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AIRFIELD ASPHALT PAVEMENT CONSTRUCTION BEST PRACTICES MANUAL

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Airfield Asphalt Pavement Construction Best Practices Manual

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DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented within. The contents do not necessarily reflect the official views and policies of the Federal Aviation Administration. The report does not constitute a standard, specification or regulation. FAA requirements take precedent over information presented in the report.

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1. INTRODUCTION

Lack of good construction practices is a major contributor to poor performing hot mix asphalt (HMA) airfield pavements. It is necessary that those involved in airfield asphalt pavement construction have some knowledge of best construction practices so that a quality product can be obtained. There are currently several references that address elements of quality for HMA pavement construction, but most of these are focused on highway pavements. While many of the construction issues are the same for highways and airfields, there are other issues that are unique for airfields.

A comprehensive manual is needed to address HMA pavement issues with emphasis on the unique characteristics of airport pavement construction. This manual will provide the Federal Aviation Administration (FAA), Department of Defense (DOD) agencies, and others dealing with construction of HMA airfield pavements with comprehensive guidance and recommendations for improved construction practices for HMA pavements.

Performance of airfield hot mix asphalt mixtures can be improved if best practices are followed during the construction process. In fact, most performance problems observed on airfields are related to construction issues. For example, the biggest performance problems typically observed include cracking and raveling due to segregation, cracking and raveling at longitudinal joints, and surface deterioration due to lack of adequate density throughout the HMA. All of these construction problems result in the potential for foreign object damage (FOD), which is much more of an issue for airfields than for highways.

It is very difficult to find experienced, qualified technicians, inspectors, and engineers who can manage an HMA construction project for the government. In recent years, more of the responsibility for design and construction testing and inspection has been contracted out to consultants, commercial laboratories, and contractors. This contracting out of work has been standard practice within the FAA for years and this is becoming standard practice within the Department of Defense (DoD). This has resulted in a gradual loss of experienced personnel within the DoD. Due to this loss of experienced personnel, a best practices manual is needed especially for those representing the government and also for those contracted to work for the government so that everyone has access to the same technical information. This manual should help to ensure that high-quality HMA is constructed.

The purpose of this document is to provide a discussion of the best practices for construction of HMA mixtures. There are often many different construction methods used to construct HMA. For certain projects, one method may work better than another; however, for other projects, this same method may not be the best approach. Various methods will be discussed and some guidance provided so that readers can make the best decisions considering their specific projects. The manual also discusses all aspects of controlling mixture quality during construction, including stockpiling, mix design, plant operations, trucking, placement, compaction, and quality-control/quality-assurance testing.

The expected audience for this manual includes inspectors, technicians, designers, and contractor personnel involved in construction and quality-control testing, as well as foremen, and superintendents. This manual discusses construction methods and things to look for to help ensure good construction is accomplished.

2. EFFECT OF CONSTRUCTION DEFICIENCIES ON DESIGNED PERFORMANCE

2.1 Introduction

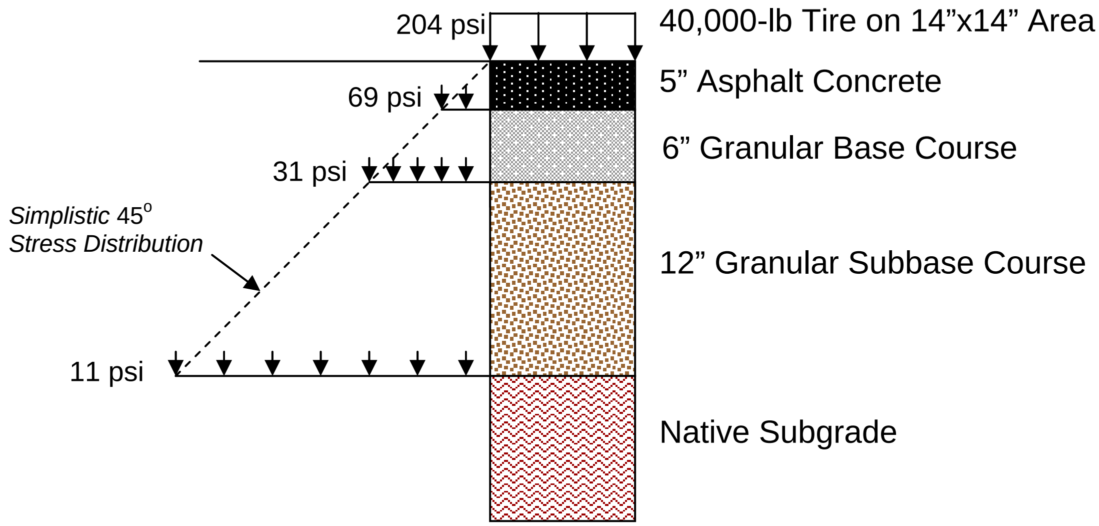
It is important during construction that materials, mixtures, and the constructed product meet the specification requirements. Hence, one might think that all the inspector and other oversight personnel have to do is ensure that the specification requirements are met. However, there are many items to control during construction, and there is seldom a project where everything goes completely according to plan. Those involved in oversight must understand materials, mix design, and construction to ensure that the specification requirements are met. It is also important to understand how deficiencies in the pavement design requirements will affect performance so that appropriate steps can be taken when issues arise. There will almost certainly be some portions of any project that will have some amount of work that is marginal, and decisions will have to be made about what actions to take. This section should help in making decisions about structural design issues.

The purpose of this section is to provide some guidance concerning the effect of deficiencies on performance. Some of the questions that this section will attempt to answer are: How does a change in thickness of the various layers affect performance? How does insufficient density of the various layers affect performance? How does plasticity index affect performance? How can a lack of positive control of drainage affect performance? The construction of a modern airfield pavement must ensure that the as-built product conforms to the design assumptions and concepts if proper performance is to be achieved. Some of the crucial design issues that may be compromised during construction are discussed in the following sections.

Official government guidance for civil and military airfield pavements is provided in the most recent version of Federal Aviation Administration Advisory Circular AC 140/5320-6, *Airport Pavement Design and Evaluation*, and in the latest version of the Department of Defense Unified Facility Criteria UFC-260-02, *Pavement Design for Airfields*. The FAA has been engaged in extensive research for pavement design over the last decade. The latest version of this advisory circular includes layered elastic design, which represents a departure from a long history of the FAA using the California Bearing Ratio (CBR) design approach.

2.2 Fundamental Airfield Flexible Pavement Design Concepts

The classical airfield asphalt concrete pavement consists of a relatively thin asphalt concrete surface on relatively thick granular base and subbase layers that rest on the natural subgrade. The asphalt concrete layer provides a smooth, all-weather operational surface that limits water penetration into underlying layers. The amount of pressure exerted by the aircraft tires affects the quality of materials needed in and near the surface, while the amount of load applied by the tires will affect the quality of the materials needed at a deeper depth. The applied surface load is distributed through each succeeding layer, which spreads the load over increasingly large areas thereby reducing the applied stress as illustrated in figure 2.1. The basic concept of flexible pavement design is to provide sufficient total pavement thickness to reduce applied stresses on the subgrade to levels appropriate for the subgrade strength and anticipated load repetitions. This is usually the fundamental concept used for most design approaches. Subgrade strength is the major parameter governing pavement thickness required for any given load.



Conceptual stress distribution in a flexible pavement for 40,000-lb wheel load on a 14”x14” contact area. This applies a 204 psi pressure to surface of the asphalt concrete. Using a simplistic *45°* approximation of the stress distribution with depth, the vertical stress on the base course surface would be 69 psi, 31 psi on the subbase course surface, and 11 psi on the subgrade. Better stress distribution estimates are available from Boussinesq, Frohlich stress concentration, layered elastic, finite element, and other analytical models.

Figure 2.1 – Typical flexible pavement showing stress reduction at underlying layers

Each pavement layer must be sufficiently strong to withstand the stress intensity to which it will be subjected, and it should not densify excessively under traffic. Hence, any changes in layer thickness or quality of material in any layer may result in reduced pavement life.

2.3 Thickness Effects

Subgrade strength has a major impact on the required design airfield pavement thickness. Deficiency in pavement thickness during construction will be reflected in premature shear rutting in the pavement before it reaches its design life. The thickness of various layers may be controlled by determining the difference in the surface elevations between the top and bottom of each layer, or by actually measuring the layer thickness.

Fatigue cracking in the asphalt concrete surface develops from a complex interaction of support conditions, mixture properties, loads, and the thickness of the asphalt concrete layer. Deficiencies in thickness during construction will result in increased strain at the bottom of the asphalt mixture, resulting in premature cracking of the surface. However, premature fatigue cracking may develop for other reasons besides inadequate thickness. This might include factors such as:

1. Poor or blocked drainage resulting in saturated subgrade or base/subbase materials with higher than anticipated deflections,
2. Overload,
3. Construction variability that results in undesirable mixture properties such as low asphalt content, overheating of the asphalt mixture, and high dust content. All of the variations result in stiffening of the asphalt mixture, eventually causing fatigue cracking, especially when thin layers of HMA are used.

2.4 Compaction

Each layer within an airfield pavement structure requires a specified degree of compaction to be achieved during construction. This compaction helps to ensure that two objectives are met. First, most desirable engineering properties of pavement materials improve with improved compaction and higher density (e.g., strength increases and permeability decreases). Consequently, if an asphalt concrete layer or base course material is inadequately compacted during construction, it may have inadequate shear strength and may fail when loaded. Asphalt concrete permeability increases rapidly as compacted density is lowered, so a poorly compacted asphalt layer's increased permeability may allow penetration or accumulation of water that might lead to stripping in the asphalt concrete or weakening of underlying layers. Even if the low mixture compaction does not result in excessive permeability, it will result in increased oxidation and earlier cracking problems.

The second objective of compaction is to densify all materials in the pavement structure and subgrade so that there will be no significant further densification under traffic. The large loads and high tire pressures of aircraft make them highly effective compaction devices. Although a material may have sufficient shear strength to resist shear deformation, if it densifies under traffic, one will still have rutting and surface distortion simply because of the decrease in material volume as it compacts. Figure 2.2 shows the densification of a base course under a parked USAF C-5 aircraft at a southeastern U.S. airbase. The sustained heavy gear load resulted in densification of the base under the tires and the surface depressions visible in figure 2.2. Figure 2.3 shows rutting in an asphalt concrete that developed from additional densification of the asphalt concrete by aircraft traffic. Failure to achieve the specified compaction in any pavement layer is a major potential construction defect that may significantly shorten the life of an airfield pavement.

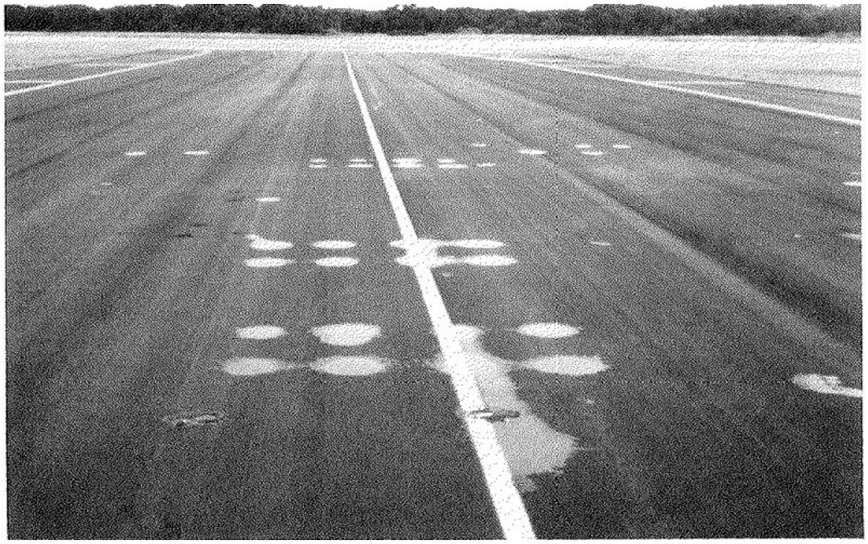


Figure 2.2 – Example of surface depressions caused by densification in the base course under a parked C-5 aircraft

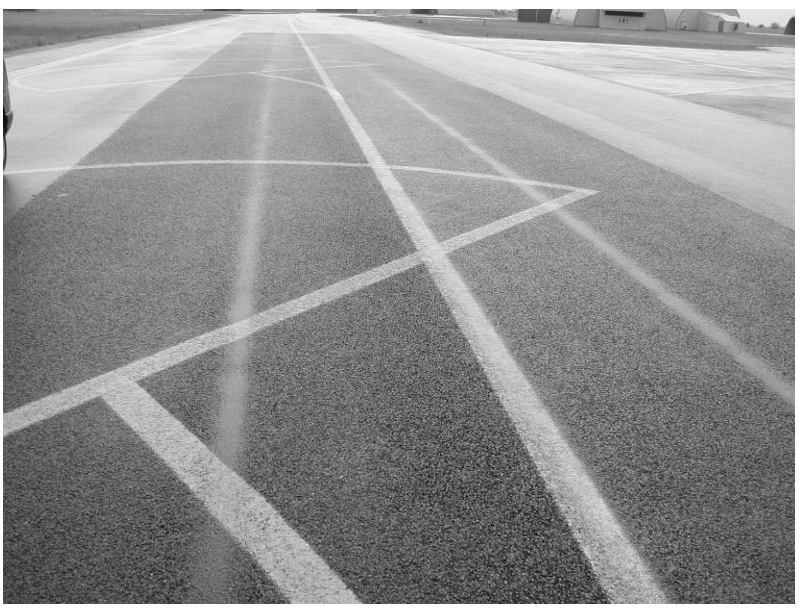


Figure 2.3 – Example of rutting caused by additional densification of an asphalt concrete by aircraft traffic (note water standing in ruts)

Past studies have found that aircraft traffic can compact granular base and subbase materials to densities greater than 100% of the density obtained in maximum modified laboratory compaction tests. For that reason, some agencies require proofrolling of subbase and base layers after the specified compaction is achieved. Proofrolling is usually limited to the runways and primary taxiways and for the most severe aircraft types. Proofrolling is discussed further in chapter 4.

As design has become more mechanistic, there has been a corresponding interest in verification of engineering properties of the in-situ newly constructed pavement. Some new instrumentation on the market allows one to measure pavement stiffness (e.g., applied load/deflection), the dynamic cone penetrometer allows an estimate of in-situ strength, and falling-weight deflectometer results can be used to calculate pavement layer modulus values. These types of devices allow practical verification of in-situ engineering properties that can be checked against design assumptions. However, they do not address rutting issues due to densification of underlying materials. Consequently, these stiffness and strength measurements are supplementary to conventional density testing and are not a substitute for it.

2.5 Water and Drainage

The oft quoted adage, "there are only three problems with pavements: water, water, and water," certainly has a kernel of truth. Some of the specific adverse impacts of water on flexible airfield pavements include the following:

1. Water on the pavement surface can pose safety problems due to loss of skid resistance and hydroplaning hazards for the aircraft.
2. Water allowed to accumulate adjacent to the pavement may attract bird life that then becomes a bird-aircraft safety hazard.
3. Water in the pavement structure usually causes a decrease in strength and stiffness of most engineering materials. This water may come from penetration through the surface, high ground water, capillary rise, frost effects, or lateral flow.
4. Soils and aggregates that are at or near saturation develop excess pore water pressure under traffic loads, thereby reducing their shear strength.
5. Moisture content change in some soils result in volume changes.
6. Increasing moisture content may lead to the collapse of some soils.
7. Moisture in asphalt concrete, especially when combined with repeated traffic loads, may lead to stripping in the asphalt concrete.
8. Water near the pavement surface can lead to blistering in the asphalt surface or seepage of moisture through the pavement surface in hot weather due to high vapor pressures.
9. In seasonal frost areas, water being drawn to the freezing zone can lead to frost heave of the pavement and severe weakening of the pavement structure during spring thaw.

The pavement structural design is usually based on soaked CBR values or some similar "worst-case" soil condition. Use of soaked CBR samples to select design strength is slightly conservative for subgrade soils and somewhat more conservative for base and subbase materials. Studies have repeatedly found that in zones of seasonal frost when conditions are right, the layers in the pavement can be saturated during the spring thaw period. The soaked CBR tests will often overestimate the resistance of the underlying materials in this case, and significant structural damage can occur during these periods.

Airfield designers typically use surface grades (this requires that the grade be closely controlled during construction) on the pavement to remove surface water, and then collect it in drainage ditches, swales, or storm-water systems to transport the water from the airfield area. The surface may be grooved, or the surface may have a special porous, high skid-resistant surface to improve traction and braking and reduce hydroplaning potential. In most cases, grooving is required on primary and secondary runways at commercial airports but is typically not used at general aviation airports.

Some designers and agencies incorporate very permeable drainage layers to rapidly remove subsurface water from the pavement structure. These drainage layers are typically an open-graded aggregate that may or may not be stabilized with portland cement or asphalt cement. The unstabilized layers are typically placed on top of the base course and can cause some construction difficulties as regrading and recompaction is often needed to correct surface rutting or movement in the open-graded materials under construction traffic. Placing a choke layer of smaller aggregates to help fill the surface voids often improves stability with negligible adverse impact on the drainage layer itself. Using small amounts of portland cement or asphalt cement to bind an open-graded aggregate together provides a better platform, but mixture proportioning must be carefully done to balance the need for permeability and strength to allow unencumbered construction operations. The drainage layer tends to be loose and difficult to compact. It is also often difficult to place and compact the overlying layer due to some movement of this loose drainage layer. Regardless of whether the drainage material is stabilized, care must be taken to avoid inadvertent clogging of the drainage material. This may happen from mud or dirt dropped by construction equipment or storm runoff or water from cleaning equipment running into the drainage layer and clogging the permeable material with fines. This can negate the value of a drainage layer before the pavement is even completed.

2.6 Compliance with Material Specifications

Failure to comply with material specifications is an area where construction can have particularly adverse impacts on the performance of an airfield pavement. Pavements are composed largely of natural materials that are inherently variable. Consequently, meeting tight airfield pavement specifications on aggregates, asphalt concrete, and other pavement components is not an easy task. A strong quality-control system by the contractor, supplemented with a good quality-assurance program by the owner, is most likely to ensure that materials placed are within specifications. This requires physical testing of materials, without which, no one knows what actually is going down in the pavement.

Airfield pavement base and subbase aggregates typically have very strict limits on physical properties. Generally, these materials are required to be well graded within specified grading bands. These requirements promote strength and stability of the material. Requirements for base courses are particularly tight as these materials are heavily loaded in the pavement structure and are major load-carrying components. Generally, military specifications mandate a coarser base aggregate, while the FAA overlaps a portion of the military base course specification and allows a somewhat finer base course aggregate in the fine aggregate particle sizes. Gap-graded materials (those deficient in a given range of particle sizes) are poor choices for use in base and subbase layers even if they meet the agency specifications. Such materials often do not provide adequate shear strength to support construction traffic and aircraft loads.

The percent passing the No. 200 sieve (fines) is a particularly crucial gradation characteristic in airfield base and subbase materials. If a dense well-graded aggregate with excess fines becomes saturated, or nearly so, pore pressures develop under traffic loads as the water cannot drain away from the loaded area. Gradations having humps (A hump in the gradation usually shows up when the slope of the gradation curve is equal to or less than the maximum density line for two or more sieves and then is steeper than the maximum density line for two or more adjacent smaller sieve sizes. This change in slope results in a relative hump and is often associated with natural sands which have a large amount of material on a few consecutive sieve sizes resulting in an increase in the slope of the gradation curve for these size ranges.) may indicate a high percentage of natural sand, which is typically not used in base courses and must be considered when used in the subbase. The surface of saturated aggregate layers will visibly pump under load and can sometimes be made to do so by just patting one's foot. To enhance internal drainage and prevent this detrimental pore pressure development, most agencies limit the percent passing the No. 200 sieve to 8% or less for airfield base courses, with somewhat more relaxed limits of typically 15% maximum for subbase materials. In frost-susceptible areas, the upper limit for passing the No. 200 sieve is often set at 3% to minimize the potential for capillary action. Failing to control fines in a base course may make it vulnerable to shear failures under load if it is loaded when saturated. Note that accurate determination of percent fines in a soil or aggregate requires a wet sieve test; dry sieving is not sufficient for accurate measurement of fines in a material.

In seasonal frost areas, controlling the fine fraction of the material used in the pavement is particularly critical. Small percentages of fines make a soil or aggregate much more susceptible to freezing effects. Frost-susceptible soils and aggregates are particularly prone to frost heave during cold weather and to extreme weakening during spring thaw. Frost failures have been observed in the base course when it is only slightly out of specification on the percentage passing the No. 200 sieve. Limited data in table 2.1 shows the effect of varying percentages passing the No. 200 sieve. Notice that it only takes a small amount of change in the percentage passing the No. 200 sieve to result in a significant effect on the amount of heave. It is important to control fines in all airfield aggregates; in seasonal frost areas, it is vital to do so.

Excessive fines are a problem:

The amount of fines will affect the permeability of the mixture. High fines will decrease permeability and make capillary action in frost areas a bigger problem. Fines are often maintained below 3% when base is placed in frost zone.

Table 2.1 – Heave rates of base course aggregate with varying fine content

Fine Content	Heave Rate, mm/day	Frost-Susceptibility Classification (ASTM D 5918)
1%	4	Low
2.5%	5	Medium
3.5%	11	High
4.5%	22	Very high

The aggregate plasticity characteristics are particularly critical. Figure 2.4 shows that there is a general trend of decreasing strength in soils and aggregates as the plasticity index increases. For this reason, the allowable plasticity index, or PI, for airfield base course and subbase aggregates is often limited to a maximum of 4 or 5 depending on the agency. A general aviation airport in the southeast used a natural clay gravel for a base course that was well graded and gave excellent strength when constructed. However, the plasticity index of the fines in the gravel was 11, which led to major losses in strength when the moisture content in the base increased. Within a few years, widespread failures developed in the base, requiring extensive repair. Allowing use of an aggregate exceeding the allowable PI limits compromises the structural strength of the layer when it is wet and thus should be avoided.

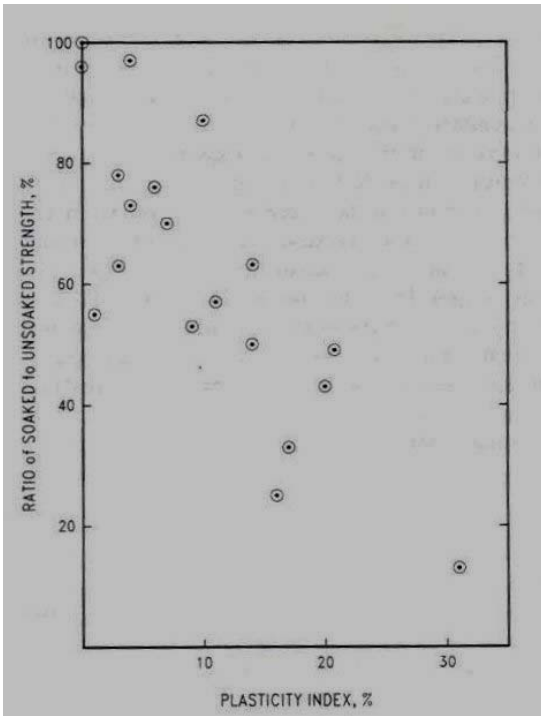


Figure 2.4 – Effect of plasticity index on retained soil and aggregate strength after soaking

Organic material tends to be highly compressible and weak. Attempts to stabilize organic soils with lime or portland cement are generally unsatisfactory as the organic materials interfere with the chemical reactions needed for stabilization. In general, organic soils are unsatisfactory as a foundation for airfields and must be removed. If the site exploration has not been adequate to identify such deposits before construction, these deposits still must be removed, and costs of removal will have to be allocated as outlined in the contract's changed conditions clauses. Organic soils tend to be more prevalent in the high latitudes rather than in the warmer climates where they are oxidized more rapidly.

Aggregate durability is a particularly difficult property to ascertain in the laboratory. The commonly specified tests such as sulfate soundness or Los Angeles abrasion are only modestly correlated with field performance. Hence, they are perhaps best thought of as screening tests, and one should always try to establish a history of past field performance for an aggregate source if at all possible. Discussion about aggregate breakdown from processing and compaction arises fairly often in the field. However, many cases of so-called aggregate breakdown are actually the result of aggregate segregation or contamination from poor stockpiling and handling practices. If aggregate breakdown is a suspected problem on a job, the aggregate stockpiling and handling should also be checked to be sure that is not the actual source of the problem. Nevertheless, there are aggregates that will break down when handled and compacted. Weathered residual aggregate particles are particularly prone to these breakdown problems.

2.7 Summary

Considerable effort is involved in the design of a modern civil or military airfield. However, if the design objectives are not achieved during construction, then the airfield pavement may fail or require major maintenance prior to the end of its expected design life. This chapter outlined some of the areas where construction deviations, sometimes only minor deviations, can have serious repercussions on pavement performance. In the end, the poorly designed but well-built airfield pavement will generally outlast the well-designed but poorly built one. Therefore, control during the construction process is very important.

3. SUBGRADE CONSIDERATIONS AND PREPARATION

3.1 Introduction

The native soil and fill materials that may be required to achieve the desired grade provide the foundation upon which the airfield pavement structure rests. The subgrade support value used in pavement design has a greater impact on the pavement design thickness than any other single factor. The general philosophy for flexible airfield pavement design is based on providing sufficient pavement thickness and stiffness to reduce the applied aircraft loads on the subgrade to a level tolerable for the subgrade strength. Hence, the ultimate cost and performance of a flexible airfield pavement is highly dependent on this critical subgrade material.

The subgrade must be brought to the design grade by cutting and filling as required by the topography and must be processed and compacted to achieve the expected design properties and to avoid future densification under aircraft traffic. Certain soils and conditions pose specific problems and will be addressed separately later in this chapter.

3.2 Construction Platform

No soil or aggregate can be placed on a yielding foundation and be expected to achieve the desired strength and needed density to resist densification and shear flow under traffic. On some sites, construction equipment mobility may be hampered such that construction costs are greatly increased or construction may be delayed or proven to be impossible.

It is extremely important that the potential site be assessed and modified as needed to provide a sound construction platform to allow proper processing and compaction of the subgrade soils and subsequent overlying areas. This may require simply draining wet areas, removal of vegetation and stumps, rerouting natural drainage channels, and providing for removal of rainwater accumulation during construction. This latter point should not be neglected as the topographic changes required in placing airfield pavements may significantly alter original drainage patterns and could result in flooding of construction worksites during rainstorms if provision for drainage is not included.

In some cases, the support conditions provided by natural conditions may not be suitable, and more elaborate efforts may be needed. Unsuitable material may have to be removed and replaced. Alternatively, the soils may be stabilized to improve their strength and provide an all-weather working platform. Geotextiles and granular fill have proven effective in dealing with soft wet areas in numerous earthwork projects. Clays are not the only problem soil. Fine sands and silts can also significantly hamper construction equipment mobility, and some blending or fill of better material may provide considerable construction benefit. In other very difficult cases, more elaborate steps involving wick drains and surcharging for deep weak materials or insulation to protect permafrost from melting may be necessary. Ideally, the need for special subgrade work and processing should be addressed during the design stage, but this is not always the case.

The subgrade and the subsequent pavement layers cannot be compacted properly if the underlying material is yielding and deflecting. Airfield pavement requirements mandate significant levels of compaction in the subgrade and in the overlying layers. To achieve this will require a sound foundation upon which to work. If the site does not provide a good foundation naturally, then a suitable foundation must be constructed prior to construction of the pavement.

3.3 Compaction

3.3.1 Requirements

The FAA and DOD requirements for compaction in airfield pavements are provided in Advisory Circular FAA AC-150-5320-6 and Unified Facility Criteria (DOD UFC 3-260-2). For a given aircraft loading and depth in the pavement, the material must be compacted to a given percentage of the modified laboratory density. For heavy aircraft the modified proctor laboratory density (ASTM D 1557) is used as the relative compaction. For lighter aircraft such as for general aviation airfields the standard proctor laboratory density is used (ASTM D 698). Different requirements exist for cohesionless and cohesive soils, with cohesionless soils requiring higher densities, all other factors being the same. These requirements extend well below the surface. For example, for a B-747, a cohesionless subgrade soil must be compacted to 100% modified density for the upper 23 in., 95% for depths of 23 to 41 in., 90% for depths of 41 to 59 in., and 85% for depths of 59 to 76 in. If the existing site soils do not meet these requirements and cannot be compacted to achieve this from the surface, then the subgrade must be removed to a sufficient depth and replaced and compacted in layers to achieve the required density.

These compaction requirements are based on the densities that would be expected in the pavement after a significant amount of traffic. Meeting these specified compaction requirements should prevent excessive densification in the pavement subgrade from aircraft traffic. (The military requirement for proofrolling to address compaction of subbases and bases to greater than 100% modified density under very heavy, high tire-pressure military aircraft is addressed in chapter 4.) These requirements provide protection against additional densification under traffic but do not address subgrade strength. Selection of soil strength for design is a separate issue addressed in the Advisory Circular and UFC mentioned above and is based upon the compaction needed to avoid densification and the likely placement conditions.

Project specifications normally dictate the percentage of modified density that is to be achieved for different layers and materials. These specifications also normally specify that compaction is to take place within specific limits of the optimum moisture content. A tolerance of +/- 1-1/2% to 2% is usually allowed around the optimum moisture content during placement.

3.3.2 Soil Behavior

Figure 3.1 illustrates a typical set of soil compaction curves for standard, intermediate, and modified levels of compaction. For any given level of compaction energy or effort, there is an optimum moisture content for which there is a maximum dry density. As the compaction energy increases from standard to modified effort, the optimum moisture content decreases and the maximum dry density increases.

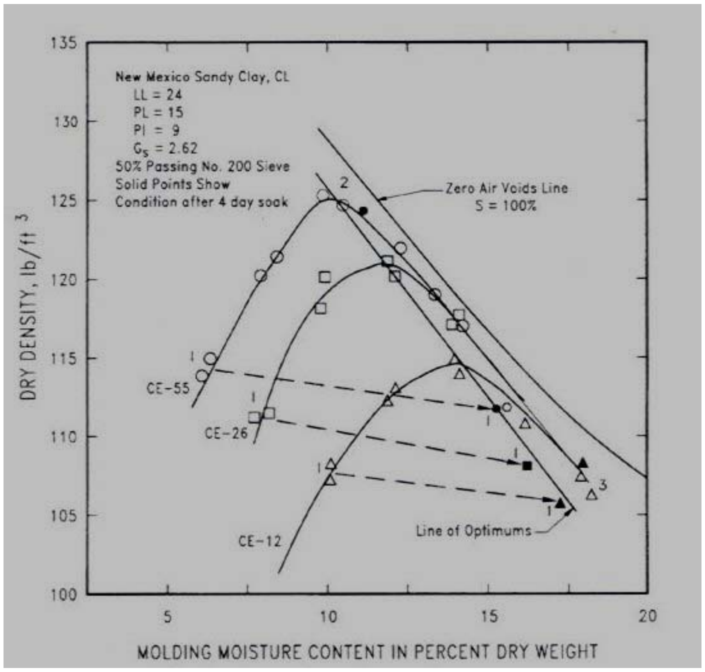


Figure 3.1 – Typical laboratory compaction curves for sandy clay

Airfield pavement work is normally specified in terms of modified proctor compaction density (ASTM D 1557). The standard proctor compaction is approximately 93% to 94% of the modified proctor compaction density for cohesionless and low plasticity materials and 88% to 91% for more plastic materials. These relationships are based on the average of thirty-five soils and are only to illustrate approximate relations between the expected densities for the two compaction energies. The actual relationship is dependent on the specific soil; therefore, approximations are not appropriate for trying to convert field and laboratory data for design or construction use.

Use modified proctor (ASTM D 1557):

Standard proctor is sometimes used for controlling compaction for soils, but for airfields the modified proctor is specified. The modified proctor will provide a density approximately 6% to 7% higher for cohesionless soils and approximately 9% to 12% higher for plastic materials.

In figure 3.1, an approximately linear line connects the points of optimum moisture content and maximum dry density at each compaction effort. This line is termed the *line of optimums*.

Also in figure 3.1, the slightly curved zero air voids line represents the point of 100% saturation. For a given moisture content, density, and specific gravity, all points will be on or below this line. The zero air voids line may be calculated as

$$\gamma_d = \frac{G_s \gamma_w}{1 + \frac{wG_s}{S}}$$

Where

γ_d = soil dry density
G_s = soil specific gravity
γ_w = unit weight of water, approximately 62.43 lb/ft^3
w = soil moisture content in decimal form (e.g., 0.10 not 10%)
S = saturation in decimal form (i.e., 1.00 not 100%)

Figure 3.2 shows compaction curves for several types of soils. Some cohesionless sands and gravels may compact when dry to density levels comparable to that achieved at their optimum moisture content, giving a double-peaked compaction curve (see the Keesler SW in figure 3.2, for example). In practice, the moisture content is somewhat higher than optimum for these sand or gravel materials since draindown of the moisture will occur. Silty soils tend to have a sharper more peaked compaction curve than more plastic soils, so moisture control of these silty soils during compaction is often critical to achieving good density.

When soil is compacted at moisture contents wet of optimum, the soil particles tend to be arranged in parallel sheets, while compaction dry of optimum tends to produce a more flocculated soil fabric. This fact has several implications for compaction of airfield subgrades.

Soil moisture content tends to increase above the moisture content used during construction after the pavement has been in place for some amount of time. The moisture content tends to approach an equilibrium content in 3 to 5 years under large paved airfield surfaces, with only minor fluctuations near the edges of the pavement.

Compacting expansive soils wet of optimum places the soils close to what is likely to be their final equilibrium moisture content. This minimizes any change in long-term moisture content over time and thereby minimizes potential soil volume change. This technique is sometimes used to help deal with expansive soils.

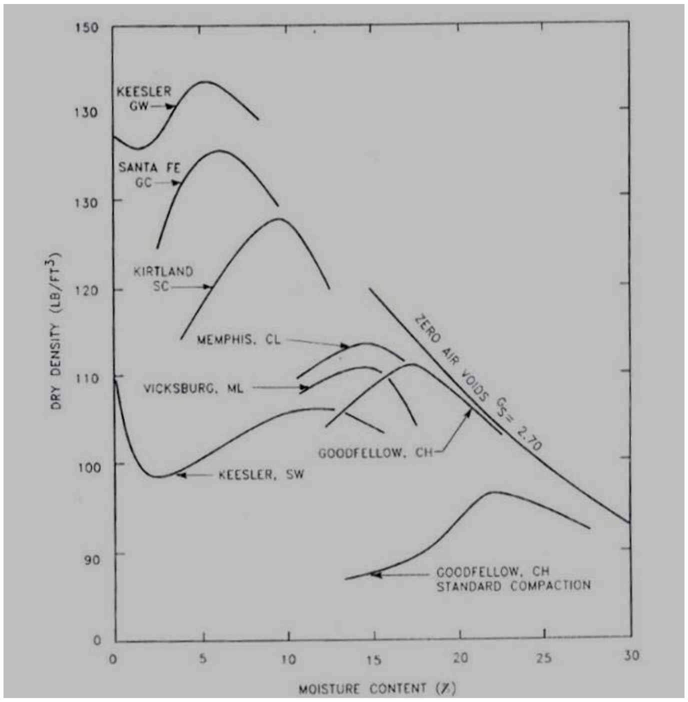


Figure 3.2 – Examples of compaction curves for different soils

> **Compact expansive soils on the wet side of optimum:**
>
> Soils will tend to become saturated after construction, and constructing on the wet side of optimum will minimize the amount of expansion.

If a soil is compacted dry of optimum, when the expected increase in moisture content occurs, it may become vulnerable to further densification under traffic. Hence, it is poor practice to place soils dry of the specified moisture content range, even if the specified density is achieved in the field. This will make the soil potentially more vulnerable to compaction under traffic later as subgrade moisture contents rise.

If a subgrade soil contains large aggregate particles, this poses problems for the laboratory compaction test. Depending on the number and size of the particles, test standards may require using a larger mold, scalping the oversize material, or replacing the oversize material with smaller particles. There is no perfect answer to this problem, so if such materials are to be encountered, the specifications should clearly specify what correction method will be used.

There are differences between soil behavior when compacted in the laboratory and under rollers in the field. The rollers will produce essentially the same shape curves as seen in figure 3.1, and the same effect of changing compaction energy is seen in the field as is seen in the laboratory. For rollers, increasing compaction energy may represent more passes, more weight, increasing tire pressure, etc. The line of optimums for a pneumatic rubber-tired roller will tend to be a little to the right of the laboratory line of optimums in figure 3.1, and a sheepsfoot roller would produce a line of optimums slightly to the left of the one shown. However, these differences are minor. Control of earthwork compaction using laboratory maximum densities and optimum moisture content has proven to be practical and workable in the field.

3.3.3 Compaction Equipment

A variety of compaction equipment is available. Each type of equipment offers different advantages and disadvantages.

Static Steel-Wheel Roller. The static steel-wheel roller (figure 3.3) has largely been replaced for heavy compaction work by the more efficient vibratory steel-wheel roller. These static steel-wheel rollers typically consist of two tandem drums, one in front and one behind, or they may have one drum in front and two large wheels behind (three-wheeled roller). The tandem steel-wheel roller is more common in asphalt concrete placement (in most cases vibratory rollers are used for compaction, but static rollers are used for finish rolling) and the three-wheel roller is sometimes encountered in compaction of some soils.

Figure 3.3 – Example of a static steel-wheel roller

Vibratory Steel-Wheel Roller. These rollers comprise the bulk of the compaction rollers used today. The most common rollers have two smooth tandem steel drums or one front smooth steel drum with pneumatic tires in the rear (figure 3.4). However, there are also towed models as well as various sheepsfoot and padfoot variations of the smooth drum. Vibratory rollers are available in a wide variety of sizes varying from the small walk-behind models to individual static drums weighing in excess of 20,000 lb. For soil compaction, the frequency of vibration is typically between 1,000 and 1,800 vibrations per minute (vpm) and may or may not be controllable by the operator. There may also be multiple settings for the drum amplitude, and this is set by changing the synchronization of the eccentric weights rotating within the drum.

Figure 3.4 – Example of a vibratory steel-wheel roller

The rotating eccentric weights within the drum impart a vibration to the soil that essentially reduces the interparticle friction and allows the particles to more easily rearrange themselves into a denser packing. Soil pressure under vibratory rollers can be 10% to 100% greater than for static conditions. These rollers are most effective with cohesionless soils, and their efficiency decreases as the soil plasticity and amount of fines increase.

Trying to compact soils in the field at the resonant frequency is largely an academic exercise (the resonant frequency is that frequency for a specific vibrator-soil system for which displacements are a maximum). The resonant frequency is very specific to test method and specific vibrator-soil system. It also changes with soil density and moisture content, which may be changing during the compaction process. Fortunately, effective compaction is achievable in the field at a variety of frequencies and finding and operating at a specific resonant frequency is not necessary. Compaction sometimes tends to improve as frequency increases up to a point, which may or may not be the resonant frequency, after which further increases in frequency have relatively little effect.

Static drum weight is a reasonable indicator of the effectiveness of a vibratory roller. To achieve high density required for airfield levels of compaction, the roller should probably have a static drum weight of at least about 125 to 150 lb/lineal in. of drum width and a frequency of operation within the range of 1,000 to 1,800 vpm. Sandy cohesionless soils will respond best and can probably be adequately compacted with lighter rollers, while more cohesive and fine-grained soils will tend to require a heavier static roller. Each soil is different, and some trial and error may be needed to get the right size roller, number of passes, frequency and amplitude settings, etc., for a specific site.

Sheepsfoot and Padfoot Rollers. A sheepsfoot roller consists of a drum with protruding feet such as seen in figure 3.5. The roller may be towed or self propelled and may be static or vibratory. There are also a variety of alternative foot designs available to the classical sheepsfoot. These may be marketed as padfoot, clubfoot, or tamping foot compactors, but they are all the same in principal.

Figure 3.5 – Example of a sheepsfoot roller

The rotation of the drum brings the different feet into contact with the soil and kneads it as it imparts compaction energy to the soil. As the roller progressively compacts the soil, the feet may progressively penetrate less deeply into the soil, giving the impression the roller is "walking out" of the soil. This is not necessarily an indication of good compaction. A light roller may rapidly walk out of a soil leaving the soil at lower density than that obtained with a heavier roller that fails to walk out. In the end, the criterion of success or failure is the density achieved and not whether the roller walks out.

When the sheepsfoot roller first begins compaction, the soil is not very dense, and a significant amount of penetration is obtained. The stress on the soil is a function of the number and area of the feet on the roller in contact with the soil and the weight of the roller. The increased initial penetration of the feet into the soil allows more feet or pads to be in touch with the soil, thus applying relatively low stress. However, as the material is rolled, the density increases and the penetration of the feet decreases, resulting in fewer feet or pads being in touch with the soil. This has the desirable benefits of applying less pressure when the soil is soft and not very well compacted and applying more pressure as the soil is compacted.

The sheepsfoot and related rollers are classically best for cohesive soils. The larger and shorter feet sometimes used on the padfoot rollers may allow the equipment to be operated at higher speeds, improving construction efficiency. Also, the larger size foot reduces the intensity of loading on the soil; this lowered intensity might require more passes to achieve the target density but has the benefit of being capable of effectively compacting softer soils.

Rubber-Tired Pneumatic Rollers. These rollers (figure 3.6) probably can effectively compact the widest range of soil types and can be used on both cohesive and cohesionless soils. They are available in a wide variety of sizes and tire pressures. The tire pressure is critical in achieving high density and must be selected for the stiffness of the material being compacted. For very stiff materials, the tire pressure should be higher, maybe as high as 90 to 100 psi; however, for softer soils, the soil pressure may have to be significantly lower. When the material is being compacted dry of optimum and near optimum moisture content, the tire pressure and number of roller coverages have a major effect on the level of density achieved. However, when the material is wet of optimum, the benefit of higher tire pressure and number of roller coverages diminishes.

Figure 3.6 – Example of a rubber-tired pneumatic roller

Miscellaneous Compaction Equipment. There is a wide variety of small rollers, vibratory plates, hand- or equipment-operated impact compaction devices, etc., designed to compact soils in confined spaces, such as in trenches for lighting and other utilities. It is possible to achieve the level of compaction specified for airfields with such small equipment, but this requires, thin lifts, good compaction techniques, moisture control, and often many passes of the equipment.

