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Evaluation of the Asphalt Mixture Design Framework for Airfield Pavements in Illinois

Prepared By
Abdulgafar O. Sulaiman
Uthman Mohamed Ali
Imad L. Al-Qadi
University of Illinois Urbana-Champaign

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7. Authors Abdulgafar Sulaiman, https://orcid.org/0000-0002-7455-5039, Uthman M. Ali, Imad L. Al-Qadi, https://orcid.org/0000-0002-5824-103X		**8. Performing Organization Report No.** ICT-23-011 UILU-2023-2011
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16. Abstract

The Federal Aviation Administration (FAA) advisory circular AC 150/5100-13 authorized the use of state highway material specifications at nonprimary public-use airports serving aircraft less than 60,000 pounds gross weight. This approval is based on the condition that the safety and life span of these airports will not be adversely affected. The use of highway materials provides the Illinois Department of Transportation (IDOT) with benefits, including cost, expertise, material availability, and sustainability. This study investigated the feasibility of using highway mixes for nonprimary airport pavements. Three classes of mixes were evaluated—namely, IDOT highway mixes, IDOT state airport mixes, and FAA airport mixes. The matrix consisted of 18 mixes: 15 surface and 3 binder mixes. The 15 surface mixes comprised seven laboratory-designed (five highway, two airport) and eight plant-produced mixes (four highway, four airport). All binder mixes were airport mixes and had the same composition and mix design parameters as highway mixes. Mixture performance was evaluated using the Hamburg wheel-tracking test (HWTT) to evaluate rut potential, the Illinois flexibility index test (I-FIT) to assess cracking potential, and the tensile strength ratio (TSR) test to evaluate moisture susceptibility. Performance testing was performed at both air voids of 4% and 7% corresponding to the initial in-place densities of nonprimary airports and highways, respectively. From the HWTT results, highway mixes had lower rut potential than airports mixes. With respect to the TSR test, airport mixes had lower tensile strengths than highway mixes. However, the TSR values (ratio of conditioned to unconditioned tensile strength) were similar for highway and airports mixes. The I-FIT results demonstrated comparable results between highway and airport mixes. Airport mixes had higher and comparable flexibility index for laboratory and plant mixes, respectively. By leveraging highway construction materials and methods, nonprimary airports could be constructed with greater expertise using more sustainable pavement materials that may yield reduced costs.

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Members of the Technical Review Panel (TRP) were the following:

- William Eves, TRP Chair, Illinois Department of Transportation
- Dennis Bachman, Federal Highway Administration
- Jeff Crislip, Federal Aviation Administration
- Navneet Garg, Federal Aviation Administration
- Brian Hill, Illinois Department of Transportation
- Neal Proctor, United Contractors Midwest
- LaDonna Rowden, Illinois Department of Transportation
- Phil Silas, Illinois Department of Transportation
- Megan Swanson, Illinois Department of Transportation
- James Trepanier, Illinois Department of Transportation
- Charles Wienrank, Illinois Department of Transportation

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EXECUTIVE SUMMARY

Nonprimary airports play a crucial role in regional connectivity and transportation networks. These airports have lower traffic volume and support lighter aircraft than primary airports. They help relieve congestion at primary airports and provide more general aviation access to local communities. For nonprimary airports, the Federal Aviation Administration (FAA) allows the use of state aviation standards (FAA, 2019). State highway material specifications may also be used if the nonprimary airport serves aircraft less than 27,216 kg (60,000 lb) gross weight.

There are major structural and functional differences between highway and airport flexible pavements. Differences include traffic volume, load type, tire pressure, predominant distresses, as well as hot-mix asphalt (HMA) considerations such as design air voids, aggregate gradation, and number of gyrations in a Superpave gyratory compactor. This project aimed to develop a framework that extends the use of existing Illinois Department of Transportation (IDOT) highway pavement surface and binder HMA to nonprimary airfield pavement applications. This application would provide contractors, agencies, and other stakeholders with several advantages in terms of cost, expertise, availability, and sustainability. Economic benefits would stem from the increased number of eligible contractors, which would encourage competition and drive down construction costs. The use of locally available and recycled materials would lead to environmental benefits. These advantages could be significant, considering that nonprimary airports are more abundant than primary airports.

Three classes of mixes—namely, IDOT highway mixes, IDOT state airport mixes, and FAA airport mixes were evaluated in this study. The matrix consisted of 18 mixes: 15 surface mixes and 3 binder mixes. Of the 15 surface mixes, seven were laboratory designed (five highway, two airport) and eight were plant produced (four highway, four airport). All binder mixes were airport mixes and constituted a laboratory-designed mix and two plant-produced mixes. No highway binder mixes were evaluated, because airport binder mixes have similar mix design parameters and composition as those of highway binder mixes.

Mixture performance was evaluated using the Hamburg wheel-tracking test (HWTT) to evaluate potential rutting, the Illinois flexibility index test (I-FIT) to assess cracking potential at intermediate temperatures, and the tensile strength ratio (TSR) test to evaluate moisture susceptibility. Performance tests were conducted at air void contents of 4% and 7% to represent initial in-place densities at nonprimary airports and highways, respectively.

The HWTT results showed that highway mixes had lower rutting potential than airport mixes. This could be attributed to several factors: airport mixes do not allow the use of recycled asphalt pavement (RAP), they are designed at a reduced number of gyrations, and their low air void content requirement encourages the replacement of manufactured sand with natural sand. Natural sand is known to perform poorly under the HWTT. With respect to the indirect tensile test, airport mixes had lower tensile strength than highway mixes. However, the TSR values (ratio of conditioned to unconditioned tensile strength) were similar for highway and airports mixes. The I-FIT results

demonstrated comparable results between the two mixes. The flexibility index (FI) of airport mixes was greater for laboratory mixes and comparable for plant mixes.

The study concluded that highway materials may be used in nonprimary public-use airports serving aircraft less than 60,000 lb. By leveraging highway construction materials and methods, nonprimary airports could be constructed with greater expertise and utilize more sustainable pavement materials at lower costs.

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CHAPTER 1: INTRODUCTION

Hot-mix asphalt (HMA) is a composite material used for the construction of highway and airport pavements. It is comprised of aggregates (e.g., crushed gravel, sand, and crushed stone) bound together with asphalt binder. In highways, HMA is often used as the top layer of the road, providing a smooth and durable driving surface. In airports, it is used in the construction of runways, taxiways, and aprons, where it provides a safe and stable surface for takeoffs and landings of heavy aircraft.

The Federal Aviation Administration (FAA) is the regulatory agency in the United States responsible for the safety and oversight of civil aviation, including airport pavement construction. FAA provides funding for airport pavement construction projects through its Airport Improvement Program (AIP), which helps airports improve safety, capacity, and efficiency. The agency also provides guidance and develops standards for the design, construction, and maintenance of airport pavements to ensure they meet safety requirements and are suitable for aircraft operations. With respect to the strength and durability of pavement materials as well as the design and layout of airport runways, taxiways, and aprons, the advisory circular 150/5370-10 provides specifications (FAA, 2018). The advisory circular includes specifications on general provisions, earthwork, flexible base courses, rigid base courses, flexible surface courses, rigid pavement, fencing, drainage, turf, and lighting installation.

Highway development, construction, and maintenance within a given state is regulated by the Federal Highway Administration (FHWA) and state department of transportation (DOT). DOTs typically work with contractors to ensure that highway pavements are built to meet safety requirements, provide a smooth driving surface, and support traffic and environmental loading. This involves specifying materials that are appropriate for local weather conditions and expected traffic volume. In addition, DOTs may implement a pavement preservation program to optimize the pavement life cycle cost.

Although the airport and highway HMA pavement construction process is similar, there are some key differences between the two. The design of each pavement is tailored to meet the unique demands of its specific application. Runways are designed and constructed to support the weight of large and heavy aircraft. Hence, airport pavements typically require greater structural capacity than highway pavements (e.g., thicker HMA layer and stabilized unbound layers). In contrast, highway pavements are designed to accommodate a range of vehicle types and weights.

In comparison to several highway specifications, FAA AC 150/5370-10 (2018) requires lower design air void content and number of gyrations, higher quality asphalt binders and aggregates, other directives such as the exclusion of recycled asphalt pavement (RAP) in surface courses, and the adoption of different performance testing. Generally, airport pavements are built to strict mixture design and pavement construction specifications that consider factors such as traffic volume, tire pressure, load distribution, predominant distresses, safety concerns, and prevailing weather conditions.

Nonprimary airports support light aircraft (comparable to freight truck weight on a highway), relieve congestion at primary airports, support regional economies by connecting communities to regional and national markets, and provide improved general aviation access to the overall community. Consequently, the number of nonprimary airports surpasses those of primary airports. The State of Illinois has 107 public and private airports, of which approximately 90% are nonprimary. FAA, through AC 150/5100-13 (2019), granted two major approvals to state DOTs, given that safety and life span of nonprimary airport pavements are intact. These approvals were for the development of state aviation standards for airport pavement construction at nonprimary public-use airports and the use of materials meeting state highway specifications for airport pavement construction at nonprimary public-use airports serving aircraft less than 60,000 pounds gross weight.

The current Illinois Department of Transportation (IDOT) airport flexible pavement construction specifications were developed in the 1980s, and later revised in 2012, 2020, and 2023. Based on the similarities between highways and nonprimary airports, research is needed to evaluate if IDOT highway HMA may be used for nonprimary airports in Illinois while maintaining FAA specifications.

RESEARCH OBJECTIVES AND SCOPE

The main objective of this study was to develop a framework that extends the use of existing IDOT highway pavement surface and binder HMA to nonprimary airfield pavement applications. The scope of the study is twofold:

- Evaluate existing IDOT-certified HMA for FAA volumetric and performance requirements. Volumetric requirements include aggregate gradation, voids in mineral aggregates (VMA), air void content, and asphalt binder content. Potential performance tests include meeting the Hamburg wheel-tracking test (HWTT), Illinois flexibility index test (I-FIT), and indirect tensile strength ratio (TSR) highway requirements.

- Investigate possible modifications of the HMA to meet FAA requirements, such as binder grade and asphalt binder content.

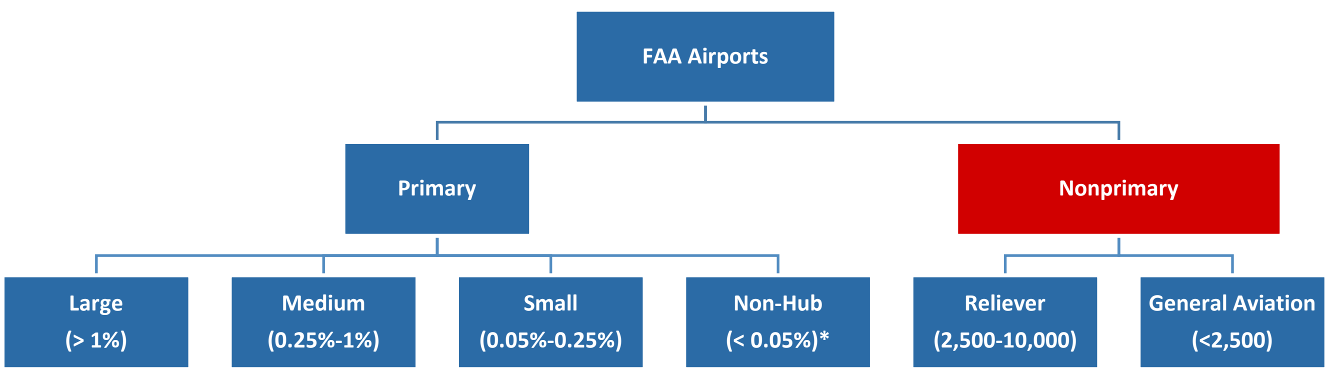
CHAPTER 2: CURRENT STATE OF KNOWLEDGE

AIRFIELDS AND ROADWAYS

Airports

According to the FAA (2016), an airport is any area of land or water used or intended for landing or takeoff of aircraft, including seaplane bases, heliports, and facilities to accommodate tilt-rotor aircraft. This also includes the area occupied by airport buildings and facilities and the rights-of-way associated with these buildings and facilities. The number of private-use (closed to the public) and public-use (open to the public) airports in the United States is about 14,400 and 5,000, respectively.

Passenger airports are classified into primary and nonprimary based on the number of passenger boardings each year. Primary airports have more than 10,000 passenger boardings each year, while nonprimary airports have fewer than 10,000 boardings each year (FAA, 2021a). The total number of boardings at passenger airports was 899,663,192 in 2018 (FAA, 2021b). Figure 1 presents a description of the various classes of airports based on the number of boardings as a percentage of the total boardings in a year for primary airports and annual number of boardings for non-primary airports.



*: above 10,000 boardings

Figure 1. Chart. Classes of airports.

Source: FAA (2022)

Nonprimary airports cut across several divisions. They could be commercial airports, which have between 2,500 and 10,000 passenger boardings, or reliever airports, which relieve congestion at a service airport. They are classified as general aviation airports when they have less than 2,500 passenger boardings per year (FAA, 2021a). This category was established for the distribution of nonprimary entitlements apportioned under the Airport Improvement Program (AIP). Generally, nonprimary airports have less than 10,000 passenger boardings, and there is a central theme of fewer passengers and service schedules that are either not well-structured or absent.

FAA oversees the planning, design and construction, runway safety, and environmental aspects of airports. FAA, through the AIP, offers grants to agencies, mostly public and sometimes private, for the planning and development of public-use airports included in the National Plan of Integrated Airport Systems (NPIAS). The NPIAS identifies nearly 3,310 existing and proposed airports that are included in the national airport system (FAA, 2021a).

Airports in Illinois

The State of Illinois is a hub for air travel. It is home to 107 public and private airports, of which approximately 90% are nonprimary. The earliest airport pavements constructed in Illinois were built with HMA surface treatments over a granular base. Portland cement concrete (PCC) pavements were later introduced in the 1940s during the Second World War.

In the 1970s, FAA issued advisory circulars (AC) AC 150/5370-10 and AC 150/5320-6C, which were promptly adopted by Illinois Department of Transportation—Division of Aeronautics (IDOT-DOA) and consultants. During that decade, airport designs moved toward thicker HMA layers and lime-treated subgrade. IDOT-DOA published its own construction specifications in 1985, the Standard Specifications for Construction of Airports, based on the FAA specifications, but was amended to suit the environmental and construction practices and materials specific to the state of Illinois, such as binder type and use of IDOT standardized aggregate gradations (Van Dam, 1995). Additional changes in pavement design included shorter PCC joint spacing and a minimum 75 mm (3 in) HMA layer thickness, among others.

In the early 1990s, there were major modifications to the FAA and IDOT-DOA material specifications, especially with respect to performance and quality assurance/quality control (QC/QA) elements of the specifications. Today, most Illinois airport runways, taxiways, and aprons are constructed of HMA. To ensure aircraft safe operation, airport pavements must be constructed and maintained without deviations or bumps. IDOT-DOA maintains general overall administrative responsibility for these airport pavements (Vavrik, 2001). Figure 2 presents the distribution of nonprimary airports servicing aircraft less than 27,216 kg (60,000 lb) gross weight in Illinois.

Figure 2. Map. Location of airports in Illinois servicing aircraft less than 27,216 kg (60,000 lb).

Source: ICT (2022)

Roads

Roads are transport corridors for the movement of goods and people. The U.S. boasts the largest road network system in the world with over 8.8 million lane-miles (Federal Highway Administration, 2022a). In the U.S., roads are grouped into three functional systems according to the type of service they provide: arterials, collectors, and local roads. Arterials consist of the interstate system as well as other freeways and important highways that supplement them. Collectors provide access within residential neighborhoods, commercial and industrial areas, and downtown city centers. They also connect arterials with local roads and streets. Local roads serve homes, businesses, farms, and small communities, providing a high level of access but limited mobility. Arterials (including the interstate system), collectors, and local roads account for about 11.1%, 20.1%, and 68.8% of the nation's total miles, respectively. The interstate system accounts for only 1.2% of the nation's road-miles but carries 24.1% of total travel, while local roads account for 68.8% of the road-miles but serve only 13.2% of total travel (Federal Highway Administration, 2022b).

Roads in Illinois

The road network in Illinois consists of segments of the interstate highway system and the U.S. numbered highway system as well as state routes and local roads. Illinois is at the heart of the country's road network, ranking as the state with the third-most interstate mileage at 3,516 km (2,185 mi), after Texas and California (Federal Highway Administration, 2022a). This interstate system is spread across 23 routes, including major corridors such as the coast-to-coast I-80 and I-90 along with I-70, which extends from the east coast to Utah. These major corridors are accompanied by several north–south corridors such as I-39, I-55, and I-57 and other east–west corridors such as I-24, I-64, and I-74. There are 25,700 km (15,969 mi) of state highways and 7,847 bridges providing accessibility to interstate routes all over the state (Batty, 2022). IDOT, founded in 1972 and headquartered in Springfield, oversees state-maintained public roadways. The Illinois Roadway Analysis Database System (IROADS), a graphical application, provides access to current and planned projects, roadway attributes, bridge inventory information, and pavement condition data and images. Figure 3 presents the road network with a focus on the major highways in Illinois.

COMPARISON OF AIRFIELDS AND HIGHWAYS

Pavements are designed and constructed to move traffic, usually via a vehicle, between two points in a safe and smooth manner. The definition of pavement encapsulates both highways and airfields, where in the former, vehicles include cars, trucks, etc. and in the latter, vehicles include airplanes, jets, etc. However, there are major differences between airfields and highways primarily due to the type of traffic using the pavements and their operational requirements (Miller et al., 2009). The next sections discuss these differences in detail.

Figure 3. Map. Major highways on the Illinois road network.

Source: IDOT (2022)

Design Parameters

Traffic Volume

Traffic volume is the number of vehicles passing a specified point on a road in a given unit of time and direction (Mughda, 2018). This is a key input in all transportation engineering projects, including signal timing, pavement design, transportation planning, congestion management, air pollution modeling, and emergency evacuation plans, among others (Castro-Neto et al., 2009). In highway design, traffic volume is commonly estimated in terms of parameters such as the annual average daily traffic (AADT). Roess et al. (2004) define AADT as the "average 24-hr volume at a given location over a full 365-year." According to FHWA (2019), the most traveled location by AADT in Illinois is I-90 in Chicago with an AADT of 321,700. This gives an average daily traffic (ADT) of over 100 million. Conversely, the total movement (a landing or takeoff of an aircraft) at the busiest airport in the U.S., Hartsfield–Jackson Atlanta International Airport, was 879,560 in 2017 (FAA, 2021b). For comparison, in terms of AADT, this is equal to an AADT of about 2,500. This factor of magnitude between traffic volume is one of the major points of difference between airfields and highways.

Tire Pressure

Rutting is generated mainly by densification and shear deformation in various stages of pavement life (Xu et al., 2008; Bonaquist & Mogawer, 1997). Load levels, temperature variations, and interface bonding are critical factors affecting HMA mechanical responses and rutting characteristics (Garg et al., 2018; Zou et al., 2017; Harvey & Popescu, 2000). Garg et al. (2018) reported that tire pressure of commercial aircraft significantly increases from about 1.2 MPa to 1.5 MPa (174 psi to 218 psi) compared to truck tire pressure, which range between 0.72 MPa to 0.76 MPa (105 and 110 psi) (Park, 2013), or much lower for passenger cars. This led Ling et al. (2020) to suggest that findings based on highway–vehicle tire pressure cannot meet the requirements of airfield pavements, considering that high temperature coupled with high tire pressure would further accelerate rutting accumulation.

Traffic Speed

The viscoelastic response of HMA pavements when loaded varies with both temperature and traffic speed (Al-Qadi et al., 2008; Katicha et al., 2008). An important measure of this response is the dynamic modulus of HMA, which is dependent on temperature and loading time (traffic speed). The operational ground speeds of aircraft are higher than vehicles. Aircraft operate at ground speeds up to 378 kmph (235 mph) (Wakefield & Dubuque, 2022), while the highest possible speed limits on any interstate is 137 kmph (85 mph), as allowable only on Texas State Highway 130 in San Antonio, Texas. This difference in operational speeds would affect pavement design, as there is an established trend of dynamic modulus increasing with increasing traffic speed and decreasing with increasing temperature, given that the pavement is smooth.

Bodin et al. (2016) evaluated the effect of traffic speed and temperature on HMA moduli for pavement structural design considerations. They presented a method to determine an equivalent HMA modulus via viscoelastic modelling, which represents the effect of temperature and loading speed on critical tensile strains. They considered two thick flexible pavement configurations representative of typical French pavement designs, and the results agreed with the usage of 10 Hz for 70 kmph (43.5 mph) at intermediate temperatures in France. In the code of the ALIZE airfield computer software, speeds of 100 kmph (62 mph), 30 kmph (19 mph), and 10 kmph (6 mph) were adopted for runways, taxiways, and aprons, respectively (Heymsfield & Tingle, 2019). In the numerical analysis by Hernandez and Al-Qadi (2015), to calculate the effect of wheel configuration on critical airfield pavement responses during takeoff, speeds of 240 kmph (149 mph) and 340 kmph (211 mph) were considered for the low and high boundary takeoff speeds, respectively.

Pavement Thickness

The previously mentioned parameters (tire pressure, traffic volume, and speed) lead to different pavement design requirements for both airfields and highways and, ultimately, different pavement thickness requirements. For highways, the minimum design thickness requirement for pavement layers is a function of design traffic, per the AASHTOWare design method. However, in airfield pavements and in accordance with AC 150/5320-6F, the minimum design thickness requirement is based on aircraft gross weight. While a 25.4 mm (1 in) HMA layer (or even surface treatments) are allowed for traffic less than 50,000 equivalent single axle loads (ESALs) on highways, the minimum HMA layer thickness for an airfield (less than 5,670 kg [12,500 lb]) gross weight is 76.2 mm (3 in) with additional crushed aggregate base and subbase requirements. Generally, typical airfield pavements are thicker than roadway pavements.

Design Method and Software

FAA Rigid and Flexible Iterative Elastic Layered Design (FAARFIELD)

FAARFIELD, developed in 2007, is the standard FAA design procedure for airport pavements. It was developed to replace the methods used at the time, which included the layered elastic design of the Federal Aviation Authority (LEDFAA 1.3), FAA design charts based on the California bearing ratio, and Westergaard methods (Kawa et al., 2002). FAARFIELD incorporates 3D finite-element computational models in the airport pavement design process. The software was initially designed for rigid pavements because of the flaws of the layered elastic design method in computing critical stresses in rigid pavements under complex gear loads. FAARFIELD presents advantages such as shorter computation time, improved pavement failure models, overlay design algorithm, user experience, and a revamped aircraft library (Kawa et al., 2002).

The introduction of newer aircraft models with more complex gear geometries after the Boeing 747 reduced the relevancy of the older design curves. Hence, Brill (2010) conducted a calibration study. Brill (2010) applied a calibration factor of 1.12 to FAARFIELD design stresses to ensure that FAARFIELD rigid pavement design thicknesses were compatible with the earlier procedure for aircraft traffic, including the Boeing 747. Over the years, FAARFIELD has seen a wide range of modifications, from technical aspects to user interface and user experience (Brill & Kawa, 2017; Tuleubekov, 2016). The current version is FAARFIELD 2.0, which was accompanied by AC 150/5320-6G: Airport Pavement Design and Evaluation. Pavement capacity is determined using the aircraft classification rating method. Yavari and Balali (2015) compared four runway pavement design software—namely, FAARFIELD, LEDFAA, TKUAPAV, and PCASE (Pavement-Transportation Computer Assisted Structural Engineering). Because its pavement analysis is based on the 3D finite-element method, the FAARFIELD prediction behavior of aircraft loading on pavement was the most realistic.

AASHTOWare

The American Association of State Highway and Transportation Officials (AASHTO) published the *Mechanistic-Empirical Pavement Design Guide* in 2008, releasing the first version of the complementary software program, currently named AASHTOWare Pavement ME Design in 2011, both of which were based on mechanistic-empirical (ME) principles (Pierce et al., 2014). These principles incorporate factors that directly relate to pavement performance, such as traffic, climate, material, and existing soil conditions—a significant change from previous empirical-based methods developed from the AASHO Road Test.

Three hierarchical levels of input are available in AASHTOWare, depending on the level of input accuracy required. Level 1 is the highest and most costly input parameter knowledge and is based on measured parameters and site-specific traffic information. Level 2 employs regional values that are calculated from other site-specific data or parameters using correlation or regression equations. Level 3 inputs are based on expert opinions and global or regional averages (Pierce et al., 2014).

Flexible Pavement Distresses

Predominant Distresses

Pavement distresses are caused by factors such as construction quality, subgrade soil characteristics, material characteristics, traffic loading, moisture, temperature, and the environment (Tamrakar, 2019). Distressed pavement, however, is often a result of a combination of factors rather than just one root cause. When the subgrade is of low quality, pavement flexes, easily causing severe distresses. Also, low-quality materials and poor construction affect durability—for instance, moisture entering the structure during upwelling of the groundwater table may deteriorate pavements with poorly constructed shoulder drainage or pavement layers, or both (Ragnoli et al., 2018; Adlinge & Gupta, 2005).

Some factors are more predominant than others on distresses, depending on the type of pavement and its location, especially when good construction materials and construction practices have been upheld. Based on this, distresses can either be load or environmental related. The multiple factors of magnitude by which highway traffic volume exceeds airfield traffic volume causes load-related distresses to be more pronounced in highways than airfields, especially nonprimary airports that experienced lighter aircraft and fewer movements.

Rutting on airfields is a major consideration in HMA airport pavement mix design, prompting FAA's strict material requirements aimed at ensuring high-quality HMA. There are numerous aggregate tests, such as aggregate angularity, soundness, durability, and shape. In addition to aggregate, asphalt binder significantly affects the rutting potential of HMA, as it influences aggregate particle mobility during traffic loading. For mixes, in addition to volumetrics, the asphalt pavement analyzer test has been used to evaluate rutting potential of airport mixes. White (2018) reported that distresses such as stripping, horizontal deformation, groove closure, and early aging urged airports and designers to adopt modified binders around 2000. Garg et al. (2018) reported that next-generation aircraft are expected to have higher wheel loads and tire pressures, which may increase rutting potential in HMA airfield pavements.

