SERRI Report 70015-011

Full Scale Testing of Hot-Mixed Warm-Compacted Asphalt for Emergency Paving

SERRI Project: *Increasing Community Disaster Resilience Through Targeted Strengthening of Critical Infrastructure*

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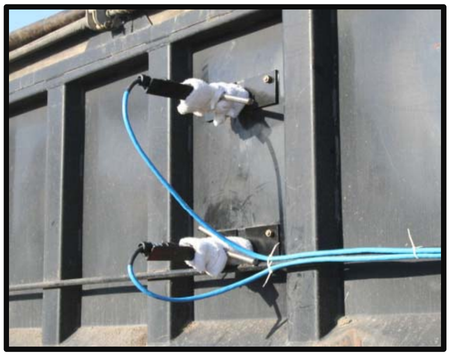
Mississippi State University CMRC Construction Materials Research Center

"An Industry, Agency & University Partnership"



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SERRI Project: Increasing Community Disaster Resilience
Through Targeted Strengthening of Critical Infrastructure

FULL SCALE TESTING OF HOT-MIXED WARM-COMPACTED ASPHALT FOR EMERGENCY PAVING

Performing Organization Report No. CMRC-12-01

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SYMBOLS

$\%G_{mm}$	Percent of theoretical maximum specific gravity
±PCS	Percent above or below the primary control sieve
AC	Asphalt content
ACP	Accumulated Compaction Pressure
APA	Asphalt Pavement Analyzer
AWD	Asphalt Workability Device
B1	PG 67-22, Hunt Refining Company
B2	PG 67-22, Ergon Asphalt & Emulsions, Inc.
B3	PG 67-22, Ergon Asphalt & Emulsions, Inc., dosed with Evotherm 3G™
BBR	Bending Beam Rheometer
cov	Coefficient of variation
C	Degrees Centigrade
CR	Cohesion ratio
DSR	Dynamic Shear Rheometer
F	Degrees Fahrenheit
FA_c	Fine aggregate ratio
FC	Field compacted
G_b	Specific gravity of binder
G_{mb}	Bulk mixture specific gravity
G_{mm}	Theoretical maximum mixture specific gravity
G_{sb}	Bulk specific gravity of aggregate
G_{se}	Effective specific gravity of aggregate
HLWT	Hamburg Loaded Wheel Tester
HMA	Hot Mixed Asphalt
HVS	Heavy Vehicle Simulator
JB	JB Kwikweld used to attach thermocouples
k	Cooling rate constant
L_1	Density measurement location defined in Figure 3.12
L1	Location of two bead thermocouples, 3.8 cm from inner edge of truck bed
L_2	Density measurement location defined in Figure 3.12
L2	Location of two bead thermocouples, 41.9 cm from inner edge of truck bed
L_3	Density measurement location defined in Figure 3.12
L3	Location of two bead thermocouples, 73.7 cm from inner edge of truck bed
L4	Location of two bead thermocouples, 111.8 cm from inner edge of truck bed
LAC	Linear Asphalt Compactor
LM	Laboratory mixed
MTV	Material Transfer Vehicle
N_F	Total number of passes with compactor (total number of P_F)
NI	National Instruments™
NMAS	Nominal Maximum Aggregate Size
N_S	Number of static passes with roller
N_T	Sum of vibratory and static passes with roller
N_V	Number of vibratory passes with roller

$P_{0.075}$	Fines content
PAV	Pressure Aging Vessel
P_b	Total asphalt content
$P_{ba(mix)}$	Percent binder absorbed on a mix mass basis
P_{be}	Effective asphalt content
$P_{12.5}$	Number of passes required to achieve 12.5 mm of rutting in PURWheel
$P_{12.5(R)}$	Ratio of $P_{12.5}$ values (wet to dry ratio) expressed as a percentage
PCSI	Primary control sieve index
P_F	One full roller pass (left and right pass)
PG	Performance grade
PL	Existing parking lot prior to placement of asphalt test strips
P_L	Left pass by roller during compaction
PM	Plant mixed
PQI	Pavement Quality Indicator
P_R	Right pass by roller during compaction
RAP	Reclaimed Asphalt Pavement
RD_{APA}	Rut depth measured by APA
RD_{PW}	Rut depth measured by PURWheel
RH	Relative humidity
RR_{PW}	Rutting rate in PURWheel expressed as mm of rutting per 1,000 cycles
RTFO	Rolling Thin Film Oven
RTV	Silicone used as space filler for thermocouples
SGC	Superpave Gyrator Compactor
SIP	Stripping Inflection Point
SSD	Saturated Surface Dry
STAPs	Short Term Aging Protocols
rpm	Revolutions per minute
t	Thickness of asphalt mat, asphalt cores, or compacted asphalt specimens
t	Time
t/NMAS	Relationship between the mat thickness and nominal maximum aggregate size
$T_{Air\text{-}PL}$	Air temperature at parking lot where test strips were placed
t_{bp}	Time when the mix was dumped into the paver
$T_{B\text{-}PL}$	Existing parking lot temperature serving as a base for the test strips
TCE	Trichloroethylene
T_{comp}	Compaction temperature
T_e	Embedded temperature of mat
t_{ep}	Time when the mix was placed, but no compaction occurred
t_h	Haul time
t_L	Time when truck was loaded and pulled out from the plant silo
t_{Load}	Time from when mix was loaded into truck from silo (min)
$t_{Load\text{-}1}$	Time from when first truck was loaded for the day (min)
t_{Lp}	Time when truck left the plant
T_M	Temperature near the middle of the asphalt layer
T_{mix}	Mixing temperature
T_{oven}	Oven setting during phase 1 short term aging protocol testing
T_{Post}	Post-haul temperature

$T_{Post}(t_h)$	Post-haul or arrival temperature after haul time t_h
T_{Pre}	Mixture production temperature or pre-haul temperature
T_s	Asphalt surface temperature measured by hand held device
t_{Screed}	Time referenced when the screed first passed over the location of interest
T_{Screed}	Mix temperature behind the paving screed
T_{STAP}	Short term aging protocol temperature
T_{ta}	Ambient temperature surrounding truck
V_a	Air voids
$V_{a(P)}$	Percent air voids in laboratory compacted specimens during production
V_{a-N}	Air voids measured with data collected from nuclear gage
V_{a-P}	Air voids measured with data collected from a PQI
V_{a-T166}	Percent air voids measured using AASHTO T166
$V_{a-T166-Post}$	Air voids measured post-haul using AASHTO T166
$V_{a-T166-Pre}$	Air voids measured pre-haul using AASHTO T166
V_{a-T331}	Percent air voids measured using AASHTO T331
$V_{a-T331-Post}$	Air voids measured post-haul using AASHTO T331
$V_{a-T331-Pre}$	Air voids measured pre-haul using AASHTO T331
vpm	Vibrations per minute
Δ5k	The difference in wet and dry PURWheel rut depths at 5,000 passes
VMA	Voids in mineral aggregate
WMA	Warm mix asphalt
WMT	Warm mix technology

ACRONYNMS

AASHTO	American Association of State Highway and Transportation Officials
AMRL	AASHTO Materials Reference Laboratory
APAC	Trade name of paving company
ASTM	American Society for Testing and Materials
BCD	Burns Cooley Dennis
BW	Vibratory roller type manufactured by Bomag
CEE	Civil and Environmental Engineering
DHS	Department of Homeland Security
DoD	Department of Defense
DOT	Department of Transportation
FEMA	Federal Emergency Management Agency
LTPPBind	Long Term Pavement Performance Superpave binder selection software
MDOT	Mississippi Department of Transportation
MSU	Mississippi State University
NAPA	National Asphalt Pavement Association
NCAT	National Center for Asphalt Technology
NCHRP	National Cooperative Highway Research Program
NRF	National Response Framework
ORNL	Oak Ridge National Laboratory
PTSi	Paragon Technical Services, Inc
SERRI	Southeast Region Research Initiative
TRB	Transportation Research Board
USACE	United States Army Corps of Engineers
USCS	Unified Soil Classification System

EXECUTIVE SUMMARY

The primary objective of this report was to perform full-scale testing of hot-mixed and warm-compacted asphalt for disaster recovery purposes (e.g. respond to hurricane damage). After disasters such as hurricanes, power is often out for large distances, which limits the use of conventional construction approaches. Use of hot-mixed and warm-compacted asphalt hauled from a considerable distance (i.e. a location with power and functioning infrastructure) should drastically reduce recovery time by increasing efficiency of all activities associated with response and recovery.

This report was phase 2 of the emergency paving effort. Phase 1 was a laboratory study that ended with two primary questions: 1) can hot-mixed and warm-compacted asphalt be delivered to a location of interest at a temperature of 105 C or higher; and 2) can hot-mixed and warm-compacted asphalt be compacted to 11 to 14% air voids after a very long haul distance? The answer to both questions was yes.

Phase 2 consisted of producing asphalt concrete at a full-scale facility, loading the material into trucks (some trucks were instrumented), hauling the material for different amounts of time, and compacting the material into test strips on a parking lot. The process was monitored from production, to transport, to paving, to compacted material properties. Approximately 175 laboratory compacted specimens were tested, alongside approximately 750 field cores and over 100 field sawn slabs.

Asphalt concrete could be hauled 1.0 to 10.5 hr and be placed with a paver. The mix was subsequently compacted to 6.8 to 11.6% air voids based on AASHTO T166. Testing including workability, binder grading, wheel tracking, and moisture damage revealed no formidable problems for emergency paving. An emergency pavement compacted to even modest levels should last at least a few thousand truck passes.

For haul distances of 8 hr or less, there was no compelling case to use any mix type (traditional hot mixed asphalt, foamed asphalt, or Evotherm 3G™ modified asphalt) over another in terms of in place air voids. Foamed asphalt, though, would be a logical choice in Mississippi since many asphalt plants have the necessary equipment. Evotherm 3G™ modified asphalt was the only product the research team felt comfortable taking to 10 hr (+) haul times. It's compaction ability at these haul times was fairly remarkable, especially considering it was not different than traditional hot mixed asphalt at conventional haul times.