3.3.4 Lift Thickness

Most airfield pavements specify that the compacted lift thickness for subgrade, subbase, and base course materials should not exceed 6 inches in thickness. This represents the best compaction that most compactors can achieve, but there are some large compactors, especially vibratory rollers in sandy soils, that can achieve specified levels of compaction in thicker lifts. Usually, the increase in thickness of materials that can be effectively compacted will be modest (i.e., the increase in thickness might be a few inches but not significantly more).

If a thicker lift is proposed for compaction, the specified compaction must be achieved throughout the entire lift thickness. This means the density at the bottom of the lift must be measured by excavation or other technique and compared to the density specified. It is not sufficient that the average density over a thick lift meet the density. The density in the lower part must meet the specification.

3.3.5 Measuring Field Density

Most agencies accept the sand cone test (ASTM C1556) or the nuclear gauge density (ASTM C6938) for determining the density of soil in the field for compliance with compaction specifications. Less commonly, one may also encounter the rubber balloon test (ASTM D2167) and the drive cylinder (ASTM D2937) used for determining field density. Research is developing newer test instruments that are nondestructive and do not rely on a nuclear source. The sand cone and nuclear gauge do a little better in reducing test error than the rubber balloon or drive cylinder methods.

The sand cone test should be considered as the default standard procedure, and a certain percentage of the measurements should be done with the sand cone (e.g., 1 in 10 tests should be with the sand cone) even if the nuclear gauge is allowed. However, the nuclear gauge offers faster and easier testing in the field, with immediate results. Since it uses a nuclear source for estimating density, the nuclear gauge also has specific licensing, training, security, and exposure-monitoring requirements that must be met.

The nuclear gauge may be operated in either backscatter or direct transmission mode. In backscatter mode the attenuation of alpha radiation is measured with both the source and detector on the surface, while with the direct transmission mode the attenuation is determined with the source at a depth up to 12 in. below the surface and the detector on the surface. The direct transmission mode is a more accurate approach and is the recommended approach for nuclear density determinations to be used with compaction specifications for soils and granular materials. The attenuation of the alpha radiation is used to determine the soil wet density using a previously established calibration.

> When using the nuclear gauge for density testing, the direct transmission mode is normally used for soils and granular layers, and the backscatter mode is normally used for asphalt layers.

The moisture content of soil is determined by the nuclear gauge by measuring the slowing of fast neutrons by the hydrogen in the water in the soil mass. Both the source and detector for these measurements are on the surface, so moisture content determinations are a backscatter type measurement. The nuclear gauge may provide significant error in moisture content measurements in some soils. It is prudent to periodically check the nuclear gauge moisture content with oven-dried samples.

The project specifications should explicitly state what test method will be used to determine soil density in the field to ascertain compliance with the specification compaction requirements. The skill and experience of the testing technician also has a major impact on the quality of the measured field density. Adequately trained testing personnel are critically important.

3.3.6 Troubleshooting Compaction Problems

Airfield compaction requirements are rigorous. However, to prevent soils from densifying under traffic and to ensure that the desired engineering properties are achieved, such requirements need to be met. If problems achieving the specified compaction level are encountered, the following issues often are the cause of the problem.

Proper Moisture Content. Be sure the soil is at the proper moisture content for compaction. If the soil is too dry or too wet, achieving the specified airfield compaction may prove difficult if not impossible. Silty soils often have a very narrow compaction curve, so achieving tight control over the moisture is particularly critical. It is the moisture content at the time of compaction that is important. If the weather conditions favor evaporation (e.g., low humidity, wind, intense solar radiation), the soil may be drying out before the compaction is complete. More water must be added during compaction, or the loss in water from the soil due to evaporation must be accounted for at the start of compaction. Failure to compact the soil at the proper moisture content is probably the number one cause of compaction problems in the field.

Acceptability of Rollers and Proper Rolling Procedures. Proper compaction cannot be achieved with undersized rollers. Trying to compact with undersized equipment is one of the most common causes of poor field compaction. Switching to a larger roller or to one with a higher tire pressure for pneumatic rollers may be necessary to achieve the specified compaction. The roller type should also match the soil, as discussed earlier. For vibratory rollers, varying the vibration amplitude or frequency may improve results. Rollers should be operating at approximately walking speed to be most effective. Excessive speed to nominally maximize production can contribute to low density results. The roller operators should use a consistent pattern so that the area to be compacted is uniformly covered with a uniform compactive effort. On some cohesionless materials, a vibratory roller may loosen the upper few inches of the soil; finishing the rolling pattern with a few passes with the vibrator off may help raise the in-situ density measurement.

Foundation Support. If the foundation below the layer to be compacted yields or deflects, compaction will prove impossible. To achieve high levels of compaction, one must have a firm foundation. Any underlying soft materials, wet or spongy areas, debris, etc., must be dried or removed and replaced with satisfactory compacted fill before compaction can proceed.

Critical items for meeting density requirements:

- Maintain optimum moisture content.
- Utilize proper rollers and rolling techniques.
- Ensure adequate support from underlying materials.
- Use satisfactory lift thickness.
- Control uniformity of materials.
- Ensure accuracy of test results.

Lift Thickness. Generally, it is difficult to achieve high compaction levels in lifts thicker than 6 in. or so (compacted thickness). Commonly, equipment manufacturers are overly optimistic on the depths of effective compaction for their equipment, especially when one is trying to meet the high compaction levels required in airfield pavements. Shifting to thinner lifts may prove helpful in achieving the desired compaction level, but compaction will again become difficult if the layer thickness is reduced too much.

Uniformity of Materials. The specified field moisture content range and minimum density are based on tests of a specific material in the laboratory. Small variations in soil properties such as percent fines or plasticity may result in significantly different laboratory compaction results. The laboratory soil must be representative of the field soil for the specified compaction requirements to be meaningful. If the soil in the field is varying widely, it may be useful to develop a family of compaction curves or to use 1-point field compaction results to adjust the required density for varying soil conditions. Segregation of materials is also a significant problem.

Accuracy of Test Results. Ensure that the laboratory test procedures and the field density measurements are performed strictly in accordance with the specified procedures. There may be errors in the procedures themselves (e.g., a small air gap under a nuclear gauge can underestimate the actual field density, or the wrong over-size particle correction may be used in the laboratory test and give different target densities). There are limitations in any field measurement of density and in the test procedures themselves. Consequently, the specification should clearly lay out what specific laboratory test procedures will be used to establish the required field compaction results and how the in-situ soil will be assessed to ascertain compliance.

3.4 Stabilization

The subgrade may be stabilized for a variety of reasons, including:

1. To increase strength of a weak subgrade
2. To provide an all-weather construction platform
3. To mitigate adverse engineering behavior
4. To improve soil properties for construction

General guidance for selecting type of stabilizer:

Passing No. 4	Passing No 200	Soil Classification	Recommended Stabilizer	Limits for LL and PI and Comments
>50	0-5	SW or SP	B	
			C	
			L-C-F	PI<25
>50	5-12	SW-SM, SP-SM, SW-SC, or SP-SC	B	PI<10
			C	PI<30
			L	PI>12
			L-C-F	PI<25
<50	0-5	GW or GP	B	Well-graded material only
			C	
			L-C-F	PI < 25
<50	5-12	GW-GM, GP-GM, GW-GC, GP-GC	B	PI<1, well-graded material only
			C	PI<30
			L	PI>12
			L-C-F	PI<25
>35	12-50	SM, SC, or SM-SC	B	PI<10
			C	PI< 20 + (50 - %No. 200)/4
			L	PI>12
			L-C-F	PI<25
<35	12-50	GM, GC, GM-GC	B	PI<10, well graded, <30% No. 200
			C	PI< 20 + (50 - %No. 200)/4
			L	PI>12
			L-C-F	PI<25
---	>50	CH, CL, MH, ML, OH, OL, ML-CL	C	PI<20
			L	PI>12

B-bituminous, C-cement, L-lime, F-fly ash
Summarized from TM 5-822-14, Soil classification system from Mil-Std 619B

Although strength gain is often thought to be the objective of stabilization, the other potential uses for subgrade stabilization listed above are also important. There are several stabilization technologies available today, and selection of the stabilization method for any specific project should consider the following: objectives to be achieved by the stabilization, the soil to be stabilized, and an assessment of the economic, construction, and potential durability issues associated with each potential stabilizer.

3.4.1 Lime Stabilization

Hydrated lime ($Ca(OH)_2$) and quicklime (CaO) to be used for stabilization and their dolomitic variants should meet the requirements of ASTM C977. Hydrated lime is a fine powder and can cause dust problems. Quicklime is more granular without the dusting problems of hydrated lime. However, quicklime is caustic, and protective safety equipment is needed. Such safety procedures are well established. Note that ground limestone ($CaCO_3$) and dolomite ($MgCO_3$) are often sold as "agricultural lime," but these agricultural products are ineffective as soil stabilizers.

Several actions occur when lime is mixed with soil, and these actions are the basis for lime's effectiveness as a stabilizing agent. Most of these actions are rapid reactions and will have taken place within an hour of addition of the lime. Pozzolanic reactions for strength gain are much slower, however, and normally 28 days are allocated for these reactions and the strength gain to take place. The stabilization effects from lime include:

1. Reduction in moisture content. When quicklime (CaO) is added to a soil, the moisture in the soil reacts with the quicklime to form hydrated lime ($Ca(OH)_2$), thereby lowering the soil moisture content. The hydration reaction is highly exothermic, which also evaporates water from the soil. Hence, quicklime is a very effective drying agent for soils.

2. Reduction in soil plasticity. Calcium cations from the lime replace sodium and potassium cations in the clay mineral, thereby reducing the plasticity characteristics of the soil (usually measured as the Atterberg limits). This process is known as cation exchange and is the key to several important potential stabilization applications for lime. With a reduction in plasticity, the ability of the clay to hold moisture on the surface of the clay particle is reduced, and adverse expansive characteristics are reduced. This is often an effective way to mitigate expansive clays. It will also make the soil easier to dry by aeration. The drop in plasticity may make a cohesive soil less sticky and easier to work with. There may also be a slight improvement in early age strength that may help with site mobility for construction equipment. The drop in plasticity also makes the soil less susceptible to loss in strength if the soil moisture content rises from rain or other sources.

3. Change in soil texture. The addition of lime makes individual clay particles clump together to make a more silty, coarser soil. This effect can be measured by running a hydrometer analysis before and after adding lime, and the gradation of

the soil material finer than the No. 200 sieve will become coarser. This coarsening of the soil makes it easier to work with, increases soil permeability, can speed drying during aeration of the soil, and may make it easier to mix other stabilizers with the soil.

4. Pozzolanic strength gain. Over time, pozzolanic reactions between clay minerals and lime may develop calcium-silicate-hydrate and calcium-aluminate-hydrate cementing compounds. A clay soil is considered pozzolanically reactive if the soil's unconfined compressive strength increases by 50 psi in 28 days when mixed with lime. The response of any specific soil-to-lime stabilization varies with the specific soil chemistry and clay mineralogy. Clayey soils can commonly achieve compressive strengths of 100 to 400 psi when stabilized with lime. To achieve pozzolanic strength, the lime-stabilized soil must be moist cured. Typically strength is assessed at 28 days or longer.

Lime stabilization is suitable for clayey soils that have clay minerals to respond to cation exchange, particle agglomeration, and pozzolanic reactions. Generally, a plasticity index of 10 or higher is sufficient for a soil to be a candidate for lime stabilization. Organics in the soil will interfere with the lime stabilization reactions, and some highly weathered soils may require a somewhat higher dosage of lime to achieve good stabilization. Quicklime can dry any soil.

Lime stabilization:

Lime works better in clayey type materials where a pozzolanic reaction can take place and provide increased strength. Some amount of time is needed for the lime to react with the material and provide strength gain.

3.4.2 Portland-Cement Stabilization

Stabilization with portland cement relies on the hydration of the cement to bind the individual soil particles into solid mass with improved strength. Depending on the soil characteristics, the amount of cement used, and cure time, compressive strength may range from a few hundred psi to several thousand psi. Generally, coarse-grained soils gain more strength than fine-grained soils. Shrinkage cracks form in cement-stabilized soils with coarse-grained soils, tending to have wider cracks with greater spacing than fine-grained soils. If the cement content is controlled at a low level, the soil can be modified with cement without the problem of shrinkage cracks, but in this case, the compressive strength will not be as high.

Sandy and gravelly soils with a distribution of particle sizes to include some fines are probably the most economical soils for cement stabilization. Very fine or gap-graded soils tend to require a large amount of cement, and very plastic clays may prove difficult to get adequate mixing. Organic soils and acidic fine sands do not usually respond well to cement stabilization.

Portland cement contains free lime, so some of the same reactions that occur for lime stabilization will also occur with cement stabilization. However, these are minor secondary beneficial effects compared with the other cement hydration effects. If one needs to achieve some of the effects of lime stabilization (e.g., cation exchange to reduce plasticity), it is more economical to use lime than to try to accomplish these objectives with portland cement.

3.4.3 Bituminous Stabilization

Bituminous stabilization coats the individual soil particles, which waterproofs the soil to a degree and binds the particles into a cohesive mass. Typically, for subgrade stabilization, emulsified or cutback liquid asphalts are mixed in-situ with the soils to be stabilized. The emulsion must break and the water must evaporate or drain, or the cutback solvent must evaporate for the benefits of stabilization to become apparent. Bituminous stabilization is most effective with coarse-grained soils. Generally, soils with a plasticity index greater than 6 and with fines exceeding 12% are poor candidates for bituminous stabilization. Some soil and aggregate particles have a strong affinity for water (hydrophilic aggregates) and are very difficult to coat with liquid asphalts. Adding lime or proprietary antistrip compound, or changing the charge of the emulsified asphalt (anionic or cationic) will often overcome this problem.

3.4.4 Fly Ash and Ground Granulated Blast Furnace Slag Stabilization

Fly ash and ground granulated blast furnace slag are waste products of coal-fired power plants and iron ore, respectively. These materials are normally not cementitious on their own. However, when combined with small quantities of lime or portland cement, these will develop pozzolanic cementing bonds. They typically will develop strength ranges intermediate between lime-stabilized clayey soils and portland cement-stabilized materials. Since they are waste products, they tend to be economical, and their slower strength gains compared to portland cement usually mean lower shrinkage and reduced shrinkage cracking issues. Some Class C fly ashes have sufficient free lime that they may be self-cementing and, depending on the CaO content, may also be effective for soil drying. These pozzolanic materials generally offer little advantage over lime for stabilizing clayey soils as the soil clay minerals are naturally pozzolanic themselves. Consequently, they are often particularly effective with more granular soils, and sometimes an upper limit of 15% to 30% fines is specified. These materials are potentially quite effective with gap-graded and nonplastic fine materials. Gap-graded materials, as well as fine sands and perhaps nonplastic silts, would require a large quantity of portland cement to stabilize effectively. The more economical fly ash and slag could serve as filler for the gap-graded material and would be economically feasible to add in sufficient quantity to coat the small particles in fine sands and silts. Fly ash has been used for stabilization in the United States for decades, but stabilization with ground granulated blast furnace slag is only now starting to see more use.

3.4.5 Proprietary Stabilizers

A wide variety of commercial products are marketed as soil stabilizers. These include a variety of acids, salts, lignins, resins, emulsions, cation exchange products, electrolytes, enzymes, and polymers, among others. Technically, sound information on these products is often scarce, and their performance varies widely. Marketing information on these products should always be verified by independent testing and never accepted at face value. Each potential stabilizer should be tested and evaluated for the specific application in mind. For instance, electrolytes depress the ability of plastic clay mineral to hold water. Such a product would be a candidate to deal with expansive soils, but it would offer little value if large increases in soil strength were needed.

3.4.6 Mechanical Methods

A variety of mechanical methods have developed that may aid in stabilization of soils with mechanical rather than chemical means. Blending an unsatisfactory material with a better material is an old and often-used technique. For instance, in some coastal areas, fine sands that are unstable may have oyster shells mixed with them to give a working platform to support traffic. Geotextiles are widely used to reinforce soils and to provide a separating medium to keep fine material from clogging overlying granular materials. In very soft materials, full or partial replacement may be needed, or more extensive site improvement may be needed using surcharge and drains, stone columns, specialized grouting, and deep mixing techniques, etc.

3.4.7 Stabilization Construction and Durability Issues

Stabilization is not a substitute for poor construction. Stabilized material must be thoroughly mixed with the soil, compacted, and generally must be cured in a favorable temperature regime. Failure to accomplish these fundamental steps will result in poor results.

Adequate mixing is crucial. Stabilized material can be mixed either in-situ or at a central plant. Central plant mixing provides a more consistent product, but for subgrade stabilization in-situ mixing is most common. Equipment is available that can provide good in-situ mixing, but quality-control programs should periodically check to ensure that the mixing is thorough.

Compaction must also be carried out to ensure that the desired properties are achieved. Essentially, conventional compaction equipment, compaction curves, and specifications are used for stabilized materials as with unstabilized materials. The addition of the stabilizing agent will change the compaction curve, so tests should be run on the soil and stabilizer combinations. However, there are several additional considerations. Depending on the stabilizing agent, there may be important time limitations on operations. For cement-stabilized materials, the compaction should generally be finished within 2 hours of the portland cement and water or moist soil coming in contact with each other. Fly ashes and ground granulated blast furnace slags usually have a little more time than portland cement before they need to be compacted, but this varies between different fly ashes and slags as their properties and characteristics can vary significantly. Lime, on the other hand, can wait for a day or more before compaction is needed. For liquid asphalts, some aeration may be needed prior to compaction to encourage emulsions to break and solvents to evaporate.

Curing is also critical to portland cement, lime, fly ashes, and slags. Moisture must be retained in these materials for the needed chemical reactions to occur. Usually, simply spraying the compacted stabilized surface with an asphalt emulsion is sufficient, but any technique that keeps the stabilized material moist during curing is adequate.

Durability to cycles of freezing and thawing is a major concern for stabilized materials in seasonal frost areas. The military mandates specific laboratory freezing and thawing tests as part of the mixture proportioning for stabilized soils. The amount of additive needed may be controlled by the requirement to pass these durability requirements, rather than to meet a specific strength. Other organizations specify a minimum strength (which varies from 500 to 1,000 psi) that the materials must exceed to be considered durable to cycles of freezing and thawing. This approach does not guarantee the material will be durable but instead relies on the observation that the percentage of samples failing a laboratory durability test decreases as the strength increases.

Temperatures above 50˚F are needed if the cementitious stabilizers are to gain strength properly. Often, it is specified that portland cement-stabilized materials are not to be placed within 7 days of the onset of freezing weather. Similarly, it is often specified that lime, fly ashes, and slags are not to be placed within a month of the onset of freezing weather.

Lime- and portland cement-stabilized soils are susceptible to damage from sulfate attack if sulfates are present in the soil, groundwater, mixing water, or other sources. It is best to avoid stabilization with these materials if sulfates are present. Low alumina content Type II or V cement will not provide protection against sulfate attack for soil stabilization because the alumina needed for the reaction is readily available in clay minerals in the soil.

Bituminous materials may be susceptible to stripping if used in wet conditions. This needs to be addressed in the design stage, and lime or liquid antistrip agents may be needed to combat this problem.

3.4.8 Summary

Subgrade stabilization may offer a variety of benefits ranging from strength improvement to providing a sound all-weather construction platform to combating adverse soil behavior. In selecting a stabilization technology, one should consider the stabilization objective (e.g., strength vs. reduction of expansion of a swelling clay), soil characteristics, placement conditions (e.g., time to potential freezing conditions), and durability issues (e.g., potential for sulfate attack). This step should identify the potential stabilizers that are feasible to accomplish the objective. Then, considerations of economy and availability of mixing and placing equipment can be used to further limit the stabilizer selection. Finally, laboratory testing should determine what quantity of stabilizing agent is needed to meet the project requirements.

3.5 Special Subgrade Soil Issues

Soils are natural products of complex mechanical and chemical weathering and various transport and deposition processes. Hence, they always tend to be variable, and some of these natural soils may require extra precaution when encountered in airport construction. A few special problems with soils are mentioned in the following sections, but the discussion of these complex issues is, of necessity, brief. References for more comprehensive coverage of the topics are provided for each section. Because of the specialized nature of these problems, the assistance of specialists in the field would be advisable.

3.5.1 Expansive Soils

Some clay minerals change volume dramatically when they gain or lose moisture. Soils containing such clay minerals will swell when their moisture content increases and will shrink when they lose moisture. Different sources provide somewhat varying assessments of a soil's potential for expansive soils, but generally any soil classifying as a CH clay under the unified soil classification system (ASTM D 2487) should be considered suspect and subject to further investigation for expansive characteristics.

Methods typically used to reduce the soil's potential for adverse volume changes include some combination of the following:

- Removal of the expansive material and replacement with nonexpansive fill.
- Surcharge of the material to limit its swelling potential.
- Control moisture changes in the material after construction using moisture barriers. (If the moisture content after construction does not change, there will be no volume change.)
- Stabilize the soil with chemicals. Lime is effective with many soils for this purpose, and there are some other specialty stabilizing agents that can help with this problem (e.g., electrolytes or proprietary cation exchange stabilizers). Stabilization alone will not be sufficient when a highly expansive clay is encountered.
- Control soil placement during construction so that the moisture content is unlikely to change after the pavement is placed. This usually requires placing the soil at moisture contents above the laboratory determined optimum values and may also include placement to lower density and use of a sheepsfoot roller to obtain a more dispersed particle distribution. Obviously, after soil placement, the soil must not be allowed to dry out before paving over it. If moisture is absorbed afterward, swelling will occur.
- Flooding to prewet the soil to limit its ability to absorb moisture later.

3.5.2 Organic Soils and Other Difficult Weak Soils

Organic soils, some wet plastic clays, sensitive quick clays, water-logged silts, and loose uncontrolled fills often pose a multitude of engineering problems including high settlements, very low-bearing capacity, lack of stability, and poor construction trafficability. Such deposits require specialized geotechnical engineering to allow the sites to be developed properly. The assistance of specialists in the field is prudent.

3.5.3 Very Soft Reclaimed Land

A number of civil and military airports and airfields have been built on reclaimed soils and dredged spoil materials, and such trends are likely to continue as new land space is needed in densely developed areas. These materials are often in a near-liquid state when initially deposited, and final settlements may be in tens of feet. In such materials, settlement calculations based on conventional one-dimensional consolidation theory widely used in geotechnical engineering is not adequate. More rigorous approaches give better estimates of likely field settlement magnitudes.

3.5.4 Seasonal Frost and Permafrost

Seasonal frost effects on pavement subgrades may result in frost heaving and dramatic loss in strength during the spring thaw. Pavement design may account for this by providing sufficient pavement thickness of nonfrost-susceptible materials to prevent frost penetration into the ground by allowing partial penetration of the frost in the subgrade, or by designing for subgrade weakening during the spring thaw. Frost requires special detailing of culverts, utilities, etc., to minimize adverse effects of differential frost effects. During construction, these seasonal frost requirements and provisions of truly nonfrost-susceptible materials are critically important to ensure proper pavement performance. Permafrost or perennially frozen ground exists in northern sections of Canada and Alaska. Pavements built on such subgrades must be composed of sufficient thicknesses of nonfrost-susceptible materials to prevent melting of the permafrost. This results in very thick and expensive pavement structures, but trying to use thinner structures will result in unsatisfactory performance. Attempts to use insulating layers to help prevent frost penetration into subgrades or to avoid permafrost melting has resulted in slippery ice formation on the pavement surface, so the practice remains largely experimental at present for airfield pavement. Asphalt pavements have been painted white to reduce the depth of melting of permafrost.

3.5.5 Tropical Deposits

The intense weathering and unique depositional environments of the tropics can provide deposits that are unfamiliar to engineers from temperate zones. For example, residual tropical soils developed from igneous bedrock or volcanic ash may be rich in unusual clay minerals. These soils often exist naturally at high moisture content and undergo irreversible changes upon drying. Consequently, oven drying often used for Atterberg limits, grain-size analysis, and compaction characteristics will provide erroneous results for such soils because of this characteristic. In many tropical environments, coral may be the only viable natural aggregate, and a number of military and overseas airfields have routinely used this material as a bound and unbound construction material for pavements.

3.5.6 Collapsible soils

Collapsible soils are prone to sudden decreases in volume when saturated, and the effect is magnified when combined with loading. Such soils often have an open texture, high void ratios, and low density (Bell, Houston, and Rollings). These soils can develop under a variety of depositional environments such as:

1. Loess. These wind-blown silts have a uniform, very open structure in the undisturbed state with particles cemented with calcite, clay, or a combination of these. Upon saturation, this metastable structure is prone to collapse. Loess constitutes about 17 percent of the US's surficial deposits and about 11 percent worldwide.

2. Alluvial soils. Torrential floods may deposit material in an unstable structure that is then cemented by clay binders or evaporate deposits that are vulnerable to solution on wetting. Such deposits are often found in arid and semi-arid regions, particularly in alluvial fans, and recent construction during the urbanization of the Southwestern US has increasingly encountered collapsible soil problems.

3. Residual soils. Leaching of colloidal and soluble materials from residual soils may leave them in an unstable structure prone to collapse on loading. This mechanism has been reported particularly on granitic residual soils on well-drained slopes.

4. Compacted soils. Soils compacted on the dry side of optimum may collapse under load and wetting. This seems to be particularly a problem for sandy soils with 10 to 40 percent clay and such failures have been reported in embankments, dams, fills, and foundations.

5. Soluble soils. Soils high in soluble minerals such as gypsum may see cavities form as these materials dissolve. This is particularly a problem in arid regions where the soil matrix may contain a high proportion of such minerals and where flow is concentrated such as where runoff from shoulders exits or at junctures between asphalt and concrete that are difficult to keep properly sealed.

Geologic assessment of site conditions coupled with field investigations and laboratory testing offer the best potential to identify these soils and their distribution in the project site, and several investigators have proposed identification models based on density, liquid limit, and geologic origin (Bell 2004, Houston et al 2002, Rollings and Rollings 1996). Response of undisturbed samples to wetting in an oedometer (ASTM D 5333) is commonly used as a laboratory evaluation tool for identifying collapse susceptible soils. Once identified, these collapsible soils may be treated by techniques such as ponding, infiltration wells, conventional impact, or dynamic compaction to induce settlement before construction. It is also possible to remove and replace the poor material if in shallow deposits, or to provide grouting or stabilization technology if discovered after construction.

4. SUBBASE AND BASE COURSE CONSTRUCTION

4.1 Introduction

Flexible airport pavements generally consist of hot mix asphalt (HMA) layers, graded crushed aggregate base, and granular subbase supported by subgrade soils. The granular base and subbase layers generally comprise the bulk of the flexible airfield pavement thickness. The total thickness of the base and subbase courses for airfield pavements can typically range between 12 in. and 48 in., depending on subgrade soil conditions and the types and volume of aircraft loadings. A typical flexible airport pavement structure is shown in figure 2.1.

4.1.1 Base Course

The base course layer is the primary structural component of a flexible airport pavement. The major function of the base course layer is to distribute the imposed wheel loads to the subbase layer and subgrade soils. The thickness of the base layer and the quality of the base materials must be adequate to prevent failure in the subgrade, withstand high shear stresses, and resist consolidation and densification. Base course materials are typically composed of select hard and durable aggregates that produce CBR values ranging between 80 and 100. The quality of the base course depends on the aggregate gradation and physical properties and the field compaction level.

Base course specifications covering the quality of the materials, construction methods, testing requirements, and acceptance criteria for airport paving projects are provided in FAA and DoD (UFGS specifications) documents.

4.1.2 Subbase Course

The subbase course is a granular layer that is constructed between the base course and subgrade soils. The material requirements for the subbase materials are not as strict as for the base course because this layer is subjected to lower stresses due to its depth below the pavement surface. Subbase materials typically produce CBR values ranging between 20 and 30.

Subbase course specifications covering the quality of the materials, construction methods, testing requirements, and acceptance criteria for airport paving projects are provided in the appropriate FAA and DoD documents.

4.2 Base Course Materials

For typical commercial airports and military airfields, the requirements outlined in Item P-209 and UFGS 32 11 24 produce a high-quality base material that produces a CBR value of 100. The recommended gradations for high-quality base materials for the FAA and DoD projects are presented in tables 4.1 and 4.2.

Table 4.1 – Base course requirements in FAA P-209

FAA Base Course Aggregate Item P-209	
Sieve Size	Design Range
2 in.	100
1 ½ in.	95-100
1 in.	70-95
¾ in.	55-85
No. 4	30-60
No. 30	12-30
No. 200	0-5

Table 4.2 – DoD base course requirements

DoD Base Course Aggregate UFGS 32 11 24			
Sieve Size	Gradation 1	Gradation 2	Gradation 3
2 in.	100		
1 ½ in.	70-100	100	
1 in.	45-80	60-100	100
¾ in.	30-60	30-65	40-70
No. 4	20-50	20-50	20-50
No. 10	15-40	15-40	15-40
No. 40	5-25	5-25	5-25
No. 200	0-10	0-10	0-10

The following material properties are typically required for base course materials used for airfield pavements:

- Clean, sound, and durable particles.
- Free of organic matter, debris, clayballs, and any objectionable materials.
- Crushed particles with at least 90% with two or more fractured faces and 100% with one or more fractured faces.
- No more than 15% flat or elongated particles.
- Percent wear by Los Angeles abrasion should not exceed 45.
- Sodium soundness loss should not be greater than 12.
- Material passing the No. 40 sieve should have a liquid limit of not more than 25 and a plasticity index less than 6.

4.3 Subbase Materials

For typical commercial airports and military airfields, the requirements outlined in Item P-154 and UFGS 32 11 16 produce subbase materials that have typical CBR values between 20 and 30. The recommended gradations for airport pavement subbase materials for FAA and DoD projects are presented in tables 4.3 and 4.4.

Table 4.3 – Subbase requirements for FAA P-154

FAA Subbase Course Aggregate Item P-154	
Sieve Size	Percent Passing
3 in.	100
No. 10	20-100
No. 40	5-60
No. 200	0-8

Table 4.4 – Subbase course requirements for DoD

DoD Base Course Aggregate UFGS 32 11 16			
Sieve Size	Gradation 1	Gradation 2	Gradation 3
2 in.	100	100	100
No. 10	50	80	100
No. 200	8	8	8

The following material properties are typically required for subbase course materials used for airfield pavements:

- Hard and durable particles.
- Percent wear should not exceed 50.
- Material passing the No. 40 sieve should have a maximum liquid limit of 25 and a plasticity index less than 6.
- Maximum amount of material finer than the 0.02 mm should be less than 3%.

4.4 Base and Subbase Course Construction

The first step in subbase and base course construction is to ensure an acceptable gradation. The components for base course will need to be stockpiled separately to help minimize segregation. If the components for the base course are placed in one stockpile, segregation will certainly be an issue. There is a wide range in gradation of subbase materials, so each project will have to be evaluated separately to determine the best way to provide a consistent product.

The various components must be blended in a mixing plant, and the correct amount of moisture must be added. Depending on weather, haul distance, etc., the amount of moisture may be decreased with time, so adding some extra moisture may be considered.

While the subbase may be spread in a number of ways depending on the material, the base course should be placed with a mechanical spreader. An asphalt paver is sometimes used very successfully for this operation. The asphalt paver typically provides better results since it can place a more uniform surface with less segregation of the aggregates. Moving base course material around with a grader or pushing the material in any other way will almost certainly result in significant segregation.

Once the material is in place, it should be compacted to meet the density requirements. The loose thickness will need to be significantly thicker than the compacted thickness in order to have sufficient material in place to provide proper density when compacted. The amount of rolldown for a 6-in. compacted layer will likely be somewhere between 1 and 2 in. This means that the loose thickness will have to be between 7 and 8 in. thick when placed to end up with a 6-in. thick compacted layer. This compaction is typically done with vibratory rollers for granular materials, such as that used in the base course. Other types or rollers such as rubber-tire rollers and static steel-wheel rollers can be used if needed.

Rolldown of subbase and base:

The base and subbase will decrease in thickness during compaction due to rolldown. It is typical for these materials to roll down approximately 20%, so approximately 7.5 in. will have to be placed to end up with 6 in. of compacted material. If this rolldown is not properly accounted for, the thickness can be in error.

The compacted layer will need to be checked for thickness, smoothness, segregation issues, and density. Typically, sand cone tests and nuclear gauges are used to check the density. If the density is insufficient, look at moisture content, rolling pattern, and condition of rollers to determine what may be the problem.

Important phases of base and subbase course construction:

- Prior to base and subbase course placement, the underlying layer should be stable. Any unstable areas should be repaired.
- Crushed aggregate base should be placed with a mechanical spreader.
- Subbase materials can be dumped from truck tailgates and spread with a motor grader; however, care should be taken to minimize segregation.
- Base and subbase course placement should begin along the centerline or high point to maintain drainage.
- Maximum compacted thickness should be 6 in. for the base course layer and 8 in. for the subbase layer; minimum compacted thickness for base and subbase layers should be 3 in.
- Moisture content of base and subbase course materials during placement and compaction should be within 2 percentage points of the optimum moisture content.
- Base and subbase course layers should be compacted to at least 100% of the referenced laboratory density.
- Finished base course surface should not vary more than 3/8 in. when tested using a 16-foot straight edge.
- Finished subbase course surface should not vary more than 1/2 in. when tested using a 16-foot straight edge.
- Completed base and subbase course thickness should be within 1/2 in. of the design thickness.

4.5 Testing

The contractor is responsible for quality-control (QC) testing, which is used for process control. The owner/agency is responsible for acceptance (QA) testing, which is used to determine pay factors and adjustments. Aggregate samples should be obtained from the source prior to production and delivery and during construction to evaluate the material properties and construction procedures.

A moisture density curve will have to be developed to determine the optimum moisture content. The optimum moisture content is that moisture content that provides maximum dry density. ASTM D1557 should be used for the compactive effort in developing the moisture density curve.

During production, in-place density testing will have to be accomplished. If density is insufficient, the method of compaction will have to be modified to provide adequate in-place density.

When gradation testing is performed on the in-place material, it will be finer than that determined on stockpile materials since some amount of breakdown will occur during the rolling process.

Important elements of the testing include:

- Obtain random samples in accordance with ASTM D75.
- Perform gradation tests in accordance with ASTM C136 and C117.
- Determine moisture-density relationship in accordance with ASTM D1557.
- Determine in-place density in accordance with ASTM D1556 or ASTM D2922/D3017.
- Measure thickness of base course using a 3 in. diameter test hole.
- Evaluate surface smoothness with a straight edge.

4.6 Proofrolling

Proofrolling aggregate base courses of flexible airfield pavements with a heavy rubber-tired roller after required compaction levels have been achieved is used to accomplish two primary objectives:

- To identify any soft or weak spots in the pavement structure prior to placing the hot mix asphalt layers
- To achieve additional compaction above that achieved by typical compaction equipment and procedures

Proofrolling is very beneficial when this procedure detects weak areas in the pavement structure. Proofrolling can avoid costly repairs and reduce the potential for aircraft operation delays.

Proofrolling of aggregate base courses of flexible airfield pavements should be performed with a heavy rubber-tired roller such as the one shown in figure 4.1. The required proofroller has four tires abreast with a gross load of 60 tons. Each tire should be loaded to 30,000 pounds or more with a tire inflation pressure of 125 psi. For heavily loaded airfield pavements, proofrolling should consist of 30 coverages with a coverage defined as one application of one loaded tire print over the entire surface.

Figure 4.1 – Typical airfield proofroller

During proofrolling, an inspector needs to be present to observe the deflection (if any) under the roller and to evaluate the stability of the base course layer.

Inspector checklist:

The following are several items that the construction inspector should check prior to and during airfield construction activities:

- Verify compacted subgrade soils are acceptable and ready to support placement and compaction of granular materials.
- Verify subgrade elevations and slopes.
- Verify source approval tests for granular materials meet specification requirements.
- Monitor moisture content of granular materials.
- Observe placement and spreading procedures to reduce segregation potential.
- Check granular layer thickness.
- Verify field compaction results comply with specification.
- Evaluate smoothness to ensure specification requirements are satisfied.
- Final grades and elevations should be determined and compared to project plans.

The following criteria may be used as a guide to evaluate the proofrolling:

- Rutting less than 0.25 in.—base layer is acceptable.
- Rutting between 0.25 in. and 1.5 in.—base course should be scarified and recompacted.
- Rutting greater than 1.5 in.—remove and replace base course material.
- Deformation greater than 0.5 in. that rebounds—indicates instability within the structure or subgrade.
- Elastic movement with significant cracking or lateral movement indicates instability.

Troubleshooting guide for base and subbase materials:

Problem	Probable Cause	Corrective Action
Surface appears loose	Low density Layer too thin/thick for compaction Insufficient rolling Low fines content	Check moisture content and density. Recondition to optimum moisture and recompact area. Adjust gradation.
Depressions or excessive movement under roller	High moisture content Weak layer under base or subbase Low fines content	Check moisture content and recondition to optimum moisture if high. Probe to find weak layer. Stabilize area. Adjust gradation.
Bird baths	Improper grade control	Perform grade survey and correct deficient areas.
Segregation	Improper stockpiling Improper trucking Operations Poor placement/handling procedures	Ensure that stockpiles are maintained properly. Use good loading and unloading procedures. Place with mechanical spreader.

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5. RUBBLIZATION

5.1 Background

Rehabilitation of old portland cement concrete pavement consistently encounters problems with reflective cracking if the surface is overlaid with asphalt concrete and in some cases with portland cement concrete. Rubblization fractures the existing portland cement concrete pavement into small fragments. The existing rigid concrete pavement is broken into a stiff granular material that now distributes load more like a granular base course than as a rigid pavement. The older large concrete slabs that once provided the movement to cause reflective cracks are now shattered into essentially granular material, and reflective cracking will not now develop in the new asphalt concrete overlay surface.

The use of rubblized concrete pavement as a base course for asphalt construction has been performed on a number of projects. In this case, the rubblized concrete acts as a base course for the asphalt pavement so the rubblization process is one form of base course construction. This process has been shown to provide a completed pavement that does not develop reflective cracking.

AAPTP 04-01, *Development of Guidelines for Rubblization*, provides a complete discussion of rubblization and guidelines for use of this procedure. This section only provides some of the findings that would be useful during construction of asphalt pavement over a rubblized concrete pavement.

> Rubblization is often not very effective when the foundation underneath the concrete is poor.

5.2 General Facts about Rubblization

Rubblized pavement generally provides an effective cracked modulus of elasticity with a value of 100,000 to 400,000 psi, which is greater than that seen for conventional high-quality aggregate base courses. Thicker pavements, pavements with higher initial modulus values, and pavements with reinforcing steel tend to produce higher modulus values. Repeated passes of the rubblization equipment increase the amount of fracturing and decrease the effective modulus of the rubblized concrete. There is mixed guidance about using rubblization with concrete experiencing alkali-silica reaction. The AAPTP 04-01 study indicated that there is no problem; however, an International Pavement Research Foundation (IPRF) study and a United States Air Force (USAF) Engineering Technical Letter indicate that an assessment is needed to evaluate the risks versus benefits for this technique when using concrete that has experienced alkali-silica reaction.

Rubblization is generally an effective method of rehabilitating a deteriorating portland cement concrete pavement and avoiding problems with reflective cracking in asphalt overlays.

5.3 Procedures

Both the resonant pavement breaker (figure 5.1) and the multihead breaker (figure 5.2) have proven effective for rubblizing concrete pavements, and they seem to share the market. Both machines should be allowed in the project specifications. Costs for rubblizing tend to be lower if either type of equipment is allowed than if only one machine is allowed in the project specifications. Conventional demolition equipment such as headache balls or impact rollers should not be used for rubblization as they are specifically designed for demolition work rather than rubblization.

Figure 5.1 – Resonant pavement breaker (courtesy of Asphalt Institute)

Figure 5.2 – Multiple head breaker (courtesy of Asphalt Institute)

In general, edge drains should be installed along the edge of airfield pavement to be rubblized, and any structures that need to be protected (e.g., utilities, drainage structures, lighting fixtures, or adjacent pavement) should be isolated with full-depth sawcuts adjacent to the structures.

A test section should be constructed at the start of construction to allow the contractor to demonstrate the adequacy of the proposed equipment and procedures for the project site conditions. Lack of good drainage can prevent proper breaking of the old concrete during the rubblization process. A test pit should be excavated to evaluate the adequacy of the rubblization. Detailed recommendations on test pits are contained in the full AAPTP report.

Test pits should verify that all specification requirements are met and should specifically examine that:

- Fracture occurs through the full depth of the PCC slabs.
- Steel is "substantially" debonded from concrete; dowels may be sawed at the joint when substantial debonding cannot be achieved.
- Particle size requirements are met.
- No contamination of subgrade fines into rubblized layer exists.
- No shear distortion of the subgrade exists.

In general, the target rubblization particle size objective for the bottom half of the layer should be that there are no particles greater than 6 in. in any dimension, and at least 75% of material (by weight) is less than 3 in. in any dimension. For the bottom half of the layer, or below the steel, there should be no particles greater than 2 times the slab thickness in any direction.

Future concepts for improving quality-control/assurance practices include utilizing either deflection testing or intelligent compaction to identify potential unrubblized areas, finding localized weak spots, and monitoring the uniformity of the rubblized layer.

If the test pit is located along the periphery of the pavement, so there is a lack of confinement at the edges, some poorly fractured material may result along the outside edge. These over-sized edge pieces should not be a cause for rejection.

If the rubblized pavement receives a direct asphalt concrete overlay, this asphalt concrete should be a minimum of 5 in. thick. The minimum asphalt concrete thickness should be placed in two lifts with the first lift at least 3 in. thick (compacted thickness) to ensure density is achieved. If an unbound granular layer is placed directly on the rubblized concrete, it should have a minimum compacted thickness of 4 in., and the agency minimum thickness of asphalt concrete over unbound bases will apply.

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6. DISCUSSION OF DENSE-GRADED MIX, SMA, AND OPEN-GRADED FRICTION COURSE

6.1. Introduction

Several mix types are used in the construction of hot mix asphalt (HMA). Each of these mix types has different characteristics, and the type should be selected based on the specific project. A discussion of the various mixture types is provided in this section. The three most common mixtures are dense-graded mix, stone matrix asphalt (SMA), and open-graded friction course. The primary difference between these three mixture types is the gradation of the aggregate used. The dense-graded mixture is designed for dense aggregate packing, which results in a somewhat lower optimum asphalt content and good mixture stability. The dense-graded mixture will typically have approximately 40% coarse aggregate, 55% fine aggregate larger than the No. 200 sieve, and 5% material passing the No. 200 sieve. The SMA mixture is a gap-graded mixture, which typically contains approximately 70% coarse aggregate, 20% fine aggregate larger than the No. 200 sieve, and 10% material passing the No. 200 sieve. This is called a gap-graded mixture due to the large amount of coarse aggregate, high filler content, and low percent of fine aggregate. The open-graded friction course typically has about 80% coarse aggregate, 15% to 20% fine aggregate, and approximately 2% passing the No. 200 sieve.

