or project conditions deteriorate in such a manner that time available for compaction is short. Being able to adjust drum amplitude more than into low and high selections may make the difference between achieving target density and failure. There is no magic number of amplitude selections that should be available or all manufacturers would have the same design. Currently, one manufacturer offers either four or five amplitudes on their large DD vibratory rollers; another offers eight amplitudes on most of their large models.

TABLE 2 Drum Impact Spacing Recommendations

Drum Diameter		Vibratory Impacts	
inches	**millimeters**	**per foot**	**per meter**
< 39.4	< 1,000	14	46
39.4–47.2	1,000–1,200	12	39
47.3–55.1	1,201–1,400	10	33
> 55.1	> 1,400	8–9	26–30

Because of the design of vibratory roller eccentrics (the unbalanced weight assemblies that create the forces of vibration), there is necessity to reduce vibration frequency whenever drum amplitude is increased. This protects the bearings (on which the eccentric weights spin) from premature failure. Vibratory roller design engineers know that there is a relationship between eccentric mass, radius of eccentricity and rotational speed that cannot be exceeded without risk of component failure during use. Speed influences bearing life to a greater degree than mass or eccentricity. This is why HF rollers are designed with lower drum amplitude than their normal frequency counterparts. Please note that all vibratory roller designers use the same formula to calculate the amplitude of the vibrating drum. A second formula is used to calculate centrifugal force—sometimes referred to as dynamic force. Both formulas incorporate eccentric moment (the product of eccentric mass and radius of eccentricity) in the equation. Only centrifugal force, however, considers vibration frequency in its calculation.

In practice, the compaction of HMA pavement mixes is performed according to several basic metrics. Typical factors taken into account when setting up the roller for a project include width of paved panel, thickness of panel being laid, target density, and productivity requirements. Adjustments to drum amplitude, and to the amount of generated dynamic force produced by the vibrating drum, are most commonly made to match a vibratory roller's performance to the thickness of the paved layer and to mix stiffness or tenderness. Selection between rollers of different rolling widths is also done to satisfy special compaction requirements on some applications. For example, sometimes relatively soft limestone aggregates are used in asphalt mixes. When compacting mixes produced with these softer aggregates, drum amplitude is often reduced so that the aggregate is not fractured. This is particularly true when rolling mixes with stone-on-stone contact like stone mastic asphalt (SMA) and some Superpave designs. Some contractors frequently choose vibratory rollers that offer more than two amplitude selections so that they can fine tune compaction forces and minimize aggregate damage. This is advantageous to both agency and contractor. In reality, having multiple amplitude selections can be a benefit; it is rarely a detriment to producing high-quality HMA pavements.

TEMPERATURE ZONE FOR ROLLING

In the good old days conventional HMA pavements were the norm, designed using either the Hveem or Marshall methods. Since the development of the Superpave design system, compaction requirements have changed. SMA mixes have also raised the bar for compaction equipment performance requirements. Staying in the proper temperature zone for rolling is more important with today's mixes than ever before. Aggregate and binder temperatures have increased at the plant, especially for PG asphalt cements. The goal is to lay down these newer mixes at temperatures above 300°F (150°C) so that there is sufficient TAC before the mixes stiffen and air voids cannot be removed. Some contractors struggle with mixes that exhibit tenderness within a certain temperature range; this too reduces TAC. Breakdown rolling needs to be accomplished quickly before mix cools below 250°F (120°C). Many roller manufacturers now equip their larger DD models with infrared temperature sensors so that the breakdown or intermediate roller operators can avoid the tender zone. Knowing pavement surface temperature also helps operators avoid creating pavement surface distress that can be caused by improper roller operation. Providing roller operators with hand-held infrared thermometers can also help.

WHEN TO VIBRATE

The question is often asked: when is it appropriate or necessary to engage vibration on the DD vibratory compactor? The answer is simple; vibrate unless there is a compelling reason not to vibrate. Remember that the design of modern DD rollers incorporates a number of automatic technologies to make the task simpler for the operator. Any compactor's ability to remove voids from a pavement layer is inversely proportional to mix viscosity. Therefore, when using vibratory rollers it is almost always recommended to make the first pass behind the paver with vibration engaged. It is recommended to continue rolling with vibration until target density has been reached. Vibrating after target air void content has been achieved is rarely a good idea due to the possibility of damage to aggregate or binder structure.

AUTOMATIC CONTROLS

The roller operator's responsibilities today, using a high-performance vibratory compactor, are really no different than they were when HMA pavements were compacted with the three-roller train; a three-wheel static followed by a pneumatic followed by a static tandem. What has changed is the emphasis on achieving target density and smoothness metrics while maintaining highest possible production. For this reason, vibratory compaction equipment manufacturers have spent considerable effort designing machines with automatic control systems.

Consider, for one example, one manufacturer's newest product introductions in the large DD product category. It's produced in three different sizes (with two different rolling widths) weighing 11, 13, and 15 tonnes. Each unit incorporates seven different automatic systems to help ensure correct machine performance under a variety of operating conditions. These include

- Automatic vibration start and stop, tied to machine rolling speed—this prevents an inexperienced or inattentive operator from vibrating when the roller is stationary.
- Automatic drum impact spacing, selectable for 8, 10, 12, or 14 impacts per foot (26, 33, 39, or 46 impacts per meter)—this optimizes productivity while reducing risk of missing smoothness requirements.
- Automatic vibration frequency selection, related directly to drum amplitude setting—this adjusts frequency up in lower amplitude selections and down in higher amplitudes to provide longest component life with no compromise in roller performance.
- Automatic sequential drum vibration—the lead drum begins vibrating first with the trailing drum's vibration engagement delayed a couple of seconds to reduce engine power demand, noise level of vibration system, and also prevents vibrating on the cold panel when rolling night joints or other cold to hot panel transitions.
- Automatic maximum rolling speed limit—this prevents the inexperienced operator from rolling too fast with vibration and creating surface depressions that might lead to negative smoothness incentives.
- Automatic drum wetting system start and stop—this makes certain the spray system operates whenever the roller is in motion and stops water spray when the roller is parked.
- Automatic flow control of drum wetting system—this sprays more water onto the drums' surfaces at faster rolling speeds and less water at slower speeds to conserve water use and minimize the cooling the pavement surface by the water from the drum wetting system.

Most compaction equipment manufacturers provide similar automatic control functions for their double drum vibratory rollers. Everyone knows that rollers are the final tool in the paving train and compaction can either make or break a paving contractor's financial success on any project.

OPERATOR SAFETY AND COMFORT

Job site safety is not optional; safety is one of the overriding concerns on every paving project. Equipment manufacturers incorporate safety features into every machine they design. For safety reasons, as example, every ride-on roller is equipped with either ROPS (roll-over protective structure) or ROPS/FOPS (falling objects protective structure) and seat belt. Similarly, for safety reasons, all pinch points and reciprocating parts like engine cooling fans are protected with guards and are labeled with warning decals.

Manufacturers also pay attention to operator comfort. They design operator stations so that controls are within the zones of comfort and reach for operators of a wide range of body dimensions and weights. They make certain that the operator has a clear view of the work zone; they also pay attention to the operator's view along the sides of the compactor so that productivity is enhanced and the possibility for damage is minimized to the machine or surroundings. They even make creature comforts like enclosed cabins with heating and air conditioning available as options on most compaction equipment.

The modern vibratory roller is a high-production tool. It is designed with features that provide practical benefits, both to agencies and to contractors. HF vibratory rollers are able to maintain pace with fast-moving paving spreads. SMA and Superpave mixes can be compacted to high density with exceptional surface smoothness, meeting or exceeding all agency specifications. Roller operators are provided with a safe and productive machine; high product reliability keeps the paving train moving for highest possible tonnage processed each day. Practical automation reduces operator strain and assures consistent pavement quality. And more is yet to come!

A very recent development has been the non-nuclear density-testing device. This work has been pioneered by TransTech Industries. They have a commercial device that is hand-held and eliminates the regulatory controls necessary for nuclear densometers. The TransTech PQI (pavement quality indicator) has been shown to be accurate and very user-friendly. One manufacturer is currently marketing a roller-mounted version of the PQI on selected models of its large DD vibratory rollers. This device is shown in the photographs in Figure 6. Results obtained during the first season of availability have been very good, with high correlation between the density readings from the roller-mounted device compared to the handheld PQI. The present model requires contact between the sensor and pavement surface; the manufacturer is jointly working on a non-contact device to determine its accuracy and dependability.

CONCLUSION

The vibratory roller is a relatively new construction tool, having been in use for 40 years or so. Its value to the paving contractor, however, has been proven on projects around the world. To achieve density and pavement surface smoothness, properly operated vibratory rollers will

FIGURE 6 Roller-mounted PQI density measuring device and operator display unit

always provide superior results compared to the traditional three-roller train. Their operation incorporates practical automation and features that provide for safety and productivity. New developments are in continuous evolution. Vibratory rollers are the contractor's choice wherever and whenever HMA pavements are rolled.

RECENT ADVANCES IN COMPACTION EQUIPMENT,
INCLUDING INTELLIGENT COMPACTION

Oscillatory Compaction of Hot-Mix Asphalt

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Historically, compaction of hot-mix asphalt (HMA) pavements was done with steel wheel static rollers until vibratory rollers, introduced in the late 1950s, became the dominant compaction equipment. As difficulties in compacting Superpave mixes were encountered during the 1980s and 1990s, equipment manufacturers made their vibratory rollers bigger, heavier, and with higher vibration frequencies. In some cases, these rollers were so powerful that, during compaction, mix aggregates were fractured, the asphalt mat cracked, and damage to nearby buildings and underground utilities occurred. This latter problem had been evident earlier in European countries due to their narrower city streets and the proximity of many old, historic buildings. Thus oscillation technology was developed in Germany in 1983 and after years of testing and usage in Europe, the oscillatory asphalt roller was introduced in the United States in early 2003.

The oscillatory system uses dual, opposed, eccentric weights rotating in the same direction around the roller drum axis to produce a rocking motion. This rocking motion produces horizontal and downward shear forces that achieve greater compaction by "massaging" the HMA —even at lower mix temperatures. Since the drum does not leave the pavement surface or bounce like a conventional vibratory roller drum, the compacted mat surface is smooth and flat and there is no damage to utilities, buildings, or bridges. Numerous tests have shown that oscillatory compaction can achieve higher densities with fewer passes when compared to a vibratory roller. Plus compaction is possible at lower temperatures (down to 150°F).

INTRODUCTION

The key to the good performance and long life in an asphalt concrete pavement is achieving proper mix density during construction. In the HMA compaction process, three very important things occur: the asphalt-coated aggregates are pressed together, air voids are reduced, and mix density increases.

- The squeezing together of the HMA aggregates increases their surface to surface contact and interparticle friction. This results in increased mix stability and greater structural strength of the pavement.
- Reducing air voids to an optimal level (about 3% to 8%) produces a pavement that is nearly impermeable—preventing air and water from entering the mix. Thus asphalt hardening, stripping and freeze–thaw damage are minimized.
- Finally, if the rollers don't achieve a high density, subsequent traffic may further consolidate the mix in the wheelpaths producing ruts that could become a safety hazard.

In the early days of asphalt pavement construction, compaction was by static steel wheel rollers. The compactive effort results from the downward force of the roller's weight—typically 8 to 12 tons (7 to 11 tonnes). See Figure 1*a*.

In the late 1950s, vibratory rollers were introduced. The vibration is generated by an eccentric weight revolving at a high speed inside the drum. The drum is actually lifted off the surface and then forced downward to impact the mix being compacted (see Figure 1*b*). The compactive effort is a function of the roller weight, the amplitude (or height) of the roller movement and number of vibrations per minute (frequency) of the drum. The forward speed of the vibratory roller and its frequency must be coordinated so that there are at least 10 impacts per foot (33 impacts per meter). This produces the maximum mix density and a smooth riding surface. Vibratory rollers account for the majority of all compaction equipment sold worldwide and have provided excellent results for many years.

However, during the late 1980s and early 1990s, as the result of an extensive research effort under the SHRP, the Superpave asphalt mix design system was developed for use in the United States. These Superpave mixes addressed the three 1980s dominant HMA pavement distresses of deformation/rutting, fatigue cracking, and low-temperature cracking. Mixes were

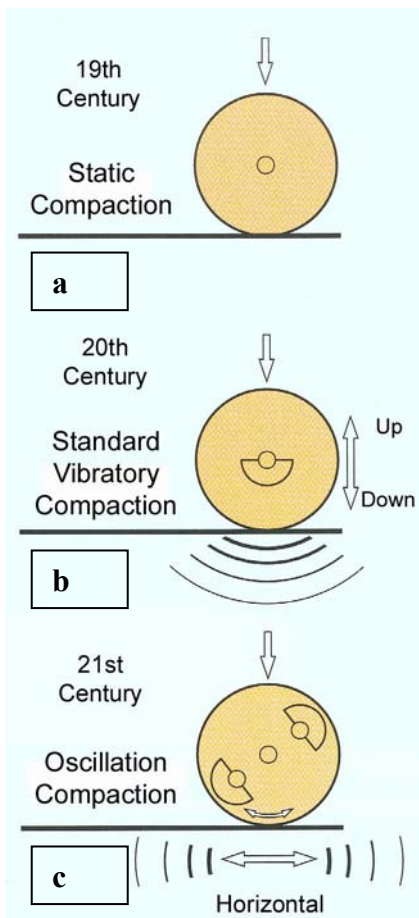

FIGURE 1 Three compaction systems for asphalt rollers.

designed to accommodate the expected traffic loading and the historical climatic conditions at the pavement's location. These new mixes, while rut resistant, proved to be difficult to compact due to the higher crushed aggregate content and the use of stiffer asphalts. Some mixes also had a "tender zone" where compaction was difficult within a certain temperature range. Equipment manufacturers tried to address these problems by making heavier rollers with higher vibration frequencies. These more powerful rollers increased the compactive effort but in many cases also caused fracturing of mix aggregates, cracking in the compacted mat, and damage to nearby buildings and buried utilities.

Prior to Superpave, several European countries were experiencing similar HMA compaction problems. However, the narrowness of many of their city streets and the closeness of historical buildings to these streets made use of larger vibratory rollers impractical in many instances. Since oscillation compaction, which had just been developed, was non-aggressive and compacts partly by the static weight of the roller and partly through horizontal shear forces (as opposed to vertical impacts), it offered promise that density could be achieved—even in these difficult situations.

OSCILLATION COMPACTION

Oscillation technology had been developed in Germany in 1983 and tested as a method of compacting asphalt pavement. The oscillatory system uses dual, opposed, eccentric weights rotating in the same direction around the drum axis. The rotating eccentrics cause the drum to rapidly move in a forward–backward rocking motion but still remain in constant contact with the pavement surface. This rocking motion causes a horizontal shear force that realigns aggregates and with the static load continuing to move aggregate vertically, greater compaction is achieved in a shorter time. Since there is no vertical impact of the drum onto the mat surface, aggregate fracture, mat cracking and damage to surrounding structures does not occur. And, because the drum does not bounce like a vibratory drum, the mat surface is smooth and flat. Figures 1 and 2 show schematically how the rotating weights produce the oscillation motion.

The oscillatory roller, shown in Figure 3, was introduced to the United States construction industry at the 2003 World of Asphalt Show in Nashville, Tennessee, by Hamm AG. It has an operating weight of 10 to 11 tons (9 to 10 tonnes) and a roller width of 66 in. (1.7 m). The front drum is a standard vibratory drum and the rear has the oscillatory system. Vibration is usually only used on the first few passes and the oscillation drum should always do the final compaction.

The reduced vibrations and lower noise levels of the oscillatory roller results in improved comfort for the operator and throughout the surrounding residential area. Oscillatory compaction also is the best system to use when paving HMA on bridges. A longer roller life and less maintenance problems are added benefits.

FIELD DENSITY TESTS

Density tests on various HMA mixes on airports, Interstate highways, and state routes have shown that the oscillatory roller can achieve higher densities when vibratory rollers could not. In many instances, the oscillatory roller improved the density from the "acceptable" to the "bonus payment" range. Figure 4 presents typical density test results.

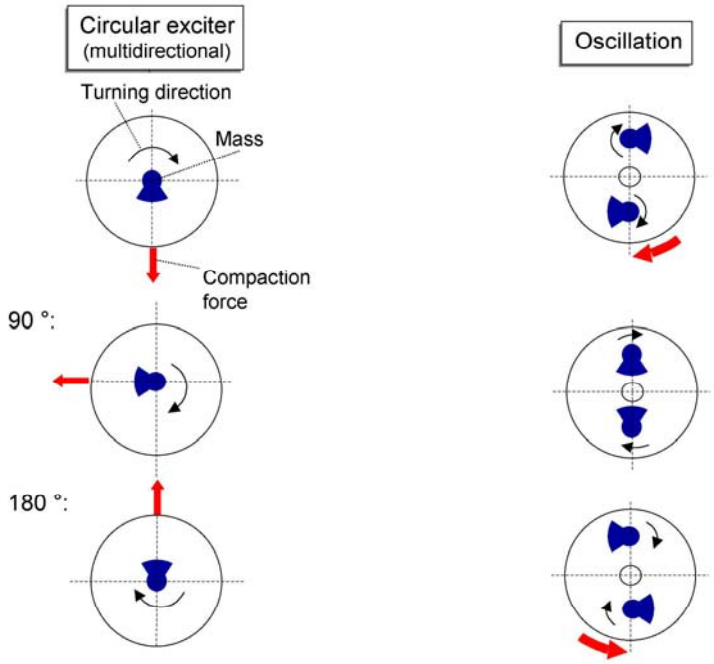

FIGURE 2 Producing the oscillation motion.

FIGURE 3 Oscillatory roller.

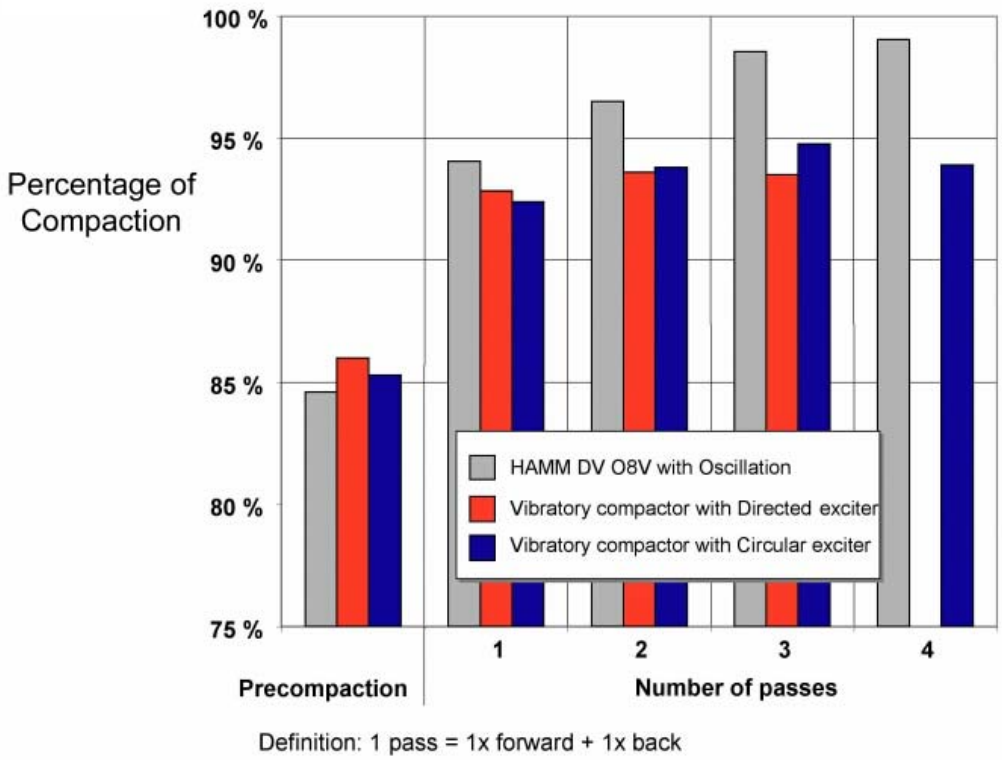

FIGURE 4 Comparison of vibratory versus oscillatory rollers.

CONCLUSIONS

Oscillation technology, when applied to asphalt compactors, has proved to be extremely valuable. The oscillatory rocking motion of the roller drum produces a shear force on the HMA surface that realigns mix aggregates horizontally while the roller weight applies a simultaneous vertical compactive force. With a variety of HMA mix types, the oscillatory roller has been able to increase mat densities higher than vibratory rollers. In fact, additional vibratory roller passes many times resulted in density decreases. Unlike the vibratory roller, the oscillatory roller drum never lifts off the pavement surface. Thus, there is no mix aggregate fracturing, mat cracking, or damage to underground utilities or nearby buildings. And the final pavement surface is smooth and flat with an excellent ride.

Vibratory Pneumatic Tire Roller

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In order to improve the efficiency of the compaction process for both stiff and tender Superpave mixtures, a vibratory pneumatic tire (VPT) roller was developed by Sakai Heavy Industries, Ltd., in 2002. The roller has been used on various types of paving projects in the United States since the first production unit was introduced in March 2003. For the evaluations conducted on normal paving operations, the VPT roller was used in the intermediate rolling position after breakdown rolling by a high-frequency double-drum vibratory (DDV) roller. Based on the evaluation of results obtained at San Francisco International Airport, King City, California, and Traverse City, Michigan, the following conclusions were reached. First, the minimum required density of the pavement was significantly exceeded with a combination of a high-frequency DDV roller and a VPT roller. Second, a vibratory pneumatic roller weighing only 9,350 kg (20,580 lb) and making six roller passes at medium amplitude achieved the same level of density as a conventional static pneumatic tire (SPT) roller weighing 27,270 kg (60,000 lb) and making 12 roller passes. It is obvious that the VPT roller is more versatile and efficient than a much heavier SPT roller. Third, the density distribution measured by cores cut from the pavement show that the density in middle or bottom portion is higher than that in top portion. It is believed that the density in top portion was lower due to excessive compactive effort applied during breakdown rolling when the pavement surface temperature was relatively low.

INTRODUCTION

With the increasing use of Superpave designed hot-mix asphalt (HMA) mixes in the United States, many paving contractors have encountered difficulties in achieving the required level of compaction. Many Superpave HMA mixes are very stiff and difficult to compact due to the high percentage of crushed aggregate in the mix. In other cases, coarse-graded Superpave HMA mixes that are designed with aggregate gradations below the maximum density line exhibit tender behavior and move or shove under the applied compactive effort (*1, 2*).

A number of factors directly affect the ability of an asphalt paving contractor to obtain the required level of density in a HMA mixture. Those factors are air temperature, base temperature, mix temperature, layer thickness, and wind velocity (*3*). In most cases, it is necessary to complete the compaction process before the surface temperature of the HMA mix reaches 80°C (176°F). For example, for a mix temperature at the paver screed of 150°C (302°F), an air temperature of 15°C (59°F), a base temperature of 15°C (59°F), and a layer thickness of 50 mm (2 in.), the time available to compact the mix before a temperature of 80°C (176°F) is reached at 18 min. However, for a mix temperature at the paver screed of 120°C (248°F), an air temperature of 5°C (41°F), a base temperature of 5°C (41°F), and a layer thickness of 25 mm (1 in.), the time available to compact the HMA mix before it cools to 80°C (176°F) is only 3 min (*4, 5, 6*).

As the time available to densify the HMA mix decreases, the speed of the compaction equipment must increase accordingly. The challenge is that there is an upper limit to what most rollers can do to simultaneously achieve both density and smoothness of the new pavement layer. In order to achieve the proper compaction of both stiff and tender mixes within the time available for compaction, a high-frequency DDV roller operated at a frequency of 4,000 vibrations per minute (vpm) was developed by Sakai Heavy Industries, Ltd. Early test results with the high frequency DDV roller, conducted in Japan in 1998 and in the United States since 2001, indicate that a greater degree of density can be obtained with the same number of roller passes, or the same degree of density can be obtained with fewer roller passes, when a high-frequency DDV roller is used in place of a DDV roller operated at a lower (normal) vibratory frequency (*7*).

In order to achieve compaction on tender Superpave HMA mixtures, a VPT roller was developed by the same company (*8*). This roller is unique in the fact that the seven pneumatic tires on the roller vibrate at various amplitude settings, providing a combination of vibratory compactive effort along with the kneading action of conventional pneumatic tires. Normal SPT rollers achieve compaction through a combination of wheel load and tire inflation pressure (*9, 10, 11*). As the wheel load increases, with the same tire inflation pressure, the compactive effort is extended deeper into the pavement layer. As the inflation pressure increases, with the same wheel load, the compactive effort is also extended further down into the pavement layer. The kneading action of the pneumatic tires increases the density of the mix and reduces the permeability of the pavement layer by "tightening up" the surface texture. It is expected that the use of a VPT roller will both improve compaction efficiency and decrease the permeability of the HMA mixture by the dynamic, instead of static, kneading effect.

Many pneumatic tire rollers currently in use are in the weight range of 8,000 kg to 15,000 kg (17,620 lb to 33,000 lb) in an unballasted and ballasted condition, respectively. Some of the larger pneumatic tire rollers vary in size from 12,000 kg to 27,270 kg (26,400 lb to 60,000 lb) in an unballasted and ballasted condition, respectively. With the advent of the large DDV rollers and problems with pickup of HMA mixes on the pneumatic tires, many of the larger pneumatic tire rollers have disappeared from paving sites in the United States. In recent years, however, the use of pneumatic tire rollers has increased after they were found to be very effective for compacting the Superpave HMA mixes (*12, 13*).

The development of a VPT roller for use in compacting HMA paving materials began in 1995 in response to a request to be able to compact a newly designed asphalt emulsion cold mix that contained portland cement as an additive to provide a stiffer mix. Many trials were conducted to find a way to compact the new emulsified cold mix using conventional vibratory rollers equipped with different vibratory systems such as a radial vibratory system with a single eccentric shaft, a vertical vibration system with dual eccentric shafts, and a nutation vibratory system. All of these rollers, however, produced hairline cracks on the surface of the asphalt emulsified cold mix that were unacceptable to the highway agency. The first prototype of a VPT roller was equipped with a set of VPT that replaced the front steel drum on a small DDV roller. This new roller was tested on the asphalt emulsion cold mix and on several HMA mixtures. Encouraging results led to the continued development of a VPT roller.

The second prototype of the VPT roller was a full-scale model weighing 8,000 kg (17,600 lb). Initially, the set of four pneumatic tires on the rear end of the roller vibrated with no vibration on the front tires. The roller, however, was found to efficiently compact not only the asphalt emulsion cold mix, but also roller compacted concrete (RCC) pavement, as well as HMA mixtures. The third prototype was also a full-scale model with four lightweight tires on each of

the front and rear axles, with the tires on one axle placed so that they would overlap the gaps between the tires on the other axle, for a total of eight tires on this prototype roller. This prototype was also used to compact various types of paving materials. The use of an even number of pneumatic tires on both axles was not desirable, however. Finally, a new VPT roller was developed, the GW750, which has an odd number of tires—seven in total.

A series of tests was conducted to evaluate the compactive effort of the VPT roller with two other rollers. The first was a Sakai SW850, a DDV roller operated at a vibratory frequency of 67.7 Hz (4,000 vpm). The second was a Sakai TS650, a SPT roller weighing 22,850 kg (50,270 lb) (*8*). Based on these test results, it was concluded that the use of a VPT roller appears to provide the "best of both worlds." The VPT roller allows for the compaction of a tender Superpave HMA mix in the middle temperature zone without the shoving or movement of the mix usually associated with the use of a DDV roller. In addition, it is expected that the permeability of the pavement surface will be lower with the use of the VPT roller than with the use of a DDV roller. Further, the combination of the kneading action of the pneumatic tires along with a vibratory compactive effort enables a reduction in the size of the pneumatic tire roller needed to achieve the same degree of density.

In order to evaluate the applicability of a VPT roller for various types of HMA mixes in the United States, several extensive field trials were conducted on three different projects. The first was at the San Francisco International Airport. The second was on a city street in King City, California. The third was on M37 in Traverse City, Michigan. In these evaluations, a VPT roller was used for intermediate rolling after breakdown compaction was accomplished using a high-frequency DDV roller. In addition, for the test at San Francisco International Airport, the compactive effort of the VPT roller was compared with that of a SPT roller weighing 27,270 kg (60,000 lb).

TEST RESULTS

Test Project 1a: San Francisco International Airport, California

In June 2003 a VPT roller was used on a Graniterock Company paving project at San Francisco International Airport. The purpose of this test was to evaluate the applicability of a VPT roller in combination with a high-frequency DDV roller, and to compare the compactive effort of a VPT roller with that of a conventional SPT roller. The first test project, 60 m (200 ft) long by 3.6 m (12 ft) wide, was constructed as part of a runway overlay project at the intersection of two primary runways. The existing surface pavement layer was milled to depths varying from 75 to 100 mm (3 to 4 in.), 60 m (200 ft) long by 90 m (300 ft) wide and then an asphalt emulsion tack coat was applied.

The HMA mixture specified by FAA had a nominal maximum aggregate size (NMAS) of 19 mm (¾ in.). The job mix formula, including aggregate gradation, voids in the mineral aggregate (VMA), binder content, asphalt binder type, and theoretical maximum density (TMD) is shown in Table 1. The HMA was hauled by bottom-dump trucks, picked up by a windrow elevator, and placed 3.6 m (12 ft) wide by a Cat AP1055B paver. The nominal compacted layer thickness was 100 mm (4 in.).

Mix surface temperature behind the paver screed was approximately 140°C (284°F). The first breakdown roller pass was made when the mix surface temperature was approximately

TABLE 1 Job Mix Formula for FAA Type 1-in.-Maximum Marshall Mix

	Sieve Size		Percent Passing
Aggregate Gradation	25 mm	1 in.	100
	19.0 mm	¾ in.	97.0
	12.5 mm	½ in.	83.0
	9.5 mm	⅜ in.	73.0
	4.75 mm	#4	52.0
	2.36 mm	#8	36.0
	1.18 mm	#16	27.0
	600 um	#30	18.0
	300 um	#50	11.0
	150 um	#100	6.0
	75 um	#200	3.8
Mix Properties	VMA (%)		13.8
	Binder content (%)		4.6
	Asphalt binder type		AR 8000
	TMD g/cm^3 (lb/ft^3)		2.569 (160.4)

110°C (230°F), intermediate rolling occurred when the mix temperature was approximately 90°C (194°F), followed by finish rolling at 80°C (176°F). It is noted that due to operational constraints, breakdown rolling was significantly delayed which resulted in much lower mix temperatures when compaction operations started.

As indicated in Table 2, during construction of the first test strip, a Sakai SW900, a high-frequency DDV roller, made six breakdown passes. This roller was operated at a speed of 4.8 km/h (3.0 mph), an amplitude of 0.33 mm (0.013 in.), and a frequency of 67.7 Hz (4,000 vpm). For the intermediate rolling, a VPT roller, a Sakai GW750 weighing 9,350 kg (20,580 lb) made six passes. The VPT roller was operated at a speed of 4.8 km/h (3.0 mph), an amplitude of 0.38 mm (0.015 in.), and a frequency of 40 Hz (2,400 vpm). For the finish rolling, a DDV roller, a Sakai SW850, made two vibratory and four static passes.

Density of the HMA pavement after each roller pass was measured by a Troxler 4640B, thin-lift nuclear density gauge since cores could not be cut from the test section. The gauge was used at three different random locations along the 60-m- (200-ft-) long test section. Density measurement for 30 s was repeated twice at the same location by rotating the gauge 180 degrees. When measured values between each gauge reading at the same location were greater than 0.05 g/cm^3 (3.1 lb/ft^3) additional measurements were taken.

In Table 2, information is presented on rolling patterns, percent of TMD with standard deviation, and the number of measurement locations. It was found that a VPT roller was very useful for compacting the HMA mix since the minimum required density level of 92.0% was easily exceeded when this roller was used in combination with the DDV roller. The DDV roller in the breakdown position achieved 90.2% of TMD after six vibratory passes. The VPT roller, in the intermediate position, increased density 2.6%, up to 92.% after six passes. After finish rolling by the second DDV roller making two vibratory and four static passes, the density level increased 1.0% up to 93.8% of TMD.

Pickup of HMA mixes on the pneumatic tires occurred at a mix surface temperature of approximately 95°C (203°F). To prevent the pickup, a release agent was used at a dilution ratio

TABLE 2 Rolling Patterns and Percent of TMD for FAA Type 1-in.-Maximum Marshall Mix

Rolling Patterns					
Rolling Process	Roller Models	Operating Variables			Number of Roller Passes
Breakdown	SW900	Speed	km/h (mph)	4.8 (3.0)	6 Vibratory
		Amplitude	mm (in.)	0.33 (0.013)	
		Frequency	Hz (vpm)	67.7 (4,000)	
Intermediate	GW750	Speed	km/h (mph)	4.8 (3.0)	6 Vibratory
		Amplitude	mm (in.)	0.38 (0.015)	
		Frequency	Hz (vpm)	40 (2,400)	
Finish	SW850	Speed	km/h (mph)	4.8 (3.0)	2 Vibratory & 4 Static
		Amplitude	mm (in.)	0.33 (0.013)	
		Frequency	Hz (vpm)	67.7 (4,000)	
Percent of Theoretical Maximum Density (TMD) by Nuclear Density Gauge					
	Density g/cm^3 (lb/ft^3)	TMD g/cm^3 (lb/ft^3)	Percent of TMD	Standard Deviation	Number of Measurement Locations
Breakdown	2.318 (144.7)	2.569 (160.4)	90.2	0.56	6
Intermediate	2.384 (148.8)		92.8	0.56	8
Finish	2.411 (150.5)		93.8	0.98	8

of 100 parts water to 1 part release agent, and sprayed continuously over the tires through the water spray system.

Test Project 1b: San Francisco International Airport, California

Based on the results of the first test project described above, further evaluation was conducted to compare the VPT roller weighing 9,350 kg (20,580 lb) with a conventional SPT roller weighing 27,270 kg (60,000 lb). Both of the rollers were used in the intermediate rolling position. Breakdown rolling was again done using a DDV roller, a Sakai SW900. This DDV roller was operated at a speed of 4.8 km/h (3.0 mph), an amplitude of 0.33 mm (0.013 in.), and a frequency of 67.7 Hz (4,000 vpm). Mix temperatures for this test project were the same as for the Test Project 1a. After the compaction, process was finished, density measurements were conducted randomly at 25 and 15 locations for the VPT and SPT roller sections, respectively. At each location, every measurement was repeated twice. Although the use of cores to measure the density would have been desirable, the nuclear density evaluation method was chosen because the limited time frame in the airport paving schedule would not allow enough time to extract cores.

In Table 3, information on the rolling patterns and percent of TMD with standard deviations based on the gauge readings in the two different sections is presented. The percent of

TABLE 3 Rolling Patterns and Percent of TMD Comparing the VPT and SPT Rollers for FAA Type 1-in.-Maximum Marshall Mix

	Rolling Patterns					
	Rolling Process	Roller Models	Operating Variables			Number of Roller Passes
VPT Roller Section	Breakdown	SW900	Speed	km/h (mph)	4.8 (3.0)	6 Vibratory
			Amplitude	mm (in.)	0.33 (0.013)	
			Frequency	Hz (vpm)	67.7 (4,000)	
	Intermediate	GW750	Speed	km/h (mph)	4.8 (3.0)	6 Vibratory
			Amplitude	mm (in.)	0.38 (0.015)	
			Frequency	Hz (vpm)	40 (2,400)	
	Finish	SW850	Speed	km/h (mph)	4.8 (3.0)	6 Static
SPT Roller Section	Breakdown	SW900	Speed	km/h (mph)	4.8 (3.0)	6 Vibratory
			Amplitude	mm (in.)	0.33 (0.013)	
			Frequency	Hz (vpm)	67.7 (4,000)	
	Intermediate	CP271	Speed	km/h (mph)	4.8 (3.0)	12 Static
	Finish	SW850	Speed	km/h (mph)	4.8 (3.0)	4 Vibratory & 4 Static
			Amplitude	mm (in.)	0.33 (0.013)	
			Frequency	Hz (vpm)	67.7 (4,000)	
Percent of Theoretical Maximum Density (TMD) by Nuclear Density Gauge						
		Density g/cm^3 (lb/ft^3)	TMD g/cm^3 (lb/ft^3)	Percent of TMD	Standard Deviation	Number of Measurement Locations
VPT Roller		2.404 (150.1)	2.569 (160.4)	93.6	0.87	25
SPT Roller		2.412 (150.5)		93.9	1.16	15

TMD measured in the VPT roller section, where the roller was operated at amplitude of 0.38 mm (0.015 in.), was essentially the same as for the SPT roller section—93.6% versus 93.9%. The VPT roller achieved that level of density using only half the number of roller passes as the SPT roller did—six versus 12 passes. The total number of roller passes in the VPT roller section was also 30% less than that of the SPT roller section (18 versus 26 passes). Based on previous test results (*8*), it was found that the compactive effort of a VPT roller operated with amplitude of 0.71 mm (0.028 in.) instead of amplitude of 0.38 mm (0.015 in.) was greater than that of a SPT roller weighing more than twice as much.

Test Project 2: King City, California

In July 2003, a VPT roller was used on a pavement reconstruction project on Main Street in downtown King City, California, by the Graniterock Company, as shown in Figure 1. The existing asphalt pavement was removed and replaced with two lifts of HMA pavement directly

FIGURE 1 VPT roller on the King City, California, project.

on an aggregate subbase material. Testing of the VPT roller was conducted on the 70 mm (2-3/4 inches) thick HMA base course.

As indicated in Table 4, the HMA was a 19-mm (¾-in.)-Maximum Medium, Type A, Caltrans Section 39 mix. The asphalt binder used was an AR 4000 at 5.0% by dry weight of aggregate or 5.3% by weight of mix. The HMA mix was delivered by bottom-dump trucks and picked up by a windrow elevator. The mix was placed 3.6-m (12-ft) wide by a Cat AP100A paver operated at a speed of 6 m/min (20 ft/min).

The surface temperature of the HMA mix behind the paver screed ranged between 140°C and 120°C (284°F and 248°F). The first breakdown pass by the DDV roller was made when the mix surface temperature ranged between 120°C and 100°C (248°F and 212°F). The intermediate rolling by the VPT roller occurred when the mix surface temperature ranged between 95°C and 85°C (203°F and 185°F). This low temperature for the intermediate rolling was due to a problem with pickup of the mix on the pneumatic tires, even though a release agent was used. The finish rolling was made using the DDV roller in the static mode at a mix surface temperature of approximately 80°C (176°F).

As indicated in Table 5, three different rolling patterns were employed in order to evaluate the compactive effort of the VPT roller by changing the roller combinations and number of roller passes. Two different rolling patterns, referred to here as Pattern A and B, used three different rollers: a Sakai SW850 DDV roller for breakdown rolling, a Sakai GW750 VPT roller in the intermediate position, and a Cat CB534 DDV roller for finish rolling. The total number of passes by these three rollers was kept at 10 over each point of the pavement surface. For example,

TABLE 4 Job Mix Formula for 19-mm (¾-in.)-Maximum Medium, Type A, Caltrans Section 39

	Sieve Size		Percent Passing
Aggregate Gradation	25 mm	1 in.	100
	19.0 mm	¾ in.	95.2
	12.5 mm	½ in.	81.8
	9.5 mm	⅜ in.	70.8
	4.75 mm	#4	49.0
	2.36 mm	#8	35.5
	1.18 mm	#16	27.0
	600 um	#30	18.7
	300 um	#50	13.1
	150 um	#100	7.2
	75 um	#200	5.0
Mix Properties	Binder content (%)*		5.0
	Asphalt binder type		AR 4000
	TMD g/cm^3 (lb/ft^3)		2.557 (159.6)

* Binder content is percent by weight of dry aggregate.

in Pattern A, four roller passes each were made by the SW850 and the GW750, and then two passes were made by the CB534. In Pattern B, the SW850 and GW750 made six and two roller passes, respectively, and then the CB534 made two passes.