The overall recommendation from this research is to use hot-mixed and warm-compacted asphalt concrete as an emergency paving material for disaster recovery applications. The approach has passed laboratory and full-scale testing.

CHAPTER 1 – INTRODUCTION

1.1 General and Background Information

The work presented in this report was developed in partial fulfillment of the requirements of Task Order 4000064719 sponsored by the *Department of Homeland Security (DHS)* through its *Southeast Region Research Initiative (SERRI)* program administered by *UT-Battelle* at the *Oak Ridge National Laboratory (ORNL)* in Oak Ridge, Tennessee. The original research (phase 1) was proposed by members of the *Department of Civil and Environmental Engineering (CEE)* at *Mississippi State University (MSU)* to *SERRI* in a document dated 1 June 2007. The project initiated on 1 January 2008. Two phase 1 Task Order 4000064719 modifications occurred; 9 September 2008 and 22 June 2010. The third task order modification occurred 13 April 2011, which added the work presented in this report to the research effort (phase 2).

The scope of work associated with phase 1 of Task Order 4000064719 included several related components comprised of six major tasks. Ten research reports were written containing all research performed for these six tasks (SERRI Report 70015-001 through SERRI Report 70015-010). Task 4 of phase 1 (SERRI Report 70015-004; Howard et al. 2010) dealt with pavement characterization and repair, and this report (phase 2) builds upon Task 4 in the area of using hot-mixed warm-compacted asphalt for emergency paving to expedite disaster recovery. This report is the final Task Order 4000064719 (Mod 3) report, could be viewed as Task 4-Ph 2, and was given the report number SERRI Report 70015-011.

The overall goal of the project was to determine how far asphalt concrete can be hauled incorporating warm mix technology and how it will perform once on site for a given application. The primary motivation of the research is to meet emergency paving needs. The research fits into the *National Response Framework* (*NRF*), which is described in NRF (2008). Howard et al. (2010) provides detailed discussion of how the emergency paving work presented in SERRI Report 70015-004 (and by extension 70015-011) fit into the *NRF*.

The objective of SERRI Report 70015-004 was to develop protocols for quickly and accurately evaluating and prioritizing pavement networks post natural disaster for initial response operations. The phase 1 work provided guidance on making informed decisions on pavement sections needing repair, as well as techniques that could be used to make the temporary repairs. The repair solution that was studied in the most detail was hot-mixed warm-compacted asphalt. All phase 1 work was in the laboratory. The target service life for the temporary repairs was 60 days in a warm and wet environment.

Phase 1 provided evidence that material haul distances and compaction conditions could be extended beyond current practice for short term use. This finding is significant as it allows the asphalt industry to respond to disasters much more effectively. With respect to current practice, air void levels were relatively high and when tested in these conditions mixtures rutted on the order of six times faster during early testing as they did during later testing. Overall rutting levels, however, did not appear to be problematic for a temporary application. Incorporation of warm mix additives was, in general, preferred for the application. Full-scale instrumented testing was recommended to determine how long a mix could be hauled and still be compacted, and thereafter how the material would perform in a temporary application.

1.2 Objectives

The general objective of Task Order 4000064719 was to investigate several specific means by which local communities may best use available resources in an effort to rapidly recover from a flooding disaster. A key component of this research was to develop solutions which may be rapidly deployed to achieve maximum benefit to the community, typically through use of on-site materials, pre-engineered components, and innovative construction materials and techniques. The specific objectives of this report are related to emergency paving as described later in this section.

A research status report released in May of 2012 by the Transportation Research Board (TRB) of the National Academies provided evidence regarding the fairly unique nature of the objectives of the current research. The report states that since September 2001 that 136 security, emergency management, and infrastructure protection related planning and implementation projects have been initiated through TRB. None of the projects listed have any connection to the process of emergency paving.

The primary objective of this report was to perform full-scale testing of asphalt containing warm mix technologies to assess the feasibility of using hot-mixed warm-compacted asphalt in response to water induced disasters (e.g. hurricane, tsunami, and river flooding events). Water induced disasters can often cause extreme pavement damage that can hamper recovery efforts. After disasters, power is often out for large distances, which limits the use of conventional construction approaches. Use of hot-mixed warm-compacted asphalt hauled from a considerable distance (i.e. from a location with power and functioning infrastructure) should drastically reduce recovery time by increasing efficiency of all activities associated with response and recovery. While the aforementioned concept is sound, the primary objective of the research was to determine if it is feasible using full-scale testing.

To accomplish the primary objective, the research built upon SERRI Report 70015-004. The two primary questions remaining from phase 1 were: can the mixture of interest be delivered to the location of interest at a temperature at or in excess of 105 C; and can the mixture delivered be compacted to 11 to 14% air voids? The full-scale field testing presented in this report addressed both these questions along the way to meeting the primary objective.

1.3 Scope

This report fully addresses phase 2 of Task Order No. 4000064719 dated 13 April 2011. This research investigated new techniques to improve reconstruction of transportation corridors for emergency response due to the forces of waves and flowing water. A noteworthy component of the project scope was to interact with potential stakeholders to ensure operational requirements and capability gaps in the areas of transportation were adequately addressed. Specific tasks are listed below.

Task 1: Align Research with Well-Understood Customer Requirements

The Seller shall continue efforts to ensure that this research effort is aligned with well-understood customer requirements. To this end, the Seller shall assist homeland security components (e.g., first responders or prospective end-users) in assessing, articulating and pinpointing their operational requirements and capability gaps associated with reconstruction of transportation corridors after hurricanes, tornadoes, and other storm-related disasters associated with extreme wind storm events. The Seller shall interface with regional emergency management practitioners or other potential end users (e.g. state DOT's) to ensure capability gaps and operational requirements are understood and being addressed by this effort.

Task 2: Build and Implement Protocols for Full-Scale Testing of Asphalt Pavement

The Seller shall perform full-scale testing of asphalt containing warm mix technologies to adequately address the critical/essential operational requirements and capability gaps identified in Task 1. The Seller shall produce enhancements through the selection, acquisition, integration and development of needed technology. The key issues envisioned are how far can the material be hauled and can the material perform adequately during disaster response.

Task 2.1: Develop Test Plans and Data Collection Methods

The Seller shall develop test plans and data collection methods to perform full-scale testing of asphalt containing warm mix technologies. The Seller shall identify key variables to plan and execute the asphalt production and pavement layer construction. Variables to be considered shall include but should not be limited to: selection of base asphalt mixture; compaction protocol; selection of trucks for hauling; inclusion of a material transfer vehicle; and temperature levels for compaction. The Seller shall also investigate techniques to allow a full-scale asphalt transport vehicle to be equipped with thermocouples (or equivalent) to measure temperature profiles within an asphalt mixture while the mixture is being hauled from the production facility to the construction site. The Seller shall also attempt to identify and implement an instrumentation technique to measure cooling rates of the warm mix technologies during transport.

Task 2.2: Perform Full-Scale Tests

The Seller shall produce full-scale asphalt mixtures incorporating warm mix additives and compact them after varying haul distances. The Seller shall carefully monitor the compaction process to determine feasible haul distances for disaster response using warm mix technologies.

Task 2.3: Conduct Laboratory Tests of Pavement Specimens from Full-Scale Tests

The Seller shall collect pavement specimens from the full-scale tests for laboratory examination and characterization. At minimum, the Seller shall attempt to develop laboratory tests to characterize the following components: 1) volumetric measurements to determine air voids, maximum theoretical density, and effective specific gravity; 2) asphalt extraction and recovery to determine as produced aggregate gradations and PG binder grade; 3) moisture damage testing; and 4) wheel tracking to investigate mixture stability.

Task 2.4: Analyze Test Results

The Seller shall analyze results of full-scale and laboratory tests to evaluate the suitability of the warm mix technologies to aid in disaster response, and provide guidance on what conditions these materials would be suitable for.

Task 3: Confirm the Practicality of Research Test Results for Disaster Response

The Seller shall discuss the feasibility of using hot-mixed warm-compacted asphalt during disaster response with prospective stakeholders and end-users. This should include input on the experimental tests and feedback on the results of laboratory/field tests. The Seller shall work closely with prospective stakeholders and end-users to determine the value, merits, and benefits of the results of this research and development effort for reconstructing asphalt pavement within transportation corridors for disaster response.

Task 4: Document Research Activities & Results

The Seller shall develop a final report describing the activities and the products developed to improve reconstruction of transportation corridors using hot-mixed warm-compacted asphalt. The Seller shall seek opportunities to ensure the knowledge gained, the technology developed, and the science-based solutions are integrated into homeland security operations for appropriate end-users at the federal, state, and/or local levels.

CHAPTER 2 –LITERATURE REVIEW

2.1 Overview of Literature Review

An exhaustive literature review of all factors that could be pertinent was beyond the scope of this project. Numerous publications exist in the areas of warm mix technology, asphalt temperatures, field compaction, volumetrics, and wheel tracking, but only literature directly applicable to the current project was referenced. One reason for this approach to the literature review is this project is phase 2 of an emergency paving effort and phase 1 (Howard et al. 2010) defined the boundaries of the project in a manner that made a comprehensive literature review during phase 2 unnecessary. A considerable amount of information is contained in Howard et al. (2010) pertaining to emergency paving that was not repeated in this report. The remainder of this chapter provides information obtained from literature and practice review that is directly applicable to the current project.

2.2 Warm Mix Technology

As of 2010, there were over twenty warm mix technologies in the US, but broadly speaking they can be divided into three general categories: chemical, foaming, and organic (wax). This project made use of chemical (Evotherm 3G™) and foaming technology. Phase 1 (Howard et al. 2010) made use of three technologies: chemical (Evotherm 3G™), sequential mixing (process suggested by some, e.g. Wendel 2011, to be able to enable mixing temperature reduction without foaming or moisture presence), and organic (Sasobit®). Sequential mixing is more suited for batch plants and if foaming is an option, the disaster recovery appeal of sequential mixing is lessened. Evotherm 3G™ and Sasobit® performed well in phase 1; either approach seems suitable for disaster recovery. Sasobit® often requires a feeding system to be brought to the facility, though is can be pre-mixed (preferred disaster recovery option). Evotherm 3G™ can be delivered pre-mixed into the asphalt binder.