6.2. Dense-Graded Mixtures

By far the most common mixture used on airfield pavements is dense-graded mix. Almost all airfield work constructed in the past has utilized dense-graded mix. If a core is cut through a dense-graded mixture, one can see that the coarse aggregate is floating in the fine aggregate matrix (figure 6.1). There is little stone-on-stone contact between the coarse aggregate particles. In the dense-graded mixtures, the fine aggregate generally carries the load making it very important that the fine aggregate be of good quality (e.g., angular, minimum amount of natural sand, hard) otherwise shear failure (rutting) may occur quickly in the mixture when subjected to traffic.

Figure 6.1 – Dense-graded mixture

Dense-graded mixtures typically have reasonably good workability and good pavement life, making dense-graded mixtures ideal for most HMA work. These mixes have low permeability when properly compacted. They are also resistant to rutting as long as the voids are not overfilled with asphalt binder and as long as a high-quality crushed aggregate is used.

6.3 SMA Mixtures

SMA is a gap-graded mixture having stone-on-stone contact. This stone-on-stone contact provides good stability, resulting in a mixture that has good resistance to rutting. This mixture was first used in Europe to resist abrasion from studded tires. Performance studies have shown that SMA mixtures are resistant to rutting and also provide good durability. The stone-on-stone contact, high filler content, and stiff asphalt binder provide the rutting resistance, while the high asphalt content provides good durability characteristics.

If a core was cut through an SMA mixture one could see the large coarse aggregate content and the small amount of fine aggregate floating in the coarse aggregate matrix. A cross section of an SMA mixture is provided in figure 6.2.

Figure 6.2 – Stone matrix asphalt mixture

6.4 Open-Graded Friction Course (also known as porous friction course)

Open-graded friction courses have been used for a number of years to provide good drainage of the water from the surface during heavy rains, thus, greatly reducing the chances for hydroplaning. The mix is typically placed approximately 1-in. thick, and the water actually drains down into the pavement and out the side of the pavement. The report for AAPTP 04-06, which evaluates the potential for using porous friction courses, has been completed and can be downloaded from aaptp.us. Many state Departments of Transportation (DOTs) use these mixtures on their high-volume roads such as interstates and other high-speed, high-volume roadways. Back in the 1970s and 1980s, these mixtures were used on airfields, but due to some performance issues they are not widely used today. The biggest performance problem with these open-graded friction courses was loss of aggregate due to raveling resulting in foreign object damage (FOD) potential. Now that modified asphalts are more widely used, these binders should be capable of minimizing any loss of aggregate that may occur during the life of the porous friction course.

If one cut a core through an open graded friction course, one could see the high coarse aggregate content and the high amount of voids in between the aggregate particles. A cross section of a core taken from an open graded friction course is provided in figure 6.3.

Figure 6.3 – Open-graded friction course

All three of these mixture types present some advantages over the other types of mix and should be selected for their appropriate application. The specifications will clearly indicate which of these mixtures is specified. In almost all cases, a dense-graded mixture, as shown in figure 6.1, is used for airfield work.

Select mix that best fits the project. Most projects use dense-graded mixtures. Three types of mixes used are:

- Dense-graded mixtures
- Stone matrix asphalt (SMA)
- Open-graded friction course or porous friction course (OGFC or PFC)

7. HOT MIX ASPHALT MATERIALS SELECTION AND MIX DESIGN

7.1 Introduction

The properties of HMA are usually defined by the binder grade or stiffness, aggregate properties, nominal maximum aggregate size, aggregate gradation, asphalt content, and volumetric properties of laboratory compacted specimens. Controlling these material and mixture properties will help to ensure that good performance is obtained. Construction properties such as in-place density, joint density, and uniformity of the in-place pavement also have a significant effect on performance.

Asphalt binder is the "glue" that holds the asphalt mixture together. The asphalt content needs to be optimized to balance stability, or resistance to rutting and shoving, with long-term durability in terms of raveling, cracking, and resistance to moisture. Generally, airfield pavements tend to experience a higher percentage of durability problems than stability problems. Stability problems such as rutting or shoving are more likely to occur in warm weather when the aircraft tire pressures are high, such as with some military aircraft, and when aircraft are moving slowly. Rutting and shoving occur more often on taxiways where traffic may be turning and moving slowly or where aircraft stack near the end of runways.

There are four steps to the mix design process:

1. Asphalt cement and aggregate selection and evaluation
2. Selection of aggregate gradation
3. Determination of optimum asphalt content
4. Evaluation of moisture sensitivity

Two mix design procedures will be discussed: Marshall and Superpave. Both mix design systems generally share the same four steps. Currently, the Marshall method is the accepted design practice for airfields, but there are cases where Superpave mix design is used. The Superpave mix design system has been used on a widespread basis for highway pavements since 2000. Superpave has been adopted by FAA as an alternative mix design procedure for airfields. This document is provided in FAA document EB 59A. The primary difference between the Marshall and Superpave design systems is the laboratory compaction method and effort used in the determination of the optimum asphalt content.

Materials selection consists of selecting the appropriate asphalt binder and coarse and fine aggregates. Once the component coarse and fine aggregates are selected, the selected gradation blend must be tested to determine the optimum asphalt content. Finally, the moisture susceptibility of the mixture should be tested. Materials selection and asphalt mix design considerations are discussed in this section. Specific information on test procedures or step-by-step directions on how to perform a mix design are not included. Appropriate guides for this purpose are referenced.

Some items that should be included in a mix design report:

- Percent passing each sieve
- Optimum AC content
- Material proportions
- AC grade
- Compactive effort—number of blows with hammer or number of gyrations with gyratory compactor
- Lab mixing temperature
- Lab compaction temperature
- Temperature-viscosity relationship for asphalt binder
- Plot of gradation
- Plot of mix properties versus asphalt content
- Specific gravity and absorption of each aggregate
- Percent natural sand
- Fractured face count of coarse aggregate
- Fine aggregate angularity
- Flat and elongated count
- Tensile strength ratio
- Antistrip agent if added
- List modifiers and amount

7.2 Material Selection

This section provides guidance on material properties and their relationship to performance.

7.2.1 Aggregates

Aggregates used in HMA may consist of crushed stone (quarried material), gravel, or slag; natural sand; and mineral filler. Aggregate retained on the No. 4 (4.75 mm) sieve is termed *coarse aggregate*, and aggregate passing the No. 4 (4.75 mm) sieve is termed *fine aggregate.* Material passing the No. 200 (0.075 mm) sieve typically comes solely from the coarse and fine aggregate stockpiles, but in some cases mineral filler may be added as a separate material.

Identification and Importance of Aggregate Tests			
Aggregate Requirements	**Test Method**	**Typical Limits for Higher Traffic***	**Importance of Requirement**
LA abrasion	ASTM C131	40% max	Used to prevent breakdown of aggregate primarily during production. Increase in dust due to breakdown is a significant problem.
Sulfate soundness	ASTM C88	12% max for Na 18% max for Mg	Used to measure resistance of aggregate to breakdown due to freeze-thaw action.
Fractured faces	ASTM D5821	75% with 2 or more fractures	Angular aggregate is needed for good shear resistance. This specifies minimum angularity. This is primarily issue for gravels.
Flat and elongated	ASTM D4791	20% max with 3:1 ratio or 8% with 5:1	Used to provide good workability and to minimize pulling and tearing during construction.
Sand equivalent	ASTM D2419	45% min	Used to ensure that the fine aggregate does not have excessive clay-like particles.
Fine aggregate angularity	ASTM D1252	45% min	Used to ensure good angularity in fine aggregate. Can be used to help evaluate shape of natural sands.
Natural sand content	----	15% max	Natural sands are typically rounded and sometimes contaminated with organics and clay particles. Amount used needs to be limited.

*Note: These limits are typical. Actual limits vary depending on specification being used.

7.2.2 Asphalt Binder

Asphalt binder is referred to by a number of names: *asphalt, asphalt cement,* and *asphalt binder*. Asphalt and tar are very different materials. Asphalt is generally a by-product of the distillation (refining) of crude oil, although it can be naturally occurring. The amount of asphalt produced from a given quantity of oil is dependent on the crude source and refining techniques. Asphalt is soluble in petroleum products such as gasoline, jet fuel, and oil. Tar is resistant to petroleum products. Tar is developed when coal is burned and is typically a by-product of the production of iron ore. The properties are much different from asphalt. Tar is generally somewhat fuel resistant and asphalt is not. Tar is sometimes used as a sealer for asphalt pavements but has not been used in the United States to produce mixtures for many years. Hence, the information in this document only includes asphalt materials.

The following sections describe three classification systems that have been used to grade asphalt binders around the world: penetration grading (used in United States up to approximately 1970), viscosity grading (used in United States from early 1970s up to late 1990s), and performance grading (used in United States from late 1990s to present). The Performance-Graded (PG) binder system is presently used throughout the United States.

7.2.2.1 Penetration Grading

The penetration grading system was adopted as ASTM D946. One of the main disadvantages of the penetration system was that it only measured the stiffness of the asphalt at one temperature. The performance of a binder at 77 °F may be deceptive to its performance at higher (summer) or lower (winter) temperatures. Penetration grading is still used in Europe and the Middle East, as well as other locations throughout the world.

7.2.2.2 Viscosity Grading

The viscosity grading system was developed by the Federal Highway Administration (FHWA) and the Asphalt Institute to address problems during construction and at high-surface temperatures under traffic. The viscosity of the binder was measured at two temperatures, 140 and 275 °F (60 and 135 °C). The first was selected to represent typical pavement temperatures on a warm summer day. The second temperature was selected to represent the binder properties near typical mixing and compaction temperatures.

The viscosity grades are specified in ASTM D 3381. Commonly used viscosity or AC grades were AC-10, AC-20, and AC-30. Two criticisms of the viscosity grading system were that it did not provide safeguards against low-temperature cracking, and it was not suitable for modified binders. The viscosity grading system was the most widely used grading system in the United States until the adoption of the Superpave Performance-Graded binder system.

7.2.2.3 Performance-Graded Binder System

In 1994, the PG binder system was developed (*6*). The PG binder system was a product of the Strategic Highway Research Program (SHRP), the same program that developed the Superpave mix design method. The PG binder system is unique in that it provides specifications for the binder over the complete range of temperatures expected during construction and in-service. The PG binder specifications are outlined in AASHTO M320.

The PG system is based on expected high and low pavement temperatures, traffic speed (or loading rate), and traffic volume. As noted previously, "PG" stands for performance grade. The first number represents the average 7-day maximum pavement temperature for which the binder would be resistant to rutting; for example, a PG 64-XX would be expected to be resistant to rutting to a pavement temperature of 64 °C (147 °F) at normal traffic speeds. The second number is the minimum pavement temperature for which the binder would be expected to resist low-temperature cracking; for example, a PG XX-22 would be expected to be resistant to low-temperature cracking to a temperature of -22 °C (-8 °F).

There are three methods to grade asphalt binder:

- Penetration grading
- Viscosity grading
- PG grading (the most common grading system used in the United States)

7.2.2.4 Binder Modification

There is a general rule of thumb for PG binders that if the high- and low-temperature numbers are added (ignoring the minus sign) and the sum exceeds 90, then the binder is likely modified. For example, the sum of the two numbers for a PG 76-22 is 98, which exceeds 90 and means that the binder is likely modified. Grades like PG 70-22 or PG 64-28, with sums of 92, are sometimes produced with no modification, and sometimes produced as modified asphalts.

There are a number of methods for modifying asphalt binders. Some of the most common are:

- Polymers
- Air blowing
- Acid modification

Elastomeric polymers, such as styrene-butadiene di-block (SB), styrene-butadiene-styrene (SBS), or styrene-butadiene-rubber (SBR, also know as latex) are preferred by many agencies. The primary reason for using these polymers is to improve the rutting resistance of the mixture. Elastomeric polymers are believed to improve the fatigue and reflective cracking resistance of the mixture, and, in some cases, improve resistance to moisture damage. There is a lot of debate about which type of elastomer is best, but there is no strong evidence to support one type over another.

Plastomers are another type of polymer modifier. Plastomers generally only stiffen the binder to help resist rutting and do not improve the elastic properties of the binder. For this reason, cracking has been a concern with some plastomers.

Separation can be a concern with some types of polymers. If the polymer is not properly agitated in the tank it can separate from the asphalt binder. Such binders are said to lack storage stability. Some polymers such as natural rubber, recycled tire rubber, and Novaphalt are generally blended on site at the contractor's asphalt plant. It is essential that proper proportions and blending be provided with the polymer being used. It is generally better to have this mixing done by the asphalt supplier and not try to do this during production of the asphalt mixture at the asphalt plant. Be aware that some separation can occur when these polymer modified asphalts are stored. Providing circulation of the binder will typically help to minimize this problem.

Air blowing has been used as a refining technique for many years to stiffen (increase the high temperature grade) binders. Basically, the binder is aged under controlled conditions at the refinery. This technique will result in a stiffening of the binder and improve the resistance of the binder to rutting.

7.2.2.5 Binder Grade Selection

General Considerations. Stiffer binders can improve rutting performance. However, for long-term durability, it is advisable to use the binders with the lower stiffness, at least for surface mixes. Stiffer binders tend to reduce workability and may make it harder to achieve the necessary compaction. Much stiffer binders may also suffer from long-term durability problems in the form of cracking. However, the use of elastic polymer modifiers to obtain these grades may help mitigate this cracking problem.

Selection of Binder Grade. Work is underway to provide detailed procedures for selecting the binder grade for airfield pavements. The specific procedure should be followed as spelled out in latest copy of FAA Advisory circular 150-5370-10.

New recommendations for binder selection for airfields have been developed as part of AAPTP Project 04-02, *PG Binder Grade Selection for Airfield Pavements*. The recommendations primarily affect the selection of the high temperature grade. While the recommendations have been made, they will have to be studied by the various agencies to determine what portion of the study will be adopted. The report can be downloaded at the following Web site: aaptp.us.

The following method has been used, without specific guidance, to select the binder grade for specific airfield paving projects. First, determine the grade of the binder that the state DOT requires in the specific area. (The DOT does a large amount of paving and generally has become familiar with the grades that work best in the local area.) Use this grade as the baseline requirement for the asphalt grade. When the tire pressures of the aircraft exceed 200 psi or in areas that aircraft tend to move slowly or stack, the high temperature grade should be bumped one or two grades higher to ensure the best resistance against rutting. Bumping the grade will almost certainly increase the cost of the binder and the in-place mixture.

Also, there are three other general recommendations to consider: 1) Be cautious when using high temperature grades less than 64 (there has been some evidence to indicate that asphalts with lower grades tend to be tender under compaction and initial traffic resulting in some surface issues), 2) be cautious when using high temperature grades above 76 (some evidence indicates that asphalts with these higher grades tend to be stiff resulting in reduced workability and difficulty in obtaining compaction requirements), and 3) be cautious when using low temperature grade below 22 (for example, be careful when using 16 or 10) (some evidence indicates that these lower grades result in asphalts that have performance issues). This is meant to be general guidance for use when no additional guidance is available. If additional guidance is available concerning the grade of asphalt to be used, the additional guidance should take precedent.

Binder suppliers have a limited number of tanks available to store different binder grades. Many HMA contractors only have two tanks available. Therefore, many state highway agencies have reduced the number of binder grades used in a state and even regionally. In many cases, these reductions were made based on experience with historically used viscosity grades under the PG binder system. Therefore, it is important to consult the state highway agency and potential binder suppliers to determine which grades are readily available.

7.2.2.6 Mixing and Compaction Temperatures

The rotational viscometer is used to determine the mixing and compaction temperature ranges for unmodified binders. Typically, two tests are run at 135 and 165 °C (275 and 325 °F). The results are plotted on a log-log graph of temperature versus viscosity as shown in figure 7.1. The mixing and temperature ranges are then picked off the graph from the recommended viscosity ranges of approximately 0.17 ± 0.02 Pa·s for mixing and 0.28 ± 0.03 Pa·s for compaction. For modified binders, the supplier's recommendations for mixing and compaction temperatures should be used. Research is currently in progress to identify a better method for the determination of mixing and compaction temperatures. Overheating certain polymer modification systems can damage the polymer resulting in a significant change in binder properties.

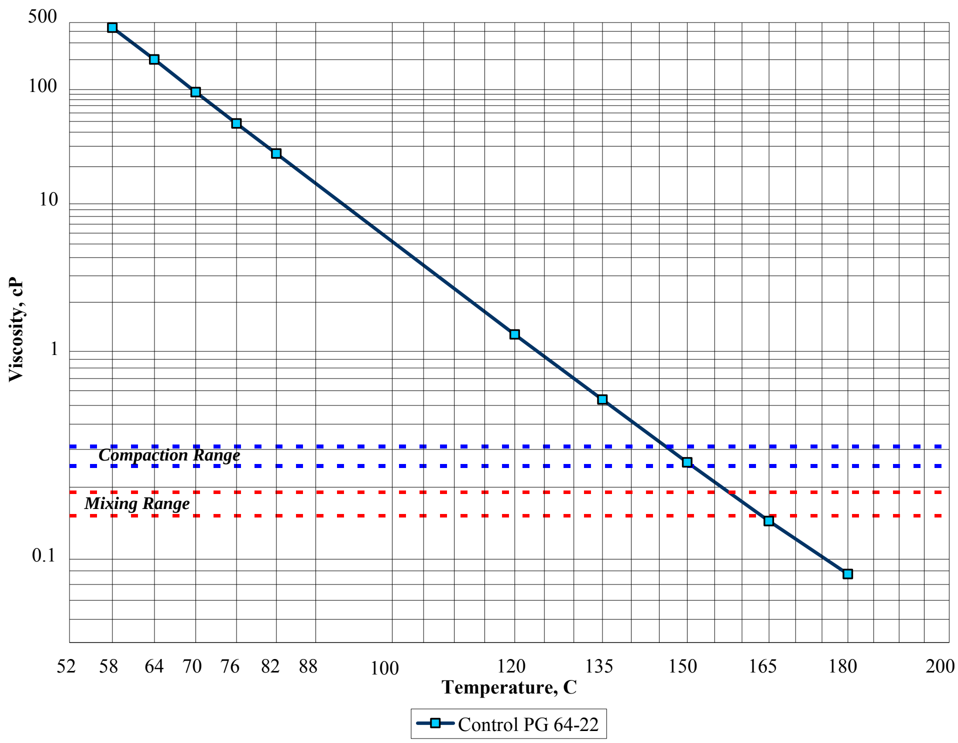

Figure 7.1 – Example mixing and compaction temperature chart from rotational viscometer

7.2.3 Other Additives

7.2.3.1 Mineral Filler

The need to use mineral filler to increase the percentage passing the No. 200 sieve to meet gradation requirements is seldom needed. Generally, the aggregates available for use have more than enough dust in the mix. However, filler in the form of hydrated lime or portland cement is often used to improve the moisture susceptibility properties of the mixture. Typically, baghouse material or material passing the No. 200 (0.075 mm) sieve is added to the mix design batches to account for the expected degree of breakdown during the mix design process.

7.2.3.2 Anti-Stripping Agents

The asphalt binder can strip from the aggregate in the presence of moisture. This phenomenon is termed *stripping* (binder strips or separates from the aggregate) or *moisture damage*. During the mix design process, the tensile strength ratio (TSR) test, ASTM D 4867, is used to assess the potential for moisture damage. This will be discussed in greater detail later in the chapter. If the mixture fails to meet the minimum TSR ratio, then an anti-stripping additive must be added to the mix. Potential anti-stripping agents include hydrated lime (as mentioned above), portland cement, or liquid anti-stripping agents. Hydrated lime and portland cement are most effective when they are mixed with damp aggregate. Liquid anti-stripping agents can be added to the binder either at the asphalt terminal or the asphalt plant. Liquid anti-stripping agents should be heat stable and should not alter the binder grade.

7.2.4 Reclaimed Asphalt Pavement (RAP)

Many projects use reclaimed asphalt pavement (RAP) in the asphalt mixture to help reduce construction cost. RAP tends to have a higher percentage of material passing the No. 200 (0.075 mm) sieve since some aggregate particles are fractured during milling and/or crushing. The amount of dust in the RAP can be a limiting factor in the quantity used in a mix design. AAPTP research project 05-06 addresses recycled mixtures.

There are a number of issues that must be considered when using RAP in a mix design. First, it is important to get a representative sample of the RAP. The asphalt content of the RAP needs to be determined by solvent extraction or the ignition furnace. If the ignition furnace is used, the correction factor for aggregate breakdown is generally not known. If the percentage of RAP in the mixture is small, say less than 10%, it probably does not matter. Also, if the ignition furnace is used to extract the RAP, and the ignition furnace is used to measure the asphalt content during production, any error resulting from aggregate breakdown will be self-correcting as long as the RAP source remains consistent. If the aggregate sources in the area are of a consistent geology, an average of the local correction factors can be used.

The aggregate consensus properties, except the sand equivalent value, can be performed on material recovered using the ignition furnace. The effective specific gravity is recommended for RAP for use in the calculation of volumetric properties.

Another concern when using RAP is the selection of the virgin binder grade. RAP has aged binder, which may exceed PG 76-XX on the high temperature side and may also have stiffened on the low temperature side; for example, what was a XX-22 may become an XX-16. Most agencies allow 15% to 20% RAP in a mix without changing the virgin binder grade. Some agencies limit RAP in surface mixes or when polymer modified binders are specified in surface mixes (such as PG 76-22) due to concerns about cracking or losing the effect of the polymer modifiers in reducing cracking. The binder from the RAP can be recovered and graded if higher RAP percentages are to be used.

Other general considerations during the mix design process when using RAP include:

- During mix design, use oven-dried RAP or determine moisture content.
- Dry RAP at approximately 140 °F (60 °C), typically for at least 2 days.
- Avoid heating RAP too long during mix design process (maximum 1 hour).
- NCHRP 9-12 recommends heating RAP at 230 °F (110 °C) for not more than 2 hours (McDaniel and Anderson, 2001).
- Use same short-term aging process and time as virgin mixes.

Many agencies commonly use a relatively small amount of RAP in their HMA mixtures, generally up to approximately 20%. If the amount of RAP is kept low, it can be used without having to change mix design methods or construction procedures. At higher RAP contents, some changes are likely required in the mix design procedures and possibly in construction procedures. Prior to using RAP, check latest guidance.

7.3 Selection of Design Aggregate Gradation

After it is shown that the aggregates meet the material specifications, the design aggregate gradation should be determined. HMA is typically comprised of a blend of three or more stockpiles. Percentages of the stockpiles used in the mix may be adjusted during production to keep a consistent gradation in the finished HMA.

7.3.1 Gradation Bands

Gradations are plotted on a 0.45 power curve. The x-axis is a logarithmic plot of the sieve size opening, in mm, raised to the 0.45 power. The y-axis is the percent passing the corresponding sieve size. The maximum density line is drawn from the origin to 100% passing the maximum sieve size. The maximum density line is supposed to represent the gradation that would result in the densest packing of the aggregate.

The FAA and Unified specifications include a range of percent passing for each sieve size. The range is somewhat wide to account for regional variations in materials. Superpave only includes a minimum number of control points. Control points include the 0.075 mm (No. 200 sieve), 2.36 mm (No. 8 sieve), and nominal maximum aggregate size (NMAS). The gradation blend for Superpave should pass between the control points. Figure 7.3 illustrates the FAA, Unified, and various components of the Superpave gradation bands.

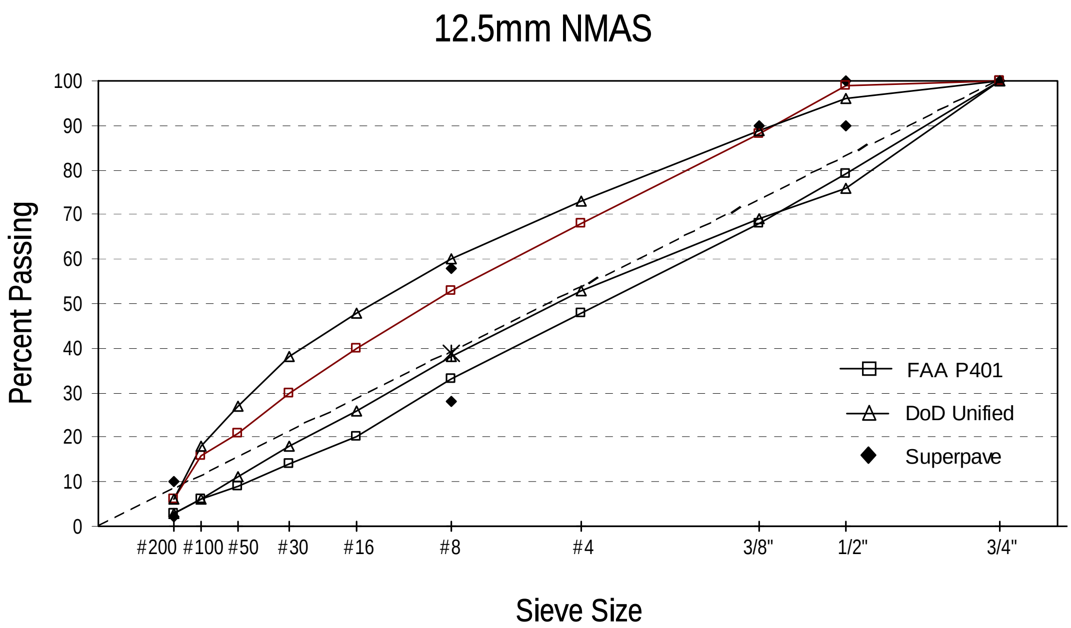

Figure 7.3 – FAA, Unified, and Superpave gradations for ¾-in. maximum aggregate size or ½-in. nominal maximum aggregate size

The maximum density line is shown in figure 7.3 by a dashed line. Gradations that are generally above the maximum density line are fine graded; they have a greater proportion of particles passing the No. 4 (4.75 mm) sieve. Gradations that are generally below the maximum density line are coarse graded; they have a greater proportion of particles retained on the No. 4 (4.75 mm) sieve. Several observations can be made from figure 7.3. The FAA and Unified specifications generally favor fine-graded mixes, and the Unified specifications allow finer mixes than the FAA specifications.

There are several good reasons for specifying gradations close to or finer than the maximum density lines for the surface layer of an airfield, including:

- Finer graded mixes are less permeable to water.
- Finer graded mixes are less likely to ravel and therefore less likely to produce FOD.
- Finer graded mixes are more workable and less likely to segregate.

7.3.2 Selecting a Trial Gradation

The first step in determining a trial gradation is to determine the stockpile gradations. Samples can be taken from a number of points around the stockpile, blended, split, and gradation analyses performed. The gradation of RAP should be determined on material extracted using solvents or the ignition furnace.

Most plants cannot uniformly feed less than 5% to 10% of a given material, so 5% to 10% would generally be the minimum amount of any one material that should be used. The exception is mineral filler, including hydrated lime, since it will be fed from a mineral filler silo if used.

7.3.2.1 Preparing Samples for Laboratory Analysis

When performing a mix design, it is important to account for the expected breakdown of the aggregate going through the plant. Ideally, the expected breakdown would be known for a given aggregate source based on experience. If not, the addition of 1% passing the 0.075 mm (No. 200) sieve is a reasonable starting point. The additional dust can be collected from the asphalt plant's baghouse or by sieving the fine aggregate.

There are a number of methods to batch aggregate samples for mix design. For large samples (e.g., more than 5,000 g), the aggregates can be dumped into a large flat pan and "bulk batched" using a flat bottom scoop. It is important that the scoop is run along the bottom of the pan. Smaller samples, such as those used in Marshall mix design, require fractionation. The recommended method is to fractionate each aggregate source or stockpile individually. It is necessary to run a washed gradation on a sample batched in the same manner as the samples prepared for mix design to determine that the gradation of the batched samples matches the design gradation.

Most common problems with aggregates during mix design and construction include:

- Excessive amount of natural sand used
- Excessive amount of minus 200 material in mixture
- Inadequately crushed coarse aggregate when using mixtures produced with gravels
- Lack of uniformity (segregation, contamination, etc)

7.4 Importance of Binder Content

The asphalt binder is the glue that holds the aggregate particles together. The binder content of a mixture is critical for performance. For long-term durability, including resistance to aging, cracking, raveling, and moisture, it is desirable to have the highest asphalt content possible without sacrificing stability. This is expressed graphically in figure 7.4. Typically, mixtures are designed for stability since stability-related failures tend to occur earlier in the service life of the pavement. Binder content is controlled mainly by the air voids in the mixture and the compactive effort used to compact lab samples.

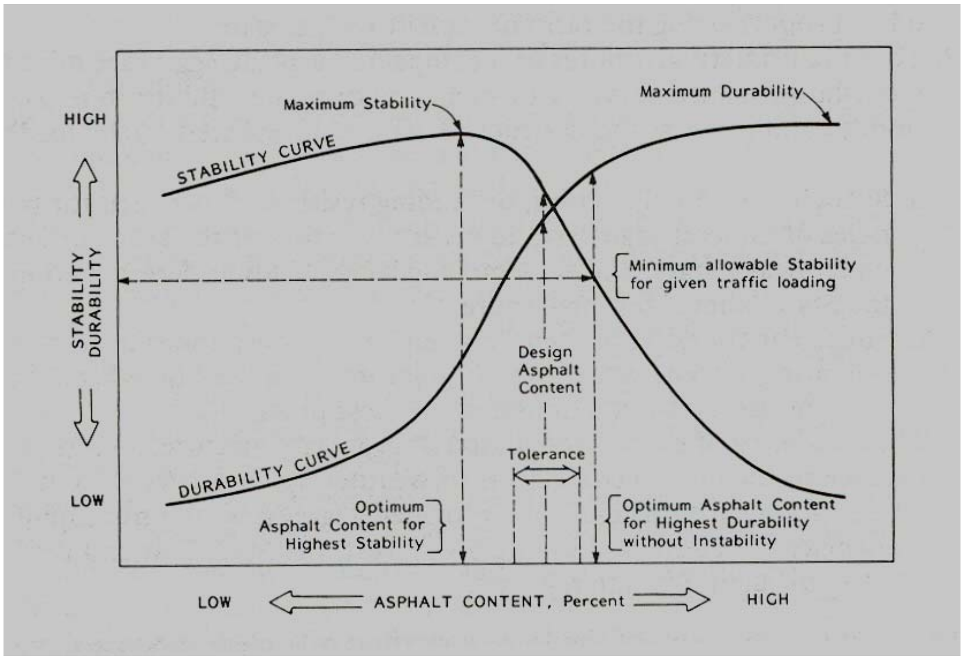

Figure 7.4 – Stability and durability as a function of binder content (Vallegra and Lovering, 1985)

Two mix design systems will be discussed: Marshall and Superpave. This section provides an overview of mix design concepts. Additional information can be found in the HMA textbook by Roberts et al., 1996. Both mix design methods use the concept of volumetric properties to determine the optimum asphalt content of the mix. The volume of an HMA mixture is made up of the sum of its component volumes. A brief introduction to volumetric properties will be provided first.

Steps in mix design process:

1. Obtain representative samples of aggregates and asphalt binder.
2. Evaluate aggregate properties.
3. Evaluate asphalt binder properties.
4. Determine desired aggregate gradation.
5. Prepare laboratory samples with desired gradation and variable asphalt contents.
6. Compact samples using required compaction effort.
7. Determine volumetrics: air voids, VMA, and voids filled with asphalt.
8. Measure stability and flow.
9. Plot volumetrics, stability, and flow versus asphalt content.
10. Determine asphalt content to provide desirable voids (typically 3.5% to 4.0%).
11. Prepare samples at selected asphalt content, and determine moisture susceptibility.
12. Determine and report results and job mix formula.

7.4.1 Volumetric Properties

In the mix design process, one of the most important factors is the volumetric properties of the mixture. The volumetric properties include air voids, voids in mineral aggregate, and voids filled with asphalt. These properties must be controlled within acceptable ranges to ensure the mixture will resist rutting and will provide good durability.

The air voids are expressed as a percentage of the total volume. For example, if there is 5% air voids, this means that 5% of the total volume is made up of air. The air voids must be controlled within an acceptable range for good performance. If the voids are too high there will not be good durability, and if the voids are too low the mixture will be susceptible to rutting. Generally, when the in-place air voids are approximately 8% or higher, the mixture will be permeable to air and water and will tend to oxidize quickly, resulting in the mixture becoming more brittle. These high air voids will also allow water to penetrate, resulting in stripping of the binder from the aggregate and potentially water getting into the underlying unbound materials.

> The performance of the HMA mixture is greatly affected by the amount of air voids in the mixture. If the air voids are below approximately 3% in laboratory compacted samples, bleeding and rutting are likely to occur as the mix densifies under traffic. If the air voids are above approximately 7% to 8% in-place after compaction, the mixture is likely to be permeable to air and water, resulting in loss in durability.
>
> It is unlikely that the in-place air voids are less than the laboratory air voids. If this is the case, be sure to check to see if test methods are followed and accurate test results are presented.

If the laboratory air voids are below approximately 3%, the mixture will tend to bleed and rut, resulting in early performance problems that will have to be repaired after a short period of time. The laboratory density is an indication of the density that the mixture will reach under traffic. Hence, if the laboratory air voids are 3%, it is estimated that the mixture will reach approximately 3% air voids after traffic has densified the mixture. Even though the initial in-place air voids will be above 3%, this will change as traffic is applied, eventually resulting in the in-place voids reaching approximately 3%.

A second volumetric property that is important to the performance of the mixture is the voids in the mineral aggregate. The property describes the amount of voids in the aggregate structure. An asphalt mixture contains aggregate, asphalt, and air. The volume of asphalt plus volume of air are considered as the amount of voids in the mineral aggregate. Hence, if the volume of a mixture is 1 cubic foot and the volume of the aggregate is 0.84 cubic feet, the volume of the asphalt is 0.12 cubic feet, and the volume of the air is 0.04 cubic feet, then the volume of the voids in the mineral aggregate is 0.16 cubic feet. Hence, in this case, the amount of voids in the mineral aggregate is 16%.

The voids in the mineral aggregate are important to ensure that there is enough room in the aggregate for sufficient asphalt to provide good durability. If the voids in the mineral aggregate (VMA) is low, sufficient asphalt cannot be added to ensure durability without reducing the air voids to a level that is too low. The gradation of the aggregate is the one property that controls VMA. The gradation has to be selected in a way that provides sufficient VMA. The recommended minimum VMA will depend on the maximum size of the aggregate. As the maximum size increases, the minimum VMA requirement is reduced. It becomes more difficult to maintain VMA when the amount of dust in the mix is high.

Material and mixture properties needed to determine volumetrics:

- Bulk specific gravity of aggregate (ASTM C 127 and C 128)
- Specific gravity of asphalt binder, typically 1.03 (ASTM D 70)
- Bulk specific gravity of compacted mixture (ASTM D 2726)
- Theoretical maximum specific gravity of mixture (ASTM D 2041)

The air voids in compacted HMA samples are used in determining the optimum binder content during mix design, monitoring production of the mix in the field, and ensuring the quality of the in-place pavement. In order to determine the air voids of the compacted HMA sample, both the bulk specific gravity of the mixture and the theoretical maximum specific gravity must be measured (The air voids are determined by expressing the bulk specific gravity as a percentage of the theoretical maximum specific gravity. The air voids are then determined by subtracting this percentage from 100.0. For example if the bulk specific gravity is 98.0% of the theoretical maximum specific gravity, the amount of air voids is 2.0%.). The theoretical maximum specific gravity is basically the mass of the binder and aggregate divided by the voidless volume of the binder and aggregate. The theoretical maximum specific gravity is determined according to ASTM D2041. A setup of the test is shown in figure 7.5.

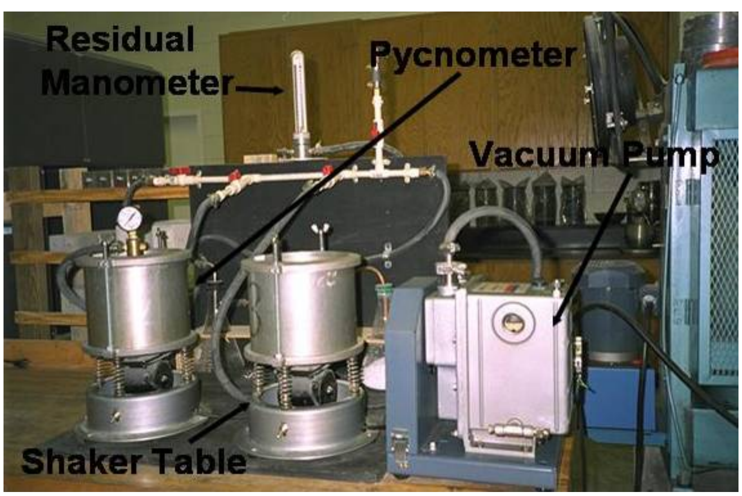

Figure 7.5 – Theoretical maximum specific gravity apparatus

7.4.2 Marshall Mix Design

In the Marshall mix design procedure, once the materials are selected and a design gradation is selected, a series of samples are mixed and compacted over a range of asphalt contents encompassing the expected optimum asphalt content. Typically, samples are compacted at four to five different asphalt contents with the asphalt content increments being 0.5%. At least three samples should be compacted at each asphalt content.

The Marshall method typically does not include any short-term oven aging to simulate the aging and absorption of the binder that typically occurs when mixed in an asphalt plant. The aggregate temperature should be set so that the resulting HMA temperature, after mixing, is equal to the desired compaction temperature. The asphalt binder should be maintained at the mixing temperature determined from the kinematic or rotational viscosity tests, or, in the case of a modified binder, the supplier's recommendations. Typically, an aggregate temperature approximately 50 °F above the recommended mixing temperature for the asphalt binder suffices. Some designers will reheat the mixture to the compaction temperature after mixing. This should not be done. If the mix is reheated, the Marshall stability will be increased significantly, and the stability results will not be useful in evaluating the quality of the mixture.

If an automatic hammer is used, it should be calibrated to the hand hammer since there is a difference in results between the two types of hammers (figure 7.6). The calibration involves preparing a number of samples and compacting some with a range of blows with the automatic hammer and compacting others with the required number of blows with the manual hammer. A plot is prepared showing the number of blows with the automatic hammer versus density. The number of blows with the automatic hammer needed to provide equal density to that obtained with the specified number of blows with the manual hammer is determined. It is recommended that the mix design be done with a manual hammer, and the automatic hammer, if used, be calibrated and used during the start of plant production.

Figure 7.6 – Automatic Marshall hammer (left) and manual hammer (right)

The air voids of the cooled sample are determined as described previously. The volume of the compacted samples is determined when determining the density of the individual samples; this volume is used to determine a corrected stability value. The samples are placed in a 140 °F (60 °C) water bath for 30 minutes. The bath temperature was selected to represent typical pavement temperatures on a hot summer day. The samples are taken out of the water bath and immediately loaded into a Marshall stability and flow-breaking head and then tested for stability and flow (figure 7.7). The sample is loaded at a rate of 2.0 in. per minute. The stability is defined as the peak load carried by the sample, and the flow is the vertical deformation at the peak load in 0.01 in.

Figure 7.7 – Marshall stability and flow equipment

After samples have been compacted and tested for density, air voids, VMA, stability, and flow, the results are plotted and analyzed (figure 7.8). Typically, the optimum asphalt content is selected as the asphalt content that provides 4% air voids in the mixture for military pavements and 3.5% for civil airports. Other properties such as VMA, stability, and flow must be within the specification requirements at this asphalt content. If they are within specifications, then the moisture susceptibility test is conducted on the design mixture. From the plot, it can be seen that the optimum asphalt content for a military mixture would be approximately 5.5% assuming that all other properties were within specifications. The optimum for civil airports would be closer to 5.7%.

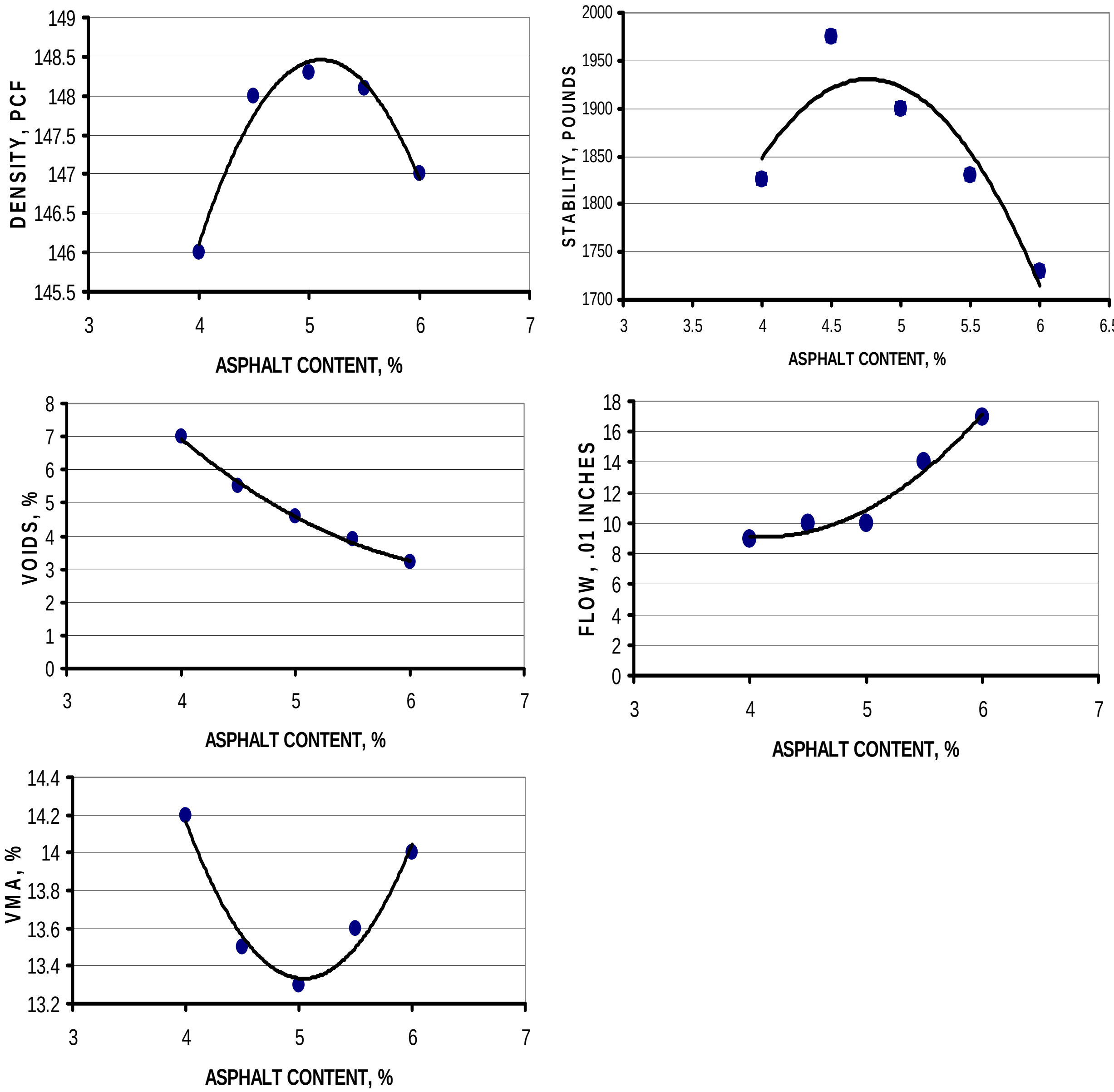

Figure 7.8 – Plot of mixture properties versus asphalt content

Table 7.1 presents FAA P401 and Unified Marshall mix design criteria. The VMA requirements are provided for different minimum aggregate sizes in table 7.2.