Using polymer-modified binders in HMA is one technique to reduce the effects of loading encountered with next-generation aircrafts (Rushing, 2018; Saqer et al., 2021; Wang et al., 2018). Based on tire pressure requirements and grade bumping specifications, in accordance with AC 150/5370-10H (FAA, 2018), polymer-modified binder mixtures are more commonly used in airfield pavements than in highways except for stone-matrix asphalt (SMA). Typically, when the asphalt binder performance grade (PG) spread between the high and low temperature is 92°C (198°F) or more, the asphalt binder has been modified (Federal Aviation Administration, 2018). Wang et al. (2018) presented a general performance of multiple-polymer modified HMA containing an anti-rutting agent, polyethylene, and styrene-butadiene-styrene (SBS). Multiple-polymer modification resulted in excellent high-temperature performance, leading to lower permanent deformation, moisture damage, and low-temperature cracking potential.

Foreign Object Debris

Foreign object debris (FOD) refers to any objects located on airfields (especially runways and taxiways) that can damage aircraft or injure air carrier personnel (Federal Aviation Administration, 2009). FOD may include twisted metal strips, components detached from aircraft or vehicles, concrete chunks from the runway, and plastic products (Xu et al., 2018). FOD poses a serious safety risk to an aircraft and a significant economic loss to airlines, and, as such, FAA has an advisory circular to this effect. The FAA AC 150/5210-24, airport foreign object debris management, presents several programs and necessary steps for airports, airlines, and the general aviation community to minimize FOD. The crash of Air France Flight 4590 that killed 113 personnel in 2000 was caused by a twisted metal strip, and similarly that of a Gates Learjet 36A at the Newport News/Williamsburg International Airport in Virginia in 2007 (National Transportation Safety Board, 2008). Hence, for airfield HMA design, runway debris as a type of FOD is of greatest concern. Aggregates chunks can fly off a runway and get stuck in the engine of an aircraft.

The FOD index is used to evaluate the potential for pavement-related FOD issues, as determined based on pavement distresses collected during the pavement condition index (PCI) survey in accordance with ASTM D-5340. This FOD index has been implemented in MicroPAVER, an airport pavement management system (Greene et al., 2004; Li et al., 2010). Li et al. (2010) investigated the ability of the current practice of PCI-based pavement maintenance plans to address the FOD maintenance requirements using 2.52 km^2 (0.98 mi^2) of airfield pavement. In general, PCI-based pavement maintenance plans can accommodate the maintenance requirements triggered by FOD-related distresses. However, some “acceptable” pavement sections based on PCI values may require maintenance based on FOD potential only. An innovative normalized PCI–FOD system was developed to identify efficiently sections that may be overlooked (Li et al., 2010).

Safety

Safety is a major component and determinant in the design, construction, and maintenance of pavements. It is affected by human behavior and several other variables such as weather, roadway geometry, visibility issues, FOD, as well as vehicle and pavement surface conditions. Of these variables, pavement engineers only have control over pavement surface conditions. Pavement surface condition is evaluated using surface characteristics such as friction, texture, smoothness, and tire–pavement noise. Friction and texture are of highest concern to pavement engineers (Merritt et al., 2015).

Friction is the force developed when a nonrotating tire slides along the pavement surface (Ong & Fwa, 2007). Pavement friction is a measure of the resistance to the relative motion between a vehicle tire and the pavement surface measured in terms of the nondimensional coefficient of friction. Pavement friction resists the relative movements between the vehicle tire and pavement surface. Hence, skid is generated due to the rolling or sliding of vehicle tires on the pavement surface (Hall et al., 2009; Wallman & Åström, 2001). In general, a low coefficient of friction means a higher probability of slippage in a fully locked braking condition. Two key mechanisms involved are adhesion and hysteresis. The former is the friction that results from the small-scale bonding/interlocking of the tire rubber and pavement surface, while the latter is the frictional force that results from energy loss during deformation (or enveloping around the pavement texture) as the tire moves across the surface (Hall et al., 2009). Pavement surface texture is related to safety as it affects the coefficient of friction and the ability of the pavement to drain water away from beneath the tire. Pavement texture is typically broken into four types—microtexture, macrotexture, megatexture, and roughness—with the first two of highest concern (Merritt et al., 2015).

Insufficient friction leads to skid-related accidents. Coarse aggregates are responsible for the majority of HMA pavement friction. The macrotexture is predominantly controlled by coarse aggregates, while the microtexture is a combined effect of coarse and fine aggregates. HMA design ensures friction via the use of wear-resistant coarse aggregates. Although aggregate selection is mostly based on the economy and availability, abrasion and polishing specifications are usually required. Silicious aggregates typically provide better abrasion resistance, while carbonaceous aggregates such as dolomites and limestones are more prone to wear and polishing. Typically, the Los Angeles (LA) abrasion specifications for airfields are harsher than those for highway pavements.

Sustainability

Sustainability is defined in terms of development by the World Commission on Environment and Development (1987) as a "development that meets the needs of the present without compromising the ability of future generations to meet their own needs." Harvey et al. (2015) recently described sustainability as being made of environmental, social, and economic needs, collectively referred to as the "triple-bottom line." As expected, the economic components have been the leading determinant to date. With increasing concerns about climate change, the environmental components have been gaining momentum. The social component, due to difficulty in measurement and evaluation, is the least developed, but is getting more attention recently. Among several tools used to measure sustainability, the four most relevant approaches are life-cycle assessment, life-cycle cost analysis, performance assessment, and sustainability rating systems (Harvey et al., 2015).

A sustainable pavement is one that meets basic human needs and effectively uses resources without significant damage to the surrounding ecosystems. A major component of incorporating sustainability in pavements is the usage of reclaimed asphalt pavement (RAP), warm-mix asphalt (WMA), and other sustainable additives. These technologies lead to less pollution, reduced extraction of nonrenewable resources, and less waste. RAP refers to removed or reprocessed pavement materials containing asphalt and aggregates that are generated when asphalt pavements are milled. WMA is a mix that is produced and placed at temperatures 20°C–40°C (36°F–72°F) lower than HMA (Vaitkus et al., 2009).

There are some differences or gaps between the incorporation of sustainability in highways and airfields. For example, while RAP can be used in highway surface courses from about 30% to 50%, the usage of RAP in airfields has been limited to intermediate courses with a ceiling of 30%. Years of research effort concentrated on highway pavements also led to a head start in technologies such as rubberized asphalt, WMA, and full-depth reclamation.

Performance Testing

Laboratory performance tests give an indication of HMA field performance. Several laboratory test methods have been and continue to be proposed to assess the pavement performance indicators in simple, cost-effective, and practical approaches. These tests range from older or empirical tests, as in the traditional Hveem and Marshall mix design tests, to newer performance tests such as the Illinois flexibility index test (I-FIT). Through the years, developed tests have been targeted at determining HMA performance characteristics and how these characteristics change throughout the life of a pavement. Performance criteria include cracking and rutting potential, ride quality, and surface friction, among others. Rutting at high service temperatures, cracking at intermediate service temperatures, and susceptibility to moisture are major performance characteristics often evaluated at the mix level. Low-temperature cracking is typically incorporated when selecting the asphalt binder for a given location.

Asphalt Pavement Analyzer

Rutting has been a major problem in flexible pavement for many years and has become of greater importance due to higher wheel loads and tire pressures, especially in warmer climates. It manifests as longitudinal depressions in the wheel path either via the repeated application of high stresses on the subgrade and/or the inadequate shear strength of the HMA. The thickness of pavement layers, traffic volume, and tire pressure are important factors that affect rutting. In general, rutting decreases the useful life of a pavement and creates a safety hazard (Skok et al., 2002).

In airfield pavements, the Asphalt Pavement Analyzer (APA) is currently used in mix design acceptance to identify HMA that may be prone to rutting. Rutting susceptibility is measured by cylindrical samples under repetitive wheel loads, usually 8,000 cycles, using a 45.3 kg (100 lb) load and a 0.69 MPa (100 psi) hose pressure and measuring the permanent deformation. Specimens are 150 mm (5.9 in) in diameter and 75 ± 2 mm (2.95 ± 0.08 in) tall at an air void content of 7 ± 0.5% tested at a high temperature of the standard SuperPave PG binder identified by the specifying agency for HMA. An automated data acquisition system obtains five rutting measurements per passing of the wheel. The APA was recommended as a rutting performance test for airport HMA design based on its ability to differentiate between mix performance measures and to identify significant improvement when polymer-modified binders are used in mix design. For example, Rushing (2018) evaluated acceptance criteria for the laboratory tests for HMA prepared using unmodified and polymer-modified binders and two different base PG binders.

Hamburg Wheel-Tracking Test

The Hamburg wheel-tracking test (HWTT) device was originally developed in Germany in the 1970s by Helmut-Wind Incorporated as a test device to measure the rutting potential and stripping susceptibility of HMA. Rutting and stripping are evaluated via a combination of steel wheel load of 71.7 kg (158 lb), generating an average contact stress of approximately 0.72 MPa (105 psi), and immersion in warm water of 50°C (122°F) (Cooley et al., 2000). The device tests two sets of cylindrical specimens simultaneously with two reciprocating solid steel wheels. Test specimens are typically compacted to an air void content of 7 ± 1%. The average speed of each wheel is 52 ± 2 pass/min, with wheels traveling about 320 mm (12.6 in) before reversing direction (Yildirim et al., 2007).

Aschenbrener (1995) evaluated factors that influence the results from the HWTT and found excellent correlation between laboratory results and highway pavements of known field performance. HWTT was sensitive to the aggregate quality, HMA stiffness, length of short-term aging, and compaction temperature. Izzo and Tahmoressi (1999) evaluated the laboratory repeatability of the test, testing configuration, test temperature, and capability to evaluate effects of antistripping additives. They found the test to be repeatable and consistent. Later, Lu and Harvey (2006) found that the test correlated poorly with performance and was strongly influenced by binder properties. Preconditioning specimens by vacuum to about 50% to 70% saturation, usage of various water temperatures for different binder grades based on environmental regions, and running HWTT under dry conditions were recommended steps for potential improvement. Also, Yin et al. (2014) introduced a novel method to analyze the HWTT, proposing three new parameters to measure moisture susceptibility and rutting potential based on the inflection point (where the curvature of the rut depth versus load cycle curve changes from negative to positive) and the viscoplastic strain increment.

Illinois Flexibility Index Test

Cracking is a major distress in HMA pavements often as a result of increased use of recycled materials such as RAP (Epps Martin et al., 2020; Willis et al., 2013). Developing and evaluating specifications for testing the performance of HMA with high amounts of recycled content led to the development of the Illinois flexibility index test (I-FIT) by Al-Qadi et al. (2015) using the semi-circular bending geometry. The measured parameter—flexibility index (FI), presented in Figure 4—indicates cracking potential and could distinguish between various HMA.

$$\mathrm{FI}=\frac{G_f * A}{|m|}$$

Figure 4. Equation. Flexibility index equation.

Source: Al-Qadi et al. (2015)

The FI is obtained from the load-displacement curve by dividing the fracture energy (G_f), which is the energy required to create a unit area of a crack, by the slope ($|m|$) at the post-peak inflection point, as presented in the equation in Figure 4. This is multiplied by the constant, *A*, a scaling factor equal to 0.01. The recommended testing temperature and loading rate are 25°C (77°F) and 50 mm/min (1.97 in/min), respectively. I-FIT is repeatable and consistent.

Tensile Strength Ratio

The presence of moisture in HMA causes loss of adhesion at the binder–aggregate interface leading to a distress commonly known as stripping. Water causes stripping in five mechanisms—namely, detachment, displacement, spontaneous emulsification, pore pressure, and hydraulic scour (Gorkem

& Sengoz, 2009). Anti-stripping additives are used to control stripping by increasing the physico-chemical bond between the binder and aggregate as well as moisture tolerance by lowering the surface tension of the binder. The most common types are hydrated lime and quicklime (Stuart, 1990). Liquid anti-strip is also commonly used.

In the laboratory, moisture susceptibility is measured using the tensile strength ratio (TSR) test. In the TSR test, according to AASHTO T 283 (2021), stripping is measured under laboratory-controlled water conditioning. Two sets of specimens are prepared with one set, referred to as the conditioned, undergoing partial vacuum saturation followed by freezing at −18 ± 3°C (-0.4 ± 5.4°F) for a minimum of 16 hours and then soaking in water for 24 hours. Both the conditioned and the control (dry) are subjected to the split tensile test. IDOT skips the freeze-thaw cycle for the conditioned set (IDOT, 2019). The TSR is the ratio of the average split tensile strength of conditioned to unconditioned samples, as presented in the equation in Figure 5. A minimum TSR of 0.70 to 0.80 is often used as a standard. The TSR is the most common method for evaluating moisture resistance. Do et al. (2019) evaluated the suitability of the TSR as a moisture susceptibility parameter by comparing it to the cohesion ratio, Marshall stability ratio, Marshall stability to flow ratio, and the dynamic immersion value. In addition to TSR, it was recommended to report the wet indirect tensile strength.

$$\mathrm{TSR} = \frac{tensile\ strength_{conditioned}}{tensile\ strength_{unconditioned}}$$

Figure 5. Equation. TSR equation.

Source: AASHTO T 283 (2021)

Dynamic Modulus

The dynamic modulus represents the stiffness of HMA when tested in a compressive, repeated load test (Bennert, 2009). It has been used to determine the structural response of HMA pavement layers (Garcia & Thompson, 2007). The dynamic modulus is greatly affected by the rate of loading, temperature, and aging. It is also one of the key pavement design parameters either when using the layered elastic design method, as in FAARFIELD, or the mechanistic-empirical design method, as in AASHTOWare. In the *Mechanistic–Empirical Pavement Design Guide*, dynamic modulus is used to simulate the time and temperature dependency of HMA (Al-Qadi et al., 2008). Gyratory compacted specimens, 150 mm (5.9 in) in diameter, are cored to diameters between 100 and 104 mm (3.9 and 4.1 in) and heights between 147.5 and 152.5 mm (5.8 and 6 in) in accordance with AASHTO T432. Load is applied at various frequencies (25 Hz, 10 Hz, 5 Hz, 1 Hz, 0.5 Hz, 1 Hz) from the highest to lowest frequency at five temperatures (−10°C [14°F], 4°C [39.2°F], 21°C [69.8°F], 37°C [98.6°F], 54°C [129.2°F]), starting from the coldest to the warmest.

Although traditionally conducted in axial compression, Kim et al. (2004) developed an analytical solution for the dynamic modulus of HMA tested in the indirect tension mode using the theory of linear viscoelasticity. The accuracy of this solution was successfully validated with the experimental data obtained from 12 commonly used North Carolina HMA mixtures tested via both axial compression and indirect tension testing. A fixed value of 1,379 MPa (200,000 psi) was adopted for P-401/P-403/P-601 HMA surface layers in the FAARFIELD software.

Binder Indicator of Low Temperature Cracking

Rutting and cracking result from traffic and environment loading. With reduced repeated loading, such as in nonprimary airports, the environment becomes a major agent. Titus-Glover et al. (2019) quantified the effects of pavement design and environmental factors on pavement performance in the absence of heavy loads. They found that the percentage of total damage related to environmental factors for flexible and rigid pavements that have been in service for 15 years with normal traffic loading was 36% and 24%, respectively. Hence, the effect of environment on pavements is not negligible. Environmental effects result in oxidation of asphalt binder, which leads to increased brittleness and aging and, ultimately, cracking (Herrington et al., 2005).

ΔT_c is an indicator of the effect of aging on asphalt binder rheology. It gives an insight into the relaxation properties of asphalt binder, which could contribute to non-load-related cracking or other age-related distresses in HMA pavement. ΔT_c is obtained from the difference between two critical low temperatures of the binder PG. It is calculated using values (creep stiffness and creep rate) from the bending beam rheometer (BBR) test, as presented in the equation in Figure 6, where $\Delta T_{c,s}$ and $\Delta T_{s.m}$ are the BBR critical temperature from stiffness and m-value, respectively. It is normally used on binders that have been long-term aged (rolling thin-film oven and pressure aging vessel [PAV]) (Baumgardner, 2021). The proposal to use 40-hour PAV conditioning and ΔT_c parameter to indicate cracking potential has been gaining attention in the literature. A ΔT_c value of less than −5°C (23°F) after 40-hour PAV conditioning suggests a high potential for cracking (Reinke et al., 2015).

$$\Delta T_c = \Delta T_{c,s} - \Delta T_{s.m}$$

Figure 6. Equation. Delta Tc.

Source: Christensen et al. (2019).

CURRENT USE AND SPECIFICATION FOR NONPRIMARY AIRPORTS

Nonprimary airports have less than 10,000 passenger boardings per year and fewer service schedules that are either not well-structured or absent. FAA allows state DOTs to develop respective state aviation standards for nonprimary airports with guidelines as provided in AC 150/5100-13 (FAA, 2019) in lieu of using AC 150/5370-10 (FAA, 2018). In AC 150/5100-13 (FAA, 2019), upon approval for airport pavement construction, states can either use their state aviation standards for general nonprimary airports or use their state highway material specifications for nonprimary airports serving aircraft less than 27,216 kg (60,000 lb) gross weight. Safety and service life of the airfield pavements shall not be compromised when constructing HMA using AC 150/5370-10 (FAA, 2018).

Currently, a few state DOTs have developed a general state aviation standard or modified highway material specification. Wisconsin and Alaska have state-developed standard specifications for airport construction. Florida also has a standard specification for construction of general aviation airports, while Missouri published a document adopting highway specifications for airfields. Generally, all state-produced specifications share major similarities with FAA's AC 150/5370-10 (2018). However, each state has implemented modifications based on local needs, climate, and materials. For example, the states mentioned specify the binder PG grade to be used for construction and typical air void ranges—unlike in AC 150/5370-10 (FAA, 2018), where an air void content of 3.5% is directly specified and there is no mention of a particular PG.

SPECIFICATION COMPARISON

Construction and materials specifications promote the production and placement of pavement materials that meet a minimum level of performance. They help communicate the required quality of the final laid product from agencies to contractors. Most construction specifications require the measurement of material characteristics that are believed to influence pavement performance (Miller et al., 2009). The following section provides a detailed background on the major types of transportation construction specifications, as defined by AASHTO (2003) and the Transportation Research Circular Number E-C037 (2002).

Types of Specifications

Construction specifications, as with several other aspects of transportation engineering such as material testing and mix design, have undergone and continue to undergo a wide scope of evolution. In the 1920s, agencies adopted method-type specifications, in which specific details such as each step in the construction process were prescribed. Liability and responsibility led to the development of quality control/quality assurance (QC/QA) specifications to allow the distribution of responsibilities between agencies and contractors. Construction and material variability cause challenges to QC/QA specifications. By the early 1960s, after the American Association of State Highway Officials (AASHO) Road Test in 1958, end-result specifications were introduced, allowing agencies to specify the final product. Then, quality assurance specifications were introduced, which require contractor quality control and agency acceptance activities throughout production and placement of pavement sections. From the late 1980s, this evolution has progressed into the development of performance-related specifications and, more recently, performance-based specifications (Gallivan, 2011). Gallivan (2011) visualized this evolution, as presented in Figure 7. The following section presents more details about these types of specifications.

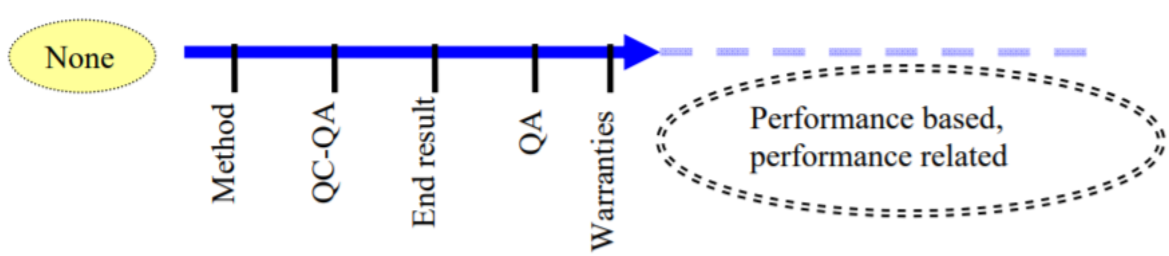


Figure 7. Chart. Specification development continuum.

Source: Gallivan (2011)

Method Specifications

Method specifications—also known as materials and method specifications, recipe specifications, or prescriptive specifications—require contractors to produce and place HMA using specified materials in agency-defined proportions using agency-defined equipment and methods to place the product in a prescribed method (AASHTO, 2003). In doing so, maximum control and responsibility are on the specifying agency. In other words, the agency hires the staff and equipment of the contractor. This type of specification may be viewed as one that prevents innovation on the part of the contractor. During the period when these types of specifications were popular, little or no testing was done, and acceptance was based on "reasonable or substantial compliance." Hence, there was no justifiable means to reject a product. As expected, these types of specifications are difficult to enforce, and financial payment is 100% across a range of quality.

End-Result Specifications

In contrast to method specifications, end-result specifications require the contractor to take entire liability and responsibility for the production and placement of HMA. Based on the degree of compliance with the specifications, the agency then either accepts or rejects the final product or applies a corresponding price adjustment.

Quality Assurance Specifications

Quality assurance specifications, previously called "statistically-based specifications," require contractor quality control and agency acceptance activities throughout production and placement of a product. Unlike the previous types of specifications, acceptance is usually based on a statistical sampling of the measured quality level of important quality characteristics. For highways or airfields, important measurable quality characteristics may include pavement density and smoothness. A "midpoint" between the method and end-result specifications, quality assurance specifications share responsibility and liability between the contractor and agency. The agency provides a practical means of achieving high-quality products, and contractors have an opportunity to try various processes and techniques. They are based on proven mathematical (statistical) principles for normal variability that provide a more realistic assessment of the degree of conformance to the specification criteria. Financially, price adjustments are related to quality level of the product, and there is room for increased payment (around 101%–105%) for superior quality work (Transportation Research Circular Number E-C037, 2002).

Performance-Related Specifications

Performance-related specifications (PRS) use quantified quality characteristics (such as asphalt content, air void content, and initial ride quality) and life-cycle cost relationships that are correlated to product performance to establish the desired acceptance levels. A distinct feature of PRS is that mathematical models are used to quantify the relationship between important quality characteristics and product performance. The models (performance-prediction models and maintenance-cost models) are based on collated empirical and mechanistic data and present a much clearer picture of what influences the performance of the constructed product than what could be visualized through engineering judgment and intuition alone. In pavement construction, apart from quality characteristics, inputs to these models would also include design variables such as traffic loading, climatic factors, drainage, and soil factors (Transportation Research Circular Number E-C037, 2002).

Performance-Based Specifications

Performance-based specifications (PBS) are "quality assurance specifications that describe the desired levels of fundamental engineering properties (e.g., resilient modulus, creep properties, and fatigue) that are predictors of performance and appear in primary prediction relationships (i.e., models that can be used to predict stress, distress, or performance from combinations of predictors that represent traffic, environment, supporting materials, and structural conditions)" (Transportation Research Circular Number E-C037, 2002, p. 9). Generally, compared to PRS, PBS use a more mechanistic and less empirical approach to define acceptance levels for fundamental engineering properties (Miller et al., 2009). As would be expected, with such a high level of complexity, there are no complete PBS. The SuperPave PG asphalt binder specifications, which were developed through the Strategic Highway Research Program, are examples of partial performance-based specifications.

Highway Specifications

Method type specifications, in which the contractor was directed by the agency to perform the work, were the first type of specifications adopted by agencies for highway pavement materials and construction (Chamberlin, 1995). The first standard test methods for HMA materials were published in 1911, while the first American Society for Testing and Materials (ASTM) specifications for materials were adopted in 1921. Specifications, sampling, testing, and construction methods in asphalt pavement construction have been covered in over 78 standards (Welborn, 1984).

A conference in 1923 resulted in the adoption of nine grades of asphalt by AASHO and ASTM in 1926 and 1947, respectively. Test methods included penetration, ductility, softening point, oven loss test, and flash point. In the 1930s, the Abson recovery test and thin-film oven test were developed, while later in the century viscosity tests were developed as a replacement for penetration in the 1960s (Welborn, 1984). The current binder specifications are the SuperPave PG tests supported by the Strategic Highway Research Program. These tests evaluate properties of binder, such as viscosity and modulus, and also create a relationship between these properties and binder-usage climatic conditions (Roberts et al., 1996).

A pivotal effort to standardize aggregate specifications was initiated in 1948 when a simplified practice recommendation, R 163-48, was approved and issued by the Bureau of Standards. This effort recommended standard sieves, aggregate sizes, and method of reporting (Welborn, 1984). Current specifications for aggregates (fine, coarse, and mineral filler) include tests methods such as unit weight, sieve analysis, specific gravity, absorption, soundness, abrasion, polishing resistance, liquid limit, plasticity index, and durability.

HMA designs and construction specifications have evolved from the Hveem mix design method to the Marshall mix design method and now the SuperPave mix design approach. The Hveem mix design method was developed by Francis Hveem in the late 1920s and selects asphalt content that yields the highest durability without dropping below a minimum allowable stability (Vallerga & Lovering, 1985). The Marshall mix design method developed by Bruce Marshall in the late 1930s seeks to select the asphalt binder content at a desired density that satisfies minimum stability and range of flow values (White, 1985). The SuperPave mix design developed during the Strategic Highway Research Program is the current mix design method. It has an asphalt binder specification based on its performance response to temperature and aging. It includes design parameters based on traffic loading and environmental conditions, selection criteria for aggregates, and a performance-based specification for mixes.

Airfield Specifications

From the 1950s, a prescriptive approach was generally adopted for the specification and design of airfields. This approach is usually based on the Marshall design method adapted for airport surfaces by the US Army Corps of Engineers during the 1940s and 1950s (White, 1985). Many airports and aviation authorities around the world retain the basis of the Marshall method in their current airfield specification (White, 2017). In this method, Marshall flow and stability as well as binder content, aggregate gradation, and field density are the primary design criteria and quality assurance parameters. In practice, these mixes generally have asphalt content between 5.4%–5.8% (by weight), 4%–6% of aggregate passing 75 mm (2.95 in) sieve (by volume), and approximately 14% (by volume) mastic (combination of binder and very fine aggregate) (White, 2018).