Evotherm 3G™ is the product resulting from several iterations of the technology. Multiple surfactant chemistries, labeled A to M, have been used. The first iteration is referred to as Evotherm™ Emulsion and used specialized surfactants to produce emulsified asphalt (approximately 30% water) with warm mix properties. The second iteration is referred to as Evotherm DAT which used the same surface chemistry and surfactant solutions (in a more concentrated form with less water) that was injected into the asphalt binder line in an otherwise traditional asphalt concrete production process. The third iteration is referred to as Evotherm 3G™, which has no added moisture.

Foamed asphalt is created when cool water is injected into hot liquid asphalt binder in small amounts (e.g. 2% of the asphalt binder mass). The cool water vaporizes and greatly expands (i.e. foams) the liquid asphalt binder for a period of time. Expansion of the asphalt binder facilitates coating, and typically allows a mixing temperature reduction. Foamed asphalt half-life, in general, is related to the binder/mixture temperature with higher temperatures producing a shorter half-life; information was presented in one reference cited by a recently released best practices document (Prowell et al. 2011). Half-life is the time for the foam volume to decrease by one-half. Another reference cited by Prowell et al. (2011) provided an alternative suggestion that mechanical foaming systems provided compaction benefits over non-foamed HMA even at typical HMA production temperatures.

Multiple asphalt producers in Mississippi have equipped their plants with foaming equipment, which made the process a logical approach for this demonstration. As a reference, foamed asphalt is currently produced in Mississippi in the 138 to 143 C range and placed in the 124 to 129 C range for typical Mississippi Department of Transportation (MDOT) applications according to their Materials Division.

2.3 Warm-Mix Technology Applications

Warm-mix asphalt (WMA) was initiated in the 1990's, so it is a relatively new development. The first field trials for warm-mix technology in the US were less than a decade ago. In the past few years, the emergence of WMA has brought to the paving industry a number of potentially promising advantages, such as fuel savings, reduced emissions, and reduced binder aging. Additional WMA advantages are extending the paving season (especially in colder climates), wider windows for night paving, longer haul distances, and serving as a compaction aid for stiff mixes (e.g. mixes with high reclaimed asphalt pavement (RAP) contents). Several sources (e.g., D'Angelo et al. 2008, Prowell and Hurley 2007) have discussed WMA, with extended haul distances being one point of discussion.

According to NCAT (2010), at least 45 states either actively use WMA technologies or have investigated their use via a trial project. Early performance of WMA projects was reported to be very good and comparable to HMA. Laboratory tests have suggested some WMA mixes may be more susceptible to rutting and moisture damage while field evaluations have not shown practical differences in rutting between WMA and HMA sections. WMA is actively being studied at the state and national levels. State level studies are too numerous to mention individually. As an example of the interest for WMA, notable research performed through the National Cooperative Highway Research Program (NCHRP) is summarized in the following list.

- 09-43: Mix Design Practices for Warm Mix Asphalt
- 09-47: Engineering Properties, Emissions, & Field Performance of Warm Mix Asphalt Technologies
- 09-47A: Properties and Performance of Warm Mix Asphalt Technologies
- 09-49: Performance of WMA Technologies: Stage I--Moisture Susceptibility
- 09-49A: Performance of WMA Technologies: Stage II--Long-Term Field Performance
- 09-52: Short-Term Laboratory Conditioning of Asphalt Mixtures
- 09-53: Properties of Foamed Asphalt for Warm Mix Asphalt Applications

Some warm mix technologies can be used at plant mixing temperatures (T_{mix}) ranging from that of traditional hot mix asphalt (e.g., 165 C) to lower mixing temperatures associated with WMA (e.g., 130 C). In a fairly short time, the additives and processes associated with the "warm" movement have proven to be highly versatile, and this versatility has resulted in some debate as to concise definitions and terminology. In the remainder of this report, warm mix technology (WMT) is the use of a "warm" additive or process in any manner, while WMA is restricted to using these same additives or processes when mixed at lower temperatures (e.g., 130 C).

2.3.1 Traditional Applications for Warm Mix Technology

Prowell et al. (2011) summarizes the National Asphalt Pavement Association (NAPA) Warm-Mix Asphalt: Best Practices 2nd Edition. Use as a compaction aid for RAP, RAS (reclaimed asphalt shingles), or other stiff mixes (e.g. low temperature compaction) appears viable and additives improve compaction at a given temperature. Table 2.1 summarizes pertinent field projects that were reported in the document.

Table 2.1. Pertinent Field Projects Summarized in Prowell et al. (2011)

State	Year	Haul	Summary
Ohio	2006	1 hr	3 WMA technologies were used
California	2008	1 to 3 hr	4 projects, polymer binders, cool weather, chunks formed
Texas	2008	96 km	121 C production temp, 107 C compaction, V_a 6.5 or less
Rhode Island	2010	2 hr	One hr of haul was on ferry, removed need to set up plant

Kristjansdottir (2006) performed a literature review and survey for Iceland focusing on warm mix technology use in cold climates. Three of the ten survey respondents mentioned long haul distances as a limiting factor to using hot mix asphalt more widely around the country when asked about the most common problems that arise during paving in Iceland. Four of the ten survey respondents mentioned long haul distances when asked "What do you think are the main benefits of warm mix asphalt?".

Table 2.2 shows examples in which WMT has been successfully used in permanent construction in a variety of conditions, supporting versatility of the technology, especially to facilitate compaction. In one case (*Pacific Coast Highway 1*), hot-mixed asphalt with WMT was successfully used with a haul distance of about 140 km. The key attribute of the data in Table 2.2 is that mixing temperatures (T_{mix}) were 116 to 152 C and temperatures at the beginning of compaction (T_{comp}) were 80 to 127 C. Based on the data presented in Table 2.2, increasing T_{mix} and targeting a similar T_{comp} appear viable for disaster recovery.

Table 2.2. Projects With Dense Gradation and Evotherm WMT

Project	PG Binder	NMAS (mm)	Site Conditions	T_{mix} (C)	T_{comp} (C)	Haul Time (hr)	Haul Dist (km)	V_a (%)
I-70, MO[a]	64-22	12.5	Overcast: 9 C	---	125	---	97	5.8
I-78, NJ	76-22	12.5	Night: cool: rain	116-143	102-127	1	58	5.1
I-70, CO	58-28	12.5	Air < 0 C	121	116	0.5	26	5.3
US 84, TX[b]	64-22	9.5	Sunny: 13 C	133	116-127	---	48-64	6.0
I-37, TX	76-22	12.5	Sunny: 21-32 C	127-130	116-118	Metro area	48	< 8.0
BU 287, TX	64-22	19.0	Sunny	116	88-110	1	80	5.5
BU 287, TX	76-22	12.5	Sunny	113-132	93-116	1	80	3 - 6
Hwy 1, CA[c]	64-16	12.5	Chilly: foggy	149-152	105-116	3-4	137	< 8.0
HVS, CA[d]	64-16	12.5	Sunny: balmy	121	80	---	---	7.1
NCAT, AL	64-22	19.0	Sunny	116-121	82-116	short	short	<5.0
NCAT, AL	64-22	12.5	Sunny	116-121	82-116	short	short	<5.0

a) Fewer roller passes required to achieve density.
b) Contractor began using Evotherm™ 3G when problems arose with foamed asphalt.
c) Very long haul on Pacific Coast Highway 1 with all laydown operations proceeding smoothly.
d) Caltrans Heavy Vehicle Simulator (HVS). Rutting performance same as control with T_{mix} of 155 C.

Hurley and Prowell (2005a, 2005b, 2006) performed three studies where Evotherm® (asphalt emulsion with approximately 70% residue), Aspha-min® Zeolite, and Sasobit® were investigated. One item of interest in the research efforts was quick traffic turn-over in high temperature conditions. There was no evidence of a difference in indirect tensile strength gain with time for mixes containing Evotherm with respect to control mixes indicating quick traffic opening is acceptable (PG 64-22 and PG 76-22 binders were investigated). No change in strength at a given age for hot and warm mixtures was reported, indicating there is no evidence to support the need for curing time prior to opening pavements to traffic when constructed with Aspha-min® Zeolite warm mix additive. A similar finding was reported when Sasobit® was the warm mix additive.

Saboundjian et al. (2011) performed a paving study in Alaska where binder was modified with Sasobit®, barge shipped approximately 1,300 km, and then used to produce WMA for a low volume road. The mix used 19 mm NMAS granite aggregate with 5% PG 58-28 asphalt binder (Sasobit® changed grade to PG 70-22). The project used 23,000 tons of mix for a 12 km stretch placed 7.5 cm thick in a single lift. During construction ambient temperatures were 7 to 13 C, and production rates were 270 to 280 tons/hr. WMA was mixed at 129 C, and placed at 110 to 121 C. The haul distance was 1.6 to 6.4 km, and there were no major problems during construction. A Bomag BW 205 vibratory roller was used throughout, and the average density was 95.5% of G_{mm} (1.3% standard deviation). During construction an infrared camera was used to monitor temperatures. A field survey just after project completion revealed no construction related distresses.

2.3.2 Emergency Applications for Warm Mix Technology

A 7.1 magnitude earthquake struck Yushu in China's Qinghai Province (sparsely populated frontier-land with few roads) on April 14, 2010 resulting in nearly 3,000 people reported dead or missing and over 10,000 injured. The information in this report was provided by individuals familiar with the work so no citation was available. A road 25 km long paved with Evotherm was the first road opened after the earthquake, and it was used for some of the initial relief efforts into the disaster area from the *Yushu Batang Airport*. As such it was named *Life Channel*. The project was high altitude (mostly 4,000 m above sea level) and low temperature (around 5 C) paving. After the earthquake, survivors and rescuers endured snow, rain, strong winds, and in some cases high altitude sickness.