Table 7.1 – Marshall mix design criteria

Test Property	FAA P401	Unified	FAA P401	Unified
	Pavements designed for aircraft gross weights ≥ 60,000 lbs or tire pressures ≥ 100 psi	Tire pressures ≥ 100 psi	Pavements designed for aircraft gross weights < 60,000 lbs or tire pressures < 100 psi	Tire pressures < 100 psi
Compaction, blows	75		50	
Stability, lb (N)	2,150 (9,560)		1,350 (6,000)	
Flow 0.1 in (0.25 mm)	10 to 14	8 to 16	10 to 18	8 to 18
Air Void, %	2.8 to 4.2	3 to 5	2.8 to 4.2	3 to 5

Table 7.2 – Minimum VMA as a function of aggregate size

FAA Maximum Particle Size		Unified		Minimum VMA, Percent
inch	mm	Gradation No.	Max. particle size, inch	
½	12.5			16
¾	19.0	3	½	15
1	25.0	2	¾	14
1 1/2	37.5	1	1	13

7.4.3 Superpave Mix Design

FAA Engineering Brief 59A provides information and guidance on using the Superpave mix design system for airfields. The AAPTP has an ongoing research project, AAPTP 04-03, "Implementation of Superpave Mix Design for Airfield Pavements," which is scheduled to be completed in 2008. The Superpave Mix Design System is summarized in AASHTO M323 and AASHTO R 35.

The following discusses the Superpave concepts and their current implementation as provided in Engineering Brief 59A.

The aggregate gradation requirements specified in the Superpave Mix Design System are not recommended for airfields. The FAA and DoD have their own specification requirements for aggregate gradation.

Once the design aggregate gradation is selected, samples are batched for a volumetric mix design. Superpave Gyratory Compactor (SGC) (figure 7.9) samples are 6 in. (150 mm) in diameter and have a target height of 4.5 in. (115 mm). Therefore, SGC samples require a larger mass of material than for Marshall, typically 4500 to 5000 grams of mix depending on the specific gravity of the aggregate. The larger sample size was selected for SGC samples in order to allow testing of mixes containing larger aggregates than that allowed for Marshall.

Figure 7.9 – Typical Superpave gyratory compactor.

Originally, the angle of gyration was measured external to the gyratory mold. However, this external calibration did not provide uniform results between different types of compactors. It was determined that internal measurements of the gyratory angle provided results that gave more uniform compaction. Devices have been developed to measure the internal angle of gyration inside the mold and under load.

Superpave specifies that one additional property be calculated: the dust to effective binder content ratio. To do this, first the effective asphalt content must be calculated. The effective asphalt content is the portion of the asphalt content that is not absorbed into the surface voids of the aggregate. The dust to effective asphalt content is then the percent passing the No. 200 (0.075 mm) sieve based on a washed gradation divided by effective asphalt content.

The volumetric criteria for the Superpave Mix Design System vary by traffic level and NMAS, as shown in table 7.3; the minimum VMA requirements by NMAS are shown in table 7.4. The Ninitial and Nmaximum requirements were developed for roads and are put in place to help ensure that rutting never occurs in the pavement. These requirements tend to encourage slightly lower asphalt content and coarser mixtures. Coarser mixtures result in more permeability issues and durability problems if not properly compacted. These requirements for Ninitial and Nmaxinum typically have no beneficial benefit on the designed mixture. These requirements are sometimes not included in specification requirements and, in fact, several research studies have recommended that they no longer be included as part of the Superpave requirements. These requirements for Ninitial and Nmaximum can be excluded from the specifications without affecting the quality of the designed mixture.

Table 7.3 – Superpave design requirements (EB 59A)

Test Property	Pavements for Aircraft with Gross Weights ≥ 60,000 lbs.	Pavements for Aircraft with Gross Weights < 60,000 lbs.
Initial number of gyrations (N_{ini})	8	7
Design number of gyrations (N_{des})	85	60
Maximum number of gyrations (N_{max})	130	90
Air voids @ N_{des}	4.0	4.0
VMA @ N_{des}, %	see table 7.4	
VFA @ N_{des}, %	65-78	65-78
Dust proportion	0.6 to 1.2	0.6 to 1.2
Dust proportion (coarse gradations)*	0.6 to 1.6	0.6 to 1.6
%G_{mm} @N_{ini}	≤90.50	≤90.50
%G_{mm} @N_{max}	≤98.00	≤98.00

*A coarse gradation is defined as passing below the restricted zone.

Table 7.4 – Superpave minimum VMA requirements

NMAS, mm	Minimum VMA, %
19.0	13.0
12.5	14.0

7.5 Moisture Sensitivity

The final step in any volumetric mix design procedure, whether Marshall or Superpave, is an evaluation of the moisture susceptibility of the mixture at optimum asphalt content. Moisture susceptibility is the condition where the asphalt will potentially separate from the aggregate or the asphalt binder itself weakens in the presence of moisture. The most commonly used test for determining moisture susceptibility is the tensile strength ratio (TSR) test (ASTM D4867) (figure 7.10). The test method specifies that samples be compacted to 7 ± 0.5% air voids. With the SGC, preparing samples 95 mm tall with an air void content of 7% is relatively easy. A volumetric calculation is done to estimate the mass of material required to give 7% air voids or 93% density.

The SGC is then set to compact to a specified height (95 mm) instead of a specified number of gyrations. A similar calculation can be used to estimate the required mass for Marshall samples, but the number of blows needs to be adjusted to achieve the required height. This is a trial and error process. A minimum of six samples need to be prepared, and generally eight are prepared to ensure that six samples with the appropriate air void content are obtained. Based on ASTM D4867, the mix is aged for 1 to 2 hours at the compaction temperature in a closed container prior to compaction.

The samples are divided into two subsets, a dry and a conditioned subset. The samples in the conditioned subset are then saturated with water under a partial vacuum. The required degree of saturation is 70 ± 5%. The saturated samples are wrapped tightly in plastic wrap. After saturation, there is an optional (for the ASTM method) 16-hour freeze cycle. This option should be used by all agencies regardless of whether it freezes in their climate. The freeze cycle produces pore pressures in the mixture, similar to that produced in a saturated pavement under loading. The freeze-thaw cycle is required for Engineering Brief 59A designs. The use of freeze-thaw cycle(s) is optional for P401 designs. The use of a freeze-thaw cycles(s) is not discussed in the Unified specification. The minimum required TSR varies by specification and ranges from 75 to 80.

Certain aggregate types, such as granite or siliceous material, are more susceptible to moisture damage than other aggregate types. Mixtures that fail TSR can be addressed in a number of ways. Liquid anti-striping agents, primarily amines, can be added to the binder either at the terminal or at the HMA plant. These additives alter the surface chemistry between the asphalt and binder, improving adhesion. Hydrated lime can also be added to the mix. Hydrated lime should be added to damp aggregate or in a slurry form to allow a chemical reaction to occur between the lime and the surface of the aggregate. The addition of hydrated lime does increase the material passing the No. 200 (0.075 mm) sieve in the mixture. It also tends to reduce the optimum asphalt content, even at the same dust content.

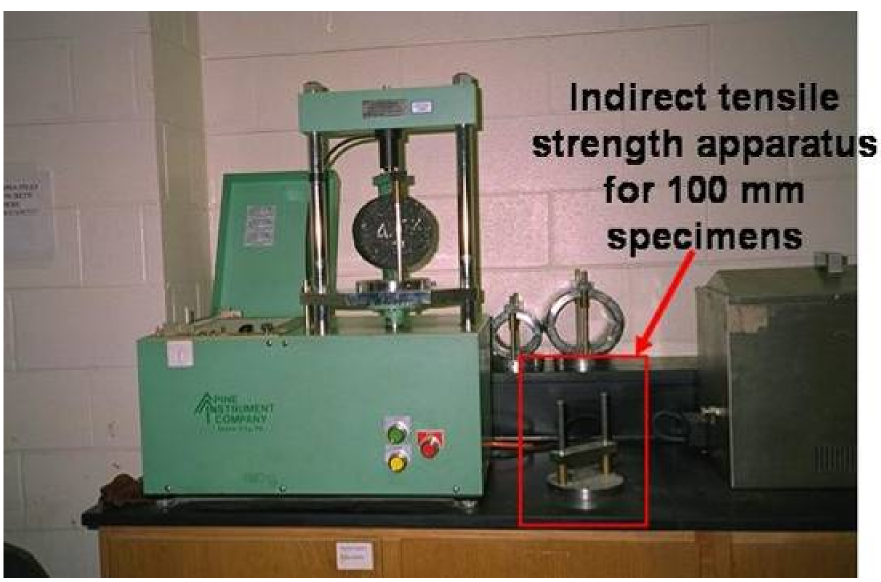

Figure 7.10 – TSR fixture and press

Primary mixture properties used to conduct and evaluate mix design:

- Air voids
- Voids in mineral aggregate
- Voids filled with asphalt
- Stability for Marshall
- Flow for Marshall
- Dust to asphalt ratio

7.6 Mix Adjustments

7.6.1 Volumetric Properties

In the mix design phase, VMA may control the initial gradation selection. VMA typically decreases during plant production if breakdown during plant production is not properly accounted for during the mix design stage. Aggregate breakdown may be accounted for by adding baghouse fines. Ideally, the design VMA should be approximately 1.0% higher than the specified minimum.

If the VMA is too low, there is insufficient room for asphalt binder to ensure durability. VMA can be increased in a number of manners. Moving the gradation away from the maximum density line, shown previously in figure 7.3, tends to increase VMA (and air voids). For airfield mixes, one typically needs to add more fine aggregate (reduce the coarse aggregate) to increase VMA since there is not a great deal of room to go coarser than the maximum density line. Reducing the percentage passing the No. 200 (0.075 mm) (dust) sieve will also increase VMA. A rule of thumb is that a 1% decrease in dust will increase VMA by approximately 1%. Dust can generally be decreased by switching a portion of crushed screenings for either a washed manufactured sand or a clean natural sand. However, there are limits on how much natural sand can be used.

It is also undesirable to have VMA that is too high. High VMA will result in higher asphalt contents, adding cost to the mix and potentially providing a mixture that will lack stability under traffic. VMA can be reduced by moving the gradation closer to the maximum density line, adding dust or possibly a small amount of natural sand. In some cases, the VMA may still be 2% to 3% above the minimum, even after adjustments have been made within the allowable parameters (e.g., maximum percent dust).

7.6.2 Stability and Flow

When evaluating stability and flow, VMA and voids should be adjusted first. A mix with low voids may have high flow; however, flow will likely decrease as the voids increase. Mixes with very high stabilities or very low flows tend to be brittle and may crack under heavy loadings such as aircraft. The exception occurs with the use of polymer modified binders. Mixes containing polymer modified binders have increased stabilities, but the flow number also tends to be higher. When modified binders are used, the flow requirement sometimes has to be waived since very high flow numbers may result. Binders modified with elastic polymers do not tend to be brittle but, rather, to improve elasticity. Stability can be increased by increasing the amount of crushed material.

The mix produced at the plant will always be a little different from the mix design due to breakdown of materials going through the asphalt plant and sampling errors. The mix design is a starting point, and some adjustments will have to be made at the startup of mix production to get the optimum mix properties. If excessive changes are required, a new mix design may be necessary.

8. PRECONSTRUCTION CONFERENCE

8.1 Introduction

Preconstruction conferences are held after the paving contract has been awarded but before the "notice to proceed" has been given to the contractor. This meeting initiates verbal communication between the owner's representatives (agency personnel) and contractor personnel. The preconstruction conference can set the tone for the working relationship between the owner and contractor and establish the communication lines. The most important part of project planning, organization and execution is communication!

8.2 Participants and Purpose

Guidance for conducting preconstruction conferences for airport paving projects is provided in the following FAA and DoD documents:

FAA Advisory Circular No. AC 150/5300-9A: *Predesign, Prebid and Preconstruction Conferences for Airport Grant Projects*
DoD Unified Facilities Guide Specification: UFGS -01 31 1900 40: *Project Meetings*

Participants in the preconstruction conference meeting should include the following:

- Sponsor/agency representative
- Project engineer
- Project inspector
- Contractor
- Subcontractors
- Testing laboratories
- Airport management
- Airport operations

The agency representative is responsible for convening and conducting the preconstruction conference as soon as possible after the contract has been awarded. The preconstruction meeting is held to review project requirements and to discuss project planning with the contractor. The scope of the project and contract documents should be discussed. This is an opportunity to discuss any nonroutine construction practices that may be required.

The preconstruction conference is conducted to ensure all parties are aware of design, construction, and safety requirements. This meeting will help all parties understand the responsibilities of the owner's representatives and the contractor's personnel. The preconstruction conference is an opportunity to discuss issues upfront before construction begins (i.e., potential problem areas and means to create solutions). The contractor should provide and discuss the planned construction schedule and how the construction will affect the airport operations.

8.3 Typical Discussion Items

Discussion items for the preconstruction conference should generally include issues relating to airport operations, security, safety, environmental factors, schedule, and construction plans. Specific items to review and discuss could include:

- Project scope
- Project specifications and drawings
- Project schedule
- Contractor and agency key personnel
- Role of key personnel
- Lines of authority: "chain of command"
- Applicable permits and licenses
- Project safety plan
- Airport security and access
- Environmental issues
- Utility locations
- Labor requirements
- Aircraft operations
- Materials submittals
- Quality-control plan (QCP)
- Testing and sampling
- Test section requirements
- HMA mix designs
- Subgrade preparation
- Subbase and base course construction
- HMA plant production
- HMA transportation and mix delivery
- Surface preparation
- HMA laydown and compaction
- QC/QA requirements

8.4 Test Section

Prior to full HMA production, the FAA and DoD require the contractor to construct a test section before HMA airfield paving operations begin. The purpose of the test section is to evaluate the quality of the job mix formula (JMF), HMA plant production, transportation, laydown equipment and methods, and compaction techniques. Full HMA production cannot begin until an acceptable test section has been constructed and accepted by the owner or engineer.

Specific details for the test section are:

- Length should be at least 300 feet.

- Width should be at least two pavers wide (20 to 30 feet).
- Construct a cold longitudinal joint.
- Place HMA in similar conditions anticipated during airfield construction (e.g., thickness, underlying structure, weather conditions).
- Use same laydown and compaction equipment and procedures that will be used during construction.

8.5 Quality-Control Plan (QCP)

The contractor is responsible for establishing and maintaining an effective quality-control plan (QCP) that will be utilized during the airfield paving project to assure that all materials and construction conform to contract plans and specifications. (For small projects, it may not be practical to establish a detailed QCP, but it is important that good construction methods and good control techniques be used.) The purpose of the QCP is to enable the contractor to provide the necessary control that will adequately provide for the production of acceptable quality materials and to provide sufficient information to assure the contractor and engineer that specifications have been met. The contractor should submit the QCP prior to the preconstruction conference to allow the engineer to review the plan and make suggestions at the preconstruction meeting.

> The QCP is the process that the contractor uses to ensure that mixture quality is controlled. This plan is very important to the success of the project. The government representative often uses the QC data as a key part of the acceptance plan. QCP plan needs to be approved and in place prior to start of construction.

8.6 Preconstruction Conference Agenda for Asphalt Airfield Paving Projects

The agenda for the preconstruction conference should include some discussion of the following items.

- Plans and Specifications
  - Review scope of the work.
  - Review any deviations from FAA or DoD specifications.
  - Review and approve procedures for change orders; supplemental agreements.

- Construction
  - Establish the relationship between sponsor and contractor. Review authority of project engineer and agency inspector.
  - Materials acceptance testing is the responsibility of the sponsor. Quality-control testing is the responsibility of the contractor.

- No work should commence or be covered until approved by inspector. Prior to initiating acceptance testing, it should be the responsibility of contractor to provide quality-control testing. Discuss penalty clause for asphalt pavements.
- Contractor must obtain all applicable permits and licenses.
- Contractor provides protection and restoration of property.
- Discuss who is responsible for locating underground utilities and who should be contracted in case of emergency.
- Contractor should repair with identical material by skilled workers, any underground cables serving NAVAIDS, weather bureau, etc.
- Contractor should maintain paved areas clear of all foreign debris resulting from hauling and construction.
- The contractor is responsible for maintaining record drawing revisions that develop from construction.
- Submit weekly construction reports.
- Discuss liquidated damages.

- Labor Standards
  - Review the schedule of minimum wage rates (Davis/Bacon), craft classification, and apprentices.
  - Discuss veteran's preference whereby preference is given to veterans and disabled veterans.
  - All contract labor stipulations between contractor and subcontractors must be included in all subcontracts.
  - Contractor submits reports of work accomplished and payrolls to the sponsor. Each payroll submittal is accompanied by a statement of compliance.
  - Discuss wage rate posters.

- Safety on Airports During Construction
  - Contractor needs to review construction safety plan if applicable.
  - It is the contractor's responsibility to erect barricades, signs, and/or lights for protection of both air and ground traffic.
  - Vehicles on landing area must have 3-ft square orange and white flags, flashing yellow lights, or escort vehicle on aircraft operational areas.
  - Discuss coordination or airport operations with construction work, issue NOTAMS.
  - Ensure specification requires contractor to construct X's for closed runways and taxiways.
  - Contractor must be ready to demobilize an area as quickly as possible if necessary.
  - Discuss airfield security issues.

- Contractor's Construction Schedule

- Equal Employment Opportunity (EEO)
  - The EEO poster should be placed at the job site.

    - Contractor will not discriminate against any employee because of race, color, religion, sex, or national origin. Contractor's noncompliance with nondiscrimination clauses of this contract can mean the contract may be cancelled. An EEO compliance inspection may be made within 30 days by a civil rights officer.

- Testing Laboratories
    - Testing laboratories must have been inspected by a national authority. The testing laboratories must meet the requirements of ASTM D3666. Examples of national authorities are the American Association of Laboratory Accreditation (AALA), American Association of State and Transportation Officials (AASHTO) accreditation program, AASHTO Materials Reference Laboratory, Cement and Concrete Reference Laboratory (CCRL), Asphalt Materials Reference Laboratory (AMRL), or Corps of Engineers (COE).

8.7 Preconstruction Conference Checklist

Items related to technical issues that should be discussed at the preconstruction conference are provided below. It is important that all parties involved in the construction understand their roles and the roles of others. If there are any questions involving the project, these should be cleared up during the preconstruction conference.

- General Items
    - Construction schedule
    - Chain of command
    - Key personnel and roles
    - Materials and mix design submittals
    - Airport security
    - Locate utilities
    - Staging area
    - Haul roads and airport access points
    - Asphalt construction workshop

- Subgrade Preparation
    - Review geotechnical report (soil borings)
    - Cut and fill elevations and slopes
    - Borrow and waste areas
    - Fill material criteria
    - Removal of unacceptable soils
    - Proofrolling requirements and acceptance criteria
    - Compaction requirements and acceptance criteria
    - Chemical stabilization
    - Testing frequencies

- Subbase/Base Course Construction
    - Material properties and acceptance criteria

- Compaction requirements and acceptance criteria
- Placement equipment and procedures
- Thickness and grade acceptance criteria
- Proofrolling requirements and acceptance criteria
- Grade survey
- Testing frequencies

- HMA Plant Production Delivery
  - Aggregate stockpile management
  - Stockpile sampling and testing
  - Calibration of HMA plant
  - Proper loading of cold feed bins
  - Storage silos and storage time
  - Segregation
  - Proper loading of trucks
  - Release agents
  - Tarp loads
  - Haul time
  - HMA mix testing procedures, frequencies, and acceptance criteria

- Surface Preparation
  - Milling operations
  - Brooming operations
  - Paint removal
  - Excess crack sealant removal
  - Prime coat on base course
  - Tack coat on HMA layers
  - Repair areas
  - Reflective crack issues

- HMA Laydown and Compaction
  - Material transfer vehicles
  - Grade control
  - Paver speed
  - Temperature: mix and ambient
  - Condition of equipment
  - Joint construction and acceptance criteria
  - Layer thickness
  - Rollers: speed, size, type
  - Tender mixes
  - Roller patterns
  - Density requirements and acceptance criteria

Good communication between all parties involved in a project is necessary if a quality product is to be obtained. The preconstruction conference sets the tone for future communications. It is important that all aspects of the work be thoroughly discussed, potential problem areas be addressed, and any questions or concerns be dealt with. All answers may not be available at the preconstruction conference, but all can agree who will provide the answers and when they will be available. At the end of the conference, it is very important to know the point of contact for each party involved in the project.

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9. PLANT PRODUCTION

9.1 Introduction

The primary purpose of an asphalt plant is to produce a uniform mixture that meets the requirements of the job mix formula (for asphalt content and gradation), plus or minus the tolerances. The plant has to be capable of heating and drying the aggregate and mixing with the desired percentage of asphalt binder. The plant should be capable of uniformly producing an HMA mixture without excessively segregating the material.

Two general types of plants will be discussed in this section: drum mix and batch plants. Batch plants have a much longer history of use, but the drum mix plants have become very popular since the late 1970s. By far, most plants currently sold in the United States are drum mix plants.

Uniformity of materials is essential during plant production:

- Ensure incoming materials, including asphalt binder and aggregates, are within specifications.
- Ensure no contamination of asphalt binder as it is added to tank.
- Use good stockpiling techniques as material is added to stockpiles.
- Use proper techniques as material is removed from stockpiles and fed into feeder bins.
- For drum mix plants, adjust aggregate feed for moisture content.
- Ensure HMA is properly mixed with correct asphalt content and segregation is minimized.
- Ensure proper silo loading procedures as well as truck loading from silo. Correct any segregation problems.

9.2 Stockpile Operations

Regardless of the type of plant that is used, stockpiles must be created and maintained so that they are uniform, without segregation, and without contamination from other materials that may be available in the stockpile area. Generally, stockpiles are created in layers so that segregation is minimized. Aggregates should not be added to a stockpile by dumping over the edge as shown in figure 9.1. This will result in the coarser aggregate running down the stockpile and the finer material remaining at the top resulting in segregation.

The stockpiles should be inspected to ensure that no segregation or contamination exists in the stockpiles. Stockpiles should be maintained so that there is no contamination from adjacent stockpiles, organics, or clayballs. Cone-shaped stockpiles are very likely to be segregated. Creating cone-shaped stockpiles tends to result in the coarse material running down the side of the stockpile, resulting in coarser material along the bottom. Paving the surface underneath the stockpiles allows for better drainage of the stockpiles and prevents contamination from the underlying materials.

Figure 9.1 – Improper procedure for adding material to a stockpile

Crusher run material (aggregate that has not been screened into sizes) should not be used to produce HMA. All aggregates should be split into appropriate sizes so that the desired gradation can be developed during the mix design and produced during plant production. Often, there will be at least four stockpiles used to develop HMA for a project. Typically, the stockpiles used to produce HMA are as follows: two coarse aggregates, one crushed fine aggregate, and one natural sand aggregate.

Care must be used to ensure that stockpiles are not contaminated with clayballs or organic material such as roots. Use of contaminated stockpiles will result in popouts in localized areas and more significant problems if the amount of contamination is excessive. An example of organics and clayballs is provided in figures 9.2 and 9.3, respectively. If a stockpile contains a significant amount of clayballs or organics, it should be rejected to ensure that popouts don't occur in the completed pavement, leading to potential FOD problems. The stockpiles should be inspected to ensure that good crushed material is being used as detailed in the specifications. The amount of natural sand is typically limited to no more than 15% for airfield pavements.

It is essential that the calibration is correctly performed to accurately feed in the amount of each material added to the mixture. The amount of natural sand used in the mixture should be controlled not to exceed that amount detailed in the specifications. Care must be exercised to ensure that the proper aggregate is fed through the correct feeder and at the right rate. It is sometimes easy to feed materials into the wrong bins or to have material overflow from one bin into the adjacent bin, resulting in increased variability in the mixture properties.

Figure 9.2 – Contamination with organics

Figure 9.3 – Contamination with clayballs

Routinely check stockpiles for:

- Clayballs
- Organics
- Consistency of materials including segregation issues
- Contamination from other materials
- Variations in moisture content
- Proper addition of materials to stockpiles
- Proper loading of material from stockpile to feeder bins

9.3 Types and Operation of Plants

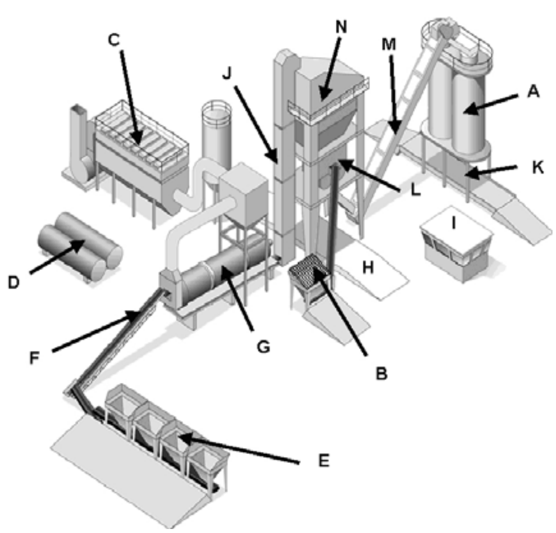
9.3.1 Introduction

There are two types of plants used to produce HMA: batch and drum mix plants. The oldest type of plant is a batch plant. The more recently developed plant is the drum mix plant, which began to be used in the late 1970s. The batch plant produces HMA in batches. Each batch is weighed into the pugmill, mixed, and either dumped into a waiting truck or added to a storage silo. The drum mix plant produces HMA on a continuous basis. Some type of storage silo is required with a drum mix plant since it is a continuous operation. A storage silo is optional with a batch plant, but one or more is almost always used.

9.3.2 Batch Plant

9.3.2.1 Components of a Batch Plant

The primary components of a batch plant are shown in figure 9.4.



A-Storage silo
B-RAP feeder
C-Baghouse
D-Binder storage
E-Cold feed hoppers
F-Cold elevator
G-Dryer
H-Plant scales
I-Control house
J-Hot elevator
K-Silo truck scales
L-Pugmill
M-Mix to silo
N-Hot bins

Figure 9.4 – Layout of batch plant (from NAPA)

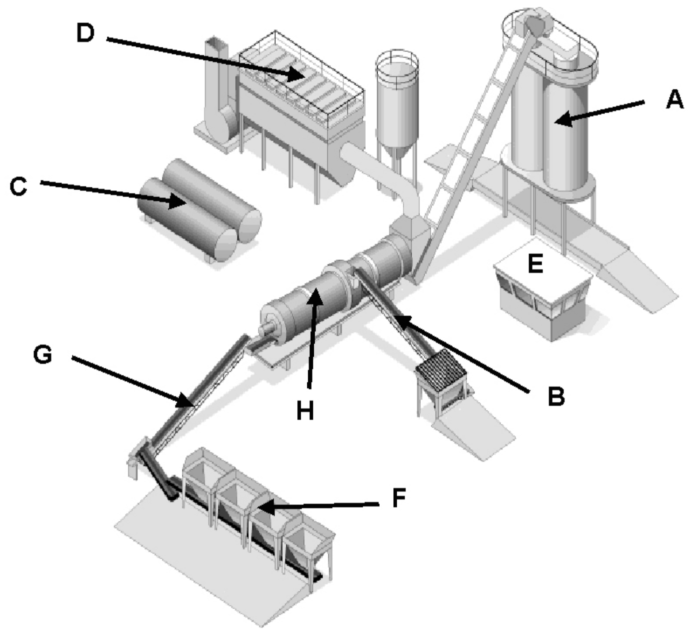
The aggregate is typically picked up from the stockpiles with a front-end loader and loaded into the cold feed hopper. For accuracy, the feeder for each of these hoppers must be calibrated for the material being fed into the plant. Each aggregate, in the correct proportion, is fed onto the collector belt underneath the cold feed hoppers and then fed onto the cold elevator and carried up to the dryer where it is heated and dried. After heating, the aggregate is carried up the hot elevator to the screening deck where it is screened into the various hot bins and then temporarily stored for use. The aggregate from the various hot bins along with the binder are then batched in the appropriate proportions and added to the pugmill where the materials are mixed. After mixing, the HMA is typically fed into the storage silos and held there until needed. Some government agencies allow the mix to be held overnight in the silos and used the following day, but this is not recommended as a general rule. Generally, the asphalt mixture can only be stored in the silo for a few hours when being used for airfields. Keeping the HMA heated for this long period of time can lead to some oxidation and possible draindown of the binder, resulting in some loss of performance in the HMA. The silos are not necessary with a batch plant but are almost always used to help ensure that the plant runs continuously, thus producing a more consistent product.

Batch plants have been used for years to produce HMA. They operate by adding the appropriate percentage of aggregates and binder to the pugmill for mixing. This process has worked well for years and continues to work well but because of many advantages of drum mix plants, the percent of the HMA market that is produced with batch plants has continued to decrease.

9.3.3 Drum Mix Plant

9.3.3.1 Components of Drum Mix Plant

Figure 9.5 shows that the number of components in a drum mix plant is significantly less than that for a batch plant. The flow of material begins in the cold feed hoppers, which should be calibrated for the materials being fed. The material is fed from the cold feed hoppers onto the collector belt and onto the cold elevator where it is weighed on the run as it is fed into the dryer/mixer. The weight of the aggregate also includes the moisture, so it is important that the moisture content of the aggregate stockpiles be known to help ensure that the aggregate weight being fed into the dryer is correct. The amount of asphalt added to the plant is controlled by this measured aggregate weight, so if it is in error then the asphalt content produced will be incorrect. All of the materials, including RAP, binder, and aggregates, are added to the dryer and mixed inside the dryer. After the material is mixed, it is moved to the silo for storage until being hauled to the project.



A-Storage silo
B-RAP feeder
C-Binder storage
D-Baghouse
E-Control house
F-Cold feed hopper
G-Cold elevator
H-Dryer and mixer

Figure 9.5 – Layout of drum mix plant (from NAPA)

Several items will be discussed about plant operations, including stockpiling operations, calibration of feeders, aggregate drying, moisture correction for drum mix plants, and storage silo operations. Certainly, one of the biggest concerns at an asphalt plant is maintaining good stockpiles that are not segregated and that are not contaminated with other materials.

Another problem that occurs at many plants is insufficient drying of aggregates. When excess moisture is in the aggregates, the mix may act tender when rolling and may strip after some time of traffic. There are methods (e.g., storing stockpiles under a roof) that can help to prevent excessive moisture.

Also, much segregation that occurs at the laydown site is actually initiated at the plant. This can happen in the stockpiles, in the storage silo, or when adding the materials to the trucks.

9.3.4 Storage Silos

Almost all plants now have storage silos for the hot mix asphalt (figure 9.6). While these silos have been used to store materials up to several days in some cases, it is recommended not to store mix used for airfield projects for more than a few hours. Uninsulated silos can be used to store the material for up to 3 hours, and insulated silos can be used for storage for up to 8 hours.

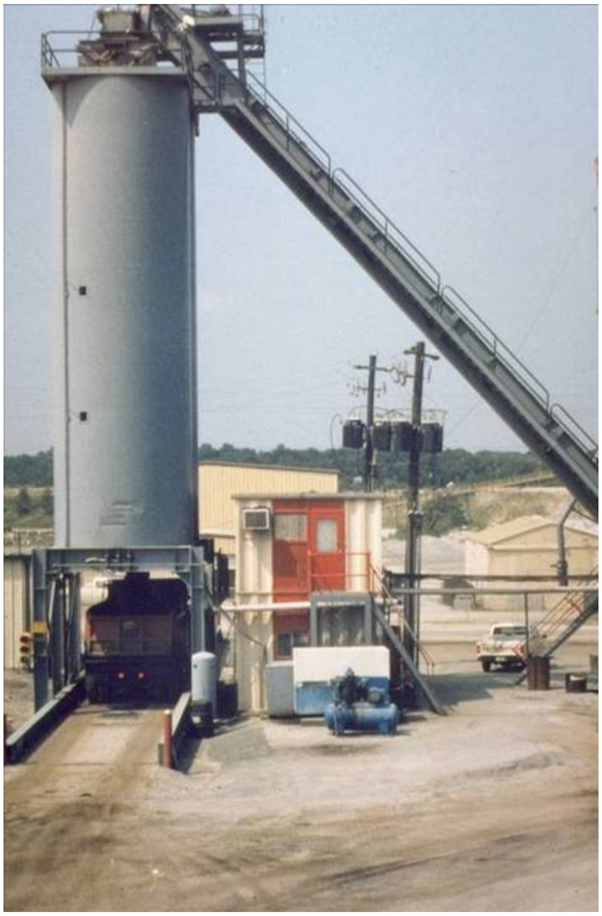


Figure 9.6 – Storage silo for HMA

Storage silos are one source of segregation, so they need to be in good operating condition to ensure that segregation does not occur.

9.3.5 Common Problems at HMA Plants

There are a number of problems that are more frequently observed at asphalt plants. These problems will be identified and discussed here.

The stockpiles should be maintained in good condition. Often, stockpiles are contaminated with materials from adjacent stockpiles or from the underlying material spread into the stockpile with the loader. This contamination will often result in increased variability in the produced mix and in nonuniform performance.

Many times, stockpiles are contaminated with clayballs and woodchips. Generally, the natural sand stockpile is the one that will be more likely to be contaminated with clayballs or woodchips. When gravels are used, this contamination can be significant also. A very small amount of contamination will generally not cause significant performance problems, but if the amount is excessive, popouts leading to potential FOD can be a problem. It is important to check all stockpiles but particularly the natural sand stockpile to ensure that it does not contain clayballs or woodchips. If the sand stockpile does contain these materials, it should be rejected for use until the problem is solved. Screening can sometimes be done to help remove the oversize clayballs and woodchips.

Another problem that sometimes results is use of too much natural sand. An inspection of the feed rates of the aggregates can give an indication of the amount of natural sand that is being added. After the material is added, it is difficult to determine the amount of natural sand. Sometimes the gradation of the final blend can be used to show if too much natural sand is used, but this can often be difficult. Knowing the gradation of the natural sand and the gradation of the final asphalt mixture usually allows the amount of natural sand to be determined.

Measuring the feed rate of each of the feeders is probably the best method of determining the amount of natural sand being added to a mixture. If too much sand is added, this can result in the mixture being tender during the rolling process, making it difficult to compact. Too much sand can also result in rutting of the mixture under traffic.

With a drum mix plant, the amount of moisture in the stockpiles must be input into the plant computer so that it can accurately determine the amount of dry aggregate being fed into the mixture. Without this correction, all moisture in the aggregate will be considered as aggregate leading to an incorrect asphalt content being added to the mixture. The moisture can vary from place to place in the stockpile, depending on time of day, and whether it recently rained. The loader operator has to operate the equipment in a way that will tend to average the moisture, and adjustments will need to be made in the plant input for moisture content when appropriate.

There is a tendency to overheat the HMA. The specifications generally allow up to 350 °F, but the actual temperature of the produced mix should generally be significantly less. The mix should not be heated any hotter than necessary to mix, haul, handle, and compact on the project. When overheated, the mixture will become brittle, resulting in more cracking as well as other problems caused by brittleness. Unmodified asphalts are typically heated to about 280 to 320 °F, depending on the grade of asphalt being used. Modified asphalts are often heated from about 310 °F, up to approximately 350 °F. As long as the rollers stay close behind the paver, these higher temperatures, up close to 350 °F, are not needed to ensure good compaction.

Silos can be one of the sources of segregation. The material is normally moved to the silo with a slat conveyor or other means and then fed into the silo. If the material is allowed to continuously feed into the silo, segregation occurs since the coarser material will tend to be thrown further across the silo than the finer material. This is prevented by using a gob hopper at the top of the silo. Material is collected in the gob hopper and then dropped as a batch into the silo. This allows the materials to stay together as a batch and minimizes segregation. It is essential that the gob hopper at the top of the silo be kept in good operating condition otherwise segregation of the material may occur.

The loader operator should keep the bucket about 1 to 2 feet above the underlying material (unless it is paved) to prevent contamination of the stockpile material. Also, the loader operator should operate from the dryer side of the stockpile, which is typically the sunny side of the stockpile.

The purpose of an asphalt plant is to blend and thoroughly mix the aggregates and asphalt binder in the proper proportions. This should be done without excessive breakdown of the aggregate and without overheating the asphalt binder. The moisture content of the aggregate should be reduced below the maximum requirement. The produced mixture should not be segregated.

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10. TRANSPORTING

10.1 Introduction

HMA is transported from the asphalt plant to the laydown site in trucks. There are a number of truck types that can be used. The type of truck used depends on personal preference and the method that will be used to supply material to the paver. The primary items to consider when evaluating trucks include the condition and cleanliness of the trucks, satisfactory release agent, loading procedures at the plant, and unloading procedures at the laydown site. It is also important to consider the use of insulated trucks for long haul distances and when hauling in cooler weather. The truck should have a tarp that can be used to cover the truck during hauling operations.

10.2 Condition of Trucks

Trucks should be in good operating condition, and the beds should not be pitted and worn sufficiently to cause the HMA to cling to the bottom when emptying at the laydown site. A truck should be able to discharge the HMA properly once it is at the laydown site. The tarps should be in good condition and able to be spread quickly over the entire upper surface of the HMA. Proper tie downs should be available and used to minimize air blowing underneath the tarp.

10.3 Release Agents

Release agents are used to coat the bottom and side of the truck bed to allow the HMA to be cleanly dumped from the truck to the asphalt paver. It is important that a proper release agent be used. In the past, diesel was often used as the release agent, but this should never be done. There are a number of proprietary materials that perform very well. These materials should not be petroleum-based or act as a solvent when mixed with HMA.

10.4 Loading and Unloading Procedures

Trucks should be loaded and unloaded in a way that will minimize segregation. Segregation is one of the most common problems that occur during HMA production and placement. Segregation often occurs when the material is loaded from the plant pugmill or silo into the truck. When loading, the material tends to form a cone in the back of the truck. As additional material is added, it tends to run down the side of the cone with the coarser material running further, resulting in segregation around the truck sides, back, and front.

Ways to minimize this segregation when loading a truck involve methods to minimize the formation of a cone and to minimize the distance that the material can travel when it is added to the truck. To minimize the formation of a cone, the material should be dropped in batches and not be allowed to continuously flow into the truck bed. So, the gates are opened and some material is allowed to drop into the truck; then, after a short period of time, the gates are closed. This procedure is repeated until sufficient material has been added to the truck. When the material continually flows into the truck, a cone develops and segregation can occur.

A second step to minimize segregation is to load the front of the truck first, the back second, and the middle third. This minimizes the distance the material can flow once it is added to the truck. Examples of satisfactory loading procedures to minimize segregation are provided in figure 10.1.

Unloading trucks is very important to minimize segregation and to provide a smooth pavement. Details about unloading are discussed in the section on laydown procedures.

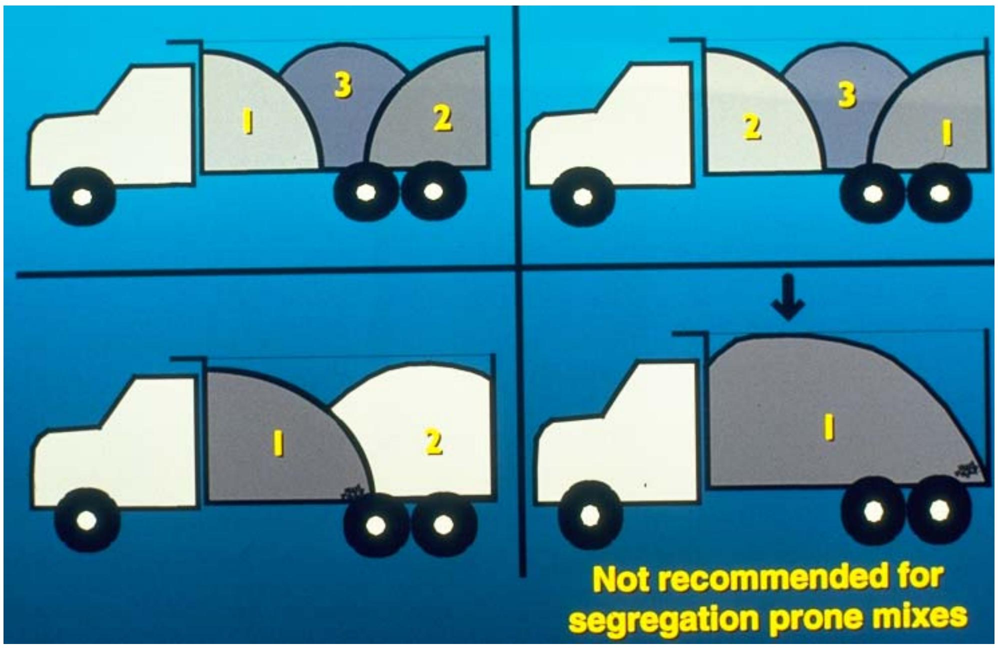

Figure 10.1 – Recommended truck loading procedures to minimize segregation

Overview of trucking:

- Truck bed should be in good shape.
- Use proper release agent. Do not use petroleum product such as diesel.
- Load trucks in a way to minimize segregation.
- If hauling long distances in cool weather, insulated trucks are desirable.
- Tarps that are in good shape should be used to cover loads during hauling.