The SW850 was operated at a speed of 4.8 km/h (3.0 mph), an amplitude of 0.33 mm (0.013 in.), and a frequency of 67.7 Hz (4,000 vpm). The GW750 was operated at a speed of 4.8 km/h (3.0 mph), an amplitude of 0.38 mm (0.015 in.), and a frequency of 40 Hz (2,400 vpm). The CB534 was operated at a speed of 3.2 km/h (2.0 mph) in static mode.

Another rolling pattern, Pattern C, used only the SW850 for breakdown rolling and the CB534 for finish rolling. The SW850 made six passes at a speed of 4.8 km/h (3.0 mph), an amplitude of 0.33 mm (0.013 in.), and a frequency of 67.7 Hz (4,000 vpm). The CB534 made two roller passes in the static mode at a speed of 3.2 km/h (2.0 mph). A total of only eight roller passes were made over each point of the pavement.

As described in the column labeled "Total" located in the lower portion of Table 5, the percent of TMD based on cores for all three rolling patterns was generally very high and reached a level of approximately 95% of TMD. This indicates that a combination of a high-frequency DDV roller and a VPT roller is very beneficial in achieving the required density. The highest density level of 95.7% was obtained in Pattern A where four, four, and two passes were made by the SW850, GW750, and CB534, respectively. With the same total number of roller passes of 10 in Pattern A and B (four, four, and two passes versus six, two, and two passes), the percent of TMD in Pattern A was slightly higher than that measured in Pattern B—95.7% versus 95.4%. This implies that two more roller passes by the VPT roller in the intermediate roller position used in Pattern A is more effective in increasing the density than two more roller passes of the DDV breakdown roller used in Pattern B. Between Pattern B and C, two passes by a VPT roller in the intermediate roller position used in Pattern B increased the percent of TMD slightly from 95.3% to 95.4%.

TABLE 5 Rolling Patterns, Percent of TMD, and Distribution of TMD in Each of the Three Layers

	Rolling Patterns					
	Rolling Process	Roller Models	Operating Variables			Number of Roller Passes
Pattern A	Breakdown	SW850	Speed	km/h (mph)	4.8 (3.0)	4 Vibratory
			Amplitude	mm (in.)	0.33 (0.013)	
			Frequency	Hz (vpm)	67.7 (4,000)	
	Intermediate	GW750	Speed	km/h (mph)	4.8 (3.0)	4 Vibratory
			Amplitude	mm (in.)	0.38 (0.015)	
			Frequency	Hz (vpm)	40 (2,400)	
	Finish	CB534	Speed	km/h (mph)	3.2 (2.0)	2 Static
Pattern B	Breakdown	SW850	Speed	km/h (mph)	4.8 (3.0)	6 Vibratory
			Amplitude	mm (in.)	0.33 (0.013)	
			Frequency	Hz (vpm)	67.7 (4,000)	
	Intermediate	GW750	Speed	km/h (mph)	4.8 (3.0)	2 Vibratory
			Amplitude	mm (in.)	0.38 (0.015)	
			Frequency	Hz (vpm)	40 (2,400)	
	Finish	CB534	Speed	km/h (mph)	3.2 (2.0)	2 Static
Pattern C	Breakdown	SW850	Speed	km/h (mph)	4.8 (3.0)	6 Vibratory
			Amplitude	mm (in.)	0.33 (0.013)	
			Frequency	Hz (vpm)	67.7 (4,000)	
	Intermediate	---	Speed	km/h (mph)	--	0
	Finish	CB534	Speed	km/h (mph)	3.2 (2.0)	2 Static

Percent of Theoretical Maximum Density (TMD) by Nuclear Density Gauge						
	Positions of Density in Cores	Density g/cm^3 (lb/ft^3)	TMD g/cm^3 (lb/ft^3)	Percent of TMD	Standard Deviation	Number of Cores
Pattern A	Total	2.447	2.557	95.7	1.0	5
		152.8				
	Top	2.409		94.2	1.5	5
		150.4				
	Middle	2.468		96.5	1.2	
		154.0				
	Bottom	2.455		96.0	0.7	
		153.2				
Pattern B	Total	2.439		95.4	0.6	2
		152.3				
	Top	2.401		93.9	0.9	2
		149.9				
	Middle	2.457		96.1	1.1	
		153.4				
	Bottom	2.455		96.0	0.6	
		153.2				
Pattern C	Total	2.437	159.6	95.3	0.3	2
		152.1				
	Top	2.393		93.6	0.7	2
		149.4				
	Middle	2.455		96.0	0.4	
		153.2				
	Bottom	2.444		95.6	0.1	
		152.6				

Cores cut from the 70-mm (2¾-in.) thick pavement layer were sawed horizontally into three slices of 25 mm, 25 mm, and 20 mm (1.0 in., 1.0 in., and ¾ in.). The density distributions are presented in Table 5 in the columns labeled "Top," "Middle," and "Bottom." The highest density was measured in the middle layers for all three different rolling patterns. In Figure 2, a "bottom-up" type of density distribution is shown, in which the degree of density in the top portion of the cores is lower than the density in the bottom and middle portions. The difference in density between the top and the middle portion of cores is very significant—2.3% (94.2 versus 96.5), 2.2% (93.9 versus 96.1), and 2.4% (93.6 versus 96.0) in Patterns A, B, and C, respectively. It is believed that the density in top portion of the pavement layer was decreased due to the excessive compactive effort applied during the breakdown rolling at a relatively low pavement temperature. The degree of density in the middle and bottom potion of the layer was increased due to the compactive effort of the DDV and VPT rollers. A similar density distribution was reported in the previous research that evaluated the density distributions obtained using three different rollers (*8*).

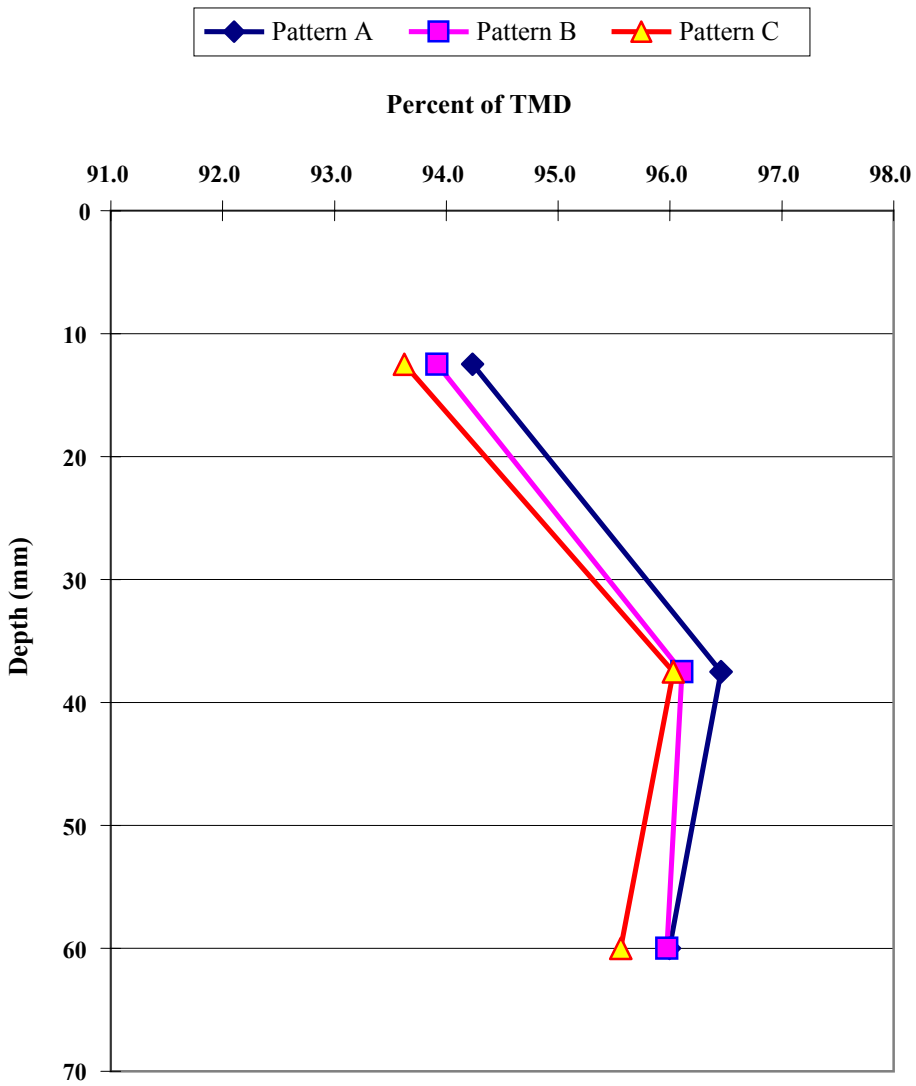

FIGURE 2 Distribution of percentage of the TMD in each of the three layers.

Test Project 3: M37 in Traverse City, Michigan

In September 2004, a VPT roller was used to compact a HMA leveling course mix on Michigan State Route M37 near Traverse City. The evaluation was conducted during normal paving operations with a combination of a high-frequency DDV breakdown roller, a Sakai SW850, the VPT roller in the intermediate position, and an Ingersoll-Rand DD130 finish roller, operated both in the vibratory and static mode.

Table 6 presents the job mix formula data including aggregate gradation, VMA, binder content, asphalt binder type, and TMD. The HMA mix was designed and produced by Team Elmer's. The mix was hauled in live-bottom trucks and discharged directly into the Blaw–Knox PF5510 asphalt paver. The mix was placed 5.1-m (17-ft) wide and 60-mm (2.4-in.) deep over a stabilized aggregate base. Mix surface temperature behind the paver screed was approximately 140°C (284°F). The first breakdown pass by the high-frequency DDV roller was made when the mix surface temperature was approximately 125°C (257°F). The intermediate rolling by the VPT roller occurred when the mix surface temperature was approximately 90°C (194°F). This delay was due to a problem with pickup of mix on the pneumatic tires even though a release agent was used. The finish rolling was carried out by the DDV roller when the surface temperature was approximately 80°C (176°F).

As indicated in Table 7, a Sakai SW850 roller was used for breakdown rolling and made five, six, and eight vibratory passes over each point of the pavement surface at a speed of 5.8 km/h (3.6 mph), an amplitude of 0.33 mm (0.013 in.), and a frequency of 67.7 Hz (4,000 vpm). Intermediate rolling was done with two or four vibratory passes by the VPT roller operating at a speed of 5.0 km/h (3.1 mph), an amplitude of 0.71 mm (0.028 in.), and a frequency of 40 Hz (2,400 vpm). Finish rolling done by an Ingersoll–Rand DDV roller at a speed of 5.0 km/h (3.1 mph), an amplitude of 0.48 mm (0.019 in.), and a frequency of 42 Hz (2,500 vpm).
The percent of TMD based on core density is presented in the column labeled "Total" in the lower portion of Table 7. It indicates that a combination of a high-frequency DDV breakdown roller and a VPT intermediate roller is very effective in meeting and exceeding the required density. Pattern A achieved the highest level of density at 94.1% of TMD. In comparing Pattern A with Pattern B, the density increased with an increase in the number of roller passes of the VPT roller after the same number of breakdown roller passes (six) with the DDV roller. In comparing Patterns B, C, and D with the number of breakdown roller passes of six, five, and eight, respectively, a similar density level was achieved for all three different number of breakdown roller passes.

Cores cut from the 50 mm (2.0 in.) thick pavement layer were sawed horizontally into two slices of 25 mm and 25 mm (1.0 in. and 1.0 in.). In all cores, it is observed that the minimum required density level of 92% was achieved throughout the cores and the density levels in top portion of the cores were always lower than those in bottom portion, as shown in Figure 3. This "bottom-up" type density distribution mirrors the previous results measured on the King City project. It is also believed that the density in the top portion was lower due to an excessive number of roller passes by the DDV roller in the breakdown rolling position at a relatively low pavement surface temperature. For example, the minimum density difference of 0.3% between in the top and bottom portion of the cores was measured in Pattern C where the minimum of five breakdown passes was made.

TABLE 6 Job Mix Formula for Michigan DOT Type 4E3L Mix

	Sieve Size		Percent Passing
Aggregate Gradation	19.0 mm	¾ in.	100.0
	12.5 mm	½ in.	96.0
	9.5 mm	⅜ in.	89.0
	4.75 mm	#4	70.3
	2.36 mm	#8	53.7
	1.18 mm	#16	40.9
	600 um	#30	30.2
	300 um	#50	15.5
	150 um	#100	7.0
	75 um	#200	5.0
Mix Properties	VMA (%)*		14.6
	Binder content (%)		5.6
	Asphalt binder type		PG64-28
	TMD g/cm^3 (lb/ft^3)		2.459 (153.5)

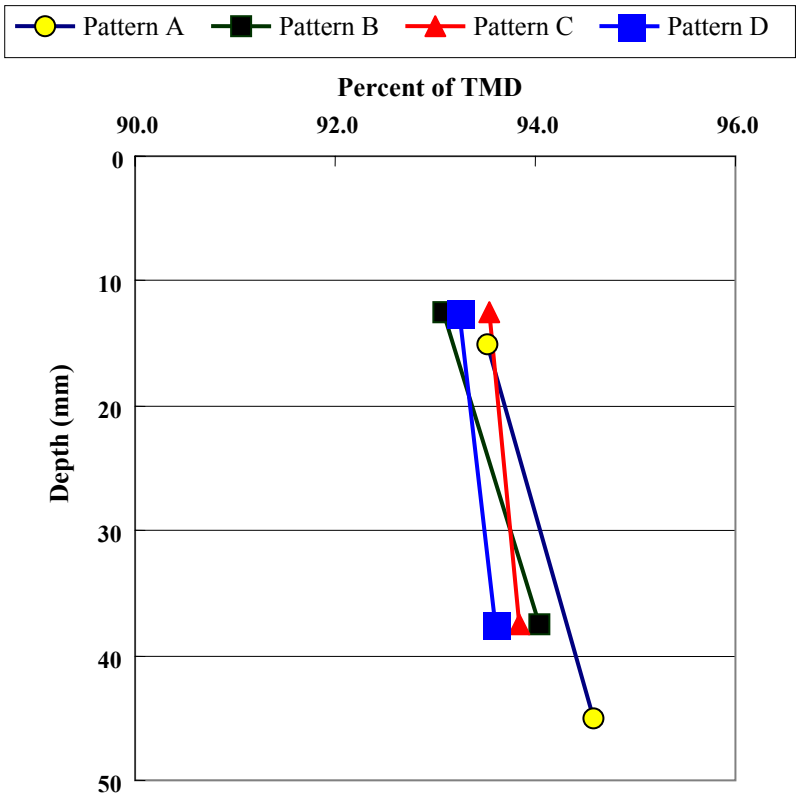

FIGURE 3 Distribution of percentage of the TMD in each of the two layers.

TABLE 7 Rolling Patterns, Percent of TMD, and Distribution of TMD in Each of the Two Layers

	Rolling Patterns					
	Rolling Process	Roller Models	Operating Variables			Number of Roller Passes
Pattern A	Breakdown	SW850	Speed	km/h (mph)	5.8 (3.6)	6 Vibratory
			Amplitude	mm (in.)	0.33 (0.013)	
			Frequency	Hz (vpm)	67.7 (4,000)	
	Intermediate	GW750	Speed	km/h (mph)	5.0 (3.1)	4 Vibratory
			Amplitude	mm (in.)	0.71 (0.028)	
			Frequency	Hz (vpm)	40 (2,400)	
	Finish	DD130	Speed	km/h (mph)	5.0 (3.1)	2 Vibratory
			Amplitude	mm (in.)	0.48 (0.019)	
			Frequency	Hz (vpm)	42 (2,500)	
Pattern B	Breakdown	SW850	Speed, Amplitude and Frequency are as same as those in Pattern A			6 Vibratory
	Intermediate	GW750				2 Vibratory
	Finish	DD130				2 Vibratory
Pattern C	Breakdown	SW850	Speed, Amplitude and Frequency are as same as those in Pattern A			5 Vibratory
	Intermediate	GW750				2 Vibratory
	Finish	DD130				2 Vibratory
Pattern D	Breakdown	SW850	Speed, Amplitude and Frequency are as same as those in Pattern A			8 Vibratory
	Intermediate	GW750				2 Vibratory
	Finish	DD130				2 Vibratory

Percent of Theoretical Maximum Density (TMD) Based on Cores						
	Positions of Density in Cores	Density g/cm^3 (lb/ft^3)	TMD g/cm^3 (lb/ft^3)	Percent of TMD	Standard Deviation	Number of Cores
Pattern A	Total	2.314	2.459	94.1	0.10	2
		144.4				
	Top	2.299		93.5	0.08	2
		143.5				
	Bottom	2.326		94.6	0.01	
		145.2				
Pattern B	Total	2.302		93.6	1.30	4
		143.7				
	Top	2.289		93.1	1.70	1
		142.9				
	Bottom	2.311		94.0	1.30	
		144.3				
Pattern C	Total	2.299	153.5	93.5	1.00	1
		143.5				
	Top	2.299		93.5	1.50	1
		143.5				
	Bottom	2.307		93.8	0.70	
		144.0				
Pattern D	Total	2.294		93.3	1.30	1
		143.2				
	Top	2.292		93.2	1.70	1
		143.1				
	Bottom	2.302		93.6	1.30	
		143.7				

Pickup of HMA mix on pneumatic tires occurred when surface temperature of HMA pavement was approximately 95°C (203°F). To prevent the pickup, a release agent was used at a dilution ratio of 100 parts water to one part release agent. Use of the release agent significantly reduced the amount of pickup on the pneumatic tires.

CONCLUSIONS

Several evaluations of the VPT roller during normal paving operations were reported—a paving project at San Francisco International Airport, the reconstruction of Main Street in King City, California, and on Michigan State Route M37 in Traverse City. Based on the test results reported in this paper, the following conclusions can be drawn.

- It was found that a VPT roller is beneficial for compacting various types of HMA mixes since the minimum required density levels of the pavement layers were significantly exceeded with a combination of a high-frequency DDV roller in the breakdown position and the VPT roller in the intermediate position.
- A VPT roller weighing only 9,350 kg (20,580 lb) and making six roller passes at an amplitude of 0.38 mm (0.015 in.) achieved the same level of density as a conventional SPT roller weighing more than twice as much—27,270 kg (60,000 lb) and making twice as many roller passes—12. It is obvious that the VPT roller is more versatile (smaller size and lighter weight) and easier to transport and operate than a heavier SPT roller.
- Based upon the density distribution within the pavement layer, the VPT roller was able to achieve a more uniform degree of the density between the middle and bottom portion of the pavement layer.

Further studies on the relationship between the density distribution of the pavement layer with depth and the rolling patterns used for various pavement surface temperature conditions, especially during breakdown rolling, are needed.

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RECENT ADVANCES IN COMPACTION EQUIPMENT,
INCLUDING INTELLIGENT COMPACTION

Compaction of Stiff and Tender Asphalt Concrete Mixes

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Many of the asphalt concrete mixtures designed using the Superpave mix design system are relatively stiff and difficult to compact. In order to properly densify these types of mixtures, it is necessary to keep the rollers directly behind the paver in order to apply the compactive effort while the mix is hot. Several different roller patterns can be employed, but the most efficient roller pattern typically uses a pneumatic tire roller for initial compaction followed by a double drum vibratory roller for intermediate, and often finish, rolling.

Some Superpave designed mixtures, however, are very tender and move under the compactive effort of the rollers. These tender mixes shove in both the longitudinal and transverse direction and often check or crack while being compacted. For tender mixtures, three temperature zones typically exist—both an upper and a lower temperature zone where density can be obtained and an intermediate temperature zone where de-compaction of the mix occurs during the rolling process.

For tender mixes, it is necessary to alter the roller patterns in order to achieve the required level of density in the upper temperature zone—before the mix starts to move and shove. The use of two double-drum vibratory (DDV) rollers operated in echelon directly behind the paver is the most efficient and effective method to achieve the desired level of density in such mixtures.

INTRODUCTION

It has often been said that the degree of compaction of a hot-mix asphalt (HMA) concrete mixture is the single most important factor that affects the ultimate performance of the pavement under traffic. Compacting an asphalt concrete mixture to an air void content of 6% or less generally increases the fatigue life, decreases the amount of permanent deformation or rutting, reduces the amount of oxidation or aging, decreases moisture damage or stripping, increases strength and internal stability, and may decrease slightly the amount of low-temperature cracking that may occur in the mix.

A HMA mixture may have all the desired mix characteristics and properties when designed in the laboratory. That same mix, however, may perform poorly under traffic if that mix is not compacted to the proper level of density on the roadway. A mix that may have only marginal properties in the laboratory will often outperform a mix with more desirable properties if the marginal mix is adequately and properly compacted.

Compaction is the process through which the asphalt mix is compressed and reduced in volume. Compaction permits the unit weight or density of the mix to be increased by placing more material into a given volume or space or by taking a given amount of material and compressing it into a smaller space or volume. As a result of the compaction process, the asphalt coated aggregate particles in the mix are forced closer together, which increases the amount of aggregate interlock and interparticle friction and also reduces the air void content of the mix.

With the advent of the Superpave mix design method in the United States, problems have been experienced in obtaining the desired degree of compaction in some of the mixtures (*1*). In a few cases, the problems have been related to the increased stiffness of the HMA material due to the incorporation of a polymer-modified binder into the mix. In most cases, however, the problems have been related to a lack of internal stability in the HMA material and the presence of a "tender zone" experienced at some point during the compaction process. Within the temperature range at which the tender zone exists, the mix moves under the applied compactive effort of the rollers and thus it is normally very difficult to achieve the required level of density.

The purpose of this paper is to review some of the possible causes for both stiff and tender asphalt concrete mixtures and to discuss how to properly compact each type of mix. For mixtures which exhibit tender characteristics, the contractor must be aware of the three temperature zones which typically exist during the rolling process.

SUPERPAVE MIXTURE PROPERTIES

Traffic Levels

The Superpave mix design procedure is based on the amount of traffic that will be traveling over the HMA pavement (*2*). The most recent revision of the Superpave system divides the mix design criteria into five different traffic categories. Those categories are based on the number of equivalent single-axle loads (ESALs) that will be pass over the pavement structure during a 20-year design period. The traffic levels used are <0.3, 0.3 to 3, 3 to 10, 10 to 30, and >30 million ESALs.

Aggregate Properties

The minimum required aggregate properties change depending on the level of traffic. The angularity of the coarse aggregate particles is measured in terms of fractured faces—both one and two fractured faces. As the amount of traffic increases, the minimum one and two face crushed content of the coarse aggregate also increases. The minimum fractured face count, however, is also dependent on the location of the layer within the pavement structure—less than or more than 100 mm below the surface of the HMA.

The minimum angularity level of the fine aggregate particles also increases as the number of ESALs increases and is related to the depth of the HMA layer beneath the pavement surface (*3*). As the amount of traffic loading increases, the fine aggregate angularity level also increases. In addition, the sand equivalent value for the mixture also increases as the number of ESALs applied to the pavement increases. The number of flat and elongated aggregate pieces in the HMA mix is also regulated based on a five to one maximum to minimum ratio. This ratio, however, is not affected by traffic category.

The gradation limits for the combined coarse and fine aggregate are based on the nominal maximum size of aggregate (NMSA), defined as the next sieve size larger than the first sieve to retain more than 10% of the aggregate. As the NMSA used in the mix decreases, the minimum voids in mineral aggregate (VMA) percentage in the HMA increases.

The purpose of the aggregate consensus properties is to provide a mixture which is stiffer and more resistant to both fatigue damage and permanent deformation. In essence, as the amount of traffic increases, the amount of uncrushed natural sand that can be added to the mix decreases and the

coarse aggregate must be more angular in order to create more interlock between the aggregate particles. Thus, for high-traffic loads, a HMA mix design using the Superpave aggregate criteria should produce a relatively stable, stiff asphalt concrete mixture. Such a mixture should be more difficult to compact to a given air void content.

Binder Properties

The Superpave system has created a whole new method of characterizing asphalt cement binder materials. The performance grading (PG) system bases the minimum requirements for the binder on the average 7-day maximum pavement temperature and on the minimum temperature to which the pavement will be subjected. Depending on the location of the pavement, the binder may or may not be modified in order to meet the minimum performance criteria.

For many lower traffic volume roadways, unmodified asphalt cement is typically specified for use in the HMA. As the number of ESALs applied to the pavement increases, however, it is often necessary to modify the binder material with some type of polymer in order to obtain the desired binder properties. In addition, as the range between the minimum and maximum ambient temperatures increases, the use of modified binder materials also increases.

The purpose of the PG system is to provide a stiffer binder for increased levels of traffic and for harsher environmental conditions. A HMA mix that incorporates a stiff binder material will normally be more difficult to compact to a given air void content.

Aggregate Gradation Requirements

For each particular HMA mix, based on the NMSA incorporated into the mix, there are control points applied to the combined coarse and fine aggregate gradation. The control points limit the maximum and minimum amount of aggregate that can pass a particular sieve. In addition, there are maximum and minimum amounts of aggregate that can pass the 0.075 mm (No. 200) sieve for each NMSA size mix.

The purpose of the aggregate gradation requirements is to increase the stiffness of the asphalt concrete mix. Extreme combinations of coarse and fine aggregate are eliminated and, theoretically, mixtures that are very densely graded—parallel the so-called maximum density line—can not pass the specifications. Such mixtures will be more difficult to compact to a given air void content.

Laboratory Compaction Parameters

The Superpave design system includes the use of a gyratory compactor to produce HMA mix specimens for laboratory testing (*4*). Three levels of compactive effort are required, based upon the number of revolutions applied. Those revolutions were originally labeled Ninitial, Ndesign, and Nmaximum. The use of the Nmaximum criteria has been eliminated and Superpave-designed mixtures are currently compacted to the Ndesign number of gyrations. As the number of ESALs applied to the pavement increases, the minimum number of revolutions of the gyratory compactor also increases.

On a practical basis, as the number of ESALs becomes greater and the Ndesign value also increases, the amount of binder added to the mix decreases. Everything else being equal, a HMA mix which contains less binder material should be more difficult to compact to a given air void content.

COMPACTION OF STIFF HMA MIXTURES

Particularly for high volumes of traffic, a properly designed Superpave mix should result in an asphalt concrete mixture which is relatively stiff and relatively difficult to compact. For such mixtures, the compactive effort applied to the mix on the roadway must be accomplished while the temperature of the HMA material is still high. A variety of compaction equipment can be used in various combinations, including DDV rollers, pneumatic tire rollers, and static steel wheel rollers, to accomplish the task.

For unmodified HMA, the typical mix laydown temperature (the temperature of the mix passing out from under the paver screed) in is in the range of 150°C to 135°C (300°F to 275°F). For a polymer-modified HMA material, the typical mix laydown temperature is in the range of 160°C to 140°C (320°F to 285°F), depending on the type of polymer used in the mix.

Most specifications require a minimum level of density be obtained in the mix. That minimum level is typically set at 92% of the theoretical maximum density (TMD) of the HMA mix—a maximum air void content of 8%. For some coarse-graded Superpave mixtures—those with gradations below the so-called maximum density line—some state highway departments have specified the minimum density level at 93% of the TMD value. This increase in the minimum density level is in response to problems with high water permeability in some of the coarse-graded HMA mixtures.

Conventional Roller Combinations

Two primary groups of "roller trains" have been employed in recent years to compact stiff HMA mixes. The most popular combination consists of a double drum steel wheel vibratory roller operated in the initial or breakdown position. This roller is followed by a pneumatic tire roller operated in the intermediate or second position. Finish or final rolling is typically completed by a static steel wheel roller.

The DDV roller is usually kept relatively close behind the paver to obtain the initial compactive effort while the mix is still hot. In general, the breakdown rolling should be completed before the surface temperature of the mix falls below 120°C (250°F). The roller is operated at the highest possible frequency level available for the particular make and model of roller and at an amplitude setting that is dependent on the thickness of the asphalt concrete mat being placed. For HMA layers less than 30 mm (1¼ in.) thick, the vibratory roller should not be operated in the vibratory mode—the roller should be run in the static mode to avoid fracturing the aggregate in the mix. For HMA layers ranging in thickness from 30 mm to about 75 mm (1¼ to 3 in.), a low amplitude setting is used on the vibratory roller. For greater thicknesses of asphalt concrete mix, a higher amplitude setting can be used without fracturing the aggregate in the mix.

When a pneumatic tire roller is used as an intermediate roller, it is usually necessary to keep the tires at the same temperature as the mat being compacted. This means that the roller can not be allowed to sit and wait for long periods of time, allowing the temperature of the roller tires to decrease to the point that pickup of the mix is experienced. For some polymer-modified HMA mixes, it may be very difficult to prevent pickup depending on the type and concentration of polymer used in the mix. Intermediate rolling should be completed, for a relatively stiff Superpave mix, before the surface of the mix reaches a temperature of approximately 100°C (210°F). The intermediate roller thus should be kept directly behind the breakdown roller.

A static steel wheel roller is usually used for finish rolling. The primary purpose of this compactive effort is to obtain the "last little bit" of density and to remove the marks, if any, left by the first and second rollers. For stiff Superpave designed mixes, finish rolling should be accomplished before the surface temperature falls below about 80°C (175°F).

Alternative Roller Combinations

For Superpave mixtures that contain high levels of polymer and are very stiff even at relatively high laydown temperatures (above 160°C or 320°F), one alternative combination of rollers that has often been used to obtain the required level of density consists of a pneumatic tire roller in the breakdown or initial position behind the paver. In this case, the pneumatic tire compactor is kept as close to the paver as reasonably possible. As in the case discussed above where the pneumatic tire roller is used in the intermediate position, the tires of the roller must be kept hot—the same temperature as the mix being compacted.

Before starting compaction at the beginning of the day, it is necessary to run the pneumatic tire roller back and forth on a previously placed pavement for a period of 10 to 15 min in order to build up heat in the tires. For most normal binder (unmodified) mixtures, the pneumatic tire roller then can be operated on the new mat directly behind the paver after mix placement starts. For some polymer-modified mixtures, however, it may be necessary to put the pneumatic tire roller on the mix behind a DDV roller for a few minutes in order to allow the temperature of the tires to fully reach the temperature of the mix. Once that has been achieved, the pneumatic tire roller can be moved into the breakdown position ahead of the DDV roller. It is noted that the pneumatic tire roller should be operated without water being sprayed onto the tires.

The second roller should be a DDV roller operated close behind the pneumatic tire roller. The frequency of vibration used for this roller should be as high as possible. The amplitude setting, however, should be related to the thickness of the mat being placed, the same as when this type of roller is used in the breakdown position. This roller pattern—a pneumatic tire roller in the breakdown position with a vibratory roller in the second position—is shown in Figure 1.

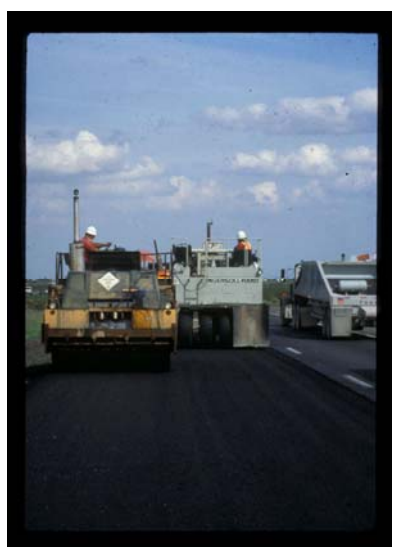

FIGURE 1 Pneumatic tire breakdown roller and DDV finish roller.

For this combination of rollers, it is often not necessary to use a static steel wheel finish roller. The tire marks left by the pneumatic tire roller are readily removed by the vibratory roller. This is due to the fact that the latter machine operates on the mat when the temperature of the mix is higher compared to the temperature of the mix under the static finish roller behind the pneumatic tire roller in the conventional roller train. In addition, the smooth steel wheel drums of the vibratory roller typically leave few, if any, marks in the mat when this roller is used behind the pneumatic tire roller. If marks are present, they can usually be erased easily by simply making a pass of the vibratory roller over the mat in the static mode without vibration.

A second alternative roller combination includes the use of two vibratory rollers, using one operated in the breakdown position and one operated in the second position. In this case, both rollers should be kept close to the paver and both machines should be operated in the vibratory mode. For most Superpave mixtures, even those that are polymer modified, a static steel wheel roller is not needed. If marks are present, they can usually be easily eliminated by making a final pass over the pavement surface with the second vibratory roller operated in the static mode. In many instances, a higher degree of density can be obtained with the two vibratory rollers being operated in echelon (essentially side by side), one on each side of the lane being place and compacted. In this situation, there is no problem with roller marks since the two vibratory rollers can easily remove their own marks due to the high mix temperature when the compactive effort is applied.

Key Factors

The key to compacting a stiff Superpave mixture is to roll the material while it is as hot as possible. Keeping the rollers—in any combination—as close to the paver as possible is extremely important. Properly designed, a Superpave mix is supposed to have enough internal stability to support the weight of the compaction equipment without pushing or shoving. With the correct aggregate properties, such as crushed count and gradation, the stiff mix must be rolled while the viscosity of the binder material is still low—when the temperature is high—in order to be able to reorient the aggregate particles under the applied compactive effort and to densify the mix.

It has been found that, in general, the required degree of density can be obtained in the mat with fewer roller passes over any point in the pavement surface when a pneumatic tire roller is used in the breakdown position, with a DDV roller in the second position, compared to the same two rollers used in the reverse positions—vibratory roller first and pneumatic tire roller second. Fewer roller passes means a more efficient and economical compaction operation.

CHARACTERISTICS OF TENDER HMA MIXTURES

Not all asphalt concrete mixtures that meet the Superpave requirements exhibit high stiffness during the compaction process. Some of these mixes are quite tender—they move or shove excessively under the weight of the compaction equipment while being rolled. These tender mixtures need to be treated very differently than stiff mixes in order to be properly compacted to the required level of density.

Until recently, emphasis has been placed on designing coarse-graded Superpave mixes. These mixes have gradations which lie on the bottom side of the so-called maximum density line when plotted on 0.45 power gradation graph paper. In some cases, the gradation of the combined coarse and fine aggregate lies completely beneath maximum density line. In other cases, a S-curve

gradation is employed. In the latter case, the gradation starts out on the fine or upper side of the maximum density line in the coarse aggregate portion of the gradation but then dips or curves down below the maximum density line as the gradation gets finer. The gradation of the aggregate then passes under—on the coarse side—of the maximum density line.

The use of a polymer-modified asphalt binder in the Superpave mix typically does not overcome the primary causes of a tender mix, as discussed below. Mixtures that contain a non-modified binder that exhibits tender characteristics generally also have tender properties when the standard binder material has been replaced with a modified binder in the same mix.

Low VMA Content

When the combined grading of the coarse and fine aggregate incorporated into the mix passes beneath the maximum density line but immediately adjacent to it, the resulting HMA mix typically has a minimal VMA content (*5*). Further, the change in VMA with a change in binder content is often very small—there is a relatively flat curve when a graph of binder content versus VMA is drawn over a range of 1.5% binder content in accordance with the Superpave mix design procedures. Such mixtures are typically very sensitive to fluids content—the combination of asphalt binder content and moisture content—due to the low VMA content of the HMA mix. For these mixes, a small variation in the amount of binder material added to the mix or the presence of residual moisture content in the mix (the aggregate does not get completely dry when passing through the batch plant dryer or through the drum mix plant) results in a mix that is tender.

If the binder content in a mix with a low VMA content is too high—usually only 0.2% to 0.3% above the optimum value—the film thickness of the binder material increases and the mix becomes over lubricated. Similarly, if all of the moisture is not removed from inside of the aggregate particles before the binder is added to the mix, some residual moisture may be retained in the internal pores of the aggregate (*6*). This moisture prevents a portion of the asphalt binder material from being absorbed into the aggregate (which was taken into account during the mix design process since the aggregate was completely dried in the laboratory before the binder material was added). The result is again an increase in the film thickness of the binder material around the coarse and fine aggregates due to the presence of the non-absorbed asphalt cement on the outside the aggregate instead of being inside the aggregate.

Even with a high crushed face content in the combined coarse and fine aggregate materials, it is very possible to design a HMA mix which is very sensitive to fluids content when the VMA content of the mix approaches the minimum value permitted for the NMSA incorporated into the mix. Aggregate gradations which plot immediately adjacent to the lower side of the so-called maximum density line usually produce mixtures with very low VMA content and thus tender mixes when an excess of fluids content—binder content or moisture content—is present in the material. Such mixtures lack the internal stability to support the weight of the compaction equipment during the rolling process.

High VMA Content

The same problem with high fluids content also exists when the combined coarse and fine aggregate material results in a HMA mix which has very high VMAcontent—more than 1.5% above the minimum value required for a given NMSA used in the mix. If the combined coarse and fine aggregate material produces a gradation similar to the one shown in Figure 2, near the

FIGURE 2 Humping up of the mix at the edge of a steel wheel roller drum.

control points at the bottom side of the graph for a coarse-graded mix, then the HMA mix will typically have a high VMA content.

According to Superpave mix criteria, the design air void content in the mix is usually 4.0%. As with any HMA mix, asphalt binder material is added to the mix to fill up the VMA until the 4.0% air void content is reached. If the mix contains a high VMA content, more binder material is added to achieve the desired air void content. This, in turn, results in higher binder content in the mix and higher binder film thickness around the aggregate particles. In this case, the mix becomes over lubricated with too much asphalt binder material. The final result is a mix that is tender during the construction–compaction process due to a lack of internal stability in the mix and thus a mix that may later rut under applied traffic loads.

Mixes with high VMA content typically do not contain enough mastic (fine aggregate and mineral filler) material in the mix to hold the mix together. The applied load is then carried by the binder material instead of by the aggregate. If the VMA is more than 1.5% above the minimum value, it is often recommended to add fine aggregate or mineral filler material to the aggregate in order to lower the VMA content of the mix. Indeed, recent changes in the criteria for coarse-graded Superpave mixtures have increased the allowable range for the dust-to-binder ratio from 0.6 to 1.2 to a range of 1.0 to 1.6 in order to increase the mastic portion of the HMA mix and significantly reduce the potential tenderness of the mix during construction and the potential for rutting or permanent deformation of the coarse-graded mix under traffic (*7*).

TENDER MIX TEMPERATURE ZONES

Movement of the Mix

HMA mixtures that have an excess of fluids content due to too much binder, too much moisture, or a lack of mastic content are normally very difficult to compact (*8*). This is due to the tendency for the mix to move under the applied compactive effort of the rollers. This movement occurs in two directions.

In the longitudinal direction, the mix will shove in front of the steel wheels of both a vibratory roller and a static roller. A bow wave will form in front of the drums and the HMA material will "hump up" before the drum of the roller reaches that point on the surface of the mix. Indeed, depending on the degree of tenderness of the mix, mix located up to 150 mm (6 in.) in front of the roller may start to move before the roller gets there. In extreme cases, the mix may start to move while it is still 300 mm (12 in.) or more in front of the first drum of the steel wheel roller. This may occur whether the roller is operated in the vibratory mode or static mode.

Such longitudinal movement of the mix is normally accompanied by checking of the mix—the development of short, transverse cracks in the surface of the HMA material. Again, depending on the degree of the tenderness of the mix, the checks or cracks can be found as close as 25 mm (1 in.) apart if the mix is very tender to 75 mm (3 in.) apart if the mix is only slightly tender. In terms of transverse length, the checks or cracks may be as short as 50 mm (2 in.) if the mix is only slightly tender to a length of 200 mm (8 in.) or more if the mix is very tender.