The paving project began in 2009 with plans to complete by June 2010. After the earthquake, the project completion date was shortened to May 25 for one of the travel lanes. On May 8, Evotherm was introduced and allowed paving for longer periods each day (mornings and evenings were especially cold), which further shortened the completion to May 18. Once complete the *Life Channel* reduced travel time from 150 to 40 min.

The project was completed with a portable plant and third-party resources. The Evotherm modified asphalt was produced at 130 to 145 C, placed at 115 to 120 C, and compacted at 110 to 60 C. Figure 2.1 provides example construction photos, and Table 2.3 provides properties of the asphalt binder and surface layers.

Figure 2.1. Photos of Yushu, China Emergency Paving

Table 2.3. Properties of Marshall Designed Mixtures Used for Yushu *Life Channel*

Property	Binder Layer	Surface Layer	Specification
Asphalt Binder Penetration at 25 C (dmm)	100 to 120	100 to 120	---
Asphalt Content	5.2	5.3	---
Passing 9.5 mm (%)	76.6	78.2	---
Passing 2.36 mm (%)	30.7	33.0	---
Passing 0.075 mm (%)	6.2	6.2	---
Marshall Stability (kN)	13.3	10.8	>8
Marshall Flow Number (0.1 mm)	29.8	31.0	20 to 40
Laboratory Compacted Air Voids (%)	4.1	4.3	3 to 6 or 3 to 5
Field Compaction: % of Standard Marshall	98.9	98.4	$\geq$ 98
Core Thickness (cm)	5.2	4.1	$\geq$ 4.8 or $\geq$ 3.8

Note: 18 cm of cement treated base had already been placed prior to the earthquake.

2.4 Asphalt Temperatures During Production, Transport, and Compaction

2.4.1 Newton's Law of Cooling

Newton's Law of Cooling states that the time rate of change in temperature of an object is proportional to the difference between its temperature and the ambient surroundings. This relationship can be expressed in the form of Eq. 2.1. Terms in Eq. 2.1 were described with terms used elsewhere in the report for consistency.

$$T_{Post}(t_h) = T_{ta} + (T_{Pre} - T_{ta})e^{-k(t_h)} \quad (2.1)$$

Where,

t_h = haul time (min)
$T_{Post}(t_h)$ = post-haul or arrival temperature after a haul time t_h (C)
T_{ta} = temperature of air surrounding truck, or ambient temperature (C)
T_{Pre} = pre-haul or mixture production temperature (C)
k = cooling rate constant (1/min)
e = 2.718…..

2.4.2 Brock and Jacob (1998)

The amount of heat lost during transport depends on 9 factors: 1) mix temperature when loaded into truck; 2) ambient air temperature; 3) if the truck bed is insulated; 4) size of truck bed in relation to tons of mix hauled; 5) length of haul; 6) speed of travel; 7) waiting time at paver; 8) if the mix is covered; and 9) traffic delays. Heat is lost quickly to the air above the mass of asphalt in the truck and through the sides of the truck bed; however the relatively low thermal conductivity of asphalt mix slows the rate of heat transfer from the middle of the mass to the edges. An outer crust of mix relatively cooler than the center of the mix mass develops resulting in an insulating effect.

Data collected with a thermal imaging camera of asphalt mix in the bed of a truck was presented; the center of the mass was above 116 C while the cooler outer crust was on the order of 82 C. An instance was cited in Australia where mix was transported 240 km from the plant to the paving site; upon arrival the outside truck body temperature was 80 C, the mix exposed top was 96 C, and the center temperature of the mix mass was 152 C.

2.4.3 Diefenderfer et al. (2007)

Diefenderfer et al. (2007) describes several WMA field trials. The data from field trial B was used by Howard et al. (2010) to perform cooling rate calculations using Eq. 2.1. HMA and WMA (Sasobit®) were used in field trial B, and both were a 12.5 mm PG 64-22 mix with 5.3% asphalt binder. Table 2.4 summarizes pertinent data provided in the study alongside the calculated (k) value, which was practically identical between HMA and WMA.

Table 2.4. Field Trial Data (Diefenderfer et al. 2007)

Mix	Weather	t_h (min)	T_{ta} (C)	T_{Pre} (C)	$T_{Post}(t_h)$ (C)	T_{Screed} (C)	k (1/min)
HMA	Sunny	105	25	164	149	138 to 149	1.11E-3
WMA	Overcast	105	23	149	135	121 to 135	1.12E-3

-- The average haul speed was 41 km/hr, and no material transfer vehicle was used.
-- T_{Screed} is the mix temperature behind the paving screed.
--The coefficient of variation (*cov*) was 6.1% for k (HMA and WMA).

2.4.4 Howard et al. (2010)

Laboratory cooling rate experiments were performed in a metal pail filled with 33 kg of a 9.5 mm NMAS mixture with 6% PG 67-22 binder (the mix was at 26% air voids during the experiments). Probe thermocouples were used to measure temperature in the center of the mix, while bead thermocouples measured temperature of the mix crust (only performed for some experiments) and of the air temperature 30 cm above the mix. A *National Instruments NI Compaq Daq 9172* chassis and *NI 9211* module were used in conjunction with a program written in *LabView*™ to acquire the temperature measurements (readings were taken every ten seconds with most measurements taken in duplicate). The experiments were performed in an oven with a 0.708 m^3 volume and a uniformity of ± 1.0 C at 100 C.

Four cooling rate experiment types were performed (*Pre-Conditioned*, *Setting Dropped*, *Incremental Cooling*, and *Expeditious Cooling*), with a beginning mix temperature of 166 to 168 C (two experiments were successfully repeated for quality control purposes but were not shown for brevity). Figure 2.2 provides details of each test and the corresponding results. Eq. 2.1 was used to calculate k at 105 min using the Figure 2.2a and 2.2b data, and the results were 6.48 E-3, 6.12 E-3, and 4.77 E-3 for oven temperatures of 104, 93, and 82 C, respectively. These k values are 4.3 to 5.8 times higher than Table 2.4 indicating a much faster cooling rate than the estimated field cooling rate using Diefenderfer et al. (2007) data.

Figure 2.2 was collected to develop a series of laboratory Short Term Aging Protocols (STAPs) for simulating hot-mixed and warm-compacted asphalt temperature loss during transport to a disaster environment. Three STAPs were developed and used to compact slabs of asphalt tested in a variety of manners. Three test times were selected: 90 min to represent normal MDOT laboratory practices; 240 min to represent a fairly long haul distance for disaster recovery; 360 min to represent a very long haul distance for disaster recovery (authors assessment was any location along Gulf Coast could be supplied within 360 min after Hurricane Katrina level event). The *Incremental Cooling* and *Expeditious Cooling* approaches were abandoned after viewing the data in Figure 2.2. Rationale for moving forward only with *Pre-Conditioned* and *Setting Dropped* data is provided in the following paragraph; Table 2.2 was used as a guide for all decisions pertaining to Figure 2.2 data.

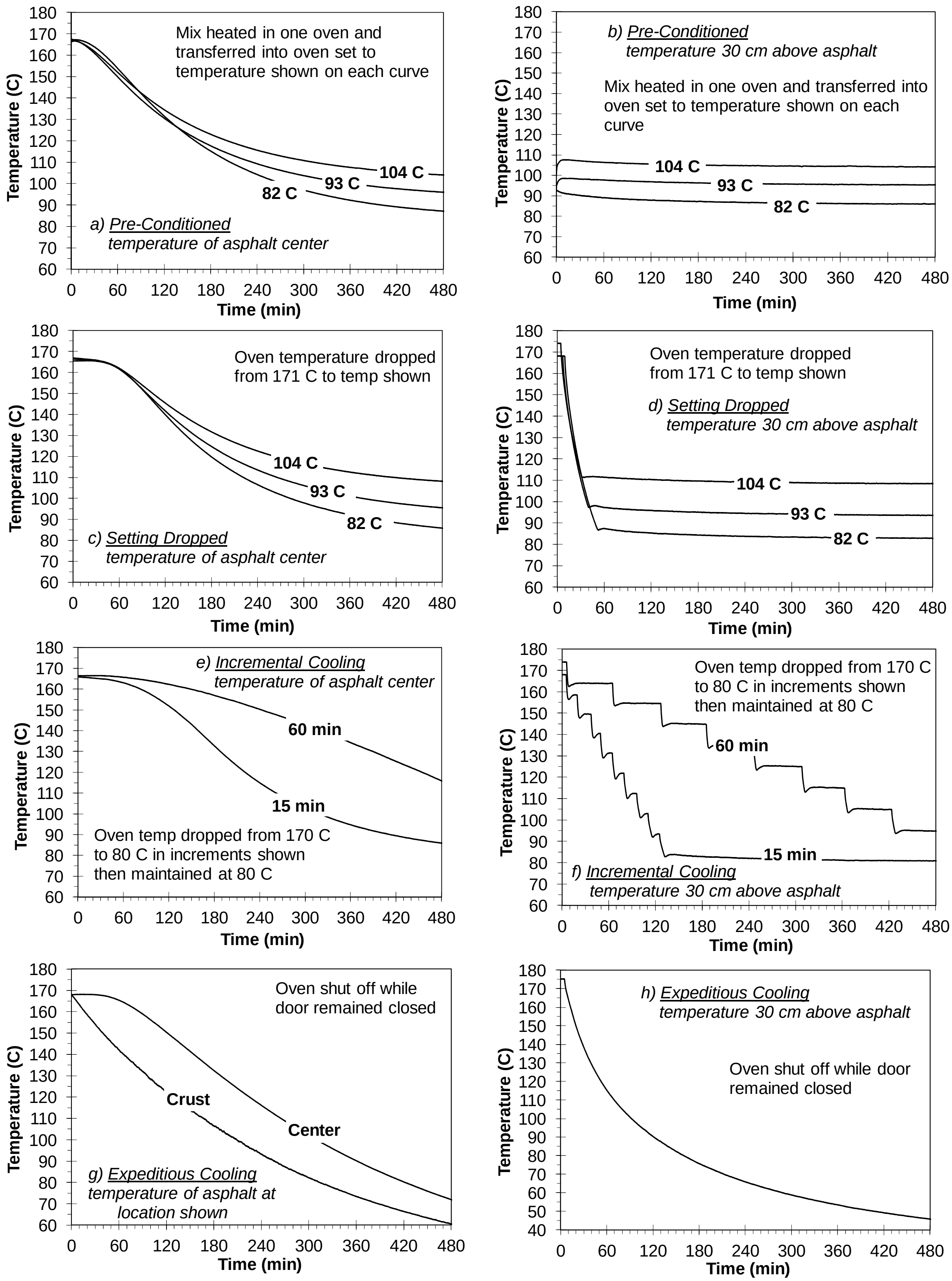


Figure 2.2. Summary of Howard et al. (2010) Cooling Rate Testing

Expeditious Cooling data was below anticipated field conditions. *Incremental Cooling* with multiple settings over time (i.e. dropping temperature every 15 min) is cumbersome for a laboratory and a similar curve can be produced with *Pre-Conditioned* or *Setting Dropped* protocols. *Incremental Cooling* while dropping temperature on 60 min intervals did not cool the material sufficiently to be compacted within a brief period after removal from the oven.