The performance tests, their limits, and criteria as defined in any pavement specification are controlled by the operational performance characteristics (OPC) of such pavement. These are the measures of pavement performance from the perspective of the user. Various OPCs have different levels of importance for airfields and highways, but all are largely categorized under safety, comfort, and appearance (Miller et al., 2009). Miller et al. (2009) conducted a series of interviews with airport operators, aircraft manufacturers, and experts to identify the important OPCs of interest on HMA airfield pavements. The two most important OPCs are braking and dynamic effects. The former is impacted by surface friction as well as the risk of hydroplaning while the later influences pilot control, passenger comfort, and can lead to aircraft damage.

Therefore, specifications define several acceptance criteria, as influenced by OPCs. These include density, aggregate gradation, asphalt content, air void, voids in mineral aggregates (VMA), roughness for a PRS, and fundamental engineering properties such as dynamic modulus and creep compliance for a PBS. The quality characteristics that most impact the structural performance and durability of a mix are asphalt content, air void content, and density (Miller et al., 2009). In FAA’s P-401 specification, acceptance criteria considered are air void content, mat and joint density, thickness of HMA layers, smoothness, and grade, as well as Marshall stability and flow (for the Marshall mix specification). Generally, highway and airfield specifications are developed by agencies such as FAA and state DOTs to provide thresholds and limits with respect to construction materials and methods that stakeholders such as contractors, consultants, and producers must meet. The next section compares three specifications pertaining to highways and airfields in the state of Illinois.

SPECIFICATION COMPARISON IN ILLINOIS

Compared Specifications

IDOT Standard Specifications for Road and Bridge Construction

This IDOT specification outlines "the general requirements and covenants applicable to all highway construction improvements as well as provisions relating to materials, equipment, and construction requirements for individual items of work on road and bridge construction projects awarded by the department" (Illinois Department of Transportation, 2021). Its requirements cover subjects such as earthwork, landscaping, subgrades, subbases, bases, surface courses, pavements, rehabilitation, shoulders, traffic control, pavement marking, and equipment, among others.

FAA Standard Specifications for Construction of Airports

This FAA specification is contained in AC 150/5370-10H (FAA, 2018) and relates to materials and methods used for construction on airports, including general provisions, earthwork, flexible base courses, rigid base courses, flexible surface courses, rigid pavement, fencing, drainage, turf, and lighting installation (FAA, 2021b). Per the development of the HMA design framework, item P-401 (HMA pavement) is of greatest importance. Item P-401 specifies comprehensive requirements for the design, production, and placement of HMA. These requirements begin with preliminary material acceptance criteria, mix composition, and laboratory design, as well as the type of construction methods, equipment, and quality control testing. A final mix and pavement acceptance criteria based on the Marshall stability and flow, air void, mat and joint density, thickness of HMA layers, smoothness, and grade are also stated (Miller et al., 2009).

IDOT Standard Specifications for Construction of Airports

This IDOT specification was developed for airfields in the state of Illinois per AC 150/5100-13C, which grants states the allowance to use state aviation standards for airport pavement construction at nonprimary public-use airports. This allowance is based on the condition that the safety and life span of the pavement will not be negatively affected. The format and content of the specification was drafted closely to the FAA *Standard Specifications for Construction of Airports*. The following subsections discuss the comparison between the standards.

Specification Criteria

Asphalt Binder

The following section discusses various parameters pertaining to asphalt binder properties.

Binder Grade

The asphalt binder grade-selection procedure specified by SuperPave assumes that the pavement is subjected to a given volume of fast-moving traffic for a given set of prevailing climatic conditions (Asphalt Institute, 1996). In addition, the specifications recommend adjusting the base high-temperature PG to account for the additional effect of heavy and slow or standing traffic. To accommodate for such cases where asphalt rutting is critical, SuperPave requires applying a one grade increase (bumping), equivalent to 6°C (42.8°F), to the base high-temperature PG grade in cases of slow traffic, and two grade increases, equivalent to 12°C (53.6°F), in cases of standing traffic (Asphalt Institute, 1996).

This recommendation was also put in place in the FAA AC 150/5370-10H specification (FAA, 2018). Different high temperature adjustments to asphalt binder grade were recommended based on the aircraft gross weight. The initial asphalt binder PG should be consistent with the recommendations of the applicable state DOT requirements for pavement environmental conditions. Chehab et al. (2019) investigated the high-temperature PG adjustment (bumping) recommendations contained in various specifications for airfield pavements. They recommended that PG bumping be applied beyond the surface layer in airfield pavements to the intermediate and base layers, as deeper layers are subjected to considerably higher compressive stresses, longer loading times, and consistently high temperatures. This recommendation was supported by two case studies (with and without PG bumping for the base layer) of airfield pavements in hot climates based on their performance.

Table 1 summarizes the PG selection and the minimum elastic recovery percentages for SBS and styrene-butadiene-rubber (SBR) modified binders required by IDOT.

Binder Low Temperature Criterion

The ΔT_c as a criterion was first proposed in a research project sponsored by the Airfield Asphalt Pavement Technology Program (AAPTP). AAPTP project 06-01 used the ductility loss of aged asphalt binder in investigating the relationships between binder properties and non-load-related cracking, with a particular focus on block cracking in airport pavements (Blankenship et al., 2010). Other full projects have been and continue to be carried out for better insight on the use of ΔT_c (Christensen et al., 2021; Reinke et al., 2015). Ten agencies had or soon will adopt ΔTc as a specification parameter, with the majority using the −5°C (23°F) criteria after 20-hour or 40-hour PAV aging (Buncher, 2019). Currently, no ΔT_c specifications have been adopted by FAA. IDOT requires ΔT_c using the −5°C (23°F) criteria after 40-hour PAV aging.

Aggregates

Aggregates constitute about 95% by weight of HMA. Hence, the properties of aggregates (coarse and fine) are pivotal to the performance of HMA when used in pavements. The Strategic Highway Research Program established four aggregate characteristics as critical in all cases for a well-performing HMA and called them “consensus properties.” These properties are coarse aggregate angularity, fine aggregate angularity, clay content, and flat and elongated particles. The Strategic Highway Research Program established three other aggregate properties as critical but source-specific and called them “source properties.” Specified values for these properties are established by specifying agencies. The properties are toughness, soundness, and deleterious materials (Jia et al., 2005). Tables 2–4 summarize the comparison of aggregate specifications. Control sieve serves as the cut off between the coarse and fine aggregate.

Table 1. Specification Comparison—Binder

<table>
<tr><th></th><th>FAA Primary</th><th>IDOT Highway</th><th>IDOT Airport (2012)</th><th>IDOT Airport (2020)</th></tr>
<tr><td>Elastic Recovery (%)</td><td>75</td><td>SBS modified binder<table><tr><td>64-28
70-22
70-28</td><td>60</td></tr><tr><td>76-22
76-28</td><td>70</td></tr></table>SBR modified binder<table><tr><td>64-28
70-22
70-28</td><td>40</td></tr><tr><td>76-22
76-28</td><td>50</td></tr></table></td><td>NS</td><td>NS</td></tr>
<tr><td>PG Grade Specificity</td><td>NS</td><td>NS[1]</td><td>64-22</td><td>IDOT Districts 1–6<table><tr><td>Surface and Top Binder</td><td>70-28</td><td>76-28</td></tr><tr><td>Lower Binder</td><td>64-22</td><td>64-22</td></tr></table>IDOT Districts 7–9<table><tr><td>Surface and Top Binder</td><td>70-22</td><td>76-22</td></tr><tr><td>Lower Binder</td><td>64-22</td><td>64-22</td></tr></table></td></tr>
</table>

NS: Not Specified

[1]: IDOT provides guidance for binder PG in its *Bureau of Design and Environment Manual*

Table 2. Specification Comparison—Aggregates

	FAA Primary	IDOT Highway	IDOT Airport (2012)	IDOT Airport (2020)
Lithology	crushed stone, crushed gravel, crushed slag, screenings, natural sand	gravel, chert gravel, crushed gravel, crushed stone, wet bottom boiler slag, crushed slag, crushed sandstone, crushed concrete, chats, crushed steel slag	crushed stone or crushed gravel, blended with crushed or natural sand(s)	crushed stone, crushed gravel, crushed slag, screenings, natural sand
Control Sieve	#4	#4	#8	#4
Anti-stripping Agent	NS	hydrated lime, slaked quicklime, or a liquid additive required if tensile strength and/or TSR criteria are not met	NS	hydrated lime, slaked quicklime, or a liquid additive

NS: Not Specified

Table 3. Specification Comparison—Coarse Aggregates

	FAA Primary	IDOT Highway	IDOT Airport (2012)	IDOT Airport (2020)
LA Abrasion (%)	40	40–45	40	40
5 Cycle Soundness (%)	12 (Na_2SO_4) 18 (Mg_2SO_4)	15-25 (Na_2SO_4)	15 (Na_2SO_4)	15 (Na_2SO_4)
Clay Lumps and Friable Particles (%)	1	0.25–0.5	0.5	0.25
Shale (%)	NS	1–4	2	1
Soft & Unsound Fragments (%)	NS	4–8	6	4
Coal & Lignite (%)	NS	0.25	NS	0.25
Other Deleterious (%)	NS	2–4	2	4
Total Deleterious (%)	NS	5–10	6	5
Equivalent IDOT Aggregate Quality	A/B	A/B/C/D	B	A

NS: Not Specified

Table 4. Specification Comparison—Fine Aggregates

	FAA Primary	IDOT Highway	IDOT Airport (2012)	IDOT Airport (2020)
5 Cycle Soundness (%)	10 (Na_2SO_4) 15 (Mg_2SO_4)	10–20 (Na_2SO_4)	15 (Na_2SO_4)	10 (Na_2SO_4)
Liquid Limit	25	NS	NS	NS
Plasticity Index	4	NS	NS	NS
Sand Equivalent	45	NS	NS	NS
Natural Sand (%)	0 to 15	NS	NS	NS
Minus No. 200 (75 µm) Sieve Material (%)	NS	3–10	6	NS
Clay Lumps and Friable Particles (%)	1	1–3	3	1
Shale (%)	NS	3	3	3
Coal, Lignite & Shells (%)	NS	1-3	3	1
Conglomerate (%)	NS	3	3	3
Other Deleterious (%)	NS	3	3	3
Total Deleterious (%)	NS	3–5	5	3

NS: Not Specified

Hot-Mix Asphalt

The following section discusses the various parameters pertaining to HMA volumetrics and performance properties and are summarized in Tables 5 and 6.

Air Voids

An air void is a pocket of air in compacted HMA that occurs between aggregate particles coated with asphalt binder. A certain air void percentage is needed in HMA to allow for additional pavement compaction during the early life of the pavement and to provide spaces into which small amounts of mastic can flow. Air void content has a significant effect on HMA durability and potential for different distresses, including rutting, stripping, fatigue cracking, and low temperature cracking (Monismith, 1992). Higher air void content leads to a more permeable mix through which air and water can be introduced, causing distresses such as stripping and raveling. Water weakens the adhesive bond between the aggregates and binders as well as the cohesive bond within the binder, leading to the disintegration of the asphalt mix and, ultimately, failure of the pavement (Kassem et al., 2011). Conversely, lower air void content leads to more stable and less permeable mixes. However, an excessively low air void content can lead to bleeding—a condition in which excess asphalt mastic squeezes out of the mix to the surface (Willoughby & Mahoney, 2007). It has been proven in the literature that air void reduction stiffens HMA material, leading to lesser rutting potential (Roy et al., 2013; Witzcak, 2002). IDOT and FAA have target air void contents of 4% and 3.5%, respectively, during mix design.

Rutting

Several tests have evaluated moisture susceptibility and rutting potential of HMA. Two major tests currently used are HWTT and APA. Generally, both tests apply repetitive loading on HMA specimens in the presence of water and measure the rut depth in the specimen with increasing load cycles. In addition to rutting potential, these tests give an indication of moisture susceptibility (Lu & Harvey, 2006; Yin et al., 2014). However, a major difference exists. In HWTT, the weight of the wheel is fixed at 72 kg (158.7 lb), which results in an average contact stress of about 0.69 MPa (100 psi) on top of the specimens. The HWTT is required in the IDOT standard specification for high ESAL mixtures. IDOT specifies a failure criterion of 12.5 mm (0.5 in) at several passes dependent on the contract plan mix binder PG, as listed in Table 6. In the AC 150/5370-10H (FAA, 2018), the APA test, using the AASHTO T340 test procedure at 1.72 MPa (250 psi) hose pressure, is recommended. A failure criterion of 10 mm (0.39 in) at 4,000 passes is specified. Alternatively, when APA is not available, HWTT can be used with a criterion of 10 mm (0.39 in) at 20,000 passes. This highlights one of the major differences between highways and airfields with respect to tire pressure, as previously mentioned in this report.

Cracking

Cracking distress is addressed via several direct and indirect ways in specifications, from procedures as simple as low and intermediate SuperPave PG criteria to HMA performance tests such as I-FIT. I-FIT has been adopted by IDOT, and the criteria are summarized in Table 6. There are no cracking tests specified by FAA.

FOD

FAA provides guidance for developing and managing an airport foreign object debris (FOD) program in AC 150/5210-24. However, this AC only provides guidance on the prevention, detection, removal, and evaluation of FOD as well as the specifications for the equipment used in FOD removal operations. Although some researchers have proposed the adoption of a FOD index (Greene et al., 2004; Xu et al., 2018), it has not been implemented by FAA. There are no FOD requirements in highway specifications.

Construction

In addition to specifications about material types and properties, agencies provide specifications about construction processes, especially as they affect quality assurance and quality control of payments. Of importance to mix design are density and control limits. Table 5 presents the suspension limits of AC 150/5370-10H (FAA, 2018) and the QC/QA QC control limits using the moving average of 4 in IDOT’s (2022) *Standard Specifications for Road and Bridge Construction*. Lastly, Table 6 presents a specification comparison between HMA for highways in Illinois and HMA for airfields in accordance with FAA.

Table 5. Compaction Density and Control Limits

	FAA Primary	IDOT Highway	IDOT Airport (2012)	IDOT Airport (2020)
Asphalt Content (%)	±0.70	± 0.2	± 0.45	± 0.45
Air Void (%)	± 0.5	± 1.0	NS	NS
Minimum VMA (%)	−1.0	−0.5	NS	NS
Mat Density (%)	94.5 (min)	92.0–97.4	94 (min)	94 (min)
Joint Density (%)	92.5 (min)	90.0–91.0 (min)	NS	NS

NS: Not Specified; min = minimum

Table 6. Specification Comparison—Hot-Mix Asphalt

	FAA Primary	IDOT Highway	IDOT Airport (2012)		IDOT Airport (2020)	
			≥ 60,000 lb	< 60,000 lb	≥ 60,000 lb	< 60,000 lb
No of Gyrations	75	30, 50, 70, 80, 90	40–50	30	50	30
Air Void (%)	3.5	4.0	2–4	2–4	2–4	2–4
VMA Min (%)	16	13.5-18.5 based on NMAS	8–16*	8–16*	8–16*	8–16*
VFA Min (%)	78*	NS	75–90	75–90	75–90	75–90
Asphalt Content	5.5–8.0 (stone) 7.0–10.5 (slag)	NS	5–7	5–7	5–7	5–7
Dust/AC Ratio (max)	0.55–1.1* (stone)	1.0	0.7–1.4*	0.7–1.4*	0.7–1.4*	0.7–1.4*
HWTT	10 mm @ 20,000 passes	12.5 mm @ PG 58-xx (or lower): 5,000 passes PG 64-xx: 7,500 passes PG 70-xx: 15,000 passes PG 76-xx (or higher): 20,000 passes	NS	NS	NS	NS
APA	10 mm @ 4,000 passes (250psi) 5 mm @ 8,000 passes (100psi)	NS	NS	NS	NS	NS
I-FIT	NS	HMA - 8.0 (STA); 5.0 (LTA Design); 4.0 (LTA Plant) SMA – 16.0 (STA); 10.0 (LTA) IL-4.75 – 12.0 (STA)	NS	NS	NS	NS
TSR (%)	85 (areas with aggregate that have a history of stripping)	85	NS	NS	NS	NS
RAP	Only for shoulders and intermediate courses (0–30%)	IDOT SSRBC Art. 1031.06 Dependent on N design, Mix Type, RAP/FRAP Type, and Asphalt Binder Grade	NS	NS	Only for base courses (0–30%)	Only for base courses (0–30%)

*: Values not explicitly stated but calculated from other stated values.

NS: Not Specified; SSRBC: Standard Specifications for Road and Bridge Construction

Field Performance Comparison

FAA recently completed a comparison of in-service performance of asphalt pavements in 40 nonprimary airports with aircraft less than 27,216 kg (60,000 lb) gross weight. The airports were in five states: Georgia, Illinois, Indiana, Michigan, and Wisconsin. FAA and state specifications were used in the construction of 21 and 19 primary and nonprimary airports, respectively (West et al., 2023). Of the 19 airports constructed using state specifications, 10 airports used state airport specifications (5 in Illinois and 5 in Michigan) and 9 airports used state highway specifications (3 in Georgia and 6 in Wisconsin). Table 7 presents the airport details based on the type of specification used. The five airports evaluated in Illinois were constructed between 2010 and 2016 using the 2012 Illinois *Standard Specifications for Construction of Airports*.

Table 7. Details of Airports Evaluated Based on the Type of Specification Used

Specification	FAA	State Airport	State Highway
Classification	General/reliever	General/reliever	General
Number of airports	21	10	9
Year paved	2007–2015	2006–2016	2003–2018
Total aircraft operations	2,546–63,200	6,854–133,110	5,600–61,000
Single-wheel load rating (1,000 lb)	10–75	12–72	12–40
Dual-wheel load rating (1,000 lb)	25–130	23–98	30–55

Only asphalt pavement runways constructed after 2003 were considered to ensure that only the SuperPave mix design approach was used in projects constructed using state highway specifications. In addition, all runways were at least three years old to have adequate changes in pavement condition (West et al., 2023). Table 8 summarizes the mix design and field density criteria for the different specifications used.

Pavement condition index (PCI) ratings, which are numerical ratings of a pavement condition based on the type, severity, and extent of distresses observed on the pavement surface during visual inspections (FAA, 2023), for each of the projects collected over time, were compiled and summarized per the type of specification used. The PCI value of the pavement condition is represented by a numerical index between 0 and 100, where 0 is the worst possible condition and 100 is the best possible condition (FAA, 2023). Both the state airport and state highway specifications were grouped into one specification type, referred to as “state.” Least squares linear regression equations were fit on PCI against age for FAA and state specifications and had coefficient of determination (R^2) values of 0.8754 and 0.8006, respectively (West et al., 2023). Furthermore, the regression equations yielded a PCI rating of about 60 after 14 years for both specification types. Analysis of variance (ANOVA) results yielded a p-value of 0.824, indicating that the effect of specification type is not statistically significant (West et al., 2023).

Table 8. Mix Design and Field Density Criteria for the Different Specification Types Used

Specification	FAA	State Airport (IL)	State Airport (MI)	State Highway (GA)	State Highway (WI)
Gyrations	50	30	Not used	50, 75, 100	75
Marshall Blows	Not used	Not used	(50 blows)	Not used	Not used
Air Void (%)	3.5	2.0–4.0	2.5	4.0	3.5–4.0
VMA for 9.5 NMAS	16.0	Specifies VFA	Not used	15.0	15.0
VMA for 12.5 NMAS	15.0	Not used	14.5	14.0	14.0
VMA for 19.0 NMAS	14.0	Not used	Not used	13.0	Not used
RAP (%)	≤ 30 (shoulders)	Only in base layers	Not used	Only shoulders or non-interstates	Only in shoulders and lower layers
Rutting test	APA for aircraft > 60,000 lb	Not specified	Not used	APA	HWT
Marshall stability (lb)	Not used	Not used	≥ 1,000	Not used	Not used
Field density (%)	95–98	93–99	93–99	93–95	≥ 92.8

In addition, most distresses observed in projects using FAA and state specifications were environmental-related and consisted of longitudinal and transverse cracking. Only 8 of the 40 projects evaluated had load-related distresses; five of which used state specifications and three used FAA specifications. In addition, none of the projects with state specifications exhibited rutting distresses, while two projects that used FAA specifications exhibited this load-related distress (West et al., 2023).

CHAPTER 3: EXPERIMENTAL METHODS, TESTING PROGRAM, AND MATERIALS

The experimental plan involved the mix design and performance testing of laboratory-produced and laboratory-compacted specimens and the performance testing of plant-produced and laboratory-compacted specimens. This chapter introduces the HMA used in the study and the performance testing methods selected to characterize these mixes. Both highway and airfield surface and binder mixes were evaluated. Eighteen mixes were used in the research testing program: 15 surface mixes and 3 binder mixes. Of the surface mixes, seven were laboratory designed (five highway, two airport), and eight were plant mixes (four highway, four airport). The binder mixes were all airport mixes, constituting 1 laboratory-designed and 2 plant-produced mixes.

ASPHALT MIXES

Laboratory-Designed Mixes

Eight laboratory-designed mixes, per the Illinois modified AASHTO M 323 specification, were used in this study and are presented in Table 9. The Bailey method was used to produce mix design trials. All highway mixes were designed to have an air void content of 4% and minimum VMA of 15% except for H4 and H5, which are Superpave5 mixes and consequently designed to have an air void content of 5% and a minimum VMA of 16%.

Table 9. Major Characteristics of Laboratory Mixes

Mix ID	H1	H2	H3	H4*	H5*	A1	A2	AB1
Design Specification	IDOT Highway	IDOT Highway	IDOT Highway	IDOT Highway	IDOT Highway	IDOT Airport	IDOT Airport	IDOT Airport
Number of Gyrations	70	70	50	50	50	30	40	30
Binder PG	64-22	70-22	64-22	64-22	64-22	64-22	64-22	64-22
NMAS (mm)	9.5	9.5	9.5	9.5	9.5	9.5	9.5	19.0
Friction Grade	C	D	D	D	D	C	C	N/A
Lithology	100% LS	50% LS and 50% GR	100% DL	50% TR and 50% LS	100% DL	100% LS	100% LS	50% LS and 50% GR
Binder Content (%)	6.0	6.2	6.2	6.0	6.1	6.0	6.0	4.9
Air Void (%)	3.9	4.0	4.1	4.9	5.1	2.0	2.8	3.1
VMA (%)	14.9	15.2	15.2	16.2	15.9	13.3	14.5	11.8
Dust/AC ratio	0.85	0.95	0.8	0.99	1.0	0.83	1.1	0.85
FRAP (%)	15	16	15.5	18	15.5	0	0	15.5
ABR (%)	11.7	12.1	11.7	14.1	11.9	0	0	15

H: Highway; A: Airport; AB: Airport Binder; *: Superpave5; LS: Limestone; DL: Dolomite; TR: Trap-rock; GR: Gravel

Airport mixes had a design air void between 2%–4%, as allowed in the IDOT Aeronautics standard specifications. Notable differences between highway and airport mixes include the number of gyrations and air void content, which are lower for airport mixes. In addition, RAP is not used in airport mixes, except for binder mixes. For relative comparison, the binder content was kept around 6.1% for surface mixes. The dust-to-binder ratio was kept between 0.8 and 1.0, per specification requirements. Superpave5 mixes were added to the matrix, as they provide an advantage of easier field compaction to required density as compared to conventional Superpave4 mixes using the same rolling pattern. A highway binder mix was not added to the laboratory mix matrix, as they typically have the same properties as an airport binder mix in terms of VMA, nominal maximum aggregate size (NMAS), and RAP incorporation.

Binders Used in Laboratory Mixes

The same PG 64-22 was used in the laboratory-designed mixes in this study except for mix H2, where a PG 70-22 was used. This would ensure relative comparison and minimize variability. The asphalt binder was sampled from Emulsicoat, Inc. in Urbana, Illinois.

Aggregates Used in Laboratory Mixes

The aggregate stockpiles used in this study were mostly from the state of Illinois, with some from Missouri. This ensured a good representation of the available aggregates used for pavement construction in Illinois. Table 10 presents more details of the stockpiles used in the laboratory mix designs. Aggregates were sampled from respective construction plants according to Illinois Test Procedure 2.

Table 10. Aggregate Lithology and Locations for Laboratory Mixes

Material ID	CM11	CM16	FM22	FM20	FM01
H1	N/A	Limestone from Nokomis, IL	NA	Crushed stone from Kankakee, IL	Gravel from Heyworth, IL
H2	N/A	Limestone and gravel from Weston and Heyworth, IL	N/A	Gravel from Heyworth, IL	Gravel from Heyworth, IL
H3	N/A	Dolomite from Rockdale, IL	Gravel from Pekin, IL	N/A	Gravel from Pekin, IL
H4	N/A	Traprock from Ironton and Huntington, MO	Limestone from Huntington, MO	Limestone from Huntington, MO	N/A
H5	N/A	Dolomite from Rockdale, IL	Gravel from Pekin, IL	N/A	Gravel from Pekin, IL
A1	N/A	Dolomite from Rockdale, IL	N/A	N/A	Gravel from Pekin, IL
A2	N/A	Limestone from Charleston, IL	N/A	Limestone from Casey, IL	Gravel from Greenup, IL
AB1	Dolomite from Rockdale, IL	Dolomite from Rockdale, IL	N/A	N/A	Gravel from Pekin, IL

IL: Illinois; MO: Missouri.

The mineral filler used was a limestone material obtained from Linwood in Iowa with 88% and 95% passing the #100 and #200 sieves, respectively. The RAP was sampled from the I-55 highway section along the Bloomington corridor in IDOT District 5. It had 9.5 mm (0.37 in) NMAS and a binder content of 4.95%.