A tender mix will also move in the transverse direction. Depending on the position of the edge of the steel wheel roller drum on the unsupported edge of the pavement layer, it is possible for a tender mix to widen out transversely during the compaction process. In some cases, if the edge of the roller drum is located just inside the unsupported edge of the asphalt concrete mat, the mix may creep out laterally a distance of 100 to 200 mm (4 to 8 in.) or more. Such transverse movement of the HMA material makes it very difficult to achieve the desired level of density in the mix at that location as well as very hard to make a good longitudinal joint if another lane is to be constructed next to the first lane.

If the mix is tender enough, the mix will hump up on the outside edge of the steel wheel roller drum. This is seen in Figure 2. The lateral or shear force at the edge of the drum is great enough to de-compact the HMA material and shove it sideways. Such movement (cutting of the mix) is usually found whenever the steel wheel roller, operated in either the vibratory or the static mode, makes any type of turning movement such as at the end of a roller pass. The de-compacted mix in the hump formed next to the edge of the drum is normally accompanied by checking or cracking of that mix, as shown in Figure 3.

FIGURE 3 Checking of a tender mix.

Three Temperature Zones

HMA mixtures that have tender characteristics still have to be compacted to a minimum level of density or a maximum level of air void content. Because of the lack of internal stability in the mix, however, the compaction of a tender mix is often very difficult. It has been found that a mix that has tender properties may be able to be compacted to the proper density by taking advantage of the three temperature zones which normally exist in that tender mix.

In the first, or upper, temperature zone, the asphalt concrete mix is relatively stable during the compaction process. Within the temperature range from laydown (160°C to 140°C or 320°F to 285°F, depending on the use of polymer-modified binder and other factors) down to about 115°C (240°F), the HMA material is stable under the applied compactive effort. In this upper temperature range, the mix will not shove or check under the rollers regardless of whether a vibratory roller or a static steel wheel roller is used to compact the mix.

The lower limit of this upper temperature zone is not an exact value—it depends on the characteristics of the mix, the rate of cooling of the mat, the thickness of the layer, environmental conditions, and the type of roller used—vibratory or static steel wheel. In some cases, this value may be as high as 120°C (250°F) or higher while in other cases it could be as low as 110°C (230°F) or lower.

The middle temperature zone—the problem range—extends from approximately 115°C (240°F) down to about 90°C (195°F). In this temperature range, the mix will move, shove, and check under the applied compactive effort. A bow wave will form in front of the steel drums of the roller and the mix will crawl longitudinally. The mix will also move laterally or transversely and the mat will widen out if the edge of the roller is not positioned properly over the unsupported edge of the asphalt concrete mat by at least 150 mm (6 in.). In general, however, the mix will not move in this intermediate temperature zone when compacted using a pneumatic tire roller. In this middle temperature zone, the HMA material lacks the internal stability to support the weight of the steel wheel compaction equipment.

As with the upper temperature zone, the temperature range for the middle zone is not an exact value. This zone may start at a temperature of 120°C (250°F) or higher or at a temperature of 110°C (230°F) or lower. The tender zone may extend down to a temperature of 80°C (175°F) or lower or may stop at a temperature of 95°C (205°F) or higher. In general, for many mixes, the tender zone ranges from approximately 115°C (240°F) down to about 90°C (195°F), as discussed above.

The lower temperature zone extends from the end of the tender or intermediate temperature zone down to approximately 70°C (160°F) or even lower. Within this temperature range, the mix is cool enough to regain the internal stability necessary to support the weight of the compaction equipment. It is possible, but usually very difficult, to achieve the required degree of density within this relatively narrow temperature range if the mix has been de-compacted during the rolling of the mix when the mix temperature was in the middle temperature zone.

COMPACTION OF TENDER HMA MIXTURES

In order to compact a HMA mixture which has tender characteristics and moves under the rollers in the middle temperature zone, a contractor has one of two choices. First, all of the compactive effort can be applied in the upper or the lower temperature zones when the mix is internally stable and does not move, shove, or check under the steel wheel rollers. In this case, no compactive effort is applied

to the mix when the mix temperature is within the middle or intermediate temperature zone. Second, if it is necessary to roll the mix while the temperature is within the middle zone, a pneumatic tire roller can be employed as the intermediate roller since the mix will typically not shove in front of the rubber tires on this type of roller as it will in front of a steel wheel roller drum in this middle temperature zone.

Avoiding the Tender Temperature Zone

In order to compact a HMA mix that is stiff, it is necessary to roll it as close behind the paver as possible, as described above. In order to compact a HMA mix that is tender, it is also necessary to roll it as close behind the paver as possible in order to take advantage of the upper temperature zone in the mix.

For most Superpave asphalt concrete mixtures that exhibit tender characteristics, the upper temperature zone ranges from the laydown temperature down to about 115°C (240°F). Within this range of temperatures, the tender mix must be compacted to as high a density as feasible, as quickly as possible. For most stiff HMA mixtures, one roller can be placed behind another roller, in a roller train fashion. For most tender HMA mixtures, however, in order to obtain the required minimum level of density (maximum air void content), it is necessary to place two rollers on the mat within the upper temperature zone.

Use of Two Double Drum Vibratory Breakdown Rollers

Perhaps the most efficient method to compact a tender mix within the upper temperature range is to use two DDV rollers in echelon (almost side by side). As illustrated in Figures 4 and 5, one DDV roller can operate on one side of the mat and the other DDV roller can operate on the other side of the mat. In essence, when one roller moves toward the paver on one side of the lane, the other roller also moves toward the paver at the same time on the other side of the same lane. The rollers operate together, moving forward and backward at the same time on different portions of the mat. The purpose of this compaction process is to get as much density in the mat as possible before the temperature of the mix drops into the middle or tender temperature range and the mix begins to shove and move under the steel wheel rollers.

FIGURE 4 Two DDV rollers in echelon behind the paver.

FIGURE 5 Two DDV rollers in echelon behind the paver.

The exact roller pattern used depends on many factors. Some of those factors include the width of each DDV roller used, the width of the lane being placed, the temperature of the mix immediately behind the paver screed, the temperature at which the mix starts to move and shove (the start of the tender zone), and the maximum frequency at which each vibratory roller can operate. Ideally, the two DDV rollers should be of the same make, model, and condition. This will assure that the compactive effort applied to the mat is equal across the width of the lane.

In order to adequately compact an asphalt concrete mixture, it is proper to extend the edge of the vibratory or static steel wheel roller drum over the unsupported edge of pavement and/or over the longitudinal joint by at least 150 mm (6 in.). Theoretically, therefore, one would need a 3.9 m (13 ft) wide roller to compact a 3.66 m (12 ft) wide pavement, allowing for a 150 mm (6 in.) overlap of the outside edges, whether unsupported or a longitudinal joint. Since no roller exists that is 3.96 m (13 ft) wide, in order to uniformly compact the mix using rollers of normal width, it is necessary to overlap the edge of the previous roller pass by a minimum of 150 mm (6 in.). Vibratory rollers used for most highway paving projects typically are manufactured in three drum widths: 1.67, 1.98, and 2.13 m (66, 78, or 84 in., respectively).

If the width of the lane is 3.66 m (12 ft) and the roller being used has a width of 2.13 m (84 in. or 7 ft), then the whole width of the lane can be covered with two passes of the DDV roller—one of the left side and one on the right side, allowing for a 150-mm (6-in.) overlap of both edges of the lane as well as a 150-mm (6-in.) overlap of the drum in the center of the lane. If a narrower DDV roller is employed, however, with a drum width of only 1.98 or 1.67 m (78 or 66 in., respectively), then three passes of the roller are required in order to compact the whole width of a 3.66 m (12 ft) wide lane and have the correct amount of overlap over the unsupported edge and/or longitudinal joint and between internal passes within the lane width.

Roller Patterns

A roller pattern that has been used to compact a 3.66 m (12 ft) wide lane with two DDVrollers that are either 1.98 or 1.67 m (78 or 66 in., respectively) wide, operating in echelon in the breakdown position directly behind the paver is as follows: Both rollers are operated at maximum frequency and at an amplitude setting that is proper for the layer thickness being placed. The first roller compacts the left side of the roadway with two passes (numbers 1 and 2) up and back in exactly the same

position, hanging over the left edge of the lane by 150 mm (6 in.). (A pass is defined as one time over a point in the pavement surface). That roller then makes a pair of passes (numbers 3 and 4), up and back, over the center of the lane. The last three passes (numbers 5, 6, and 7) are made directly on top of the first two passes on the left side of the mat. At the end of pass 7, the roller continues up to the back of the paver and then begins the pattern over again.

The second DDV roller makes its first four passes (numbers 1, 2, 3, and 4), up, back, up, and back over the right side of the mat, hanging over the right edge of the lane by 150 mm (6 in.). The next two passes, numbers 5 and 6, are made up the center of the lane over the top of the two passes completed by the other roller. For pass number 7, the second roller again moves back to the right side of the lane and makes its last pass over the top of the first four. At the end of pass 7, the roller continues up to the back of the paver and then begins the pattern over again.

Using this roller pattern, and allowing for overlap of the unsupported edge of the lane or the longitudinal joint, as well as the overlap between the roller drums, the width of the lane is compacted as uniformly and efficiently as possible. Most importantly, the compaction process can be completed before temperature of the mix reaches the middle or tender temperature zone. In this way, attempted compaction of the mix in the tender zone can be avoided by essentially putting the intermediate roller in the breakdown or initial position together with the first roller.

In most cases, the required level of density can be obtained using only the two DDVrollers operated in echelon in the breakdown position. Generally, no additional rollers are needed, even a static steel wheel finish roller. Roller marks can be taken out with the initial rolling. No shoving or movement of the tender mix occurs since no rolling is done in the intermediate or tender temperature zone. This roller pattern typically is the most efficient and economical means to compact a tender asphalt concrete mixture.

Rolling in the Tender Temperature Zone

If only one DDV roller is available and if a pneumatic tire roller is available, then the rubber tire roller can be used in the intermediate position in the tender zone. The vibratory roller should make as many passes as possible over the width of the lane before the temperature of the mix drops to the point that the mix starts to move and check with an additional roller pass. Once the mix starts to shove and de-compact under the drum, all compaction with any steel wheel roller, operated in either the vibratory or static mode, should cease.

An asphalt concrete mixture, even a tender one, does not normally shove in front of the tires of a pneumatic tire roller. The rubber tires on this type of roller tend to tuck the mix back under the tires instead of shoving it forward. Thus, in the middle or tender temperature zone, a pneumatic tire roller can be used to accomplish what a steel wheel roller can not do: compact the mix. It is important to note, however, that it may be difficult to use a pneumatic tire roller in the middle temperature zone on some polymer-modified mixtures, particularly if the mix has been modified using a latex (styrene-butadiene-rubber) type material. Pickup of the mix on the tires might be impossible to avoid in some instances.

If a pneumatic tire roller is used in the intermediate position on a tender mix, it will be necessary to use a static steel wheel roller to complete the compaction process. The finish roller is employed to remove the marks of the rubber tire roller and to increase the density of the mix to the final, required level. Care must be taken, however, that the finish roller does not operate in the middle temperature zone and actually de-compact the mix instead of compacting it. The static steel wheel finish roller should be moved closer and closer toward the back of the rubber tire roller until the mix

starts to move or shove under the drums. This is an indication that this roller is inside the middle or tender temperature zone. If this occurs, the finish roller should be kept farther back from the intermediate roller and operated completely within the lower temperature zone.

Using this roller pattern to compact a tender mix is not as efficient as using the two DDV rollers in echelon in the breakdown position since three rollers are needed—a DDVbreakdown roller, a pneumatic tire intermediate roller, and a static steel wheel finish roller. This pattern, however, can be used, if necessary, to obtain the required level of density in a tender mix.

SUMMARY

It is possible to properly compact a Superpave HMA mixture that is relatively stiff or a Superpave HMA mixture that is tender. In either case, it is suggested that the breakdown or initial roller or rollers be kept as close to the back of the paver as possible in order to compact the mix while it is hot. If the mix is stiff, the roller pattern used should ideally consist of a pneumatic tire roller, followed by a DDV roller, followed, if necessary, by a static steel wheel finish roller. If the mix is tender, the roller pattern used should consist of two DDV rollers operating in echelon directly behind the paver, followed, if necessary, by a static steel wheel finish roller, with no roller operating in the intermediate or tender temperature zone.

For tender mixes, the real solution to the tender mix problem is twofold. If the problem is with the gradation or the properties of the aggregate incorporated in the HMA mix, the aggregate should be changed. If the problem is with an excess of fluids content, either asphalt binder material or moisture, the binder content of the mix should be adjusted or the aggregate should be properly dried in the plant before the binder is added during the production of the mix.

For tender mixes, overcoming the deficiencies in the mix in the compaction process can be done; the required level of density can be obtained with an adjustment in the roller pattern used. The mix, however, might still deform or rut under traffic. A mix that can not support the weight of the rollers during the compaction process may not be able to support the weight of the applied traffic with time. Thus the proper solution to a tender mix compaction problem is to change the properties of the mix being compacted.

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Longitudinal Joint Density

Basics of Longitudinal Joint Compaction

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Most of the problems that are observed in hot-mix asphalt (HMA) pavements are related to one of three issues: insufficient compaction of longitudinal joints, segregation, or low mat density. Solving these construction problems can result in an increase in the pavement life resulting in significant cost savings.

This report deals with compaction of longitudinal joints. During the construction process much effort must be provided to the longitudinal joint to ensure a uniform, high-quality joint is obtained. Poor compaction of longitudinal joints typically results in joints cracking, opening, and raveling of the adjacent material (Figure 1).

The purpose of this paper is to give an overview of methods that have been used to construct longitudinal joints and to provide some guidance to ways to achieve better longitudinal joints.

FIGURE 1 Opening of longitudinal joint.

FACTORS AFFECTING DENSITY OF LONGITUDINAL JOINTS

There are three primary factors that affect the density that is obtained at longitudinal joints. The first factor is the amount of density obtained on the outside edge of a paving lane that will become part of the joint when the adjacent lane is placed. Since the outside edge is unconfined it is very difficult to obtain adequate compaction on this edge. The mix at the edge tends to move laterally when being compacted resulting in a lower overall density at the edge. When a poor bond exists the edge may move several inches during the rolling process.

The second factor involves how well the material located directly in the joint is compacted. Good compaction of this material requires that some overlap of the cold mat occurs during the paving process to ensure sufficient material is available in the joint for compaction.

The third item involves how well the mix on the hot side of the joint is compacted. The mix on the hot side of the joint is confined and is typically easier to compact but there is a tendency for the contractor to operate with the screed too low at the joint so that rolldown of the mix can be easily obtained. Each of these items will be discussed in more detail below.

Compaction of Free Edge

Compaction of the unconfined edge has always been difficult to perform. One reason is that the edge is not confined and tends to move underneath the rollers especially when the bond to the underlying layer is less than desired. This movement can be minimized by proper selection and application of tack coat. With a good bond between the underlying layer and the layer being compacted, there will be less movement underneath the rollers thus making it easier to compact the edge. Without a good bond, good compaction of the edge will be impossible.

One of the problems is that the edge of the newly placed HMA breaks over when compacted providing a lot of loose material immediately adjacent to the edge of the lane (Figure 2). When the adjacent lane is placed it is very difficult to adequately compact this loose material. In some cases contractors elect not to roll the outside 3 to 4 in. of material until the adjacent lane is placed. Best results are obtained when the edge is rolled with a steel wheel roller prior to placement of the adjacent lane. This provides some breakdown of the edge but the rolling is needed to compact the material between the edges and a few inches from the edge before the material cools. The edge should not be rolled with a rubber tire roller since it will round off the edges. For best results the rubber tire roller should stay a few inches from the edge until the adjacent lane is placed.

Infrared heaters have been used to help reheat the edge when placing the adjacent lane. It is assumed by those using this device that this added heat will make the HMA more workable and help to ensure additional compaction when rolling. One of the problems that have been observed with a joint heater (Figure 3) is the overheating of the material in the joint and adjacent to the joint. It is very difficult to control the heaters so that the loose material on the cold side of the joint is consistently heated to a satisfactory temperature. This results in some of the material being overheated and some material being under heated. In many cases the loose material that is overheated will have a dull brown color indicating that some damage to the asphalt binder may have occurred. There are other procedures that can be used to provide acceptable joint density that do not result in overheating of the asphalt.

FIGURE 2 Breaking over of material at edge when rolling.

FIGURE 3 Use of joint heaters at longitudinal joints.

There has been some work to develop a device that applies confinement to the free edge as the rollers are providing compaction. In theory, this confinement allows the edge to be rolled without the mix breaking down resulting in a higher density at the edge. To date the use of this device has been demonstrated on a number of projects without significant success.

Compaction of Material Directly in the Longitudinal Joint

One problem that sometimes occurs when placing the second lane adjacent to the first lane occurs when the material is not pushed tight up against the edge of the cold lane by the paver. There are several reasons that this lack of material adjacent to the joint may happen. In some cases the overlap is eliminated or so small that insufficient material is provided into the joint for compaction. This minimum overlap is sometimes done so that luting of the joint is not needed. There is always some variability in the alignment of the paver so any time the overlap is too small there will be some gaps between the two paving lanes. Any gap that is created by the paver can not be adequately rolled sufficiently to close the gap and provide adequate joint density. So it is important to ensure that sufficient overlap is always provided to ensure that even with variability there will always be some overlap.

Another problem that may occur directly in the joint is caused by the paver auger not adequately pushing the HMA against the free edge when placing the second lane. Instead the material falls from some point on the outside edge of the auger to the edge of the previous lane often resulting in a high percentage of coarse aggregate at the joint. This can be caused by paving too fast and not allowing the augers in the paver to adequately push the material to the joint. It can also be caused when the material in the auger box is not kept at a constant satisfactory height.

Another cause of inadequate material being provided at the joint is the use of paver extensions that do not provide the same initial in place density as the primary screed. With some extensions the uniformity of material behind the screed varies between the extensions and the primary screed. If the density behind the screed near the edge is less than the density behind the rest of the screed then adequate density near the joint will be difficult to obtain because sufficient material is not available for compaction. This is a smaller problem when rubber tire rollers are used because these rollers tend to provide similar pressures in low areas and thus adequately compact the low area. Steel wheel rollers, on the other hand, tend to bridge over these areas and not significantly increase density.

It is also important to extend the augers when the screed extensions are installed to ensure that there is not too much gap between the edge of the augers and the joint. If the gap is too large then the material is not pushed into the joint but is allowed to fall into the joint resulting in more coarse, loose mix at the joint.

Compaction of Material on Hot Side of Joint

The biggest issue on the hot side of the joint that leads to low joint density is failure to place enough material on the hot side of the longitudinal joint. Generally HMA is rolled down about 20% during the rolling process. This means that a layer with an uncompacted thickness of 2½ in. will end up being approximately 2 in. thick after compaction. This percentage varies with different materials, etc., but the amount of rolldown is typically somewhere around 20%. As shown in Figure 4 the newly placed material should be placed about 20% thicker than the

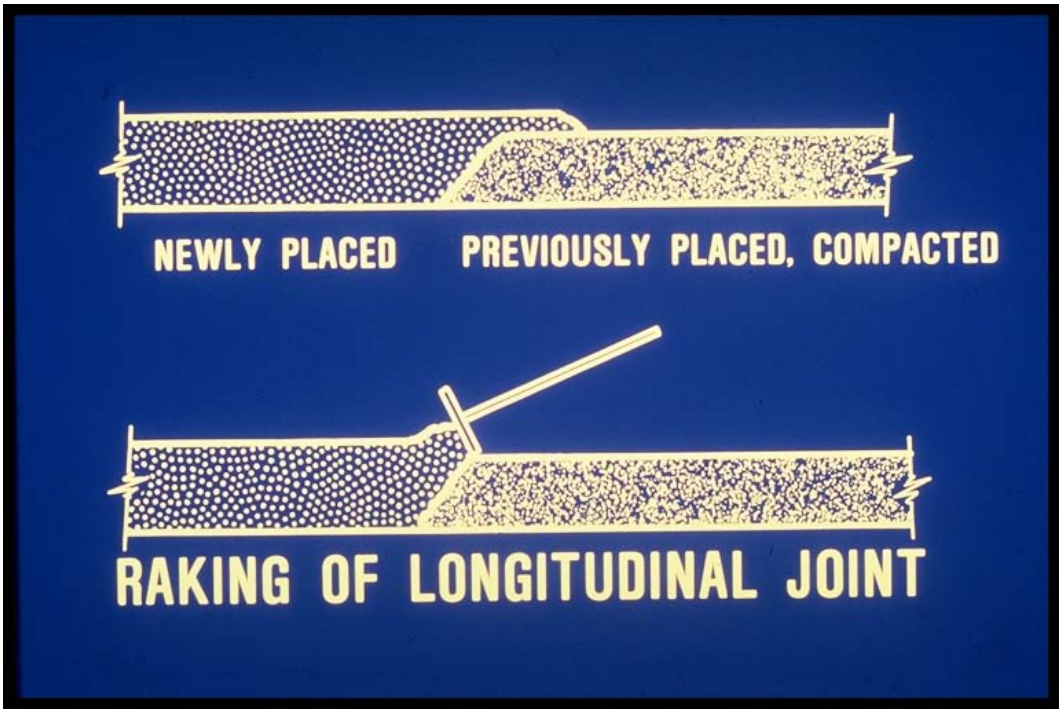

FIGURE 4 Placement of material on hot side of joint.

previously placed, compacted mix to provide satisfactory density when rolldown occurs. There is a tendency, by some contractors, to place insufficient thickness of HMA adjacent to the joint so that the material can easily be rolled down to produce a smooth joint. However, this results in inadequate density when rolled down to the same elevation as the adjacent cold mat.

When using steel wheel rollers the material on the hot side adjacent to the joint is rolled down to match the cold side. Very little additional density is obtained after it has been rolled down to the same elevation as the adjacent lane. For example, assume the material on the hot side is placed 10% thicker than the adjacent lane. During compaction the hot mix is rolled down to the same elevation as the adjacent lane and then any additional compaction with steel wheel rollers is minimum due to the bridging effect of the steel wheel roller. If rubber tire rollers are used, the material is compacted more as it is rolled since the rubber tire roller does not bridge over low spots and hence, can provide additional compaction in low spots.

Measuring Joint Density

Some state departments of transportation (DOTs) specify a minimum joint density to help ensure that high-quality joints are constructed. Due to the uneven surface and the porosity of the material in the joint it is sometimes difficult to accurately measure joint density. A nuclear density gauge is not very accurate at the joint since it is difficult to seat properly and it measures an area of HMA that includes material well outside the joint.

When cores are used to measure the joint density there may also be some error in the results. The cores may be porous resulting in some error when using the normal procedure of weighing in air and water (AASHTO T-166). For porous mixtures a test method such as the

vacuum seal method must be used. Any core that absorbs more than approximately 1% moisture when following the T-166 method should probably use the vacuum seal method or a similar method that can be used for porous mixtures. (The standard T-166 states that a core has to absorb at least 2% moisture before being required to use another test method. However, the 2% water absorption is too high for most mixes. At 2% absorbed water significant error in the measurement of voids is expected.)

Generally, a 6-in. core provides a slightly higher joint density than a 4-in. sample since the 6-in. sample has a higher percentage of area outside the joint. In most cases the difference in measured density between 4- and 6-in. samples is small but in limited cases the difference can be significant.

PROCEDURES USED TO OBTAIN DENSITY IN THE LONGITUDINAL JOINT

As discussed there are three areas that must be carefully controlled to ensure that a satisfactory joint is obtained. The first problem area is compaction of the unconfined, free edge. Some states have begun using asphalt cements for tack coats to help provide a better bond between the layer being constructed and the underlying layer. The asphalt cement tack coat improves the bond enough to help hold the free edge in place and, hence, improve the compactibility of the free edge.

Another procedure that has been used to help solve the unconfined, free edge problem is to cut back the edge 2 to 3 in. prior to placing the second lane (Figure 5). This process removes the loose material adjacent to the edge that was caused by the edge being rolled. While this process is not always easy to use on highways it has worked very well on airfields. On airfields the cutting wheel has been shown to be very important in obtaining satisfactory joint density.

When paving the hot side of the joint it is essential that material be pushed tightly into the joint so that there is sufficient material for compaction. This means that any extensions of the screed and auger be done properly to ensure that material is pushed into the joint and not allowed to simply “roll” into the joint and that the compaction underneath the screed be uniform from one end of the screed to the other.

As shown in Figure 4, the hot side of the mat should overlap the cold side of the mat sufficiently to ensure that there are no gaps between the two lanes. When the excess material is luted from the cold side to the hot side (Figure 6) some extra material is provided in the joint to help increase density during the rolling process.

Sufficient material must be placed adjacent to the joint to allow for adequate compaction. As stated earlier this is typically about 20% additional material placed prior to compaction to allow for roll down. Some trial and error will probably be necessary to establish the correct height on the hot side to ensure that adequate compaction is obtained and at the same time the mixture is rolled smooth with the adjacent lane.

Rolling of Joint

There are a number of ways that have been used to successfully compact the longitudinal joint. Generally these methods involve first rolling the joint and then proceeding to the opposite side to roll the free edge. One method is to roll on the hot side about 6 inches away from the joint as shown in Figure 7. With this method the roller can be operated in the vibratory mode. The next

FIGURE 5 Cutting back of free edge.

FIGURE 6 Luting of longitudinal joint.

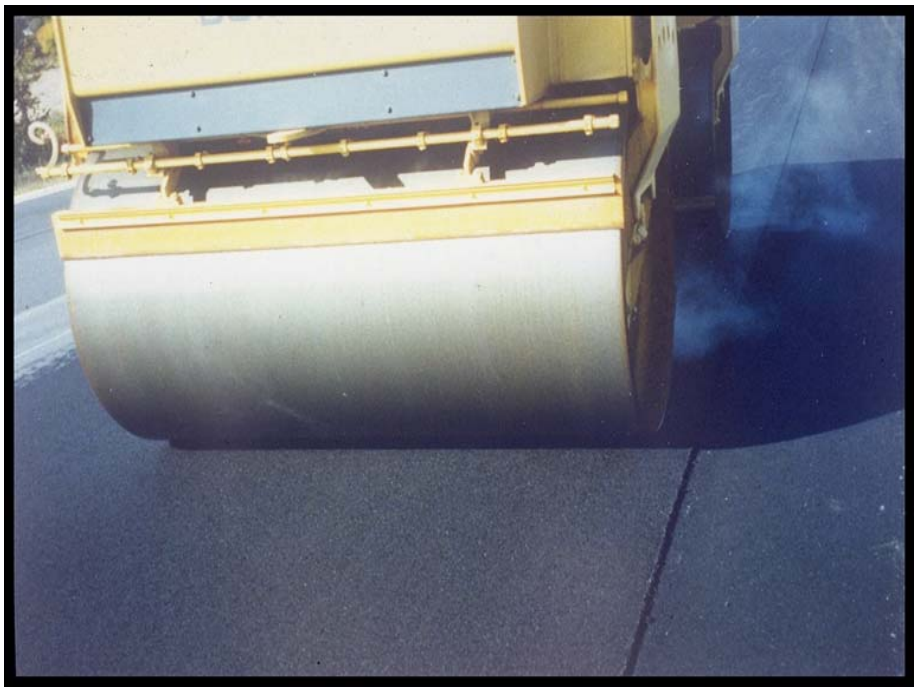

FIGURE 7 Rolling of longitudinal joint.

pass would roll down the 6-in. wide mound that is left between the surface that is rolled and the cold side. Care has to be taken with this process to ensure that when the 6-in. strip is rolled no cracks develop. If not done correctly there is a tendency to develop a crack on either side of the 6-in. strip.

A second method is to place the roller on the cold mat and overlap approximately 6 in. on the hot side. With this method the roller must operate in the static mode to ensure that no damage is done to the cooled HMA. This method applies a large amount of pressure to the material in the 6 in. overlap, resulting in improved density at the joint.

A third method is to roll on the hot side and overlap about 6 in. on the cold side. The vibrator can be on during this process. All three of these methods have worked successfully on some projects. If the results are not satisfactory it is suggested that one of the other methods of rolling be considered.

A rubber tire roller is very good for rolling longitudinal joints since the rubber tires provide a kneading action and can reach down into localized low spots to help provide compaction (Figure 8).

Use of Sealant at Longitudinal Joint

Good density is important to ensure that longitudinal joints perform satisfactorily. Another approach that has been used to obtain good performance is to apply a sealant at the longitudinal joint during construction. The sealant can waterproof the joint even though the density may still be low. One product that has been used is an asphalt sealant material that can be applied from a roll to the joint. In this case the heat from the HMA melts the asphalt material which then seals the voids in the joint. Rubberized asphalt has also been used to seal the joint during the

FIGURE 8 Use of rubber tire roller for compacting joint.

construction process. Experience has shown that sealing the joint has resulted in improved performance of the joint but care must be exercised to not apply too much sealant in the joint since this material may then bleed to the surface.

Tapered Joint

There has been significant work using the tapered joint concept. The most common approach is what is typically referred to as the notched wedge joint (Figure 9). This approach provides a notch where the thickness of the material being placed is reduced by approximately ¾ to 1 in. The material is then transitioned down to approximately zero over a distance of approximately 1 ft. This approach has been shown to generally provide good performance. It is more difficult to use when the layers are thin or when the nominal maximum aggregate size is relatively large. One advantage of this approach is the improved safety for traffic provided by the tapered wedge. It is much safer for vehicles when they drop off the overlay over the wedge rather than dropping straight down from a vertical joint.

FIGURE 9 Notched wedge joint.

SUMMARY

Poor performance of the longitudinal joint is one of the biggest problems that is seen in performance of HMA. There are many steps that can be used to improve the quality of the longitudinal joints. Some of these steps to improve longitudinal joints are identified in this report.

RESOURCES

1. Kandhal, P., and S. Rao. *Evaluation of Longitudinal Joint Construction Techniques for Asphalt Pavements (Michigan and Wisconsin Projects)*. NCAT Report 94-1, January 1994.
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3. Kandhal, P., T. Ramirez, and P. Ingram. *Evaluation of Eight Longitudinal Joint Construction Techniques for Asphalt Pavements in Pennsylvania*. NCAT Report 02-03, February 2002.

Constructing Longitudinal Joints with Lower Voids

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The Port Authority of New York and New Jersey (PANYNJ) had experienced some failures at longitudinal joints. In the past, it had directed the contractors to construct a specific type of longitudinal joint. However, many projects had poor joint densities even when the specified joint was constructed.

Therefore, it was decided to change the longitudinal joint specification into an end-result specification, wherein no specific type of joint construction is mandated, but the lower acceptable joint density is specified and payment is reduced when more than 10% of the longitudinal joints in a lot fall below the lower limit. The Port Authority found that most longitudinal joints will have good density when properly placed and compacted. The in-place density of the longitudinal joint has improved with this end-result specification.

PANYNJ had experienced raveling failures in its hot-mix asphalt (HMA) pavements and, in particular, along longitudinal joints. The PANYNJ specification for HMA pavements had payment adjustments for in-place mat density and plant voids for many years, but not for longitudinal joint voids until the last 2 years. A contractor was liable to receive up to a 50% payment reduction if less than 90% of the mat density or plant voids were not within the specification requirements, or receive up to a 6% bonus if more than 90% of the mat density or plant voids exceed the specification requirements. The acceptable lower limit for in-place mat density is 96.3% of Marshall Density at 75 blows. For plant voids, 90% must be between 2.5% and 5.5%. The pavement is evaluated on a lot basis. A day's production is a lot, which is typically about 2,000 tons for aeronautical pavement work.

The PANYNJ bases all its mix designs on the 75-blow Marshall Method because the majority of our HMA pavements are aeronautical for which we receive funding from the FAA. The FAA has not adopted the Superpave mix design methodology. PANYNJ has adopted many of the mix design requirements of Superpave such as those shown in Figure 1.

In addition, PANYNJ does not allow natural sands. All sand must be manufactured, washed stone sand, and it requires a gradation to be on the coarse side of the maximum density line. A typical 12.5-mm mix used on our runway surface courses is given in Table 1.

This mix would equate to a 12.5-mm Superpave mix at 75 gyrations. Even with our high aircraft volume and loading, PANYNJ would not recommend using 100 gyrations, because the majority of its recent pavement failures have been due to raveling. In particular, longitudinal joint raveling has been a problem. Its pavements do not exhibit plastic deformation. Mat densities, in general, are very good, averaging 5% in-place voids.

In order to reduce the joint raveling problem, PANYNJ decided to increase its joint density requirements. Until recently, bonus money would not be awarded for a lot if less than 90% of the longitudinal joints had densities above the lower acceptance limit of 93.3% of Marshall density. However, this did not provide much of an incentive for a contractor to produce quality joints to meet the end-result specification. PANYNJ does not direct the contractor as to the type of joint to construct. PANYNJ's experience is that, it is best left to the contractors to

VFA: 65%–75%
VMA: 14% for a 19-mm mix
VMA: 15% for a 12.5-mm mix
Produce asphalt at 4% total voids
Typically a PG76-22 styrene-butadiene-styrene–modified asphalt is used.

FIGURE 1 Superpave mix design requirements adopted by PANYNJ.

TABLE 1 Newark Liberty International Airport Runway 4L-22R gradation.

Sieve Size	% Passing
¾ in.	100
½ in.	94
⅜ in.	81
No. 4	51
No. 8	35
No. 30	17
No. 200	5.2
Asphalt Content %	5.1

determine how they can construct the best joint to meet the end-result specification. Longitudinal joints with hot mating mats were usually of good quality. The problem joints occurred when a joint was being made against a cold mat. Some contractors chose to make long paving runs and then drop back and pave against a cold edge that was unconfined which invariably produced a poor longitudinal joint. PANYNJ experienced failures in the joints because of poor density, and its coarse mix gradation increased the potential for raveling.

It was decided to increase the lower limit acceptance criteria for longitudinal joints from 93.3% specified by the FAA to 94.3% of Marshall density and to have payment reductions for longitudinal joints that had more than 10% of the lot below this minimum requirement. The maximum payment reduction can be 25%. The payment adjustment equations for longitudinal joints are given in Table 2 and in Table 3 for mat densities and plant voids. Furthermore, joints that were being made from two different lots, not the same days production, would also be tested for conformance to this density requirement. All density determinations are made by taking a four-inch diameter core directly over the joint for each sublot. Typically, four joint cores are taken per lot. The location of the joint cores is at the same station as the sublot mat density cores. If there are two joints at a location, both are cored and the lower of the two densities is used for payment. If two different sublots of material were used to construct the joint, the sublot with the lowest bulk density would be used to calculate the in-place density. Granted, this would give the contractor an advantage, but would remove a point of contention.

This new joint specification was adopted in 2003 for repaving Runway 4L-22R at Newark Liberty International Airport. The in-place voids data is given in Table 4.

TABLE 2 Adjustment to Contract Compensation for In-Place Joint Density

Percentage of Material Within Tolerance Limits	Percentage Adjustment of the Unit Price
96–100	106
90–96	PWL + 10
80–90	0.25 × PWL + 77.5
65–80	PWL + 17.5
Below 65	75

TABLE 3 Adjustment to Contract Compensation for In-Place Mat Density and Marshall Air Voids

Percent of Material Within Tolerance	Percent Adjustment of the Unit Price
96–100	106
90–96	PWL + 10
80–90	0.5 (PWL) + 55
65–80	2.0 (PWL) – 65
Below 65	50 or remove

TABLE 4 Runway 4L-22R

Location	Average In-Place Void (%)
Mat	4.4
Longitudinal Overall	6.5
Hot Joint	5.4
Hot to Cold Joint	8.3

The in-place voids overall for the longitudinal joints averaged 6.5%. This is good but could be better. The runway was paved with two pavers in echelon; the paving lane widths were 20 ft. Initially, the contractor ran the paving lanes about 2,000 ft. Of course, by the time the mating joint was placed against the unconfined edge, it was cold. Therefore, he did not produce longitudinal joint densities much below 10% voids, and was advised that payments would be reduced by 25% in accordance with Table 2. The contractor's position was that the in-place voids at the surface of the joint were tighter than those at the bottom due to the unconfined edge of the initial paving lane. His contention was that the joint was sealed at the surface and that water would not permeate the joint, initiating premature failure. Table 5 shows the results of joint cores tested in layers, which indicate the uniformity of poor density throughout a common tapered joint and considerably improved density on vertically cut joints.

The contractor decided to maintain the same paving pattern, but construct vertical longitudinal joints. The contractor cut back the cold mat about 6 to 8 in., tacked the vertical face, and placed the hot mat against the cut. This decreased the in-place joint voids by 2% to 4%, and the contractor did not receive a reduction in payments.

TABLE 5 Longitudinal Joints: In-Place Voids

Core Number	Type of Joint	First Lane Paved	First Lane Paved	Second Lane Paved
		Top 1”	Bottom	Top 1”
12-2	Tapered	11.0%	12.3%	9.7%
12-4	Tapered	8.7%	9.8%	10.0%
12-6	Tapered	7.9%	11.2%	8.2%
12-8	Tapered	9.5%	9.2%	8.8%
15-8	Tapered	11.7%	15.3%	14.5%
16-2	Tapered	14.3%	14.6%	12.2%
Average	Tapered	10.5%	12.1%	10.6%
13-2 B	Cut	6.7%	6.6%	5.1%
13-4 B	Cut	7.6%	8.3%	6.8%
13-4 C	Cut	5.3%	7.0%	2.2%
13-6 B	Cut	6.9%	7.6%	4.7%
20-2	Cut	8.3%	7.4%	10.4%
20-4	Cut	6.1%	5.3%	8.3%
Average	Cut	6.8%	7.0%	6.3%

In conclusion, by giving the contractor an incentive to produce a better joint by imposing reduction in payment for poor joints, the quality of the joint increased. In the future, PANYNJ may require contractors to rout and place a sealant in joints that receive a vertical cut, because they have a tendency to open in cold weather. Overall, we received a very good pavement.

Measurement of Longitudinal Joint Density in Asphalt Pavements Using Nuclear and Nonnuclear Gauges

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Several studies have shown that the long-term performance of asphalt pavement near a longitudinal joint can be predicted by the material density near the joint just after construction. However, the asphalt material near a joint is less homogeneous than the material in the middle section of a lane. As a result, estimating the density of a joint requires obtaining density measurements at several different locations. Density gauges that employ non-destructive methods are excellent tools for this purpose.

This paper outlines the key features of both nuclear and nonnuclear density gauges that can be used to measure density near longitudinal joints in asphalt pavements. It discusses the gauge properties, such as precision, lateral extent of measurement, depth of measurement, effect of an air gap between the pavement surface and the gauge, and absolute density. A good understanding of gauge properties will assist the user in selecting the proper gauge for the specific job.

INTRODUCTION

Relatively low density near the longitudinal joint has been identified as a cause of premature failure in several asphalt pavements (*1*). Several studies have shown that the area near the longitudinal joint of a multilane pavement performs well when the density in that area is as close as possible to the density in the middle section of the lane (*2,3,4,5,7*).

The quality of the final construction can be assessed by comparing the density near a longitudinal joint with density in other areas of the paved lane. Several states have proposed quality assessment criteria based on pavement density.

Estimating the density near the longitudinal joint by using any physical method is a challenging task. This is because the material near the joint is less homogeneous than the material in the center of the paved lane and the subject area is a narrow strip 6 to 8 in. wide.

Currently, the density near the longitudinal joint is estimated by measuring the density of extracted cores (destructive testing) using water displacement methods (*6*). The core specimen volume itself is small (6 in. in diameter and 2 in. thick) and the pavement area near a joint from where the specimen was extracted is not homogeneous. Therefore, the estimated density near a joint, based on density measurements of *a few* specimens, is prone to error.

The properties near the longitudinal joint can also be estimated by measuring the density using non-destructive methods (*2,7*). Since the instruments used for these measurements only require contact with the pavement surface during the measurement, the density near a joint can be measured at many locations. This improves the possibility of obtaining good measurements of the density near the longitudinal joint. The density gauges can also be used for quality control purposes during construction.