11. SURFACE PREPARATION

11.1 General

Airfield HMA layers can be placed on subgrade soils (full-depth asphalt pavement), granular base course (new construction), existing HMA surface, or existing portland cement concrete pavement. The performance of the HMA pavement is definitely affected by the surface preparation. For full-depth HMA pavements, the subgrade soils must be strong enough and firm enough and demonstrate acceptable stability under heavy construction traffic. Granular bases should also be strong and stable and not yield under asphalt trucks or the paver. For overlay of an asphalt or concrete pavement, the existing paved surface should be swept clean, the cracks sealed, and failed areas repaired. A tack coat should be applied to the existing paved surface before the overlay is applied and between multiple HMA layers to ensure an adequate bond is developed between the HMA layers.

11.2 New HMA Pavements

11.2.1 Subgrade

Subgrade soils are the foundation of HMA airfield pavements. These soils must be placed and graded according to plan grade and slope. The subgrade soils must be uniformly compacted to the required density, appropriate moisture content, and adequate smoothness. Soft or yielding areas must be repaired and corrected prior to beginning paving operations.

For full-depth asphalt pavements, the subgrade soil must be firm, hard, and unyielding in order to achieve the required density in the HMA course. The subgrade surface should be free of loose particles and significant accumulations of dust. Surface moisture control is required to prevent dusting or cracking (too dry) or surface distortion (too wet).

A prime coat is typically not required for subgrade soils. Fine-grained subgrade soils (CL and ML) generally produce a very tight surface that will prevent the prime coat material from absorbing into the subgrade material. Prime coat applications should not be considered as a substitute for proper subgrade preparation.

11.2.2 Granular Base Course

The granular base should be constructed meeting all requirements for moisture content, density, and smoothness. The base course should also be proofrolled to demonstrate stability. The base surface should be swept clean of debris and accumulations of dust. Once the base course is stable and dry and does not yield significantly during proofrolling, a prime coat should be applied, especially if this layer is to be exposed to weather for an extended period prior to covering with a HMA layer.

11.2.3 Prime Coat

The prime coat is a sprayed application of cutback asphalt or emulsified asphalt. Prime coats are rarely placed on subgrade soils when HMA will be placed directly on the subgrade, but more often prime coats are used on granular bases prior to receiving the HMA layer.

The prime coat has four purposes:

1. Weatherproofing the base material
2. Stabilizing fines in the base material
3. Promoting bonding of the pavement layers
4. Preventing base from shifting under construction equipment

The asphalt distributor (figure 11.1) is used for spray applications such as prime coats and tack coats. The asphalt distributor is a truck or trailer mounted asphalt tank with pumps, spray bars, and appropriate controls for regulating the rate at which the asphalt is applied. The distributor is equipped with a heating system to keep the asphalt material at the proper temperature for application. Every nozzle on the spray bar must be kept clean, and the spray bar must be kept at the correct height to ensure proper coverage of the prime coat. The nozzles should be set to 15 to 30° to the horizontal axis of the spray bar, and each nozzle should be set at the same angle (figure 11.2). The bar height should be set so that the roadway receives single, double, or triple coverage (figure 11.3). Using a hand wand to spread prime coat should be avoided except in very tight areas where a truck cannot reach.

Figure 11.1 – Asphalt distributor

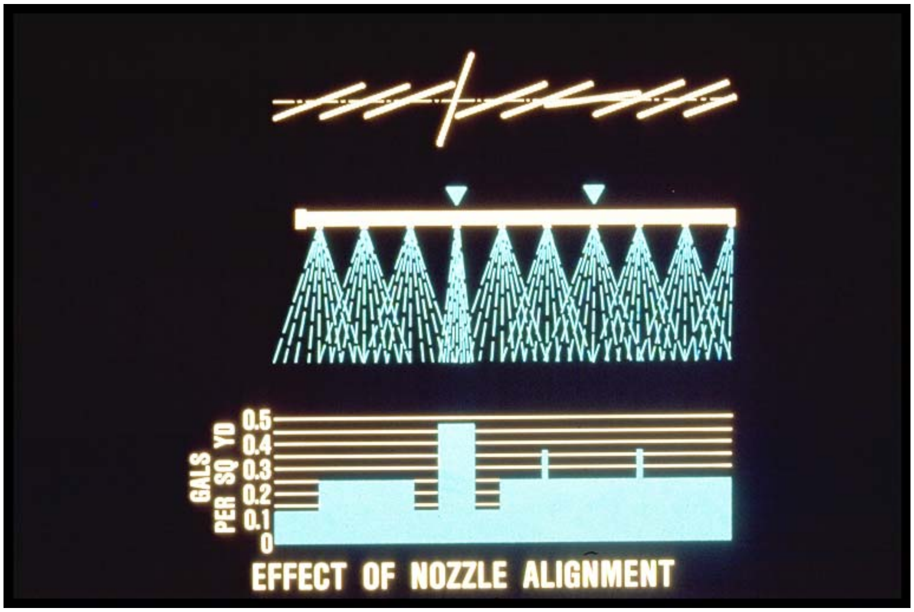

Figure 11.2 – Nozzle angle diagram

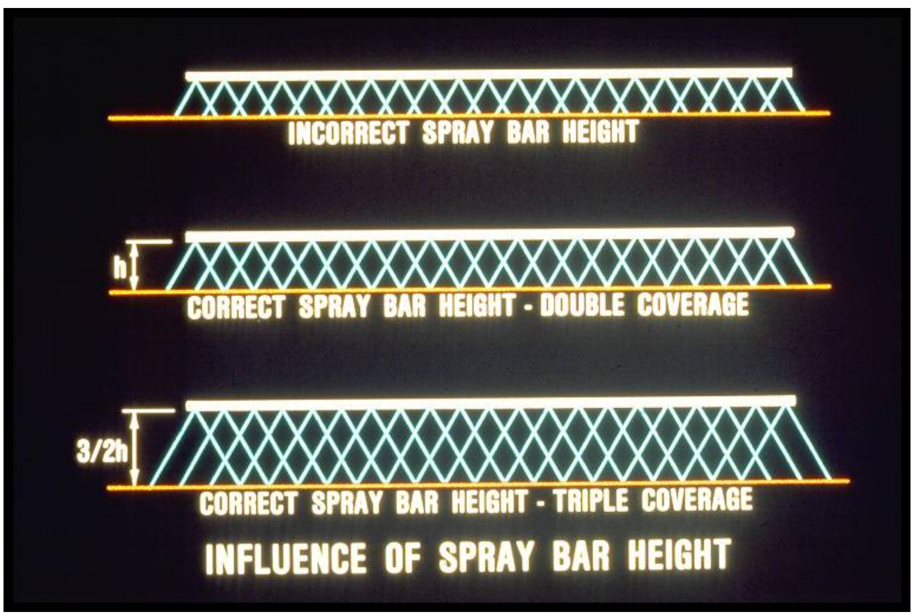

Figure 11.3 – Double and triple coverage diagram

The application of the prime coat should occur at least 48 hours before placing the HMA layer over it. No more prime coat material should be applied than can be absorbed into the underlying material in a 24-hour period. Precise application rates will be determined by the project engineer, but typical rates are 0.15 gal/yd^2 or less for tight surfaces and somewhat more for open surfaces. Excess prime will inhibit the bonding of the pavement layers, creating a slip plane in the pavement. Excess prime might also bleed upward through the asphalt pavement and result in bleeding or loss of stability. If too much prime coat material is applied, the excess should be blotted up with clean sand or screenings. When blotting excess prime coat, the aggregate material is spread over the desired area and allowed to absorb the excess prime. The area is then lightly swept to remove loose aggregate material.

Asphalt distributors should be operated correctly:

- All nozzles should be the same size, turned at the same correct angle to the spray bar, and in good shape.
- Spray bar height should be set at the proper height to give double or triple overlap.
- Coordinate truck speed and pump rate so that a proper flow of material is sprayed through the nozzles and the correct application rate is obtained.
- Spray application should be observed to ensure that a good spray of material is provided from each nozzle and that the sprayed surface looks uniform for the full width of the distributor.

11.3 HMA Overlays

The amount of surface preparation for an HMA overlay or placement of multiple HMA layers depends greatly on the existing pavement conditions. Structurally failed areas and highly distressed areas should be removed and replaced. All potholes should be properly patched. Large cracks greater than 3/8 in. wide should be cleaned and sealed. Airfield pavements with excessive rubber deposits and crack sealant should have these materials removed by suitable means prior to placing an HMA overlay. Existing airfield paint should also be removed prior to placing a new HMA layer. Milling (remove high points) or leveling courses (fill in low areas) may be required to re-establish adequate pavement grades and smoothness. Stress-absorbing interlayers such as geotextile fabrics and aggregate chip seals may be used to reduce and retard reflective cracks that occur and develop in the HMA overlay.

11.3.1 Repairs and Patching

Highly distressed failed areas and potholes should be removed and replaced prior to placing the new HMA overlay. Patches are required for pavement areas that exhibit alligator cracking, failed patches, depressions, shoving, slippage, and rutting. Each of these distresses is considered structural and requires permanent repair methods.

Full-depth patching involves removing all pavement structure materials that have failed. This could include the HMA layers, granular base and subbase courses, and the subgrade soils. The repair should extend to a depth that encounters strong and stable materials. For small localized failed areas, the pavement should be cut back into sound pavement (no distress) and squared up with vertical sides. Small areas can be patched using handwork and handheld compaction equipment. Large patch areas should be placed with a paver and compacted with standard rolling equipment.

Performance of patched areas is dependent on the following:

- Remove existing materials including HMA layers, granular layers and subgrade soils necessary to produce a firm foundation capable of supporting compaction of repair materials
- Edges of removed area should be straight and vertical
- Apply tack coat to vertical edges and bottom of excavation
- Backfill repair area in layers with suitable material and provide adequate compaction. Layers should not exceed 6 inches compacted thickness.
- Make sure patched area is flush with adjacent pavement and surface is smooth.

11.3.2 Milling

Milling is often used as a surface preparation technique prior to overlaying an existing asphalt pavement. Milling can remove minor surface irregularities, restore surface grades, allow an overlay to match an existing elevation, and provide opportunity for minimum clearance to be maintained in the paving process. The removal of high spots by milling can be in lieu of using a leveling course to fill in the low spots. Milled material is often transported back to the HMA plant where it will be recycled into a future HMA pavement.

The milling equipment available today can vary the width and depth of milling. The milling machine (figure 11.4) can create a level pavement surface with proper use of automatic grade and slope control, like that used on asphalt pavers. Various widths of pavement can be milled; existing equipment allows milling widths of 6 in. to greater than 12 feet. The depth of milling can range from ¾ in. to 14 in. Milling machines can also operate at a range of speeds; it is important to ensure that the maximum speed for a particular milling machine is not exceeded.

When milling is complete, the airfield surface will exhibit a textured surface that will provide a smooth ride. These grooves can help the overlay adhere to the milled surface as long the dust from the milling process is removed before the overlay is placed. Often, multiple passes of a broom are necessary to ensure that the pavement is sufficiently clear of dust. To ensure that an adequate bond is formed between the pavement layers, the percent tack coat will likely need to be increased slightly on the milled surface due to the increased surface texture. This increase in amount of tack coat is dependent upon a number of factors that are specific to the milling machine equipment, but 20% to 30% is typical. The increased amount of tack is necessary to promote a good bond between the two layers, but it should also be noted that too much tack can cause slippage between the layers or bleeding of the tack up through the surface of the pavement.

It is important to note that milling will only remove surface irregularities. Milling will not treat structural defects in the pavement. Therefore, if rutting is caused by the subgrade instability, milling the high sides of the rut to make the pavement level will likely only be a temporary fix to the rutting problem.

Figure 11.4 – Typical milling machine and drum with milling teeth

11.3.3 Paint Removal

Before overlaying an existing pavement, paint, rubber, and excess crack sealant should be removed from the surface of the pavement. Paint may also need to be removed for restriping or air traffic alignment changes. Rubber and excess crack sealant may need to be removed because of skid-resistance concerns. The procedures for paint removal are discussed in detail in chapter 17.

11.3.4 Rubber Removal

Rubber buildup can be a problem on runway ends where landing aircraft deposit tire rubber on the pavement upon impact with the pavement (figure 11.5). This occurs because the tires on the aircraft landing gear are virtually at rest just prior to landing and, upon landing, instantaneously accelerate to landing speed before the plane decelerates to a stop. This buildup of tire rubber on runways ends can cause friction loss or create a lack of bond if the pavement is to receive an overlay.

Rubber removal can be accomplished by many of the same methods as paint removal; however, waterblasting is the most common. Rubber removal should be as complete as possible without damaging the pavement surface. Eighty-five percent removal is typically specified for asphalt pavements.

Figure 11.5 – Rubber buildup on runway end

11.3.5 Excess Sealant Removal

Before pavements are overlain, any excess crack sealant must be removed. Excess crack sealant is material that has been used to fill cracks. In extreme cases, excess crack sealant can also cause friction-loss problems. Excess crack sealant can cause a bump to form in the overlay pavement (figure 11.6) during the rolling process. Excess sealer is removed during the milling operation if milling is required on the project.

Figure 11.6 – Bump caused by excessive crack sealant

11.3.6 Crack Sealing

If the cracking in a pavement is not severe enough to receive a patch or not frequent enough to merit an entire surface treatment, cracks may need to be sealed before receiving an overlay. Crack sealant is usually a hot-poured bituminous material that is injected into a crack to keep water from traveling through the crack.

If the crack is less than 1/4 in. wide, it is unlikely that the sealant will enter the crack and the overlay should proceed without sealing. If wider cracks exist, they should be cleaned to remove oxidized pavement, loose aggregate, and foreign debris before sealing. Cleaning the cracks will promote adhesion between the crack-sealing compound and the crack wall. Potential crack cleaning equipment includes:

1. Routing equipment
2. Concrete saw
3. Sandblasting equipment
4. Waterblasting equipment
5. Hand tools (wire brushing)

Routing should be accomplished with a router that is at least 1/8 in. wider than the crack. This is to remove residual sealant, oxidized pavement, and any loose aggregate.

Using a concrete saw to clean cracks requires that the blade be stiffened with washers or old blades. After the sawing operations, the crack should be cleaned using a water jet to remove sawcuttings and debris.

Cracks of different widths and depths require different treatment. Table 11.1 shows the proper procedures for hairline to large cracks. When the depth of a crack is greater than ¾ in., a backing material should be used to maintain a sealant reservoir depth of ¾ in.

Table 11.1 – Crack descriptions and cleaning requirements

Crack Size Designation	**Crack Width, in.**	**Crack Cleaning Requirements**	
		Sandblasting, Waterblasting, or Wire Brushing	**Compressed Air**
Hairline	Less than ¼	Sealing not required*	
Small	¼ to ¾	X	X
Medium	¾ to 2	X	X
Large	Greater than 2	Should be repaired using pothole repair techniques	

*Pavements that have frequent cracking may require a surface treatment.

Cracks that have been previously sealed may need to be resealed if the sealant is not performing. These cracks must be cleaned of existing sealant before they can be resealed. The same routing and/or cleaning techniques as described above can be used on these cracks.

Crack sealing can only begin when the cracks are clean and dry. Care should be taken not to entrap air while placing the sealant in the crack. The sealant should be recessed ¼ to ½ in. below the surface of the pavement if it is to receive an overlay. If no overlay is going to be placed, the sealant should only be recessed 1/8 in.

When the surface of the pavement is milled, it becomes difficult to locate and seal cracks unless they are fairly large. Typically, a crack will be wider at the top than underneath; therefore, any milling will reduce the size of the visible crack.

11.3.7 Brooming

When all needed repairs and/or milling are complete, the pavement surface will need to be cleared of dust and debris before the overlay is placed. This is accomplished with a mechanical broom. Brooming is necessary to promote bond between the overlay and underlying material.

11.3.8 Tack Coat

Tack coats are used to promote the bond between an existing portland cement concrete or HMA pavement layer and an HMA overlay. The tack coat is typically a sprayed layer of diluted emulsified asphalt, but cutback asphalt or asphalt cement can also be utilized. Diluted emulsified asphalt is preferred over cutback asphalt for environmental reasons. Cutback asphalts contain solvents that must evaporate for the tack to set up and be able to bond the two layers together. Generally when cutbacks or asphalt emulsions are used, the applied rate should be increased by approximately 10-20 percent to allow for the loss of cutter material or water.

Application of tack coat is very important for good performance:

- Always use tack coat on underlying layer when placing HMA.
- Use properly calibrated distributor in good condition.
- Select proper grade of asphalt to use—generally asphalt emulsion.
- Ensure that excessive water is not added to emulsion.
- Ensure that proper application rate is uniformly applied.
- Without good tack coat, slippage will likely occur during rolling or during traffic.
- Protect tack coat after application until covered with HMA.

The surface on which the tack coat is applied must be cleared of dust and debris by a broom prior to application. If the surface has been milled, multiple passes with the broom may be required to ensure a clean surface to which the tack can bond.

The application of a tack coat should generally occur just prior to the overlay of HMA. However, there have been a number of cases where tack coats have been left in place for a few days and still work successfully. It is important that the tack coat not get contaminated or worn off by traffic or construction equipment. If the tack coat has been worn, some additional material may have to be applied to ensure a good bond.

The amount of tack coat required for a pavement may be specified as a residual rate or application rate. The residual rate is the amount of tack remaining after the water evaporates. The application rate is the rate at which the tack is dispersed out of the distributor truck. The amount of residual asphalt in the asphalt emulsion and cutback asphalt will have to be determined based upon the amount of asphalt material left on the pavement after water and other volatiles have evaporated or been driven off. If straight asphalt cement is used, the residual rate is equal to the application rate. The typical application rate for tack coat is 0.05 gal/yd^2 for new pavement layers up to 0.15 gal/yd^2 for older oxidized, cracked, or milled asphalt pavements and portland cement concrete pavements. The project engineer usually determines the proper application rate. Too little tack will not provide a proper bond between layers resulting in slippage cracks, and too much tack could cause bleeding into the overlay mix as well as loss of bond. A poor application of tack coat is shown in figure 11.7. A good application of tack coat is shown in figure 11.1.

Figure 11.7 – Poor uneven application of tack coat

11.3.9 Reflective Crack Treatments

Currently, no rehabilitation techniques have been shown to “prevent” reflective cracks; however, there have been several methods and materials that have been able to “reduce” and “retard” reflective cracks. The latest information should be published shortly after this document was published.

Three methods have been used to reduce reflective cracking when overlaying an existing HMA surface with cracks:

- Geotextile interlay or band aid over cracks
- Asphalt rubber membrane interlayer (ARMI)
- Surface treatment or chip seal as a bond breaker

In most cases, there is not a need for a reflective crack treatment. This is especially true when the surface is milled. However, if there is significant cracking or when overlaying concrete pavement, some type of treatment to reduce reflective cracks is useful.

The most common reflective crack treatment used on airfield pavements in recent years is the geotextile interlayer. This rehabilitation method involves applying a heavy tack coat (~ 0.25 gal/yd^2) and a full-width geotextile fabric prior to the asphalt overlay. In recent years, there has been a tendency to use a band aid approach to cover the crack with a geotextile that simply covers the crack and is not placed the full width. This reduces the cost and reduces the possibility that performance problems may result due to the application of the treatment.

Besides reducing and/or delaying the reflective cracks, the asphalt-impregnated geotextile fabric produces a moisture barrier that controls surface water infiltration into the pavement structure. Geotextile fabrics can improve pavement performance when existing cracks are less than 3/8 in. wide. Reflective crack treatments typically perform considerably better in warm and mild climates than in cold climates. The typical HMA overlay thickness should be between 2 and 4 in. in order to realize significant improvements.

Surface preparation:

- Mill surface, if required to provide proper grade.
- Patch failed or damaged areas.
- Seal cracks.
- If applicable, apply reflective crack treatment.
- Clean surface.
- Apply tack coat.
- For unbound layers, apply prime coat.

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12. HANDWORK AND OTHER WORK AROUND LIGHTING FIXTURES AND OTHER UTILITIES

12.1 Introduction

Most HMA is placed with an asphalt paver; however, HMA can be placed by hand in situations where the paver cannot place it adequately. Generally, this occurs around the edges of paved areas, utilities, and lighting (although lighting can often be cut out), and around fillets between taxiways and runways.

Handwork should be minimized:

- It is very difficult to place mix by hand and achieve the same density, uniformity, and smoothness as when placed with a machine.
- Material should not be broadcast since this will result in segregation.
- Material placed by hand will have to be placed a little thicker since it will roll down during compaction (more than mixture placed with a machine).
- Handwork behind the paver to fill in deficiencies in the mat will result in loss in uniformity and performance. If pulling and tearing is occurring, solve the problem; don't try to cover it up with hand work.

12.2 Shovel Work

When necessary, HMA can be placed by shovel (figure 12.1). In general, it should be placed in full shovelfuls and not cast or thrown, called *broadcasting*. Broadcasting typically results in aggregate segregation, with the larger particles moving further or rolling away from the smaller particles.

Figure 12.1 – Handwork around concrete structure

All material should be homogenous and contain no chunks. Chunks of HMA that do not easily break apart should be removed and discarded.

Use safe work practices when obtaining HMA by shovel. The safest place to obtain HMA from the paver is in front of the screed on the side of the paver (figure 12.2). While the hopper may seem acceptable, this area usually contains backing trucks whose drivers have limited visibility of the hopper.

Figure 12.2 – Obtaining HMA in front of the screed on the side of the paver

12.3 Lute and Rake Use

Once HMA is placed by hand, it should be prepared for compaction by spreading and smoothing the surface with a lute (figure 12.3). A lute has a smooth surface on one side with teeth on the other side (figure 12.4). The lute can also be used to push the freshly placed hot material at a joint off of the cold side and into the joint, called *bumping*. Bumping is often used on longitudinal joints to build up a small ridge of mix for the roller to compact (figure 12.5). On some jobs where joints are cut back after paving, handwork is not needed on the joint because it will later be removed.

Lute and rake precautions are:

- Check the handworked surface with a straightedge or template before rolling to ensure uniformity.
- Minimize working time with the lute and/or rake. Excessive surface working can result in segregation leading to a rougher more open surface and can also lead to loss of temperature making the mixture difficult to work with.
- Segregated materials resulting from handwork should be removed from the HMA mat entirely along with any other excess material.
- Account for compaction when placing HMA in its loose state. As a general rule-of-thumb, HMA compacts about ¼ to ⅜-in. for every inch of mat thickness. Therefore, if the final mat thickness of 1 in. is desired, place about 1¼ to 1⅜ in. of loose HMA. Sometimes this is referred to as "fluffing up" the mix.
- Do not broadcast material with a lute.

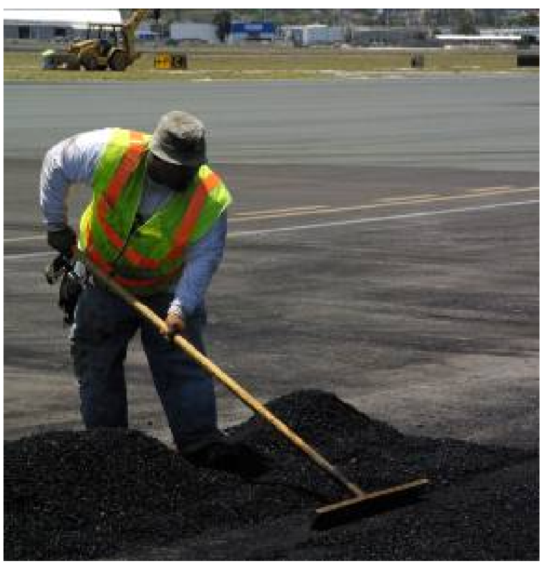

Figure 12.3 – Using a lute

Figure 12.4 – Lute with a smooth edge (left side) and a rake edge (right side)

Figure 12.5 – Bumping the longitudinal joint

12.4 Work around Fillets, Light Fixtures, and Utilities

Airfields typically use fillets between runways and taxiways and embed lighting fixtures into the pavement for aircraft guidance. Tight fillet radii and embedded objects present a unique paving challenge.

12.4.1 Fillets

Intersections between runways and taxiways use fillets to provide a smooth curved pavement edge (figure 12.6).

Figure 12.6 – Chicago's O'Hare International Airport showing runways and taxiways and filets at connection points (image from Google Maps, 2007)

Fillets present a special paving challenge because airfield pavements are normally paved in long straight pulls. In most instances, fillets can be paved by gradually extending the screed on the fillet side, while paving in a straight line; this is often called *winging out*.

Fillet edges constructed in this manner are likely to be less than perfect; however, handwork on the edge, final grading near the edge, and adequate compaction can usually make them quite presentable. If necessary, edges can also be sawcut.

12.4.2 Light Fixtures

Airfields equipped for low-visibility operations use lights to provide visual guidance to pilots for takeoff, landing, and taxiing. Lights are usually at the edge of paved areas or embedded in paved areas. Those embedded in the pavement area are usually flush with the final pavement surface and can be difficult to pave around.

Light fixtures (figure 12.7) that are spaced apart so that each fixture is an individual unit (e.g., runway centerline lights) can generally be paved over by fitting the fixture with a sacrificial cover (e.g., plywood cut to shape) and paving over it. The fixture can be located with a global positioning system (GPS) device prior to paving over it. After paving, the fixture can be located using the recorded GPS coordinates and cored to expose the fixture. The light can then be installed and shimmed up to the proper elevation.

Figure 12.7 – Light fixture on a taxiway shoulder

Groups of closely spaced light fixtures (e.g., guard lights) are often set in a portland cement concrete (PCC) base. Arrangements are most easily constructed when the PCC base extends up to the final pavement elevation. In this case, the taxiway can be fully paved with HMA, and then a slot can be sawcut for the PCC base and light fixtures. Arrangements are more difficult to construct when the PCC base does not extend up to the final pavement elevation. In this case, the PCC and fixtures must be constructed first, and then HMA must be paved in between the light fixtures to reach the final pavement elevation. Since the fixtures are closely spaced, there is generally not enough room to use a roller between fixtures. Therefore, fixture areas are typically compacted using handheld plate compactors, which may result in lower compaction levels and more roughness in these areas.

12.4.3 Utilities

In addition to light fixtures, utility access can also be located in the pavement. Items like manholes can be set in a concrete base that is slightly bigger than the manhole itself. When milling existing pavement, these utilities must be carefully located and skipped over with the milling machine. Options for removing material near these utilities are:

1. Hand removal. Jackhammers (figure 12.8) can be used to remove HMA close to utilities.
2. Small milling machines. Some small milling machines (figure 12.9) can be maneuvered around utilities, which can save time over hand removal.

Figure 12.8 – Jackhammering HMA around a manhole

Figure 12.9 – Small milling machine

Paving around utilities and other obstructions requires handwork and the use of hand-operated compaction equipment.

> Compaction next to manholes, light fixtures, and other structures adjacent to the HMA can be difficult. Keep steel-wheel rollers off of these structures, especially when vibrating. It is generally easier to roll the adjacent HMA mix when rolling parallel to the edge of the adjacent structure. Good density is especially important here, otherwise water will get into the voids and raveling will eventually occur. Use of small compaction equipment that is easier to maneuver may be required. Use of rubber-tire rollers may also be helpful in these areas. Control segregation in these areas, otherwise raveling will result.

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13. LAYDOWN

13.1 Introduction

The objective of the laydown operation is to place the HMA to the desired thickness or elevation, with a uniform surface without segregation, and without any pulling or tearing. The asphalt paver is designed to do this if operated correctly. A schematic of a typical paver is provided in figure 13.1.

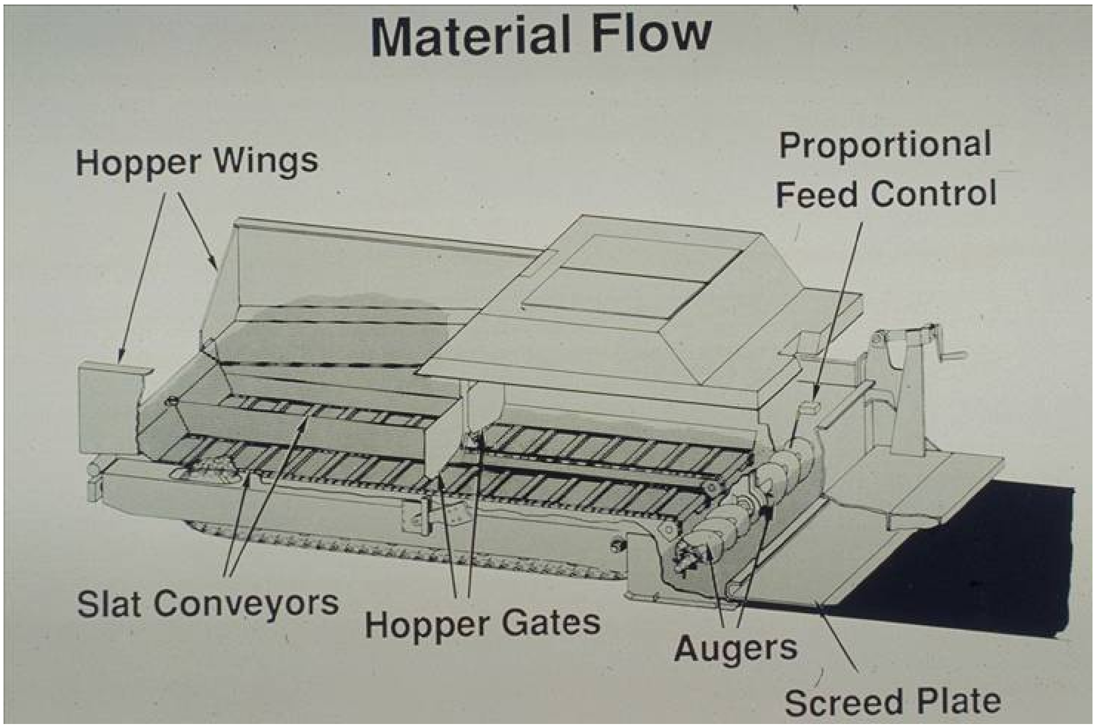

Figure 13.1 – Schematic showing material flow through a paver

13.2 Condition of Equipment

Equipment needs to be in good operating condition. The bottom of the screed needs to be in good condition with no pits, no build up of old asphalt, or any other problem that would cause the surface of the mix to pull and tear. The bottom of the screed should be plane with no warping, excessive wear, etc. The screed can be checked to see whether it is plane by raising the screed and pulling a stringline underneath it. The stringline should show that the screed is plane or the screed should be adjusted to make it plane. Sometimes, there is a crown set into the screed. The crown is not generally used on airfields, so usually the paver should not be set to pave a crown.

If there are extensions, they should be in good shape and should produce the same surface texture as the rest of the paver. Extending the screed without extending the augers is not acceptable for airfields. The end of the auger should generally be between 1 and 2 feet from the side plate at the end of the screed. If the space between the end of the auger and the end plate is too great, this can lead to segregation and low compaction at the edge of the lane/longitudinal joint.

The auger has to feed material underneath the gearbox (figure 13.2), otherwise a crack may eventually occur in the centerline of the paver. This deficiency in material underneath the gearbox will lead to a streak being formed in the pavement surface. During construction, make sure that there is not a streak occurring in the mix at the center of the paver. This will almost certainly lead to a crack at that location at some future date.

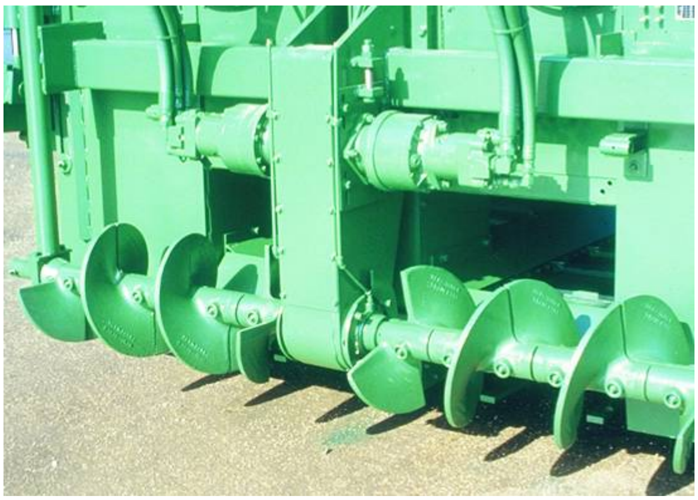

Figure 13.2 – Gearbox at center of auger

13.3 Transfer of Material from Truck to Paver

There are generally three ways material can be transferred from the truck to the paver:

- Dump directly into the paver.
- Place in windrow and then use pick-up machine to add to the paver.
- Feed into material transfer vehicle, which feeds material into the paver.

Dumping the HMA directly from the truck into the paver is probably the most common way of placing HMA. Of course, the advantage of this procedure is that no additional equipment is needed, but there are some potential issues. If these issues (potential for segregation and roughness, for example) are solved, then this procedure can produce a satisfactory product. Some of the disadvantages of dumping straight into the paver include:

- If the paver has to change locations, or if it is otherwise busy unloading other trucks, each truck will have to hold its HMA until its time to dump into the paver.

This results in potential cooling of the mix and a delay in getting the truck back to the plant.
- The trucks tend to bump the paver when backing into position, or resist movement of paver if brakes are pressed too hard during unloading operation.
- Segregation with some materials is a problem when dumping material straight into the paver. End-of-truckload segregation is a significant problem on many projects.

The windrowing method (figure 13.3) is often used in the western part of the United States. The advantage of this method is that it reduces trucking costs because the material can be placed on the surface being paved and does not have to be kept in trucks until loading into the paver. Some disadvantages of this method include:

- Quantity of material in windrow must be closely controlled otherwise some material will have to be removed or added later.
- Some segregation can occur as the material is windrowed.
- If windrow gets too far ahead of paver, mixture can cool prior to being paved, and significant material may be lost if rainfall begins or if there is equipment breakdown requiring that the paver be stopped.

Figure 13.3 – Placement of HMA using windrowing process

The preferred method of placing HMA is to use a material transfer vehicle (MTV) to collect the material from the loaded trucks and to load the paver. The quality of the placed mat is typically improved when this method is used. The MTV (figure 13.4) has been shown to provide the following improvements:

- Improved smoothness since paver can operate independently of trucks.
- Reduced aggregate segregation since there is some remixing with the MTV that minimizes segregation.

- Reduced temperature segregation due to remixing. This temperature segregation is caused by HMA cooling around the sides of trucks, especially during cold weather and on long haul distances. Insulated trucks will help.

Figure 13.4 – Use of material transfer vehicle (MTV) in placing HMA

13.4 Thickness of Placed Material

The loose asphalt mixture will compact under rollers reducing the initial thickness of the placed material. Typically, HMA will compact from 20% to 25%, depending on the type of mixture, the amount of initial compaction provided by the paver, and the amount of final compaction by the rollers. This has to be considered when setting the initial layer thickness, and this is very important especially when tying into other structures or when matching a longitudinal joint. If the material on the hot side of a joint is not placed thick enough, satisfactory compaction will be very difficult to obtain, typically resulting in low joint density, which is one of the major problems on airfield pavements.

When a transverse joint is constructed, the material on the hot side of the joint has to be thick enough to allow for compaction. The paver must be set up to begin with a thicker section. This is accomplished by placing strips of wood (figure 13.5), with thickness equal to the amount of compaction that will occur, underneath the paver when setting up the paver on the cold material.

Figure 13.5 – Strips underneath paver to provide sufficient thickness for rolldown

13.5 Construction of Longitudinal Joint

For airfields, it is essential that good construction of the longitudinal joint is obtained. (More information on construction of the longitudinal joint is provided in chapter 15). If inadequate density is obtained in and around the joint, cracking and raveling may occur resulting in FOD potential. Airfields, due to the width of their pavements, typically have a large number of longitudinal joints, making the quality of the construction of these joints even more important.

During the laydown operation, it is important that the paver provide some overlap of material with the adjacent lane. There may be disagreement about the amount of overlap, but there can be no argument that some overlap is needed. Without some overlap, it is impossible for the paver to continuously and exactly match the joint. This results in some areas having gaps in the place joint, resulting in a crack being built into the pavement surface. Typically, an overlap of somewhere between 1 and 2 in. is recommended.

The height of the screed at the joint has to be sufficiently high to allow for adequate material to be placed so that when rolled down even with the adjacent lane adequate compaction is obtained. This requires that the thickness of the new lane, when placed, be approximately 20% (this will vary with different mixtures but this is a good approximation of what the rolldown will be) greater than the thickness of the adjacent compacted lane. If sufficient thickness is not provided, there is no way that adequate compaction can be obtained when rolled smooth.

After placing the new lane, the material that overlapped the cold mat should be raked off the cold side and onto the hot side directly in the joint (figure 12.5). The material should not be broadcast across the mat since this will result in some of the coarse aggregate segregating on the surface that may be dislodged at a later date.

13.6 Smoothness and Grade Control

Smoothness is typically controlled when placing HMA by keeping the paver moving, using an MTV, and using a ski on one side of the paver and a joint matcher on the other side of the paver, if there is an adjacent lane that has been placed. Keeping the paver moving is very important since any stopping and starting will typically result in a bump that cannot be rolled smooth with the rollers. The MTV facilitates keeping the paver moving. Without the MTV, the paver will have to stop and start when changing from one truck to another and may be bumped some when it engages the truck. With the MTV, the paver can keep moving independently of the trucks, thus resulting in a significantly smoother pavement.

The ski (figure 13.6), which is typically 30 to 40 feet long, helps to take out the effects of dips and bumps in the smoothness of the finished surface. This ski is used as a reference for the paver to match, and it tends to bridge over low spots and take out high spots. This use of a ski will result in an improvement in the overall smoothness of the finished mat.

Figure 13.6 – Ski for improved surface smoothness

The joint matching shoe is used to match the edge of a previously placed mat. The paver follows the signal of the joint matcher, resulting in a constant difference between the top of the newly paved surface and the previously compacted surface. As mentioned earlier, it is important that sufficient thickness of material be placed to allow for adequate compaction, but not so much material that does not allow for the material to be rolled down plane with the adjacent lane. The joint matcher can consist of a sliding shoe a few inches long that slides along the adjacent pavement and allows the paver to match the adjacent joint, or an ultrasonic sensor device can be used to do the same thing without having to touch the adjacent pavement.

There are three ways that have been used to control the elevation of the completed surface. When placing new pavements, or when placing overlays over surfaces that need to have the grade improved (due to drainage issues or other problems), positive control of surface elevation is important.

There are three ways that have been used to control the surface elevation:

- Match established stringlines set to proper grade (figure 13.7).
- Use laser to establish plane to follow
- Use design elevation and GPS to build pavement to desired elevation.

Figure 13.7 – Use of stringline to control grade

Some have attempted to control grade by matching one side of the paver to a stringline or completed surface and setting the slope-control device within the paver. The slope can be checked with a carpenter level by pasting something underneath the level such as a coin on one end to provide the required slope when the level is lying flat on the pavement surface. This procedure can be done on projects that are not very wide, but for wide areas like parking aprons, taxiways, and runways, using slope control with the paver is not acceptable. There is always some error in the outside edge when the slope-control procedure is used, and each lane that is placed simply builds in more error than the first when this procedure is used. For one lane this error is typically acceptable, but for multiple lanes this is generally not accurate. It will result in the latter lanes being placed having too much error.

13.7 Paver Speed

The paver speed should be adjusted so that the material is being placed at approximately the same rate as the material is being produced at the asphalt plant. This will allow the paver to continue to move and will not result in a lot of stopping and starting, which leads to bumps in the surface. Most pavers are designed to place a relatively large amount of material per hour, of course, depending on thickness etc., so keeping up with the production at the asphalt plant is usually not a problem. The problem that occurs most often is for the paver to place material considerably faster than the plant, resulting in the paver having to stop and start often, which leads to increased roughness of the surface.

If the volume of material produced by the plant or plants is too large, then an additional paver may be needed. The paver should not move so fast that HMA cannot be placed with a uniform texture, consistent initial density, and level surface. When paving too fast, there is a tendency for tearing and pulling, as well as segregation due to the auger having to throw the material to the sides of the paver. In general, paving too fast results in a nonuniform surface.

13.8 Weather Conditions

The best work with HMA can be obtained on warm to hot days with no rain and no wind. This allows the mixture to maintain its heat longer, resulting in a very uniform surface during placement and better opportunity to obtain adequate compaction.

HMA should not be placed during rainfall. A few drops of rain do not cause a problem, but it is very difficult to clearly define when the amount of rainfall is excessive. Mixture can be produced and maintained in a silo during rainfall, and this is not a problem. This will hold heat for a reasonable amount of time, and the mix does not come in contact with water.

When water begins to accumulate on the surface, it is best that paving cease. The water can cause the mix to cool quickly, result in loss in bond, and cause stripping of the lower portion of the layer being paved.

The mixture is typically protected while in trucks with the tarps that cover the trucks, so a quick shower does not typically result in a significant problem unless there is a high amount of rainfall or for a prolonged amount of time. The water can leak in around the tarp and, with time, cause some problems with the mix. However, once the material is delivered from the truck to the paver, windrow, or material transfer vehicle, the material begins to quickly become wet during rainfall, resulting in more rapid cooling of the mixture and general loss of quality due to the moisture in the mixture.

When a quick shower does begin to occur during the paving operation, the material being placed should be quickly placed and compacted, and no additional material should be added to the paver until the rainfall has stopped and until the surface is reasonably dry. The material can be kept in trucks for an extended period of time before it has to be placed. However, once it is placed, it begins to cool quickly and absorb more water. This can result in a significant loss in performance.

Paving during windy conditions can be a problem. The wind does not directly affect the quality of the mixture, but it causes the mixture to cool faster, making it more difficult to place smoothly and obtain adequate density. The wind will likely make it more difficult for the crew to safely perform their jobs, especially during high winds. The wind has little effect on the cooling rate of the mixture when the mix is kept in trucks, but once placed in a thin layer, the wind can quickly cool the mixture to a temperature that results in loss in mixture workability and compactibility. During windy conditions, the number of rollers may need to be increased to provide the amount of rolling necessary to obtain satisfactory density before the mixture cools to an undesirable temperature.

Paving during cold weather can have a similar effect as paving during windy conditions. The major problem with paving in cooler weather is the rate at which the HMA cools during placement and compaction. Obviously, thinner layers of HMA will cool faster than thicker layers, so the effect of temperature is more significant for the thinner layers.

Some judgment has to be used when deciding when the weather is bad enough that paving should not be done. Typically, according to specifications, the air temperature has to be somewhere above 40 °F before it is recommended that paving begin. However, on a windy, cloudy, damp day, the minimum temperature at which paving will be done should be even higher. If there is a need to pave during low temperatures and windy, damp, conditions, then more rollers will almost certainly be needed otherwise adequate density will not be obtained.