Plant-Produced Mixes

Ten plant-produced mixes with diverse mix properties were sampled for testing. They included four highway and six airport mixes, as presented in Table 11. Plant-produced mixes were blended and split in accordance with the Illinois modified AASHTO R 76. Fine-graded mixes were added to the matrix, as they provide a smoother and less porous surface as well as easier field compaction to required density. A highway binder mix was not added to the plant mix matrix, as they typically have the same properties as an airport binder mix in terms of VMA, NMAS, and RAP incorporation.

Table 11. Major Characteristics of Plant Mixes

Mix Property	PH1	PH2[FG]	PH3	PH4[FG]	PA1	PA2	PA3	PA4	PAB1	PAB2
Design Specification	IDOT Highway	IDOT Highway	IDOT Highway	IDOT Highway	IDOT Airport	IDOT Airport	FAA	FAA	IDOT Airport	FAA
Number of Gyrations	50	50	50	70	40	30	50	75	30	75
Binder PG	64-22	64-22	64-22	64-22	64-22	64-22	64-22	64-22	64-22	64-22
NMAS (mm)	9.5	9.5	9.5	9.5	9.5	9.5	12.5	9.5	19.0	12.5
Friction Grade	C	C	C	C	C	D	C	C	N/A	N/A
Lithology	100% LS	100% LS	100% LS	100% LS	100% LS	100% DL	100% LS	100% LS	100% LS	100% LS
Binder Content (%)	6.0	6.3	5.9	6.0	6.1	6.4	6.0	6.2	6.2	5.7
Air Void (%)	4.0	4.0	4.0	4.0	3.0	2.0	3.5	3.5	4.0	3.5
VMA (%)	15.5	15.2	15.3	15.9	15.0	14.3	15.9	15.5	15.2	14.3
Dust/AC Ratio	0.83	0.89	0.98	0.9	0.91	1.04	0.87	0.97	0.95	1
RAP (%)	15	15	16	10	0	0	0	0	16	20
ABR	12.5	11.9	15.9	7.8	0	0	0	0	23.8	18.1

PH: Plant Highway; PA: Plant Airport; [FG]: Fine-graded mix; PAB: Plant Airport Binder; LS: Limestone; DL: Dolomite.

Aggregates Used in the Plant Mixes

Table 12 presents the details of the aggregate materials used in the plant mixes.

Table 12. Aggregate Lithology and Locations for Plant Mixes

Material ID	CM11	CM16	FM20	FM02	FM01	MF
PH1	N/A	Limestone from Manteno, IL	Limestone from Manteno, IL	N/A	Gravel from Paxton, IL	Fly ash from Decatur, IL
PH2[FG]	N/A	Limestone from Manteno, IL	Limestone from Manteno, IL	N/A	Gravel from Paxton, IL	Fly ash from Decatur, IL
PH3	N/A	Limestone from Nokomis, IL	Crushed stone from Kankakee, IL	N/A	Gravel from Heyworth, IL	Limestone from Ocoya, IL
PH4[FG]	N/A	Limestone from Nokomis, IL	Limestone, Pana, IL & Granite, Ironton, MO	N/A	Gravel from Heyworth, IL	Limestone from Ocoya, IL
PA1	N/A	Limestone from Elgin, IL	Limestone from Lagrange, IL	Gravel from Elgin, IL	N/A	West Chicago, IL*
PA2	N/A	Dolomite from Rockdale, IL	N/A	N/A	Gravel from Pekin, IL	Urbana, IL*
PA3	N/A	Limestone from Huntington, MO	Limestone & Traprock from Huntington and Ironton, MO	N/A	N/A	Quincy, IL*
PA4	N/A	Limestone from Fairmouth, IL	Limestone from Cayuga and West Lebanon, IN	N/A	N/A	Limestone from Thornton, IL
PAB1	Limestone from Elgin, IL	Limestone from Elgin, IL	Limestone from Sycamore, IL	Gravel from Elgin, IL	N/A	West Chicago, IL*
PAB2	Limestone from Fairmouth, IL	Limestone from Fairmouth, IL	Limestone from Cayuga and West Lebanon, IN	N/A	N/A	Limestone from Thornton,IL

IL: Illinois; MO: Missouri IN: Indiana; *Lithology not specified

Binder Grade of the Plant Mixes

Plant mixes were selected to ensure that they contained PG 64-22 binder. As with the laboratory mixes, the decision to use the same binder grade was to ensure relative comparison and minimize variability.

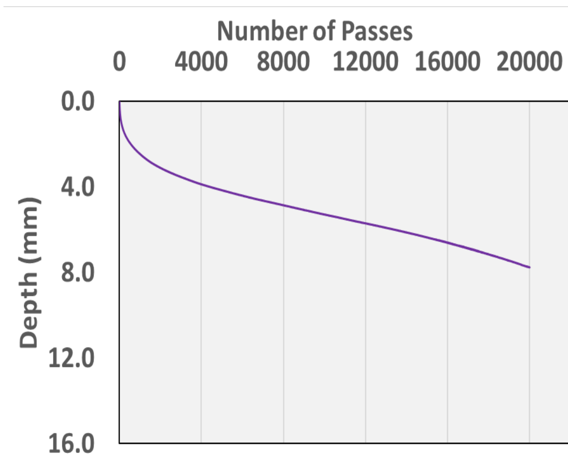
MIX PERFORMANCE CHARACTERIZATION

Mix performance testing was performed for all mixes in the study. Two air void contents (7% and 4%) were considered for each test and mix. Air void specimens of 7% were used to represent highway mix evaluation while the 4% air void specimens were tested to replicate the relatively high compaction density achieved at nonprimary airports. The following section describes all performance tests in detail.

Hamburg Wheel-Tracking Test

The HWTT was conducted to predict potential rutting and moisture susceptibility performance of mixes. Two 62 ± 1 mm (2.44 ± 0.039 in) specimens were submerged in a temperature-controlled water bath at 50°C (122°F) and repetitively loaded using a reciprocating steel wheel weighing 71.7 ± 0.45 kg (158 ± 1 lb), in accordance with the Illinois modified AASHTO T324, as presented in Figure 8-A. The wheels made 52 ± 2 pass/min, and an automated system measured the deformation yielding results, as presented in Figure 8-B.

A. A specimen on the smart tracker machine

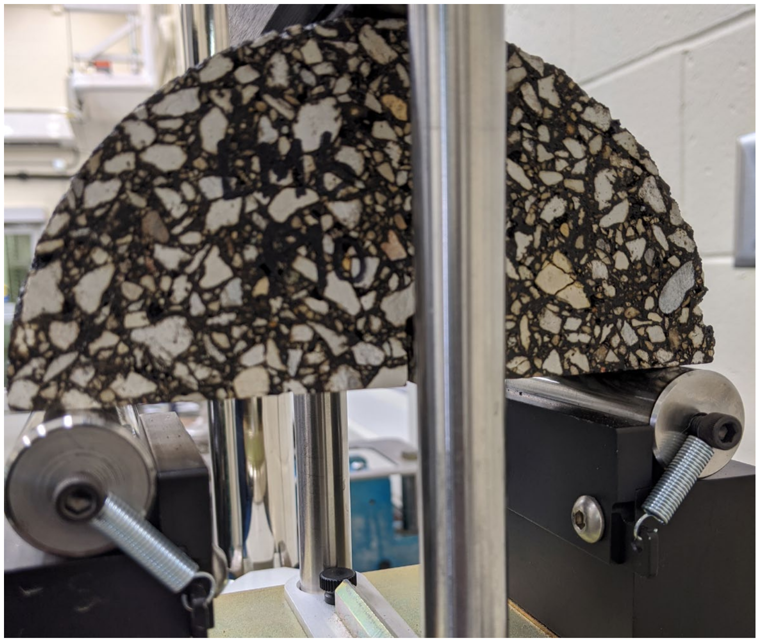


B. Typical HWT test result

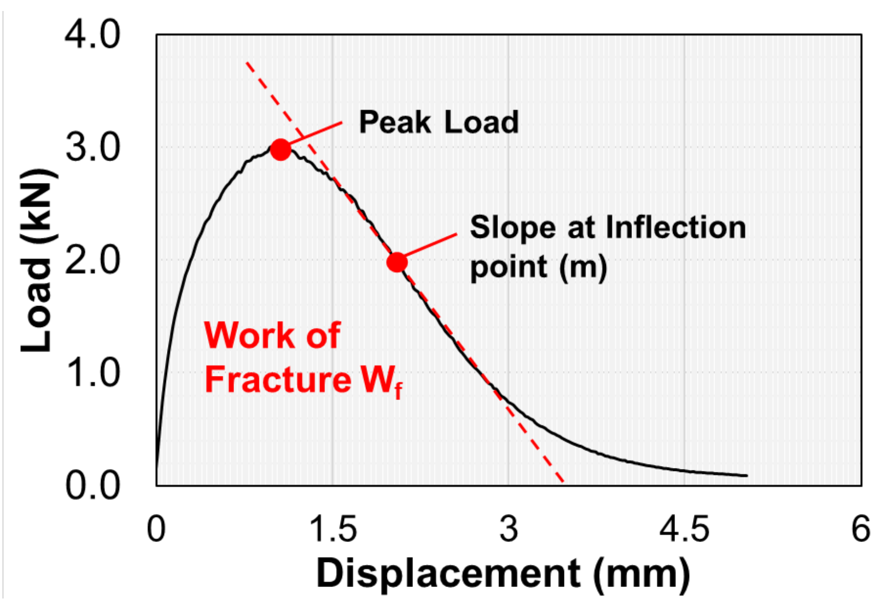
Figure 8. Graph. HWTT machine and a typical test result.

Illinois Flexibility Index Test

Cracking potential of mixes was evaluated using I-FIT according to the Illinois modified AASHTO T393. Compacted samples of 160 mm (6.3 in) height and 150 mm (5.9 in) diameter were sawed to a thickness of 50 ± 1 mm (1.97 ± 0.039 in). These specimens were then fabricated into semicircular halves and notch cut into the flat rectangular surface. Notch depth and width were 15 ± 1 mm (0.59 ± 0.039 in) and 1.5 ± 0.1 mm (0.059 ± 0.0039 in), respectively.



A. A specimen on the I-FIT fixture



B. Typical I-FIT test result

Figure 9. Photo and Graph. I-FIT specimen and a typical test result.

The specimens were conditioned for 2 hours in a 25°C (77°F) water bath and then placed onto the test fixture presented in Figure 9-A. A load line displacement (LLD) at a rate of 50 mm/min (1.97 in/min and an automated system measured the load and LLD as the test progressed, as presented in Figure 9-B.

Tensile Strength Ratio Test

The TSR test was used to evaluate moisture susceptibility and was performed in accordance with the Illinois modified AASHTO T283. Six compacted cylindrical specimens of 95 mm (3.74 in) height and 150 mm (5.9 in) diameter were used. Specimens were categorized into two groups of three, ensuring that the average air void content for each group was similar. Three specimens (conditioned) were saturated to between 70% and 80% and then conditioned in a 60°C (140°F) water bath for 24 hours. All specimens were then conditioned for 2 hours in a 25°C (77°F) water bath and then placed onto the test fixture shown in Figure 10.

Figure 10. Photo. A specimen on the TSR fixture.

An indirect tension test was performed on each specimen by applying a load at a constant rate of 50 mm/min (1.97 in/min). An automated system reported the peak load. The tensile strength values were computed for each specimen, and the TSR was calculated as the ratio of the average tensile strengths of conditioned to unconditioned specimens.

CHAPTER 4: RESULTS AND DISCUSSIONS

This chapter presents the results of the various performance tests conducted along with a discussion of the results and findings. For highway mixes, specimens at an air void of 7% were prepared per the Illinois modified AASHTO T324 requirement in the *Illinois Manual of Test Procedures* as well as practical reasons because highway pavements are usually compacted to about 93% density in the field. In contrast, airport mixes for nonprimary airports are compacted to 96% density in the field. Therefore, laboratory specimens for airport mixes were prepared at an air void of 4%. To facilitate relative comparisons between highway and airport mixes, both 4% and 7% air void specimens were prepared for all laboratory performance tests.

HAMBURG WHEEL-TRACKING TEST

Laboratory-Designed Mixes

Hamburg wheel-tracking tests were performed for laboratory-designed mixes, and the results are summarized in Table 13.

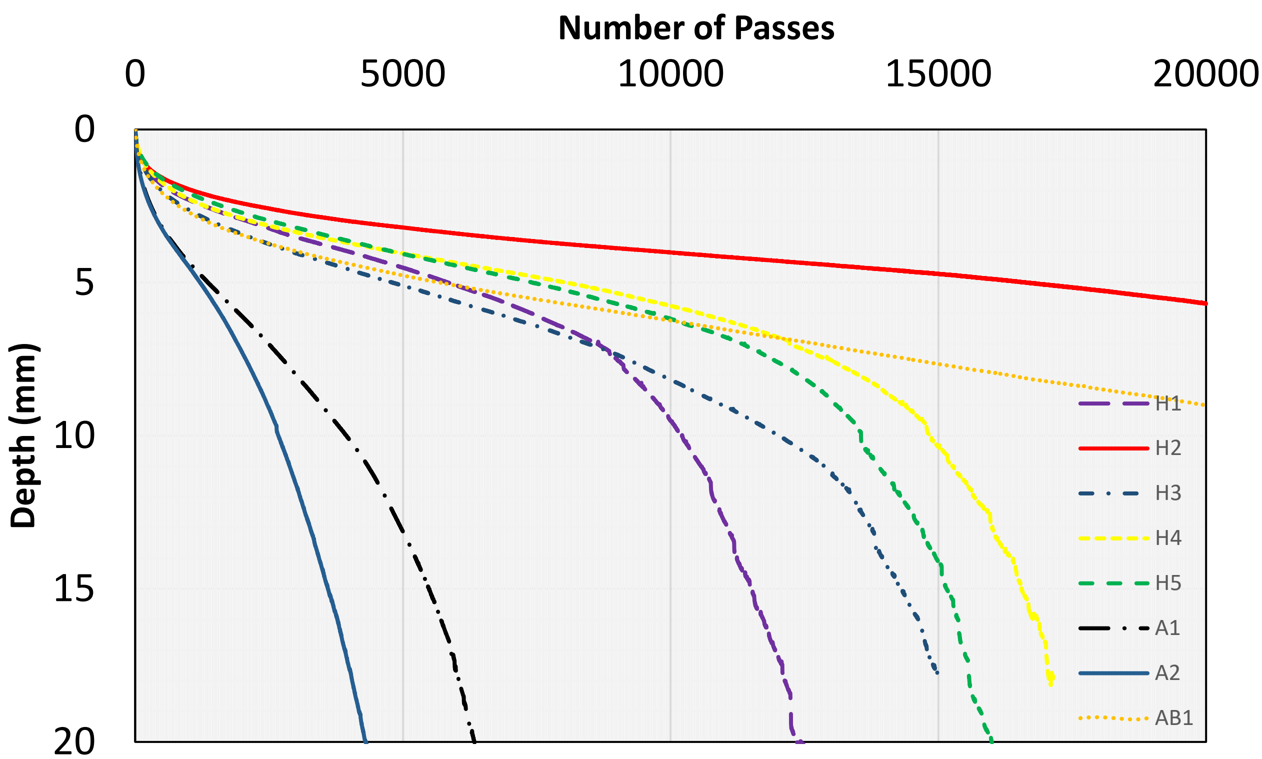
Table 13. HWTT Results for Laboratory Mixes

	Air Void (%)	**7.0**	**7.0**	**4.0**	**4.0**
Mix	Minimum # Passes	Rut depth (mm)	Passes to 12.5 mm criteria	Rut depth (mm)	Passes to 12.5 mm criteria
H1	7,500	6.1	10,920	1.2	> 20,000
H2	15,000	4.7	> 20,000	4.0	> 20,000
H3	7,500	6.4	13,560	4.6	13,742
H4*	7,500	4.8	15,926	3.0	> 20,000
H5*	7,500	5.0	14,492	3.4	15,938
A1	7,500	Failed (> 15)	4,608	Failed (> 15)	6,353
A2	7,500	Failed (>15)	3,170	Failed (>15)	4,894
AB1	7,500	5.6	> 20,000	2.5	> 20,000

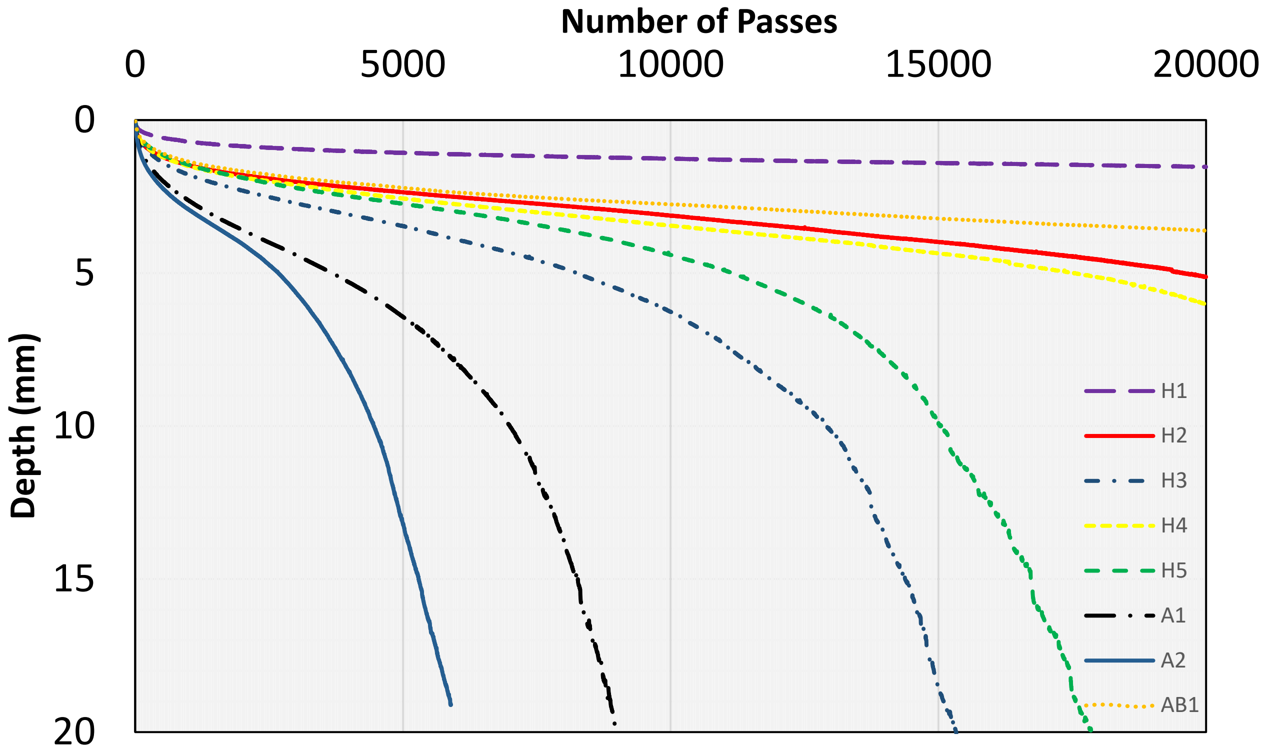
*: Superpave5 mixes

All laboratory mixes at 7% air void content had rut depth less than 12.5 mm (0.49 in) at their required number of load passes, except airport laboratory mixes A1 and A2, which failed at just 4,608 and 3,170 passes respectively. The majority of these two mixes' fine aggregate portion was natural sand, which has rounded surfaces that contribute to faster rut progression under the HWTT. They contain little or no manufactured sand content. Mix A1 and A2 had 0% and 5.2% manufactured sand, respectively. Manufactured sand has crushed faces, which cause slower rut progression. Mix H2, which had a PG 70-22 binder, and mix AB1, which was an airport binder mix with 19 mm (0.75 in) NMAS, had rut depths lower than 12.5 mm (0.49 in) after 20,000 passes. All other highway mixes were comparable in terms of rut depth at the specified number of passes and the number of passes to 12.5 mm (0.49 in) rut criteria. All highway mixes required more than 10,000 passes to get to 12.5 mm (0.49 in).

All mixes were also tested at 4% air void content. From discussions with contractors and available data, airports are usually constructed to higher densities of 95%–97%. The HWTT results for this air void content are presented in Table 13. The major differences in lower rut depths and higher passes to 12.5 mm (0.49 in) rut criteria are due to the lower air void content. The rut progression curves in Figure 11-A and 11-B demonstrate a similar profile except they are shifted to the right in Figure 11-B, indicating that it took a higher number of passes to get to that same rut depth. The rut depth was reduced for mixes A1 and A2 with air void reduction from 7% to 4%. The number of passes to 12.5 mm rut depths increased from 4,608 and 3,170 at 7% air void content to 6,353 and 4,894 at 4% air void content, respectively. However, these were still less than the 7,500-pass threshold for a mix with PG 64-22 binder. Appendix A presents the volumetrics properties of all HWTT specimens.



A. HWTT results for laboratory mixes at 7% air void content



B. HWTT results for laboratory mixes at 4% air void content

Figure 11. Graphs. HWTT results for laboratory mixes.

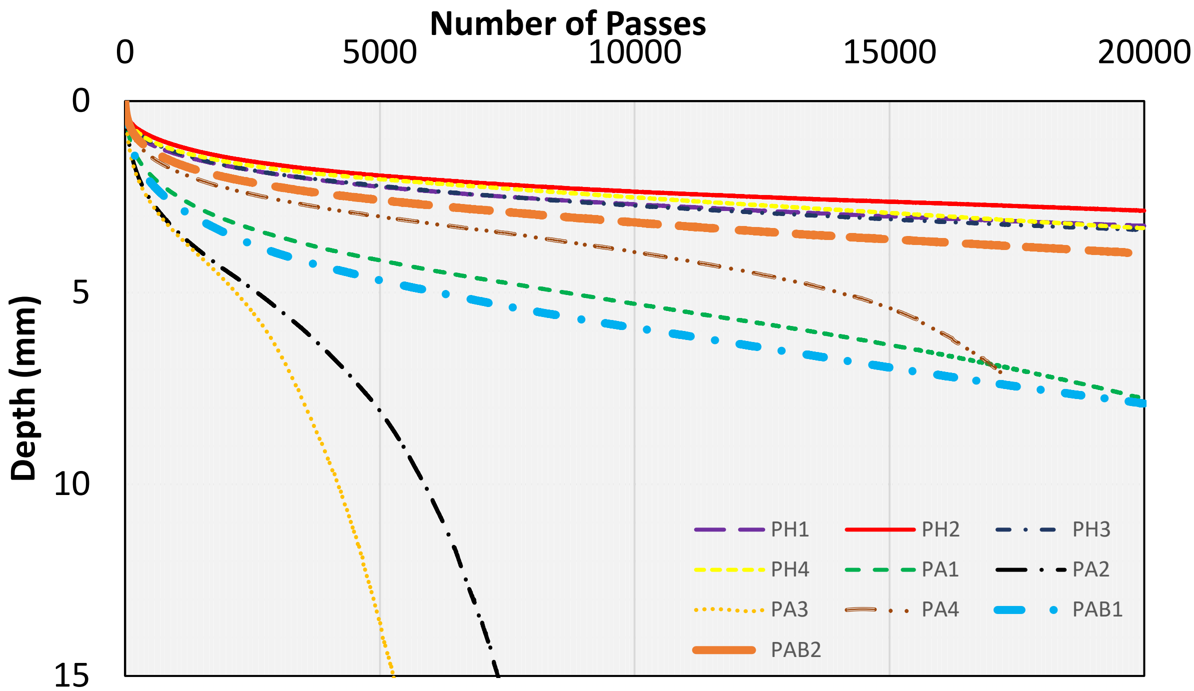
Plant-Produced Mixes

Similar to the laboratory-designed mixes, HWTT was performed for plant-produced mixes, and the results are summarized in Table 14. At 7% air void content, all plant mixes had rut depth less than 12.5 mm (0.49 in) at their required number of passes except mixes PA2 and PA3, airport plant mixes, which failed at just 6,244 and 4,768 passes, respectively. PA2, just like airport laboratory mix A1, did not contain any manufactured faced-crushed sand. The fine aggregate portion of this mix was only natural sand, which has a rounded surface that could lead to faster rut progression under the Hamburg wheel. Airport plant mix PA3, with manufactured sand constituting 55.5% of its aggregate blend and 5.3 mm in the APA from its design mix sheet, was the anomaly and had the fastest rut progression of all plant mixes. Two additional bags of this mix were split to verify the mix G_{mm} after which HWTT samples at 7.0% were reproduced. The new samples failed at 5,028 passes, which is similar to that shown in Table 14. The G_{mm} samples were extracted, and the binder content was 6.0% as per the mix design, while the gradation was very similar to that in the job mix formula and is shown in Appendix A. Airport mixes PA1 and PA4, which had manufactured sand in their blend, had better performance than PA2. The plant-binder mixes PAB1 and PAB2 had exceptionally better performance because of their 19 mm (0.75 in) NMAS. All other highway plant mixes were comparable in terms of both rut depths and number of passes to 12.5 mm (0.49 in). All highway plant mixes required more than 20,000 passes to get to 12.5 mm (0.49 in).