Currently there are two types of gauges used to measure density of asphalt pavements. The nuclear gauge uses gamma rays to directly measure the density. The nonnuclear gauge uses electric fields to measure the dielectric constant and relates this to density. The industry uses nuclear gauges operating in backscatter modes for density measurements. Some nuclear gauges have a general backscatter mode and others, the thin-layer gauges, are designed specifically to obtain the density of the top-most layer in a multilayer pavement. For more information about manufacturers of density gauges, consult any industry buyer's guide such as the ones published by asphalt magazines.

These gauges have been primarily designed to measure density of heterogeneous materials. With the low level of homogeneity in hot-mixed asphalt, the techniques are tailored to probe large volumes. When these gauges are used for longitudinal joint evaluation, where the density in a narrow strip near the joint (within 6 to 8 in.) is required and the material is less homogeneous than in the middle section of a lane, it is even more important to understand the gauge properties in selecting the proper gauge for a job.

This paper outlines the key features of the nuclear and nonnuclear gauges that can be used to measure density near longitudinal joints in asphalt pavements. It discusses the gauge properties, such as precision, lateral extent of measurement, depth of measurement, effect of an air gap, and absolute density. As examples, for the purpose of clarification of these features, the Troxler Model 3440 will be used as a general backscatter nuclear gauge, the Troxler Model 4640 as a thin-layer backscatter nuclear gauge, and the Troxler Model 2701 as a nonnuclear gauge.

LONGITUDINAL JOINTS

In a multilane pavement, the first paved lane is referred to as the cold lane and the adjoining lane as the hot lane. In the cold lane, the area near the joint (within 6 to 8 in.) has a lower density than in the middle section of the lane. The surface course of a pavement may have a thickness of 1 to 4 in. and may have a joint with a nearly vertical boundary or a wedge-shaped boundary such as in the Michigan wedge joint.

Because of the way the joint is constructed, the area very near the joint (0 to 4 in. from joint) may not be flat, and may have a different texture when compared with the surface over 4 in. away from the joint. The area on either side of the longitudinal joint may also be at different elevations. The unconfined cold lane joint edge may also be coated with 'tack' or other adhesives for various purposes.

Generally a joint is considered well constructed when the density near the joint is within 2% to 3% of the density near the center of the lane, and may not be less than 90% of the maximum theoretical density. In well-constructed pavements, this leads to having a density contrast of approximately 2.5 to 4 pounds per cubic foot (PCF) between areas near the joint and the center of the lane.

For discussing the gauge parameters, four laboratory geometries for the longitudinal joint were considered.

- Geometry 1: A joint with a near vertical edge with a top layer thickness of 4 in. (Figure 1).
- Geometry 2: A joint with a 45-degree edge with a top layer thickness of 1.5 in. (Figure 2).
- Geometry 3: A joint with a 1.5 : 12 wedge with a top-layer thickness of 1.5 in. (Figure 3).
- Geometry 4: A joint with a 45-degree edge with a top-layer thickness of 1.5 in. with the edge made with low-density material (Figure 4).

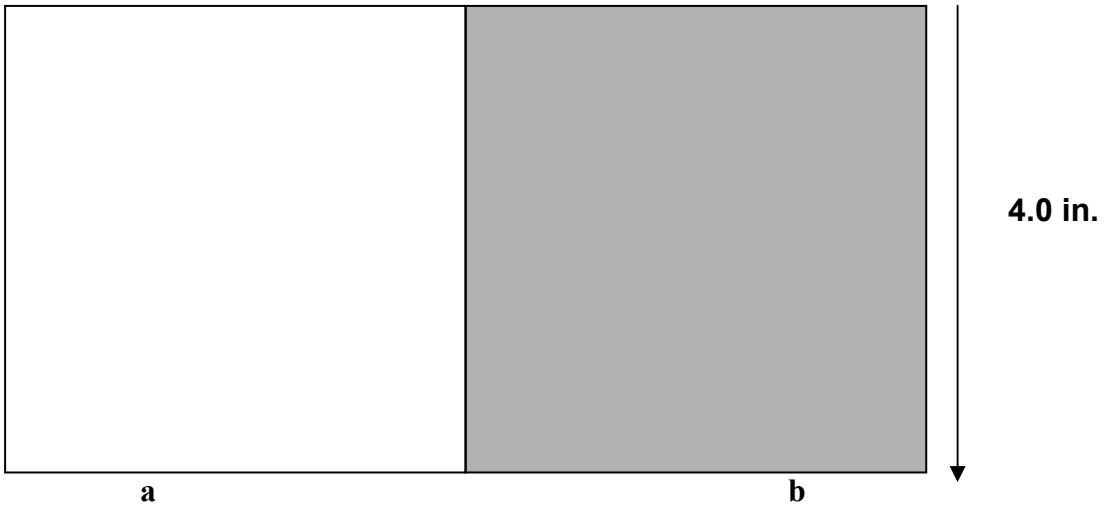

FIGURE 1 A side view of a simulated longitudinal joint with a nearly vertical edge: butt joint (Geometry 1): (*a*) high-density medium and (*b*) low-density medium.

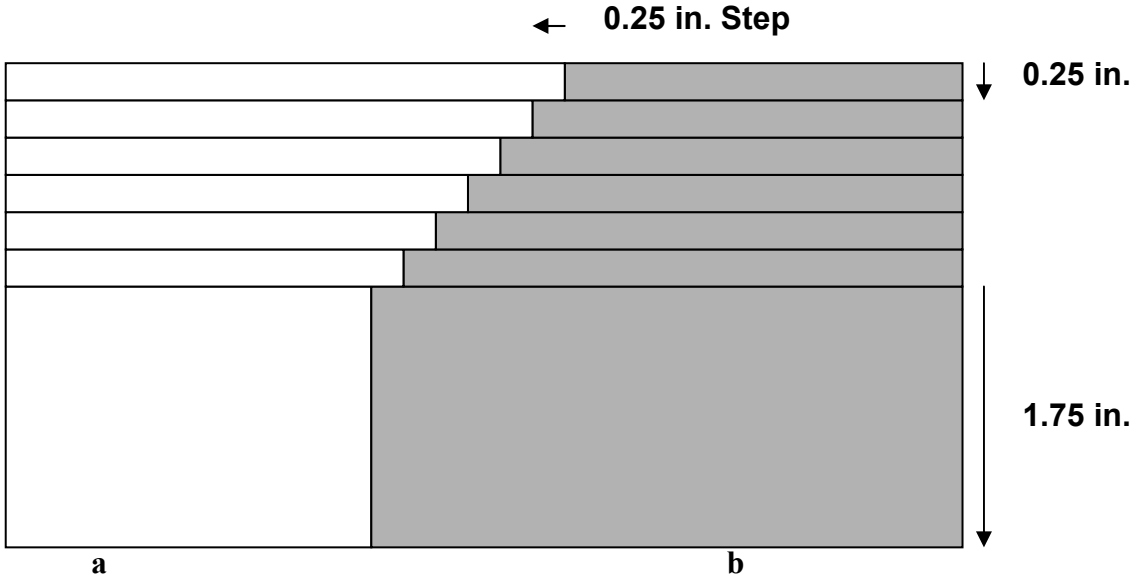

FIGURE 2 A side view of a simulated longitudinal joint with a 45-degree edge–wedge joint (Geometry 2): (*a*) high-density medium and (*b*) low-density medium. Six 0.25-in.-thick sheets were used on each side of the joint to construct the top 1.5 in. of the joint.

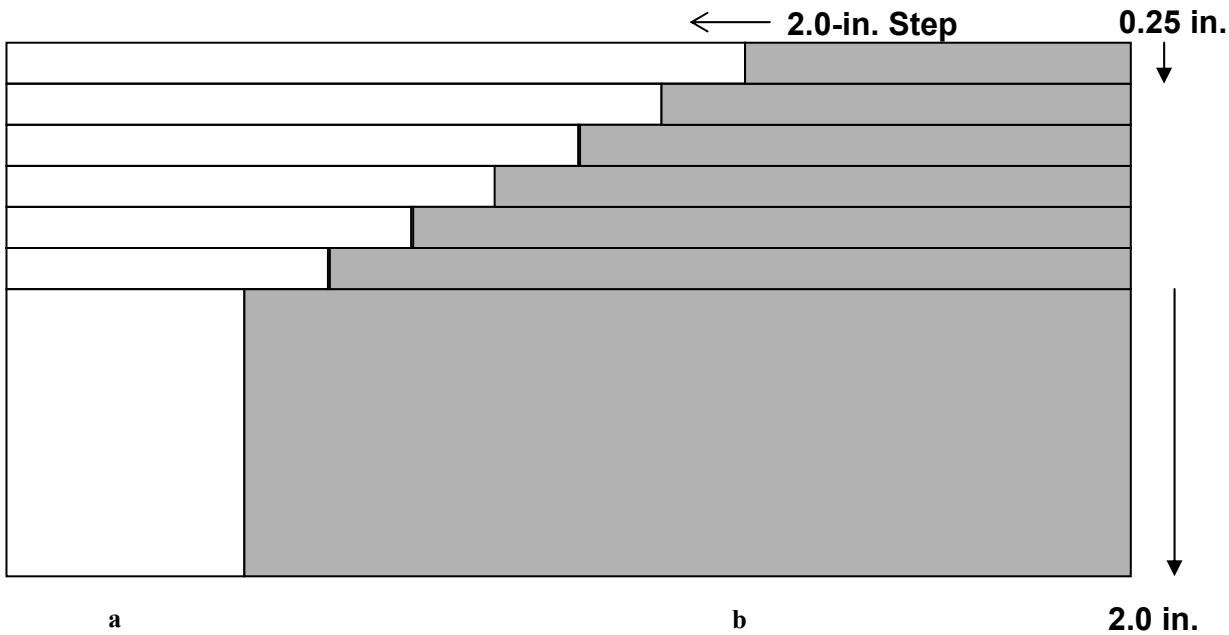

FIGURE 3 Side view of a longitudinal joint with a 1.5: 12 wedge (Geometry 3): (*a*) high-density medium and (*b*) low-density medium.

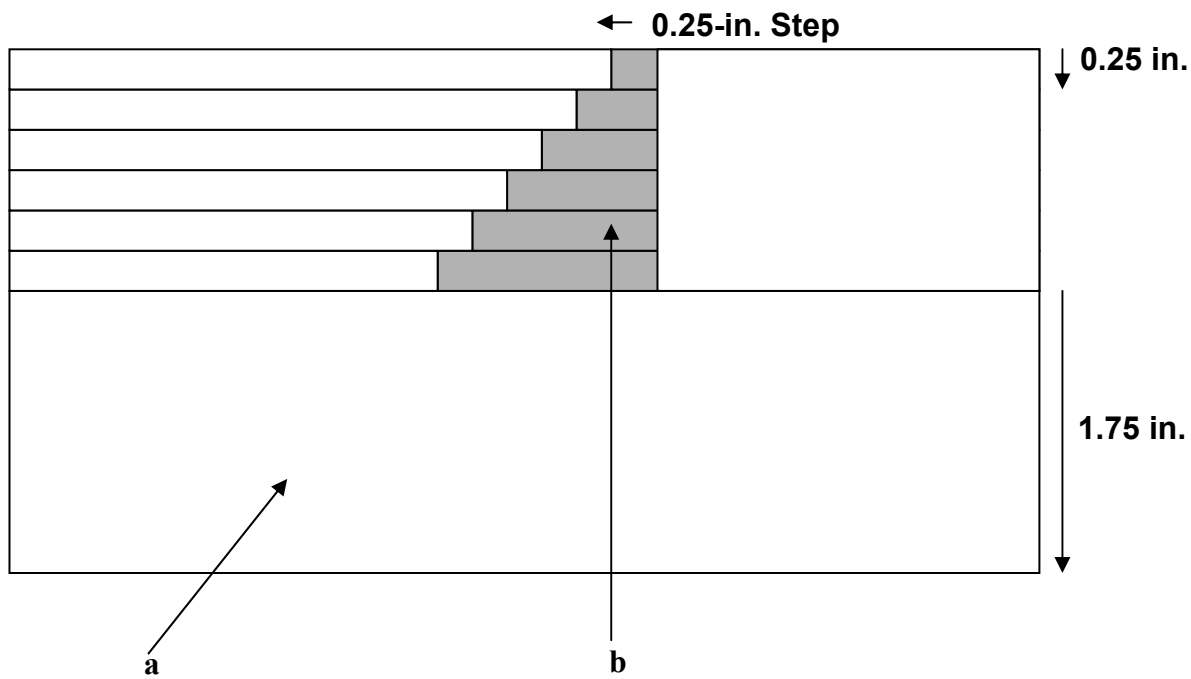

FIGURE 4 Side view of a longitudinal joint with a 45-degree edge with the edge made with a low-density material (Geometry 4): (*a*) high-density medium and (*b*) low-density medium.

For this study, homogeneous natural and synthetic materials of known density were used for constructing these joints. The materials used for the two sides were different in Geometry 1, 2, and 3. In Geometry 4, the same material was used for the two sides of the staircase shaped low- density region.

DENSITY MEASUREMENT OF CORES USING WATER DISPLACEMENT METHODS

Consider the estimation of density near a longitudinal joint by extracting a 6-in. diameter core and by using the AASHTO T-166 method for measuring the density (*6*). Assume that the acceptable density of the pavement is 150 PCF. As shown in Figure 5, assume also that:

1. The area near the longitudinal joint in the hot lane has a density of 150 PCF,
2. The area within 6 in. from the longitudinal joint in the cold lane has a density of 142 PCF, and
3. The other areas of the cold pavement have a density of 150 PCF.

Assume that the acceptance criterion for the longitudinal joint density is within 2% of the density in the middle section of the lane.

Next, as illustrated in Figure 5, three cores are extracted. First a core right on the joint (A), second a core from the area within 6 in. of the longitudinal joint in the cold lane (B), and third a core in the area 3 to 9 in. from the longitudinal joint in the cold lane (C). Determining the density of the core, density values of 146, 142, and 146 PCF for cores A, B, and C, respectively are obtained. The low-density area can be best seen in the measurement of the density in core B. Based on the density of core A, this longitudinal joint fails the acceptance test.

The density at the joint may be subjective. With a tapering edge in the cold lane, core A may give a density less than 146 PCF as more cold material would be present in the core. With more material build up in the hot side of the edge, core A may give a density value higher than 146 PCF thus incorrectly passing the acceptance test. Therefore, the location for density measurement should be decided considering the geometry of the joint, properties of the asphalt mix, as well as the properties of the paver screed.

Figure 6 shows the calculated density of 6-in. diameter cores extracted at various locations across the joint. Also shown here are the point-density (density determined from a pencil thin core) distribution across the joint and the density of 4-in. diameter cores for comparison. As can be seen from Figure 6, the density of the material in inhomogeneous areas depends on the specimen volume.

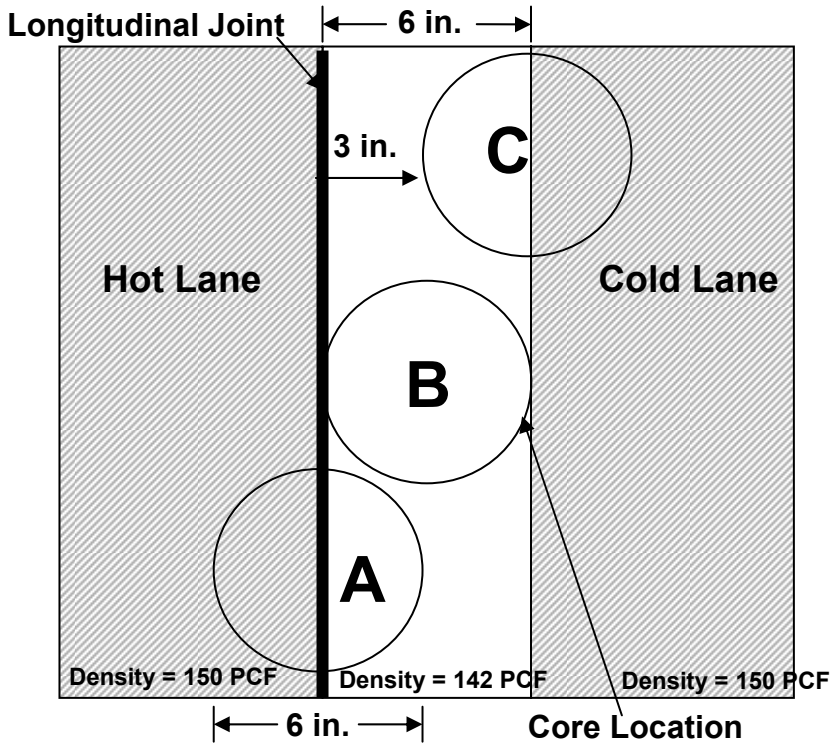

FIGURE 5 Possible locations of extracting 6-in. diameter cores for determining low density areas near a longitudinal joint.

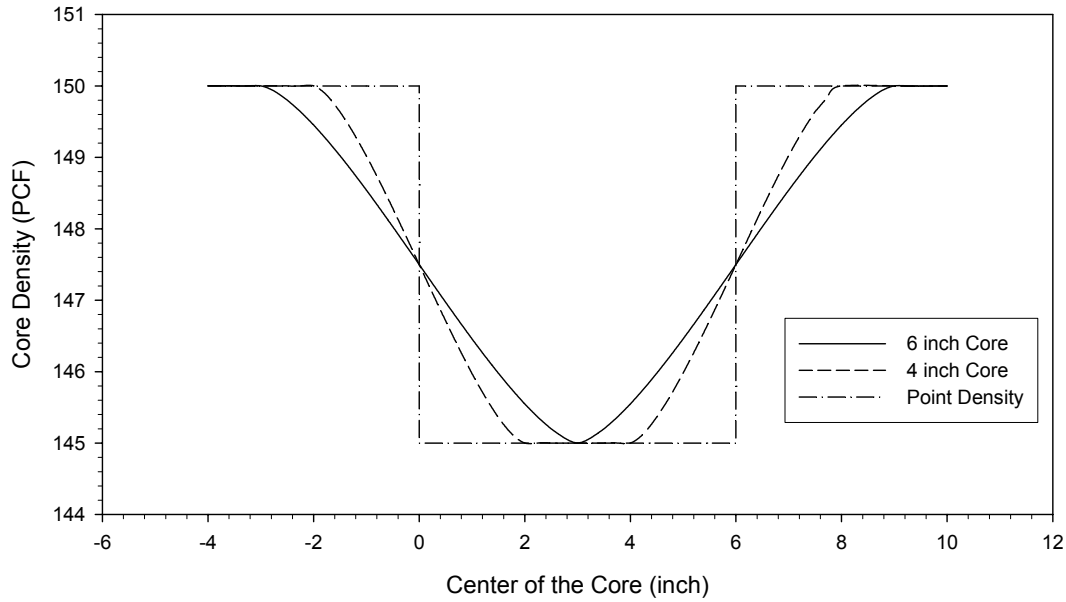

FIGURE 6 The calculated density of cores of three diameters: 6-in., 4-in., and 0 (point density). The dotted dashed curve shows the point density distribution across the longitudinal joint.

DENSITY MEASUREMENT USING GAUGES

Gauge Parameters

Precision

The operator's manual of a gauge should show the instrumental precision of the gauge. The reported instrumental precision was determined from multiple measurements made on homogeneous natural and synthetic materials. The instrumental precision depends on the material density for gauges that use non-linear density calibration models. In general, the measurement precision in the field is poorer than the instrumental precision.

Depth of Measurement

Currently, there is no industry standard for defining and determining the depth of measurement (DOM) of density gauges. Until recently there has been no interest for such a standard since gamma rays used in nuclear gauges are highly penetrating and can probe deeply into a material. The DOM can be defined based on measurements made on a layer of one type of material of a particular thickness placed on another type of material. For nuclear gauges a layer of aluminum is placed on a layer of the magnesium-aluminum composite. Because aluminum has a density of approximately 160 PCF and the magnesium-aluminum composite has a density of approximately 135 PCF, the layer structure using these materials can be used to simulate typical field conditions. The DOM is defined as the thickness of the top layer for which the bottom layer contribution to the density reading is less than 5%. With this definition, an aluminum top layer of the DOM thickness resting on a thick magnesium-aluminum composite would result in a density reading of 158.8 PCF providing practically no density information of the 135 PCF bottom layer.

This tells us that for a better estimation of the density, the DOM should be comparable to the layer thickness of the joint. When the DOM is equal to the layer thickness of the joint, such a gauge provides more complete density information. If the DOM is less than the layer thickness, this gauge estimates the density of the top-most portion of the material only up to the DOM thickness. This gauge cannot provide the density of the lower part of the layer. If the DOM is greater than the layer thickness, this gauge gives the 'weighted' average density of the top layer and the top part of the bottom layer.

Because of the differences in the DOM of different types of gauges, when placed on a location near a joint, they will provide different density readings.

Lateral Extent of Measurement (Prior to Paving the Hot Lane)

All gauges have their sensor mounted on a base with a width of at least 7.5 in. The actual sensing area is about 4 to 5 in. wide. The lateral extent of the measurement is determined by the active width of the sensor and the depth of measurement of the gauge.

When the two lanes near the longitudinal joint are at different elevations, it may be impossible to determine the true density when a gauge is placed centering the joint. The air gap formed between one half of the sensor and the lane results in an error in the density reading. However, in this study, it was found that these gauges can be placed about 1 in. away from the

longitudinal joint even when the adjoining lane is not completed without effecting the readings. Figure 7 shows the geometry used in this study.

Because of the differences in the lateral extent of measurement of different types of gauges, when placed on a location near a joint, they may provide different densities.

Air Gap Effect

An air gap between the gauge base and the asphalt surface skews the gauge readings for both nuclear and nonnuclear gauges. An air gap can be easily formed when a gauge is placed on debris on the asphalt surface, when the asphalt surface is not flat, or with a sudden change in the elevation of the asphalt surface near the longitudinal joint. Generally, an air gap causes a reduction in the density reading.

Absolute Density of the Pavement

Because of the wide variety of asphalt mixes used in the industry, the gauge manufacturers calibrate their gauges to an average asphalt mix (factory calibration). When using on a particular job site, the gauge should be calibrated for that particular mix by correlating the readings to core density readings (field calibration). Such readings are obtained at randomly selected location in the middle of the pavement to estimate a constant bias between the two methods. Since the density contrast is low near a properly constructed longitudinal joint (2.5 to 4 PCF), a regular field calibration performed may be sufficient. For an improperly constructed longitudinal joint with high density contrast (6 to 10 PCF), it is recommended to verify the slope of the calibration curve. A special calibration can be performed to determine the slope and the bias. Because of the differences in the specimen volume involved in the core method and the gauge methods, cores taken near the joint should not be used to calibrate gauges in the field.

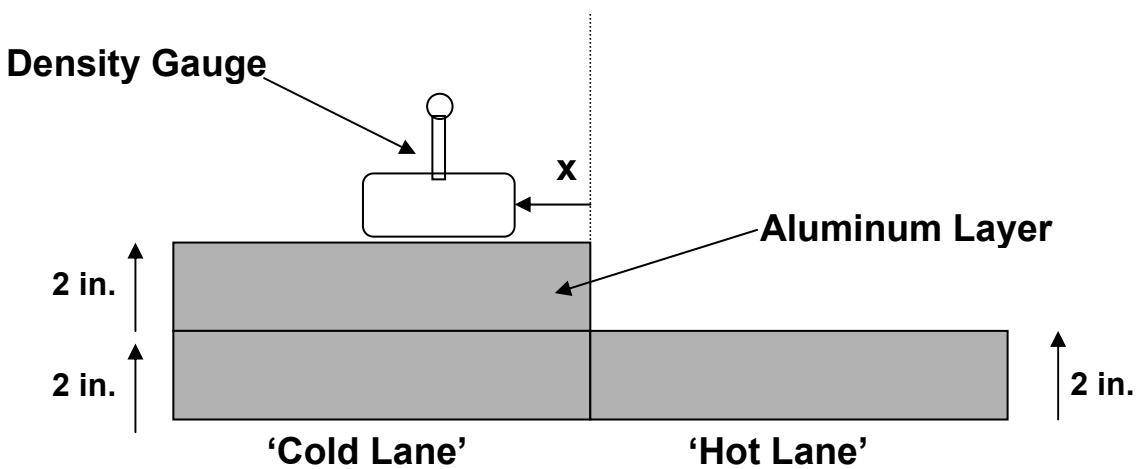

FIGURE 7 Material slab geometry used to measure the effect of the gauge density reading in the cold lane near the edge prior to paving the hot lane.

Gauge Density Measurements on Various Simulated Joint Geometries

Gauge measurements were made on four laboratory geometries for the longitudinal joint. We used homogeneous natural and synthetic materials of known densities for constructing these geometries.

DENSITY GAUGES

Backscatter Nuclear Density Gauge (M 3440, Backscatter Mode of M 4640)

Precision

The field precision of the nuclear gauge depends primarily on the counting time. A counting time of 2 min provides about 0.4 to 0.7 PCF precision at 1-sigma. It is recommended to use the average of two 1-min readings to estimate the spot density near a joint.

Depth of Measurement

The DOM is about 2.6 to 3 in.

Lateral Extent of Measurement (Prior to Paving the Hot Lane)

When the long side of the gauge base is placed right near the joint in the cold lane, the gauge overestimate the density (Figure 8). This is because the air boundary scatters fewer photons (gamma-rays) towards the detector. When a flat surface is available 1 in. from the joint, the gauge can be used to laterally probe the low-density areas (Figure 9).

Air Gap Effect

When a gauge is placed right on the joint with a 1 mm elevation difference between the adjacent lanes, the density is under-estimated by about 1 to 2 PCF.

Gauge Density Measurements on Various Simulated Joint Geometries

Geometry 1 Figure 10*a* shows the measurements. The discontinuity at the vertical joint was located at the 0 in. location. This boundary is sensed by the gauge when the center of the sensor is within 2.5 in. from the joint.

Geometry 2 Figure 10*b* shows the measurements. The step-wise discontinuous boundary started at the 0 in. with a thickness of 1.5 in., and extended to –1.25 in. (see Figure 2). The right end of the boundary was sensed by the gauge when the center of the sensor was about 2 in. to the right. The left end of the boundary was sensed by the gauge when the center of the sensor was about 4.5 in. to the left. When the gauge was placed centered on the joint, gauge density showed a bias towards the low density side. The measurements are asymmetric because of the deep penetration of the measurement.

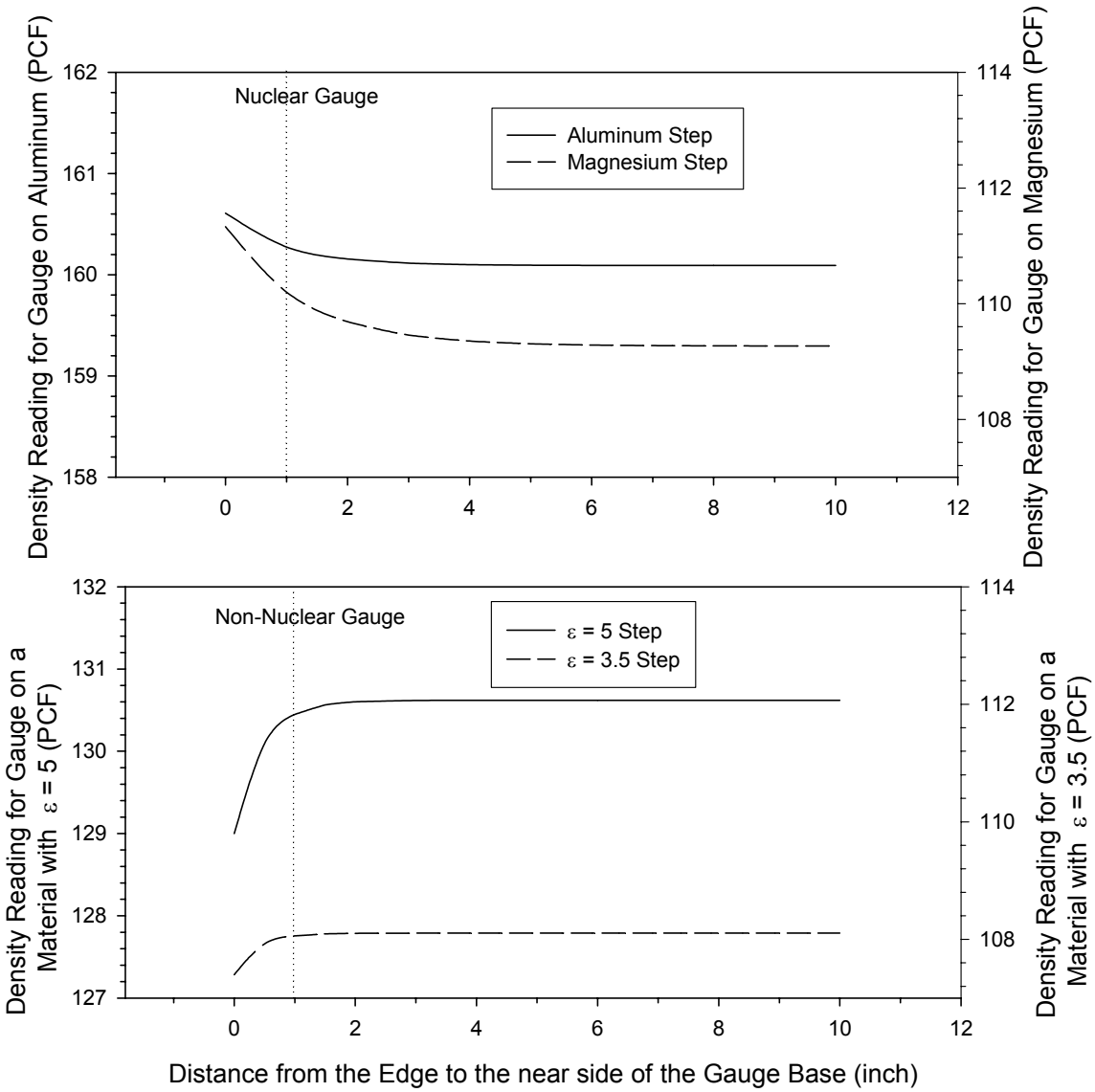

FIGURE 8 The cold lane gauge density dependence on the gauge distance to the edge prior to paving the hot lane.

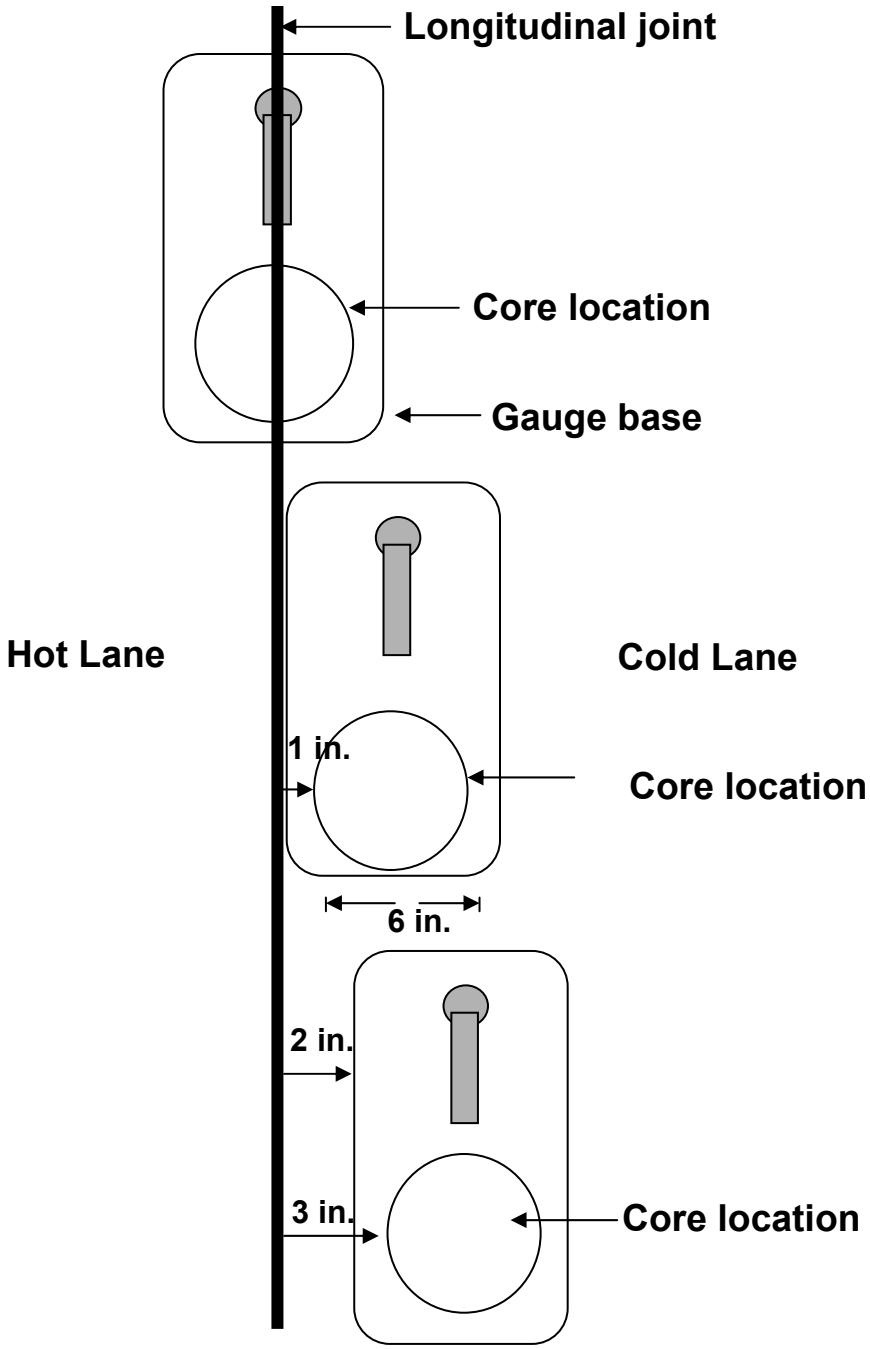

FIGURE 9 Positioning of the gauge base to probe a similar volume as represented by a core specimen.

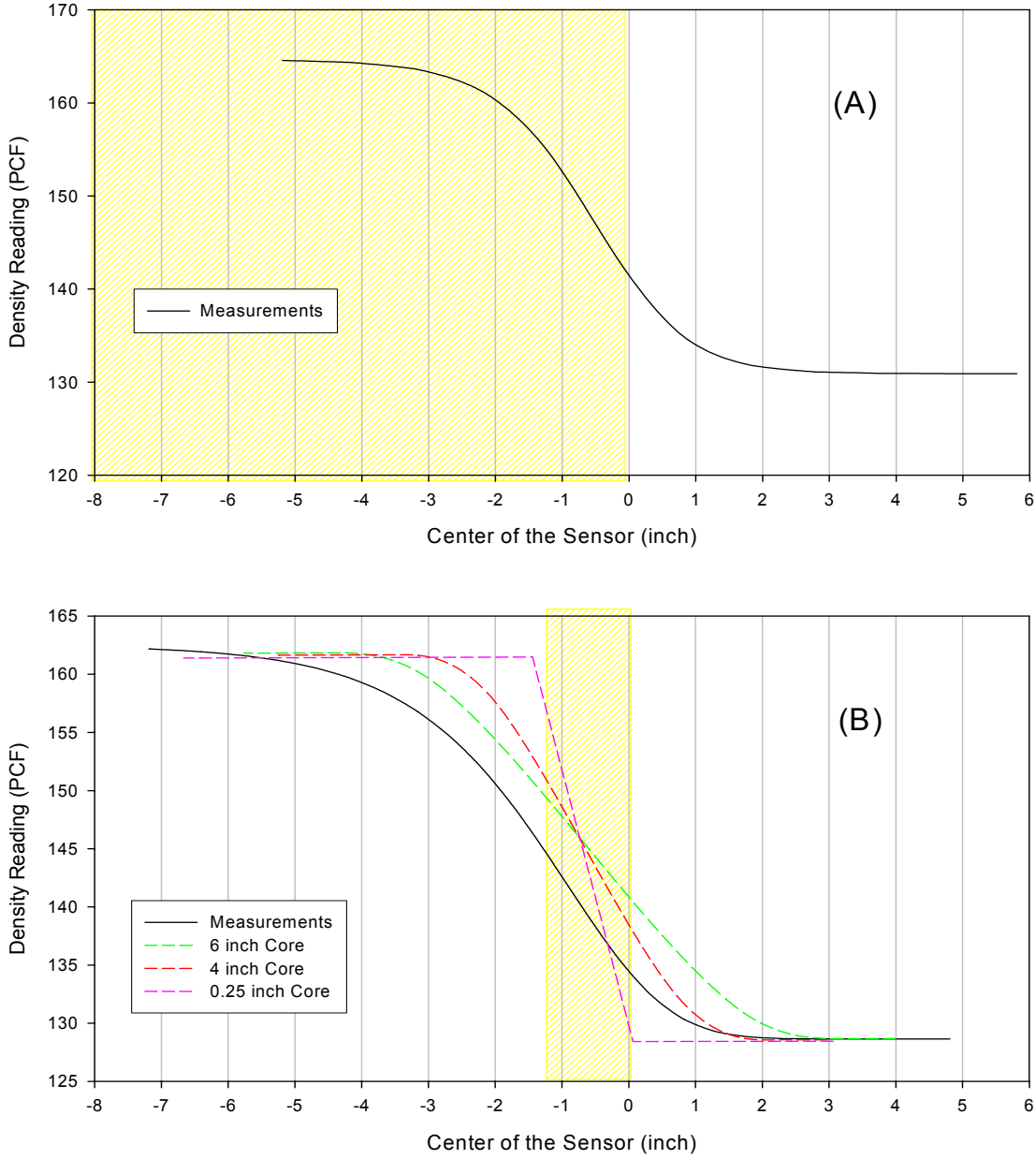

FIGURE 10 Measurements with a general backscatter nuclear gauge (M 3440) for (*a*) Geometry 1 and (*b*) Geometry 2.

Geometry 3 Figure 11*a* shows the measurements. The step-wise discontinuous boundary started at 0 in. with a thickness of 1.5 in. and extended to –10 in. (see Figure 3). The right end of the boundary was sensed by the gauge when the center of the sensor was about 0.5 in. to the right. The left end of the boundary was sensed by the gauge when the center of the sensor was

about 4 in. to the left. When the gauge was placed centered on the joint, gauge density showed a strong bias towards the low-density side because of the deep penetration of the measurement.

Geometry 4 Figure 11*b* shows the measurements. The step-wise discontinuous boundary started at 0 in. with a thickness of 1.5 in., and extended to –1.5 in. (See Figure 4). This region contains a material lower in density than on either side of the discontinuity. The right end of the boundary was sensed by the gauge when the center of the sensor was about 3.5 in. to the right. The left end of the boundary was sensed by the gauge when the center of the sensor was about 3 in. to the left. When the gauge was placed centered on the joint, gauge density showed a strong bias towards the small low density region.