To help establish desired temperatures at the end of the three STAPs, data from the *Pre-Conditioned* and *Setting Dropped* protocols was averaged and plotted in Figure 2.3 alongside bands representing the overall standard deviation. The STAPs used are also shown alongside the oven setting used (T_{oven}) for the time (t) of each STAP. The oven was at the prescribed temperature and the material was placed in the oven just after mixing at a temperature $T_{mix.}$ At the STAP conclusion, the material exited the oven at temperature T_{STAP}, and the material cooled approximately 6 C after removal from the oven before compaction began at temperature T_{comp}.

The STAPs incorporated reduced temperature along the general path shown by the fitted equation in Figure 2.3. T_{STAP} values were equivalent to Diefenderfer et al. (2007) extrapolated values at 90 minutes, but lower at 240 and 360 min. This data indicates the laboratory collected data represented in Figure 2.3 may have a faster cooling rate than would occur in the field, but since the field data represents a considerable extrapolation there is some uncertainty in the comparison. T_{comp} values were 6 C below T_{STAP} values and within one standard deviation of the fitted equation from the Figure 2.2 cooling rate experiments.

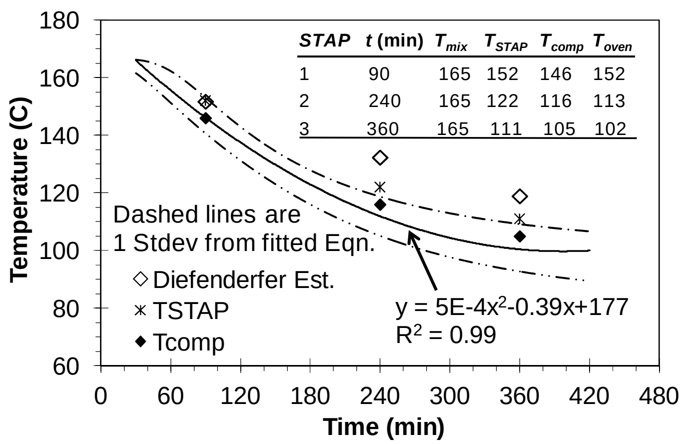


Figure 2.3. Development of STAP Temperatures

Diefenderfer et al. (2007) extrapolated temperatures were deemed higher than field compaction temperatures in many applications (Table 2.2 supports this position in general terms, in particular the *Pacific Coast Highway 1* project). With the uncertainty surrounding laboratory to field correlations, the researchers favored mixture compaction temperatures over cooling rates. In other words, the mixture compaction temperature was deemed more important than the specific nature of the cooling rate curve that led to the mixture compaction temperature. This was partly due to the uncertainty pertaining to a cooling rate curve that would lead to a mixture compaction temperature in the field, and the higher level of certainty on desired field compaction temperatures (i.e. Table 2.2). The approach selected was to cool the asphalt along the general trend shown in Figure 2.3 for the time of interest to a temperature 6 C above the desired compaction temperature. The approach taken, while not ideal, was believed to be the best available approach at the time as full-scale data for the time durations of interest was not available.

2.4.5 Williams et al. (2011)

Williams et al. (2011) performed a field study in Mississippi where twelve projects were studied and forty-four locations were evaluated within the twelve projects. During one part of the evaluation temperature data was collected throughout compaction; Figure 2.4 was generated using the data collected. Pavement surface temperature (T_S) was collected using a hand held infrared device, and the temperature near the middle of the pavement layer (T_M) was collected with an embedded thermocouple. As seen in Figure 2.4, the temperature near the middle of the pavement layer was on the order of 8% higher than the pavement surface temperature.

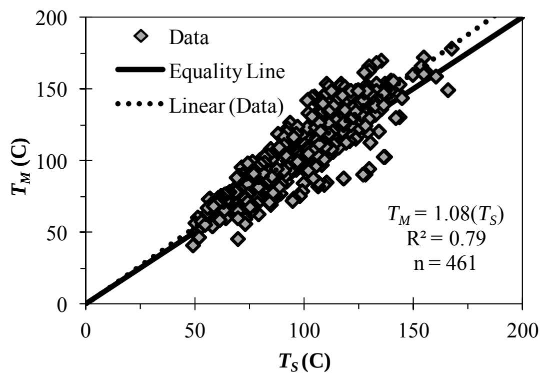


Figure 2.4. Surface and Internal Mat Temperature (Williams et al. 2011)

2.4.6 Adams (1960)

Adams (1960) reported pavement density can be negatively affected when the base material under the asphalt is below ≈ 24 to 27 C. Base (or underlying) temperatures above 27 C resulted in no additional density improvement. When the base temperature was 27 C, the air temperature was ≈ 13 C. Roadway density was not affected by base temperature increases from 6 to 18 C where air temperature was 5 to 10 C. Base temperature exceeded air temperature throughout the study. Test data was collected in central Louisiana and was obtained every 180 m over a one year period. A 10 ton 3-wheel and 10 ton tandem wheel roller was used on a 19 mm maximum aggregate size asphalt placed in 5 cm lifts.

2.4.7 Prowell et al. (2011)

Prowell et al. (2011) reported cooling rate was driven by the difference between the mix and air temperatures, which was stated to be important since the majority of the compaction that occurs is while mix is at its highest temperature. Similarly, Goh and You (2009) suggested that WMA cooling rates are slower than HMA cooling rates due to the smaller difference between production and ambient temperature. PaveCool and MultiCool are computer programs that can determine the compaction window for a given set of conditions in terms of the mix temperatures. Prowell et al. (2011) used MultiCool to calculate cooling rates for 163, 135, and 107 C arrival temperatures and it took 5, 7, and 12 min for the mix to cool to 42 C below the arrival temperature.

2.5 Compactability of Asphalt Concrete

Leiva and West (2008) identified the following factors as affecting HMA field compaction: aggregate type, gradation (NMAS is an important gradation parameter), environmental conditions, asphalt binder characteristics, compaction equipment, roller operation, and lift thickness (t). Aggregates with rough surface texture, cubical shaped particles, and highly angular particles were reported to require increased compactive effort. In general, asphalt mixtures with higher fines content were reported more difficult to compact than mixtures with lower fines content.

Environmental conditions that affect HMA field compaction were identified by Leiva and West (2008) as underlying surface temperature, ambient temperature, and wind speed. These environmental conditions affect the mix temperature by controlling mat cooling rates. The longer the environmental factors allow the HMA temperature to remain within an optimum compaction temperature range, the more time to achieve the desired density. Compaction is accomplished by three equipment types: paver screed, steel wheeled rollers, and pneumatic rollers. Roller type and operational characteristics (mass, dynamic force, wheel load, and tire pressure) affect compaction. Roller speed is important in that lower roller speed decreases the mix shear rate and allows aggregates to rearrange into more dense configurations.

Leiva and West (2008) defined Accumulated Compaction Pressure (ACP) to quantify the total compactive effort applied to an asphalt mat. ACP was defined as the summation of pressure applied by each pass of each roller in the field compaction process. Generally speaking, ACP is the downward force divided by the roller area in contact with the asphalt mat, though some of the terms in the approach have some latitude in their definition. Test results suggested total compactive effort was mainly affected by lift thickness and mix temperature. The ACP approach is useful when multiple rollers are used between multiple projects.

West and Leiva (2010) developed correlations between laboratory and field compaction. The most easily compacted mixes were fine graded. The relationship between the mat thickness (t) and NMAS, or t/NMAS, was investigated and it was observed that when the relationship was below 3 more compaction energy was required. Mixes with temperatures less than 107 C at the first pass of the breakdown roller required higher compaction energy, as did mixes with a lift thickness less than 50 mm due to temperature loss. A multiple regression analysis produced a model relating laboratory and field compaction with an R^2 of 0.82 that required four inputs: primary control sieve index (PCSI) and fine aggregate ratio (FA_c) both determined by Bailey method; surface temperature at beginning of field compaction, and the number of gyrations to reach field density for specimens compacted to field lift thicknesses.