13.9 Tying into Adjacent Pavements

There is often a need to tie into adjacent pavements or structures when placing HMA. One example is when a keel section on a runway is cut out of the existing pavement and repaved at some greater thickness than the original pavement. This results in the new pavement section being somewhat higher than the existing pavement that it has to tie into. There have been many projects built where the mix in this center keel section is tied into the old existing pavement on the sides by tapering out from some typical thickness, say 2 in. down to zero thickness. This has typically resulted in difficulty obtaining density in the section that has been tapered and eventually results in raveling of the area. Feathering from some thickness to zero thickness should not be allowed except in unusual situations.

Any time there is a need to tie a new overlay or new pavement into some existing pavement, the existing pavement needs to be milled so that a minimum thickness of at least 2 times the maximum aggregate size is provided for all mix placed. This method will result in higher density and much better performance of the material tying into the old existing pavement.

13.10 Segregation

One of the biggest problems that results in loss of HMA performance is segregation. This is a problem that can be seen during the placement of HMA, and steps need to be taken as soon as observed to identify the potential causes of the problem and to correct the problem.

There are two types of segregation that occur. The first is the separation of the coarse aggregate from the remainder of the mixture, resulting in some material in place only having coarse aggregate and a relatively small amount of asphalt binder. This results in the compacted mixture having a large amount of air voids in the segregated area. Most often segregation occurs at the end of truck loads but can occur at other times as well.

The other type of segregation is temperature segregation. This results when the material is hauled from the plant to the job site. During this haul, the material around the sides, top, and bottom of the truck bed cool more than the remaining material, especially during long haul distances. This cool material results in localized locations of asphalt mixture in the paved mat having lower temperature than the rest of the mat. This temperature segregation is caused by the cooler material not remixing with the warmer material when fed through the paver. This cooler mixture results in localized reduced density in the compacted mixture and eventual performance problems due to this low density. Since the lower density material is in small random areas, the loss in performance will likely be the result of raveling of the material in these random areas. Of course, this raveling could result in FOD issues.

The amount of both types of segregation can be reduced by using good paving practices. For the coarse aggregate segregation, this can be minimized by good stockpiling practices, proper use of storage silos, good truck-loading techniques, use of material transfer vehicle, and proper use of paver. The paver hopper should keep a significant amount of material in it at all times, and the wings should not be closed more often than needed. This type of segregation typically shows up at the end of truckloads.

For the temperature segregation (figure 13.8), there are several steps that can be used to reduce the problem. These steps include not hauling HMA for excessive time, using insulated trucks, and using a material transfer vehicle. So, during the laydown operation, the one step that will minimize this problem for both aggregate and temperature segregation is the use of a material transfer vehicle. Experience has shown this to be very effective in reducing segregation.

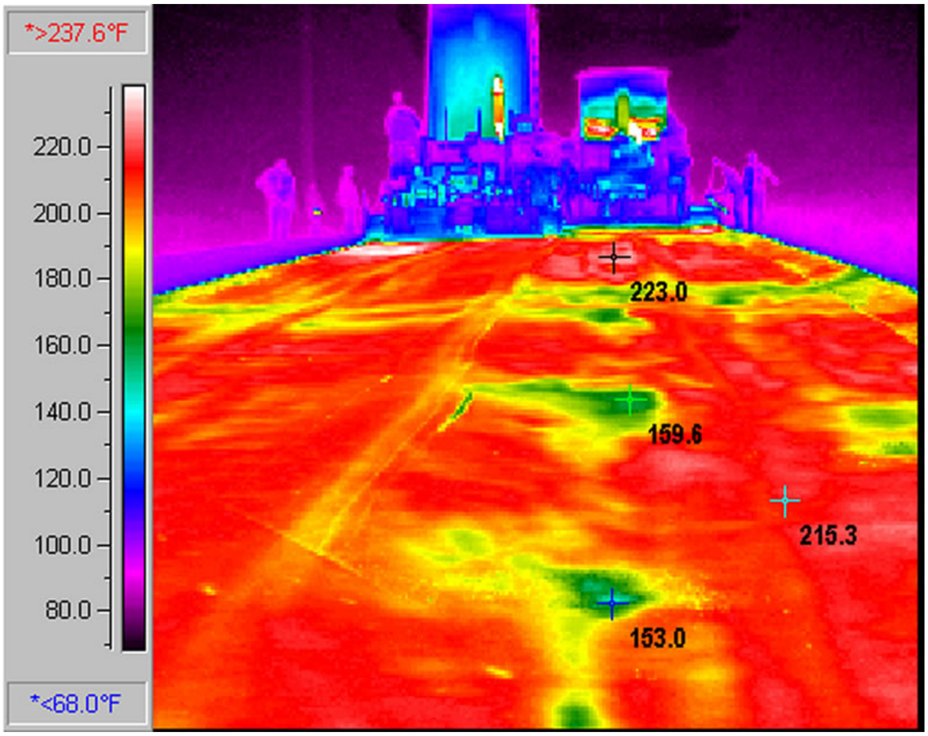

Figure 13.8 – Example of temperature segregation

13.11 Contamination

Care must be used during the laydown operation to ensure that contamination of the mixture does not occur. There are several sources of potential contamination including loose HMA that has cooled, segregated, and been thrown back into hopper; use of diesel to clean equipment and tools; and use of improper release agent in the trucks. This material that has cooled, become contaminated with tack coat and other materials, and likely segregated can result in loss in performance. Care must be used when cleaning tools with diesel. This should be done outside the area being paved, and all diesel should be removed from the equipment before coming in contact with the HMA. Diesel should definitely not be used as a release agent to cover the beds of trucks. There are a number of release agents available that work fine and that are not detrimental to the asphalt mixture.

Placement of hot mix asphalt should accomplish the following:

- Proper grade
- Uniform density throughout the HMA
- Uniform materials—no segregation
- Good smoothness

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14. COMPACTION

14.1 Introduction

Compaction is a very important part of the overall process. Without adequate compaction, asphalt mixtures will most certainly perform poorly. This section will provide some guidance on the types of rollers available and how these rollers should be used to obtain adequate compaction.

Low compaction can cause:

- Decreased mixture strength
- Increased oxidation and loss in durability
- Increased permeability to air and water
- Additional compaction under traffic resulting in rutting

14.2 Types of Rollers Available

There are a number roller types, and additional rollers with improved characteristics are routinely being develop by the compaction equipment companies. However, at this point, the primary rollers that are used for HMA include static steel-wheel rollers, rubber-tire rollers, and vibratory steel-wheel roller. There are other roller types including intelligent compaction, vibratory rubber-tire rollers, and steel-wheel rollers with a tensioned rubber belt around the two steel wheels. Most of the discussion that follows, however, will relate to the three primary types of rollers that are widely used today.

14.2.1 Static Steel-Wheel Roller

The static steel-wheel roller (figure 14.1) has been used to compact HMA for many years. Of the three primary rollers, the static steel roller was the first to be used. The primary purpose of the static steel roller today is to roll out bumps; this is known as finish rolling. Other types of rollers are used for compaction, in most cases; this static steel-wheel roller is the last roller in the rolling train. Typically, these rollers weigh 10 to 12 tons. In some cases, contractors have used vibratory rollers in the static mode to roll out bumps, and these may do a very good job. However, the pressure applied to the HMA is typically significantly much higher with the conventional static steel-wheel roller than with the vibratory roller operated in the static mode.

Figure 14.1 – Static steel-wheel roller

14.2.2 Rubber-Tire Roller

The rubber-tire roller (figure 14.2) began to be used in the mid-1900s. This roller has been shown to provide good density without the shoving and checking that sometimes occurs with the steel-wheel rollers. The biggest problem that has occurred with this type of roller is the sticking of the asphalt mixture to the tires. There are a number of release agents that prevent or at least minimize this pick-up problem. HMA containing asphalt binder that includes some type of modifier is generally more likely to pick up than HMA that does not contain a modified asphalt binder. The roller shown in the figure has skirts around the tires to help keep the tires hot. When the tires get hot, they are less likely to pick up the asphalt than when they are cooler. These skirts are very important to use when paving in cooler weather.

Figure 14.2 – Rubber-tire roller with skirts

Rubber-tire rollers are typically used as the second roller in the rolling train. The first roller is some type of steel-wheel roller that is used to breakdown the HMA. Once the material has been initially rolled, it can then be rolled with the rubber-tire roller. Rubber-tire rollers have been used for breakdown rolling when the mix is tender and tends to move under steel-wheel rollers. One big advantage of the rubber-tire roller is that it does not move the mixture laterally when rolling, resulting in check cracking. It is generally believed that the rubber-tire roller will provide a tighter denser surface on the compacted HMA.

14.2.3 Vibratory Steel-Wheel Roller

In the late 1960s, the vibratory steel-wheel roller began to be used to compact asphalt mixtures. Initially, the vibratory rollers that were used to compact soils were tried, but they did not work very successfully. It was learned that for HMA the roller needed to have higher frequency and lower amplitude. Once this adjustment was made, these rollers began to be very successful at compacting HMA.

There are several variations on the vibratory steel-wheel roller. Some rollers can vibrate both wheels, and others can only vibrate one wheel. Also, there are some vibratory rollers that have one vibrating wheel with the other side of the roller containing a number of rubber tires. The thought is this roller can be used to provide steel-wheel compaction as well as rubber-tire compaction. Generally, this roller with rubber tires on one side and a vibratory drum on the other is not recommended. The asphalt binder sometimes sticks to the rubber tires resulting in problems, especially when the roller is used as the breakdown roller.

Figure 14.3 – Vibratory steel-wheel roller

14.2.4 Miscellaneous Roller Types

As mentioned earlier, there are a number of nontraditional rollers that have been used to compact HMA. One type of roller is the vibratory rubber-tire roller. The use of rubber tires is beneficial because it reduces the moving and checking of the HMA. The vibration is assumed to be useful since it should provide more force for the weight of the roller, and this vibration should help to knead the material together to a denser state. This roller has been used on some projects, but it has not been widely adopted.

A second type of roller is the steel-wheel roller that uses a tensioned rubber belt to cover the two steel wheels. The purpose of the belt is to reduce checking (development of hairline cracks). Experience has shown that it does reduce checking; however, it has not been widely adopted.

A third roller type that is beginning to see some use is the intelligent compactor. This type of roller has been used to compact soils, and some are beginning to use it to compact HMA. The roller measures the stiffness of the underlying materials as it is being compacted. The stiffness is related to the density. So, if the stiffness can be measured by the roller, this can be used to estimate the density. This equipment records the measured stiffness on a continuous basis and uses GPS to prepare a map plotting the stiffness results versus location. This map can provide color codes, showing areas with lower density and higher density. As the density increases, the angle of the vibratory force is changed from vertical to more horizontal, thus decreasing the damage that may occur to a stiffer mixture. This process works very well for soils, but there are some problems that are being investigated for HMA. One of the biggest problems is that the stiffness of the HMA changes with temperature, making it difficult to develop a correlation between stiffness and density. However, work is underway to solve this problem and to develop a correlation between temperature, stiffness, and density.

14.2 Operation of Rollers

Most contractors have developed roller patterns that work well for compacting HMA. When they begin a new job, the pattern may have to be modified slightly since the best method of rolling HMA will change from one mix to another. Some mixtures are more tender than others or stiffer than others, resulting in a need for modification of the rolling technique. The longitudinal joint is almost always rolled first, and then the roller moves to the far side of the paving lane and works back toward the longitudinal joint with each succeeding pass.

One problem that often occurs with rollers is a tendency to operate the rollers too fast. Generally, a rubber-tire roller can be operated at a faster rate of speed than a steel-wheel roller, but a general rule of thumb for all rollers is that they should operate at about walking speed. If the mix is rolled too fast, it will provide less compaction per pass and will tend to push and shove the mix, especially when stopping and starting. It is easier to achieve compaction when rolling at a slow rate of speed, and it is easier to produce a smooth surface.

Another common problem when rolling is the development of roller marks (figure 14.4), especially when turning, stopping, etc. These roller marks are difficult to roll out with the finish roller, and sometimes they eventually result in a crack occurring at the location of the roller mark.

Figure 14.4 – Roller marks

Most mixtures will support rolling with steel-wheel rollers for a certain number of passes, and then the mixture begins to move laterally with additional rolling. One must pay close attention to the rolling operation to ensure that excessive checking (figure 14.5) does not occur during the rolling operation. These hairline cracks are generally superficial, only going about 1/4 to 1/2 of an inch into the mixture. However, this checking can provide a place for more severe problems, such as raveling and full-depth cracking, to occur. When checking begins to occur, it can be stopped or at least minimized by using a rubber-tire roller more and reducing the amount of rolling with the steel-wheel roller.

The rubber-tire roller has been shown to actually help reduce the size of this checking, and in some cases to close up these surface cracks.

Figure 14.5 – Roller checking

For airfield pavements, it is recommended that the rubber-tire roller weigh approximately 4,500 pounds per tire and the tire pressure be set at approximately 90 psi. If the roller is lighter or if the tire pressure is lower, it will be more difficult to obtain adequate density. The rated size of the roller does not provide a good indication of the actual weight of the roller when it is being used. The rated rate considers that the roller is filled with ballast, and in practice these rollers are often not filled. So, the only way to know the actual weight of a roller is to weigh it.

14.3 Weather Conditions

When paving in cold weather, it is very important that the rollers stay close behind the paver so the material is rolled when hot and not after a significant amount of cooling. Increasing the mixture temperature to provide more time for compaction cannot solve the problem of paving in cold weather. The mixture temperature can be increased slightly, Compaction of HMA is easier to obtain in hot weather when there is little or no wind. However, there are many cases when the weather is not ideal but work must continue. When the weather is cold (say 40s and low 50s) and there is much wind, compaction methods will need to be different than for good weather. The primary difference between compaction in cold weather and compaction in hot weather is the amount of time available to compact the mixture. It is generally believed that once the mixture cools to about 175 ºF no additional compaction can be obtained. This number is somewhat arbitrary, but it is a reasonable estimate and is at least a good starting point for identifying the minimum temperature at which compaction can be obtained.

During cold weather, there is simply less time to roll the mixture before it cools to an unacceptable level. This may mean additional rollers are needed but a significant increase in temperature may damage the asphalt cement excessively, and it may make the mixture tender and difficult to roll until cooling to a lower temperature. Having to let the mix cool before rolling defeats the purpose of increasing the mixture temperature at the plant. In cold weather, the mixture must be rolled quicker without the rollers traveling faster. This likely means that additional rollers will have to be made available.

14.4 Tender Mixtures

Many times, mixtures are tender when trying to roll. There are a number of reasons for this including excessive moisture in the mixture, too much natural sand in the mixture, too much rounded gravel, mix temperature that is too high, and lack of good bond to underlying material. When the mixture is tender, this may require the method of rolling to be modified. For example, there have been times when tender mixes have been allowed to cool to a point that they were no longer tender and then rolled to meet density requirements. There have been times when a rubber-tire roller was used to solve the tender mixture problem. There have been times when the mixture design was modified slightly to solve the problem. In most cases, when density is not being obtained it is a rolling problem, and this is what needs to be changed.

One has to be careful in adjusting the mixture to solve a tender mix problem. This can be done, but most problems can be solved by modifying the rolling procedures. The typical solution for a tender mixture problem is to reduce rolling with steel-wheel rollers and increase the amount of rolling with rubber-tire rollers.

14.5 Inadequate Bond between Layers

There are many times when a lack of bond between layers will show up as a tender mix problem. There are also times when a tender mix problem will result in loss of bond between the two layers. So, when a tender mix problem occurs, one step to take is to ensure that there is a good bond between the layers. When there is poor bond, it will eventually likely result in slippage of the asphalt mixture (figure 14.6). When slippage occurs, it is almost always the result of the top layer slipping past the underlying layer.

Figure 14.6 – Typical slippage problem

When cutting cores for density, it is important to observe the bond between layers. If the layers separate with little effort, then the bond is not adequate. This may require that the type of tack coat be changed or more time provided for the tack to cure. Some agencies use an asphalt cement as the tack coat, and experience has shown that this works very well. So, if inadequate bond is obtained with an emulsion, one option is to use an asphalt cement.

If the bond is not adequate, it will be impossible to obtain density, and much checking is likely to occur during the rolling process. This will eventually lead to slippage under traffic and a need to repair the damaged area.

14.6 Layer Thickness

The thickness of the layer being compacted can have a tremendous effect on the amount of compaction that can be obtained. The thickness needs to be at least 2 to 3 times the maximum aggregate size, or at least 3 to 4 times the nominal maximum aggregate size to allow for good compaction. If the layer thickness does not meet these requirements, it is difficult to knead the materials together without excessive damage to the coarse aggregate. It is also important that the mixture be at least 1.5 in. thick to provide enough thickness so that the mixture will hold its heat for enough time to allow for adequate compaction. Mixtures being placed less than 1.5 in. thick will cool quickly, resulting in much difficulty obtaining density.

14.7 Rerolling to Improve Density

There are many cases where adequate density was not obtained initially, and one option that is sometimes considered is rerolling. In most cases, rerolling the mixture after several hours or days has little effect on the density of the mixture. Certainly, rerolling with steel-wheel rollers is not recommended since this can damage the mixture and the aggregate. If rerolling is done, it should be done with a rubber-tire roller only, and the weight and tire pressure must be adequate as discussed earlier for rubber-tire rollers. Sufficient testing must be done to evaluate the effect of rerolling. Remember that there is always variability in test results, and it is possible that any measured increase in density after rerolling is a result of normal variation in sampling and testing and not an actual increase in the pavement density. So, significant testing may be necessary to show that rerolling actually increases density for a given project.

Reasons for inadequate density:

- Inadequate rollers
- Inadequate rolling pattern
- Improperly trained operators
- Rollers moving too fast
- Cold windy conditions
- Poor tack coat
- Layer of HMA too thin
- Tender mix
- Harsh mixture with poor workability

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15. LONGITUDINAL JOINT CONSTRUCTION

15.1 Introduction

During the construction of hot mix asphalt (HMA) pavements, a longitudinal construction joint is formed when a lane of HMA is constructed adjacent to a previously placed lane of HMA. Because airfield runways, taxiways, and aprons can be very wide, there can be many longitudinal joints constructed on airfield projects (figure 15.1). Since there can be numerous longitudinal joints on a project, their performance is vital for the overall performance of the project. Unsatisfactory performance of longitudinal joints has been called one of the biggest problems for HMA airfield pavements.

Figure 15.1 – Aerial view of runway with many longitudinal joints

15.2 Typical Distresses at Longitudinal Joints

Typical distresses near longitudinal joints are cracking and raveling. Figure 15.2 illustrates the deterioration of a longitudinal joint due to raveling and a small pothole that was created from cracking and secondary cracking. As shown in figure 15.2, both cracking and raveling can lead to loose materials on the pavement surface. Loose materials can lead to foreign object damage (FOD), which can result in aircraft damage. In addition to FOD, severe cracking and raveling, if not properly maintained, can lead to aircraft operational control problems.

Figure 15.2 – Deterioration of longitudinal joint due to cracking and raveling

The primary cause of cracking and raveling near longitudinal joints is low density in the joint. Low density leads to high permeability, which, in turn, means that air and water can penetrate into the HMA layer and cause the joint to become brittle and weak. Thus, as the permeability increases, there is more potential for cracking and raveling. Therefore, construction of a quality longitudinal joint is directly related to good performance of the longitudinal joints.

15.3 Terms and Definitions

When discussing longitudinal joints, there are several terms that must be defined:

- Cold lane—During the construction of any airfield pavement, there is an initial lane that is placed. After placement of the lane, the paving train is "backed-up" to begin placement of the adjacent lane. The term *cold lane* refers to the HMA lane that was placed first. This lane has had the opportunity to cool and, thus, is referred to as the cold lane.
- Hot lane—The term *hot lane* refers to the second HMA lane that is placed subsequent to the cold lane (figure 15.3).
- Hot joint—The term *hot joint* refers to the circumstance where two lanes of HMA are placed and both lanes remain hot enough to allow adequate compaction to be obtained in the joint. When the mix drops below 175 °F, it is typically difficult to obtain density in the joint. An excellent example of a hot joint is when echelon paving is done. Another example is when short lanes are being constructed and the equipment can finish the first lane and set up and pave the second lane before the first lane cools excessively.
- Cold joint—Cold joints are longitudinal joints in which the temperature of the cold lane falls below some reasonable temperature, usually around 175 °F.

Figure 15.3 – Hot lane being placed next to adjacent cold lane

15.4 Construction of Quality Longitudinal Joints

Construction of quality longitudinal joints is not easy and takes a team of skilled people with construction experience. The team involves the paving foreman, paver operator, raker, roller operators, and inspector.

Best approach for obtaining good density in longitudinal joint:

- Compact free edge using good techniques.
- Remove 2 to 6 in. from edge with cutting wheel.
- Overlap 1 to 3 in. when second lane is placed.
- Pave thick enough adjacent to the joint to allow for rolldown (typically approximately 20%).
- Roll with procedures discussed in section on compaction.
- Specifications should clearly specify joint density and require that cores be taken directly in the joint to verify joint density.

Proper construction of longitudinal joints starts with construction of the first lane. When the cold lane is placed, there is at least one unconfined edge of HMA. The unconfined edge must be constructed in a straight line (figure 15.4). Edges that are not straight are difficult to match when paving the hot lane and significantly increase the potential for low-density areas near the joint. A stringline or other appropriate method should be employed to ensure a straight edge on the cold lane.

Figure 15.4 – Paving a straight line

Compaction of the unconfined edge should be conducted using a steel-wheel roller operating in either a static or vibratory mode. The edge of the roller should extend over the unconfined edge by roughly 6 in. (figure 15.4). This practice will ensure that the entire compactive effort of the roller is applied to the vertical face of the unconfined edge. Additionally, this practice will reduce the potential for HMA to shove sideways during the compaction of the layer. If the roller operates near the unconfined edge but does not overhang the edge, the HMA mix has a tendency to shove laterally under the roller, thus making it more difficult to obtain a reasonable density in the edge of the paving lane. Even by compacting the unconfined edge using this best practice, there will be a zone of HMA mix at the edge that has a lower density than the rest of the mat. How this zone of low density is handled is the key to construction of quality longitudinal joints.

Another good practice during flexible pavement construction is to stagger (offset) the longitudinal joints from the bottom of the pavement structure to the surface course. If longitudinal joints are placed on top of each other, there will be a greater tendency for cracking to occur in the joint. Both the FAA and DoD have a requirement that longitudinal joints be offset by at least 1 ft from the longitudinal joint of the underlying layer. Additionally, if possible, the longitudinal joint in the surface course should be at the centerline of the crowned pavement surface.

15.4.1 Longitudinal Joint Construction Techniques

There are a number of methods that can be used to construct quality longitudinal joints. However, it must be stated again that the key to constructing quality longitudinal joints is to have a knowledgeable construction team with a desire to construct quality longitudinal joints.

15.4.1.1 Echelon Paving

Echelon paving refers to the situation where two (or more) pavers are used (figure 15.5) at the same time. The two pavers are spaced closely such that the lane placed first does not cool significantly before the second lane is placed. In essence, this creates a hot joint and is considered a good method for constructing longitudinal joints. This process of using two pavers will effectively cut out 1/2 of the cold longitudinal joints. It is generally difficult for a single HMA asphalt plant to provide sufficient mixture to the paving process to keep both pavers supplied with HMA. This could lead to many stops and starts of the paving train, which will affect pavement smoothness.

Figure 15.5 – Paving in echelon

When sufficient HMA mixture can be supplied to maintain operation of two pavers, echelon paving is a good process to use to reduce joint density problems. The following are important issues related to echelon paving:

- Compaction of the longitudinal joint should be delayed until both pavers have placed mix.
- The distance that the screed end gate of the trailing paver overhangs, the first lane should be no more than 1 in. The end gate of the second paver should be set at the same level as the screed of the first paver so that the end gate does not drag mix placed in the first lane.
- No raking is required.
- Any rollers trailing the first paver but in front of the second paver should stay at least 6 in. away from the unconfined edge on the side of the second paver.
- Once the HMA from the second paver has been placed next to the unconfined edge, the roller trailing the second paver will compact the mix across the joint.

- When using echelon paving techniques and the best practices highlighted above, the density of the longitudinal joint is generally equal to the density of the adjacent mat.

Standard practices for constructing quality longitudinal joints using the cut back method:

- The longitudinal edge of the cold lane is cut back a distance of 2 to 6 in. The DoD requires a maximum of 3 in. be cut away.
- For best results, the mix should be cut while the HMA is still plastic. In other words, the mixture should be around 120 to 140 °F when cutting takes place.
- The cutting wheel generally has a diameter of about 10 in., with the cutting angle being 10 to 20° from the vertical toward the mat. The back side of the cutting wheel should be beveled in order to push the trimmings away from the cut face. The cut should provide a clean sound vertical face for the full depth of the course.
- The cutting wheel can be mounted on an intermediate steel-wheel roller or motor grader.
- The trimmings should be removed prior to placement of the hot lane. These trimmings can be collected and used as reclaimed asphalt pavement (RAP).
- It is very important that an experienced skilled operator be used for the cutting wheel. Cuts must be straight, without wander (figure 15.7), and at the desired distance into the cold lane.
- An application of tack coat should be applied to the cut vertical face of the cold lane prior to placement of the hot lane.
- When placing the hot lane, the cold lane should be overlapped with mix by 1 to 2 in. The height of the overlap should be equal to the amount of rolldown experienced with the mix. Rolldown is generally ¼ in. for every 1 in. of compacted thickness.
- The overlap material should be luted off the cold mat and into the joint. The material should be bumped from the cold lane to *just* across the hot lane (figure 12.5). Extra material should never be broadcast onto the hot lane. Broadcasting extra material onto the hot lane results in loose coarse aggregate on the surface that will be rolled into the mat but that can pop out with time, resulting in FOD potential.
- One common approach to compacting the joint is to have the roller positioned over the hot lane with a 6-in. overhang onto the cold lane. The steel-wheel roller can be operated in either a static or vibratory mode. There are other acceptable ways to roll the joint, but this is probably the most common.

15.4.1.2 Cutting Back the Joint

When construction of hot longitudinal joints is not feasible, both the FAA and the DoD require cutting material from the unconfined edge of the cold lane. As described previously, the construction of an unconfined edge results in a zone of low density. The intent of cutting back the unconfined edge is to remove the zone of low density (figure 15.6).

Figure 15.6 – Cutting back free edge prior to placing adjacent lane

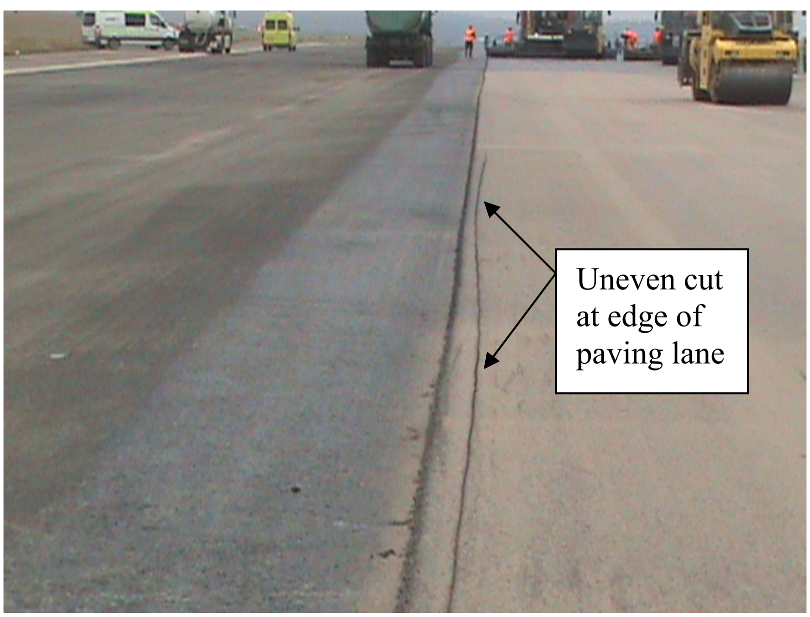

Figure 15.7 – Unacceptable cut of free edge

15.4.1.3 Other Techniques for Constructing Longitudinal Joints

Currently, the FAA and DoD only allow the use of hot joints or cut back joints for the construction of longitudinal joints. However, there are other alternatives that have led to good-performing longitudinal joints on highways. Following are some of these techniques, along with brief discussions. More detailed discussions on these techniques can be found in the final report for AAPTP Project 04-05 entitled, *Improved Performance of Longitudinal Joints on Asphalt Airfields*.

- Combination of notched wedge joints and rubberized asphalt tack coat (joint adhesive)—With this technique, a notched wedge (figure 15.8) is constructed in the cold lane. The notched wedge is generally created by an attachment on the end gate of the paver. The notch is generally ½ in. A small roller is generally pulled by the paver to compact the wedge (taper). After constructing the notched wedge, and prior to placing the hot lane, a rubberized tack coat is placed on the notch (figure 15.9).
- Rubberized asphalt tack coat—Use of a rubberized tack coat is similar to that described above, without the use of the notched wedge. In this practice, the joint adhesive would be placed on the unconfined vertical edge of the cold lane.
- Notched wedge joint—Similar to that described above, a notched wedge is formed at the unconfined edge of the cold lane. The notched wedge technique is not applicable for lift thicknesses of less than 1 ½ to 2 in.

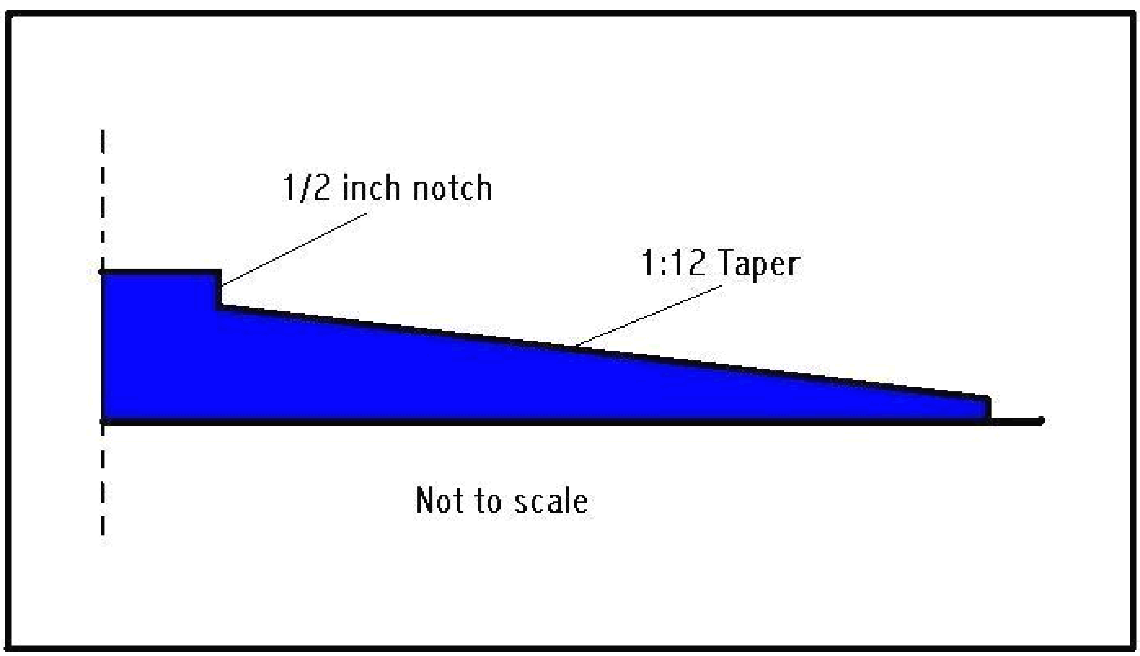

Figure 15.8 – Schematic of notched wedge joint

Figure 15.9 – Application of rubberized tack coat at notch

15.5 Density Requirements for Longitudinal Joints

Currently, both the FAA and DoD have density requirements for longitudinal joints. However, there are some differences between the two owner agencies.

Following are the recommended practices for density requirements:

- Density measurements should be made on cores that are cut centered on the joint.
- One core should be cut randomly from each sublot.
- In-place density should be based upon theoretical maximum density (TMD). The TMD used for calculating the in-place density should be the minimum or average (depending on specification) of the material from the two sides of the joint. Specifications should be clear on which value to use.
- A minimum in-place density of 92% should be achieved in the joint.

Loss of performance due to raveling and cracking in the longitudinal joints is one of the biggest problems facing airfield managers. Using good construction practices to obtain adequate density in the joints is the best single approach for preventing this problem. Following the methods provided here should help to ensure adequate density is obtained in the joints.

16. GROOVING OPERATIONS

16.1 Purpose of Grooving

Grooving enhances skid resistance and reduces the potential for hydroplaning. It is commonly required as a safety precaution on high-speed airfield pavement surfaces used by turbojet aircraft. In this role, it is appropriate for runways and high-speed taxiways, but is not needed on regular taxiways and ramps. It is required on both asphalt and portland cement concrete surfaces. Typical grooves are shown in figure 16.1. The actual size and spacing should be as outlined in the specifications.

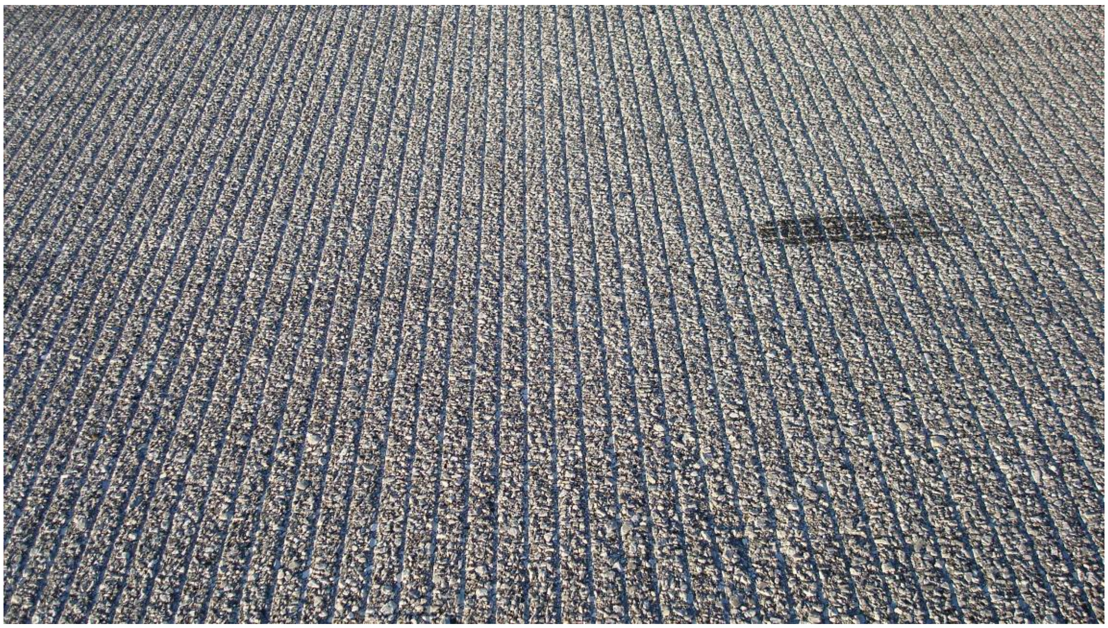

Figure 16.1 – Appearance of typical grooves after traffic

16.2 Characteristics of Grooved High-Speed Airfield Surfaces

Airfield surfaces are grooved transversely, in contrast to highway surfaces that are more commonly grooved longitudinally. The grooves for airfields are 1/4-in. wide, 1/4-in. deep, and spaced at 1 1/2-in. center-to-center spacing (figure 16.2). The width and depth of the grooves are generally allowed a tolerance of +/- 1/16 in., and the spacing is usually allowed a tolerance of +/- 1/8 in. for military airfields. Furthermore, the depth of at least 60% of the grooves should not be less than 1/4 in., according to FAA standards.

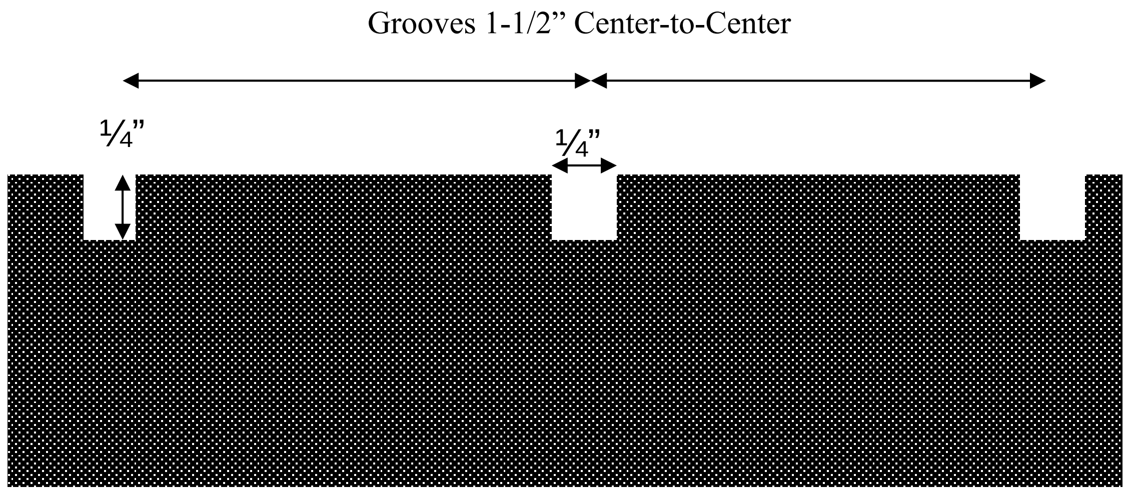

Figure 16.2 – Groove pattern

Grooves shall be perpendicular to the direction of aircraft traffic and continuous for the length of the runway. The grooves shall not vary more than 3 in. in alignment for 75-ft lengths along the runway, allowing for realignment every 500 ft along the runway.

Grooves shall be located no closer than 3 in. to transverse joints and no further than 9 in. from such joints. At the pavement edges, the grooves will extend to within 10 ft of the pavement edge and will terminate within this zone as required to allow efficient operation of the equipment. Grooving will not pass through saw kerfs that are placed for airfield lighting. However, they will continue through longitudinal joints.

The primary runway shall always be grooved in a continuous pattern. Intersecting secondary runways and high-speed taxiways will adjust their grooving pattern to not interfere with the standard pattern used on the primary runway. The FAA standards stipulate that the secondary runway and high-speed taxiway grooving at the intersection will follow a sawtooth step pattern.

16.3 Equipment and Procedures

The grooves are cut with diamond saw blades mounted on a cutting head (figure 16.3). Typically, the grooving machine will simultaneously cut grooves in an 18- to 40-in. transverse width in a single pass. The machine should be self-propelled and have a positive method of maintaining alignment as it cuts across the pavement width. The pavement must be continuously cleaned of all debris and dust during the grooving operation. The debris and flushed sediment from sawing operations shall not be allowed to enter storm-water drainage systems, natural waterways, or left on the airfield verges. Responsibility for and acceptable methods of disposal of this material should be clearly stipulated in the contract specifications.

Figure 16.3 – Grooving equipment

The hardness of the aggregate in the asphalt concrete will affect sawing operations and blade wear. However, general experience is that asphalt concrete can be grooved in a single pass, regardless of aggregate type. Lighter volatile oil fractions will evaporate from the asphalt concrete after placement, which will stiffen the asphalt concrete. Hence, various specifications require waiting 14 (e.g., Alaska airport specification) to 30 days (military airfields). The longer one waits, the less likely one is to encounter problems with raveling of cut edges.

16.4 Issues

Excessive raveling or aggregate dislodgement during initial cutting of the grooves may indicate worn or damaged saw blades, improper adjustment or speed of equipment, problems with the asphalt concrete mixture characteristics, poor asphalt concrete density, or inadequate curing of the asphalt concrete.

If problems are encountered during grooving, consider the following issues:

- In-situ asphalt concrete characteristics to see if any contributions to the problem can be identified (e.g., low asphalt cement content, low density, unusually hard aggregate or unusually soft asphalt cement used on the project).
- Condition and operation of sawing equipment to see if those can be adjusted to improve results.
- Examine curing conditions to see if this could be adjusted to improve grooving conditions (e.g., warm dry environment or longer cure periods favor loss of volatiles and stiffening of the asphalt concrete). Grooving may be easier to perform in early morning, late afternoon, or nighttime since the asphalt mixture will be cooler then and better able to resist the sawing action.

It is important to have an initial test section that establishes the expected condition of the grooves before full production grooving is underway.

A survey of common distresses found on grooved asphalt concrete runways is shown in table 16.1. Groove wear and closure are particularly unique to grooved asphalt concrete airfield surfaces. Assessments of these issues from several studies suggest that slow-moving heavy loads, sharp turning traffic, and warm climates all contribute to potential problems with grooved runway surfaces. Some improvement in performance of grooved surfaces has been reported when smaller maximum-size aggregates are used.

In general, grooved runways for the military have performed well despite heavy loads and use in extremely hot climates. Their asphalt mixes are quite stiff, similar to FAA airfield mixes. However, the military's standard detailing requires portland cement concrete in the 1,000 ft ends of the runways, and the grooved asphalt surfaces tend to be in the runway interior where it is exposed to relatively high-speed traffic.

Where slow-moving heavy loads may exist (e.g., intersection of primary taxiway and runway or runway ends) and especially in hot climates, there is an advantage to using an asphalt concrete mix with stiffer binder characteristics.

Table 16.1 – Common distresses found in a survey of grooved asphalt concrete runways ("Surveys of grooves in 19 bituminous runways" by R. Melone, FAA-RD-79-28, Federal Aviation Administration, 1979)

Distress	Description
Wear	Groove depth of 1/8 in. or less compared to standard depth of 1/4 in.
Groove closure	Groove width measuring 3/16 in. or less compared to standard of 1/4 in.
Rubber deposits	Rubber on grooves and on runway
Cracks	Reflective cracks propagated along grooves
Migration	Shoving, tender asphalt mixture resulting in a wavy pattern
Deep/shallow cutting	Adjacent grooves of varying depths from defective sawing methods
Rounding	Wearing away of the sharp groove edges
Spalling	Disintegration, breaking up of asphalt concrete surface
Chipping	Breaking away of aggregate or filler materials in sharp edges of grooves
Erosion	Washing out of fine filler or binder material, leaving exposed aggregate

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17. PAINTING AND STRIPING

17.1 Markings

Airfield markings (figure 17.1) are an airfield's temporary and permanent paint markings on all applicable runways, taxiways, and aprons. Markings are painted and must conform to Section P-620 of FAA Advisory Circular 150/5370-10B, which is the FAA's standard for the construction of airports.