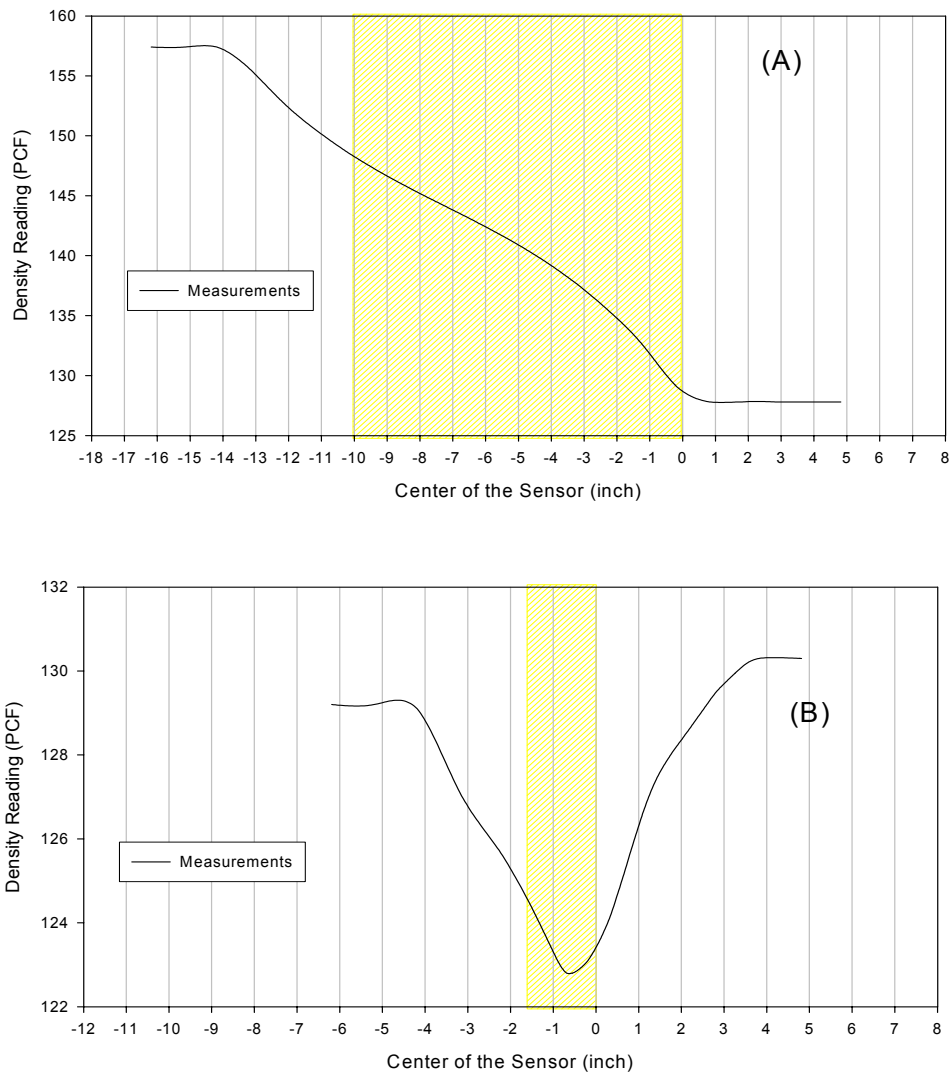

FIGURE 11 Measurements with a general backscatter nuclear gauge (M 3440) for (*a*) Geometry 3 and (*b*) Geometry 4. The shaded area in (*a*) shows the 12:1.5 joint and in (*b*) shows the low-density region.

Thin-Layer Nuclear Density Gauge (M 4640)

Precision

The field precision of the nuclear gauge depends primarily on the counting time. A counting time of 2 min provides about 0.4 to 0.7 PCF precision at 1-sigma. It is recommended to use the average of two 1-min readings to estimate the spot density near a joint.

Depth of Measurement

The DOM can be set to the top-layer thickness for layers of thickness 1 to 2 in.

Lateral Extent of Measurement (Prior to Paving the Hot Lane)

When the long side of the gauge base is placed right near the joint in the cold lane, the gauge overestimates the density (Figure 8). This is because the air boundary scatters fewer photons (gamma-rays) towards the detectors. When a flat surface is available 1 in. from the joint, the gauge can be used to laterally probe the low-density areas (Figure 9).

Air Gap Effect

When a gauge is placed right on the joint with a 1-mm elevation difference between the adjacent lanes, the density is under-estimated by about 2 PCF.

Gauge Density Measurements on Various Simulated Joint Geometries

Geometry 1 Figure 12*a* shows the measurements. The discontinuity at the vertical joint was located at 0 in. location. This boundary is sensed by the gauge when the center of the sensor is within 1.5 to 2 in. from the joint.

Geometry 2 Figure 12*b* shows the measurements. The step-wise discontinuous boundary started at 0 in. with a thickness of 1.5 in., and extended to –1.25 in. (see Figure 2). The right end of the boundary was sensed by the gauge when the center of the sensor was about 1.5 in. to the right. The left end of the boundary was sensed by the gauge when the center of the sensor was about 2.5 in. to the left. When the gauge was placed centered on the joint, gauge density showed a bias towards the low density side.

Geometry 3 Figure 13*a* shows the measurements. The step-wise discontinuous boundary started at 0 in. with a thickness of 1.5 in., and extended to –10 in. (see Figure 3). The right end of the boundary was sensed by the gauge when the center of the sensor was about 1.5 in. to the right. The left end of the boundary was sensed by the gauge when the center of the sensor was about 2 in. to the left. When the gauge was placed centered on the joint, gauge density showed a strong bias towards the low-density side.

Geometry 4 Figure 13*b* shows the measurements. The step-wise discontinuous boundary started at 0 in. with a thickness of 1.5 in., and extended to –1.5 in. (see Figure 4). This region contains a material lower in density than on either side of the discontinuity. The right end of the

boundary was sensed by the gauge when the center of the sensor was about 3.5 in. to the right. The left end of the boundary was sensed by the gauge when the center of the sensor was about 2.5 in. to the left. When the gauge was placed centered on the joint, gauge density showed a strong bias towards the small low density region.

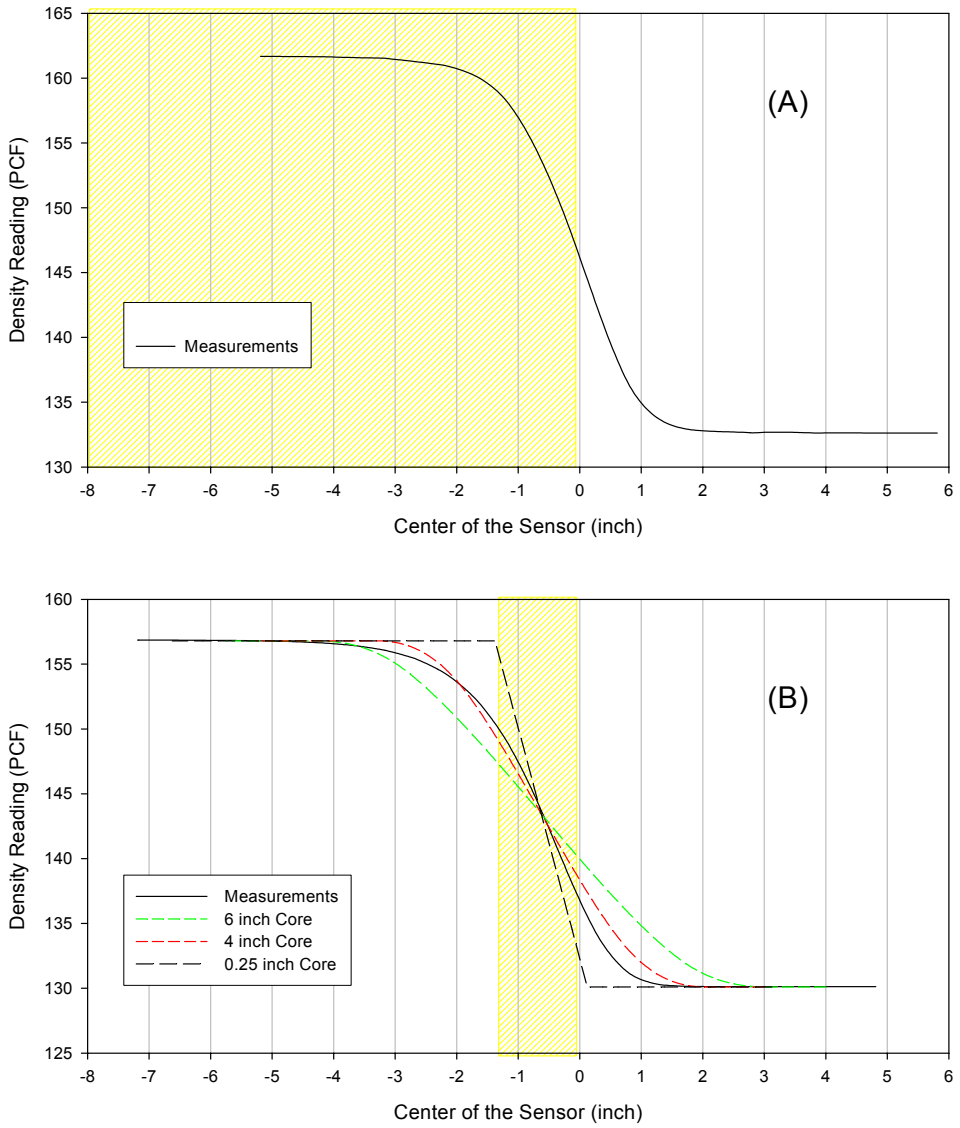

FIGURE 12 Measurements with a thin-layer backscatter nuclear gauge (M 4640) for (*a*) Geometry 1 and (*b*) Geometry 2. The shaded area in (*b*) shows the 45-degree joint.

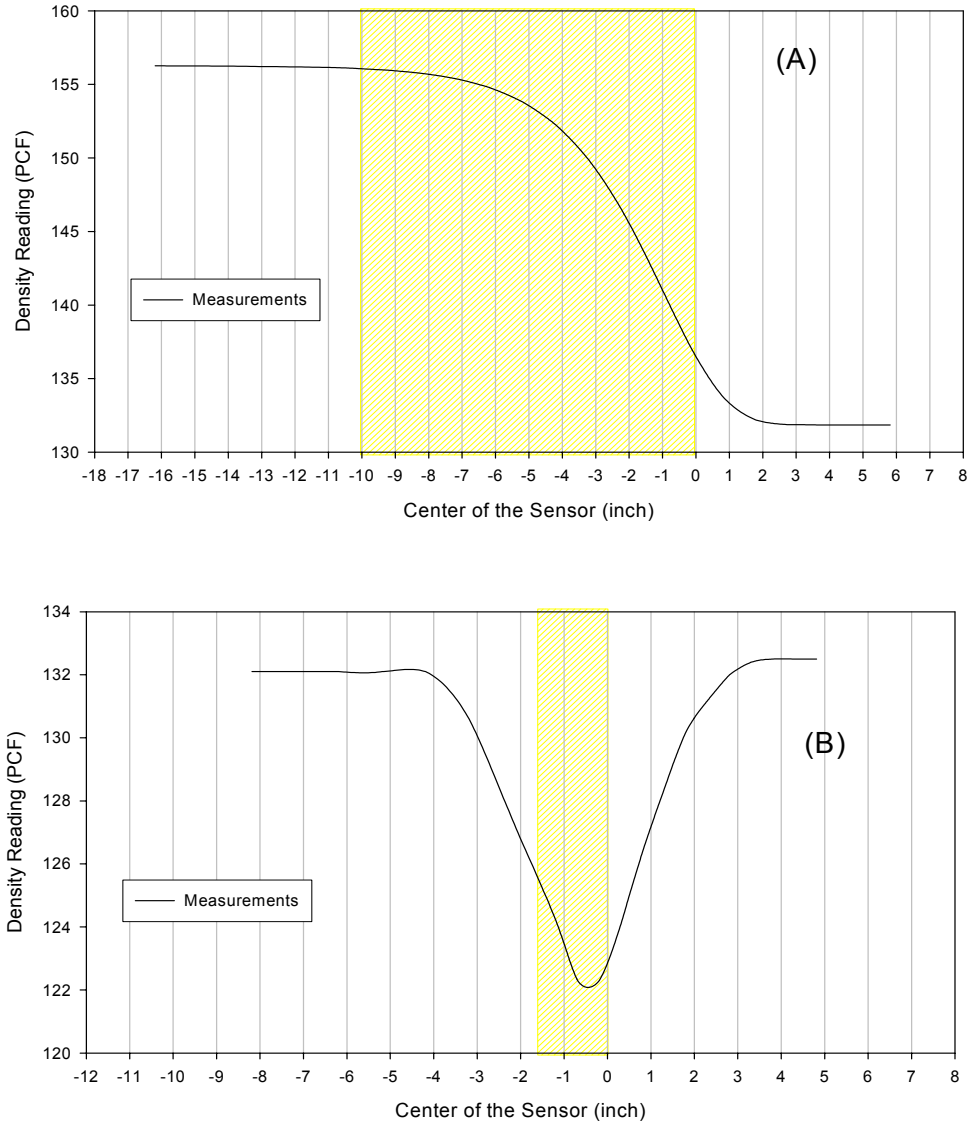

FIGURE 13 Measurements with a thin-layer backscatter nuclear gauge (M 4640) for (*a*) Geometry 3 and (*b*) Geometry 4. The shaded area in (*a*) shows the 12:1.5 joint and in (*b*) shows the low-density region.

Nonnuclear Density Gauge (M 2701)

Precision

The field precision depends on the homogeneity and texture of the HMA material. The average of 4 to 8 readings on places slightly shifted about the measurement location provides 0.5 to 1.2 PCF precision at 1-sigma level for most of the surface course materials.

Depth of Measurement

The response of a nonnuclear gauge is related directly to the dielectric constant of the material, hence it is difficult to select the appropriate materials for estimating the DOM. For example, some HMA mixes have high densities but low dielectric constants. Others have low densities but high dielectric constants. The dielectric constant of different asphalt mixes ranges from 3 to 7 depending on the aggregate and binder used in the design. By using a top layer of material with a dielectric constant of approximately 3 placed on another material with a dielectric constant of 7, the DOM is about 1 in.

Lateral Extent of Measurement (prior to paving the hot lane)

When the long side of the gauge base is placed right near the joint in the cold lane, the gauge underestimates the density (Figure 8). This is because the air reduces the overall dielectric constant. When a flat surface is available 1 in. from the joint, the gauge can be used to laterally probe the low-density areas (Figure 9).

Air Gap Effect

When a gauge is placed right on the joint with a 1-mm elevation difference between the adjacent lanes, the density is under-estimated by 0.2 PCF at 95 PCF, 2 PCF at 123 PCF and 6 PCF at 160 PCF readings. The densities specified are for a gauge with a regular factory calibration.

Gauge Density Measurements on Various Simulated Joint Geometries

Geometry 1 Figure 14*a* shows the measurements. The discontinuity at the vertical joint was located at the 0 in. location. This boundary is sensed by the gauge when the center of the sensor is about 2 to 2.5 in. to the right of the joint.

Geometry 2 Figure 14*b* shows the measurements. The step-wise discontinuous boundary started at 0 in. with a thickness of 1.5 in., and extended to –1.25 in. (see Figure 2). The right end of the boundary was sensed by the gauge when the center of the sensor was about 2.5 in. to the right of the joint. The left end of the boundary was sensed by the gauge when the center of the sensor was about 1.5 in. to the left.

Geometry 3 Figure 15*a* shows the measurements. The step-wise discontinuous boundary started at 0 in. with a thickness of 1.5 in., and extended to –10 in. (see Figure 3). The right end of the boundary was sensed by the gauge when the center of the sensor was about 2.0 in. to the right. The left end of the boundary was sensed by the gauge when the center of the sensor was about -6 in.. When the gauge was placed 4 in. to the left of the joint, gauge density showed a strong bias towards the high density (dielectric) side.

Geometry 4 Figure 15*b* shows the measurements. The step-wise discontinuous boundary started at 0 in. with a thickness of 1.5 in. and extended to –1.25 in. (see Figure 4). This region contains a material lower in density than on either side of the boundary. The right end of the boundary was sensed by the gauge when the center of the sensor was about 2 in. to the right. The left end of the boundary was sensed by the gauge when the center of the sensor was about 0.5 in. to the left. When the gauge was placed centered on the joint, gauge density showed a weak bias towards the small low density region.

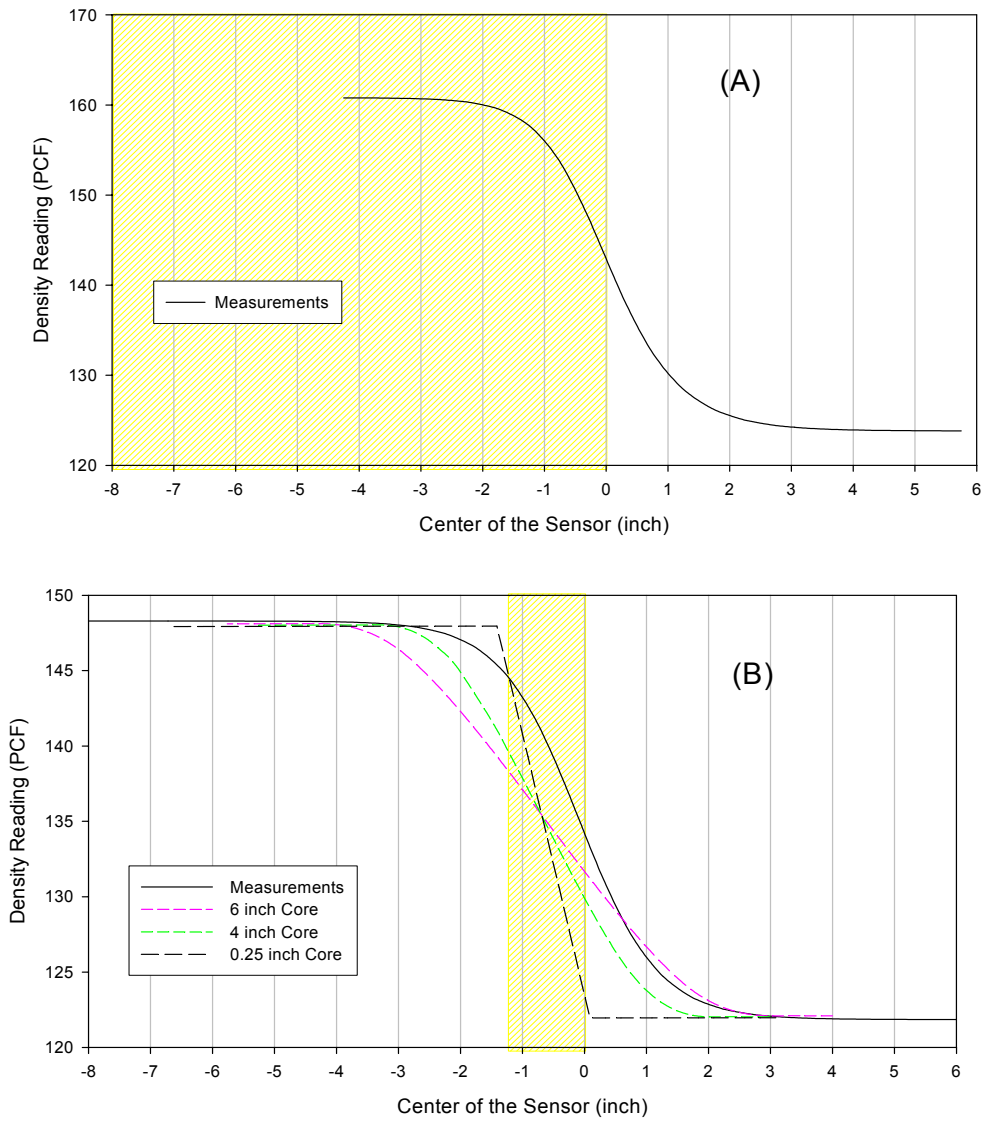

FIGURE 14 Measurements with a nonnuclear gauge (M 2701 B) for (*a*) Geometry 1 and (*b*) Geometry 2. The shaded area in (*b*) shows the 45-degree joint.

DISCUSSION OF FINDINGS

Measuring density near inhomogeneous or density-segregated areas around longitudinal joints is a challenging task. For the conventional method of density determination using core extraction, the final density depends on the diameter of the core. Two cores having a perfect geometry centered at the same point with the same height but different diameters, when extracted near a joint, may not provide the same density. Different types of density gauges when placed on the same location near a joint may also provide different density values because of their differences in the measurement properties.

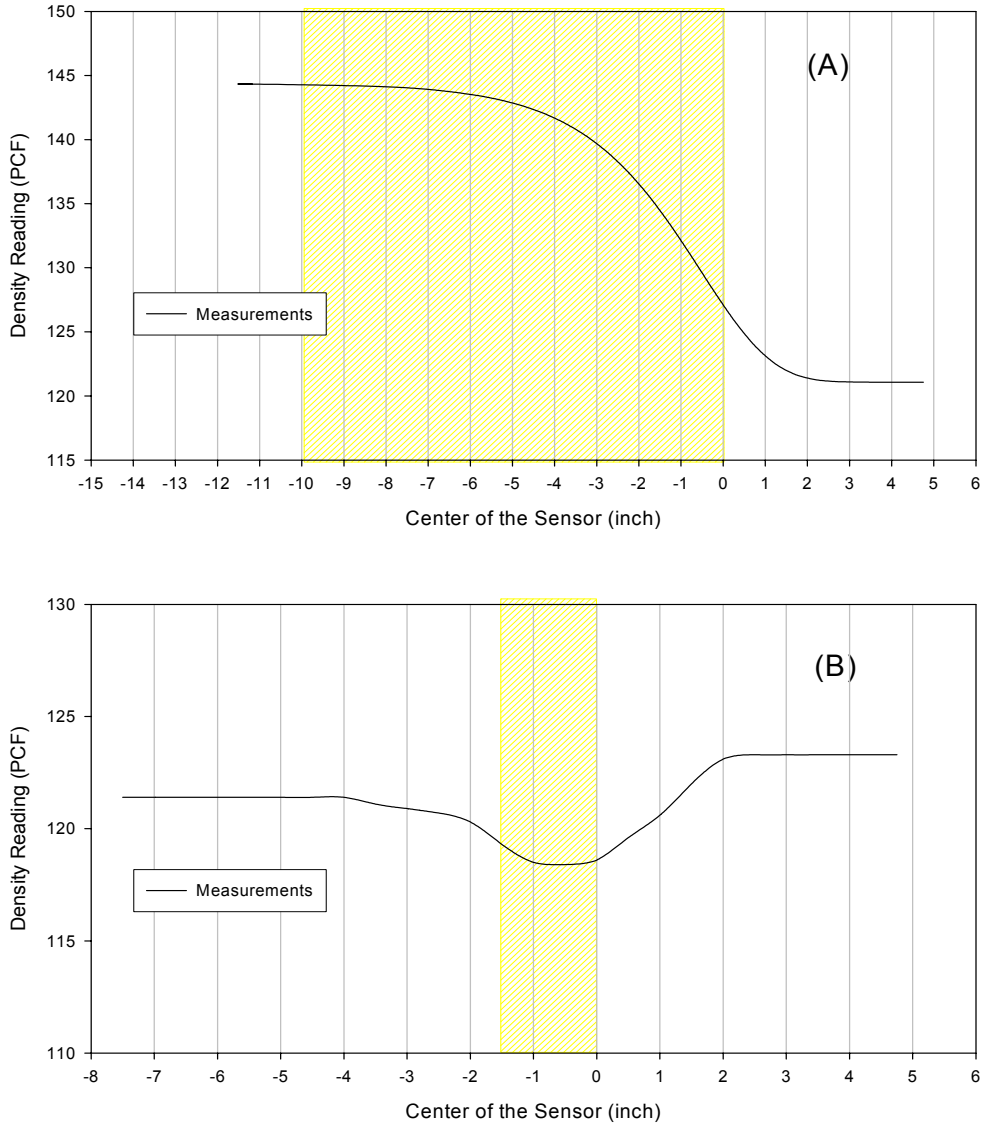

FIGURE 15 Measurements with a nonnuclear gauge (M 2701 B) for (*a*) Geometry 3 and (*b*) Geometry 4. The shaded area in (*a*) shows the joint and in (*b*) the low-density area.

The *material density* as determined from the core extraction method is currently used for predicting the quality or long-term performance of the joint. The acceptance criteria for joint density has been developed over many years of experience. When density gauges are used for joint evaluation, because of the differences in the measurement properties, it is necessary to develop specific acceptance criteria for each type of gauge and each type of geometry.

The joint quality during construction can be controlled using density gauges. When constructing the cold lane, density near the joint can be monitored during compaction. When completing the joint, density near the joint on hot lane can also be monitored during compaction.

This study attempted to document the gauge properties for three common types of density gauges. The study of laboratory models for joints has shown that the nuclear gauges have a high sensitivity of probing low-density regions in an inhomogeneous section. For the density range used in HMA materials, the gamma-ray detection probability exponentially increases with the decreasing density in the surrounding region. For nonnuclear gauges, the gauge reading is biased towards the material closest to the sensor, thus reducing the ability to read the true density of the interested volume.

Based on this study, following recommendations can be made when using density gauges for longitudinal joint evaluation.

1. Before selecting a gauge for evaluating longitudinal joints, determine the measurement precision for the particular pavement. For nuclear gauges, the counting time determines the precision. For nonnuclear gauges the number of readings needed for determining an average (by moving the gauge about 2 in. around the location) determines the precision.
2. Based on the top-layer thickness, determine the type of gauge to use. For 1-in. thick layers, use a thin-layer nuclear gauge or a nonnuclear gauge; for 1.0- to 2-in. thick layers, use a thin-layer nuclear gauge; and for 2- to 3-in. thick layers, use any nuclear gauge set to the general backscatter mode. For layers with thickness less than 1-in., a thin-layer gauge or a nonnuclear gauge may be able to use with some caution as the bottom-layer affects the density reading.
3. The transverse position or location relative to the longitudinal joint should also be considered in order to obtain the highest density contrast. This should be decided based on the geometry of the longitudinal joint.
4. Since the air-gap density error for any type of a gauge is significant when compared to the acceptance density criteria, gauges should be placed centered on the joint only when the two lanes near the joint are at the same elevation. Also remember to check for the flatness of the surface. If the surface is not flat, obtain measurements at the closest flat location. For any gauge, always ensure that the base is in complete contact with the asphalt surface.
5. The material near a joint is inhomogeneous therefore the density values as determined by different techniques may be different. The density of cores extracted near joints may not be used for calibrating density gauges.

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Construction of Durable Longitudinal Joints

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The durability of longitudinal joints in asphalt concrete pavements is a major problem at many locations across New Jersey. After a short period of time under traffic, these joints tend to ravel. In some cases the raveling is severe enough to completely erode the mix at the joint leaving a gap between the lanes.

There are a number of factors during construction that directly affect the durability of a longitudinal joint. The first is the compaction of the unsupported edge of the first lane of mix placed. The second is the amount of overlap of mix on the second lane over the top of the first lane. The third factor is related to the raking of the mix at the joint. The final primary factor is the compaction of the mix at the joint when the second lane is placed adjacent to the first lane.

The construction of a longitudinal wedge joint is also discussed. Construction of a durable longitudinal joint is a question of good workmanship by the contractor using proper construction techniques.

INTRODUCTION

The durability of longitudinal joints in asphalt concrete pavements is a major problem at many locations in New Jersey. Because of maintenance of traffic considerations, most asphalt pavement resurfacing is done one lane at a time. One lane is paved and then the adjacent lane is paved, with a cold joint between the two lanes. After a short period of time under traffic, these joints tend to ravel. Typically, however, the raveling takes place only on one side of the joint. An example of this type of raveling is shown in Figure 1.

Sometimes the raveling at the longitudinal joint is severe enough to completely erode the surface course material adjacent to the joint and expose the underlying pavement layer, leaving a gap between lanes. This type of failure is illustrated in Figure 2. Not only is the long-term durability of the pavement compromised, the safety of the traveling public is also a major concern.

Longitudinal joints between traffic lanes or between the mainline pavement and the adjacent roadway shoulder can be properly constructed (*1*). Care must be taken to accomplish four primary tasks—compaction of the unsupported edges of the first lane paved, overlap of the mix on the second lane over the top of the first lane, raking of the mix off the first lane, and compaction of the joint between the two lanes (*2*). Using the proper construction techniques, the cost of building a durable longitudinal joint is no more expensive than building a poor longitudinal joint.

FIGURE 1 Raveling at a longitudinal joint.

FIGURE 2 Severe erosion of mix at a longitudinal joint.

COMPACTION OF THE FIRST LANE

One of the keys to the construction of a durable longitudinal joint is proper compaction of the unsupported edge of the first lane of pavement placed (*5*). The mix placed by the paver will have a slope on its outside edge. The amount of slope depends on the type of end or edger plate on the paver screed but typically is approximately 60 degrees. This wedge, which is shown in Figure 3, does not receive the same amount of compaction as the rest of the mix due to its shape and position.

The type of roller used and its position in regard to the unsupported edge of the pavement significantly affects the amount of density that can be obtained. A pneumatic (rubber) tire roller normally cannot be used within about 150 mm (6 in.) of the unsupported edge of the lane without pushing the mix sideways due to the high pressure in the rubber tires. A steel wheel roller, however, can be operated in three different locations in regard to the unsupported edge of the lane. Two of those positions, however, do not provide the proper compactive effort needed to achieve the required amount of density.

As seen in Figures 4 and 5, if the edge of the drum of a steel wheel roller, operated either in the vibratory mode or in the static mode, is inside the unsupported edge of the pavement lane, two things may happen. First, the mix has a tendency to widen out—to move in a transverse direction. This is due to the shear loading on the mix at the edge of the steel drum. The amount of movement is depended on the properties of the asphalt concrete mixture—a tender mix will shove or move more than will a stiff mixture. In addition, a crack will typically form at the edge

FIGURE 3 The shape of the wedge inhibits proper compaction.

FIGURE 4 Edge of the roller drum inside the unsupported edge of pavement.

FIGURE 5 Edge of the roller drum inside the unsupported edge of pavement.

FIGURE 6 Crack formed from rolling inside the edge of pavement.

of the drum, as shown in Figure 6. The sideways movement of the mix causes this crack. Further, the movement of the mix creates a dip at the unsupported edge of the lane, making it much more difficult to match the joint when the second lane is placed. Placing the edge of the roller drum inside the edge of the lane is not good practice.

The edge of the steel drum can be placed directly over the unsupported edge of the lane. This is shown in Figure 7. In this case, the mix at the unsupported edge will still move sideways or transversely under the force of the roller. Although the mix will widen out, no crack will typically form since the edge of the steel wheel roller drum is right at the edge of the lane. Due to the transverse movement of the mix, however, the opportunity to obtain density at the unsupported edge is not possible.

The proper location for the edge of the steel drum is illustrated in Figures 8 and 9. The drum is extended over the edge of the lane by approximately 150 mm (6 in.). In this case, there will be no transverse movement of the mix since there is no shear loading at the edge of the steel drum. Since the mix does not move transversely, no crack is formed. Density is achieved because the edge of the steel drum is compacting air instead of shoving the mix sideways.

An example of the movement of the unsupported edge of the lane is shown in Figures 10 and 11. In the first figure, Figure 10, the paver operator is placing the mix in a straight line. The unsupported edge of the mix is straight. The angle of the mix at the unsupported edge—the wedge—is also seen in this same figure.

In Figure 11, a double drum vibratory roller is shown compacting the unsupported edge of the same stretch of pavement. Because the roller operator is not running the roller in a straight line, the edge of the lane is no longer straight. Where the edge of the steel drum is positioned over the unsupported edge of the lane, as where the roller is currently located in the picture, the mix remains in place and does not move transversely. In the foreground of the figure, however, the roller drum was inside the edge of the lane and the mix has moved sideways. The amount of transverse

FIGURE 7 Edge of the roller drum directly on the unsupported edge of pavement.

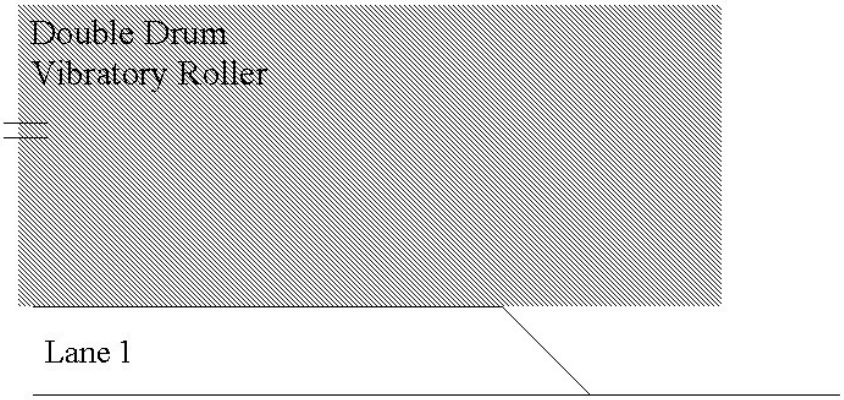

FIGURE 8 Edge of the roller drum extended over the unsupported edge of pavement.

FIGURE 9 Edge of the roller drum extended over the unsupported edge of pavement.

FIGURE 10 Paver placing unsupported edge of pavement.

FIGURE 11 Transverse movement of the mix at the unsupported edge of pavement.

movement is directly related to the location of the edge of the steel roller drum relative to the unsupported edge of the asphalt pavement.

In order to construct a durable longitudinal joint, it is necessary to compact the unsupported edge of the first lane correctly. This is accomplished by extending the drum of a steel wheel roller over the unsupported edge of the lane by approximately 150 mm (6 in.). If this is done, the asphalt concrete mix will not move transversely and a crack will not form in the mix at the edge of the steel drum due to shear loading at the edge of the drum. This lack of movement will allow the second lane to be properly placed and compacted against the edge of Lane 1.

OVERLAP OF MIX FROM LANE 2 TO LANE 1

The second key to the construction of a durable longitudinal joint is related to the amount of overlap of the end or edger plate on the paver screed over the edge of Lane 1 when placing mix for Lane 2. Two items need to be considered. The first is related to the thickness of the uncompacted mix from Lane 2 over the top of the compacted mix at the edge of Lane 1. The second is related to the transverse amount of overlap of the mix from Lane 2 over the top of Lane 1.

Dense graded asphalt concrete mix typically compacts at a rate of 6 mm (¼ in.) per 25 mm (1 in.). This means to achieve a compacted thickness of 25 mm (1 in.), the mix usually must be placed from the back of the paver screed at an uncompacted thickness of about 31 mm (1¼

in.). Further, to obtain a compacted thickness of 50 mm (2 in.), the uncompacted mix must be placed to a thickness of approximately 62 mm (2½ in.). When a mix from Lane 2 is placed over the top of the compacted mix on Lane 1, the mix needs to be high by the amount of compaction that will occur.

The amount of overlap of mix from Lane 2 onto Lane 1 is critical in the construction of a durable longitudinal joint. If an excessive amount of mix is placed over the edge of Lane 1, it will have to be removed by raking the joint or it will be crushed by the rollers. If not enough mix is placed over the edge of the first lane, a depression or dip will occur on the Lane 2 side of the longitudinal joint. In either case, the joint will not perform properly under traffic. The amount of transverse overlap needed is in the range of 25 mm (1 in.) to 40 mm (1½ in.) for proper longitudinal joint construction.

An excessive amount of overlap of the mix from the new lane (Lane 2) onto the old lane (Lane 1) is shown in Figure 12. In this figure, it appears that the edger plate on the paver screed is about 150 mm (6 in.) over the top of Lane 1. With this excessive amount of mix placed on the compacted lane, it is necessary to remove the excess material by shoveling the extra mix off of the pavement, not by raking the mix onto the new lane. In Figure 13, the edger plate is not firmly in place on the asphalt surface, allowing too much asphalt to overlap on the adjacent lane.

FIGURE 12 Excessive amount of overlap mix from Lane 2 over Lane 1, which creates the need for improper raking of the joint.

FIGURE 13 Improper placement of the edger plate results in too much overlap.

The proper amount of overlap is illustrated in Figure 14. In this case, the amount of overlap is in the range of 25 mm (1 in.) to 40 mm (1½ in.). Given this amount of overlap, no mix has to be moved off the top of Lane 1. No raking of the mix at the longitudinal joint is needed.

When milling of the existing asphalt concrete pavement surface is done, a vertical face is formed along the edge of the cutting head on the milling machine. This is significantly different than the slope that is formed by the edger plate on the paver screed. In this case, due to the vertical edge of the adjacent lane of compacted mix, the amount of overlap must be controlled very carefully. To properly construct the longitudinal joint, the amount of overlap of mix from Lane 2 over the non-milled surface should be about 6 mm (¼ in.) to 12 mm (½ in.), maximum.

RAKING THE LONGITUDINAL JOINT

If the proper amount of mix is placed in the proper place, no raking of the mix at the longitudinal joint is necessary. If an excessive amount of mix is placed over the top of Lane 1, raking the mix is needed but this typically results in very poor density at the joint. The third key to a durable longitudinal joint is not to rake the joint during construction.

Figure 15 illustrates improper raking of the longitudinal joint. When raking is done, the amount of mix needed at the joint is usually pushed into the hot mix on Lane 2 by the person doing the raking. By setting the rake down on the compacted mix of Lane 1 and pushing the rake

FIGURE 14 Proper amount of overlap of mix from Lane 2 over Lane 1.

FIGURE 15 Improper raking of the longitudinal joint.

transversely into the mix at the joint, the mix is shoved on top of the hot mix on Lane 2. This makes the mix too low on the Lane 2 side directly at the joint and also too high on the Lane 2 side a short distance away from the joint. Essentially, the mix ends up at the same elevation of each side of the joint. The problem is that the mix on one side of the joint is compacted (Lane 1) and the mix on the other side of the joint is not yet compacted (Lane 2).

In order for the rollers to be able to compact the mix on the hot side of the longitudinal joint, the asphalt concrete mix must be high—6 mm (¼ in.) for each 25 mm (1 in.) of compacted thickness. If the joint is raked flat, the rollers will not be able to compress the mix since it will already be at the same elevation as the compacted mix in Lane 1. This will result in very low density at the longitudinal joint on the Lane 2 side of the joint.

Sometimes a raker will attempt to "bump the joint" with the rake. In this case, the mix in the overlap of Lane 2 on Lane 1 is not pushed over the top of Lane 2 but merely humped up at the joint. If the new mix is at the proper height, the extra material right at the joint will have no place to go vertically. This will result in a bump or ridge along the joint. The rollers will then have a tendency to ride on the ridge of extra mix and not be able to properly compact the hot mix adjacent to the ridge.

Proper raking of the mix on the Lane 2 side of the joint is not raking the mix at all. (Please note location of rake in Figure 16.) Looking back at the illustration shown in Figure 1, it is easy to determine which side of the joint is the Lane 1 side and which side of the joint is the Lane 2 side. The left side of this joint was paved first—Lane 1. The raker placed the rake down on the compacted mix on Lane 1. The raker then pushed the overlapped mix across the top of Lane 2 and into the interior of Lane 2, not leaving the mix on the Lane 2 side of the joint high enough to be properly compacted. Essentially the raker made the elevation of the mix on both sides of the joint—both the compacted and the uncompacted side—the same. This did not provide any mix for the rollers to compact. Given the low level of density on the Lane 2 side of the joint, the mix raveled with the application of traffic.

If raveling occurs at the longitudinal joint, it is most often caused by the raking of the joint and the transverse movement of mix needed at the joint into the interior of the second lane. Raveling typically occurs on the Lane 2 side of the longitudinal joint.

COMPACTING THE LONGITUDINAL JOINT

The final key to constructing a durable longitudinal joint is the location of the rollers during the compaction of the mix at the joint (*6*). The rollers can be placed in several different transverse locations. Only one of those locations, however, provides for the efficient compaction of the longitudinal joint.

In the past, it was often common practice to compact the longitudinal joint from the Lane 1 or the cold side of the joint. The steel wheel roller, operated in the static mode, was located with most of the drum on Lane 1 with only 150 mm (6 in.) to 300 mm (12 in.) of the width of the drum extending over the joint and over Lane 2. Such a compaction operation is illustrated in Figure 17.

This type of compaction operation, however, is very inefficient for a number of reasons. First, most of the weight of the roller is on the previously compacted mix. While the roller is moving over the cold mix, the temperature of the new hot mix in Lane 2 is decreasing, thereby

FIGURE 16 Proper raking at the joint (note location of rake).

FIGURE 17 Compaction of the longitudinal joint from the cold side.

reducing the opportunity to obtain the desired level of density in the new asphalt concrete mix. Second, a vibratory roller cannot be operated in the vibratory mode on the cold side of the joint, on Lane 1, since this may fracture the aggregate in the compacted mix. This reduces the amount of compactive effort that can be applied to mix at the joint. In addition, if there is a different cross slope between the two lanes such as when the joint is located at a crown section on the roadway, only a minimum amount of the weight of the roller will actually be in contact with the mix at the joint due to the different slopes between the two lanes. Rolling the mix from the cold side—the Lane 1 side—is very inefficient and results in a significantly reduced amount of density in the mix at the joint.