Cooley and Williams (2009) studied ten paving projects in Mississippi (five with 9.5 mm NMAS and five with 12.5 mm NMAS). Testing included monitoring of asphalt compaction by measuring density following each roller pass, measuring temperature at four mat locations (surface, top, middle, and bottom of the layer) and obtaining compaction equipment and environmental information. A multiple regression model was developed (Eq. 2.2), and a subsequent sensitivity analysis revealed the effects on compaction shown in bulleted form below. Cooley and Williams (2009) recommended the Mississippi DOT increase their currently allowed maximum lift thickness for 12.5 mm mixes to 7.5 cm.

$$\%\,G_{mm} = 7.6e^{-5}(ACP)(P_{0.075}) + 3.4e^{-3}(T_M)(t/NMAS) + 4.2e^{-2}(V_{a(P)})(\pm\,PCS) \tag{2.2}$$

Where,

$\%\,G_{mm}$ = percent of theoretical maximum specific gravity achieved during compaction
ACP = accumulated compaction pressure of Levia and West (2008) with units of psi
$P_{0.075}$ = fines content
T_M = temperature near the middle of the asphalt layer with units of ^{o}F
t/NMAS = layer thickness divided by nominal maximum aggregate size
$V_{a(P)}$ = percent air voids in laboratory compacted specimens during production
$\pm$ PCS = percent above (+ or finer) or below (- or coarser) the primary control sieve

- ACP approximately 2% change in % G_{mm} (higher ACP, higher % G_{mm})
- $P_{0.075}$ approximately 0.2% change in % G_{mm} (higher $P_{0.075}$, higher % G_{mm})
- T_M just under 2% change in % G_{mm} (higher T_M, higher % G_{mm})
- t/NMAS approximately 3% change in % G_{mm} (higher t/NMAS, higher % G_{mm})
- $V_{a(P)}$ just over 2% change in % G_{mm} (higher $V_{a(P)}$, lower % G_{mm})
- $\pm$ PCS approximately 3.5% change in % G_{mm} (+ or finer mixes, higher % G_{mm})

Brown et al. (2004) reported that several studies have reported a t/NMAS ratio of 4 is preferred rather than the most commonly used minimum value of 3. Pavement density that can be obtained under normal rolling conditions is clearly related to the t/NMAS ratio. It was recommended that the t/NMAS ratio be at least 3 for fine-graded mixes and at least 4 for coarse-graded mixes. Ratios less than these suggested values were reported useable, but that the consequence is a greater than normal compactive effort. In most cases, a t/NMAS of 5 did not result in the need for additional compaction to obtain the desired density. Brown et al. (2004) noted that care must be exercised with lift thicknesses that become too large in the context of achieving adequate density.

Superpave mixes are often coarse graded (below restricted zone). Surveys and contractor feedback resulted in the assessment that approximately 40% of coarse-graded Superpave mixes experience some tenderness according to Brown et al. (2000). Huber et al. (2002) also reported tender mix behavior in some coarse graded Superpave mixes.

Brown et al. (2009) provides some data related to compaction times. Of pertinence to this work were plots that provided time estimates for compaction, defined as the time for the mix to cool from 120 C to 80 C for a range of base temperatures. For a 6.3 cm thick mat (the lower end of the thicknesses evaluated in this study) the compaction time ranged from 12 to 18 min, while for 7.5 cm thick mat (the upper end of the thicknesses evaluated in this study) the compaction time ranged from 15 to 23 min.

The recommended temperature of the asphalt mat where compaction should end is often referred to as the cessation temperature. Values on the order of 80 C are typically recommended. For example, Kristjansdottir (2006) cited 79 C as a reasonable cessation temperature, and PaveCool 2.4 (Chadboum et al. 1998) has 80 C as the default value for PG58 and higher binder grades. Williams et al. (2011) used PaveCool 2.4 for a field compaction project and used 80 C as the cessation temperature.

2.6 Test Methods

Pertinent information on test methods used in this study and interpretation of their results is presented in this section. Several of these test methods are widely used, so information presented was limited to that directly related to emergency paving.

2.6.1 Measurement of In Place Density

Kandhal and Koehler (1984) reported the nuclear gage cannot be relied upon completely by the user agency for acceptance purposes, rather nuclear density is most suited for contractor compaction quality control. Nuclear density values were higher in three projects, and lower in four projects compared to core densities. Eight projects were studied.

Brown et al. (2009) suggests underlying layer effects can be addressed by cutting cores and adjusting nuclear gage readings to these cores. The text also suggests overall recommendations on nuclear gages vary. The authors indicated the best use of nuclear gages was in the development of rolling patterns. Brown et al. (2009) also suggested that moisture can heavily affect non-nuclear gages.

Williams et al. (2011) studied twelve field projects (forty-four test locations) in Mississippi including five 12.5 mm mixes. The mixes included four different crushed gravel sources, one of which was used in this project. ACP as per Levia and West (2008) was plotted versus in place air voids measured by a nuclear density gauge that had been offset using cores cut from each section. A linear trendline was fit to the data and the final in place air voids were predicted with the resulting equation. Figure 2.5 plots the final in place air voids predicted by the trendline equation versus values measured on cores by T166. The values correlate very well, which is not surprising since the offset used for the *x*-axis was taken from the *y*-axis. The main point of Figure 2.5 is that when a nuclear gage density offset taken at a relatively high density was applied to mix at several densities in the form of a trendline equation the correlation, while scattered, was not highly skewed.

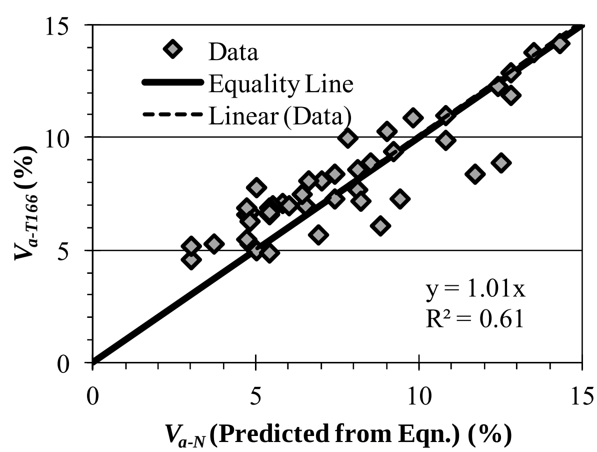


Figure 2.5. Comparison of In Place Nuclear Density and T166 (Williams et al. 2011)

2.6.2 Short Term Aging

Asphalt binder hardening occurs in two distinct stages: 1) short term aging during production, transport and placement principally due to volatilization and oxidation of the binder; and 2) long term aging after construction principally due to environmental effects (Bell 1989; Bell et al. 1994a). The effects of short term aging may be an important issue for hot-mixed and warm-compacted asphalt hauled long distances for disaster recovery. Long term aging is of less concern for disaster recovery due to the short service life anticipated.

For traditional applications, laboratory work (Bell et al. 1994b) and field validation (Bell et al. 1994a) resulted in a recommendation of short term oven aging of loose asphalt mix at 135 C for 240 minutes. This recommendation was implemented in the first iteration of Superpave (Cominsky et al. 1994).

AASHTO R30-02-2006 recommends 120 $\pm$ 5 minutes of short term aging at the compaction temperature for volumetric mix design and short term aging of 240 $\pm$ 5 minutes for mixture mechanical property testing. Current MDOT volumetric mix design requires 90 minutes of loose mix short term oven aging time at the compaction temperature (MDOT 2006).

2.6.3 Asphalt Volumetrics

Measurement of mixture bulk specific gravity (G_{mb}) and maximum mixture specific gravity (G_{mm}) was a key component of this project. G_{mb} measurement was evaluated by Howard and Doyle (2012) over a wide range of air voids using multiple methods, two of them being AASHTO T166 and T331. Eq. 2.3 summarizes the approach. Data was collected by the authors, and additional data was taken from previous research cited in literature. Table 2.5 summarizes the findings and shows that air voids calculated by T331 are generally higher than when calculated with T166.

$$V_{a-T331} = C_1\left(V_{a-T166}\right) + C_2 \tag{2.3}$$

Where,

V_{a-T331} = percent air voids measured by AASHTO T331
V_{a-T166} = percent air voids measured by AASHTO T166
C_1, C_2 = regression constants

Table 2.5. Comparison of T166 and T331 (Howard and Doyle 2012)

	Eq. 2.3 Prediction of $V_{a(T331)}$ for Varying $V_{a(T166)}$		
Source	**V_{a-T166} = 4%**	**V_{a-T166} = 7%**	**V_{a-T166} = 10%**
Literature Cited by Howard and Doyle (2012)	3.5 to 5.8	6.9 to 10.2	9.9 to 14.7
Data Collected by Howard and Doyle (2012)	4.4 to 6.0	7.8 to 9.9	11.0 to 13.7

G_{mm} measurement was evaluated by Azari (2012) where one objective was to compare results of manual and mechanical agitation. One motivation for the research was that analysis of the AMRL Proficiency Sample Program demonstrated mechanical agitation provides less test result variation than manual agitation. Results showed that G_{mm} values from manual agitation were always smaller than the highest G_{mm} values from mechanical agitation. It was reported that in most cases, manual agitation G_{mm} values were equivalent to G_{mm} produced by the mid-range intensity mechanical device settings. The difference between air voids from manual and mechanical agitation ranged from 0.2 to 0.4%. Azari (2012) reported that the use of manual agitation for G_{mm} measurement is not suggested. Vibration devices when operated at their optimum settings were concluded to produce G_{mm} values that were not statistically different.

2.6.4 Wheel Tracking

Izzo and Tahunoressi (1999) performed a Hamburg Loaded Wheel Tester (HLWT) repeatability evaluation, and also performed a comparison of slab and Superpave Gyratory Compactor (SGC) prepared specimens. It was concluded that specimens molded in the SGC could be used for moisture evaluation with the HLWT in the comparative evaluation of one material to another. A correlation could not be developed between rectangular slab specimens and SGC specimens due to variability.

Aschenbrener (1995) evaluated the HLWT on slabs measuring 25.9 cm wide, 32.0 cm long, and 4.1 cm thick with 6 to 8% air voids. Each specimen was loaded for 20,000 passes or to 20 mm of rutting. A maximum rut depth of 4 mm after 20,000 passes has been specified in Hamburg, Germany. Colorado found the specification to be severe for their pavements and proposed a rut depth of less than 10 mm after 20,000 passes. Pavements categorized as good field performers had stripping inflection points greater than 10,000 passes. High maintenance pavements had stripping inflection points between 5,000 and 10,000 passes. Poor performing pavements had stripping inflection points less than 3,000 passes. The HLWT was found to have the potential to discriminate between pavements of varying field stripping performance.

The HLWT has been adopted by the Texas DOT. Texas DOT specification criteria are defined as minimum number of passes to reach a 12.5 mm rut depth when the test is performed at 50 °C. The criterion is 10,000 and 15,000 passes for PG 64 and PG 70 binders respectively. It has been suggested that these values are too conservative and alternate criterion of 5,000 and 10,000 passes should be used for PG 64 and PG 70 binders, respectively (Rand 2006).