Figure 17.1 – New striping at the Port of Friday Harbor airport in WA

17.1.1 Paint Type

Markings are usually applied after HMA paving and grooving (if needed) is complete. Markings are also periodically applied on existing facilities in order to maintain proper visibility. Markings can be applied by large automated trucks (figure 17.2) or small hand-operated sprayers (figure 17.3). In general, marking paint consists of three main components:

1. Pigment—provides color and opacity.
2. Binder—binds the pigment together and provides adhesion.
3. Vehicle (or solvent)—adjusts the viscosity of paint and is volatile, meaning that it evaporates and does not become part of the permanent paint film. The pigment and binder are carried in the vehicle.

The makeup of each of these components determines the characteristics of the paint. P-620 allows for four types of paint markings: waterbourne, epoxy, methyl methacrylate (MMA), and solvent-based point.

Figure 17.2 – Striping truck

Figure 17.3 – Hand-operated sprayer for temporary markings

17.1.1.1 Waterbourne

This paint is specified by Federal Specification TT-P-1952D, the U.S. government standard for latex (acrylic) paint. Like most latex/acrylic paint, it involves a pigment combined with an acrylic (plastic) binder and is carried in a water vehicle, thus the name "waterbourne." It uses ammonia and methanol as drying agents. It can dry to a track-free state in 2 to 15 minutes but requires 24 to 48 hours to fully cure. It comes in two basic types depending upon curing conditions: Type I is used for normal weather curing conditions, which P-620 defines as 50% relative humidity, moderate temperatures, and slight breezes. Type II is used for adverse curing conditions, which P-620 defines as night striping, higher humidity (around 80%), low air movement, and lower surface temperatures down to 12 °C (55 °F).

Advantages of waterbourne paint:

- It is relatively low cost when compared to epoxy or MMA markings.
- It is a nonhazardous material. It contains virtually no volatile organic compounds (VOC) and cleans up with water. Thus, disposal and cleaning is simple and relatively environmentally friendly.
- The highest coverage rate (115 ft^2/gallon) requires less time and paint.

Issues with waterbourne paint:

- It can potentially react with iron sulfide in HMA aggregate causing discoloration. It may be possible to add a rust inhibitor to the paint; however, this solution is unproven.
- Applied over a new HMA surface, the paint may discolor to a golden brown. It is likely that this discoloration will disappear over a short period of time, often assisted by rain.
- Paint could bond to the HMA surface sufficiently to build up stresses in the HMA during temperature changes. This could lead to cracks that are often observed along the edges of the paint markings as the paint expands and contracts with temperature changes.

17.1.1.2 Epoxy

This paint, which is actually a hard plastic, is specified by P-620 as "a two-component, minimum 99% solids type system." In epoxy paint, the pigment is set in a binder of thermosetting epoxide polymer that cures or hardens when mixed with a catalyzing agent to form a hard plastic. Thus, epoxy comes as two separate components that harden when mixed together.

Advantages of epoxy paint:

- It is more durable than waterbourne or solvent-based paint. Paint can be expected to last 5 years or more.

Issues with epoxy paint:

- Lower coverage rate (90 ft^2/gallon) requires more time and paint.
- Durability means that subsequent removal will require more time and could cause more damage to the underlying HMA.

17.1.1.3 Methyl Methacrylate (MMA)

This paint, which is an epoxy that uses methyl methacrylate as the epoxy resin, is specified by P-620 as "a two-component, minimum 99% solids type system." Polymerized MMA (a different product but close relative) is often sold under the brand name of "Plexiglas."

Advantages of MMA paint:

- It is more durable than waterbourne or solvent-based paint. Paint can be expected to last 5 years or more.

Issues with MMA paint:

- The lowest coverage rate (45 ft^2/gallon) requires more time and paint.
- Durability means that subsequent removal will require more time and could cause more damage to the underlying HMA.
- It is solvent-based.

17.1.1.4 Solvent-Based Paint

This paint is specified by Federal Specification A-A-2886A, the U.S. government standard for solvent-based paint. The pigment is combined with an acrylic binder and carried in a volatile solvent.

Advantages of solvent-based paint:

- The highest coverage rate (115 ft^2/gallon) requires less time and paint.

Issues with solvent-based paint:

- Generally the paint comes in 55-gallon drums subject to an Environmental Protection Agency (EPA) surcharge for VOC content. This can make solvent-based paints twice as expensive as waterbourne paints.
- Cleanup is done with undesirable volatile organics such as toluene and gasoline.
- If striping equipment is used for waterbourne paints, as is typically the case, the conversion to solvent-based paints and subsequent cleanup (so the equipment can once again use waterbourne paints) is labor-intensive, expensive, and creates hazardous waste.

17.1.2 Retroreflectivity

Glass beads (figure 17.4) are often added to improve night visibility because they are retroreflective (i.e., they send light back where it came from regardless of the original angle of the light). These beads have requirements for gradation, roundness, specific gravity, crushing strength, and index of refraction. Some markings are required to have glass beads, while other markings are only recommended to have glass beads. It is recommended that Advisory Circular No. 150/5370-13A be reviewed to determine when glass beads should be used.

Glass beads for airfield markings are typically either Type I or Type III. Type III is about 10 times more expensive per pound but has a higher index of refraction, which is supposed to provide better reflectance (figure 17.5). However, the air force studied Type I and III beads and found that the Type III beads lost reflectivity at a faster rate than Type I. However, the FAA and the air force indicate superior reflectivity with Type III.

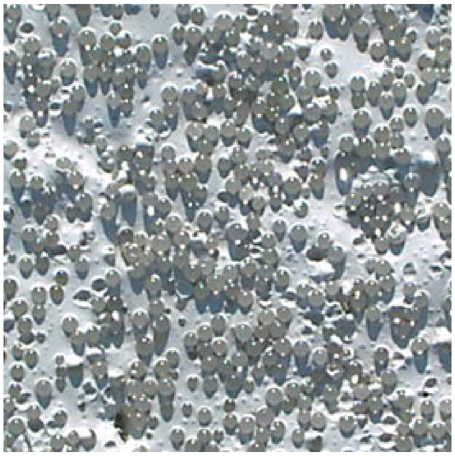

Figure 17.4 – Magnification of retroreflective glass beads embedded in marking paint (photo credit: Swarco)

Figure 17.5 – Type III glass beads in the arrowhead with a higher index of refraction (1.9) provide better visibility than Type I glass beads in the tail with a lower index of refraction (1.5) (photo credit: D. Spiedel)

Glass beads should be applied as soon as possible to fresh paint (figure 17.4). If the paint begins to cure and form a thin surface skin, the beads may not embed properly. Proper embedment is usually taken to be about 50% of the bead in the paint. Glass beads are generally added at between 7 and 15 lbs/gallon, depending upon paint type and glass bead type. Glass beads also provide friction for airfield markings. If glass beads are not used, silica sand can be used to improve friction.

17.1.3 Marking Application

Airfield markings are typically applied by a pavement marking contractor who will likely have experience in both airfields and roadways. Typically, markings are located by a separate surveying contractor, with the marking contractor often doing just the painting.

Low temperatures may inhibit adhesion to the HMA, while high temperatures may inhibit curing. Therefore, P-620 allows marking application only when the HMA surface is at least 45 °F and rising and surface temperature is at least 5 °F above the dew point. Marking application should also stop if HMA surface temperature exceeds 120 °F.

Airfield markings are generally applied more precisely than roadway markings. The FAA does not allow deviation from a straight line by more than ½ in. in 50 ft. If applying to porous friction course, use a 75% application rate in two directions. This helps paint multiple sides of exposed aggregate and allows paint to penetrate the large surface voids. Empty containers for paint and glass beads should be saved in order to verify amount of material and application rate.

17.1.4 Temporary Markings

The FAA recommends waiting 24 to 30 days after paving to apply permanent markings. Typically, this results in the final markings being applied after grooving operations.

Temporary markings can be applied so that the airfield can be used in the interim. Temporary markings are usually waterborne paint, do not require glass beads, and are applied at 30% to 50% of the specified application rate. The final marking application must still be at 100% of the specified rate to adequately set the glass beads.

Airfield markings that are no longer needed should be removed. FAA Advisory Circular 150/5340-1H, *Standards for Airport Markings*, states:

> "Pavement markings that are no longer needed should be physically removed by sand blasting, chemical removal or other means, not painted over. Painting over the old markings merely preserves the old marking, will require additional maintenance, and in certain conditions, can be misleading to pilots."

Additionally, markings should be removed prior to overlays or seal coats, and the surfaces should be properly prepared when they are to be painted over. Most paint marking contractors will have substantial experience with roadways and parking lots but may or may not have substantial airfield experience.

17.1.5 General Removal Guidance

The specification for marking surface preparation is often "85% of loose and flaking paint." This involves removing most of the loose and flaking paint but does not involve removing the old paint that is still adhered to the pavement. Owners and contractors should be aware of this distinction. If paint removal is desired, it should be specified as paint removal. A typical paint removal specification may call for waterblasting or milling to remove 95% of the poorly bonded paint from the existing surface.

Old paint should be substantially removed prior to an overlay or seal coat because it could result in a weak bond between the existing pavement and new HMA or surface treatment. Substantial removal is usually specified as 85% paint removal.

If new markings will coincide with the old, then removing 85% of loose and flaking paint should suffice. If new markings will conflict with old ones, then 90% to 95% of the conflicting paint should be removed.

If more than six layers of paint have been applied since the pavement was new, or since the paint was previously removed, consider removing 85% to 90% of the existing paint prior to repainting the markings.

Several different methods of paint removal exist and have been used satisfactorily. The goal is to remove existing paint while minimizing damage to the existing pavement surface. When removing paint from HMA, some degree of surface abrasion will happen, which usually leaves a visible scar (figures 17.6 and 17.7). Typically, this scar disappears over time. Newer HMA scars less than older aged HMA.

The basic methods of paint removal are:

- The most common method of removal, the grinding machine, has small grinding heads (figure 17.8) that scrape the pavement surface to remove paint. Grinding machines work well for old HMA surfaces with noticeable cracking, and they will not worsen existing cracks. Grinding's limitation is that since it scrapes the surface it cannot remove paint inside grooves or large cracks without removing a significant amount of asphalt mixture. Grinders leave an etched surface that will disappear over time.
- Generally, milling uses larger heads and produces deeper scars than a grinding machine. Milling machines are a good option when the entire surface will be overlayed because the surface condition is not as important.
- The least damaging method is sandblasting. It is generally slow, leaves sand that may have to be cleaned up, and requires operator protective gear to prevent inhaling the silica in the sand.
- Shotblasting is like sandblasting only steel pellets are used in place of sand. HMA is generally too soft for shotblasting, so it should generally be avoided.
- Waterblasting (figure 17.9) involves using high-pressure water, from 5,000 to 40,000 psi. Generally, lower pressures (below about 20,000 psi) should not be used on badly cracked HMA since the water will penetrate the cracks and, through hydraulics, expand the cracks and possibly move pavement chunks. Ultra high-pressure waterblasting (25,000-40,000 psi) will not further damage badly cracked pavement because the water is used at a much lower rate and acts more like a grinder or milling machine.

Figure 17.6 – Paint removal scar resulting from grinding new HMA (photo credit: D. Spiedel)

Figure 17.7 – Paint removal scar resulting from waterblasting old weathered HMA (photo credit: D. Spiedel)

Figure 17.8 – Grinder attached to a skid steer loader (photo credit: D. Spiedel)

Figure 17.9 – High-pressure waterblasting truck (photo credit: D. Spiedel)

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18. QUALITY CONTROL/QUALITY ASSURANCE

18.1 Introduction

Good QC/QA practices are necessary to ensure that good material is produced and the desired performance is obtained. This section outlines guidance on some of the details for QC/QA to ensure that a quality product is obtained.

18.2 Quality-Control Plan Specific to Asphalt Operations

One of the requirements of most specifications is that the contractor, responsible for providing quality-control (QC) testing, is required to provide a QC plan to the design engineer. This plan will provide details about how the HMA will be controlled during construction. Some items that should be included are testing for aggregate, asphalt content, mixture volumetrics, mix design, in-place density, grade, and smoothness. The plan should be very clear what tests will be conducted, how many tests will be conducted per lot, and what will be done if test results do not meet the specification requirements.

The QC plan should be submitted to the government's representative, and no work should begin until the QC plan is approved. The plan should include a discussion of the laboratory, equipment, and the qualifications of the QC staff. It should be very clear who will be the point of contact for the QC laboratory if there is any questions or if there are problems that need to be corrected. Generally, those technicians involved with testing should, as a minimum, be certified within the local state DOT certification program or certified through some nationally recognized certifying group.

QC/QA program is very important to ensure a quality product. There are a number of things that should be considered in a QC/QA program:

- Experience and certification of QC/QA staff
- Condition of lab facilities and equipment
- Approved QC plan
- Timeliness, completeness, and accuracy of QC reports
- Good oversight of overall program

18.3 Preconstruction Checklist

Details of the preconstruction meeting have already been discussed. However, it is important to mention here that all aspects of QC should be very clear to the QC group and to the government's representative at the conclusion of the preconstruction meeting. If there are any questions that need to be answered, then it is important that these questions be addressed during the meeting or an answer provided a short time later if an answer cannot be provided right away.

18.4 Timely Reporting of Data

On many projects, the QC/QA data is not reported on a timely schedule. This is not acceptable. There is no reason that QC and QA data cannot be obtained and reported within 24 to 48 hours after the work has been completed. As a general rule, the testing for all mix placed one day should be completed and summarized by the end of the following day. Thus, there is no reason that all people needing this data cannot have it within the 24 to 48 hours. This timely submission of data will allow decisions about any problem to be made early and can give the contractor information that may be necessary to make mixture changes as early as possible. This timely submission of information will be advantageous to the contractor and to the government. It will help to prevent controversial issues that may otherwise develop.

18.5 Evaluation of Overall Mixture Quality

Some airfield agencies, such as FAA, use the percent within limits to analyze conformance to the specification requirements; other airfield agencies may use the mean absolute deviation; and others may use a go-no-go specification where the material represented by each test result is considered to be within or outside the specifications, based on the individual test results. Even though there are considerable differences in these specifications, they all encourage the contractor to produce mixture near the job mix formula or near the middle of the specifications, with the variability of the mixture being as low as practical.

Regardless of which type of specification is used, a quality product can be obtained. It is necessary that the contractor and QC personnel understand the specifications and be able to analyze the test results to determine the overall acceptability of the work being performed.

18.6 Mix Design

The mix design must be conducted by the contractor and submitted to the government's representative for approval. Most airfield asphalt mixtures are presently designed using the Marshall method, but more and more airfield mixes are beginning to be designed using the Superpave method. Be sure to note which method is specified prior to conducting the design. It is also important to determine from the specifications the amount of compactive effort. Is it 50 or 75 blows with the Marshall hammer, or is it some number of gyrations with the gyratory compactor?

When the Marshall mix design method is specified, be sure that a manual hammer is used, or make sure that the mechanical hammer is calibrated to the manual hammer. This is required in the ASTM method but is not always followed during actual mix design practice. It is also important that this same method be used to evaluate the mixture during production of HMA.

The approach to calibrate a mechanical hammer is relatively simply. However, this needs to be done for each mixture that is used. For simplicity, it is recommended that the mix design be performed with a manual hammer. There is not too much effort involved with compaction during mix design, so this should not be a problem. Once production begins, calibrate the mechanical hammer, if used, to a manual hammer. The process involves compacting three to five samples of plant-produced material with the appropriate number of blows (50 or 75, typically 75) with the manual hammer. Also, compact three to five samples at each of three different blows with the mechanical hammer, say 70, 85, and 100. Plot the results and determine the number of blows with the mechanical hammer needed to produce a mixture having the same density as that in the manual hammer for the specified number of gyrations. This calibration should be submitted, along with QC reports, to the design engineer.

After the job mix formula is produced, the contractor has to have some ability to adjust the mixture as plant production begins and continues throughout the project. Without this ability to adjust the mixture, the volumetrics and other properties of the mixture cannot be controlled within the specification limits. Adjustments are almost always necessary to provide best results, but these adjustments should only be made with knowledge of the government's representative.

18.7 Control of Materials and Mixture during Production

18.7.1 Introduction

The properties that must be controlled during mixture production include mixture volumetrics, gradation, asphalt content, in-place density, grade, and smoothness. Each of these items will be discussed in the following sections.

18.7.2 Construction of Test Sections

On airfield projects, a test section should be constructed at the beginning of the project for projects that are 3,000 to 5,000 tons or larger. There are two primary purposes of the test section. One is to verify that the mixture design is satisfactory. In most cases, some adjustment of the mix design will be required based on the laboratory test results during construction of the mix design.

Another reason for constructing a test section is to verify that the mixture can be compacted with the rollers and roller pattern selected by the contractor. If there are compaction problems, the equipment and procedures can be modified as needed to correct the problem. If necessary, corrections can be made to the process and another test section constructed or, in some cases, the contractor can move on to full-scale production based on results.

18.7.3 Mixture Volumetrics

The mixture volumetrics must be determined in the laboratory during mixture production. These properties are important to ensure good mixture durability and good resistance to rutting. The primary properties included in volumetrics are percentage of air voids, percentage of voids filled with asphalt, and percentage of voids in the mineral aggregate (VMA). The primary function of the VMA requirements is to ensure that there is sufficient space in the compacted aggregate for the minimum recommended amount of asphalt binder. If the VMA is too low, the amount of asphalt that can be added to the mixture (without closing up the air voids) is low, and this will result in loss in long-term durability. The VMA requirement is typically a function of the maximum or nominal maximum aggregate size. As the maximum aggregate size gets smaller, the VMA requirements are typically higher, resulting in a higher optimum asphalt content. The VMA requirements increase as the mix gets finer (smaller maximum aggregate size) since the mix will not close up as much and higher VMA can be obtained.

The primary purpose of the air voids and the voids filled with asphalt is to ensure that the mixture has good resistance to rutting and good durability. If the laboratory air voids are lower than about 3%, rutting is likely to occur. If the in-place air voids are higher than about 8%, long-term durability is affected due to the higher permeability of the mixture and the more rapid oxidation of the asphalt binder. The effect of voids filled with asphalt on performance is the reverse of that for air voids. Generally, the amount of voids filled with asphalt should not be too large otherwise loss of stability may occur.

During plant production, these volumetric properties must be checked several times per day depending on the project specifications. These properties are very important to ensure good performance of the mixture. These are the most important tests during mix production for predicting the performance of the asphalt mixture.

Samples are generally taken from the back of a truck or from behind the paver. (Some agencies actually take samples out of the paver hopper, but this is not safe and it is difficult to get samples that are not segregated, so this method is not recommended). There are advantages and disadvantages for taking samples out of the truck or from behind the paver. Generally, many engineers believe that taking the sample from behind the paver will give the results of the end product and, thus, be a better predictor of the actual mixture properties. These individuals believe that this is the best location to sample the asphalt mixture. There are problems with taking the mixture from behind the paver, including:

- Area has to be patched, sometimes leaving a bump.
- Mixture has to be taken back to the lab, which is typically at the plant, for testing.
- Mixture has to be reheated, which is not recommended.
- Plant adjustments cannot be made based on results since some segregation of the mixture may have occurred after production.
- Mixture may get contaminated with some of the tack coat.

Many engineers believe that the best place to take the sample is out of the truck bed before it leaves the asphalt plant. Disadvantages include the following:

- It is difficult to obtain a representative sample from the truck.
- It is not the end product on the airfield.

However, representative samples can be taken with care by shoveling some of the top surface off near the middle of the truck and taking samples at two to three locations near the middle of the truckload. This will result in a representative sample as long as care is taken not to take the sample close to the side of the truck. This sample taken from the side will typically be segregated, having a coarse gradation and a low asphalt content. Advantages of taking samples out of the back of the truck are:

- Samples can be taken at the plant, and samples do not have to be reheated.
- The plant can be adjusted based on test results from this location.

Based on the two choices, it is recommended that the best practice for taking HMA samples is from the loaded truck before it leaves the plant area. This process has been used for years by many engineers with good success.

Three locations that have been recommended, by some, for taking samples of HMA mixture:

- Back of truck at asphalt plant (preferred)
- From paver hopper (not acceptable due to safety problem and difficulty in obtaining representative sample)
- From in-place uncompacted mix behind the paver (satisfactory but not advisable since mix will have to be reheated and there will likely be a bump where mix was removed)

The samples obtained are compacted in the laboratory, and the density measured from the samples is used along with other properties to determine the air voids, voids filled with asphalt, and VMA. One property that is measured during production that is also important is the theoretical maximum density. This is the density that would be obtained if the mixture could be compacted to a point where there were no air voids. The theoretical maximum density is needed to allow calculation of the air voids and voids filled with asphalt in the mixture. This is measured in accordance with ASTM D2041.

18.7.4 Aggregate Gradation

The aggregate gradation must be controlled otherwise the mixture volumetrics will vary excessively. As mentioned earlier, the mixture volumetrics are very important in ensuring good performance of the mixture.

The gradation can be measured at a number of locations including the stockpiles, trucks or railcars hauling in aggregates, cold feed bins, or extracted from asphalt mixture. All of these locations provide some important information. The bottom line is the gradation of the aggregate in the mixture must be consistent otherwise problems will occur. The other tests (stockpiles, trucks, railcars, or feeders) are good for controlling the gradation to help ensure that the gradation in the final mixture has low variability and is within the specification requirements. Hence, conducting gradations at various points is necessary to control the gradation of the aggregate going into the mixture, but the gradation of the aggregate in the mixture is important from an acceptance standpoint.

The gradation of the mixture should be conducted several times per day to ensure that proper control of gradation is occurring.

18.7.5 Asphalt Content

The asphalt content must be closely controlled to ensure good performance. The amount of asphalt in the mixture is the one item that closely controls the mixture volumetrics and, thus, performance. If the asphalt content is too low, the air voids in the mixture will be too high, resulting in potential for durability problems. On the other hand, if the asphalt content is too high, the air voids in the mixture will be too low, resulting in potential for rutting.

There are many ways to measure the asphalt content, but the two most common methods are the solvent extraction test and the ignition test. Due to the solvent issue, the extraction test is becoming less popular and the ignition test is becoming more popular. Careful sampling is needed to ensure that a representative sample is obtained and to ensure that an accurate measure of the asphalt content is obtained. If a segregated sample is obtained, the measured asphalt content will be too low because the gradation is too coarse. So, if the measured asphalt content is low, one thing to quickly look at is the gradation of the aggregate in the same sample. If the gradation of the aggregate is too coarse, this likely means that a segregated sample was obtained and tested, hence, the test results may not be representative of the mixture being produced.

The asphalt content should be measured several times per day to ensure that good control of the actual asphalt content going into the mixture is obtained. During testing, it is important to know if an HMA mix has moisture in it. If this moisture is not removed before testing, there can be an error in either of the two tests since the moisture will be counted as asphalt content. The moisture can either be removed by drying prior to test, or the moisture content can be measured with a split sample and the asphalt content test result corrected based on the measured moisture. The most common method is to store the mix in an oven for some amount of time to remove the moisture before the asphalt content test is conducted.

18.7.6 Moisture Sensitivity

The moisture sensitivity test is conducted during mix design but is not conducted often during production and placement of the mixture in the field. It is recommended that moisture sensitivity tests be conducted on a regular basis during field production, especially for larger projects. The best time to measure the moisture susceptibility is during mix design. If the results are marginal, then conduct during construction of the test section and weekly during production.

18.7.7 Compaction

Compaction is needed to reduce the in-place air voids in the mixture down below 7% to 8% and to provide sufficient strength in the mixture to support the traffic. There are a number of roller types that can be used to obtain compaction, as discussed earlier. The best way to measure in-place density is by obtaining a number of cores from randomly selected locations and determining the density of these cores. It is important that cores be taken at random otherwise the results will likely be biased. If there is a location where the density appears to be low, additional testing can be taken in these areas, but the results should be used to evaluate the local areas and should not be used to evaluate the lot of placed asphalt mixture.

There are other ways to measure density including nuclear gages and non-nuclear gages, but these have to be calibrated to that density measured with a core so, at best, they can only approach the accuracy of a core. It is recommended that cores be used for acceptance of density of the material and that other more rapid, nondestructive procedures be used for quality control. These nondestructive gages are typically very good for a quick check on the density, but they generally don't have the accuracy needed for acceptance.

Some airfield specifications set requirements for density in the longitudinal joints. This is very important for airfield pavements since there are typically a large number of joints, and this is often the first location that performance issues occur. When there is a joint specification, it is important that the cores for density be taken directly in the joint. There have been a number of cases where cores were taken 6 in. to 1 ft outside the joint. This is not acceptable for determination of joint density. Generally, the joint can be compacted to a density within about 2% of the density obtained in the mat, and the specifications set up to allow for this difference between mat and joint densities.

Cores for density measurement are typically taken the morning after construction. The weather is cooler in the morning, and this makes it easier to take samples without damaging them. The samples are allowed to dry, and then density measurements are made after drying. Using this process allows the density results to be available approximately by noon the day following construction.

There are several ways to measure density of HMA including measurement of cores taken from in-place mixture, nuclear gages, and non-nuclear gages. While each of these have some benefits, cores should be taken and measured for acceptance of density. The nuclear gages and non-nuclear gages are good for establishing rolling patterns and for getting a quick measure of density, but these should not be used for acceptance testing because they are less accurate than cores.

The number of cores to be cut for density will vary depending on the specification, but typically four to ten samples will be taken on each day's work.

18.7.8 Grade Control

It is important to check the grade as individual lots are constructed. This process will ensure that good grade is being produced during the construction process and will allow any corrections to be made as needed. If one waits until many lots have been constructed, any problem is magnified because of the amount of material that has been placed.

If there are small grade problems or small bumps, these can sometimes be solved by grinding the surface with equipment containing a gang of saw blades. When done properly, this process will do an excellent job of removing bumps. If a low spot is the problem, this process of grinding is generally not effective.

When grinding is done, the surface does not normally require sealing afterward. Some agencies seal these ground surfaces, but this is not necessary. The amount of grinding should be limited to localized areas. Grinding large areas throughout the paved surface will make the pavement look like it has a lot of patches, and this is generally not desirable.

Skin patching for grade control or to fill in low spots is not acceptable and should not be allowed. If there is a low spot that has to be repaired, about the only satisfactory way to do this is to remove the material to some acceptable depth (at least 1.5 in.), fill to the proper grade, and compact.

18.7.9 Control Charts

The use of control charts is essential for proper control of a product. Without control charts, it is difficult to understand how the process is working and to make adjustments in the process to improve mixture properties.

Typically, two types of plots are recommended. The first is a plot of individual values, and the second is a plot showing the running average. The individual plot is important since it shows all of the individual data, but it is difficult to see trends due to the large amount of scatter in the data. The running average is important since it smoothes out the data and clearly shows trends.

The individual plots should include gradation, asphalt content, volumetrics, and in-place density. The more information that is plotted the more useful the charts can be. However, this should not become so cumbersome and take so much time that it does not get done. So, there is usually a limited amount of data that is plotted. As a minimum, there should be a plot of two to three gradation sizes typically including the No. 4 sieve and No. 200 sieve, asphalt content, air voids, VMA, theoretical maximum specific gravity, mat density, and joint density. Additional plots can be made, but the items listed are the most common properties that are typically plotted.

When the air voids change, this has to be a result of changes in lab density or theoretical maximum density since these are the only two factors in the calculation of air voids. Things that affect the lab density primarily include gradation and asphalt content. The thing that affects the theoretical maximum density is the asphalt content (unless the specific gravity of the aggregate changes). So, one can quickly determine the cause or causes for varying test results. Always keep in mind that one potential cause of problems is test error. Be aware that testing error does sometimes occur even with the best technicians and equipment.

Purposes of control charts:

- Provides good summary of data
- Shows trends in data, allowing adjustments to be made prior to significant problems
- Helps to assign cause and effect to problems

18.8 Troubleshooting Construction Problems

18.8.1 Introduction

During the construction process, there are a number of problems that can potentially occur. Some of these problems include aggregate issues such as gradation, asphalt binder issues such as asphalt content, mixture issues such as volumetrics, and construction issues such as density. It is important to identify these problems when they first occur and to be able to analyze the situation and determine the cause of the problem and provide corrective action.

The potential problems are identified below along with a discussion of methods used to identify the problem and steps needed to solve the deficiency. Identifying and correcting the problem quickly will save time and money and will be good for the owner and the contractor performing the work. No one benefits when problems exist for a long period of time before being identified and corrected.

18.8.2 Asphalt Binder Issues

18.8.2.1 Asphalt Binder Properties

Asphalt binder properties are important to ensure good performance of the hot mix asphalt. The binder must maintain sufficient flexibility to resist thermal cracking during cold weather and fatigue cracking under repeated loading. The binder must maintain sufficient stiffness during hot weather to prevent rutting. The binder must maintain its ability to bond to the aggregate particles.

The properties of the asphalt binder must be satisfactory to ensure good performance. The first step in obtaining acceptable asphalt binder properties is to select the proper grade of asphalt cement. Normally, the asphalt supplier will provide a certificate showing that the asphalt binder meets the specified properties. This certificate, which should be provided with all supplies of asphalt binder, should be reviewed to ensure that the results meet specification requirements.

The next step to ensure that satisfactory asphalt properties are obtained is to confirm that the mixture is not overheated. Overheating the asphalt binder will damage the asphalt, resulting in a stiffer asphalt binder. In general, the asphalt binder should never be heated to over 350 °F. This is a maximum recommended temperature, and damage can begin to result even at lower temperatures. In practice, the binder temperature is normally significantly less than the 350 °F. The temperature is normally higher for modified asphalt binders than for binders that do not contain a modifier.

Another issue that can result in undesirable asphalt properties is contamination of the asphalt. This contamination can occur when the proper grade of asphalt binder is contaminated with another grade of asphalt binder or some other type of petroleum product such as oil, diesel, etc. This contamination can result in a significant change in asphalt binder properties and result in significant loss in performance. The most likely source of contamination is when a binder brought by a supplier is added to a tank that already contains some amount of the contaminating material. The tanker transporting the asphalt binder could also have had some amount of contaminating material in it when loaded at the refinery.

It is not always easy to identify when binder problems exist, but there are some signs that may indicate when problems are occurring. One approach is to look at how the hot mix asphalt appears in the truck. The natural slope of the material added into the truck should be reasonably constant. If the slope of the stacked material suddenly changes significantly, this can possibly be caused by a binder problem. Of course, there are many other causes of a change in the slope, such as temperature, moisture in the mix, etc., so one should be careful concluding, just from this change, that there is a binder problem.

If the mixture being produced suddenly appears to have a dull brown color, this is an indication that the asphalt may have been overheated. Continually checking the mixture temperature will help to identify when this is a problem. Also, smoke coming from the mixture would be an indication that the mixture is being overheated.

Another indication of a potential asphalt binder problem is the way the material acts under rollers. When the asphalt binder has some problems, it often shows up when rolling the mixture. If there have been no problems with compaction and suddenly there are problems, this may indicate a change in the properties of the asphalt binder. This can happen as a result of contamination, overheating, or a change in the properties of the binder delivered to the asphalt plant.

<table>
<tr><th>Potential Binder Problems</th><th>Effect on Performance</th><th>How to Identify Problem</th></tr>
<tr><td>Wrong grade of asphalt is used</td><td>Optimum asphalt grade is selected. A different grade will likely increase rutting or cracking potential.</td><td rowspan="3">Will likely stack differently in truck. Will have change in Marshall stability results. May smoke more if contamination with diesel or similar material. May become tender or unworkable.</td></tr>
<tr><td>Contamination</td><td>Will change grade of asphalt and, thus, adversely affect performance.</td></tr>
<tr><td>Overheat asphalt binder</td><td>Binder will become stiffer and likely less resistant to cracking, especially when overheated in thin films.</td></tr>
</table>

Another indication that the asphalt binder properties have changed can be determined by the Marshall stability and flow. If the binder has become softer due to contamination or some other problem, the stability will likely decrease and the flow may also change. If the binder has become stiffer due to contamination or overheating of the asphalt, the stability will likely increase and the flow may be lower. When the asphalt has been overheated, it will sometimes appear a dull brown color. This overheating will result in a stiffer binder and some loss in flexibility of the mixture (higher stability and lower flow).

When it is expected that there is an asphalt binder problem, there are several ways that this can be investigated to confirm the problem. The first step may be to look at the certificate of compliance for the binder. It is unlikely that this will indicate a problem, but it may indicate that an incorrect binder was delivered to the project. It may indicate that the binder properties changed, even though the binder still meets the requirements for the asphalt grade being used. Sometimes a slight change in binder properties can make a significant change in the mixture properties.

It is recommended that a sample of each load of binder shipped to the asphalt plant be taken and stored for future testing in case a problem occurs. This sample can be quickly tested to determine if the binder meets the requirements specified. A consistency test should be used to provide a quick evaluation of the consistency of the asphalt binder. A good consistency test is the dynamic shear rheometer, which is one of the asphalt binder tests. If there is a change in the asphalt binder, there will most likely be a change in the test results with the dynamic shear rheometer. If the binder being produced continues to meet the requirements for this test, then it will likely meet all of the other requirements specified.

It is possible for asphalt binder to be damaged after receiving it from the supplier. This damage is usually the result of overheating. It is possible that it was damaged prior to mixing with the aggregate, but most likely the damage takes place after mixing with the aggregate since the properties can be changed in thin films much easier than when in bulk. The only way to evaluate the properties of the binder after being mixed with aggregate is to extract the binder from the mixture, recover the binder from the solvent, and conduct the binder tests on the aggregate. The binder properties will be changed some after being heated and mixed with the aggregate in the asphalt plant, so judgment has to be used when evaluating these results.

18.8.2.2 Asphalt Binder Content

The quality of the mixture is directly controlled by the asphalt binder content. If the asphalt binder content is too high, the mixture will tend to rut and bleed; if the asphalt binder content is too low, the mixture will be difficult to compact and will tend to have durability problems. The proper asphalt binder content is typically selected to provide a laboratory air void content of 3.5% to 4% under standard compaction effort. So, if the laboratory air voids are high, the asphalt content is typically low, and if the laboratory air voids are low, the asphalt content is typically high.

There are tests used to directly measure the asphalt content. These test results would be the most likely indicators of a change in asphalt content. The two most common tests for asphalt content are the solvent extraction test and the ignition test. In the asphalt extraction test, a solvent is used to remove the asphalt from the sample, and the asphalt content is determined. In the ignition test, the asphalt binder is burned from the mixture, allowing the amount of asphalt to be determined. Both tests provide acceptable results if the test is conducted properly.

In a batch plant, the asphalt content is controlled by weighing in a given amount of aggregate and asphalt into each batch of asphalt mixture. The asphalt and aggregate scales should be certified on a regular basis, and if there is any doubt about the accuracy of the scales, weights should be available to check the accuracy. It is also important that there be no leaks of materials when weighing otherwise the weight shown on the scales will not provide an accurate measurement of how much material actually went into the mixture. Samples for testing should be taken in a way to ensure that representative samples of the material being evaluated are obtained and segregated samples are not obtained. If segregated samples are obtained for testing, the measured asphalt content will be in error.

In a drum mix plant, the aggregate belt scales and the asphalt meter must be properly calibrated to provide an accurate measure of the asphalt content. Also, the moisture content of the aggregate must be input into the plant so that the aggregate weight on the belt can be adjusted for the amount of moisture available in the aggregate. Otherwise, the amount of asphalt binder added to the mixture will be incorrect.

When there are binder content problems, it is possible that this can be seen by carefully inspecting the mixture. If the asphalt content is low, the aggregate may not be fully coated, and the mixture may set up higher in the truck. If the asphalt content is high, the mixture may appear wet and may slump more in the truck. Generally, the mixture with high asphalt content will be easier to compact in the field than the mixture with lower asphalt content.

Laboratory air voids are determined often during construction, and this is a good indicator of the asphalt content. If the asphalt content is high, the air voids will be low, and vice versa. Another property that can be used to check the asphalt content is the theoretical maximum density (TMD). The TMD has to be determined to measure the air voids. If the TMD is lower than normal, this is an indication that the asphalt content is higher, and vice versa.

When the asphalt content is high, the flow value will almost always be higher and the stability is likely reduced. These two properties are regularly conducted on the mixture and are good indications of changes in asphalt content.

When the asphalt binder content is incorrect, it is important to locate the problem and correct it since the quality of the mixture is greatly affected by the asphalt content. Once the reason for the asphalt content problem has been identified, it is normally fairly easy to solve the problem and begin producing a mixture with the desired asphalt content.

18.8.3 Aggregate Gradation

The aggregate makes up approximately 95% by weight of the hot mix asphalt. The gradation of this aggregate is important to ensure that the mixture has adequate strength. The variability of the aggregate gradation must be minimized so that the volumetric properties of the mixture can be closely controlled. Generally, a dense-graded mixture is preferred for airfield pavements since this type of grading will typically provide the most mixture strength.

There are several steps in the process that are necessary to control the gradation of the aggregate in the asphalt mixture. The first step is to ensure that the gradation of the aggregate received from the aggregate supplier is satisfactory. This can be done by taking samples and testing the material as it is delivered to the asphalt plant site and to ensure that the material is stockpiled in a way that does not create segregation.

The material must then be consistently fed into the hopper that feeds the aggregate into the mixture. The hopper feeder must be calibrated to provide the proper proportion of each aggregate being added into the plant. The feed rate for each aggregate can get out of calibration when equipment or personnel working around the hopper accidentally bump the controls or when the materials change in moisture resulting in a change in the rate of feed.

The aggregate will break down some when the material flows through the plant. This breakdown is typically greater when the Los Angeles Abrasion is higher. This excessive breakdown can result in a significant change in the aggregate gradation and the volumetric properties of the mixture. This breakdown will almost always result in higher dust content, thus reducing the amount of air voids in the mixture.

It is sometimes difficult to accurately sample the aggregate when in stockpiles, hot bins, or in the mixture. Sampling problems can often result in errors in measuring the gradation of the aggregate.

Gradation problems can be identified at a number of locations in the production process. However, the location that is most important is in the produced mixture. There are two types of tests that are normally used to recover the aggregate from the asphalt mixture. One method is to recover the aggregate from the ignition test. A second method often used is to recover the aggregate from the mixture through the asphalt extraction test. When the aggregate gradation is determined to be unacceptable, then all points in the process (delivery from the quarry, stockpiles, loading into the cold feed hopper, and material flowing through the plant) should be checked to determine where the problem is occurring.

After the location of the gradation problem has been identified, steps can be taken to solve the problem.

18.8.4 Mixture Volumetrics

The volumetrics are important to ensure that the mixture will not rut and will have good durability. The primary component that controls volumetrics is the amount of asphalt binder in the mixture. The three properties included in volumetrics are air voids, voids in mineral aggregate (VMA), and voids filled with asphalt. The VMA is primarily controlled by the aggregate gradation and is used to make sure that the amount of asphalt in the mixture exceeds some minimum desired amount. The air voids and voids filled with asphalt properties are primarily controlled by the amount of asphalt in the mixture. All of these properties help to ensure the proper amount of asphalt is added to the mixture to get optimum performance.

There are several items that will affect the volumetrics in a laboratory compacted mixture. One item is the compactive effort that is applied to the mixture. When the compactive effort is increased, and everything else is controlled, the air voids are reduced, and vice versa. There are some differences between compactors, molds, etc. So, there are some variations in volumetrics due to slight variations in the way tests are conducted in the laboratory.

Another item that affects the volumetrics of a mixture is the amount of asphalt in the mixture. If the asphalt content is increased, then the air voids are decreased and the voids filled with asphalt are increased, etc. So, the asphalt content is very important for controlling volumetrics.

The temperature of the samples compacted in the laboratory has to be controlled to provide adequate compaction. If the temperature is too hot there might be some minor affect on compaction, but if the temperature is too low there can be a significant effect on the compacted density.

The aggregate gradation affects the volumetrics. The volumetric properties of the mixture are most affected by the amount of filler in the aggregate. Typically, as the amount of filler is increased, the air voids and voids in the mineral aggregate are decreased, and voids filled with asphalt are increased.

The amount of asphalt and the gradation of the aggregate will affect the volumetrics. When the air voids are low, this is typically caused by one of two things: the asphalt content being too high or the VMA (which is controlled by the aggregate gradation) being too low. Typically, when this is an aggregate problem, it results from the mineral filler in the mixture being too high. Once the cause of the problem is identified, the problem can be corrected and the volumetrics brought back into the specification requirements.

18.8.5 Marshall Stability

The Marshall stability provides a measure of the strength of the HMA and its ability to resist rutting and movement under traffic. During the mix design, the mixture is proportioned to provide adequate Marshall stability. Good crushed aggregates, along with the proper grade of asphalt cement and optimum asphalt content, will typically ensure adequate stability. If there is a stability problem, it usually indicates that the quality of the aggregate is not adequate, the percentage of mineral filler is too low, the grade of the asphalt binder is not acceptable, or the amount of asphalt in the mixture is excessive. It is also possible that there is some error in the test if the stability drops suddenly. The most likely error in measuring the stability is in the temperature of the test. The temperature of the mixture significantly affects stability; if this is not controlled, there can be significant error in the measurement of stability.

It is important to control the stability of the mixture to ensure that good performance is obtained. One measure of mixture stability is the Marshall stability test. This test is conducted on samples throughout the day during mixture production, so a problem should be quickly identified. If there is a problem, check the aggregate, grade of asphalt cement, asphalt content, and test method to ensure that the test is conducted in accordance with the standard procedures. If necessary, adjust the mixture proportions or quality of materials to bring the stability back into the specifications.

Generally, samples of the asphalt mixture should not be reheated, compacted, and tested for stability. Reheated samples will almost always provide higher stability values due to the hardening effect on the binder. Any stability results should be obtained from tests of samples that were immediately compacted in the laboratory after sampling.

18.8.6 Aggregate Segregation

Segregation of aggregate is a problem that results when the coarse aggregate separates from the remainder of the mixture. This segregated material has lower asphalt content and an open surface texture that will likely lead to early raveling. The reason for the lower asphalt content in the segregated material is the lower surface area of the coarse aggregate.

When coarse aggregate and fine aggregate are mixed and then handled, the coarser aggregate will tend to segregate from the finer mixture. This is a problem that has been faced for years when working with aggregates. One way to minimize the amount of segregation is to keep the coarse and fine aggregate materials separate until they are mixed at the plant. However, at some point, the coarse aggregate and fine aggregate has to be mixed to produce the asphalt mixture. After the material is mixed in the plant, steps have to be taken to ensure that any segregation is minimized when moving the mixture from the mixer to storage silos to trucks and to the paver.