Sometimes a steel wheel roller is placed just inside the longitudinal joint on the hot side of the joint—the Lane 2 side. This is done to "pinch the joint" but is not a good practice. With a steel wheel roller, if the mix being placed is tender, locating the edge of the steel drum some 150 mm (6 in.) inside the joint will result in the mix being pushed sideways, similar to the problem at the unsupported edge of the pavement on Lane 1. Because of the side support from Lane 1, however, the mix will simply hump up adjacent to the joint. This will result in a longitudinal ridge being formed along the Lane 2 side of the joint. This, in turn, will result in poor compaction of the mix at the joint since the roller on subsequent passes over the joint will ride on the high spot in the mat—the ridge—and not compact the mix next to the ridge at the joint.

A much better place to position the roller, either a steel wheel roller or a pneumatic tire roller, is a short distance over the top of the joint from the hot side of the joint. For a rubber tire roller, the center of the outside tire of the roller, at the end of the roller with an even number of tires, is placed directly over the top of the longitudinal joint. Placing the roller in this position permits proper compaction of the mix at the joint as well as compaction of the mix on Lane 2. This is an efficient way to compact both the mix at the joint and the mainline pavement.

For a steel wheel roller, the majority of the weight of the drum is placed on the hot mix on Lane 2 with only 150 mm (6 in.) or so of the width of the drum extending over the top of the joint and over the top of Lane 1. This is shown in Figures 18 and 19. Such a rolling pattern allows the roller to apply most of its weight to the new hot asphalt concrete material while still compacting the mix at the joint. In addition, if there is a different cross slope between the two lanes, rolling from the hot side of the joint—the Lane 2 side—will typically achieve a higher amount of compaction at the joint.

In summary, when compacting the longitudinal joint between lanes 1 and 2, the rollers should not be placed on the cold side of the joint. The most efficient location to place the rollers, either pneumatic tire or steel wheel, is on the hot side of the joint with one tire or a small amount of the width of the drum—150 mm (6 in.)—extending over the top of the joint. This type of rolling pattern will result in higher compactive effort being applied to the mix at the longitudinal joint and thus higher density at the joint. In addition, by rolling from the Lane 2 side, much more of the new mix is being compacted at the same time as the mix at the joint, resulting in a more efficient overall compaction operation.

FIGURE 18 Compaction of the longitudinal joint from the hot side.

FIGURE 19 Compaction of the longitudinal joint from the hot side.

CONSTRUCTION OF A WEDGE JOINT

Some governmental agencies require the construction of a wedge joint at the longitudinal joint. This type of joint is shown in Figure 20. The original purpose of the wedge joint was to allow traffic to safely pass over the longitudinal joint from one lane to another while minimizing the difference in the drop off between the lanes. This makes sense from a traffic safety standpoint at the time of construction, but normally does not provide for a very durable longitudinal joint over the long term (*7*).

The wedge joint is typically formed by attaching a piece of metal to the edger plate on the paver screed. This form is used to create both vertical face at the top portion of the unsupported edge and the wedge or slope at the bottom portion of the joint. In most cases, the height of the vertical face is approximately half of the depth of the pavement course. The width of the wedge is typically 300 mm (12 in.) for many projects where a wedge type joint is used.

Two problems typically occur with the construction of this type of joint. First, it is very difficult to properly compact the wedge section. Because of the narrow width of the wedge, a full size roller cannot be used for compaction. Often a very small single drum, static, steel wheel roller is towed behind the paver over the wedge. The amount of compactive effort applied to the mix by this drum, however, is minimal. Depending on the width of the wedge, most of the weight of the roller rides on the adjacent, lower pavement surface and the outside end of the

FIGURE 20 Longitudinal wedge joint.

wedge receives little or no compaction. The slope of the roller drum is usually different than the slope of the top of the wedge. This lack of density in the wedge may provide for deterioration of the longitudinal joint, from the bottom up, within a few years.

Second, the vertical face at the top of the wedge joint is difficult to match when the second lane is placed adjacent to Lane 1. This was discussed above in regard to paving against an adjacent vertical face. The amount of overlap between the mix placed on Lane 2 over the top of the compacted mix on Lane 1 must be kept to a maximum of 12mm (½ in.).

Improved traffic safety at the time of construction may be offset by increased deterioration of the joint with time and traffic.

OTHER CONSIDERATIONS

Some individuals believe that it is necessary to apply a tack coat to the unsupported edge of the first lane before mix in Lane 2 is placed against that edge (*8*). There does not seem to be any definitive evidence that the application of a tack coat provides any benefit in terms of the long-term durability of the longitudinal joint. In general, the application of the tack coat is not very uniform because the material is typically placed using hand spray methods. Further, even when placed using an asphalt distributor, the distributor does not always run in a straight line compared to the unsupported edge of Lane 1. Thus the tack coat does not always end up evenly applied on the joint.

Paving in echelon is sometimes done with the idea of creating a hot longitudinal joint and eliminating future deterioration at the joint. In this process, one paver is used to place the mix in Lane 1. A second paver is used to place the mix in Lane 2. The two pavers are usually located within 10 m (30 ft) or so of each other. The ultimate performance of the mix at the longitudinal joint depends primarily on the amount of overlap of mix placed by the second paver over the top of the mix placed by the first or front paver. If that amount of overlap is kept to a minimum—less than 25 mm (1 in.)—an excellent longitudinal joint will be constructed. If the amount overlap is too much or too little, however, either a ridge or a gap will be formed at the longitudinal joint.

Paving in echelon just for the purpose of creating a hot longitudinal joint is typically not very beneficial or economical. In most cases, the amount of mix produced at the asphalt plant governs the rate of paving. Most asphalt pavers, operated without any type of material transfer device, can place more than 545 tonnes (600 tons) of mix per hour. Splitting the plant mix production to two pavers instead of one simply increases the cost of placing the mix.

Sometimes it is believed that it is beneficial to cut back the unsupported edge of the first lane to eliminate the under-compacted mix at the edge of the lane and in the slope or wedge at the side of the joint. This is typically unnecessary. First, this is a very costly operation. Second, the cutting has to be done in a straight line so that the joint can be matched with the second pass of the paver. Third, the amount of overlap of mix on the second lane over the vertical face of the cut joint has to be carefully controlled—12 mm (½ in.) or less in overlap distance, similar to the vertical edge with a milled pavement. There does not seem to be any significant data which indicates that the cutting back the joint results in a more durable longitudinal joint on a long-term basis.

SUMMARY

If the longitudinal joint is properly constructed, there generally is no need to apply a tack coat to the unsupported edge of the first lane—Lane 1. Further, if the longitudinal joint is properly constructed, there is no need to cut back the unsupported edge of the first lane before the second lane is placed adjacent to it. If the longitudinal joint is properly constructed, there is no need to use two pavers running in echelon.

Proper construction of the longitudinal joint between pavement lanes consists of four primary steps. First, the unsupported edge of Lane 1 must be compacted by placing the drum of a steel wheel roller about 150 mm (6 in.) over the top of the unsupported edge—compacting air at the edge of the drum. Second, the amount of mix placed over the top of Lane 1 when the mix in Lane 2 is placed should be limited to a distance of 25 mm (1 in.) to 40 mm (1½ in.). Third, the mix placed at the joint when the second lane is constructed should not be moved with a rake but should remain where placed by the edger plate on the paver screed. Last, the mix at the longitudinal joint should be compacted from the hot side of the joint—the Lane 2 side—with the outside tire on the rubber tire roller directly over the joint or the drum of a steel wheel roller extending 150 mm (6 in.) over the top of the joint.

Durable longitudinal joints are a workmanship issue. Proper construction techniques will provide for a long life longitudinal joint without raveling or deterioration.

The following photos (Figures 21 and 22) document projects that have had success with proper joint construction. For each of these projects, the longitudinal joints were constructed as described above.

FIGURE 21 New Jersey Route 80, paved in 1990, with the joints still in good condition.

FIGURE 22 New Jersey Route 80, paved in 1992, with joints still in good condition.

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Incentives–Disincentives for Construction Quality

Fundamentals of Percent Within Limits and Quality Control–Quality Assurance Compaction Specifications

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Most state highway agencies (SHAs) are using statistically based quality control/quality assurance (QC/QA) specifications for hot-mix asphalt (HMA) construction. The basic objective of these statistically based acceptance specifications is to specify and measure quality characteristics (mix properties like asphalt content, gradation and in-place density) that are related to pavement performance, then to pay the contractor for the quality provided. The percent within limits (PWL) statistical method is typically used to quantify quality provided (and assumed pavement performance). The PWL is then used to determine payment through Pay Factors (PF) giving consideration to agency and contractor risk. Pay factors, which include incentives (bonuses) and disincentives (penalties) are assigned for different PWL values (levels of quality/assumed pavement performance) and serve as a basis for payment. Typical QC/QA specifications include composite pay factors with in-place density or air voids normally being the most heavily weighted component. This paper provides a brief overall description of QC/QA specifications with emphasis placed on how variability influences PWL and pay factors. Components of overall variability (controllable and uncontrollable) are defined. The potential influence of sampling location and method, test method, and materials/construction variability, and use of QC or QC and QA data in PWL determinations are discussed. The importance of adequately considering all these sources of variability in establishing rational specification limits or tolerances is highlighted. The paper is concluded with actual examples of how changes in sources of variability impact PWL and payment for existing SHA specifications.

INTRODUCTION

Many SHAs use statistically based specifications for HMA construction (*1*). The specifications are commonly referred to as QC/QA specifications. The basic objective of these statistically based acceptance specifications is to specify and measure quality characteristics (mix properties like asphalt content, gradation, and in-place density) that are related to pavement performance, then to pay the contractor for the quality provided. Acceptance sampling–testing and the PWL statistical method are used to quantify quality provided (and assumed pavement performance). The contractor is given the responsibility for process and quality control sampling and testing which is verified with limited quality assurance testing by the specifying agency. This essentially places the contractor in responsible charge of its earnings while limiting resources needed by the specifying agency to manage the work.

The PWL method is based on simple statistical principles and quantifies the amount of in-place HMA that is within given specification limits (target value plus and/or minus tolerances) based on limited sampling (*2*). Specification limits are limits placed on quality characteristics

and used to define suitable and defective material quality from a pavement performance perspective. The PWL is then used to determine payment through pay factors. Pay factors, which include incentives (bonuses) and disincentives (penalties) are assigned for different PWL values and serve as a basis for payment. In establishing acceptance sampling plans and pay factors an effort is made to satisfy the specifying agencies quality level goals and at the same time minimize the risks to both the specifying agency and the contractor of wrongfully accepting or rejecting in-place HMA.

Most QC/QA specifications include composite pay factors (*3*). Composite pay factors are used when a specification includes multiple quality characteristics. A weight is assigned to each quality characteristic and a pay factor is computed for each quality characteristic. Then a weighted average pay factor, termed a "composite pay factor" is calculated to determine payment. The composite pay factor is typically multiplied by the contract price for HMA to determine final payment.

Statistically based QC/QA specifications offer several advantages over method specifications, but must be developed and implemented with adequate consideration of the underlying statistical principles incorporated in them, as well as how controllable and uncontrollable variability can ultimately affect payment. In-place density or air voids is normally a heavily, if not the most heavily weighted pay factor component in typical QC/QA specifications. Therefore it is particularly important that specifying agencies and contractors understand the influence of these items on compaction pay factors. Unfortunately there is a lack of consistency among QC/QA specifications currently used across the country, particularly in acceptance sampling (*1*). Therefore two specifications can have the same specification limits, use the PWL statistical model and the same pay factor tables, but provide different payment due to differences in sampling and testing frequency, the number of tests per samples, sampling techniques, and lot and sublot definitions.

This paper provides a brief overall description of QC/QA specifications with emphasis placed on how variability influences PWL and compaction pay factors. Components of overall variability (controllable and uncontrollable) are defined. The potential influence of sampling location and method, test method and materials/construction variability, and use of QC or QC/QA data in PWL determinations are discussed. The importance of adequately considering all these sources of variability in establishing rational specification limits or tolerances is highlighted. The paper is concluded with actual examples of how changes in sources of variability or specification limits impact PWL and payment for existing SHA specifications.

TYPICAL QC/QA SPECIFICATIONS

Figure 1 is a macro view of common components (from an implementation perspective) of a typical statistically based QC/QA specification. The components include: acceptance sampling, QC and QA, comparison testing (f- and t-testing), quality-level analysis (PWL determination), and pay factor determination. Several details are obviously excluded from the figure.

QC is normally the responsibility of the contractor (or the contractors representative) and QC sampling and testing is conducted at a relatively high frequency. The contractor may also conduct process control (PC) sampling and testing to help manage a production process. The primary difference in QC and PC testing is that QC testing is ultimately used for payment and operational control. PC testing is only used to determine whether a process (i.e., HMA

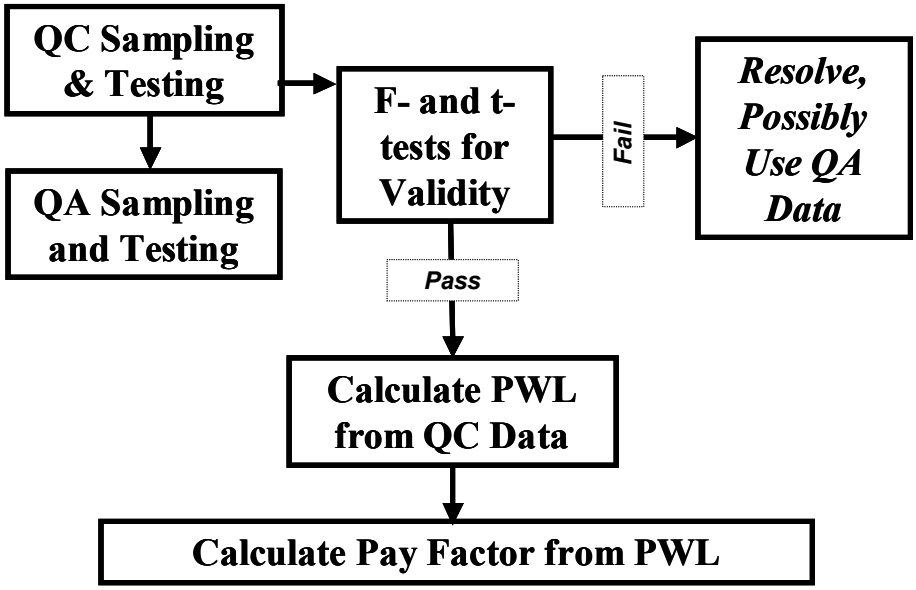


FIGURE 1 Macro view of typical statistically based QC/QA acceptance specification.

production) should be continued or shutdown (go versus no-go testing) in QC/QA specifications. QA testing is normally conducted by the specifying agency or its representative at a significantly lower frequency than QC testing. An example of the ratio of QC to QA testing might be 10:1.

Statistical tests are then conducted to assure that the QC and QA data come from the same population. Common tests are the f-test for equal variance and the t-test for equal means. If the data are determined to have come from the same population, the QC data is commonly used in the quality level analysis to determine PWLs. If unequal variance and/or means are observed, then the QA data maybe used or the QC and QA data may be pooled, or independent assurance testing may be conducted and used. Unfortunately, there is inconsistency in these processes among specifying agencies.

PWL calculation and pay factor determination are fairly simple processes. There is some consistency among specifying agencies in the mechanics of the PWL calculation, but little consistency among specifying agencies when it comes to pay factors and weight of quality characteristics in composite pay factors.

In reality, a typical statistically based QC/QA specification is more complicated than these four items suggest. There are actually seven key components of a statistically based acceptance plan (*4*). Mahoney and Muench provide a concise summary and review of the seven items in "Quantification and Evaluation of WSDOT's Hot-Mix Asphalt Concrete Statistical Acceptance Specification" (*5*).

Acceptance Plans

The seven components of a statistically based acceptance plan are

1. Acceptance sampling type,
2. Quality characteristics,

3. Specification limits,
4. Statistical model,
5. Quality-level goals,
6. Risk assessment, and
7. Pay factors.

A description of each follows.

Acceptance Sampling Type

The two basic acceptance sampling types are attribute and variable sampling. With attribute sampling each sample is inspected for a given attribute or property (quality characteristic) and the only information collected is whether or not the sample passes or fails relative to a standard. With variable sampling the quality characteristic(s) (asphalt content, in-place density, etc.) of each sample is measured and the value is retained. QC and QA tests are performed at specified frequencies to obtain these measurements. Measured quality characteristics are continuous variables and assumed to be normally distributed (*6*). Most SHAs use the variable acceptance sampling method making it possible to quantify mean and variability of each quality characteristic.

A lot defines a quantity of material. Lot sizes are usually specified by tonnage (i.e., 2,000 tons per lot) or frequency per production period (i.e., one per day). Lots are normally divided into sublots (i.e., three to five sublots per lot) for sampling and testing purposes. Sampling within a lot is normally done on a random basis to satisfy basic statistical requirements. An array of HMA sampling methods and locations are used throughout the country. Measurement of quality characteristics on the random samples (materials testing) then takes place. Selection of tests to be performed, test standards to be employed, options with the test standards to be employed, and replicate tests to be performed to report a measurement value all have to be selected by the specification developer. Engineering judgment and consideration of practical logistics normally are used to make these selections. Ultimately, the selections impact pay factors as will later be discussed.

Quality Characteristics

Quality characteristics are material properties that an acceptance plan requires measurement of to determine quality. HMA mixture properties (asphalt content, gradation, and volumetrics), in-place density or air voids, and smoothness are the quality characteristics commonly found in HMA QC/QA specifications. It is important that quality characteristics are independent (not correlated) to prevent biased pay factors.

Quality characteristics are measured at the time of construction, and are assumed to be related to long-term pavement performance. Therefore, in the ideal situation excellent relationships between quality characteristics and pavement performance would exist. Additionally the sensitivity of the relationships to variability in the quality characteristics would exist. Unfortunately, such relationships do not exist for the array of HMA materials and mixtures used in the unique environmental and loading conditions present throughout the United States. Some relationships were developed as part of the WesTrack project as illustrated in Figure 2 (*7*). In most cases engineering judgment is used to select quality characteristics.

Specification Limits

Specification limits are limits placed on quality characteristics used to define acceptable and unacceptable material quality. Specification limits are normally based on engineering judgment and in some cases statistical analysis. Normally a target value and tolerances are assigned for each quality characteristic. The target value plus and minus the tolerance define the specification limits as illustrated in Figure 3. In some cases a single (either upper or lower) specification limit is used, as is the case for in-place density in many HMA QC/QA specifications. Specification tolerances must account for variability inherent in HMA production and testing. Hughes indicates that there are four types of variability that must be considered in establishing specification limits: materials, sampling, testing, and manufacturing/construction variability as illustrated in Equation 1 (*8*).

$$\sigma_T^2 = \sigma_m^2 + \sigma_s^2 + \sigma_t^2 + \sigma_{m/c}^2 \quad (1)$$

where

σ_T^2 = total variance,
σ_m^2 = materials variance,
σ_s^2 = sampling variance,
σ_t^2 = test method variance, and
$\sigma_{m/c}^2$ = manufacturing and construction variance.

Hughes further indicates that the materials producer and/or contractor can only control manufacturing/construction variability. So specification tolerances must be large enough to allow for typical materials, sampling, and testing variability plus some level of manufacturing/ construction variability. If they are not, then the contractor will be penalized for variability that is beyond his control.

Statistical Model

A quality-level analysis (QLA) is a statistical procedure that provides a method for estimating the percent of each lot of material or product that is expected to be within specification limits. To perform a QLA a statistical model is need. An acceptance plan statistical model is used to relate random sample test results to the distribution of quality characteristic for a given sample set (lot). The random sample test results provide the average sample measurement (x = mean or average) and variability of that measurement (σ=standard deviation). The distribution of the measured quality characteristic is estimated from x and σ in the form of a normal probability distribution as illustrated in Figure 4. The definition of *Quality* is simply the portion of the overall quality characteristic distribution that lies within specification limits. Commonly used quality terminology include percent within limits (PWL) and/or percent defective (PD). These terms are

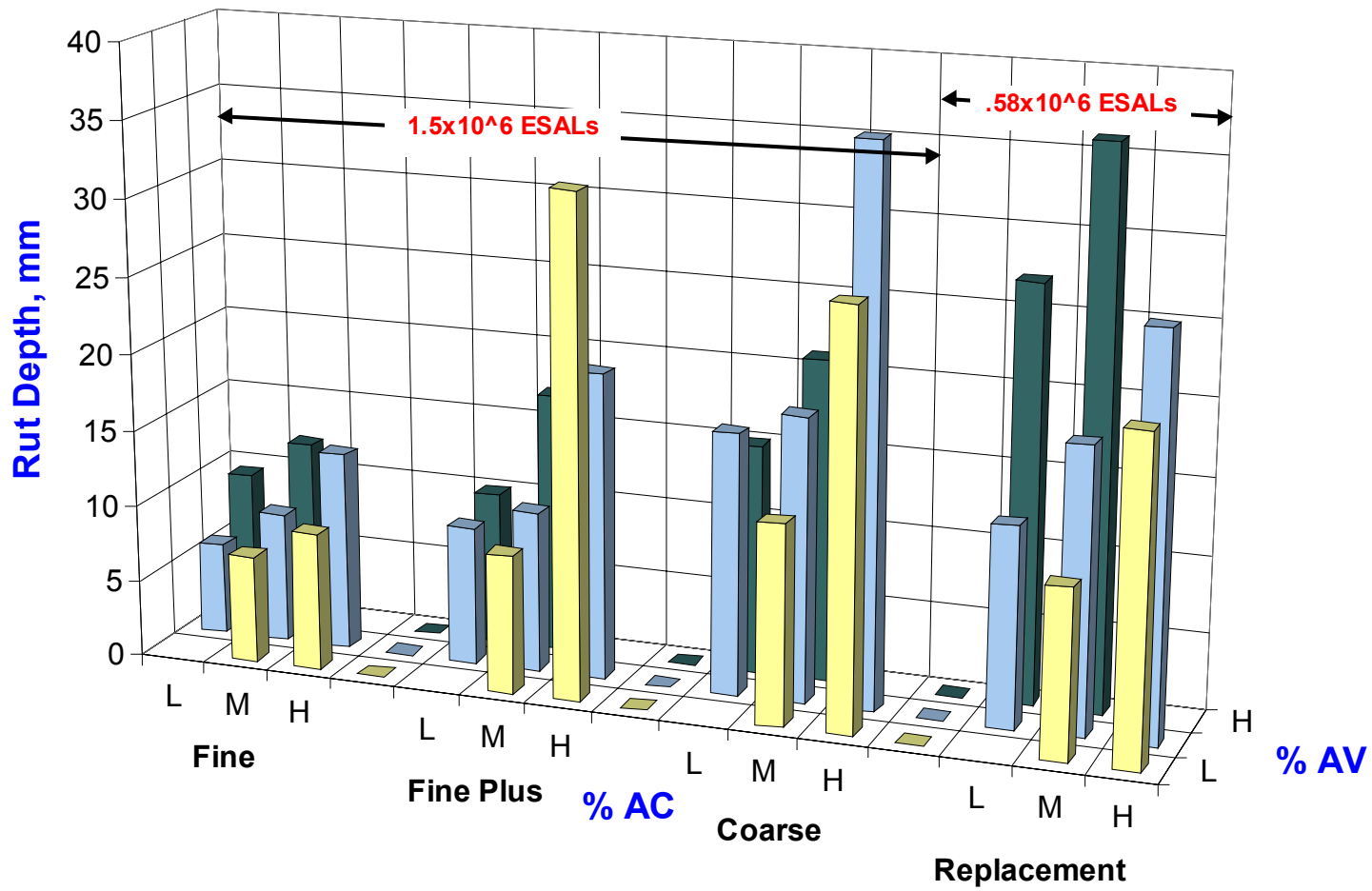


FIGURE 2 Influence of variability in asphalt content and air voids quality characteristics on rutting performance.

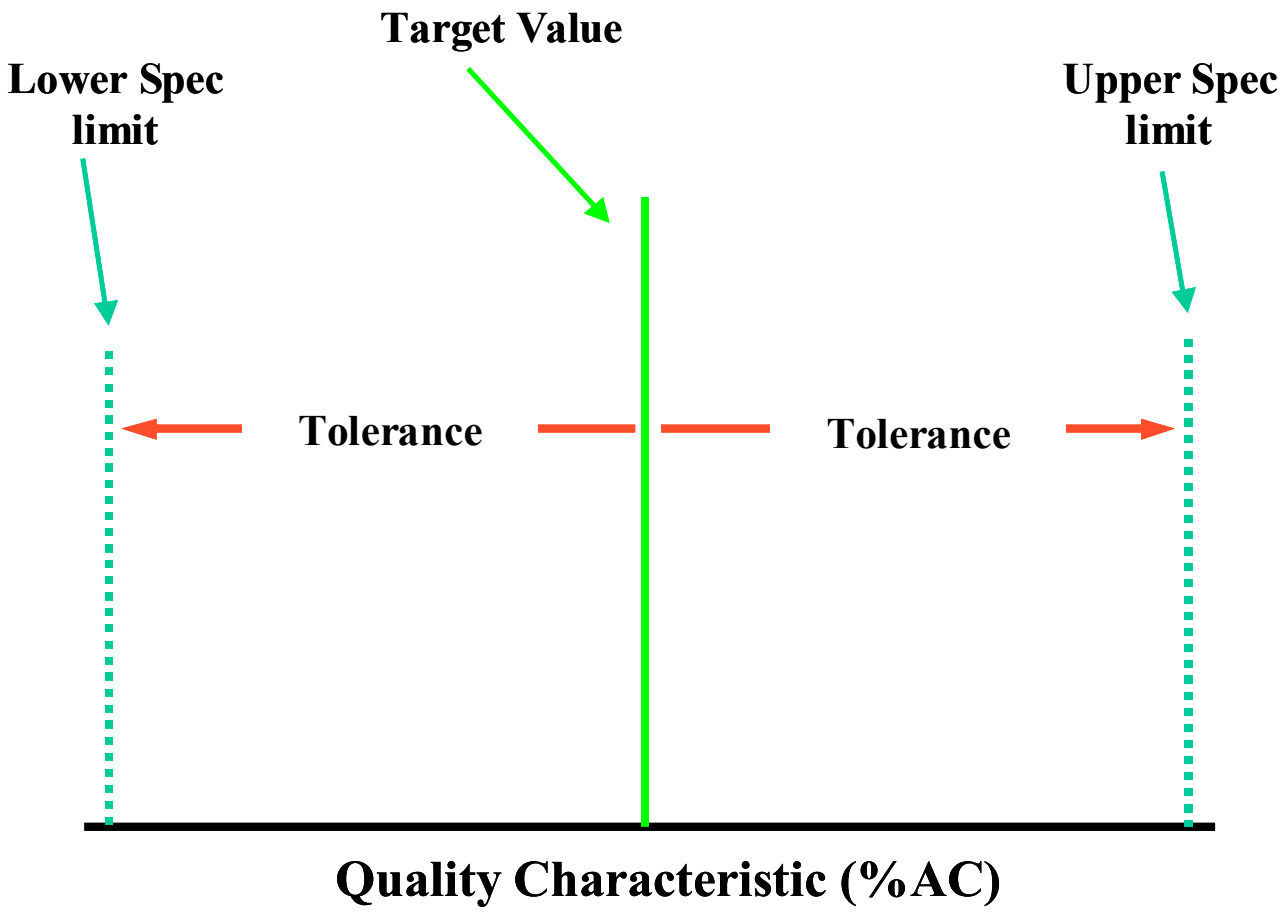
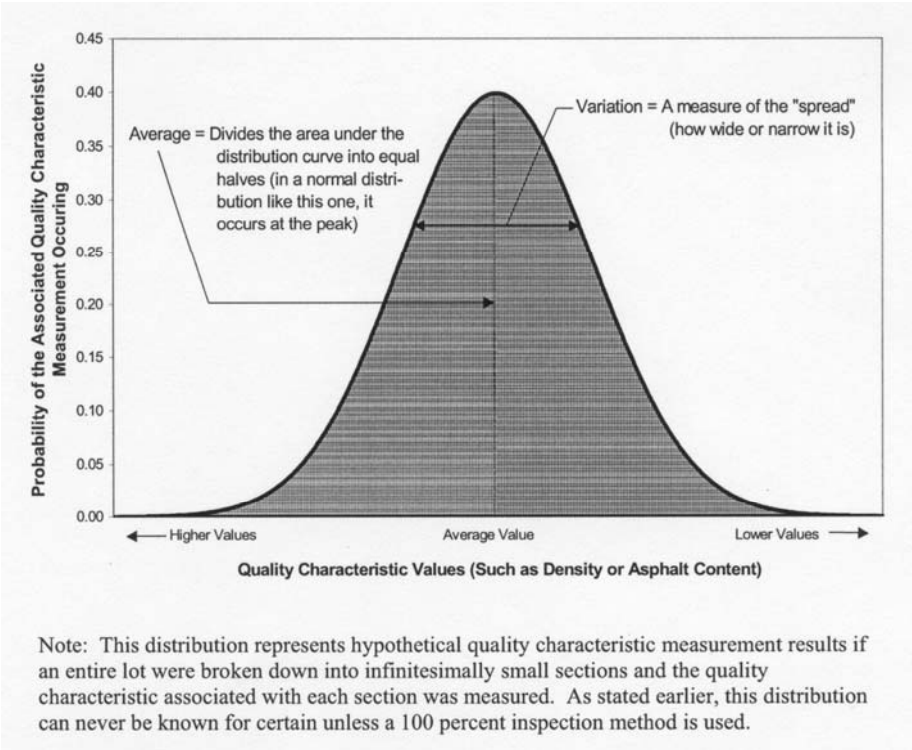


FIGURE 3 Illustration of quality characteristic specification limits based on a target value and tolerances.



Note: This distribution represents hypothetical quality characteristic measurement results if an entire lot were broken down into infinitesimally small sections and the quality characteristic associated with each section was measured. As stated earlier, this distribution can never be known for certain unless a 100 percent inspection method is used.

FIGURE 4 Quality characteristic distribution [after Muench and Mahoney (*5*)].

illustrated in Figure 5. PWL is equal to 100 PD. Details of PWL calculations can be found in *Quality Assurance Guide Specifications* (*2*).

Quality Level Goals

There are two quality level goals in a statistically based QC/QA specification; acceptable quality limit (AQL) and rejectable quality limit (RQL). AQL is minimum level of quality (PWL) at which the material is considered fully acceptable. RQL is the maximum level of quality (PWL) at which the material is considered unacceptable (rejectable). Selection of appropriate AQL and RQL levels is not consistent among all specifying agencies. In fact, the selection is subjective with typical AQL being set at PWLs of 90 to 95 and RQL levels between 30 and 75. One would initially think that the AQL should be set at a PWL of 100. The selection commonly is not based on finding at the AASHO Road Test as noted in the following quote from Quality Assurance Software for the Personal Computer FHWA Demonstration Project 89, Quality Management (*9*):

> Although the construction measures observed at the AASHO Road Test did have considerable variability, it was equally clear that many of the pavements and structures built under these conditions performed very satisfactorily. What had not been realized previously is that the existences of a relatively small percentage of tests falling outside specification limits was normal and not necessarily detrimental to performance. This led to the definition of the acceptable quality level (AQL)…Typical values used in the highway field are…percent within limits values of PWL = 95 or PWL = 90.

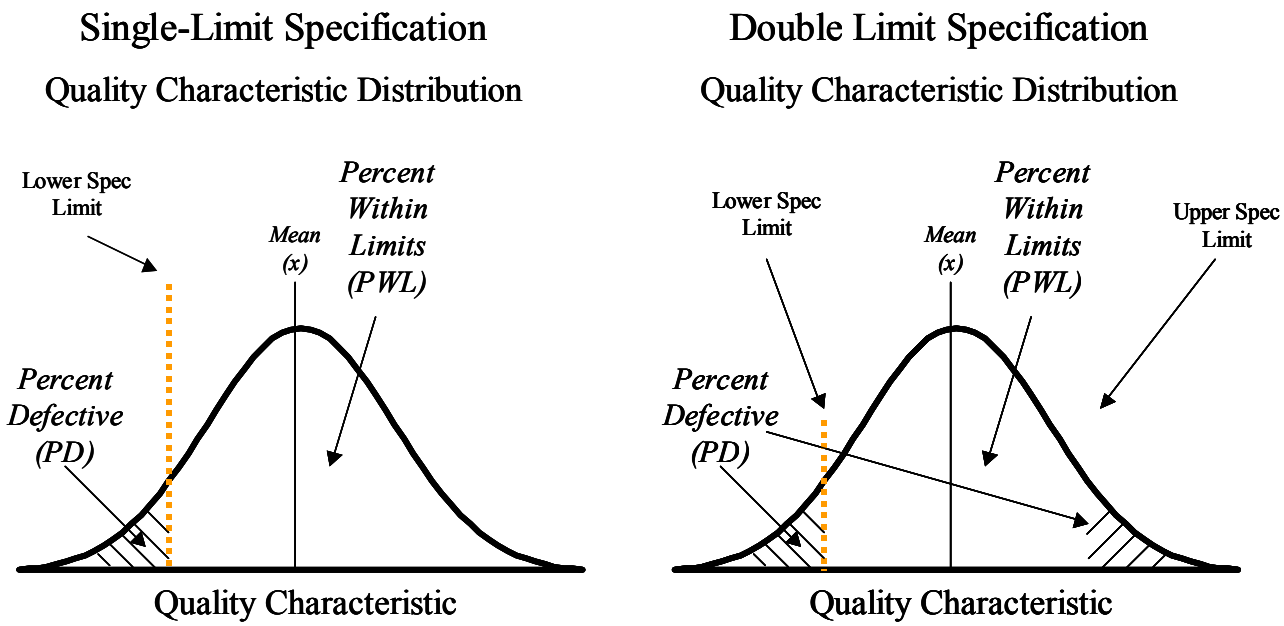


FIGURE 5 Quality characteristic distributions with quality defined by illustrated PWL and PD.

Risk

Risk is inherent is a statistically based QC/QA acceptance specifications because a small number of random samples (rather than the entire material) are used to estimate the quality (PWL) of a large amount of material. The risk exists because there is a possibility that the few random samples used will not be truly representative of the entire material provided. If the samples are not representative, than an incorrect quality estimate will erroneously be made (incorrect PWL). There are two types of risk: seller's risk (α) which the contractor assumes and buyer's risk (β) which the owner (specifying agency) assumes (*6*). The seller' risk (α) is the risk that acceptable quality material will be rejected when it is in conformance with the specifications. Buyer's risk (β) is the risk that unacceptable quality material will be accepted when it is not in conformance with the specification. Risk is illustrated in Figure 6.

Risk can be calculated and should be appropriately minimized and balanced between the seller and buyer. Criticality (implications of a failure) of a quality characteristic must also be considered. The AASHTO recommended levels of risk are summarized in Table 1 (*10*). For a fixed number of samples if seller's risk is reduced, then buyer's risk is increased and vice versa.

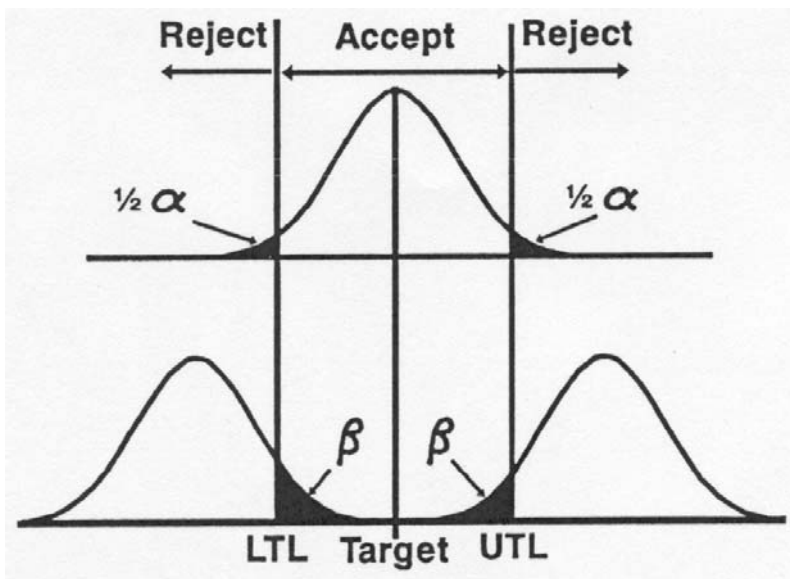


FIGURE 6 Illustration of seller's (α) and buyer's (β) risk (*10*).

TABLE 1 AASHTO-Suggested Risk for QC/QA Specifications

Criticality of Quality Characteristic	Seller's Risk (α)	Buyer's Risk (β)
Critical	5.0%	0.5%
Major	1.0%	5.0%
Minor	0.5%	10.0%
Contractual	0.1%	20.0%

The only way to simultaneously lower risk is to increase the number of samples. Thus specification developers are forced with balancing sample size, sampling and testing costs, and practical logistics (sampling and testing turn around time and management).

Operating characteristic (OC) curves can be used to recognize and control seller and buyer risks at suitable levels. OC curves show the relationship between the quality of a lot and the probability of acceptance of that lot for a given sample size. Figures 7 and 8 are OC curves for sample sizes of 5 and 50, respectively. Note the reduction in risk as sample size is increased. Ultimately a specifying agency must select what they deem acceptable levels of risk for integration into their specifications.

Pay Factors

Quality is actually related to payment via pay factors. A pay factor is a multiplier applied to a contract unit price that is a function of PWL. Many QC/QA specifications incorporate incentives (bonuses) for quality greater than AQL (PWL = 90 to 95) and disincentives (penalties) for quality less than AQL and greater than RQL. A pay factor of 1.0 is normally assigned to the PWL equal to the AQL. Just as AQL and RQL are variable among specifying agencies, so are pay factors. Mahoney et al., reports pay factors ranging from a high of 1.00 to 1.12 to a low of 0.50 to 0.75 (5). Because HMA performance is dependent on multiple quality characteristics most QC/QA specifications incorporate composite pay factors. A composite pay factor is typically calculated by first determining the pay factor for each individual quality characteristic, then by calculating a weighted average for all of the characteristics in accordance with Equation 2.

$$\text{Composite Pay Factor} = \frac{\sum\left(PF_n \times Wt_n\right)}{\sum Wt_n} \times 100 \qquad (2)$$

where

PF_n = individual pay factors, and
Wt_n = weight assigned to each individual pay factor.

It is not uncommon for QC/QA specifications to incorporate separate composite pay factors for loose HMA (asphalt content, gradation, volumetrics) and in-place HMA (in-place density and smoothness). Like many other parameters in statistically based QC/QA acceptance specifications, minimum and maximum pay factors and weights (for composite pay factors) are assigned based on engineering judgement. The intent is to either pay the contractor a bonus for the better than acceptable level of quality because the road with last longer, or to penalize the contractor for less than acceptable level of quality because the life of the road is compromised. Unfortunately, robust relationships between quality and life cycle costs do not exist. Thus the highly variable range of pay factors and weights reported in (*1,3,5*).