Brown et al. (2001) suggested an 8 mm pass/fail rut depth criteria for use with the Asphalt Pavement Analyzer (APA) for the conditions used in this study. Generally speaking, this criteria would apply to SGC compacted specimens at 7% air voids measured with T166. The PURWheel Laboratory Wheel Tracker as used in this report does not have established failure criteria as of the writing of this report.

CHAPTER 3 – EXPERIMENTAL PROGRAM

3.1 Overview of Experimental Program

The experimental program incorporated laboratory and field components. Asphalt was produced at a full scale facility, loaded into trucks (one truck was instrumented), hauled for different amounts of time, and compacted into test strips. During compaction density was monitored with non destructive gages, temperatures were monitored, and site conditions were recorded. Cores and slabs were taken from the compacted test strips and tested for density, rut resistance, and moisture susceptibility. Loose mixture was taken pre-haul and post-haul and tested for binder content, binder properties, gradation, and workability. Loose mixture was also compacted into mix design verification specimens and into specimens to be tested for rut resistance and moisture susceptibility. This chapter describes all components of the experimental program except for the instrumentation program that measured cooling rates of the asphalt during transport, which is provided in Chapter 4.

3.2 Materials Tested

Figure 1 provides the 65 gyration Superpave asphalt mix design used herein. The 12.5 mm nominal maximum aggregate size (NMAS) mixture has 36% passing the 2.36 mm sieve, which by definition makes it coarse graded. The breakpoint, however, is 39% so the mix is fairly close to the breakpoint. Three binders were used: 1) Hunt Refining Company PG 67-22 from Tuscaloosa, AL and referred to hereafter as B1; 2) Ergon Asphalt & Emulsions, Inc PG 67-22 from Vicksburg, MS and referred to hereafter as B2; and 3) Ergon Asphalt & Emulsions, Inc PG 67-22 from Vicksburg, MS dosed with 0.5% of M1 Evotherm 3G™ and referred to hereafter as B3. Hunt Refining Company typically supplies binder to the APAC Columbus facility, so it was used for preliminary testing for logistical reasons. All remaining strips were produced using binder supplied by Ergon Asphalt & Emulsions, Inc. SS1 emulsion was used as tack coat for the project at a rate of 0.32 L/m^2 (0.07 gsy).

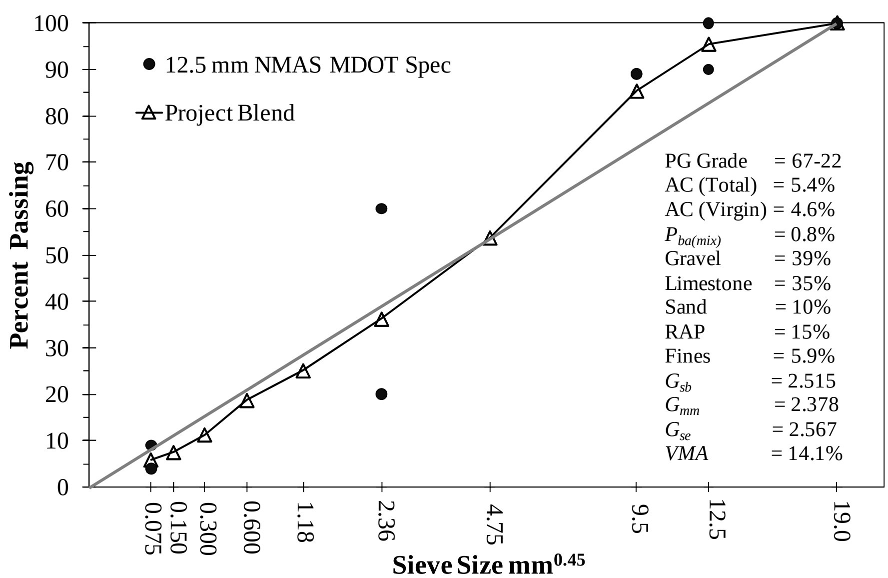


Figure 3.1. Asphalt Mix Design: MDOT MT125.08077

The Figure 3.1 mix design was produced three ways: 1) with no foam or additive to serve as the control for the study and referred to hereafter as *HMA*; 2) with foamed asphalt having 2% moisture on a binder mass basis and referred to hereafter as *Foam*; 3) with Evotherm 3G™ and referred to hereafter as *Additive*. The target mixing temperature was 160 C (320 F) for the entire study; actual values varied from 148 to 164 C. Other mixing temperatures could be used in a disaster environment on a case by case basis. The decision was made to keep mixing temperatures in line with those typically used for hot mixed asphalt applications in Mississippi. Mixing temperature increases, however, could be problematic at some point but investigation of this behavior was beyond the scope of this study.

3.3 Field Test Site

The field test site was a parking lot at the APAC Columbus facility, which is shown in Figure 3.2. The site is approximately 40 m (130 ft) square and for purposes of this project the latitude was taken as 33 degrees. The parking lot had several cracks of varying sizes (crack sizes generally ranged from 0.3 to 3.8 cm) , which were not sealed prior to paving. The cracks were cleaned, and any areas where testing would be conducted were filled with tamped sand, and the excess sand swept prior to paving the test strips. Figure 3.3 provides example photographs of the cracks present in the parking lot, and Figure 3.4 shows sand filling the cracks.

(a) Photograph of Test Site on June 30, 2011 prior to crack cleaning

Figure 3.2. Test Site Photographs

Figure 3.3. Example Photos of Cracks Present in Parking Lot

Figure 3.4. Sanded Cracks Prior to Sweeping

A crack map was developed using a TOPCON HyperLite + rover and base with Glonass Satellites and an Allegro CE data collector running Carlson Survece software. Cracks were mapped by assigning them numbers and measuring coordinates at 0.6 to 0.9 m intervals along the length of the crack. Crack size was not recorded after the decision was made to fill cracks within testing areas with sand. Removing cores and slabs only from areas without cracks was considered but was abandoned in favor of filling cracks with sand. There were too many cracks to avoid them during specimen removal in an efficient manner. Figure 3.5 shows the crack map developed with test strips and test zones superimposed onto the map for visual identification of the sections where specimens were removed.

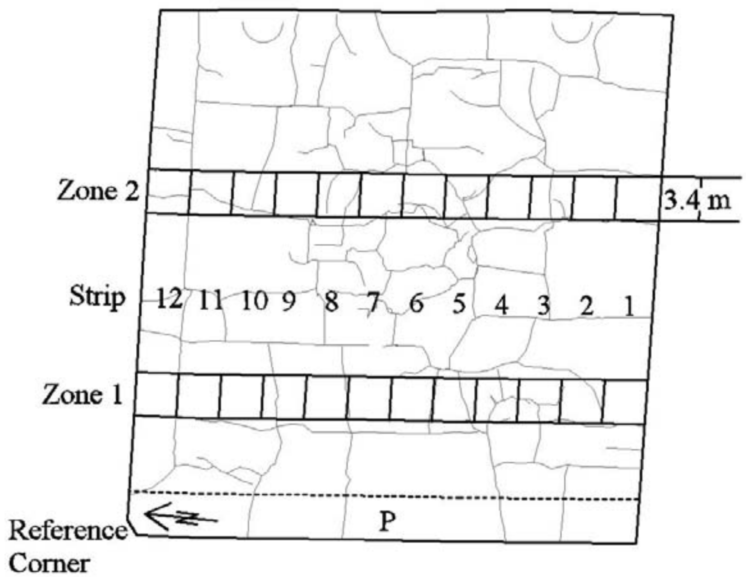


Figure 3.5. Crack Map of Parking Lot

3.4 Production and Placement Equipment

For consistency purposes, all mixture production, hauling, and placement occurred with the same equipment. Key components of the process used in this program are discussed in the remainder of this section. A material transfer vehicle (MTV) is notably absent from the test program. The decision was made not to use an MTV since the material may have difficulty passing through the MTV, especially since it would be cold as continuous mix was not placed in this project. An MTV may not be available in a disaster environment, which is another rationale for not using one in this project.

3.4.1 Asphalt Production Facility

Figure 3.6 provides photographs of the asphalt production facility. The plant is equipped with three 200 ton capacity storage silos, a 2011 model GenCore Green Ultrafoam GX2™ foaming system, a 1991 model ASTEC, Inc Double Barrel® counter flow drum with a 350 ton/hr capacity, and a PM96 aggregate blending system. The facility can be powered with recycled fuels or natural gas.