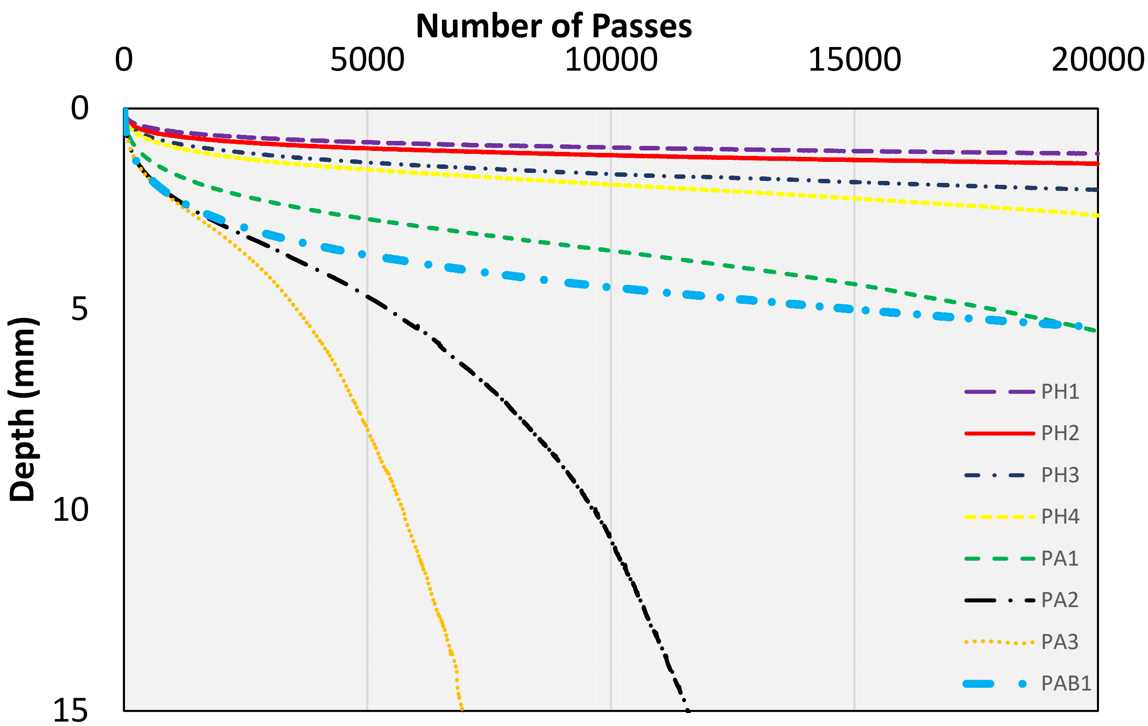
Plant mixes were also tested at 4% air void content. Like laboratory mixes, the major differences were reduced rut depths and higher passes to 12.5 mm (0.49 in) due to the lower air void content. Likewise, the plots in Figure 12 demonstrate a similar trend, with the curves in Figure 12-B shifted to the right relative to Figure 12-A. This indicates an increased number of passes at 4% air void content to obtain the same rut depths as at 7%. Mix PA2 had reduced rut potential at 4% air void content, requiring more than 7,500 passes to get to the 12.5 mm (0.49 in) threshold unlike at 7%. The number of passes to 12.5 mm rut depth for airport plant mix PA3 increased from 4,768 at 7% air void content to 6,462 at 4% air void content, but this was less than 7,500 passes required for a mix with PG 64-22 asphalt binder. The HWTT was not performed at 4% air void content for mixes PA4 and PAB2 because of insufficient materials. Appendix B presents the volumetrics properties of all HWTT specimens.

Table 14. HWTT Results for Plant Mixes

	Air Void (%)	**7.0**	**7.0**	**4.0**	**4.0**
Mix	**# Passes**	**Rut depth (mm)**	**Passes to 12.5 mm**	**Rut depth (mm)**	**Passes to 12.5 mm**
PH1	7,500	2.5	> 20,000	0.9	> 20,000
PH2	7,500	2.2	> 20,000	1.1	> 20,000
PH3	7,500	2.5	> 20,000	1.5	> 20,000
PH4	7,500	2.3	> 20,000	1.7	> 20,000
PA1	7,500	4.7	> 20,000	3.2	> 20,000
PA2	7,500	Failed (17.6mm)	6,244	6.7	10,696
PA3	7,500	Failed	4,768	Failed (17.5mm)	6,462
PA4	7,500	3.4	>20,000	–	–
PAB1	7,500	5.4	> 20,000	4.1	> 20,000
PAB2	7,500	2.9	> 20,000	–	–



A. HWTT results for plant mixes at 7% air void content.



B. HWTT results for plant mixes at 4% air void content.

Figure 12. Graphs. HWTT results for plant mixes.

ILLINOIS FLEXIBILITY INDEX TEST

Laboratory-Designed Mixes

I-FIT was performed on laboratory-designed mixes, and the results are presented in Figures 13 through 15. The fracture energy and the post-peak slopes are presented in Figures 13 and 14, respectively.

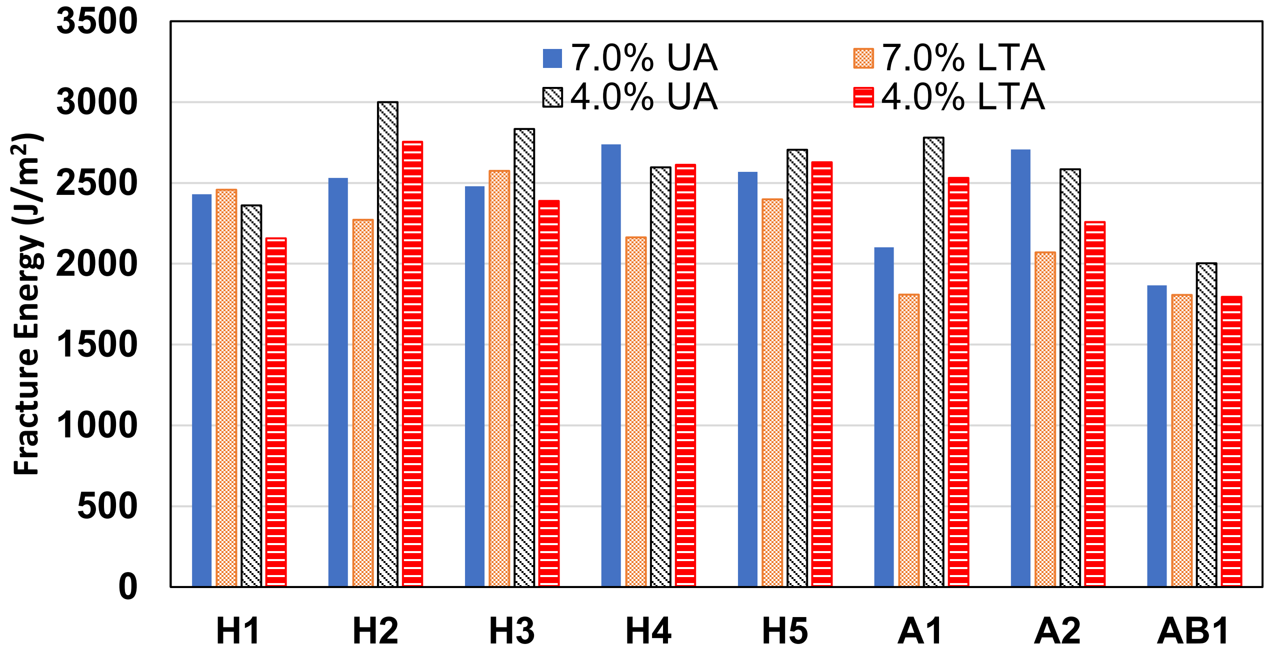


Figure 13. Bar plots. Fracture energy results for laboratory mixes.

All fracture energies were comparable with values above 1,800 J/m^2. Highway mixes had relatively higher values than airport mixes. The post-peak slopes in Figure 14 were higher for highway mixes than airport mixes except for AB1, a binder mix, indicating an increased crack propagation rate.

The flexibility index (FI) values present a full picture of the cracking potential of a mixture. Two mixtures could have similar FI values but very different fracture energies and post-peak slopes. It is desired to have a high fracture energy and relatively low post-peak slope rather than a low fracture energy and much lower post-peak slope yielding similar FI values.

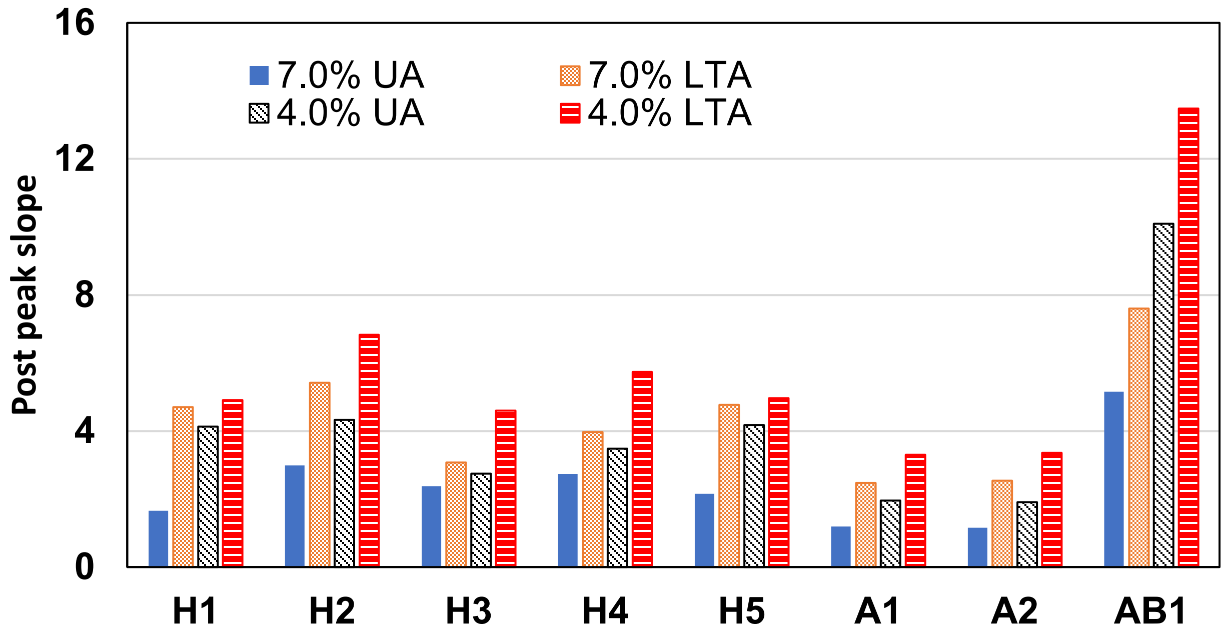


Figure 14. Bar plots. Post-peak slope results for laboratory mixes.

The FI results for the laboratory mixes are presented in Figure 15. The red (top) and black (bottom) horizontal lines represent the IDOT threshold of 8.0 and 5.0 for unaged and aged laboratory mix specimens, respectively.

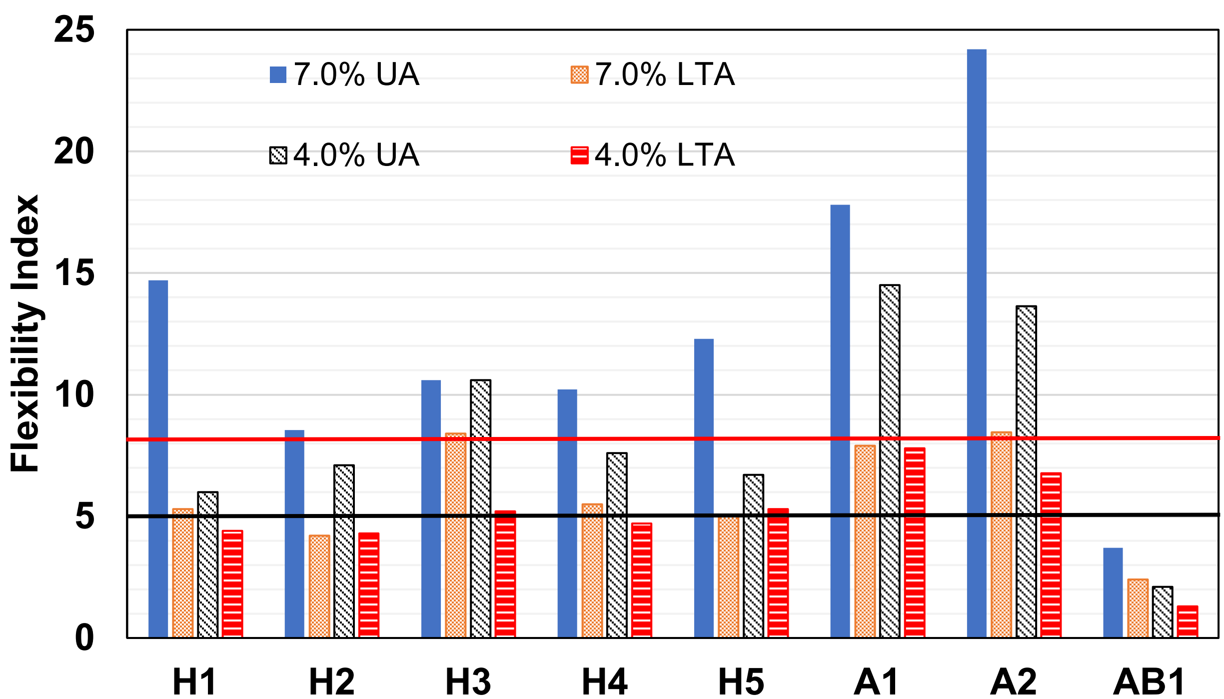


Figure 15. Bar plots. Flexibility index results for laboratory mixes.

At 7% air void content, all mixes passed the unaged threshold except AB1, which is a binder layer mix, and the I-FIT is not a requirement. Given that aged FI is most important for field performance, all the mixes passed the aged threshold except for H2, a highway mix with PG 70-22, and airport binder mix AB1. At 4% air void content, there was a decrease in FI for all aged mixes. Appendix C summarizes the properties of all I-FIT specimens, including the slope, fracture energy, strength, ligament length, and thickness.

Plant-Produced Mixes

I-FIT was also performed on plant-produced mixes. Figure 16 displays the fracture energies. Airport mix PA1 had slightly higher values than the highway mixes, which had comparable values. All unaged specimens had fracture energies higher than 1,800 J/m^3. No clear trend in the fracture energies was observed with aging. I-FIT was not performed at 4% air void content for airport plant mixes PA4 and PAB2 because of insufficient materials.

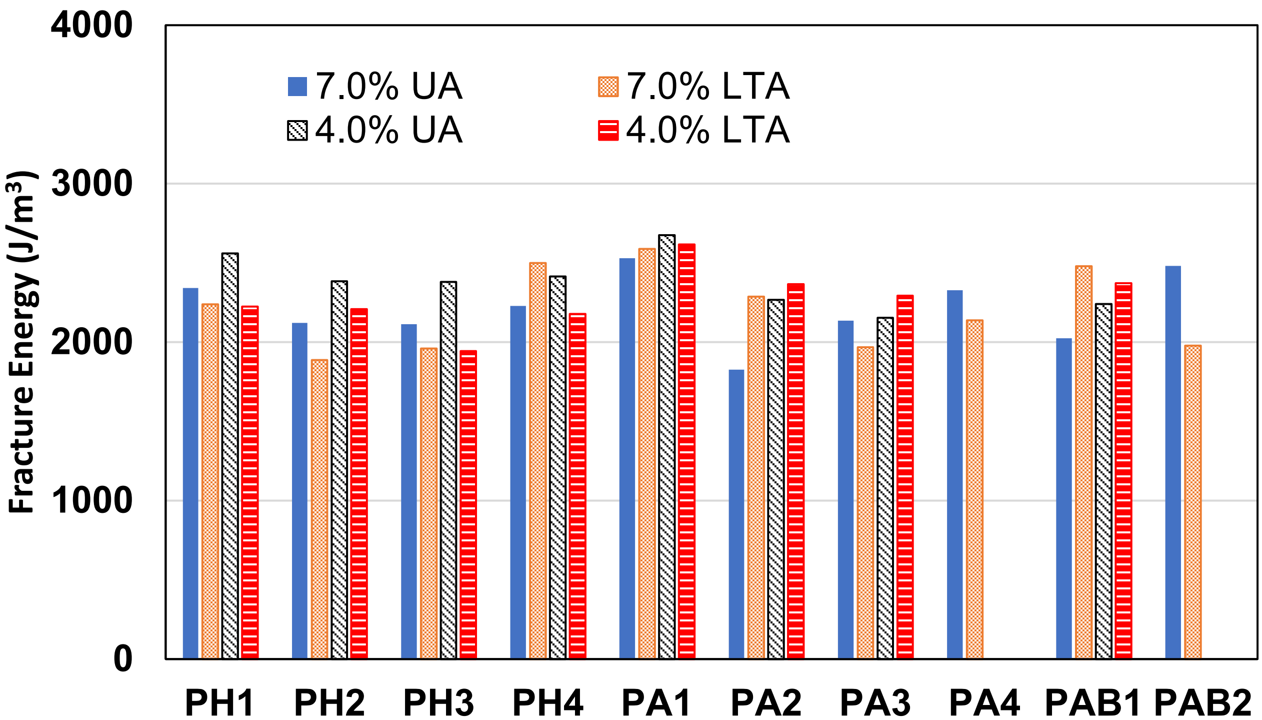


Figure 16. Bar plots. Fracture energy results for plant mixes.

Figure 17 presents the post-peak slope values. Most of the mixes had post-peak slopes between 2.0 and 2.9 for unaged specimens at 7% air void content. Plant highway mix PH4 and airport mix PA3 had the lowest post-peak slope of 1.5 for unaged specimens at 7%, indicating greater ductility. In addition, as would be expected, the slope increased with aging because of the increase in specimen brittleness.

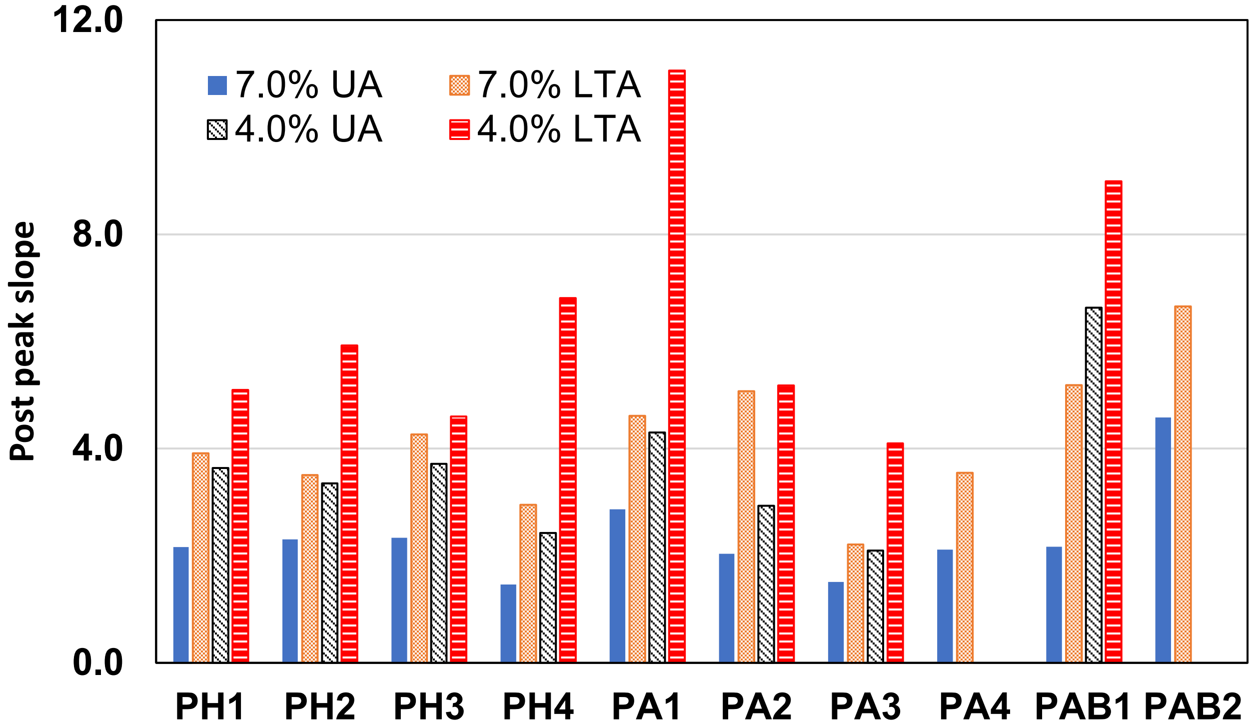


Figure 17. Bar plots. Post-peak slope results for plant mixes.

The FI values for the plant mixes at the unaged and aged conditions at both 7% and 4% air void contents are presented in Figure 18. The red (top) and black (bottom) horizontal lines represent the IDOT thresholds of 8.0 and 4.0 for unaged and aged plant mix specimens, respectively. At 7% air void content, all mixes passed both the unaged and aged FI thresholds except airport plant mix PAB2, which had aged FI less than 4.0. It should be noted that I-FIT is not required for binder mixes. Airport plant mix PA3 and highway fine-graded mix PH4 had similar results and were the best-performing mixes. Given that aged I-FIT is most important for field performance, PH4 and PA3 had aged FI values of 8.5 and 9.0, respectively, which is significantly higher than the 4.0 threshold. Aging at both 7% and 4% air void contents resulted in lower FI values compared to unaged specimens at the same air void content. Testing specimens at 4% air void content also resulted in lower FI values except for airport plant mix PA2, which had aged FI of 4.6 at both air void contents. Appendix C summarizes the properties of all I-FIT specimens, including the slope, fracture energy, strength, ligament length, and thickness.

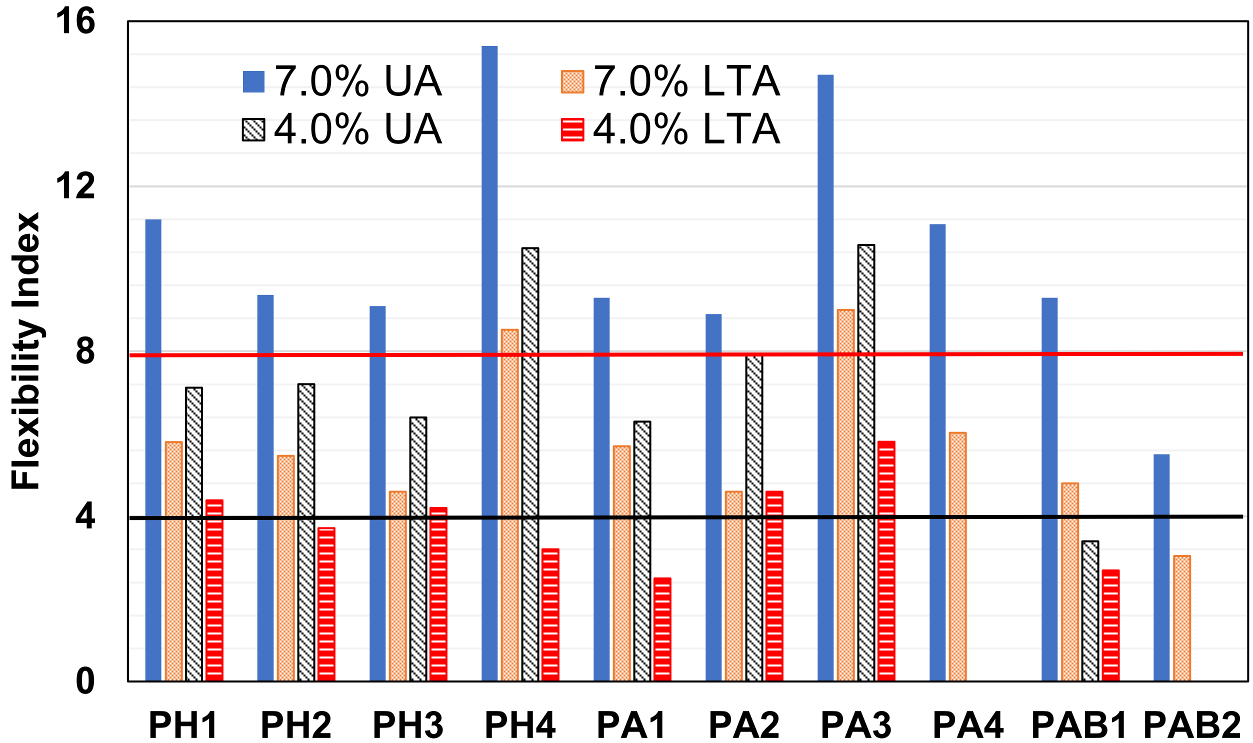


Figure 18. Bar plots. Flexibility index results for plant mixes.

TENSILE STRENGTH RATIO

Laboratory-Designed Mixes

The TSR test was performed for laboratory-designed mixes, and the results are presented in Figure 19. The IDOT tensile strength and TSR thresholds for mixes with virgin binders are 0.4 MPa (60 psi) and 0.85, respectively. This is presented by the black (bottom) and red (top) horizontal lines, respectively. For mixes with polymer-modified binders such as H2, the tensile strength threshold is increased to 0.55 MPa (80 psi). Although all laboratory mixes exceeded their respective tensile strength threshold, highway mixes had higher strengths compared to airport mixes except for binder mix AB1, with 19 mm (0.75 in) NMAS, which had similar strength at 4% air void content. Tensile strength increased with the decrease in air void content from 7% to 4%. In addition, all laboratory mixes regardless of the air void content had TSR above the 0.85 threshold with no clear trend between highway and airport laboratory mixes. Appendix D presents the properties of all TSR specimens, including the tensile load and cross-sectional area.