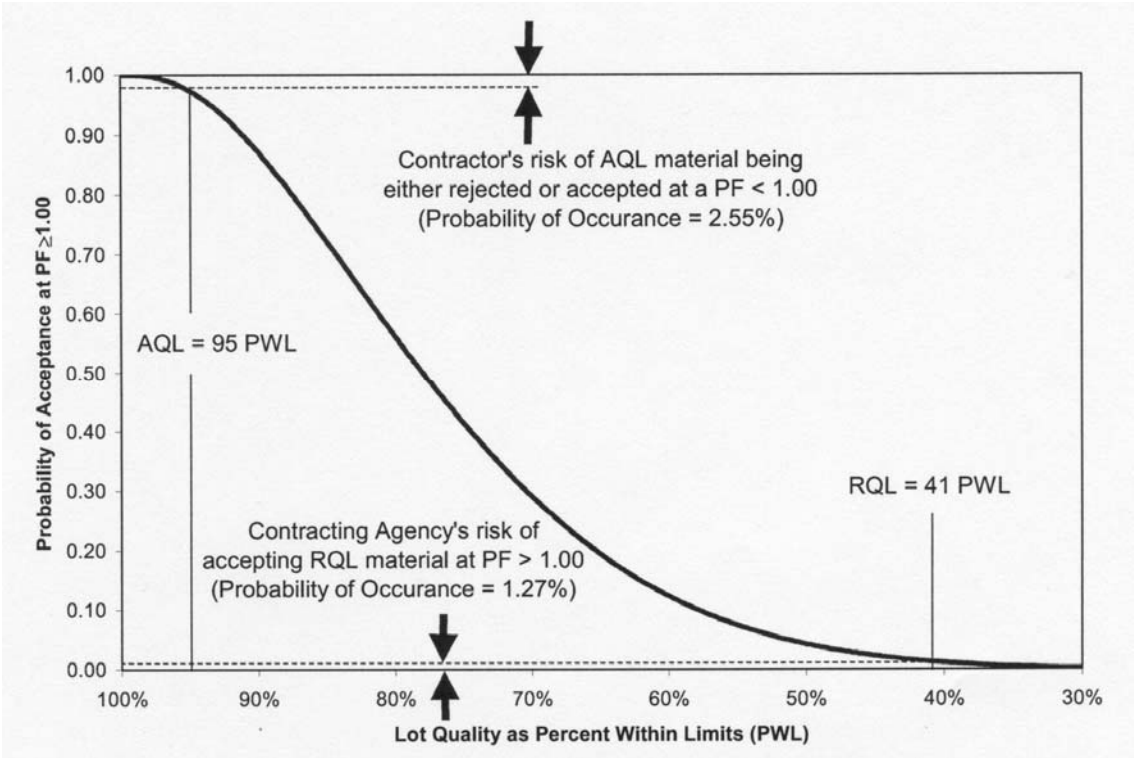


FIGURE 7 Typical OC curve showing risk for *n* = 5 [after Muench and Mahoney (5)].

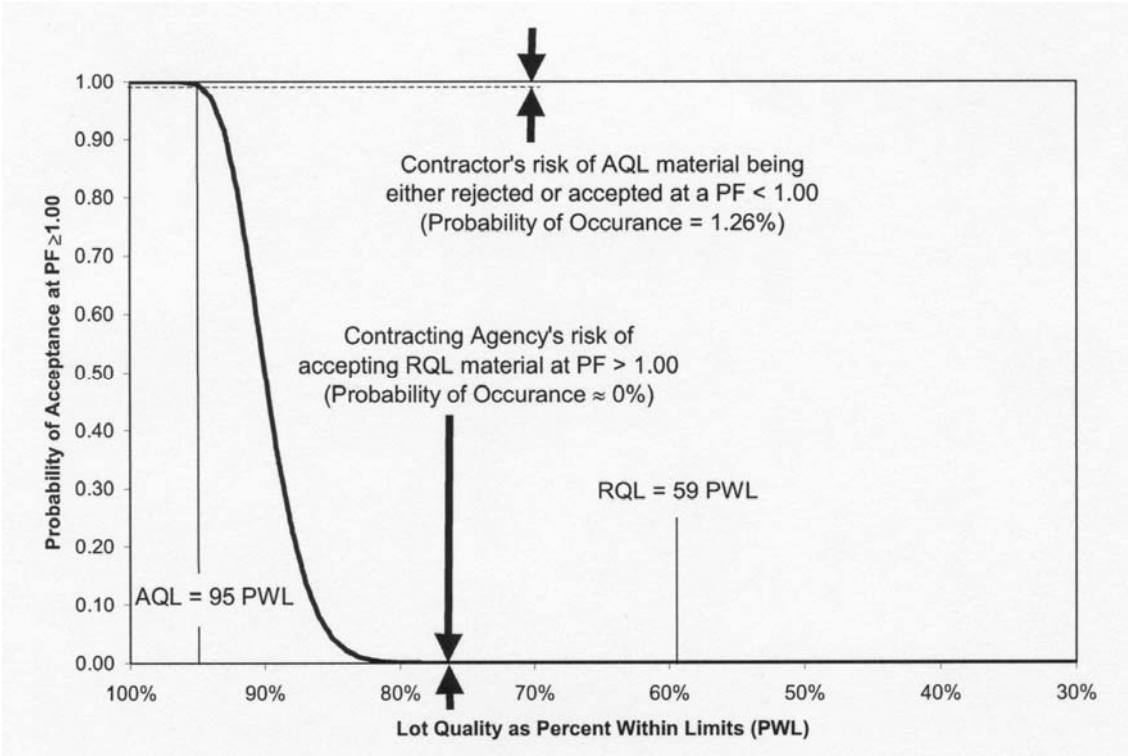


FIGURE 8 OC curve showing risk for *n* = 50 [after Muench and Mahoney (5)].

Observations

A common theme in the description of statistically based QC/QA specifications provided above is that there is a lack of consistency among QC/QA specifications and that many specification parameters are selected based on engineering judgment or historical practice, rather than on a rational basis. The following sections illustrate the impact of some differences among specifications and how sensitive the specifications are to changes in variability in quality characteristics and specification limits.

VARIABILITY AND ESTABLISHMENT OF SPECIFICATION LIMITS

As previously discussed, Hughes indicates there are four sources of variability (see Equation 1): materials, sampling, testing, and manufacturing/construction and that the material producer/contractor only has the ability to control the manufacturing/construction component (*7*). The authors suggest that in some cases the material producer/contractor has the ability to control materials variability, particularly when it comes to quarry/pit mining practices. Some might argue that this is controlling manufacturing variability. Others have also suggested three components of total variability as shown in Equation 3 (*11*).

$$\sigma_T^2 = \sigma_s^2 + \sigma_t^2 + \sigma_{m/c}^2 \tag{3}$$

where

σ_T^2 = total variance,
σ_s^2 = sampling variance,
σ_t^2 = test method variance, and
$\sigma_{m/c}^2$ = materials/manufacturing and construction variance.

Because of the significant effort it takes to clearly define the distribution of the total variance among the components, little rigorous research has been done to do so. What has been done and reported in the literature is limited to a couple of quality characteristics (*12*). Estimates of sampling, testing, and materials/manufacturing and construction variability of 10% to 30%, 30% to 50% and 30% to 40%, respectively have been reported. Figure 9 shows a distribution among the components for asphalt content. Based on this information one could suggest that about 40% to 80% of a specification tolerance should be for sampling and testing variance and the remainder should be for materials/manufacturing/construction variance.

Stroup-Gardner et al., suggests establishing specification limits based on known test method variability (*11, 13*). If the percent distribution of each component (σ_s, σ_t, and $\sigma_{m/c}$) are know then σ_T can be estimated by taking the square root of Equation 4 solved for σ^2_T as a function on any single component.

$$\sigma_T^2 = P_s\sigma_s^2 + P_t\sigma_t^2 + P_{m/c}\sigma_{m/c}^2 \tag{4}$$

where

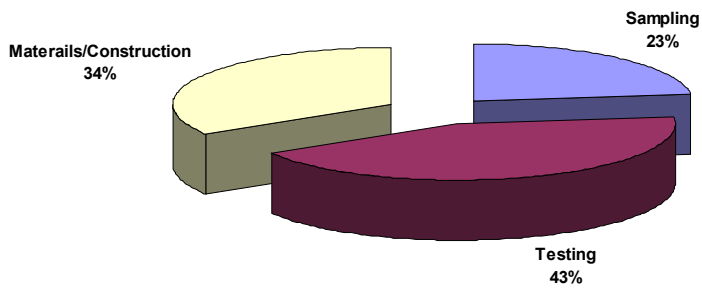


FIGURE 9 Example distribution of total variability among components.

P_s = percent contribution of sampling variance to total variance,
P_t = percent contribution of testing variance to total variance,
P_t = percent contribution of materials/manufacturing/construction variance to total variance,
σ_T^2 = total variance,
σ_s^2 = sampling variance,
σ_t^2 = test method variance, and
$\sigma_{m/c}^2$ = materials/manufacturing and construction variance.

She reported the percent contribution of total variance due to testing as 10% to 30% and suggested that specification limits be set at a target value plus and minus 3 times σ_T.

At this point it is worth noting that with some QC/QA specifications just QC data is used to calculate PWLs, where in other QC and QA data are pooled for the calculation of PWLs. Others only use QA data. It is well established that within laboratory testing variability is less than between laboratory testing variability. Therefore when QC and QA data are pooled larger specification tolerance would be required to obtain equivalent PWLs, if that were an objective. Test method precision and bias statements are a good source of σ_t data that can be helpful when establishing specification limits.

Unfortunately good estimates of the components of variance for all of the quality characteristics included in QC/QA specifications are not available. Therefore specification limits are normally selected simply using engineering judgment even though a rational technique is available.

REDUCING VARIABILITY AND SPECIFICATION LIMITS

It is very reasonable for specifying agencies to reduce specification tolerances or limits over time and with experience (to a point). If changes can be made in specification protocol and/or test methods to reduce σ_s or σ_t then specification limits could immediately be reduced to reflect those reductions as illustrated in Figure 10. An example of this would be to change sampling method and/or location for gradations. Williams et al., recently reported that using different HMA mat

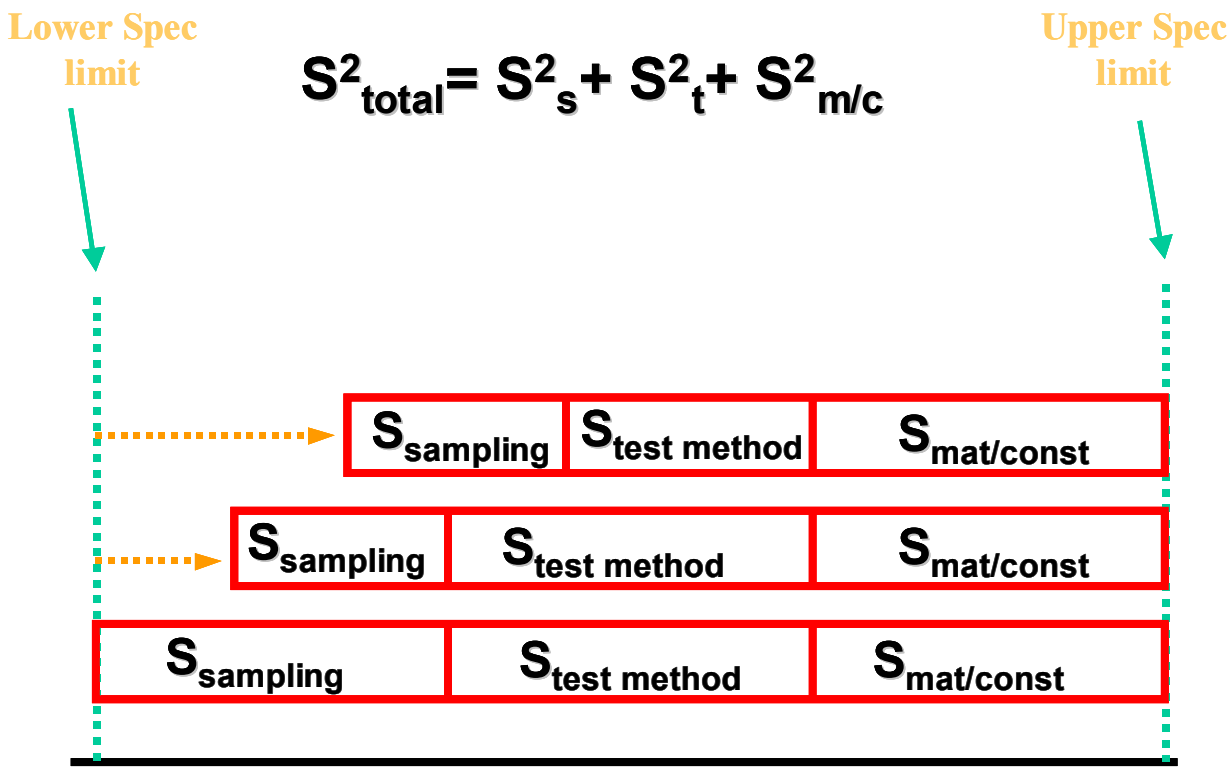


FIGURE 10 Conceptual illustration of reduction in specification tolerances as sampling and/or testing variability are reduced.

sampling techniques (shovel, plant and shovel, ring and plate) resulted in observably different amounts of variability in HMA mixture test results (*14*). He reported the ring and plate method to generate the least amount of sampling variability. When considering pay factors he indicated that coarse graded mixtures were more prone to pay adjustments than fine graded mixtures with the likely cause being segregation in the coarse graded mixtures. This work actual suggests that different specification limits may be appropriate for different sampling method and mixture types.

Figure 11 illustrates the influence of sampling location on gradation for a coarse graded 19-mm Superpave mixture (*7*). Forty-five samples were collected off a coldfeed collector belt, 45 were obtained from loose HMA sample from trucks and 46 were 150-mm diameter by 75-mm thick cores taken from the finished mat. Asphalt content determinations were made on the loose truck and cores samples using the ignition oven and gradations were determined on the remaining aggregates. Note that differences in standard deviations among the sampling locations were up to 2.7% (see #4 sieve). It should be noted that the reduction in variability from the plant and truck samples to the cores is likely due to the fact that a material transfer device was used on the project. This example clearly illustrates the fact that sampling location must be considered when selecting specification limits. To further illustrate this fact, the data in Figure 11 were input in the Maryland DOT HMA QC/QA specification to see the influence of gradation sampling location on composite pay factor. The outcome is summarized in Table 2 and the difference in PWL is 1.5%, which is significant.

In an effort to achieve bonuses materials producers and contractors will gain experience and improve facilities, equipment, technical expertise (personnel), and manufacturing/production processes. With these improvements quality should improve primarily through the reduction of $\sigma_{m/c}$ to a point beyond which it cannot be reduced any further. The logical specifying agency response to the incremental improvement would be to reduce specification limits by the amount of the improvement, which is logical. However, it should only be done once industry has had an opportunity earn a fair return on the investment made to achieve the incremental improvement. It is important that when investments are made, bonuses can be earned for a long enough period to recuperate major capital investments in items such as facilities and equipment that may be amortized over a 5-year period. If industry does not have the opportunity to earn a fair return on investment over time through earned bonuses, then construction costs will fictitiously be inflated to provide the return on investment.

Things that can be done to immediately reduce variability (σ_s and/or σ_t) in quality characteristic measurements include:

- Require all laboratories conducting QC and QA testing to be AASHTO accredited;
- Require all technicians working of QC/QA projects to be certified, preferably by a national (NICET), regional (WAQTC), or state agency;
- Selecting sampling locations and sampling/splitting methods that result in the lowest amount of variability;
- Select test methods that result in the lowest amount of variability;
- Eliminate options within test methods to reduce between laboratory variability; and
- Use only QC rather than pooled QC and QA data.

The first two items would create additional costs to the materials producer/contractor, though nominal in the big picture of construction costs. Selecting sampling locations and

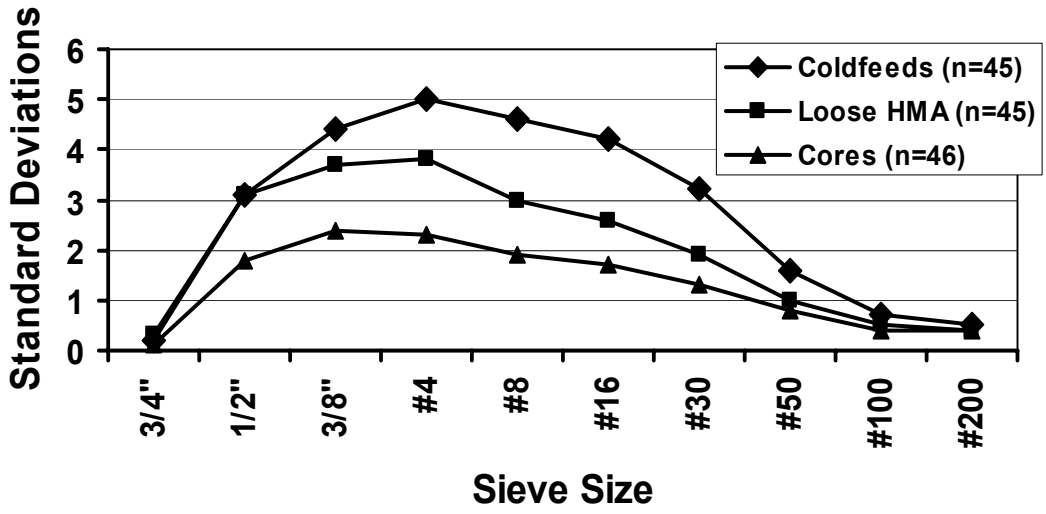


FIGURE 11 Effect of sampling location on gradation variability.

TABLE 2 Example Illustration of the Influence of Sampling Location on PWL

Sieve Size	Sampling Location and Standard Deviation	
	Truck	Cores
#4	4.9	2.2
#8	3.1	1.8
#200	0.6	0.5
PWL	98.4	99.9

sampling/splitting methods that result in the lowest amount of variability may result in cost to both the specifying agency and the materials producer/contractor. The last three items would require research on the specifying agencies part and revision of documents (specifications and test standards) therefore increasing agency costs.

It is very important that when a specifying agency decides to change specification tolerances because it appears as though a reduction in $\sigma_{m/c}$ has occurred that the following be considered:

- Determine if a real reduction has occurred, and if so has it only occurred in a small population of the material producers/contractors.
- Were significant capital costs made by industry to achieve the reduction, and if so, has the investment been recovered? And
- What impact will the change have on pay factors for the average contractor based on a statistical analysis of actual QC/QA data collected on projects constructed using the current specification?

Unfortunately arbitrary changes in specification limits can only result in two potential negative outcomes:

- Construction costs will increase without an improvement in quality which in unfair to the taxpaying community; and/or
- Pressure will unfairly be placed on the ethics of testing laboratory personnel.
- An example of the impact of an arbitrarily proposed change in specification limits is presented in the following section

Regardless of the reason for changing specification tolerances or limits it should only be done if the costs (both agency and materials producer/contractor) associated with the changes will be recuperated by those incurring them.

EXAMPLE OF ARBITRARY SPECIFICATION LIMIT CHANGE

A large state DOT in the western United States planned a specification limit change in its statistically based QC/QA specification under internal political pressure due to a perception that contractors were regularly achieving undue bonus pay factors. The planned change was in asphalt content specification tolerances. At the time the asphalt content specification was the job mix formula target value plus and minus 0.5%. The planned change was to arbitrarily reduce the tolerances to plus and minus 0.3% with no other changes in the specification.

Fortunately, an agency/industry forum existed in the state through which recommendations for resolution to issues are generated and bought to the state DOT for consideration. The planned change was immediately brought to the forum that has technical representatives of both parties. A task force was formed and a rational plan was developed to assess the impact of the planned change. The plan very simply consisted of collecting and analyzing asphalt content data from projects constructed under the QC/QA specification over about a five year period. The analysis consisted of

- Determining typical deviations in mean asphalt content from the target value and typical standard deviations;
- Determining what asphalt content pay factor the average contractor received under the current specification limits;
- Generating a series of relationships between asphalt content pay factors and deviations in mean asphalt content from the target value and standard deviation combinations over a range of asphalt content specification limits; and
- Reporting what the impact of the proposed change would have on the average contractor performing the work (average pay factor and percentage of contractors that would receive bonuses)

A statistical analysis of asphalt content data from over 75 projects with each project having over 10,000 tons of HMA placed was jointly conducted by an agency statistician and an industry technical expert. The data were collected from a database the agency had populated over time and directly from agency resident engineers and contractor quality control managers for project that had recently been completed and were not yet in the agency database. Prior to

performing the analysis several aspects of the data in the database were closely reviewed to assure that bias was not present. Examples of aspects considered included:

- Were all districts in the state well represented?
- Were all contractor sizes (large and small) well represented?
- Did all contractors included do about the same amount of QC/QA projects through the state
- Were the projects well distributed among the 5-year period?
- Did the projects incorporate a distribution of HMA tonnage (not just small or large jobs)

The data revealed that the typical deviation in mean asphalt content from the target value (offset) was just over 0.10% and the typical standard deviation was 0.20. Figures 12, 13, and 14 show the relationship between asphalt content pay factor and different offset/standard deviation combinations for specification limits of ±0.5%, ±0.4%, and ±0.3%, respectively. The analysis revealed following:

- The average contractor was receiving a pay factor of almost 1.04 under the current specification;
- By reducing the specification limits to ±0.3% the average contractor would receive a pay factor of 0.94;
- Overall there would be a reduction in average asphalt content pay factor of approximately 10%;
- It would be impossible to earn a maximum pay factor (1.05) even if the asphalt content were exactly on target (zero offset) and the only thing contributing to standard deviation were test method variability.

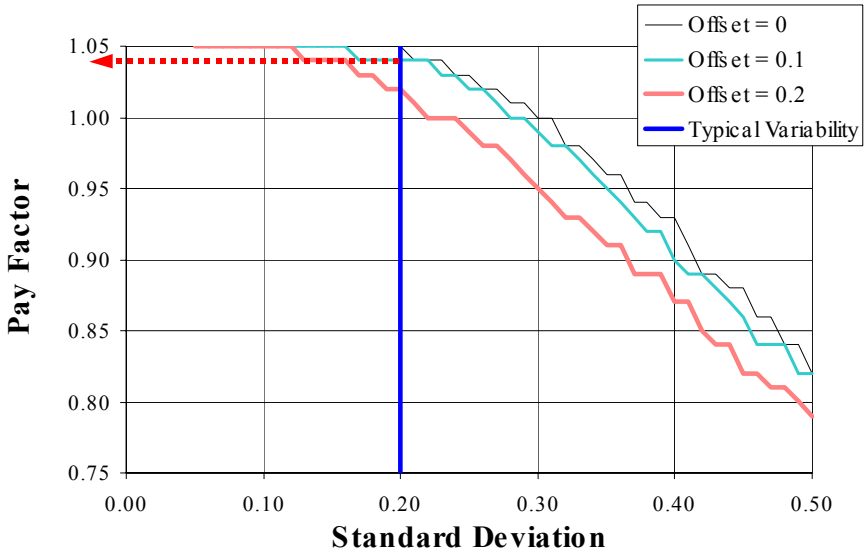


FIGURE 12 Pay factor and typical offset/standard deviation combination relationships with ±0.5% specification limits.

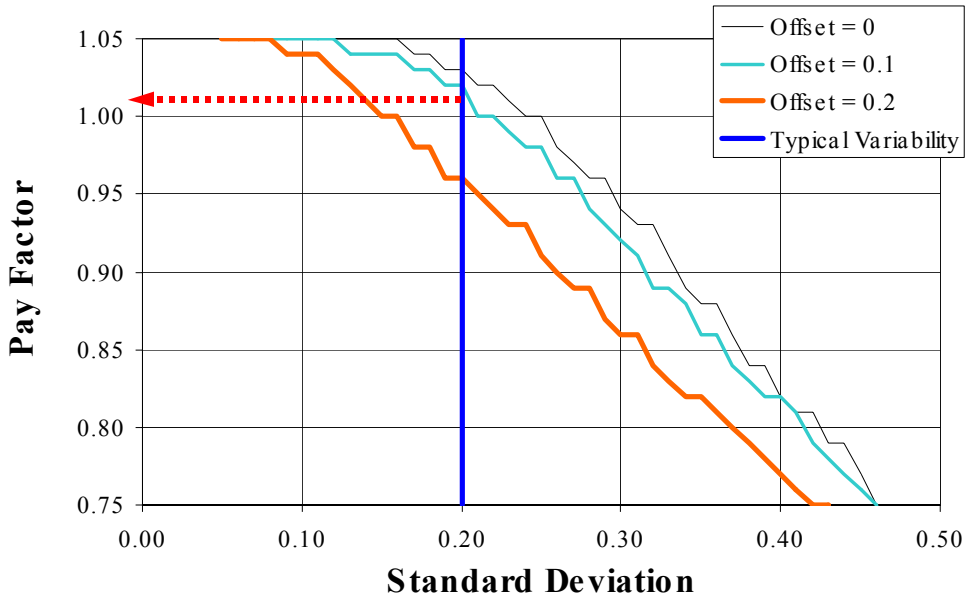


FIGURE 13 Pay factor and typical offset/standard deviation combination relationships with ±0.4% specification limits.

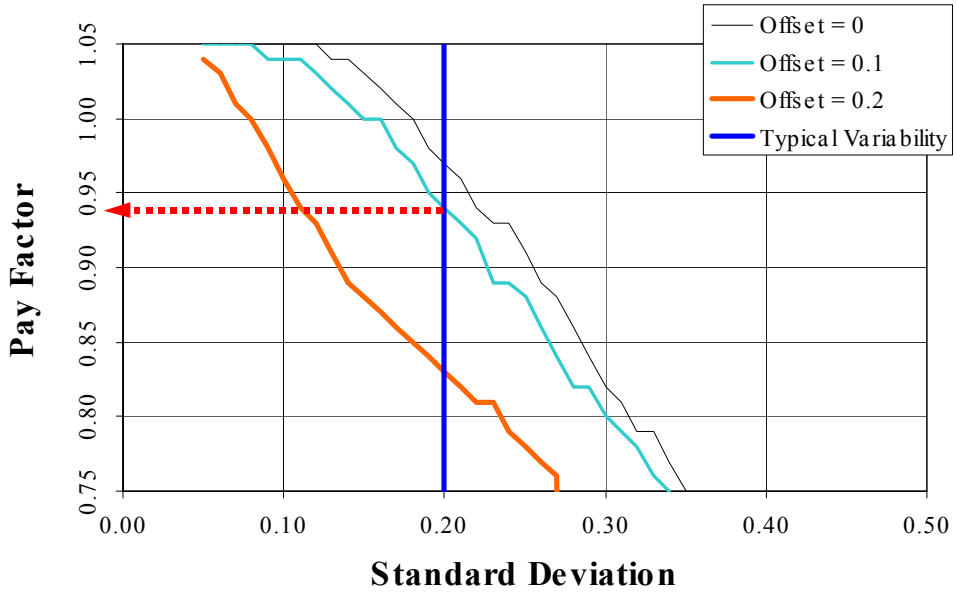


FIGURE 14 Pay factor and typical offset/standard deviation combination relationships with ±0.3% specification limits.

For a 100,000 ton project with a HMA contract price of $40 per ton, the change in composite pay factor (weight for asphalt content quality characteristic is 0.30) due to the specification change would be $120,000 (from a bonus of $48,000 to a penalty of $72,000). Based on the rational analysis conducted the specifying agency elected to change the asphalt content specification tolerances from ±0.5% to ±0.45%. The change was reasonable and fair to both the agency and industry and both parties agreed to it because of the rational (rather than arbitrary) basis for it.

SUMMARY AND CONCLUSIONS

Many state highway agencies use statistically based QC/QA specifications for HMA construction. The basic objective of the specifications is to specify and measure quality characteristics that are related to pavement performance, then to pay the contractor for the quality provided. Payment may include a bonus or penalty. There is a lack of rational in QC/QA specification due to the fact that robust relationships between quality measured and pavement performance simply do not exist. This makes it impossible to rationally determine life cycle costs and equitably develop pay factors.

The contractor is normally given QC responsibility and ultimately has the ability to control their financial success. Although many agency QC/QA specifications appear to be very similar, the reality is they are not. This is because of the differences that exist in the specifications in relation to: sampling and testing plans including lot and sublot definitions as well as number of tests per lot, selection of quality characteristics, sampling locations and methods, and replicate measurements per test; test methods and options within test methods; selection of specification limits; use of just QC or pooled QC and QA data for PWL determination; selection of AQL and RQL levels; assignment of seller's and buyer's risk; and pay factor multipliers and pay factor weights when composite pay factors are used. The influence of the array of differences listed above ultimately influence the outcome of the specification when implemented. Unfortunately engineering judgment or historical practice is commonly used when selecting specification criteria and the influence of the selections are essentially unknown at the time. This has become more evident with the use of QC/QA specifications that appear to be the same but generate different pay factors due to what appear to be subtle differences in items such as sampling plans.

The influence of changes in variability (due to any combination of factors) on QC/QA specifications was discussed. The influence of changes in variability and specification limits was illustrated by example to be quite significant in terms of ultimate payment.

In developing a statistically based QC/QA acceptance specification all of the items listed above must be defined/selected. Each and every selection ultimately has an impact on final acceptance and payment. Therefore it is extremely important that QC/QA specifications be developed based on know of all these items as well as experience with variation in each.

RECOMMENDATIONS

It is recommended that:

- Specifying agencies and contractors work together to develop specifications that are fair and equitable to both parties;
- Specification developers have a good working knowledge of the ultimate influence of specification parameter selections on pay factors;
- Specifications be used in a shadow form prior to full implementation;
- Specifications be refined over time as contractors gain knowledge/experience and equipment refinements become available;
- When refinements are made they are based on analysis of prior work and statistical analyses;
- Efforts to minimize variability in sampling and testing be continually supported; and
- Efforts to develop rational relationships between quality measured and pavement performance be continually supported.
- Databases be developed when specifications are used so that data are available for analysis and ultimately the basis for rational change in specifications.

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Impact of Incentives on In-Place Air Voids

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Specifying agencies face an on-going challenge with how to get the best quality hot-mix asphalt (HMA) overlays possible within the limits of available funding. Specifications for HMA have evolved from recipe type to end product statistically based specifications for acceptance, with a future trend towards performance related specifications. The Arizona Department of Transportation (ADOT) has been using some form of an end-product specification for HMA for about 20 years. The initial version of this specification was not a true incentive specification; positive pay lots could only be used to offset penalties and could not result in incentive pay. In 1990, this specification was changed to include a true incentive when the percent within limits was 100. Corresponding to this change, a drop in average in-place air voids from around 8.5% to 7.5% was seen. This reduction in in-place voids is expected to translate to improved pavement performance especially with respect to fatigue life. Although it is not easily possible to quantify the dollar value of this improved performance, ADOT believes that it more than offsets any additional costs incurred by the use of an incentive for compaction.

INTRODUCTION

Specifying agencies face an on-going challenge with how to get the best quality HMA overlays possible within the limits of available funding. Specifications for HMA have evolved from recipe type to end product statistically based specifications for acceptance, with a future trend towards performance related specifications. ADOT has been using some form of an end-product specification for HMA for about 20 years. This paper will concentrate on the compaction portion of ADOT's specification and its impact on the quality of HMA pavements constructed as measured by the in-place air voids.

HISTORY OF ADOT'S END-PRODUCT COMPACTION SPECIFICATION

ADOT began a gradual implementation of end-product specifications for HMA in the early 1980s. Development of the initial end product specification was a joint effort between ADOT and industry. The specification clearly outlined the responsibility of each party. The contractor is responsible for the selection and processing of materials that meet minimum quality standards; preparation of mix designs within established design criteria; performance of quality control measures; and to produce, place, and compact the HMA. ADOT is responsible for performing all acceptance and quality assurance testing. It is the intention of the specification to provide minimum direction to the contractor on how to reach the desired end product. Acceptance and total pay factors is based on the statistically determined percent within limits for specified properties.

The first two projects utilizing the new specification were successfully constructed in 1984. After review of the results of the initial two projects, refinements were made to the specification and additional projects were constructed in 1985. Acceptance for compaction was based on in-place density, as determined by the testing of 10 cores per lot (1 day's production). The target value for compaction was 98% of the density obtained in the lab using 75 blow Marshall compaction. The initial version of this specification was not a true incentive specification, positive pay lots could only be used to offset penalties and could not result in incentive pay.

In 1989, after construction of more than 6 million tons of HMA under this initial end product specification, an independent review of ADOT's specification was conducted (*1*). The study evaluated interlaboratory proficiency sample and acceptance test data, as well as data from other states and made recommendations for changes that should be made to the specification. The key findings of this study regarding the compaction portion of the specification include

(a) Few lots during the evaluation period were out of specification tolerances for mixture properties, however approximately 17% of the lots were outside the in-place density (compaction) limits, most often below the specified limits;

(b) The point of initiation of reduced pay factors favors the contractor in the ADOT specification when compared with other existing specifications, it was recommended that a PT (percent within limits) of 90 be used for zero-pay factor and the PT level below which a lot should be rejected should be considered with a PT of 50 as the suggested value; and

(c) In-place effective air void content should be used for field density or compaction control. The study recommendations were reviewed by ADOT and industry and agreed upon changes were incorporated into the 1990 Standard Specifications (*2*).

The most significant change in the 1990 specifications is the strengthening of the compaction requirements. The compaction tolerance change was slight, decreasing from ±4.5 pcf to ±4.0 pcf. The point of rejection was changed from a percent within limits of less than 20, to a percent within limits of less than 50. This means that at least 50% of the lot has to be within the upper and lower limit. In addition, the specification was changed to a true incentive specification where it was possible for the contractor to receive up to $1.00 per ton bonus for compaction.

DETAILS OF ADOT'S END-PRODUCT SPECIFICATION FOR COMPACTION

Control of compaction is the sole responsibility of the contractor when the end-product specification is in use. The contractor is responsible for the establishment of a rolling pattern and to have sufficient numbers and types of rollers to meet the requirements of the specification. Under the specification currently in use, the target value for compaction is 98% of the Marshall lab density, with a tolerance of ±4 pcf. The exceptions to this are the approximately 12 projects per year, constructed since 1995, using the specification for Superpave type mixtures that uses in-place air voids (target value 7%) to specify compaction. These projects are not included in this discussion.

Determination of the percent within limits is based on 10 cores per lot (one shift's production) taken at random locations designated by ADOT. The cores are randomly located both longitudinally and transversely on the pavement course with the outside one-foot adjacent

to an unconfined edge excluded from compaction pay factor determinations. At each of the random locations a second core is taken and held for dispute resolution, in case the contractor elects to question the compaction test results. The dispute resolution process utilizes an independent testing laboratory without knowledge of the specific project conditions. In general, contractors do not elect to referee compaction results because compaction penalties are generally for under compaction. Cores held for referee typically show a slight reduction in density due to lack of confinement during storage, and thus would not improve the contractor's pay.

ADOT utilizes a stepped rather than continuous pay factor table. The pay factors are fixed dollar amounts based on the percent within limits. This means that the bid price for HMA has no impact on the value of the pay factor applied. The maximum incentive is $1.00 when compaction is 100% within limits, and the maximum penalty is $3.00 when the percent within limits is less than 55. The pay factor step function is illustrated in Figure 1. The lot is in reject for compaction and subject to removal when the percent within limits is less than 50. Historically, lots in reject for compaction have been allowed to remain in place at maximum penalty when the in-place air voids are less than 10%, provided other materials properties are acceptable.

ISSUES WITH SPECIFICATION

Ideally, ADOT would like to see HMA compacted so that it has in-place air voids in the vicinity of 7%. The target value for the specification under discussion is based on 98% of the lab density. Setting the target value in this manner works well when the lab density is at or near the mix design target value, producing compaction target values that are in the range of 7% in-place air voids. However, in the case where the lab density is on the high side (still within the acceptable

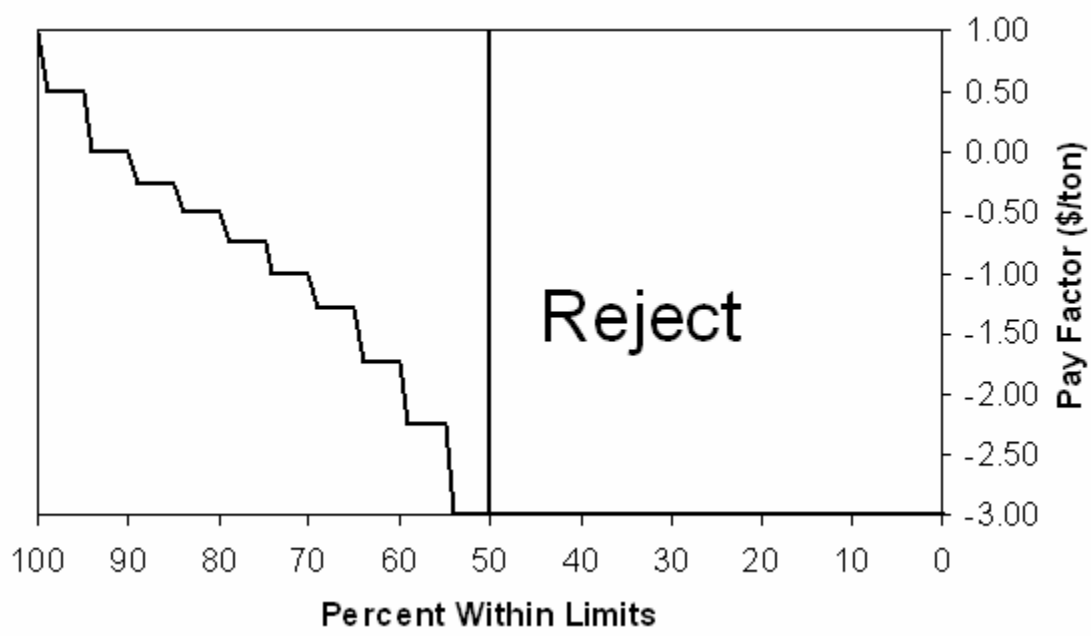

FIGURE 1 Compaction pay factor function.

limits of the specifications) the target density for compaction is relatively low and bonus may be paid for in-place void levels in the 10% to 11% range. Clearly, this is not a compaction level that is worthy of a bonus. The reverse is also true, in the case where lab density is on the low side (still within the acceptable limits of the specifications) the target density for compaction is relatively high and large penalties or reject status may be accessed for in-place void levels in the 6% to 7% range. Clearly, this is not a compaction level that should be penalized or in reject status.

Figure 2 illustrates the relationship between average in-place air voids and the percent within limits for lots placed during a thirteen-year period. This figure helps illustrate the shortcomings of ADOT's specification that bases pay factors on the in-place density (target value based on the lab density). Lots with percent within limits less that 50 are subject to removal. Most of the lots in this range do have high in-place air voids (greater than 10%) and warrant rejection, however there are a number of lots that probably don't warrant rejection and in some cases don't warrant even a penalty. It is worth noting that the impact of standard deviation on the percent within limits is not considered in this figure and some of the lots with reasonable average in-place have low percent within limits because of high variability (high standard deviation). At the other end of the scale, lots with percent within limits greater than 90 earn a bonus under these specifications. It can be clearly seen that a large number of lots with high in-place air voids (greater than 10%) earned a bonus.

To address this specification anomaly, ADOT is in the process of changing the compaction specification so that the target value is based on in-place air voids (7%) and is independent of the lab density of the mix. It is felt that this will result in lower in-place air voids on average and tighten the range of air voids.