There are many ways to help minimize segregation, beginning with the delivery of the aggregate to the asphalt plant. The more an aggregate is handled, the more likely it is to become segregated. The stockpiles must first be formed correctly to minimize potential for segregation.

The asphalt mixture must be added to the silos in a way that does not encourage segregation. This requires that a hopper be utilized at the top of the silo to collect the mixture and drop it into the silo as a batch. The material must be fed from the silo into the truck in a way that minimizes segregation. The material should be dropped in batches, loading the front of the truck bed first, back of the truck bed second, and middle of the truck bed last. This approach has been shown as necessary to minimize segregation.

Using a material transfer vehicle at the laydown site has been shown to reduce segregation due to its remixing of the hot mix asphalt. Care must be taken when loading the paver and placing the material otherwise segregation will occur.

Segregation typically occurs in a few localized areas, and testing of random samples is not likely to identify the segregation problem. Segregation is almost always first identified by visual observations. Segregation typically occurs in isolated areas such as at the end of truck loads, and this can best be detected visually. The appearance will be coarse textured and open. Segregated spots can best be identified shortly after a rainfall when the mixture is beginning to dry. Areas that absorb more water will be the slowest to dry. Segregated areas will absorb more water due to the higher voids and, therefore, will stay wet longer.

When segregation is observed, steps should be taken to evaluate items that affect segregation, as discussed above. Although some mixtures are more likely to segregate than other mixes, taking steps to evaluate and correct the segregation problem will typically result in a satisfactory mixture, even for mixtures that do tend to segregate.

Effect of change in mixture property on performance:

Mixture property	**Result in changes in mixture properties**
Air voids	Typical laboratory air voids are 3% to 4.5%. If the air voids are reduced, rutting and bleeding are likely to occur. If the air voids are increased, some loss in durability is expected.
Voids filled with asphalt	Typical voids filled with asphalt in the laboratory range from the low 70% to the low 80%. If the voids filled with asphalt are increased, rutting and bleeding are likely. If the voids filled with asphalt are decreased, then durability problems are expected.
Voids in mineral aggregate (VMA)	Typical VMA is 12% to 15%, depending on the nominal maximum aggregate size. If the VMA is increased during production, then the air voids are likely to be increased. This could result in mixture durability problems. If the VMA is decreased during production, then the voids are likely to be decreased. This could result in mixture stability problems.
Stability	Marshall stability is typically at least 2,200 pounds. If the stability increases during the production process, this is typically an indication that the mixture is getting stiffer. This improves stability of the mixture but if too high can lead to a brittle mixture.
Flow	Marshall flow typically is between 10 and 15. If the flow increases, it is an indication that the mixture stability is decreasing, and rutting can be a potential problem. If the flow decreases, it is an indication that the mixture stiffness is increasing, and this can result in a brittle mixture. Mixtures using modified asphalts will often have high flow numbers; this requirement is often waived for modified asphalts.

18.8.7 Compaction of Mixture

Good compaction is necessary to provide a watertight mixture that will provide sufficient strength to support the expected traffic. When mixtures are compacted to an acceptable density, they will provide good performance for an extended period of time. However, if density is inadequate, these mixtures will be permeable to air and water and they will begin to experience problems at an earlier date.

The first step in obtaining adequate compaction is to have a sufficient number of good rollers. Almost all projects will use at least one vibratory roller, one static steel-wheel roller, and many will use rubber-tire rollers. Static steel-wheel rollers are used as the finish roller on most HMA projects. The vibratory roller is very useful as a breakdown roller and is sometimes used as the finish roller if the vibrator is turned off. The one problem with a steel-wheel roller (static or vibratory) is that it tends to shove the asphalt mixture after some amount of rolling, resulting in check cracking. The rubber-tire rollers do not shove the asphalt mixture, but they do have a tendency to pick up the asphalt mixture as the mix sticks to the tires when rolling. Typically, the rubber-tire rollers ought to weigh at least 4,500 pounds per tire, with a tire pressure of approximately 90 psi. Significantly smaller rollers, or rollers with significantly lower tire pressures, will not be able to compact the mixture to the desired density.

Good density must be obtained in the mat as well as in the longitudinal joints. Having the ability to obtain good joint compaction takes experience. Reasonable compaction must first be obtained on the free edge of the HMA. When the free edge is compacted, there is some break over of material, resulting in loose material adjacent to the edge. The loose material needs to be cut back with a cutting wheel and disposed. The cold edge constructed previously must be overlapped with the paver when the adjacent hot lane is placed. The height of the screed at the joint must be sufficient to allow for adequate compaction when the hot mixture is rolled smooth with the adjacent cold material. Use of a rubber-tire roller at the joint has proven to be useful in obtaining adequate density.

Once adequate rollers have been obtained, experienced operators should be used and an acceptable rolling pattern must be set up. The rolling pattern should be set up based on experience from previous projects as well as experience from the project being constructed. The initial test section can be useful in verifying and modifying an existing roller pattern.

For quality control and setting up rolling patterns, the density can be measured with a nondestructive gage such as nuclear or non-nuclear gages. However, for acceptance, cores should be taken since this is the most accurate method of measuring density. Cores are taken at random so that the results are not biased in favor of the owner or with the contractor. Samples for mat and joint density must be taken and analyzed separately to ensure that good compaction is obtained in the joint and in the mat.

Compaction is difficult to obtain when layers of asphalt mixture are placed too thin. Generally, the thickness of a layer should be at least 3 times the maximum aggregate size of the aggregate. If the layer is too thin, it will cool quicker and there will be some bridging of the coarser aggregate making it difficult to compact. Many times, when a level course is placed or patching is done, the thickness is insufficient and adequate compaction is difficult to obtain. The mixture temperature must be adequate. If the temperature is too high, the mixture will tend to shove and move underneath the roller, resulting in check cracking and lack of adequate density. If the mixture is too cold, the mix will be stiff and not workable enough to compact.

When the asphalt content in the mixture is low, it will be difficult to obtain adequate density. Hence, sudden difficulty in compaction may indicate an asphalt content problem.

If it is determined that compaction is deficient, the critical items mentioned above should be evaluated to troubleshoot the problem and to determine recommended changes to the construction process. The necessary changes in the operation should be made, and improved compaction should be obtained.

18.8.8 Bond between Two Layers of HMA

Good bond between two HMA layers is essential for ensuring good performance of the HMA. If good bond is not obtained, one layer will slip past the underlying layer, resulting in slippage cracks and eventual disintegration of the pavement surface.

The steps for obtaining a good bond include selecting a suitable tack coat material, cleaning the underlying surface, applying the proper amount of tack coat, preventing contamination of the tacked surface, properly curing tack coat material prior to overlay, and rolling the overlay in a way that provides adequate density and does not result in lateral movement of the mixture under the rollers (the lateral movement will tend to weaken the bond).

There is not a bond test that is widely used, and in most cases, no bond test is conducted. However, it is simple to subjectively determine when a poor bond exists. When cores are taken for density measurements, the cores are typically drilled through the top layer and into or through the underlying layers. If the top layer separates from the underlying materials during the coring operation, or if the top layer can be easily removed from the underlying layers, the bond is poor and should be improved. When a good bond exists, the top layer will often have to be sawed from the remaining layers. This is indicative of a good bond between the top two layers.

If a bond problem exists, possible causes of the bond problem should be investigated and corrections to the process should be made.

18.8.9 Grade and Smoothness Issues

The grade has to be controlled to ensure that birdbaths don’t exist and to ensure that the HMA ties into adjacent structures. If the grade is not controlled, there may be birdbaths and the HMA may not tie into adjacent structures without a significant bump. Smoothness is important to minimize vibrations and potential damage to aircraft and to provide comfort to the passengers.

The grade is primarily controlled by one of three ways: stringlines, lasers, or GPS. Each of these methods must be set up correctly to provide the desired grade in the completed product. Smoothness is obtained by using good paving techniques and by using a moving straightedge. Good paving practices include keeping constant head of material in front of the paver, operating the paver at a constant speed, not stopping and starting, and using proper rolling techniques to prevent roller marks.

Grade is measured by determining the elevations at points along the pavement surface to ensure the elevation of the pavement surface complies with the specified elevations. If birdbaths occur during the paving process, this indicates that proper grade has not been obtained.

Smoothness can be measured with a straightedge or with a profilograph. Both of these forms of measuring smoothness are needed. The straightedge is primarily used to measure bumps in the pavement; the profilograph primarily measures the smoothness that the aircraft feels when traveling over the pavement.

If there is a grade or smoothness problem, look at the potential causes of the problems mentioned above to determine how to solve the problem.

19. RECYCLED HOT MIX ASPHALT

19.1 Introduction

The use of reclaimed asphalt pavement (RAP) became popular in the 1970s and has become even more popular since 2000 due to the high increases in cost of asphalt. Three motivating factors influenced the increased use of RAP in the 1970s. These include 1) the increased cost of virgin asphalt binder, 2) the development of the cold milling machine, and 3) the development of the drum mix plant, which made it easier to add RAP to a mixture and get a quality product. The specific benefits of using RAP include:

- Reduced costs of construction
- Conservation of aggregates and binders
- Reduced stockpiles of reclaimed asphalt

Typically, one of two methods is used to reclaim asphalt pavement. Many projects require the top few inches of HMA to be milled, and this material can be used for producing recycled HMA. The second method of asphalt reclamation is called "slab" or "chunk" removal. This process is not used very often but has been used with success when full-depth removal was required. When the chunks are removed, they must be crushed to provide smaller usable particles.

19.2 RAP Processing and/or Crushing

When creating stockpiles of RAP to be included in a hot recycled HMA mixture, large chunks of RAP must be broken up into smaller more manageable pieces. Usable RAP material should be 100% passing a 2-in. sieve. The goal of this processing is not to crush the aggregate in the RAP but to break up the conglomerations of the aggregate and binder into smaller pieces. It is preferred that the RAP be split into at least two stockpiles to make the product more uniform and easier to control. The split in sizes is usually somewhere between 1/2 in. and 1 in.

Generally, crushing of millings is minimal (only the oversized material needs to be crushed) as long as the teeth on the milling machine are kept in satisfactory condition. The milling machine typically breaks down the RAP into a size that is small enough to be fed right into the plant. Many times, the amount of RAP that can be used in a mixture is controlled by the amount of dust in the RAP. If the dust is too high, the combined recycled mixture will contain too much dust to meet the specifications; this will also likely reduce the VMA to an unacceptable level.

19.3 Handling and Stockpiling

It is preferable to keep RAP from different projects separate because this will yield the most consistent RAP product. In fact, it is desirable for airfields to use RAP collected from milling the surface of the pavement to be overlaid. This provides some assurance about the overall quality of the RAP being used. However, it is acceptable for RAP from different projects to be used, but one must be sure that the RAP is suitable. RAP can be blended from a number of sources to provide an acceptable material. Once the stockpile is characterized (binder content, aggregate gradation, and properties), it is generally preferred that no additional RAP be added to the stockpile. Millings and unprocessed chunk RAP should always be kept separate until the chunk RAP is processed.

RAP should be stockpiled in conical piles; otherwise, store the RAP so that it does not absorb excessive moisture. While conical piles should not be used with virgin aggregates for fear of segregation, RAP does not tend to segregate as easily due to the adherent nature of the material. RAP stockpiles tend to develop an 8- to 10-in. crust over the entirety of the pile. This crust, while easily broken through with a loader, is helpful in shedding water and may even assist in preventing reconsolidation of the RAP. Flat tops on RAP stockpiles will often have or develop depressions that will collect rainwater. This must be avoided.

RAP stockpiles should be placed on paved, sloped yards in order to facilitate drainage, when possible. Covering stockpiles may also be an option. The cost of the extra fuel to dry RAP, in many cases, will exceed the cost of the cover. Equipment should be kept from operating on the RAP stockpile if at all possible. Equipment activity on the RAP stockpile will consolidate the RAP and make it difficult to be used in a HMA without additional processing. This will also likely increase the amount of dust in the mix, which may already be an issue.

19.4 Characterizing RAP

Since RAP comes from an existing pavement, it is possible that the constituent aggregate will meet required specifications. However, it is certainly likely that the aggregate does not meet the specification requirements but will meet the specifications when blended with the virgin aggregate. When characterizing a RAP stockpile for use in a mix design, the stockpile must be thoroughly blended. Once the characterization begins, no more material should be added to the stockpile. When sampling the RAP, it is very important to obtain a representative sample. ASTM D 75 *Standard Practice for Sampling Aggregates* is a good reference for obtaining a representative sample. The RAP should be tested for binder content, gradation, and other specified aggregate tests such as bulk specific gravity, coarse and fine aggregate angularity, flat and elongated particles, and abrasion resistance. When determining the binder content, it must be ensured that the RAP is dry. This can be accomplished by drying the RAP in an oven. This drying will preclude the measurement of water as binder.

The following RAP stockpile characteristics may be determined:

- Moisture content
- Asphalt content
- Gradation
- Aggregate properties
    - Specific gravity

    - Angularity (fine aggregate angularity and fractured face count)
    - Flat and elongated count
    - LA Abrasion
- Binder properties
    - Viscosity
    - Penetration
    - Dynamic shear rheometer
    - Bending beam rheometer

If a paving project is going to be milled and the millings are to be used in the new HMA, it may be difficult to determine the critical properties of the RAP in a timely manner for the new HMA design. A potential solution would be to take 6-in. cores from the pavement and crush them using a laboratory crusher to perform the necessary testing on the RAP for inclusion in the new HMA.

19.5 Mix Design

The design of hot recycled RAP mixtures is very similar to the design of conventional HMA using only virgin materials. However, the RAP does require some special treatment during mix design. Further information on the mix design procedure can be found in chapter 7, "Hot Mix Asphalt Materials Selection and Mix Design."

19.6 Introducing RAP into the Plant

For both batch and drum plants, RAP begins in a separate surge bin than the virgin aggregate. In most cases, contractors now use two sizes of RAP, so two bins would be needed. Usually, these bins have steeper sides to allow the RAP to slide more easily. The bins may also be fitted with some sort of pneumatic jet vibration system to loosen the RAP because the RAP is more susceptible to bridging and clogging than virgin aggregate. As the RAP proceeds out of the surge bins, it may pass under a magnet to remove metal debris and over a scalping screen, sometimes called a grizzly, that removes large chunks of RAP that may have made it into the stockpile. These large chunks are discarded, stockpiled for future processing, or crushed in an in-line crusher on the plant, and run back through the grizzly (figure 19.1).

Figure 19.1 – In-line crusher and screening unit on RAP feed system

There are a few different methods for adding RAP to a drum mix plant, but perhaps the most common is to introduce the RAP about halfway down the drum from the flame at the RAP collar (figure 19.2). Introducing the RAP here avoids overheating the RAP and the production of excessively high levels of hydrocarbons. Another way that RAP is added to a drum mixer is in a double barrel drum (figure 19.3). In the double barrel, the aggregate is heated in the inner drum and then moved to the outer drum where the RAP and new asphalt binder is added. This gets the asphalt binder and RAP away from the flame and provides a good approach for mixing.

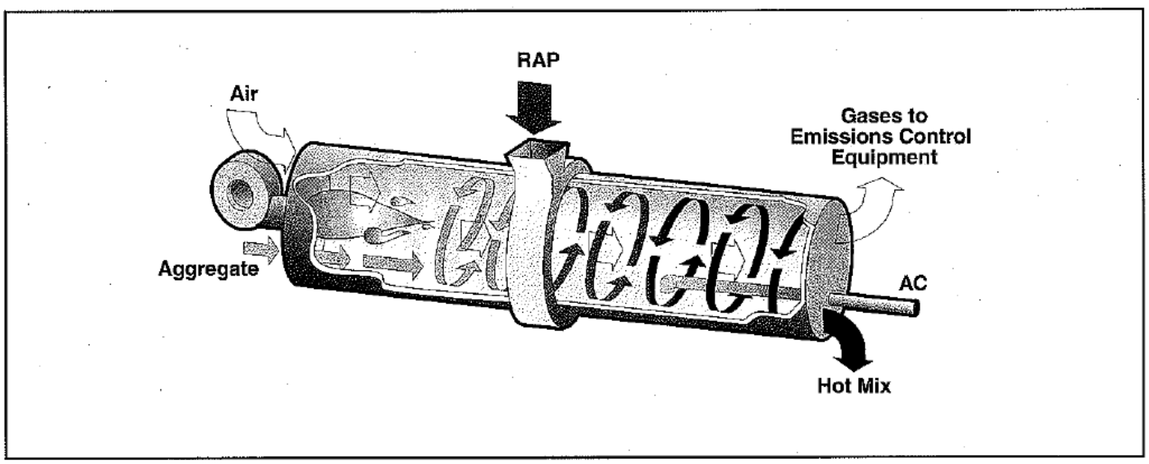

Figure 19.2 – RAP in parallel flow drum plant

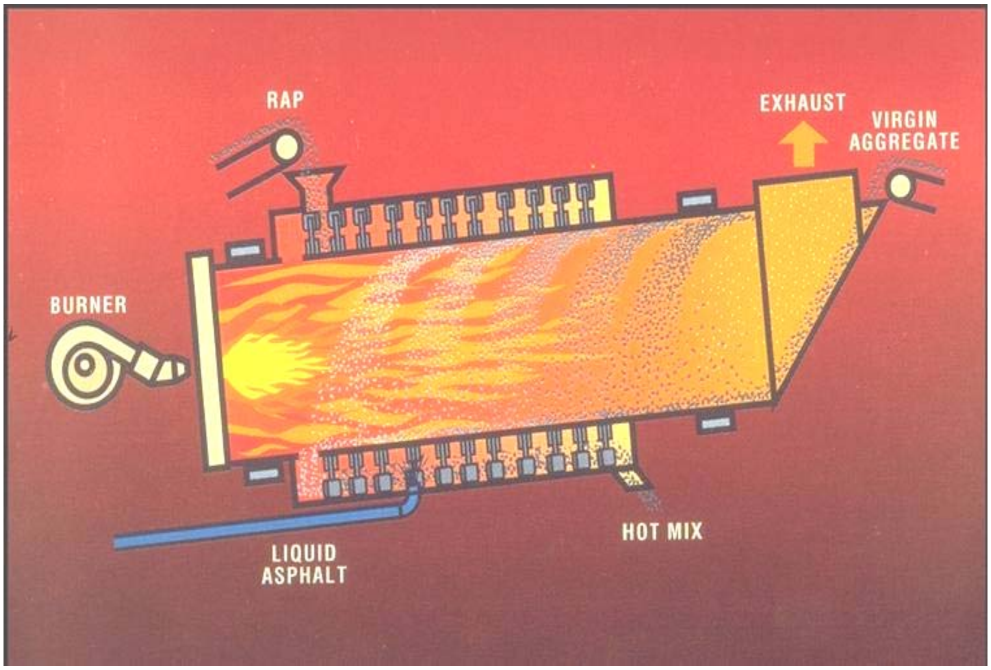

Figure 19.3 – Schematic of double barrel plant

There are also different methods for introducing RAP into batch plants. One method is to weigh the RAP directly into the weigh bucket at the batch plant (figure 19.4). If this method is used, the aggregate must be superheated to dry and heat the RAP by conduction. This is called the "Minnesota Method."

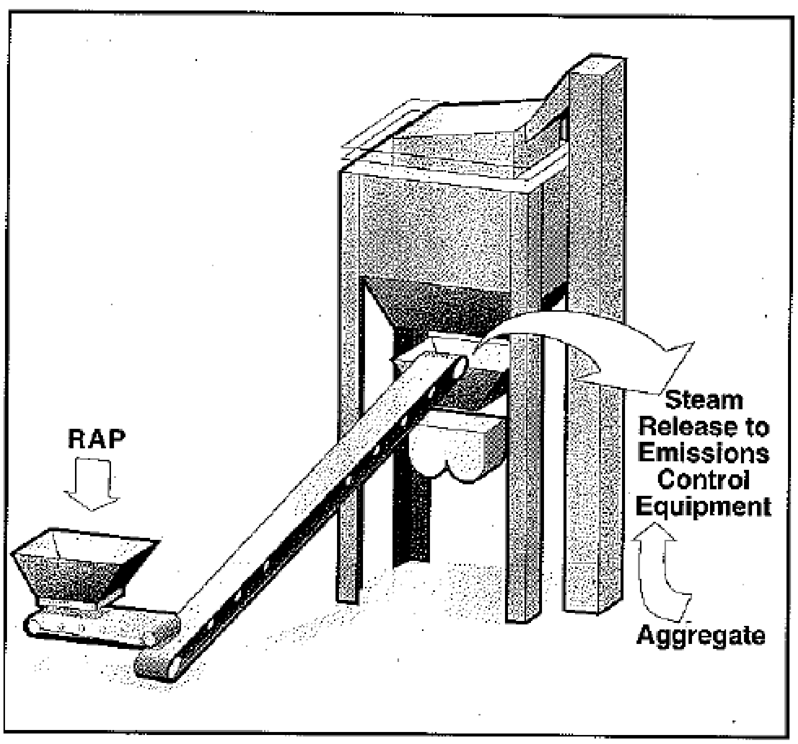

Figure 19.4 – Batch plant weigh bucket recycling technique (NAPA)

19.7 Laydown and Compaction

Laydown and compaction of HMA that contains RAP is the same as conventional HMA methods. All relevant density and smoothness criteria should be met. See chapter 13, "Laydown," and chapter 14, "Compaction," for specific guidance in these areas.

19.8 Fractionating RAP

Fractionating involves processing the RAP over a screen to create, as a minimum, one coarse RAP and one fine RAP stockpile. The breakpoint for fractionation is often the ¼-in. sieve. With a coarse and a fine stockpile, the RAP can be more precisely controlled as it is introduced into the plant. Theoretically, with this control, more RAP can be introduced into mixtures. However, many RAP operations do not currently use enough RAP to justify the expense of extra equipment and labor for fractionation.

19.9 RAP Critical Items

The following are general guidelines for the use of RAP in HMA:

- 100% of RAP chucks passing the 2-in. (50 mm) sieve.
- Maximum 2% deleterious materials.
- No aggregate particle in RAP mix should exceed maximum aggregate size at the time of discharge into transport vehicle. This is a potential problem with 3/8-in. (9.5 mm) NMAS mixes.
- Maximum RAP aggregate size should be less than half the layer thickness.
- Virgin materials and RAP blend should meet applicable aggregate properties, gradation, and volumetric properties.
- Use RAP gradation in calculation of mix gradation and fractured faces.
- Treat RAP as a stockpiled aggregate.
- Consider AC content of RAP when determining the trial asphalt content

The following critical items should be considered prior to utilizing recycled hot mix asphalt on airfield pavements:

- Maximum RAP chunk size should not exceed 2 in.
- RAP stockpile properties need to be consistent and similar.
- Blend of new virgin asphalt binder and RAP asphalt binder should meet specified asphalt binder requirements for project location.
- New virgin asphalt binder should not be more than one standard grade different than the specified asphalt grade for project location.

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20. PERFORMANCE PROBLEMS

20.1 Introduction

There are a number of performance problems that can occur in HMA mixtures. These problems will be discussed along with potential causes of problems and methods to prevent or solve the problems once they occur. Problems include rutting, bleeding, slippage, raveling, birdbaths, blisters, thermal cracking, popouts from contamination with organics or clayballs, and longitudinal joint issues.

20.2 Rutting

Rutting is a problem that sometimes occurs, especially when the pavement is subjected to traffic having high tire pressures and heavy loads. Rutting can be prevented, or minimized to the point of insignificance, in a properly designed mixture by using good-quality materials and good construction procedures. Rutting can occur in the asphalt mixture or in the underlying materials: base, subbase, and subgrade. While there are many things that can cause rutting, the emphasis here will be on rutting that occurs as a result of asphalt mixture problems. The location of the rutting can be determined by cutting a trench across the area that is rutted to show a good cross section of the pavement (figure 20.1). A stringline pulled tight across the trench can be helpful in identifying in which layers the rutting is taking place. Generally, if the rutting is taking place in the asphalt, the bottom of the asphalt mixture will be parallel to the stringline pulled across the trench.

Figure 20.1 – Trench cut across a rutted pavement

Narrow ruts generally indicate that the rutting is taking place in the asphalt mixture; wider ruts generally indicate that the rutting is taking place underneath the asphalt mixture.

The primary causes of rutting in the asphalt mixture include poor aggregate quality, incorrect grade of asphalt cement, and low air voids in laboratory compacted mixture. The two aggregate problems that are most likely to lead to rutting are excessive use of natural sand and improperly crushed gravel. The specifications place limits on the amount of natural sand and also on the fine aggregate angularity (FAA) test. If these specifications are followed, natural sand should not be an issue; however, in the past, there have been many instances where the contractor used too much natural sand (exceeded the specifications), and poor performance resulted. There is no test to accurately measure the amount of natural sand once the mix has been produced, so the amount of sand has to be measured during production at the asphalt plant to really know how much sand is in the mixture.

Using gravel that has not been properly crushed has also been a problem. The specifications provide limits on the amount of fractured faces, and these requirements need to be met to ensure that the mix will perform. When an aggregate fails to meet the specification requirements for fractured faces, the reason is typically simple. If a ¾-in., maximum-size aggregate is needed, fractured faces can be ensured if the material smaller than ¾ in. is removed first and then the larger size fractured. Aggregate producers tend to reject as little of the undersize as possible so that the specifications are just met. This optimizes the profit for the producer. There is nothing wrong with this; it is just good business. However, it does typically result in the aggregate barely meeting the specifications. The biggest problems occur in areas where the natural gravel is fairly small to begin with. In this case, a lot of material would have to be wasted to make a material meeting the aggregate specifications. So, again, the producer tries to waste as little of the product as possible and still meet the specification requirements. When crushed stone is used, the issue of fractured faces is no longer a problem.

Another significant cause of rutting in asphalt mixtures is the use of asphalt cement that is not stiff enough. The new PG grading system is much improved over previous methods, and if used effectively, should produce an asphalt mixture that does not rut due to insufficient stiffness in the asphalt binder. There is much discussion about the grade of asphalt that should be used on a project. The most common grade of asphalt cement used in the United States is PG 64-22. This is an unmodified grade and, in most of the climates in the United States, it should provide reasonably good performance. It is important that the grade of asphalt cement outlined in the specifications be used. But it is also important to know how the grade of asphalt affects the rutting performance. If the PG 64-22 grade is increased to 70-22 or 76-22, then the mixture should be even more resistant to rutting. However, when the grade is increased, the asphalt cement has to be modified, resulting in more cost for the material. So, if rutting is a problem in an area, one way the problem might be reduced is to increase the high-temperature grade of the asphalt cement.

Probably the most common cause of rutting is the amount of air voids in the mixture. If the asphalt content is high, or the dust content is high, the air voids will likely be low. Anytime the air voids are around 3% or lower, rutting is potentially a problem. So, it is important to continue to monitor the air voids during production. If the air voids are low, steps should be taken to modify the mixture so that the air voids are acceptable.

When rutting occurs, it often can lead to complete failure of the paved surface with time. The rutted areas will hold water in the surface, and this can be a problem for traffic especially if on the high-speed areas of the runway. When rutting occurs, the normal repair method is to remove the rutted areas, typically with a milling machine, and replace with a high-quality mixture. Prior to doing any repair, it is important to identify the cause and extent of the rutting. One must make sure that all unsatisfactory material is removed when the rutting is repaired. This may involve deep removal of the material in some cases, but in most cases, it only involves removing the top one or two layers of HMA.

20.3 Bleeding

Bleeding generally occurs when there is too much asphalt in the mixture and some of the excess asphalt "bleeds" to the surface (figure 20.2). Bleeding can also be caused by stripping of the asphalt from the aggregate or from contamination of the mixture with materials such as diesel. Bleeding can result in loss of friction of the pavement surface, and it may also be indicative of rutting potential.

Figure 20.2 – Bleeding in an asphalt mixture

When the mixture contains excessive asphalt binder, there may not be an early indication of the bleeding problem. However, as traffic is applied to the pavement, the mixture densifies eventually resulting in the voids in the mixture closing to an unacceptable level. For most dense-graded mixtures, this unacceptable level is less than about 3% to 4% air voids. When the voids level is this low, the asphalt binder is squeezed out of the mixture under additional traffic, resulting in bleeding. It is therefore very important to control the laboratory and field air voids during construction of the mixture. The amount of air voids measured in the laboratory is an indication of the amount of air voids that will be in the asphalt pavement after traffic. So, if the laboratory air voids are low during construction (say below 3%), they are likely to be low after some amount of traffic has been applied, even if the initial in-place air voids are significantly higher.

It is not uncommon to see small bleeding spots in the completed pavement surface after some amount of time due to contamination. Contamination that leads to bleeding is most often caused by diesel or some other type of solvent that may have been used during the paving project. There have been many instances when diesel was used to coat the truck bed prior to adding the HMA. This process has sometimes been used to prevent the HMA from sticking to the truck. This process is likely to result in some of the diesel accumulating in a portion of the HMA mixture in sufficient quantity to cause the asphalt cement to strip off of the aggregate. When this stripping occurs, the asphalt cement will migrate to the surface, resulting in a bleeding spot. This process will often lead to a pothole in HMA (figure 20.3).

Figure 20.3 – Bleeding due to diesel contamination

Sometimes, there is excessive prime coat or tack coat applied to the underlying layers. This excessive material may migrate up through the layer of HMA when sufficient traffic is applied, resulting in bleeding on the surface. So, prior to overlaying a surface, make sure that there is not too much tack or prime coat.

Stripping of the asphalt mixture can lead to the asphalt binder moving to the surface resulting in bleeding. Stripping of the asphalt binder from the aggregate due to action of water is something that does not happen too often on airfields but is a significant problem on highways. Water in the mixture under pressure from traffic can cause the asphalt to be stripped away from the aggregate. This usually happens over a significant period of time and results in the free asphalt migrating to the surface as traffic is applied. The moisture sensitivity of a mixture should be checked as part of the mixture design; when this is a potential problem, it should be corrected before mixture production occurs. This potential stripping problem can be solved by adding a small amount of lime to the mixture or by adding a liquid anti-stripping material to the asphalt binder. Both processes have been used with some success.

Bleeding of an asphalt pavement surface can be a difficult problem to solve. If the bleeding is localized, the localized area can be patched by removing the bad material and replacing it with acceptable material. However, if bleeding occurs over a large portion of a paving project, a decision has to be made about what to do. If rutting is not a problem and some loss of friction is not a problem, it may be possible to accept some bleeding. However, if loss of friction is important or if rutting is occurring, it is likely that the entire surface will have to be removed and replaced.

20.4 Slippage

Slippage of one layer along the underlying layer occurs when the bond between the two layers is insufficient to prevent movement between the layers. This is a fairly common problem. When slippage occurs, this may show up as U-shaped cracks in the pavement surface, or it may be obvious from noticing a painted strip that is no longer straight (figure 20.4).

Figure 20.4 – Slippage of HMA surface showing distorted line

One obvious sign of slippage potential is during the coring operation for density measurement. Typically, a core will be cut through the top newly constructed layer and into one of the underlying layers or all the way through the HMA layers. The top layer is then removed for density testing. If the top layer can be easily removed, this is an indication that the bond is not very good and slippage is likely to occur once subjected to traffic. Sometimes, the top layer will just pop off during coring; this is a definite sign of slippage potential. Sometimes, the top layer can be removed by hand or with a slight tap of a sharp object on the interlayer; this is also a sign of slippage potential. A good bond will require that the top layer be removed by sawing along the interface or by much effort with a sharp object and hammer.

Slippage may occur for a number of reasons. One reason is that the wrong type of tack coat is used. Generally, an asphalt emulsion or asphalt binder is used for tack coats. Another problem is the tack coat may be diluted with too much water. Generally, no additional water needs to be added to the emulsified tack coat. If water is added, it may dilute the emulsion so much that there is not enough binder remaining to provide a good bond.

Another cause is lack of calibration of the distributor, resulting in uneven or inaccurate application rates. Contractors seldom calibrate their distributor, often resulting in a very uneven spray with unknown quantities. One item that is often the cause of slipping is excessive rolling with a steel-wheel roller. Some rolling with a steel-wheel roller does not cause any problem, but at some point, the mix will typically begin to move laterally under the roller leading to loss of bond between the two layers, eventually resulting in slippage.

When slippage occurs, it is common practice to remove the area of slippage with a milling machine and to provide an overlay. It is important to remove material deep enough to go all the way through the slippage zone, otherwise the problem will not be solved. If the top layer slipped along the interface between the top two layers, for example, the top layer should be removed by milling, ensuring that the milling machine cuts all the way through the interface and into the underlying layer.

20.5 Raveling

Raveling generally occurs in areas that were not well constructed. This is typically in areas where handwork was done, where joints exist (figure 20.5), or where segregation of the mixture occurs. Generally, areas that tend to ravel are those with low density or low asphalt content, or those that are segregated. One of the biggest problems with raveling on an airfield is that it can lead to foreign object damage (FOD) issues.

Figure 20.5 – Raveling next to longitudinal joint

One issue that leads to raveling is segregation. When the mixture segregates, significant amounts of coarse aggregate accumulates in one area of the pavement. This section that is segregated will have higher coarse aggregate content, lower asphalt content, and a higher amount of air voids. This area will tend to lose some of the coarse aggregate with time, and in some cases, can ravel all the way through the surface layer. There are steps that can be taken during the construction process to minimize segregation, but best construction procedures are not always used.

Another location where raveling occurs is in longitudinal and transverse joints when the mixture is not properly compacted. It is difficult to obtain adequate density in these areas, and as a result, some raveling may occur. Proper compaction techniques have been discussed earlier and should be followed to ensure that good compaction is obtained.

If there are small localized areas of raveling, the best solution may be to remove and replace. However, if raveling is occurring throughout a pavement surface, the entire layer may have to be removed and replaced, or at least overlaid. In some cases, some type of seal coat may be applied to open surfaces prior to the start of raveling to help seal the surface and minimize any potential for raveling.

20.6 Birdbaths

Birdbaths are low spots that will hold water after some amount of rainfall (figure 20.6). These can cause water to accumulate and can cause problems, especially for runways where high speeds are needed. Birdbaths also tend to attract birds, which can be a hazard for aircraft that may ingest these birds during takeoff or landing causing potential engine damage.

Figure 20.6 – Birdbaths on airfield

Birdbaths are caused by a lack of adequate control on the pavement surface elevation. When pavements are built, or sometimes when overlays are constructed, there are elevations that the surface is required to meet. These elevations were developed to ensure good smoothness and proper drainage of surface water. If the pavement surface is not built to these design elevations, birdbaths are likely to occur. Methods to control grade were discussed earlier under chapter 13, "Laydown." If these recommended methods are followed, birdbaths are much less likely to be a problem.

When birdbaths are built into the pavement, it is difficult to correct these at a later date. For example, there might be a birdbath at a particular location, but simply removing the birdbath to some depth and replacing will probably not solve the problem. The water might be moved slightly to an adjacent location by removing and replacing, but the problem is typically not completely solved by simply cutting out a birdbath and patching. It requires that the surrounding area be surveyed and a new designed elevation be determined. The pavement should be removed and replaced in a way to meet this new designed elevation. The best solution is to ensure that good grade control is provided during the construction process.

20.7 Blisters

At times, when there is some amount of water in the underlying layer that is being overlaid, blisters may occur during the construction process and during hot weather after construction (figure 20.7). These blisters result from the top layer being pushed up by water vapor underneath. If the voids in the mixture are high, the water escapes through the surface; if they are low, the water vapor cannot escape, and this results in a blister. The hot asphalt heats the underlying water turning it to vapor, resulting in this upward pressure. When this occurs during construction, the blister should be punctured with a screwdriver or with a drill to allow the water vapor to escape. This will allow the overlay to lay back down and rebond, normally solving the problem.

Figure 20.7 – Blister in newly constructed pavement

Sometimes, blisters will result in a pavement after it has been down for some amount of time. This will normally result during hot weather when the underlying water vapor is at its highest. Puncturing these blisters at this time is also typically a good solution. Most of the time, after the pavement has aged some, this vapor will not cause blisters. It will simply cause the water to seep through the surface and show up as water on the surface (figure 20.8). In most cases, this water seeping through the surface is not a significant problem. However, if excessive, it can result in stripping the asphalt off the aggregate and potentially lead to raveling.

Figure 20.8 – Water from vapor pressure seeping up through pavement surface

The solution to this buildup of vapor pressure is to provide some type of positive subsurface drainage. When this is done, the water is taken away from the pavement and pressure does not build up. After the pavement is in place, there is no easy solution. Drilling blisters as they occur will often eventually solve the problem, but in some cases, the problem may continue for the life of the pavement.

20.8 Thermal Cracking

Thermal cracking occurs when the temperature of the asphalt mixture drops below the temperature needed to cause cracking. As the temperature drops, the HMA becomes stiffer and shrinkage begins to occur. At some point, the mixture can no longer resist the buildup of stresses and the HMA cracks. This type of cracking is called *thermal cracking*. Thermal cracking typically occurs in the transverse direction and is approximately evenly spaced in the longitudinal direction (figure 20.9). This is a problem that typically occurs in colder climates but can also be seen in warmer climates.

Figure 20.9 – Thermal cracking running in transverse direction

The best method to minimize thermal cracking is to select an asphalt binder that will resist cracking at lower temperatures. With PG-graded asphalts, this is done by selecting an acceptable low-temperature asphalt grade. For example, a PG 64-28 will be more resistant to thermal cracking than a PG 64-22, as discussed earlier. It is important to select the grade of asphalt that is recommended for the climate in which the HMA will be used.

One significant cause of thermal cracking is overheating of the HMA during production. The specification for the maximum temperature to which HMA should be heated during production usually states approximately 350 °F. This is the maximum temperature. Once this temperature is approached or exceeded, the amount of damage to the asphalt binder in the HMA begins to increase. After overheating and some oxidation once the mixture is in place, the mixture becomes brittle, eventually resulting in the development of thermal cracking.

Excessive use of -200 material (material passing the no. 200 sieve) may have the same effect as using a stiffer grade of asphalt binder. When more -200 material is used, this stiffens the asphalt binder and, thus, increases the possibility of thermal cracking.

Usually, thermal cracking is not something that will greatly affect the performance of the HMA. Locations that are cracked do create a bump. Also, moisture can get into the cracks if not properly sealed. For this reason, sealing of the cracks as they occur is recommended. Also, some raveling adjacent to the crack can occur; keeping the cracks sealed will help to minimize this potential for cracking.

20.9 Reflective Cracking

Reflective cracking is often a problem when overlaying old asphalt pavements or when overlaying concrete pavements (figure 20.10). The existing joints or cracks tend to reflect through the overlay. The time it takes for the crack to reflect through is a function of the overlay thickness, climate, and traffic, as well as many other factors. There are a number of things that have been done to minimize reflective cracking including using geotextiles, crack relief layers, etc. To date, there is no one process that has been universally accepted as a method to prevent reflective cracking. Of course, rubblization is one approach that has been used when overlaying concrete pavements, but no other method has been widely accepted.

Figure 20.10 – Reflective cracks in HMA over existing concrete pavement

The milling operation, which is often used prior to overlaying asphalt pavements, has been shown to reduce reflective cracks since the most severe portion of an existing crack, the surface, is removed during milling. While this process has been shown to be useful, it does not ensure that reflective cracks will be prevented. AAPTP project 05-04 has dealt with the reflective cracking issue.

20.10   Popouts from Contamination with Organics and Clayballs

The use of aggregates with clayballs or organics is not recommended for use during HMA production for airfield pavements. These organics or clayballs will tend to swell when coming in contact with moisture, resulting in popouts (figures 20.11 and 20.12). This is usually not a significant problem if not excessive, but it may pose some FOD issues.

Figure 20.11 – Potential popout due to clayball

Figure 20.12 – Clayball in core

Generally, these clayballs and organics will come from rock recovered from an old riverbed. This will potentially show up in gravels and natural sands that are typically recovered from these old riverbeds. It is important to inspect sources of material to be used in the HMA before production, and it is even more important that the gravels and natural sands be inspected because of this potential for clayballs and organic material. Crushed stone can have the same issues, but it is not as likely. If the sources of material have significant organics or clayballs, the material should not be allowed for use, and other sources should be obtained.

In most cases, a small amount of organics or clayballs will only be a nuisance and will not result in significant problems. About the only solution for this problem is to remove and replace, but unless there is a significant problem, the best answer is to leave it in place until time for another overlay. Any small localized problem areas can be easily corrected by cutting out the bad material and replacing it.

20.11 Longitudinal Joint Issues

Longitudinal joints pose one of the biggest potential problems for good performance of HMA pavements. Transverse joints create a problem, but typically there are not very many of them and, therefore, the overall problem is minimal. However, on airfields, the number and length of longitudinal joints is significant. The joint formed between each paving lane is a longitudinal joint. It is difficult to obtain satisfactory density in these joints, and that typically results in some loss in performance. Figure 20.13 shows poorly constructed longitudinal joints and some segregation adjacent to the joints due to improper luting of the overlap material at the joint.

Figure 20.13 – Poorly constructed longitudinal joints

There are primarily three reasons for obtaining low density at longitudinal joints. First, it is difficult to compact the free edge of HMA. This is the primary reason that a cutting wheel is used to remove the loose material at the edge of the pavement after being rolled. The second issue is that some contractors don't overlap the existing edge enough when the second lane is placed. Typically up to 2 in. is needed for the overlap. Any hot material that overlaps onto the cold surface should be pushed back with a lute to provide extra material in the joint. This will provide additional material in the joint during the rolling process that should help to increase density. The third issue is that there is a tendency not to raise the screed high enough adjacent to the joint when the second lane is paved. This results in insufficient material for compaction. Contractors are concerned that raising the screed too high will not allow the mixture to be rolled smooth and even with the adjacent material; therefore, there is a tendency to keep the screed too low.

If joint density is not satisfactory, there is not an easy solution for correcting the mixture already placed. The pay factors that are used in the specifications are there to encourage the contractor to obtain good density. When the constructed joint is so bad that it must be repaired, this must be done with care. If it is a short section, this can be cut out and patched. However, if this is a long section, about the only way to do this effectively is to remove sufficient material so that a paver can be used to replace the removed material.

Good QC/QA is essential for good performance. Mixture and construction deficiencies can result in a number of performance problems as listed below:

- Rutting
- Bleeding
- Slippage
- Raveling
- Birdbaths
- Blisters
- Thermal cracking
- Reflective cracking
- Popouts
- Poor longitudinal joints

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