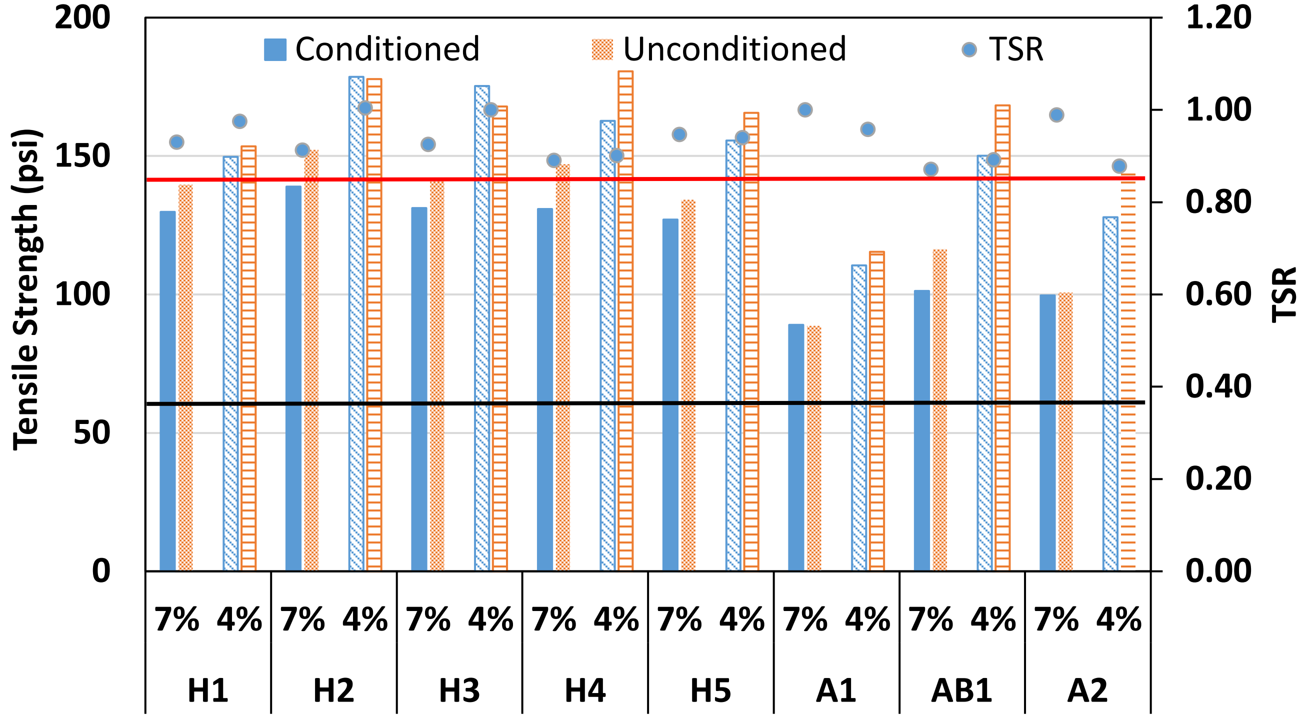


Figure 19. Bar plots. TSR test results for laboratory mixes.

Plant-Produced Mixes

All plant mixes met the minimum tensile strength of 0.4 MPa (60 psi) at both 4% and 7%, as presented in Figure 20. Similar to the laboratory mixes, highway plant mixes generally had higher tensile strength values than airport mixes, except for binder mix PAB1, which has 19 mm (0.75 in) NMAS. At 7% air void content, all plant mixes met the minimum TSR of 0.85 except for highway plant mixes PH4 and PH1, which had 0.82 and 0.8, respectively. These results were not expected, as both plant mixes had satisfied the TSR requirement during their design phase, as shown in their respective mix design sheets. A decrease in air void from 7% to 4% resulted in an increase in tensile strength for all plant mixes. The TSR at this reduced air void were all above 0.85 except for mixes PA2 and PAB1, which had 0.83 and 0.84, respectively. TSR test was not performed on mixes PA3, PA4, and PAB2 because of insufficient materials. Appendix D presents the properties of all TSR specimens, including the tensile load and cross-sectional area.

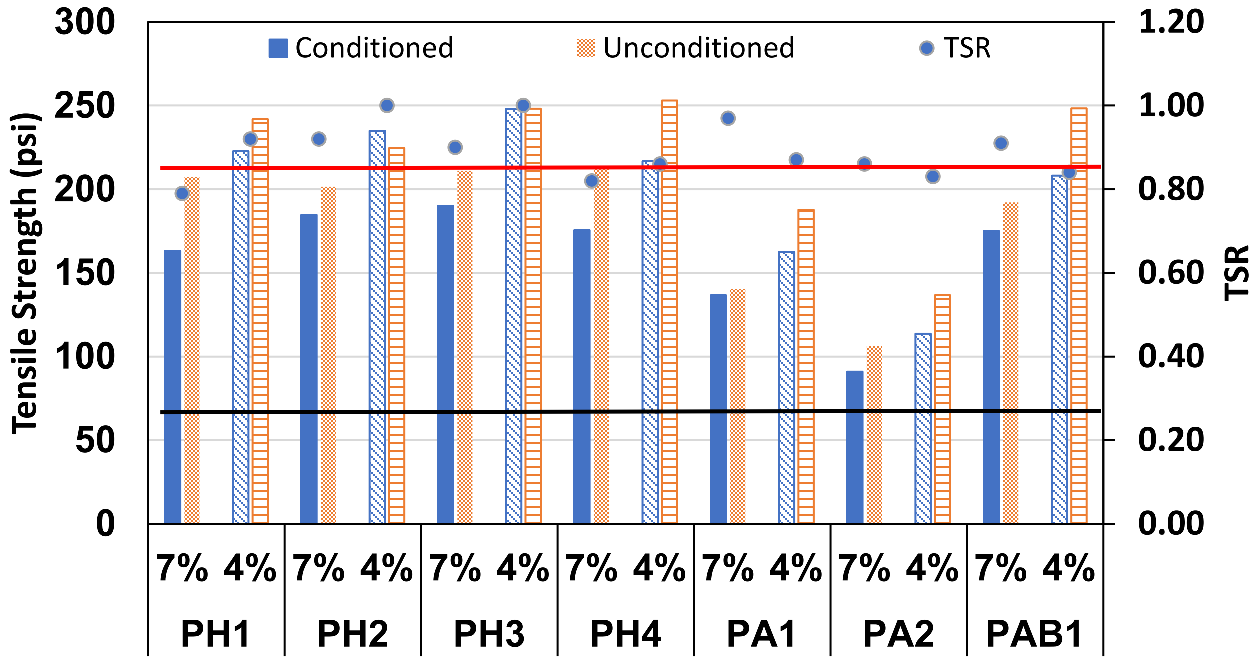


Figure 20. Bar plots. TSR test results for plant mixes.

PERFORMANCE RANKING

Figure 21 presents a comparison of all laboratory-designed mixes at 7% and 4% air void contents. For each air void content, mixes were ranked from 1 (worst) to 8 (best). The average ranking of these mixes per test was then used to plot the following bar plots, with the best mix placed at the top of the chart and the worst placed at the bottom. For mixes with a similar rank average (e.g., mixes H2 and H1 of Figure 21-A), the actual values from the test were used as a tiebreaker.

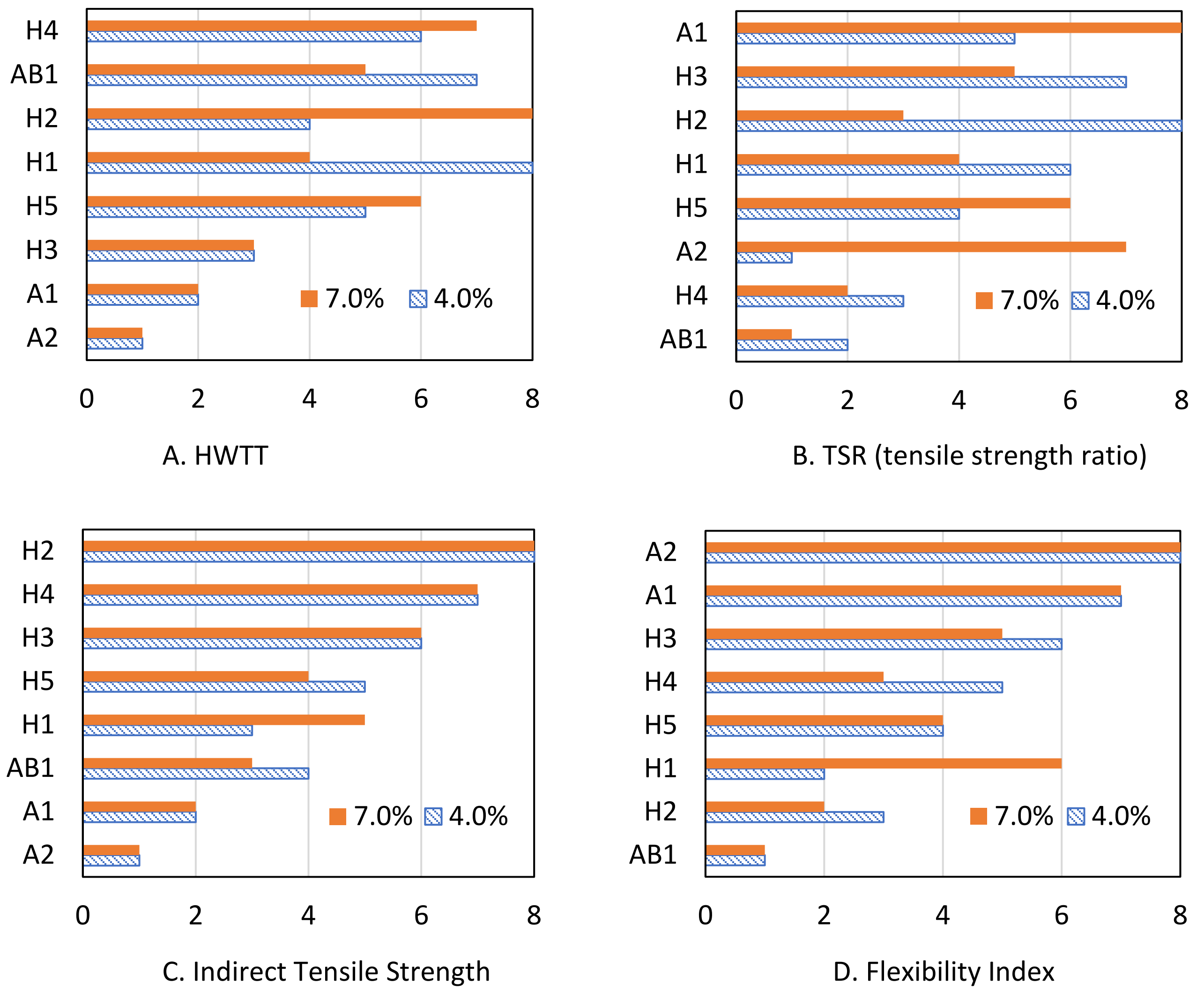


Figure 21. Bar plots. Performance test ranking of laboratory mixes.

Considering HWTT (Figure 21-A), most of the mixes (63%) had a similar ranking or a one-rank difference at 4% and 7%. This number was 25%, 88%, and 75% when considering the tensile strength ratio, tensile strength, and flexibility index results, respectively. Mix H4 was the best-performing mix in terms of HWTT. From Figure 21-B and Figure 21-C, there were differences in the rankings when using the tensile strength ratio and indirect tensile strength, respectively, as criteria. For the tensile strength, airport mixes have lower values while the tensile strength ratio is dependent on the lithology, absorption, and properties of the aggregates used in the mix.

Similarly, Figure 22 presents a comparison of all plant mixes at 7% and 4% using a similar approach as explained above. Seventy-five percent of the plant mixes had a similar HWTT ranking or a one-rank difference at 4% and 7%. The proportions were 29%, 100%, and 63% for the tensile strength ratio, indirect tensile strength, and flexibility index results, respectively. Airport plant mixes PA4 and PAB2 were not included in Figure 22 because no testing was performed at 4% air void content due to insufficient material.

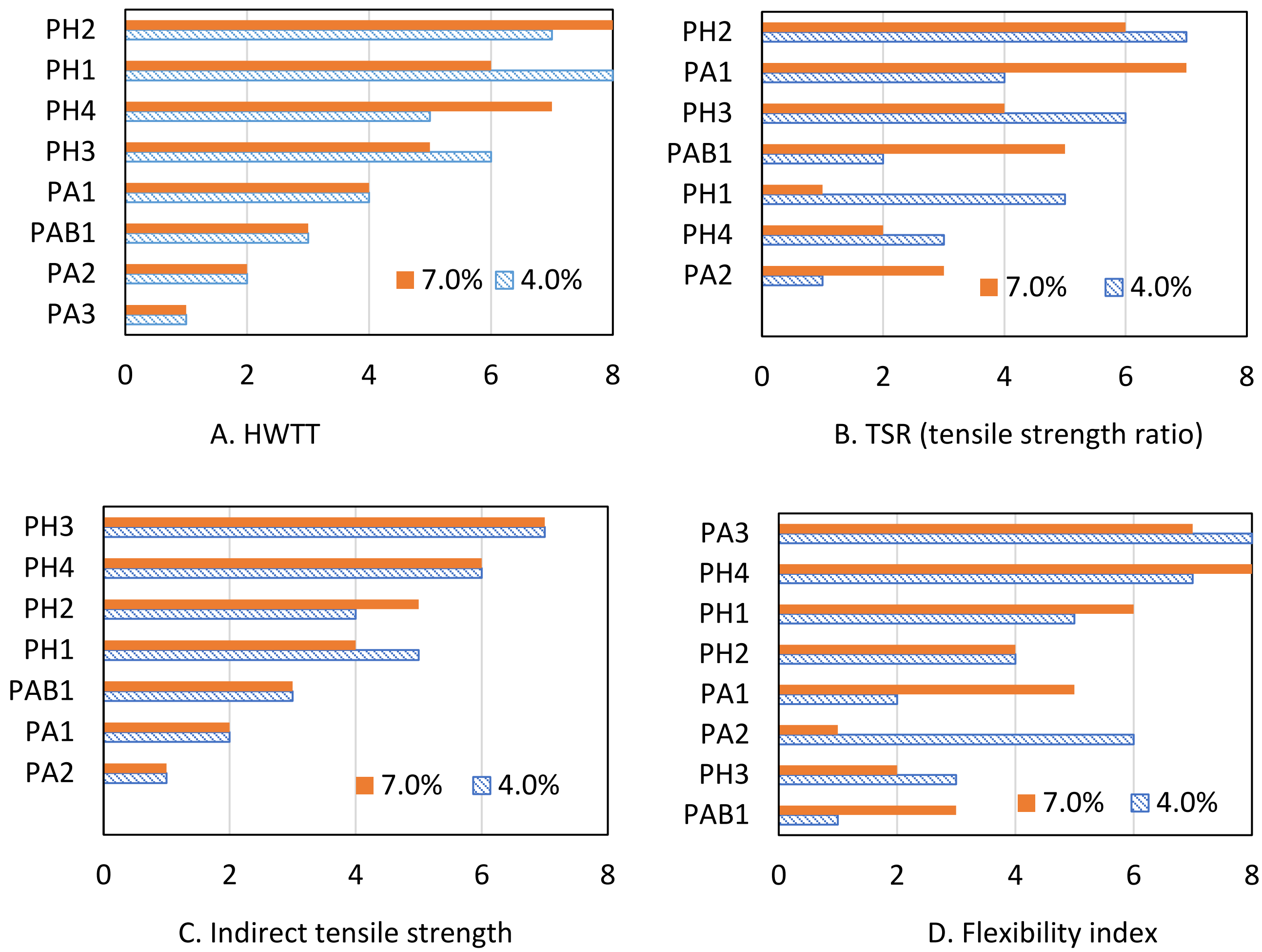


Figure 22. Bar plots. Performance test ranking of plant mixes.

IMPACT OF AIR VOID CONTENT ON PERFORMANCE

Figure 23 presents the correlation between the HWTT, TSR, and FI values at 7% and 4% air void contents. A positive correlation is observed for all mixes with considerably high coefficient of determination (R^2) values. The HWTT graphs were plotted using the rut depth (mm) values, while the TSR and I-FIT graphs were plotted using the tensile strength (psi) and FI values, respectively. In agreement with the plots from Figure 22, the indirect tensile strength values had the highest correlation, while FI had the lowest correlation. This would be expected, as the impact of air voids on FI is complex due to HMA inhomogeneity (Al-Qadi et al., 2022).

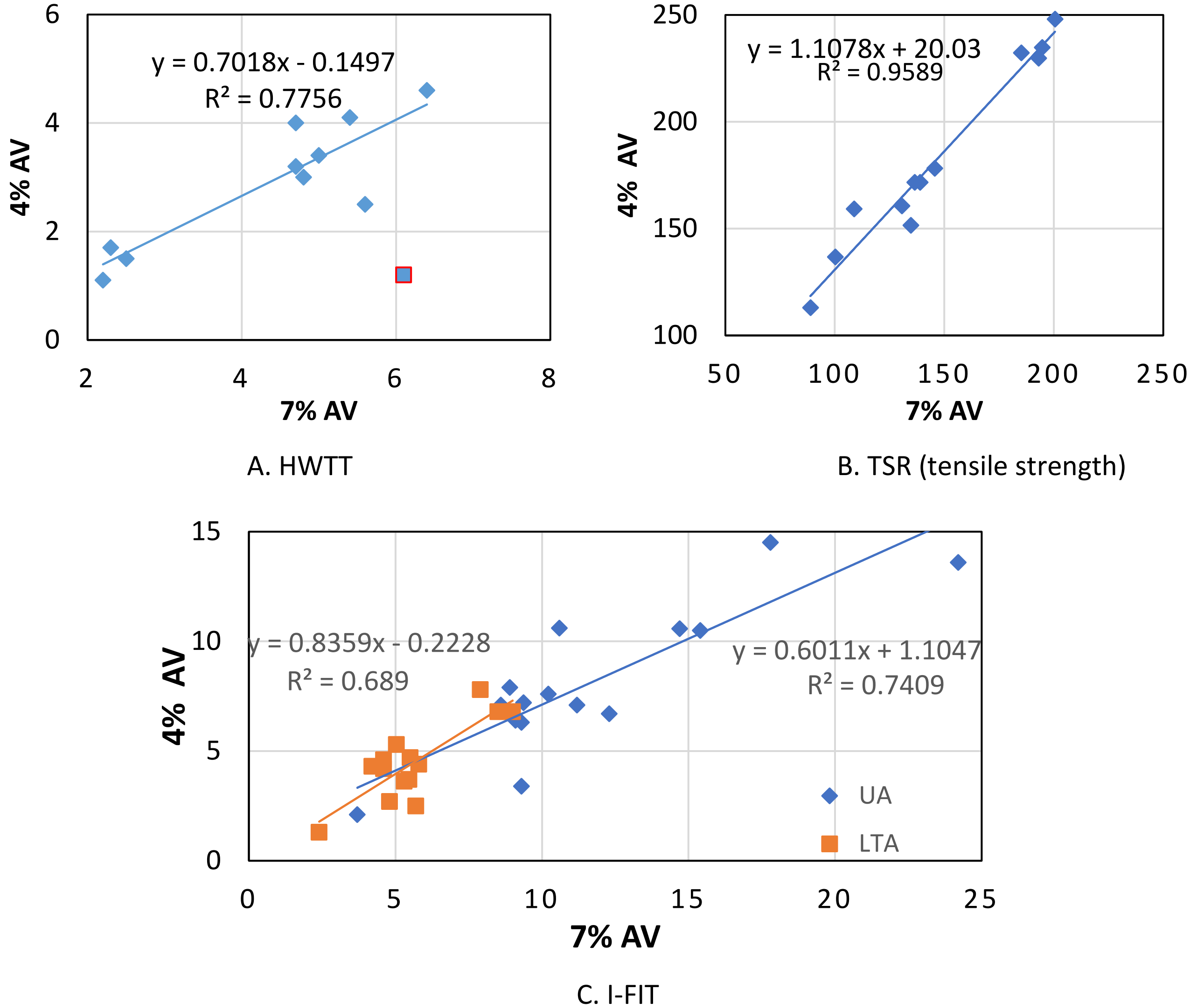


Figure 23. Graphs. Performance test correlation plots of 7% and 4% air void content.

From Figure 23, using the R^2 values and rules of plane geometry, the following conclusions can be made from the equations of the lines of best fit. Figure 23-A had a slope less than one (0.7018), indicating the y-axis value increases slower than the x-axis value. Hence, for a typical mix, the rut depth is lower at 4% than at 7% air void content. The opposite is true for Figure 23-B, which had a slope higher than one (1.1078), indicating the y-axis value increases faster than the x-axis value. Consequently, for a typical mix, the tensile strength is higher at 4% than at 7% air void content. There are two cases in Figure 23-C. Short-term aged specimens had a slope less than one (0.6011), indicating that FI values are lower at 4% than at 7% air void content. Long-term aged specimens had a higher slope that is closer to one (0.8359), indicating that embrittlement due to aging results in faster crack propagation (post-peak slope) and, consequently, lower FI, which does not distinguish between mixes and the binder plays a greater role than air void content, as expected. Note that the mixes tested had 9.5 mm (0.374 in) NMAS (except binder mixes and PA3), the same PG 64-22 (except H2), similar binder content, and asphalt binder replacement (ABR) less than 15% from RAP. The relationship between FI and air void is complex and may not be similar for a different set of mixes as those tested in this study and was presented by Rivera-Perez et al. (2018).

Based on the test results, specimens at 4% air void content had lower rut depths, higher indirect tensile strength, and similar TSR to specimens at 7% air void content. However, the FI values were lower at 4% than at 7%. Using the correlation equation in Figure 23-C, the current highway FI threshold of 8.0 for both unaged laboratory and plant specimens at 7% air void content would correspond to 6.0% for unaged specimens at 4% air void content. Similarly, the aged specimen thresholds of 5.0 and 4.0 for laboratory and plant mixes, respectively, at 7% air void content would correspond to 4.0 and 3.0 for laboratory and plant mixes, respectively, at 4% air void content.

Highways mixes are typically compacted to lesser densities (around 93%) than nonprimary airport mixes (around 96%). Typical highway mix density might not meet the functional requirements of airports (less porous surface), while compacting to a higher density might lead to aggregate breakage or higher construction costs. The choice of compaction density is the purview of IDOT, but this study has demonstrated that when comparing two mixes, no major alteration in relative performance is expected when specimens' air void contents (related to field densities) are changed. Appendix E presents the impact of air void content on each mix per test.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

SUMMARY AND FINDINGS

The state of Illinois is a hub for air travel and is home to 107 public and private airports, approximately 90% of which are nonprimary. At nonprimary airports that serve aircraft with gross weight less than 27,216 kg (60,000 lb), HMA in accordance with FAA-approved state highway specifications may be used. Currently, the majority of highway pavement HMA in Illinois is designed based on the SuperPave mix design method. Personnel pertaining to road highway agencies, pavement engineering industry, product manufacturers, and contractors have become proficient and experienced in material selection, design, and construction of SuperPave HMA. It is, thus, practically and economically beneficial to design HMA for nonprimary airports in accordance with highway HMA specifications when applicable, given that pavement's expected performance and safety operations are maintained.

The main objective of this project was to develop a framework that extends the use of existing IDOT highway pavement surface and binder HMA to nonprimary airfield pavement applications. Nine highway surface mixes (five laboratory and four plant) and six airport surface mixes (two laboratory and four plant) were used in this project. In addition, three airport binder mixes (one laboratory and two plant) were also evaluated. The mixes contained aggregates from various regions of Illinois and included limestone, crushed gravel, traprock, and dolomite. All mixes used PG 64-22 except one highway laboratory mix, which contained PG 70-22. The use of the same PG binder and relatively similar binder content enabled better evaluation of the aggregate blends. Other factors considered included the number of gyrations with highway mixes designed at 50 or 70 gyrations, while airport mixes were designed at either 30 or 40 gyrations for IDOT and 50 or 75 gyrations for FAA. In addition, two SuperPave5 highway laboratory mixes, which are designed at 5% air void content and compacted to 95% density in the field, were included. Furthermore, two highway fine-graded plant mixes were also considered, as these mixes would provide a smooth surface for the aircraft. Only 9.5 mm NMAS surface mixes were tested except for airport plant mix PA3 designed using FAA specifications that had 12.5 mm NMAS. In Illinois, highway mixes with 12.5 mm NMAS are primarily stone-matrix asphalt (SMA) and are more expensive than conventional mixes. Table 15 presents the test results summary for all mixes at 7% air void content.

Table 15. Test Results Summary for All Mixes

	Laboratory Mix	**Laboratory Mix**	**Plant Mix**	**Plant Mix**
Mix Type	*Highway*	*Airport*	*Highway*	*Airport*
Rut Depths (mm)	4.8–6.1	5.6 – > 15	2.2–2.5	4.7–17.6
Tensile Strength (psi)	127–139	89–101	163–190	90–175
TSR	0.91–1.00	0.88–0.99	0.9–0.92*	0.86–0.97
Fracture Energy (J/m^3)	2194–2739	1808–2102	1887–2498	1827–2286
Unaged FI	8.6–14.2	17.8–24.2	9.1–15.4	8.9–14.7
Aged FI	4.2–8.4	7.9–8.5	4.6–8.5	4.6–9.0

*Two highway plant mixes had TSR values of 0.8 and 0.82.

Major findings are the following:

- Highway mixes exhibited less rutting potential than airport mixes when tested using the Hamburg wheel. This could be attributed to the presence of RAP and manufactured sand in highway mixes, which were absent in IDOT airport mixes except for one mix, which included manufactured sand. The low air void requirement (2.0%–3.0%) of IDOT airport mixes promotes the use of natural sand over manufactured sand. FAA airport mixes had manufactured sand, but with varied performance. One of the FAA airport mixes had very low rutting potential while the other had one of the fastest rutting progressions of the mixes evaluated.

- Fracture energy values were similar for both laboratory-designed and plant-produced highway and airport mixes. Airport mixes have higher FI values for laboratory-designed mixes and similar values for plant-produced mixes. Generally, airport mixes have greater flexibility and thus lower cracking potential than highway mixes. This is due to not incorporating RAP and consequently higher virgin asphalt binder content in airport mixes.

- The tensile strength of highway mixes was higher than airport mixes for both laboratory-designed and plant-produced mixes. This could be attributed to the presence of RAP and the use of manufactured sand in highway mixes. Both sets of mixes had similar TSR values.

- SuperPave5 highway mixes, which are designed at 5% air void content and compacted to 95% field density, could be used to satisfy the functional requirements of airport mixes, which have to be easily compactable and with less porous surfaces, as noted in airport mix specifications. SuperPave5 mixes exhibited relatively lower potential to both rutting and cracking potential. In addition, high in-field density would limit the ingress of environmental agents such as air, water, and snow.

- To prevent environmental related distresses such as block cracking, airport mixes designed to FAA standards have lower air void content, lesser number of gyrations, and higher density when compared to highway pavements. This is to provide a "sealed up" surface that prevents the entrance of climate stressors. Fine-graded mixes can help provide this smoother "sealed up" surface, preventing the entrance of environmental agents. In this study, considering only plant mixes, the two fine-graded highway mixes, PH2 and PH4, had high tensile strengths and the lowest rutting potential. In addition, PH4 had the highest FI, while PH2 had comparable FI values to the other highway plant mixes.