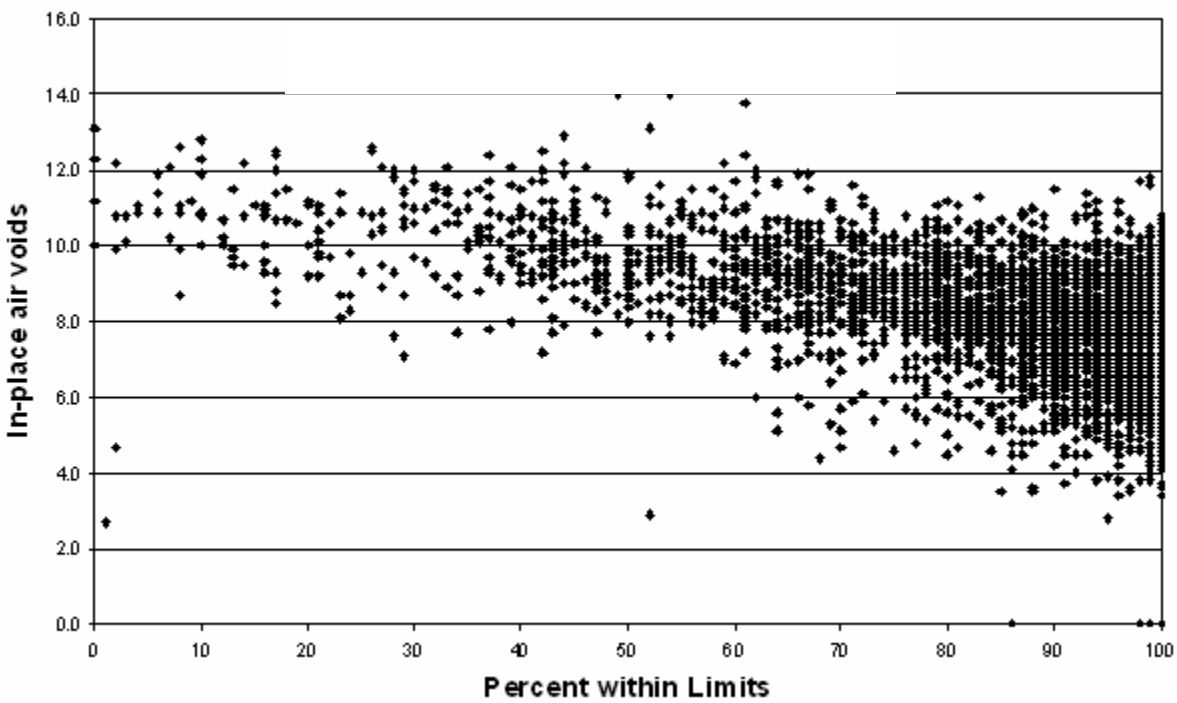

FIGURE 2 Lots placed with 98% lab density target.

For an incentive/disincentive specification to be effective in improving compaction, the incentive must be of sufficient magnitude to make it worth the extra effort required by the contractor to achieve it. ADOT's pay factors are based on fixed dollar amounts rather than a multiplier of the bid price and have not been changed since the inception of the end product specification. As a result, they may not have the same impact they had when first implemented. ADOT is currently reviewing its various pay factors for HMA to determine whether the magnitude of incentives and disincentives needs to be adjusted.

IMPACT OF INCENTIVES ON COMPACTION RESULTS

Since the inception of the end product compaction specification ADOT has maintained a database of test results. The availability of these data makes it possible to examine the impact of changes in the end product compaction specifications on the as constructed in-place air voids. Figure 3 illustrates the in-place air voids for the majority of compaction lots from projects constructed between 1985 and 1998 for which the end product specification was utilized. As you consider this figure, keep in mind that the compaction pay factor was based on in-place density, not in-place air voids and because of the specification anomaly described previously, any point on this chart may represent a lot with a positive pay factor.

Examination of this figure indicates that during the late 1980s and very early 1990s a considerable number of lots had in-place air voids in the 10% to 12% range with the average in-place air voids in the vicinity of 8.5%. Beginning in the early 1990s, there was a general reduction in the in-place air voids with the majority of the lots falling below 9.5% with the average in-place air voids in the vicinity of 7.5%. This improvement in compaction corresponds to the period of time when ADOT was transitioning to the new specifications that included

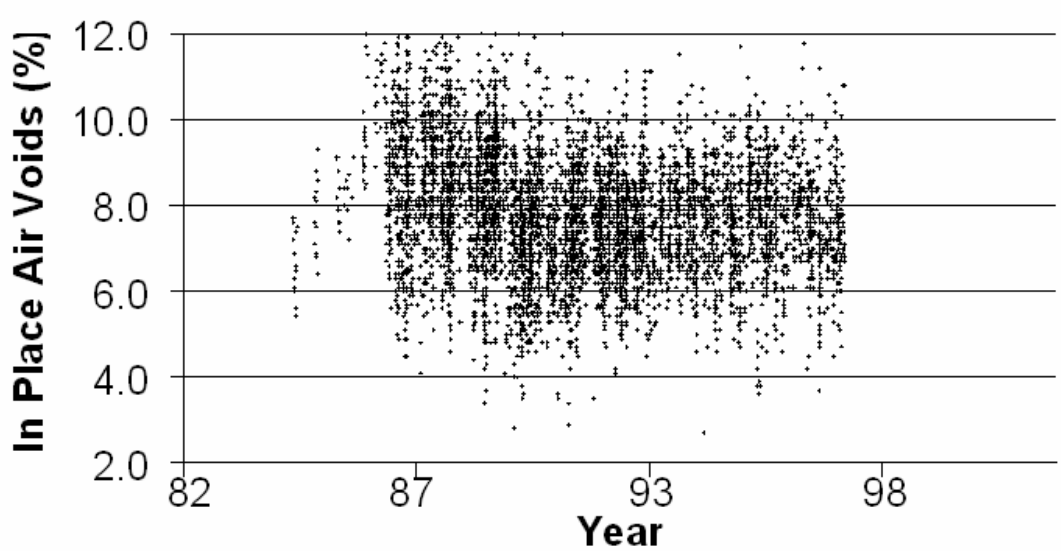

FIGURE 3 In-place air voids over time.

incentives for compaction. Because of the way new specifications are implemented, it can take several years for the full transition to occur. This reduction in in-place air voids can be expected to translate to improved performance of the HMA especially in fatigue.

There are several possible reasons for this improvement in compaction: contractor's became more skilled, changes to the percent within limits level for lot rejection, tightening of the tolerance, and implementation of an incentive for compaction. While all of these factors likely had impact on the improvement seen, it is believed that the factor with the greatest impact was the addition of the compaction incentive. An additional benefit of the incentive is that it potentially gives a bid advantage to contractors who are more skilled in obtaining compaction because they can reduce their bid price based on a proportion of the bonus they expect to receive.

SUMMARY

ADOT has utilized end product type specifications for HMA since 1984. The initial specification for compaction incorporated positive pay only as an offset to penalties received and thus was not a true incentive specification. In 1990, several changes were made to the specification including the addition of incentives for compaction. The addition of incentives for compaction corresponded to an approximately 1% reduction in average in-place air voids on ADOT end product HMA projects. This reduction in in-place voids is expected to translate to improve pavement performance especially with respect to fatigue life. Although it is not easily possible to quantify the dollar value of this improved performance, ADOT believes that it more than offsets any additional costs incurred by the use of an incentive for compaction.

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Percent Within Limits Experience on a Design–Build Project
Virginia Route 288

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INTRODUCTION

In December of 2000, under the Public-Private Transportation Act of 1995 (PPTA), the Virginia Department of Transportation (VDOT) awarded a $236 million design–build contract to APAC–Virginia, Inc., of Danville for the completion of Route 288, a new, four-lane highway, that extends 17.5 mi with 28 bridges and 10 interchanges around Richmond (Figure 1). APAC–Virginia, as the lead prime contractor and project manager, formed a partnership with Koch Performance Roads, Inc. (KPRI), and CH2M HILL to assist with the project. Koch Performance Roads provided pavement design, quality control/quality assurance/independent assurance (QC/QA/IA), public relations, and a limited performance-based 20-year pavement warranty contract. CH2M HILL had the lead design role and participated in the quality management and inspection of the project.

FIGURE 1 Aerial view of completed portion of Route 288.

On November 19, 2004, the highway came to fruition as VDOT opened the final segment of Route 288 to traffic. This last link between Lucks Lane and Route 60 (Midlothian Turnpike) in Chesterfield County allowed motorists, for the first time, to make a continuous 32-mi trip from Interstate 64 (I-64) in Goochland County to I-95 in Chesterfield County. The completed roadway is expected to bolster the economy of Chesterfield, Powhatan, Goochland, Henrico, and Hanover Counties, as they will be tied together with a direct highway link.

This particular delivery method (i.e., design–build–warranty) required an integrated performance-based decision-making process driven by the continuous assessment of the construction quality in the context of pavement design and long-term performance. A classic accept–reject specification system was ill fitted to fit this project's goals. This paper focuses on some of the practical applications of statistical process control using a percent within limits (PWL) specification approach and associated learning as an important component of the QC/QA/IA process in a design–build–warranty project.

The first section of the paper will discuss the overall QC/QA/IA elements and processes including test types, test frequencies, and specification limits in addition to the particular areas where PWL was applied. This section will be followed by a description of the factors effecting the attainment of density, a summary of project incentive–disincentive results, and some examples of the application of PWL. Finally, the paper will address perspectives, or lessons learned, and recommendations for future projects under similar delivery methods.

QUALITY ASSURANCE FRAMEWORK AND PERCENT WITHIN LIMITS

The overall QA process for the project was framed in a Quality Assurance Control Inspection Manual. The manual describes the quality organization and responsibilities, the inspection program and its related administrative functions, and the implementation of the system. The manual also contains specific information about the frequency of materials testing and the quality control and acceptance plan for the asphalt binder and the bituminous plant mix, including incentive–disincentive measures and payment calculations. To further ensure an acceptable outcome, a combination of process control and performance-related specifications were used to encourage the contractor to provide predictable results instead of conforming to specification minimums. Gradation and asphalt content were two process control parameters, while density, thickness, and smoothness were used as performance-related measures. Additionally, all specifications required quantity-based (e.g., tonnes) testing frequencies, as opposed to time-based (e.g., production day) testing frequencies.

Generally speaking, the quality index (QI) measure was used for the acceptance and determination of PWL and subsequent pay factors for asphalt binder content, gradation, construction thickness, field density, and surface smoothness. The QI uses both the average and standard deviation within each lot to estimate the population and determine the percentage of the lot within the specification limits (PWL). In principle, the percentage of the lot between the acceptable quality level (known as AQL or quality receiving 100 or more percent pay) and the rejectable quality level (known as RQL or quality requiring removal and replacement) is to remain in place and be used for subsequent pay factor determination. The quality levels for the Route 288 project were set at 90% for AQL and 60% for RQL.

The acceptance–rejection procedure consisted of determining the total PWL on a lot-by-lot basis. The project lot size was set at 1,200 metric tonnes (MT) for test strips, 2,400 MT for

normal production, and 3,600 MT for establishing uniform production. Uniform production was defined as two consecutive lots meeting uniform specification compliance, verification, and validation comparisons. All individual sample results for each lot were used to determine the statistical mean, standard deviation, and the quality level. If the total PWL for all properties was found to be 60% or higher, the lot was accepted as is in-place. If the quality level of any property on any lot was found to be less than 60%, the lot was rejected and subsequent removal and replacement was evaluated in the context of performance characteristics. (Sample results from rejected lots were not used for pay factor determination nor were any payment made for materials not incorporated into the finished pavement.)

All data, including singular results less than 60 PWL, were retained and used for pay factor computation on all accepted lots.

The initial quality plan called for an assessment of incentive–disincentive pay at the completion of a lift for the entire project. The project was scheduled over a multiple-year period and did not provide a timely recognition of economic (i.e., dollar) impacts on work performed. To encourage a direct and immediate reaction to production, KPRI provided APAC an interpretive letter that allowed for incentives to be paid or disincentives collected after the completion of five lots. The total project incentive–disincentive available amounted to $750,000 (incentive)–$2,250,000 (disincentive). The interpretive letter segregated 10% or $75,000 (incentive)–$225,000 (disincentive) of the total to be used exclusively for smoothness.

The selected upper and lower specification limits encouraged uniform production, handling, and placement while acknowledging some of the process and material limitations. Additionally, a strong engineering point of view on critical design distresses was used to derive a balance between excessive initial cost and eventual long-term costs. A ranking of priorities for the selected specification criteria may be inferred from the percentage weights shown in Table 1.

TABLE 1 Upper (USL) and Lower (LSL) Specification Limits

Mixture Properties /Tests	LSL	Target[1]	USL	Percentage Weight (%)[2] Non-surface	Percentage Weight (%)[2] Surface
Gradation 4.75-mm sieve	–5.0 %	JMF	+5.0%	5	5
Gradation 0.60-mm sieve	–4.0%	JMF	+4.0%	5	5
Gradation 0.075-mm sieve	–1.5%	JMF	+1.5%	5	5
Binder content (%AC) w/25%+ RAP	–0.3%	JMF	+0.3%	15	10
Binder content (%AC)	–0.2%	JMF	+0.2%	E	E
In-place compaction, %G_{mm} base	94%	96%	98%/99%	45	NA
In-place compaction, %G_{mm} binder and surface	93%	95%	97%	E	30
Profilograph index (PI) mm/km Binder lift—zero blanking band	—	—	473	P/F	NA
PI mm/km Surface lift—zero blanking band	—	—	316	NA	45
Design (D) lift thickness (mm)[3]	D–(0.10*D)	varies	NA	25	NA

[1] JMF: Job mix formula
[2] E: equivalent % applies as in prior row, not both; P/F: pass or fail only.
[3] Mainline typical HMA thickness; Surface: 50 mm, binder 2; 65 mm, binder 1; 75 mm, base; 100 mm

An interactive custom software tool known as quality builder (QB) was developed by KPRI early in 2001 to capture test results and to provide process control charts, graphs, summary information, and real-time PWL calculations for each lot (Figure 5). The software is a web-based system that provides instant access to all licensed users, including field personnel alongside the paving operation. The program was enhanced to accommodate changes as the design–build project evolved.

FACTORS AFFECTING DENSITY ON ROUTE 288

The ability to achieve the specified densities was affected by various factors, such as mix designs, material selection, equipment, site coordination, and field conditions. Such traditional factors, in conjunction with multiple specified quality indicators, required careful balancing throughout the production of hot-mix asphalt (HMA). This balancing process was very important as density is well known to be a key driver of long-term performance, whereas unacceptable compaction properties can lead to one or more undesirable distresses (e.g., stripping, cracking) over the life of the pavement.

The first batches of HMA placed on the project occurred in the fall of 2001 with excellent success in reproducing the laboratory verified mix designs. Both the base and first binder lift mixes met required quality levels. As the cooler temperatures of the late season approached, meeting density and asphalt content became somewhat problematic as the second binder lift was being placed. A strict enforcement of the specification called, at the time, for removal and replacement of some of the material. However, the following months of winter shutdown allowed for further assessment of the performance properties of the out-of-specification materials, and also for an expanded mechanical characterization of the local project materials and mixes with the goal of improving the performance prediction knowledge base. This additional characterization encompassed the performance testing of empirical and fundamental mechanical properties, such as repeated shear strain, resilient modulus (Mr), complex shear modulus (G*), and stripping/rutting with Hamburg wheel tracking. The pavement designer in conjunction with the warranty engineer analyzed and assessed performance risks and value associated with the placed material, creating an alternate strategy to meet the design requirements of this specific location.

Specific attention to detail was important on this design–build project to ensure that the PWL density specification could be achieved. Training the QC staff to recognize, react, and mitigate variability at all opportunities became an important factor in achieving density. Overall, training was a key determining factor in attaining quality throughout the construction of the project as changes and discoveries took place. Higher demands of PWL specifications and successful training resulted in active QC staff coordination and leadership in the overall operations of both plant and field operations.

The contractual relationship between APAC, KPRI, and VDOT allowed KPRI to perform additional performance testing on several job mix formulations during the off-season, in order to determine the mixture sensitivity to variations in asphalt content, volumetrics, and gradation. Changes were expected when using recycled asphalt pavement (RAP) as an additional material source.

Extreme weather presented a challenge to the successful compaction of all pavement layers. Weather conditions critically impacted the second- and third-year operations of the

project, where unusual rainy conditions prevailed with 50-year record precipitation and tornadoes. Several actions were taken during these periods of time to better manage the compaction requirements: adjusting production rates at the HMA plant to accommodate wet aggregates and moisture impacts; proof rolling soils and aggregate base layers to ensure a stable platform; and incorporating additional equipment and crews from across the country, to expedite final completion.

The uniformity in density was further improved by using a material transfer device (MTD) for most paving operations, along with well-maintained pavers and generally three rollers per spread. Trucking equipment availability, spacing, and coordination created a challenge with multiple paving spreads and trucks being provided by independent haulers. The timing of mix delivery, as influenced by the trucking factors, was probably the greatest variable and daily challenge to overcome.

The ability to achieve density was also impacted by non-contiguous work sites. In the context of this 20-year pavement performance project, the 28 bridges dominated the design–build job. The bridges were on the critical path, had first access priority, and created non-adjacent work sites to be graded, stabilized, and improved with the desired uniform quality pavement structure. Typical field conditions ranged from mile long stretches of three and four lane urban highway type roadway to multiple interchange loops, ramps, and flyovers. The coordination of HMA placement amidst lime soil stabilization, bridge construction and grading work, created a challenging environment for the asphalt operations and required of good communication, leadership, and sound managing efforts (Figure 2).

FIGURE 2 Construction partnering efforts on Route 288.

In a design–build project, the design team must be involved during construction. Subsurface conditions in some areas of the project were slightly different than originally designed for, creating an opportunity for modification of the typical section with the goal of speeding the construction efforts or reducing costs while maintaining the prescribed quality levels. The predominant project subgrade conditions consisted of weak saturated clay dominant soils with design California bearing ratio (CBR) values generally below 5. Some areas of the project were found to exhibit better soil conditions for which alternate pavement sections were designed. In addition to the alternate pavement sections, the project tied into 10 unique interchanges ranging from an Interstate freeway on one end, to a low-volume county road on the other. The challenge of attaining the required uniform density for each tie-in was surmounted by the field crews with proper pre-planning, coordination, and training.

INCENTIVE–DISINCENTIVE RESULTS ON ROUTE 288

The overall project incentive–disincentive results are tabulated in Attachment 1 by lift, plant, tonnages and incentive–disincentive payment outcomes. The total amount of HMA placed on the project was 490,305 MT. This included approximately 67% 329,040 MT of warranty pavement, with 61% 169,947 MT of that on mainline, and 39% 129,277 MT on ramps and loops. Overall 16.4% of the incentive was achieved for non-smoothness related properties and a disincentive of 38.2% for smoothness was imposed. The following sections describe particular PWL applications and summary results for the specification parameters of density, smoothness, and thickness.

Density Results

Field compaction was measured on cores and compared with the average lot maximum specific gravity for compliance with the specifications. It was generally found that the density results for the project conformed well to the target requirements with the exception of the Binder 1a mixture which showed the lowest total PWL and consequently the lowest pay factor (Table 2).

TABLE 2 PWL Density Results: All Lifts

Parameter	Base	Binder 1	Binder 1a	Binder 2	Surface
Target (%)	96	95	95	95	94
USL (%)	99	97	97	97	96
LSL (%)	94	93	93	93	92
No. of samples	220	170	26	229	168
Average (%)	96.15	95.17	96.11	94.46	94.33
Standard deviation (%)	1.23	1.38	1.48	1.37	1.21
Upper PWL (%)	100	91	73	97	92
Lower PWL (%)	97	95	99	86	98
Total PWL (%)	97	86	72	83	90
Individual pay factor (IPF)	1.05	0.97	0.87	0.95	1.00
Weight factor (%)	45	45	45	45	30

Two important specification changes were made based on the initial test strip work. The first was related to a higher than expected variability in the base lift density due mainly to the incorporation of RAP. Performance testing (e.g., rutting and fatigue testing) was conducted early in the project to determine if the upper and lower specification limits of plus or minus two percent points could be changed to plus three and minus two percent points. The findings indicated that the robust aggregate structure (i.e., gradation and aggregate properties) in the base mix, the higher than average asphalt content for improved impermeability and flexibility, and the three additional HMA lifts, provided a manageable performance risk profile allowing the change to occur. This new wider band of operation resulted in a better PWL for the base layer. The second change was related to a balancing process between density and smoothness for the final lift. After placing several lots of surface mix, it was found that conformance to the initial target of 95% of Gmm was achieved at the expense of smoothness. In this case, the combination of an engineered modified asphalt binder, a robust aggregate structure (i.e., gradation and aggregate properties), and an adequate pavement design, among other things, allowed for the reduction of the surface lift target density from 95% to 94%. This new value was further determined to be a superior balance for overall quality versus retaining the original target.

Density targets remained consistent throughout the remainder of the project; unlike the other mixture quality indicators, which were subject of job mix formula modifications. Lift thickness varied, as expected, with pavement section (e.g., different thickness for ramps vs. mainline paving).

Smoothness Results

Project smoothness was measured using a California type profilograph, and the specification criteria used a zero blanking band with a final pavement surface lift USL of 316 mm/km (20 in./mi), and final binder course USL of 473 mm/km (30 in./mi) (Table 3).

TABLE 3 PWL Smoothness Results for Warranted Pavement

Parameter	Mainline NB	Mainline SB	Mainline All	Ramps All[1]
USL (mm/km)	316	316	316	347
Number sublots	219	151	370	131
Average (mm/km)	277	248	266	291
Standard deviation (mm/km)	78	58	72	57
Upper PWL = Total PWL	69	88	76	84
Individual pay factor	0.85	0.99	0.90	0.96
Total no. of lots	20	15	35	20
Initial no. of lots <60 PWL	8	3	11	10
Corrected sublots (%)	56/219 = 26%	28/151=19%	84/370=23%	45/131=34%

[1] Excludes 25 sublots or three lots <60 PWL that were allowed to remain in place with maximum disincentive applied.

The PWL specification for smoothness was the least understood of the quality indicators. Conventional projects in this geographical area, at the time of construction, used two methods for characterizing smoothness: a 10-ft straightedge to detect bumps or dips, and a profilograph using a 0.2-in. blanking band criterion measured over 0.1-mi increments. Traditional criteria consisted of meeting a specified target for a day's work, with an incentive–disincentive specification based upon comparing a fixed target value with the results of the day's paving.

Route 288 similarly used both smoothness characterization methods. However, the profilograph specification used a zero blanking band with an incentive–disincentive specification based on PWL. This project specification required each individual wheel path measures averaged for sublots of 0.1 mi and lots of 1.0 mi. In a PWL smoothness specification, no target levels are used; only an USL level is provided. This USL value was initially misunderstood and presumed to represent a target level. This is the fundamental difference between classic specifications and statistical process control.

The first data reduction efforts on the project showed several lots to have less than 60 PWL. The smoothness specifications required corrective action, consisting of either diamond grinding or removal and replacement. Diamond grinding was chosen by APAC as the preferred alternative. Immediate lessons were learned from this initial experience, and several actions were implemented in an effort to improve smoothness performance over the remainder of the project. Such actions included training to address equipment issues (e.g., paver laser controls), continuous paving to reduce incidence of bumps, and more uniform delivery of materials. Additionally, the charts shown in Figure 1 were created to provide "target areas" that visually display the impact of average results and variability (i.e. standard deviation) on the application of PWL. The shaded colored areas in Figure 3 represent target values that provide superior results if consistently achieved within the 90 to 100 PWL range. For example; 10 mainline surface sublots with a standard deviation of 50 and an average of 214 mm/km would result in 100 PWL, or as Figure 3 shows, in order to achieve 100 PWL with a standard deviation of 50, the average smoothness results must be no higher than 214 mm/km.

As has been previously described, the Route 288 pavement structure was constructed in a non-continuous manner due to the priorities imposed by the 28 bridges on the project. Consequently, smoothness measurements were made on smaller noncontinuous segments as surface paving was completed. The labor-intensive 25-ft profilograph measurements were determined to be inefficient and costly, and with less than 50% of the surface paving completed APAC decided to purchase a high speed profiler (HSP). The HSP proved to deliver faster results, consume less labor, and was more responsive in facilitating changes dictated by the paving operations. During the final season of paving, the HSP was used extensively to quickly gather test results and provide feedback to paving crews. This was particularly helpful as weather impacts required accelerated operations and the use of multiple paving crews, many of which were onsite temporarily and were not as familiar with the project specifications.

The final incentive–disincentive amounts were independently computed for mainline, ramps and loops. The mainline smoothness compliance to specification was measured to have a 76 PWL, for which a corresponding $76,000 disincentive was applied. The ramps and loops smoothness compliance resulted in 84 PWL, for which a corresponding $13,000 disincentive was applied. An additional $90,000 penalty was applied for three lots allowed to remain in place.

Surface

USL (mm/km)	316			Mainline	
Qu	0.241	0.511	0.821	1.211	2.041
PWL	**60**	70	80	**90**	**100**
Std dev 1	**316**	315	315	**315**	**314**
5	**315**	313	312	**310**	**306**
10	**314**	311	308	**304**	**296**
20	**311**	306	300	**292**	**275**
30	**309**	301	291	**280**	**255**
40	**306**	296	283	**268**	**234**
50	**304**	290	275	**255**	**214**
60	**302**	285	267	**243**	**194**
70	**299**	280	259	**231**	**173**
80	**297**	275	250	**219**	**153**
90	**294**	270	242	**207**	**132**
100	**292**	265	234	**195**	**112**

USL (mm/km)	347			Ramp/Loop	
Qu	0.241	0.511	0.821	1.211	2.041
PWL	**60**	70	80	**90**	**100**
Std dev 1	**347**	346	346	**346**	**345**
5	**346**	344	343	**341**	**337**
10	**345**	342	339	**335**	**327**
20	**342**	337	331	**323**	**306**
30	**340**	332	322	**311**	**286**
40	**337**	327	314	**299**	**265**
50	**335**	321	306	**286**	**245**
60	**333**	316	298	**274**	**225**
70	**330**	311	290	**262**	**204**
80	**328**	306	281	**250**	**184**
90	**325**	301	273	**238**	**163**
100	**323**	296	265	**226**	**143**

Binder

USL (mm/km)	473			Mainline	
Qu	0.241	0.511	0.821	1.211	2.041
PWL	**60**	70	80	**90**	**100**
Std dev 1	**473**	472	472	**472**	**471**
5	**472**	470	469	**467**	**463**
10	**471**	468	465	**461**	**453**
20	**468**	463	457	**449**	**432**
30	**466**	458	448	**437**	**412**
40	**463**	453	440	**425**	**391**
50	**461**	447	432	**412**	**371**
60	**459**	442	424	**400**	**351**
70	**456**	437	416	**388**	**330**
80	**454**	432	407	**376**	**310**
90	**451**	427	399	**364**	**289**
100	**449**	422	391	**352**	**269**

USL (mm/km)	526			Ramp/Loop	
Qu	0.241	0.511	0.821	1.211	2.041
PWL	**60**	70	80	**90**	**100**
Std dev 1	**526**	525	525	**525**	**524**
5	**525**	523	522	**520**	**516**
10	**524**	521	518	**514**	**506**
20	**521**	516	510	**502**	**485**
30	**519**	511	501	**490**	**465**
40	**516**	506	493	**478**	**444**
50	**514**	500	485	**465**	**424**
60	**512**	495	477	**453**	**404**
70	**509**	490	469	**441**	**383**
80	**507**	485	460	**429**	**363**
90	**504**	480	452	**417**	**342**
100	**502**	475	444	**405**	**322**

FIGURE 3 Effect of average smoothness and standard deviation on PWL.

Thickness Results

Pavement thickness is always an important design component of the overall structure. On Route 288, each lift provides unique properties to address various potential distresses, in addition to contributing to the overall structural capacity. The typical section for mainline paving included four lifts of HMA: 50 mm of surface mix over 65 mm of binder 2 mix over 75 mm of binder 1 mix over 100 mm of base mix (Figure 4).

The importance of lift thickness was reflected in the incentive–disincentive computation, as 25% of the total available incentive was apportioned to the thickness of the non-surface lifts. The specifications also prescribed the lower specification limit (LSL) at 90% of design thickness (e.g., the base lift target was set at 100 mm with a LSL of 90 mm). Additionally, the specifications included a 13-mm maximum cumulative deviation from total design thickness.

This design–build project, unlike a traditional low bid project, did not require unit prices for materials. The contractual agreement with VDOT was a lump sum type contract, which encourages no more than tolerable deviation from minimum specified. However, the requirement for total (all lifts) minimum thickness encouraged the contractor to pave lower layers at or above target values to avoid thicker, more expensive, surface mixtures in the event the overall section does not meet minimum total thickness.

Table 4 depicts an example that illustrates the impact of PWL on two sets of lots exceeding the RQL and yet having significantly different financial outcomes. The initial two test strips were completed in the fall of 2001, with full production resuming on June 12, 2002. Thickness results for the initial two lots or 5,260 MT averaged 94.7 mm. The average thickness for the succeeding two lots, after resuming operations, averaged 101.3 mm. The application of

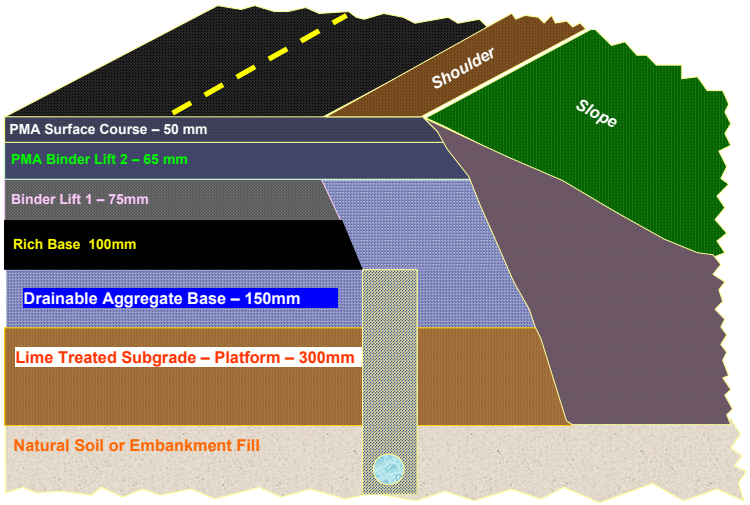

FIGURE 4 Typical mainline cross section for Route 288.

TABLE 4 Base Thickness Results for Lots 1–4

Parameter	Initial 2 Lots	Post Change (next 2 lots)
Tons	5,260	6,067
Average thickness (mm)	94.67	101.31
Standard deviation (mm)	6.35	5.68
PWL (%)	77	98
Pay factor	91%	106%
Incentive–disincentive factor	(.09)	.06
Initial cost benchmark ($/ton)	26.57	26.57
Weight factor	0.25	0.25
Incentive (disincentive) ($)	($3,145)	$2,418
Incentive (Disincentive) ($/Ton)	($0.60)	$0.40

the incentive/disincentive formula resulted in a net difference of $1.00 per ton.

PERSPECTIVES ON 288 PWL: WARRANTOR AND CONTRACTOR

This design–build–warrant project shifted the long-term performance risks of the warranted pavement from VDOT, the owner, to KPRI, the warranty holder. The warrantor assumed the more traditional owner role of specifying, testing, and inspecting the HMA to ensure compliance with the design and specifications, while executing in the context of a 20-year long-term fiscal and performance-based obligation to VDOT and the general public. The warranty is performance based as the condition of the road is measured annually over a period of 20 years. The warranty condition criteria are actual performance measures derived from terms negotiated with the owner. This shift of risk and responsibility, in addition to the design–build framework, contributed greatly to the innovation, change, achieved quality, and unique relationship between

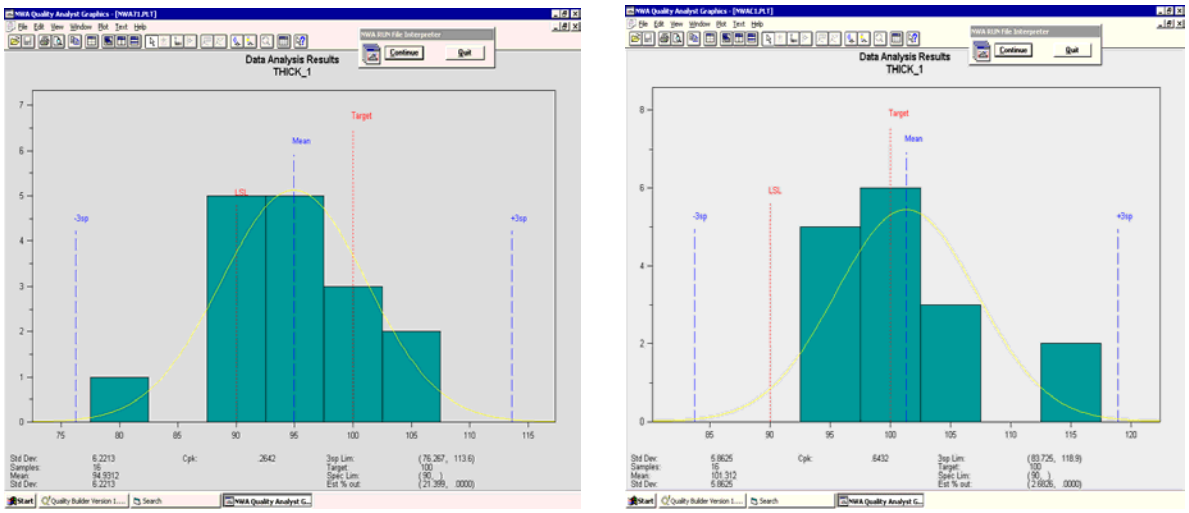

FIGURE 5 Example snapshot from quality builder software program.

warrantor and contractor. In this project, the contractor, owner, and warrantor's interests were commonly aligned to enable a more cooperative working relationship.

PWL specifications contributed to reduced variation of HMA production and placement activities on Route 288. Incentives and disincentives provided a constant feedback measure of quality to the producer. The warrantor's prevalent decision-making process is always driven by the relationships between conformance to targets with low variability and predicted pavement performance (including future maintenance requirements). Ideally, payment of incentives will result in a higher quality product with lower long-term maintenance needs. Similarly, disincentives should result in dollars available for unanticipated earlier maintenance needs. In addition, the large amount of data collected during the construction of the project will be used to compare the predicted and actual pavement performance, providing adjustments to be applied in future projects.

The combination of process and performance-related quality indicators specified on Route 288 represent an interim step in the evolution of performance specifications. Such steps were taken to bridge the gap between existing specifications and final performance measures required by the warranty terms and conditions. A direct transfer of the performance criteria to the supplier/contractor, without a collaborative and partnering mindset, would have been impractical and costly due in large part to unknown risks.

FINDINGS AND RECOMMENDATIONS

- Design–build–warranty projects should have an incentive–disincentive cost structure based upon short- and long-term performance outcomes. Such a framework should consider the most critical distresses and their associated probability and impacts to future maintenance costs.
- Training QC staff, material suppliers, paving crews, and others affecting the overall pavement structure, as to the impacts of their actions, was very effective for density operations and should be expanded to include other factors.

- Predictable quality results were obtained for most of the quality indicators as overall project incentive was achieved.
- PWL on smoothness requires more training on interpretation of values early in the process. Establishing target values with a better understanding of variability would result in more consistent attainment of incentives.
- Short- and long-term costs of corrective actions, whether remove and replace for mixtures or diamond grinding for smoothness, should be considered and determined well in advance to fully assess the risk associated for failing to meet 60 PWL. These costs can include but are not limited to lost time, equipment cost, material and labor costs, and additional maintenance costs.
- The use of properly calibrated high-speed profilers is recommended, as compared to a standard profilograph, because it provides an efficient and effective alternate.
- Coordination of material delivery to the project site is a major and important operational consideration in being able to attain quality as measured by the acceptable criteria and associated variance.
- As described throughout the paper, this design–build–warranty project reinforced the value of training, flexibility, proper equipment, and constant communication. Equally important was the constant integration between the pavement designer, the QC managers and the warranty manager in all phases of production and laydown operations.

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APPENDIX

All Non-Smoothness Quality Properties: Incentive–Disincentive

Base

Plant–Location	Tons	Eligible Incentive/(Disincentive)	Attained	Percent
RKV–Mainline	31,823	$66,774/($200,322)	$41,683	62.4
CHE–Mainline	18,551	$39,513/($118,539)	$16,933	42.9
RKV–Ramp	27,753	$59,113/($177,339)	$34,953	59.1
CHE–Ramp	6,416	$13,666/($40,998)	$5,856	42.9
Noneligible[1]	6,365	—		
Total	90,907	$179,067/($537,201)	$99,426	55.5

[1]Test strips and material placed exclusively on shoulders, as a separate operation, were excluded for incentive–disincentive.

Binder 1

Plant–Location	Tons	Eligible Incentive / (Disincentive)	Attained	Percent
RKV–Mainline	21,494	$46,426/($139,278)	$4,446	9.6
CHE–Mainline	18,721	$40,437/($121,311)	$12,766	31.6
RKV–Ramp	21,937	$46,601/($139,803)	$1,996	4.3
CHE–Ramp	4,940	$11,703/($35,109)	$2,950	25.2
Noneligible[1]	5,308	—		
Total	72,400	$145,167/($435,501)	$22,157	15.3

[1] Test strips and material placed exclusively on shoulders, as a separate operation, were excluded for incentive–disincentive.

Binder 1a

Plant–Location	Tons	Eligible Incentive/(Disincentive)	Attained	Percent
RKV–Mainline	1,986	$4,290/($12,870)	($938)	(7.3)
CHE–Mainline	9,296,	$20,079/($60,237)	($27,469)	(45.6)
RKV–Ramp	1,706	$3,684/($11,052)	($794)	(7.2)
CHE–Ramp	—	—	—	
Noneligible[1]	—	—	—	
Total	12,988	$28,054/ ($84,162)	($29,201)	(34.7)

[1] Test strips and material placed exclusively on shoulders, as a separate operation, were excluded for incentive–disincentive.

Binder 2

Plant–Location	Tons	Eligible Incentive/(Disincentive)	Attained	Percent
RKV–Mainline	33,249	$68,516/($205,548)	$5,117	7.5
CHE–Mainline	3,510	$5,123/($15,369)	$0	0
RKV–Ramp	24,964	$53,199/($159,597)	$2,463	4.6
CHE–Ramp	2,477	$5,548/($16,644)	$0	0
Noneligible [1, 2]	19,703	—	7,581	—
Total	83,902	$132,386/($397,158)	$	5.7

[1] Test strips and material placed exclusively on shoulders, as a separate operation, were excluded for incentive–disincentive.
[2] The initial eight lots were excluded from Incentive/disincentive consideration—all test strips, however lots 3–8 were counted within the basis for total tonnages.

Surface

Plant–Location	Tons	Eligible Incentive/(Disincentive)	Attained	Percent
RKV–Mainline	31,318	$98,025/($294,075)	$4,599	4.7
CHE–Mainline	—	-	—	—
RKV–Ramp	19,650	$61,504/($184,512)	$1,000	1.6
CHE–Ramp	138	$433/($1,299)	$0	0
Noneligible [1]	17,736	—		
Total	68,842	$159,962/($479,886)	$5,599	3.5

[1] Test strips and material placed exclusively on shoulders, as a separate operation, were excluded for incentive–disincentive

All Lifts

Plant–Location	Tons	Eligible Incentive/(Disincentive)	Attained	Percent
RKV–Mainline	119,869	$284,032/($852,096)	$54,907	19.3
CHE–Mainline	50,078	$105,152/($315,456)	$2,231	2.1
RKV–Ramp	96,010	$224,102/($672,306)	$39,618	17.7
CHE–Ramp	13,971	$31,350/($94,050)	$8,806	28.1
Non eligible [1]	49,112	—	—	—
Total	329,040	$644,636/ ($1,933,908)	$105,562	16.4

[1] Test strips and material placed exclusively on shoulders, as a separate operation, were excluded for incentive / disincentive

[2] The initial eight lots and all test strips were excluded from incentive–disincentive consideration; however lots 3–8 were counted within the basis for total tonnage. In summary, $30,364 were excluded from eligible incentive and ($91,092) from potential disincentive.

Smoothness Incentive–Disincentive

Location	Eligible Incentive/(Disincentive)	Attained	Percent
Mainline	$45,532/($136,596)	($76,000)	(55.6)
Ramps/Loops	$29,468/($88,404)	($13,000)	(14.7)
Total	$75,000/($225,000)	($86,000)	(38.2)

Test strips and material placed exclusively on shoulder, as a separate operation, were excluded for incentive–disincentive.

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