This study concluded that the adoption of highway mixes in nonprimary airports is viable and would results in the following advantages:

- Environmental benefits through the use of recycled materials (RAP).

- Technical advantages in the form of the proficiency and expertise of the highway construction industry in highway mixes.

- Economic gains by using available and readily produced materials and increasing the number of eligible contractors, which fosters competition.

RECOMMENDATIONS

- While the highway mixes exhibited good performance in the laboratory, in-place compaction to over 95% density was not evaluated. It is recommended that IDOT review compaction data from previous projects to ascertain the feasibility of achieving high densities for nonprimary airport applications.

- IDOT Aeronautics division may exploit mix testing currently conducted by the highway division, but slightly modify their thresholds. For example, the HWTT and TSR thresholds would be maintained, but FI values may increase to 11.0 and 6.0 for unaged and aged specimens, respectively, at 7% air void content, which would ensure the mixes have good FI and, consequently, low cracking potential when compacted to 96% in the field.

- SMA was not included in this study because of high initial costs. However, the ongoing IDOT-ICT project R27-216 is evaluating the use of local aggregates in SMA, which may result in making these mixes economically competitive, allowing SMA to possibly be used in nonprimary airport applications. The higher binder content in SMA would help withstand environmental loading, which is the primary driver of pavement deterioration in nonprimary airports with aircraft gross weight less than 27,216 kg (60,000 lb).

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APPENDIX A: EXTRACTION DATA FOR PA3

Table 16. Extraction of the Two Additional Gmm Samples for Airport Plant Mix PA3

EXTRACTION OF 2 GMM SAMPLES FOR PLANT MIX PA3 Date 7/25/2023

Weight of EMPTY container before (g)	5163.4		Weight of EMPTY Fines cup BEFORE(g)	1306
Weight of SAMPLE + container before (g)	8061.7		Weight of Fines cup AFTER (g)	1442.8
Weight of SAMPLE + container AFTER (g)	7751.7		Weight of Fines recovered (g)	136.8
Total weight of SAMPLE BEFORE (g)	2898.3			
Total weight of SAMPLE AFTER (g)	2725.1		Weight of EMPTY container AFTER (g)	5164.5
Total weight of Asphalt binder (g)	173.2		Net Loss/Gain (g)	1.1
Percent asphalt binder (%)	6.0		Percent Net Loss/Gain (%)	0.021
Target percent asphalt binder (%)	**6.0**			

Sieve Size		Weight retained	Cummulative weight retained	% Retained	% Passing	Target (JMF) %	Difference	Specification
3/4"	19.0 mm	0	0	0.0	100	**100**	0.0	100
1/2"	12.5 mm	112.5	112.5	4.1	95.9	**97**	-1.1	90 - 100
3/8"	9.5 mm	327.9	440.4	16.2	83.8	**85**	-1.2	72 - 88
#4	4.75 mm	569.5	1009.9	37.1	62.9	**66**	-3.1	53 - 73
#8	2.36 mm	345.7	1355.6	49.7	50.3	**51**	-0.7	38 -60
#16	1.18 mm	407.1	1762.7	64.7	35.3	**35**	0.3	26 - 48
#30	0.6 mm	405.8	2168.5	79.6	20.4	**20**	0.4	18 -38
#50	0.3 mm	248.8	2417.3	88.7	11.3	**11**	0.3	11 - 27
#100	0.15 mm	113.7	2531	92.9	7.1	**8**	-0.9	8 - 18
#200	0.075 mm	51.8	2582.8	94.8	5.2	**5.2**	0.0	3 - 6
Pan	Pan	2.8	2585.6					

Total sample BEFORE sieving	2587.2	Amount of Fines	139.6
Total sample AFTER sieving	2585.6	Percent minus #200	5.1
Percent loss/gain after sieving	-0.06		

APPENDIX B: HAMBURG WHEEL-TRACKING TEST

LABORATORY-DESIGNED MIXES

Table 17. Laboratory Mixes' HWTT Specimen Volumetrics

	Dry Weight (kg)	Submerged Weight (kg)	SSD Weight (kg)	G_{mb}	G_{mm}	Air Void (%)
H1	2522.3	1460	2526.1	2.366	2.462	3.9
H1	2523.1	1459.1	2526.6	2.364	2.462	4.0
H1	2525.4	1461.1	2530.6	2.361	2.462	4.1
H1	2524.4	1460.8	2528.9	2.363	2.462	4.0
H1	2457.6	1387.5	2465.8	2.279	2.462	7.4
H1	2471.4	1402.2	2477.8	2.298	2.462	6.7
H1	2463.2	1394.3	2471.1	2.288	2.462	7.1
H1	2464.3	1395.2	2472.1	2.288	2.462	7.1
H2	2507.1	1443.5	2508.9	2.353	2.444	3.7
H2	2501.1	1440.6	2504.2	2.352	2.444	3.8
H2	2503.5	1441.8	2506.3	2.352	2.444	3.8
H2	2499.1	1440	2501.5	2.354	2.444	3.7
H2	2409.7	1360.3	2416.6	2.281	2.444	6.7
H2	2408.5	1364.6	2418.4	2.286	2.444	6.5
H2	2406.5	1361.4	2418.5	2.277	2.444	6.8
H2	2409	1365.7	2421.2	2.282	2.444	6.6
H3	2547.5	1482.7	2549.2	2.389	2.475	3.5
H3	2542.1	1478.9	2544.5	2.386	2.475	3.6
H3	2543.5	1474.7	2545.4	2.376	2.475	4.0
H3	2543	1479.7	2544.6	2.388	2.475	3.5
H3	2452.6	1401.6	2461.4	2.314	2.475	6.5
H3	2452.5	1398.4	2460	2.310	2.475	6.7
H3	2454.2	1403.2	2463.4	2.315	2.475	6.5
H3	2452.2	1393.6	2460.1	2.299	2.475	7.1
H4	2524.2	1459.4	2526	2.367	2.455	3.6
H4	2509.6	1450.4	2512.2	2.364	2.455	3.7
H4	2509.7	1446.7	2514.2	2.351	2.455	4.2
H4	2511.2	1446.3	2513.6	2.353	2.455	4.2
H4	2422	1370.2	2428.7	2.288	2.455	6.8
H4	2421.6	1369.1	2429	2.285	2.455	6.9
H4	2423.6	1374.3	2431.7	2.292	2.455	6.6
H4	2422.3	1372	2431.6	2.286	2.455	6.9
H5	2547.9	1479.7	2550.6	2.379	2.474	3.8
H5	2540.8	1473.1	2544.6	2.371	2.474	4.2

	Dry Weight (kg)	Submerged Weight (kg)	SSD Weight (kg)	G_{mb}	G_{mm}	Air Void (%)
H5	2541	1474.7	2544.4	2.375	2.474	4.0
H5	2543.7	1474.9	2547.3	2.372	2.474	4.1
H5	2446.7	1396	2456.8	2.306	2.474	6.8
H5	2444	1390.2	2456.2	2.293	2.474	7.3
H5	2444.9	1391.6	2457	2.295	2.474	7.2
H5	2444.4	1390.6	2454.4	2.298	2.474	7.1
A1	2552.8	1477	2555.3	2.367	2.477	4.4
A1	2564	1480	2564.3	2.365	2.477	4.5
A1	2558.8	1482.5	2561.4	2.372	2.477	4.2
A1	2558	1480.4	2559.1	2.371	2.477	4.3
A1	2455.5	1403.6	2473.7	2.295	2.477	7.3
A1	2466.6	1410.9	2483.4	2.300	2.477	7.1
A1	2464.9	1402.3	2472.3	2.304	2.477	7.0
A1	2468.1	1403.7	2475.5	2.303	2.477	7.0
A2	2514.4	1448.2	2516.6	2.353	2.438	3.5
A2	2505.1	1438	2506.8	2.344	2.438	3.9
A2	2502	1436.9	2504.2	2.344	2.438	3.8
A2	2502.6	1436.4	2504	2.344	2.438	3.9
A2	2414.1	1362.2	2422.9	2.276	2.438	6.6
A2	2414.4	1361.8	2422	2.277	2.438	6.6
A2	2419.4	1364.5	2426.8	2.278	2.438	6.6
A2	2420.3	1364.9	2427	2.279	2.438	6.5
AB1	2569.4	1519	2575	2.433	2.524	3.6
AB1	2569.6	1519.2	2574.9	2.434	2.524	3.6
AB1	2570.6	1514.5	2575.6	2.423	2.524	4.0
AB1	2561.3	1511.6	2566.4	2.428	2.524	3.8
AB1	2467.7	1440.7	2482.8	2.368	2.524	6.2
AB1	2467	1439	2484	2.361	2.524	6.5
AB1	2451.1	1430.2	2475.2	2.346	2.524	7.1
AB1	2452.8	1428.4	2470.4	2.354	2.524	6.7

PLANT-PRODUCED MIXES

Table 18. Plant Mixes' HWTT Specimen Volumetrics

	Dry Weight (kg)	Submerged Weight (kg)	SSD Weight (kg)	G_{mb}	G_{mm}	Air Void (%)
PH1	2532.5	1465	2534.9	2.367	2.479	4.5
PH1	2547	1479.6	2549.8	2.380	2.479	4.0
PH1	2546.6	1478	2548.7	2.378	2.479	4.1
PH1	2545.4	1479.5	2548.1	2.382	2.479	3.9
PH1	2448.3	1400.4	2464.5	2.301	2.479	7.2
PH1	2451.8	1403.4	2467.3	2.305	2.479	7.0
PH1	2449.2	1409.6	2470.4	2.309	2.479	6.9
PH1	2451.6	1410.2	2471	2.311	2.479	6.8
PH2	2557.9	1485	2560.6	2.378	2.475	3.9
PH2	2560.8	1485.2	2562.9	2.376	2.475	4.0
PH2	2556.4	1483.1	2561.7	2.370	2.475	4.2
PH2	2558.3	1483.2	2561.5	2.373	2.475	4.1
PH2	2463.4	1398.3	2469.2	2.300	2.475	7.1
PH2	2465.5	1406.9	2477.6	2.303	2.475	6.9
PH2	2468.6	1407.8	2476	2.311	2.475	6.6
PH2	2469.5	1408	2479.4	2.305	2.475	6.9
PH3	2504.5	1443.6	2508.6	2.352	2.452	4.1
PH3	2508	1443.8	2509.6	2.353	2.452	4.0
PH3	2509.6	1443.4	2510.2	2.352	2.452	4.1
PH3	2507.1	1442.9	2508.7	2.352	2.452	4.1
PH3	2423.3	1375.1	2436	2.284	2.452	6.9
PH3	2421.6	1371.7	2431.5	2.285	2.452	6.8
PH3	2423.3	1370.7	2432.9	2.281	2.452	7.0
PH3	2421.7	1372.6	2432.9	2.284	2.452	6.9
PH4	2533.4	1461.4	2534.8	2.360	2.445	3.5
PH4	2519	1448.2	2521.8	2.346	2.445	4.0
PH4	2518.5	1446.9	2520.1	2.347	2.445	4.0
PH4	2520.6	1450	2522.1	2.351	2.445	3.8
PH4	2435.4	1374.2	2445	2.274	2.445	7.0
PH4	2436.5	1374.9	2446.2	2.274	2.445	7.0
PH4	2435.2	1371.8	2442.4	2.275	2.445	7.0
PH4	2437.1	1372.7	2443.4	2.276	2.445	6.9
PA1	2538.5	1479.6	2539.8	2.394	2.502	4.3
PA1	2538.2	1478.6	2540	2.391	2.502	4.4
PA1	2539	1478.5	2540.3	2.391	2.502	4.4
PA1	2542.7	1483.8	2544.2	2.398	2.502	4.2
PA1	2433.8	1393.1	2444.8	2.314	2.502	7.5

	Dry Weight (kg)	Submerged Weight (kg)	SSD Weight (kg)	G_{mb}	G_{mm}	Air Void (%)
PA1	2438.7	1397.8	2450.7	2.316	2.502	7.4
PA1	2439.1	1402.3	2454.7	2.318	2.502	7.4
PA1	2440.3	1402	2456.7	2.314	2.502	7.5
PA2	2475	1415.2	2476.8	2.331	2.428	4.0
PA2	2475.4	1415.3	2477.6	2.330	2.428	4.0
PA2	2477	1416.5	2479.4	2.330	2.428	4.0
PA2	2478.4	1420.5	2480.2	2.339	2.428	3.7
PA2	2396.1	1341.5	2402.2	2.259	2.428	7.0
PA2	2394.7	1345.1	2402.6	2.264	2.428	6.8
PA2	2395.6	1343.9	2400	2.268	2.428	6.6
PA2	2396.2	1344.7	2401.6	2.267	2.428	6.6
PA3	2437	1393.3	2438.8	2.331	2.443	4.6
PA3	2436.2	1393.7	2439	2.331	2.443	4.6
PA3	2438.2	1394.1	2439.7	2.332	2.443	4.5
PA3	2437.3	1393.8	2439	2.332	2.443	4.5
PA3	2425.2	1361.8	2432.5	2.265	2.443	7.3
PA3	2426.2	1363.8	2434.3	2.266	2.443	7.2
PA3	2427.7	1364.2	2433.1	2.271	2.443	7.0
PA3	2427.5	1363.9	2432.9	2.271	2.443	7.0
PAB1	2493.9	1468.4	2499.6	2.418	2.514	3.8
PAB1	2493.3	1467.9	2499	2.418	2.514	3.8
PAB1	2493.9	1466.2	2498	2.417	2.514	3.9
PAB1	2496.7	1468.8	2501.6	2.417	2.514	3.9
PAB1	2400.9	1406.1	2427.8	2.350	2.514	6.5
PAB1	2402	1402.5	2424.8	2.350	2.514	6.5
PAB1	2404.1	1404.3	2428.4	2.348	2.514	6.6
PAB1	2403.1	1402.8	2424.9	2.351	2.514	6.5

APPENDIX C: ILLINOIS FLEXIBILITY INDEX TEST

LABORATORY-DESIGNED MIXES

Table 19. Laboratory Mixes' I-FIT Specimen Properties

	Design AV	Condition	FI	Slope	Fracture Energy (J/m^2)	Strength (psi)	Ligament (mm)	Thickness (mm)
H1	4.0	UA	4.8	-4.59	2202.46	88.42	60.06	50.67
H1	4.0	UA	5.18	-4.68	2421.99	94.55	59.22	49.52
H1	4.0	UA	7.91	-3.11	2458.99	85.98	60.53	49.37
H1	4.0	LTA	4.02	-5.06	2035.61	98.88	60.21	49.12
H1	4.0	LTA	4.24	-4.83	2050.11	86.85	60.45	49.88
H1	4.0	LTA	4.94	-4.83	2384.68	92.98	59.95	49.94
H1	7.0	UA	13.11	-1.75	2294.77	57.55	59.08	49.85
H1	7.0	UA	15.41	-1.68	2588.76	60.72	59.97	49.37
H1	7.0	UA	15.62	-1.54	2406.12	55.21	59.78	49.33
H1	7.0	LTA	5.33	-4.66	2482.12	91.39	59.04	50.73
H1	7.0	LTA	4.33	-5.28	2286.74	95.61	60.06	49.7
H1	7.0	LTA	6.24	-4.17	2601.72	92.84	59.68	49.58
H2	4.0	UA	7.98	-3.37	2690.05	88.19	59.85	50.46
H2	4.0	UA	6.76	-4.54	3066.79	96.85	60.16	50.18
H2	4.0	UA	6.4	-5.06	3239.41	103.63	59.5	49.85
H2	4.0	LTA	4.38	-7.04	3080.73	117.58	59.48	49.38
H2	4.0	LTA	2.61	-8.54	2225.55	113.97	59.44	50.09
H2	4.0	LTA	6.02	-4.91	2955.91	114.59	59.82	49.26
H2	7.0	UA	9.57	-2.73	2613.46	73.19	60.22	50.65
H2	7.0	UA	7.27	-3.38	2456.75	74.81	60.72	49.35
H2	7.0	UA	8.81	-2.86	2519.97	76.22	59.14	49.2
H2	7.0	LTA	4.86	-5.02	2439.17	93.5	59.33	50.68
H2	7.0	LTA	4.1	-5.74	2356.11	99.25	60.23	49.94
H2	7.0	LTA	3.67	-5.5	2021.12	95.74	60.34	50.77
H3	4.0	UA	12.83	-2.07	2655.59	72.72	59.46	49.65
H3	4.0	UA	8.78	-3.43	3013.01	93.27	59.07	49.17
H3	4.0	UA	10.34	-2.74	2832.91	80.64	59.38	49.38
H3	4.0	LTA	5.4	-4.38	2364.67	93.84	59.78	49.79
H3	4.0	LTA	5.13	-4.54	2331.09	93.34	60.26	50.05
H3	4.0	LTA	5.07	-4.87	2469.28	101.41	60.31	49.12
H3	7.0	UA	12.48	-2	2495.18	67.2	60.15	50.26
H3	7.0	UA	9.66	-2.55	2464.14	69.88	59.05	50.59
H3	7.0	UA	9.56	-2.59	2477.01	65.1	60.92	50.8

	Design AV	Condition	FI	Slope	Fracture Energy (J/m^2)	Strength (psi)	Ligament (mm)	Thickness (mm)
H3	7.0	LTA	9.17	-2.74	2513.05	69.42	60.78	49.17
H3	7.0	LTA	7.59	-3.03	2299.92	70.27	59.6	49.62
H3	7.0	LTA	8.39	-3.47	2910.93	83.3	60.57	49.72
H4	4.0	UA	8.51	-2.89	2458.27	76.6	59.06	50.76
H4	4.0	UA	7.7	-3.56	2742.65	92.23	60.48	49.22
H4	4.0	UA	6.47	-4	2589.43	88.41	60.09	49.1
H4	4.0	LTA	3.46	-7.28	2520.15	109.88	59.55	50.38
H4	4.0	LTA	5.84	-5.08	2965.6	111.33	60.72	49.68
H4	4.0	LTA	4.84	-4.85	2348	90.51	59.98	50.91
H4	7.0	UA	11.75	-2.2	2585.06	72.28	60.61	50.05
H4	7.0	UA	8.41	-3.28	2757.39	84.84	58.43	49.51
H4	7.0	UA	10.5	-2.74	2875.75	80.83	59.35	50.15
H4	7.0	LTA	4.66	-4.29	1998.96	87.22	60.02	50.14
H4	7.0	LTA	6.59	-3.53	2326.46	85.07	60.06	50.47
H4	7.0	LTA	5.3	-4.08	2163.36	93.36	59.85	49.43
H5	4.0	UA	5.52	-4.41	2434.89	91.09	60.8	50.53
H5	4.0	UA	8.31	-3.08	2558.54	79.36	60.55	50.8
H5	4.0	UA	6.21	-5.03	3121.91	115.36	59.73	50.01
H5	4.0	LTA	4.13	-5.2	2146.57	92.05	60.92	4.13
H5	4.0	LTA	6.09	-4.26	2595.16	101.22	59.05	6.09
H5	4.0	LTA	5.78	-5.44	3142.44	113.68	60.42	5.78
H5	7.0	UA	10.3	-2.37	2441.89	63.64	60.61	50.05
H5	7.0	UA	10.32	-2.3	2372.96	67.82	60.66	49.92
H5	7.0	UA	16.15	-1.79	2891.28	65.19	58.43	49.51
H5	7.0	LTA	5.06	-4.05	2049.39	73.54	60.09	50.68
H5	7.0	LTA	4.99	-4.87	2431.41	89.33	59.98	49.33
H5	7.0	LTA	5.04	-5.38	2714.11	101.25	59.15	49.09
A1	4.0	UA	17.46	-1.57	2740.94	59.45	60.42	50.03
A1	4.0	UA	13.67	-2.04	2789.46	66.29	60.71	49.79
A1	4.0	UA	12.38	-2.27	2810.19	67.99	59.61	50.34
A1	4.0	LTA	7.09	-3.36	2381.66	74.67	59.1	50.97
A1	4.0	LTA	8.73	-2.3	2008.79	65.13	60.53	50.55
A1	4.0	LTA	7.57	-4.23	3200.99	88.55	60.17	51
A1	7.0	UA	20.51	-1.04	2132.65	44.63	60.09	50.55
A1	7.0	UA	16.55	-1.34	2218.22	48.17	60.28	50.54
A1	7.0	UA	16.3	-1.2	1956.34	42.08	59.08	49.68
A1	7.0	LTA	7.73	-2.28	1763.27	56.62	59.98	50.61
A1	7.0	LTA	5.1	-3.31	1686.55	63.53	60.5	50.91
A1	7.0	LTA	10.85	-1.82	1974.88	54.21	59.11	50.87

	Design AV	Condition	FI	Slope	Fracture Energy (J/m^2)	Strength (psi)	Ligament (mm)	Thickness (mm)
A2	4.0	UA	11.94	-2	2387.24	59.29	60.15	49.24
A2	4.0	UA	13.47	-1.96	2640.03	63.78	59.89	50.17
A2	4.0	UA	15.49	-1.76	2726.2	66.54	60.93	49.7
A2	4.0	LTA	6.7	-3.42	2292.18	82.53	60.74	49.2
A2	4.0	LTA	6.13	-3.68	2254.21	81.51	59.85	49.4
A2	4.0	LTA	7.5	-2.97	2228.46	72.5	60.97	50.8
A2	7.0	UA	27.67	-0.97	2683.67	49.03	59.4	49.12
A2	7.0	UA	18.75	-1.52	2850.3	54.6	60.48	49.38
A2	7.0	UA	26.41	-0.98	2587.86	41.81	59.91	50.62
A2	7.0	LTA	7.96	-2.69	2140.75	64.67	59.99	50.48
A2	7.0	LTA	6.86	-3.1	2127.59	68.79	60.91	50.37
A2	7.0	LTA	10.55	-1.84	1940.57	52.81	59.85	50.07
AB1	4.0	UA	2.88	-6.79	1957.27	103.92	59.46	49.53
AB1	4.0	UA	1.69	-11.43	1931.27	118.57	60.83	49.78
AB1	4.0	UA	1.76	-12.05	2121.15	109.29	60.29	49.08
AB1	4.0	LTA	1.51	-12.63	1906.87	123.86	59.18	50.26
AB1	4.0	LTA	1.26	-16.25	2043.94	125.55	60.06	50.65
AB1	4.0	LTA	1.24	-11.56	1435.38	99.79	60.33	50.09
AB1	7.0	UA	2.68	-5.83	1563.98	83.01	60.48	49.92
AB1	7.0	UA	4.05	-4.21	1706.58	76.62	60.39	49.39
AB1	7.0	UA	4.29	-5.43	2327.83	91.75	59.59	49.35
AB1	7.0	LTA	2.46	-6.88	1690.95	94.1	59.66	49.03
AB1	7.0	LTA	2.62	-7.3	1909.81	105.22	60.3	50.35
AB1	7.0	LTA	2.12	-8.61	1822.28	93.54	59.25	49.43

PLANT-PRODUCED MIXES

Table 20. Plant Mixes' I-FIT Specimen Properties

	Test Air Void	Condition	FI	Slope	Fracture Energy (J/m²)	Strength (psi)	Ligament (mm)	Thickness (mm)
PH1	4.0	UA	8.4	-3.21	2697.44	84.73	60.49	49.92
PH1	4.0	UA	6.64	-3.68	2442.16	90.28	59.96	49.06
PH1	4.0	UA	6.31	-4.02	2537.56	84.42	60.22	49.95
PH1	4.0	LTA	4.06	-5.65	2294.06	97.59	59.04	49.84
PH1	4.0	LTA	4.75	-4.67	2217.33	88.3	60.12	50.26
PH1	4.0	LTA	4.35	-4.97	2162.93	87.93	59.73	50.36
PH1	7.0	UA	14.29	-1.71	2443.91	65.16	59.01	50.74
PH1	7.0	UA	9.83	-2.4	2359.44	67.29	60.73	50.11
PH1	7.0	UA	9.34	-2.38	2222.55	67.2	59.97	49.75
PH1	7.0	LTA	5.58	-4.05	2261.51	82.94	59.04	49.2
PH1	7.0	LTA	5	-4.19	2096.09	84.23	60.45	49.81
PH1	7.0	LTA	6.78	-3.48	2359.89	79.62	59.29	50
PH2	4.0	UA	7.82	-2.99	2337.68	74.9	60.46	50.9
PH2	4.0	UA	7.72	-3.18	2454.42	82.51	59.87	50.27
PH2	4.0	UA	6.07	-3.89	2359.73	81.67	60.4	50.65
PH2	4.0	LTA	3.5	-5.15	1803.49	82.77	60.47	51
PH2	4.0	LTA	3.94	-6.09	2400.43	101.97	59.85	49.38
PH2	4.0	LTA	3.7	-6.53	2418.63	87.97	60.09	50.36
PH2	7.0	UA	9.14	-2.28	2084.6	60.72	59.8	50.42
PH2	7.0	UA	10.73	-1.88	2017.34	62.03	59.3	50.16
PH2	7.0	UA	8.24	-2.75	2264.72	68.82	60.12	50.32
PH2	7.0	LTA	4.49	-3.9	1750.99	75.13	59.74	50.83
PH2	7.0	LTA	5.65	-3.81	2150.98	77.59	59.88	49.44
PH2	7.0	LTA	6.26	-2.81	1759.59	69.65	59.36	49.43
PH3	4.0	UA	6.18	-3.73	2303.93	73.58	60.39	49.27
PH3	4.0	UA	6.55	-3.44	2253.62	80.04	60.94	49.65
PH3	4.0	UA	6.48	-3.98	2580.99	87.06	60.54	49.39
PH3	4.0	LTA	4.17	-4.91	2047.16	84.08	59.82	49.64
PH3	4.0	LTA	3.81	-4.18	1592.71	72	59.62	49.47
PH3	4.0	LTA	4.65	-4.71	2191.59	97.81	59.26	50.9
PH3	7.0	UA	9.02	-2.24	2019.94	66.47	59.67	49.7
PH3	7.0	UA	9.6	-2.35	2256.1	64.55	59.81	50.8
PH3	7.0	UA	8.52	-2.42	2062.77	63.68	60.41	50.87
PH3	7.0	LTA	4.59	-4.21	1931.42	73.51	59.06	50.89
PH3	7.0	LTA	5.18	-4.17	2159.02	80.93	59.98	50.99
PH3	7.0	LTA	4.05	-4.41	1786.28	77.3	59.08	49.21

	Test Air Void	Condition	FI	Slope	Fracture Energy (J/m²)	Strength (psi)	Ligament (mm)	Thickness (mm)
PH4	4.0	UA	13.49	-1.78	2401.62	65.54	59.81	50.37
PH4	4.0	UA	10.34	-2.39	2471.82	67.49	60.23	49.41
PH4	4.0	UA	7.64	-3.1	2367.33	77.12	59.08	50.78
PH4	4.0	LTA	3.51	-6.15	2157.31	111.37	59.61	50.24
PH4	4.0	LTA	2.76	-6.86	1896.37	98.7	60.65	49.44
PH4	4.0	LTA	3.35	-7.41	2483.15	113.44	60.61	49.91
PH4	7.0	UA	13.19	-1.68	2216.09	54.48	60.47	49.98
PH4	7.0	UA	16.17	-1.43	2312.71	57.26	60.36	49.37
PH4	7.0	UA	16.85	-1.28	2156.69	51.98	59.67	50.02
PH4	7.0	LTA	10.96	-2.85	3125	68.73	60.64	50.72
PH4	7.0	LTA	6.87	-3.18	2185.28	72.69	59.92	50.19
PH4	7.0	LTA	7.75	-2.82	2184.88	76.53	60.28	50.26
PA1	4.0	UA	6.85	-3.59	2457.55	85.75	60.44	49.85
PA1	4.0	UA	5.68	-4.69	2665.06	102.48	59.05	50.86
PA1	4.0	UA	6.27	-4.63	2904.46	98.8	59.58	49.71
PA1	4.0	LTA	1.77	-12.84	2272.07	117.56	59.38	49.18
PA1	4.0	LTA	2.23	-11.94	2663.55	133.45	59.56	49.59
PA1	4.0	LTA	3.47	-8.39	2911.11	131.78	59.4	49.55
PA1	7.0	UA	6.38	-3.67	2342.76	81.27	59.27	49.44
PA1	7.0	UA	12.17	-2.13	2592.35	66.37	60.84	49.37
PA1	7.0	UA	9.48	-2.8	2655.73	75.79	60.45	50.55
PA1	7.0	LTA	6.8	-3.6	2449.32	91.92	60.13	50.77
PA1	7.0	LTA	5.42	-5.16	2795.34	103.12	59.74	49.98
PA1	7.0	LTA	4.97	-5.07	2521.27	100.82	60.79	49.89
PA2	4.0	UA	7.21	-3.25	2343.68	69.63	60.73	50.14
PA2	4.0	UA	7.01	-3.21	2251.68	69.35	59.95	50.1
PA2	4.0	UA	9.42	-2.34	2205.32	69.5	60.86	50.66
PA2	4.0	LTA	4.56	-5.44	2478.52	98.2	59.92	50.14
PA2	4.0	LTA	4.64	-5.1	2365.15	93.35	60.91	49.4
PA2	4.0	LTA	4.5	-5	2251.27	99.8	61	50.31
PA2	7.0	UA	8.49	-1.95	1655.72	48.52	60.29	50.98
PA2	7.0	UA	9.74	-2.28	2220.41	58.43	59.56	50.3
PA2	7.0	UA	8.54	-1.88	1604.58	47.21	60.23	50.23
PA2	7.0	LTA	5.56	-4.34	2413.37	88.1	59.39	50.91
PA2	7.0	LTA	4.8	-4.84	2325.19	95.36	59.42	49.05
PA2	7.0	LTA	3.52	-6.03	2120.63	98.42	60.11	49.45
PA3	4.0	UA	8.41	-2.61	2195.45	71.11	60.45	50.05
PA3	4.0	UA	12.4	-1.67	2070.93	55.02	60	49.61
PA3	4.0	UA	10.92	-2.01	2195.08	56.81	59.85	50.57

	Test Air Void	Condition	FI	Slope	Fracture Energy (J/m²)	Strength (psi)	Ligament (mm)	Thickness (mm)
PA3	4.0	LTA	4.05	-4.88	1977.49	90.12	59.59	49.02
PA3	4.0	LTA	5.95	-4.05	2407.77	86.21	59.14	50.68
PA3	4.0	LTA	7.42	-3.36	2492.33	85.41	60	49.19
PA3	7.0	UA	17.53	-1.28	2243.65	48.25	60.98	49.45
PA3	7.0	UA	15.7	-1.25	1962.58	51.9	60.22	49.78
PA3	7.0	UA	10.99	-2	2198.49	56.74	59.56	50.51
PA3	7.0	LTA	9.84	-2.23	2194.02	61.69	59.44	50.86
PA3	7.0	LTA	7.33	-2.42	1773.71	60.01	60.62	50.41
PA3	7.0	LTA	9.83	-1.97	1935.67	53.1	59.57	49.32
PA4	7.0	UA	10.39	-2.12	2203.42	55.97	59.98	50.09
PA4	7.0	UA	12.25	-1.84	2254.68	54.98	60.84	50.04
PA4	7.0	UA	10.6	-2.38	2523.39	66.27	60.45	49.73
PA4	7.0	LTA	6.87	-3.59	2464.96	75.75	60.78	50.8
PA4	7.0	LTA	5.67	-3.59	2036.59	72.75	60.97	50.31
PA4	7.0	LTA	5.53	-3.46	1912.69	63.7	60.52	49.81
PAB1	4.0	UA	3.39	-7.04	2389.39	111.93	60.57	49.95
PAB1	4.0	UA	3.33	-6.74	2243.12	107.48	60.33	50.16
PAB1	4.0	UA	3.42	-6.11	2087.53	94.01	60.69	50.62
PAB1	4.0	LTA	3.12	-7.38	2303.27	101.48	60.94	50.6
PAB1	4.0	LTA	2.7	-8.91	2404.54	111.15	59.77	49.58
PAB1	4.0	LTA	2.26	-10.68	2408.39	115.8	60.83	49.32
PAB1	7.0	UA	8.5	-2.06	1751.73	52.36	59.34	49.33
PAB1	7.0	UA	9.75	-2.04	1989.06	57.67	59.48	49.32
PAB1	7.0	UA	9.71	-2.4	2331.35	65.09	60.08	49.27
PAB1	7.0	LTA	4.33	-5.47	2367.14	96.49	60.02	49.58
PAB1	7.0	LTA	5.28	-4.82	2546.14	95.23	60.2	49.79
PAB1	7.0	LTA	4.79	-5.27	2526.09	98.08	60.01	50.28
PAB2	7.0	UA	5.99	-4.27	2556.25	90.52	60.97	49.09
PAB2	7.0	UA	5.93	-3.94	2337.22	93.33	60.24	50.1
PAB2	7.0	UA	4.61	-5.54	2551.65	100.56	59.82	49.54
PAB2	7.0	LTA	3.81	-5.74	2185.55	96.36	59.74	50.65
PAB2	7.0	LTA	2.86	-6.5	1857.98	96.54	60.54	50.95
PAB2	7.0	LTA	2.45	-7.72	1891.14	106.46	59.1	50

APPENDIX D: TENSILE STRENGTH RATIO

LABORATORY-DESIGNED MIXES

Table 21. Laboratory Mixes' TSR Specimen Properties

	Condition	Load (lb)	Cross Sectional Area (in^2)	Tensile Strength (psi)	Test Air Void
H1	Conditioned	5292.3	34.70	152.5	4.0
H1	Conditioned	5506.2	34.70	158.7	4.0
H1	Conditioned	4785.8	34.70	137.9	4.0
H1	Unconditioned	5358.8	34.68	154.5	4.0
H1	Unconditioned	5130.8	34.76	147.6	4.0
H1	Unconditioned	5507.9	34.73	158.6	4.0
H1	Conditioned	4150.7	34.70	119.6	7.0
H1	Conditioned	4579.1	34.69	132.0	7.0
H1	Conditioned	4774.6	34.65	137.8	7.0
H1	Unconditioned	5078.3	34.69	146.4	7.0
H1	Unconditioned	4731.7	34.66	136.5	7.0
H1	Unconditioned	4707.0	34.66	135.8	7.0
H2	Conditioned	6404.2	34.69	184.6	4.0
H2	Conditioned	6121.6	34.70	176.4	4.0
H2	Conditioned	6065.9	34.70	174.8	4.0
H2	Unconditioned	5748.5	34.69	165.7	4.0
H2	Unconditioned	5902.7	34.70	170.1	4.0
H2	Unconditioned	6858.8	34.69	197.7	4.0
H2	Conditioned	4561.7	34.66	131.6	7.0
H2	Conditioned	4683.9	34.64	135.2	7.0
H2	Conditioned	5196.6	34.69	149.8	7.0
H2	Unconditioned	5772.1	34.73	166.2	7.0
H2	Unconditioned	5271.8	34.68	152.0	7.0
H2	Unconditioned	4803.6	34.68	138.5	7.0
H3	Conditioned	5972.5	34.62	172.5	4.0
H3	Conditioned	5954.4	34.70	171.6	4.0
H3	Conditioned	6305.1	34.70	181.7	4.0
H3	Unconditioned	5791.5	34.70	166.9	4.0
H3	Unconditioned	5573.4	34.73	160.5	4.0
H3	Unconditioned	6120.2	34.70	176.4	4.0
H3	Conditioned	4294.2	34.69	123.8	7.0
H3	Conditioned	4534.2	34.69	130.7	7.0
H3	Conditioned	4832.3	34.74	139.1	7.0

	Condition	Load (lb)	Cross Sectional Area (in^2)	Tensile Strength (psi)	Test Air Void
H3	Unconditioned	4800.8	34.74	138.2	7.0
H3	Unconditioned	5000.0	34.70	144.1	7.0
H3	Unconditioned	4953.3	34.66	142.9	7.0
H4	Conditioned	5655.9	34.66	163.2	4.0
H4	Conditioned	5163.0	34.63	149.1	4.0
H4	Conditioned	6115.9	34.77	175.9	4.0
H4	Unconditioned	6214.4	34.70	179.1	4.0
H4	Unconditioned	5873.5	34.73	169.1	4.0
H4	Unconditioned	6710.9	34.70	193.4	4.0
H4	Conditioned	4542.1	34.70	130.9	7.0
H4	Conditioned	4472.9	34.70	128.9	7.0
H4	Conditioned	4611.3	34.72	132.8	7.0
H4	Unconditioned	4730.9	34.66	136.5	7.0
H4	Unconditioned	5154.3	34.69	148.6	7.0
H4	Unconditioned	5412.6	34.70	156.0	7.0
H5	Conditioned	5908.3	34.69	170.3	4.0
H5	Conditioned	4747.8	34.73	136.7	4.0
H5	Conditioned	5548.9	34.70	159.9	4.0
H5	Unconditioned	5823.2	34.66	168.0	4.0
H5	Unconditioned	5672.1	34.65	163.7	4.0
H5	Unconditioned	5729.9	34.68	165.2	4.0
H5	Conditioned	4317.9	34.68	124.5	7.0
H5	Conditioned	4467.4	34.66	128.9	7.0
H5	Conditioned	4439.7	34.69	128.0	7.0
H5	Unconditioned	5028.7	34.66	145.1	7.0
H5	Unconditioned	4471.1	34.69	128.9	7.0
H5	Unconditioned	4464.7	34.72	128.6	7.0
A1	Conditioned	3949.8	34.65	114.0	4.0
A1	Conditioned	3832.6	34.65	110.6	4.0
A1	Conditioned	3711.1	34.68	107.0	4.0
A1	Unconditioned	3943.7	34.62	113.9	4.0
A1	Unconditioned	4130.8	34.63	119.3	4.0
A1	Unconditioned	3919.8	34.66	113.1	4.0
A1	Conditioned	3240.2	34.65	93.5	7.0
A1	Conditioned	2862.8	34.53	82.9	7.0
A1	Conditioned	3140.4	34.62	90.7	7.0
A1	Unconditioned	2932.0	34.58	84.8	7.0
A1	Unconditioned	3009.1	34.55	87.1	7.0
A1	Unconditioned	3261.9	34.59	94.3	7.0

	Condition	Load (lb)	Cross Sectional Area (in^2)	Tensile Strength (psi)	Test Air Void
A2	Conditioned	3955.2	34.69	114.0	4.0
A2	Conditioned	4440.5	34.73	127.9	4.0
A2	Conditioned	4917.0	34.69	141.7	4.0
A2	Unconditioned	4727.2	34.69	136.3	4.0
A2	Unconditioned	5197.0	34.69	149.8	4.0
A2	Unconditioned	5228.9	34.66	150.9	4.0
A2	Conditioned	3443.7	34.69	99.3	7.0
A2	Conditioned	3690.0	34.62	106.6	7.0
A2	Conditioned	3219.8	34.66	92.9	7.0
A2	Unconditioned	3264.0	34.69	94.1	7.0
A2	Unconditioned	3659.0	34.69	105.5	7.0
A2	Unconditioned	3547.5	34.62	102.5	7.0
AB1	Conditioned	5212.0	34.65	150.4	4.0
AB1	Conditioned	5348.0	34.66	154.3	4.0
AB1	Conditioned	5046.5	34.68	145.5	4.0
AB1	Unconditioned	5908.0	34.69	170.3	4.0
AB1	Unconditioned	5951.0	34.70	171.5	4.0
AB1	Unconditioned	5661.0	34.69	163.2	4.0
AB1	Conditioned	3836.0	34.65	110.7	7.0
AB1	Conditioned	3244.0	34.66	93.6	7.0
AB1	Conditioned	3453.0	34.67	99.6	7.0
AB1	Unconditioned	3719.0	34.56	107.6	7.0
AB1	Unconditioned	3891.0	34.56	112.6	7.0
AB1	Unconditioned	4449.0	34.60	128.6	7.0

PLANT-PRODUCED MIXES

Table 22. Plant Mixes' TSR Specimen Properties

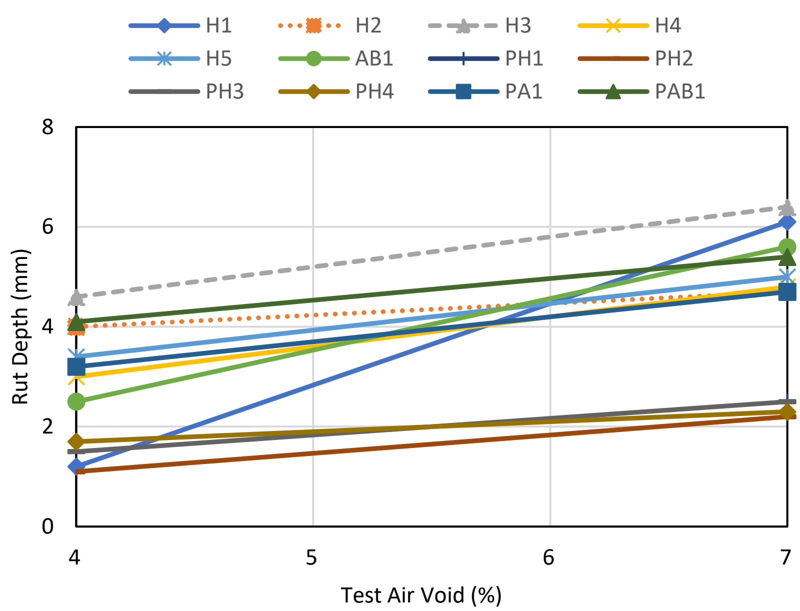
	Condition	Load (lb)	Cross Sectional Area (in^2)	Tensile Strength (psi)	Test Air Void
PH1	Conditioned	6901.8	34.70	198.9	4.0
PH1	Conditioned	7425.0	34.70	214.0	4.0
PH1	Conditioned	8852.1	34.69	255.2	4.0
PH1	Unconditioned	8096.5	34.69	233.4	4.0
PH1	Unconditioned	8439.5	34.69	243.3	4.0
PH1	Unconditioned	8626.9	34.69	248.7	4.0
PH1	Conditioned	5310.7	34.67	153.2	7.0
PH1	Conditioned	5945.3	34.69	171.4	7.0
PH1	Conditioned	5701.9	34.68	164.4	7.0
PH1	Unconditioned	6932.6	34.66	200.0	7.0
PH1	Unconditioned	7198.3	34.69	207.5	7.0
PH1	Unconditioned	7426.2	34.69	214.1	7.0
PH2	Conditioned	7559.8	34.69	217.9	4.0
PH2	Conditioned	8273.9	34.69	238.5	4.0
PH2	Conditioned	8610.0	34.69	248.2	4.0
PH2	Unconditioned	6421.1	34.69	185.1	4.0
PH2	Unconditioned	8536.8	34.69	246.1	4.0
PH2	Unconditioned	8410.1	34.70	242.4	4.0
PH2	Conditioned	6795.7	34.69	195.9	7.0
PH2	Conditioned	7152.1	34.65	206.4	7.0
PH2	Conditioned	5266.9	34.65	152.0	7.0
PH2	Unconditioned	7294.0	34.70	210.2	7.0
PH2	Unconditioned	6024.4	34.70	173.6	7.0
PH2	Unconditioned	7642.6	34.69	220.3	7.0
PH3	Conditioned	8659.7	34.69	249.6	4.0
PH3	Conditioned	9424.1	34.70	271.6	4.0
PH3	Conditioned	7723.9	34.70	222.6	4.0
PH3	Unconditioned	8354.7	34.70	240.8	4.0
PH3	Unconditioned	9547.3	34.69	275.2	4.0
PH3	Unconditioned	7915.7	34.69	228.2	4.0
PH3	Conditioned	6145.2	34.70	177.1	7.0
PH3	Conditioned	7080.6	34.69	204.1	7.0
PH3	Conditioned	6544.7	34.66	188.8	7.0
PH3	Unconditioned	5675.7	34.69	163.6	7.0
PH3	Unconditioned	7830.4	34.66	225.9	7.0
PH3	Unconditioned	8444.3	34.69	243.4	7.0
PH4	Conditioned	7269.4	34.70	209.5	4.0

	Condition	Load (lb)	Cross Sectional Area (in^2)	Tensile Strength (psi)	Test Air Void
PH4	Conditioned	7992.5	34.69	230.4	4.0
PH4	Conditioned	7287.8	34.69	210.1	4.0
PH4	Unconditioned	8439.6	34.69	243.3	4.0
PH4	Unconditioned	8713.4	34.69	251.2	4.0
PH4	Unconditioned	9178.4	34.69	264.6	4.0
PH4	Conditioned	6089.1	34.66	175.7	7.0
PH4	Conditioned	6082.7	34.70	175.3	7.0
PH4	Conditioned	6081.9	34.65	175.5	7.0
PH4	Unconditioned	6703.1	34.70	193.2	7.0
PH4	Unconditioned	7938.7	34.70	228.8	7.0
PH4	Unconditioned	7630.4	34.70	219.9	7.0
PA1	Conditioned	5217.1	34.57	150.9	4.0
PA1	Conditioned	5430.0	34.63	156.8	4.0
PA1	Conditioned	6243.3	34.69	180.0	4.0
PA1	Unconditioned	7413.4	34.66	213.9	4.0
PA1	Unconditioned	6195.5	34.69	178.6	4.0
PA1	Unconditioned	5906.6	34.62	170.6	4.0
PA1	Conditioned	4527.9	34.70	130.5	7.0
PA1	Conditioned	4853.9	34.65	140.1	7.0
PA1	Conditioned	4822.7	34.65	139.2	7.0
PA1	Unconditioned	4485.6	34.61	129.6	7.0
PA1	Unconditioned	4896.7	34.70	141.1	7.0
PA1	Unconditioned	5210.6	34.69	150.2	7.0
PA2	Conditioned	3734.7	34.71	107.6	4.0
PA2	Conditioned	3651.6	34.68	105.3	4.0
PA2	Conditioned	4435.7	34.65	128.0	4.0
PA2	Unconditioned	4344.0	34.67	125.3	4.0
PA2	Unconditioned	4752.8	34.69	137.0	4.0
PA2	Unconditioned	5113.0	34.66	147.5	4.0
PA2	Conditioned	3193.6	34.64	92.2	7.0
PA2	Conditioned	3305.0	34.61	95.5	7.0
PA2	Conditioned	2953.2	34.70	85.1	7.0
PA2	Unconditioned	3115.4	34.69	89.8	7.0
PA2	Unconditioned	3666.9	34.66	105.8	7.0
PA2	Unconditioned	4273.3	34.69	123.2	7.0
PAB1	Conditioned	7403.2	34.69	213.4	4.0
PAB1	Conditioned	6456.8	34.73	185.9	4.0
PAB1	Conditioned	7801.8	34.69	224.9	4.0
PAB1	Unconditioned	8023.9	34.62	231.8	4.0
PAB1	Unconditioned	8797.4	34.66	253.8	4.0

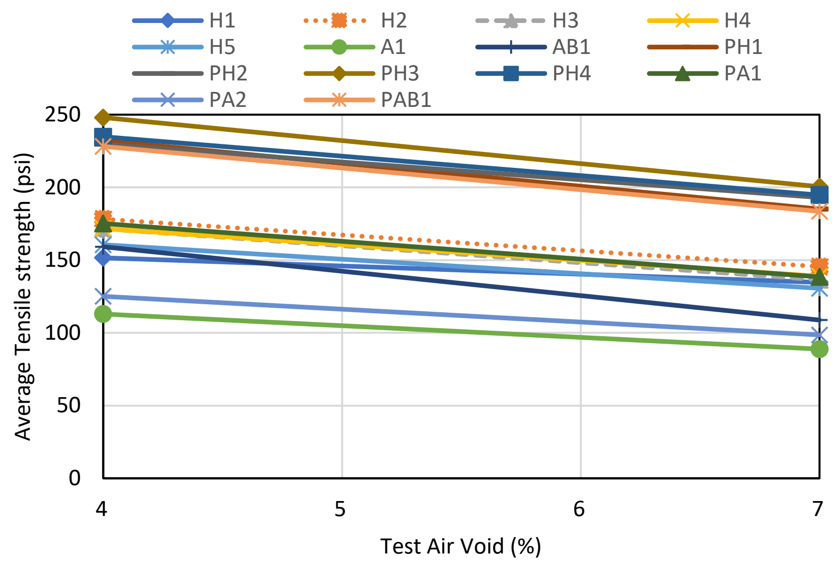
	Condition	Load (lb)	Cross Sectional Area (in^2)	Tensile Strength (psi)	Test Air Void
PAB1	Unconditioned	8989.5	34.65	259.4	4.0
PAB1	Conditioned	5661.8	34.69	163.2	7.0
PAB1	Conditioned	5946.6	34.65	171.6	7.0
PAB1	Conditioned	6585.6	34.62	190.2	7.0
PAB1	Unconditioned	6064.7	34.70	174.8	7.0
PAB1	Unconditioned	6426.2	34.74	185.0	7.0
PAB1	Unconditioned	7489.5	34.63	216.3	7.0

APPENDIX E: IMPACT OF AIR VOID CONTENT ON PERFORMANCE OF EACH MIX

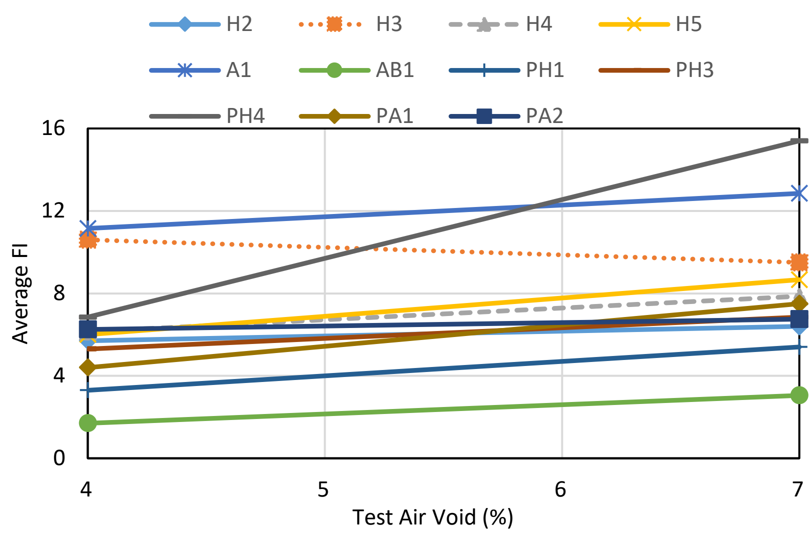
This study involved the testing of mix specimens using the HWTT, I-FIT and TSR at 4% and 7% air voids. Volumetrics were measured for each specimen and ensured they were between ±0.5% of the test air void. The influence of air void content on mixture and pavement performance has been extensively studied in literature (Harvey & Tsai, 1996; Seo et al., 2007; Tedla et al., 2023; Zaltuom, 2018; Zeiada et al., 2014). Perhaps the most popular study was by Linden et al. (1989) where the influence of compaction (specifically, air voids) on performance of pavement surfaces was investigated using existing literature, a questionnaire survey and performance data. The existence of a correlation between air void and performance was established and has been subsequently confirmed by later studies. Figure 24 plots the HWTT, TSR and I-FIT for all mixes at 4% and 7% air voids. As expected, for all mixes, there was a relationship between air void content and performance values. With every increase in air void from 4% to 7%, there is an increase in rut depth (mm) for the HWTT, a decrease in tensile strength (psi) for the TSR and an increase in flexibility index for the FI. However, for the FI, caution needs to be taken when generalizing as several other factors such as binder content, aggregate size, distribution, and orientation among others affect the flexibility of a mix.



A. HWTT



B. TSR (tensile strength)



C. I-FIT

Figure 24. Graphs. Performance tests values at different air void content.

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