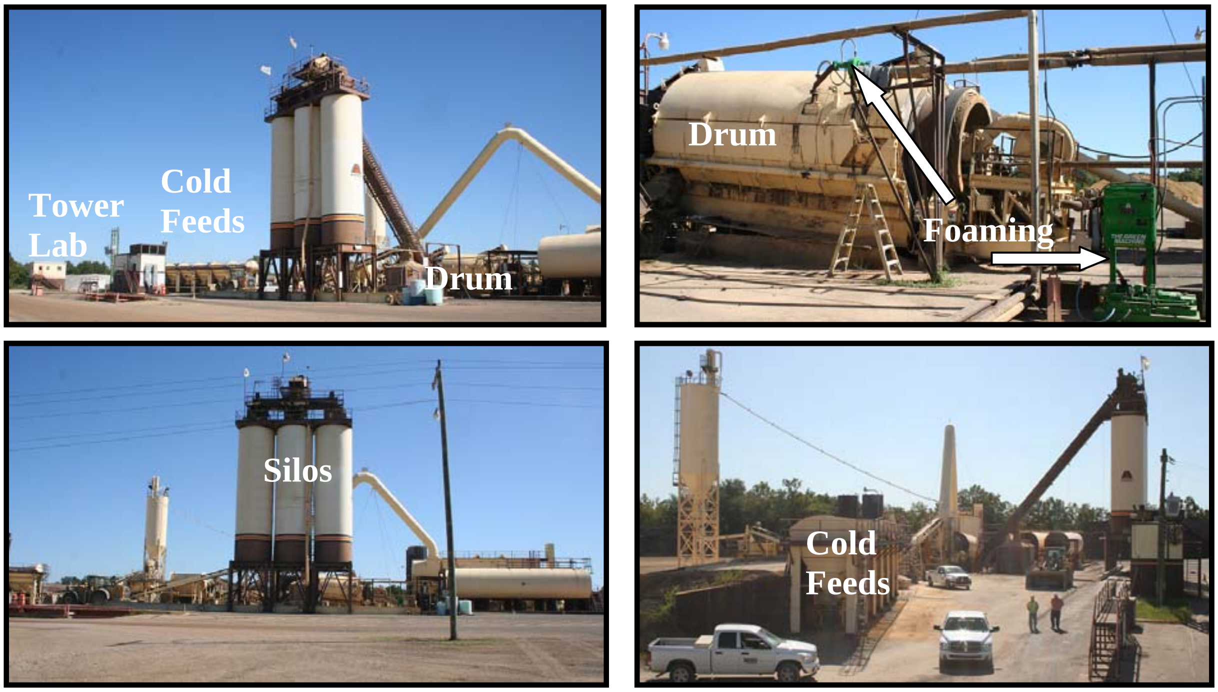


Figure 3.6. Photos of APAC Columbus Asphalt Production Facility

3.4.2 Trucks

Figure 3.7 shows the truck that was instrumented which is representative of all trucks. The trucks were not insulated since insulated trucks are not common in Mississippi, but the trucks were tarped. Two white dots are shown in Figure 3.7 to denote probe insertion locations; the probes were centered in the fifth depressed area (each depressed area was 43 cm wide) from the front of the truck bed. The Figure 3.7 truck bed was 7.32 m long, 2.44 m wide, and 1.22 m tall (measured from the bottom of the bed to the top of the wood side boards which were 0.15 m tall). The probes were placed 3.35 m from the front of the truck bed as shown in Figure 3.7. Chapter 4 provides detailed information related to the instrumentation probes.

Five trucks were used during the experiment (the Figure 3.7 truck and four additional trucks). It was attempted to use the same four trucks in the same positions within the convoy for all test days, but one truck and driver were unavailable on November 3 making it necessary to substitute a truck for this day. The effects of this change are believed to be insignificant to the results of the study.

Figure 3.7. Asphalt Hauling Truck and Corresponding Instrumentation

3.4.3 Sweeper and Tack Coat Distributor

Figure 3.8 shows the Broce Broom sweeper used to clean the surface of the parking lot just prior to paving. Also shown is the Etnyre® Black-Topper® Centennial Series asphalt distributor that was used to place the SS1 emulsion.

Figure 3.8. Sweeper and Tack Coat Distributor

3.4.4 Paver

Figure 3.9 shows the Grayhound CR461R Cedarapids paver used. The paver hopper has a capacity of 11.5 tons and the paver can hold 17 tons when all components are fully loaded. The screed width is 3.05 m to 6.10 m; note a 3.05 m screed setting typically produces a 3.20 m wide mat once compacted.

Figure 3.9. CR461R Asphalt Paver

3.4.5 Vibratory Roller

An Ingersoll Rand DD-138 HFA 13.5 tonne high-frequency vibratory asphalt compacter was used throughout the project (Figure 3.10). The DD-138 HFA is one of the more aggressive compactors used in the Mississippi market. Pneumatic compactors were not used since it is unlikely that two compactors would be available in a disaster environment as would be the case at many conventional construction projects.

The DD-138 HFA operating weight is 15,752 kg (30,325 lb), with the front drum weighing 7,202 kg (15,880 lb) and the rear drum weighing 6,551 kg (14,445 lb). The compactor is 6 m (19.7 ft) long and 2.34 m (92 in) wide. The drums are 2.14 m (84 in) wide, 1.4 m (55.1 in) diameter, and the shell is a nominal 20 mm (0.78 in) thick. The drums vibrate at a frequency of 45.0 to 66.7 Hz (2,700 to 4,000 vpm), with an amplitude range of 0.35 to 0.88 mm (0.014 to 0.035 in) and a centrifugal force range of 163 to 186 kN (36,680 to 41,720 lb) per drum. Vibration is divided into 8 settings, with each setting corresponding to a vibration frequency and centrifugal force (the higher the setting the lower the vibration frequency and the higher the centrifugal force). Both compactor drums vibrate.

Figure 3.10. DD-138 HFA Asphalt Compactor

3.5 Laboratory and Field Test Methods

3.5.1 Parking Lot Temperature and Humidity Measurement

Figure 3.11 illustrates the process of measuring air and parking lot temperatures, alongside relative humidity. A *SPER Scientific Model 800024* data logger was used with bead thermocouples to measure air temperature ($T_{Air\text{-}PL}$) at the parking lot (edge of strip 12) where the test sections were placed. The same data logger and thermocouple types were also used to measure temperature of the existing parking lot within 2 cm of the surface (T_{B-PL}) by drilling a small hole, inserting bead thermocouples, and filling the hole with bituminous material. The existing parking lot served as the base layer for the test strips, which is why this term was labeled with a subscript B for base. Four replicates were made of each measurement and temperatures were written to a file every four minutes. Relative humidity (RH) was recorded periodically with a hand held TH Pen Thermo-Hygrometer model 8708.

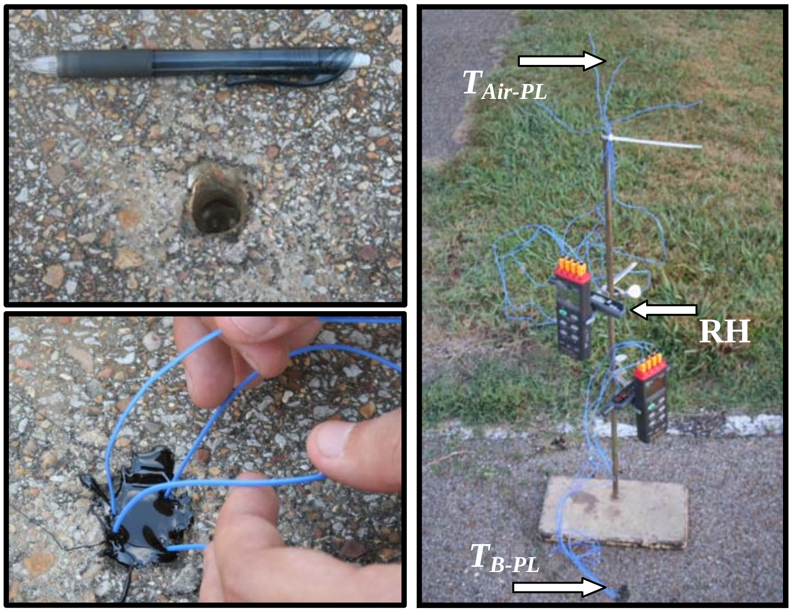


Figure 3.11. Parking Lot Temperature and Humidity Measurement

3.5.2 In Place Density Measurement

In place density was monitored during placement of the test strips to make decisions regarding applied compactive effort. Two in situ density devices were used: 1) PQI Model 301 manufactured by Trans Tech; 2) Troxler Model 3440 nuclear density gage. Figure 3.12 shows each gage measuring in situ density, alongside the symbol used to denote air voids calculated using readings taken from the gage. Figure 3.12 also shows the pattern used to collect in place density readings.

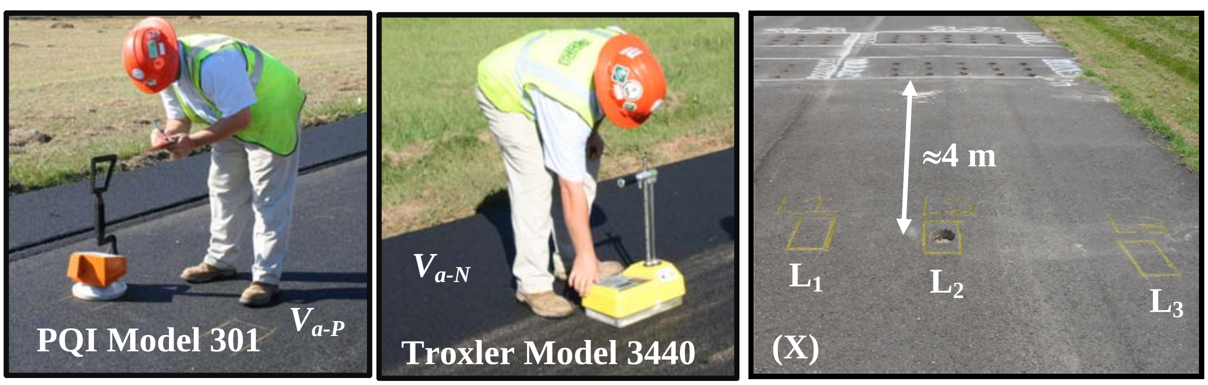


Figure 3.12. In Situ Density Measurement

Locations were marked L_1, L_2, and L_3, and they were spaced to correspond to the roller pattern used (See Section 3.6). For strip P, these locations are shown in Figure 3.22, and for strips 1 to 12 they were approximately 4 m from the edge of Test Zone 2 (defined in Figure 3.19 and illustrated in Figure 3.12). For strips 1 to 12, density at L_1, L_2, and L_3 were monitored during construction and they were measured again approximately one month after compaction (no traffic occurred during this period). L_2 was cored after the measurements taken one month after compaction (L_2 has been cored in Figure 3.12) for comparison of in place and laboratory measured air voids.

3.5.3 In Place Asphalt Temperature Measurements

Surface temperature (T_s) was measured with a hand held infrared device (Gilson MA-372) as shown in Figure 3.13a. Immediately behind the screed three small paint dots were made evenly across the width of the asphalt mat in zone 2 near the in place density measurements and temperature was measured just to one side of the paint dot; these values were averaged and reported. Generally speaking, these same three spots were recorded after each roller pass made on either side of the test strip and the results were plotted versus time after the mix passed under the paver screed (t_{Screed}) at the location of the paint dots. The surface temperature at t_{Screed} of 0 was referred to as T_{Screed} (i.e. $T_s = T_{Screed}$ at t_{Screed} = 0 min).

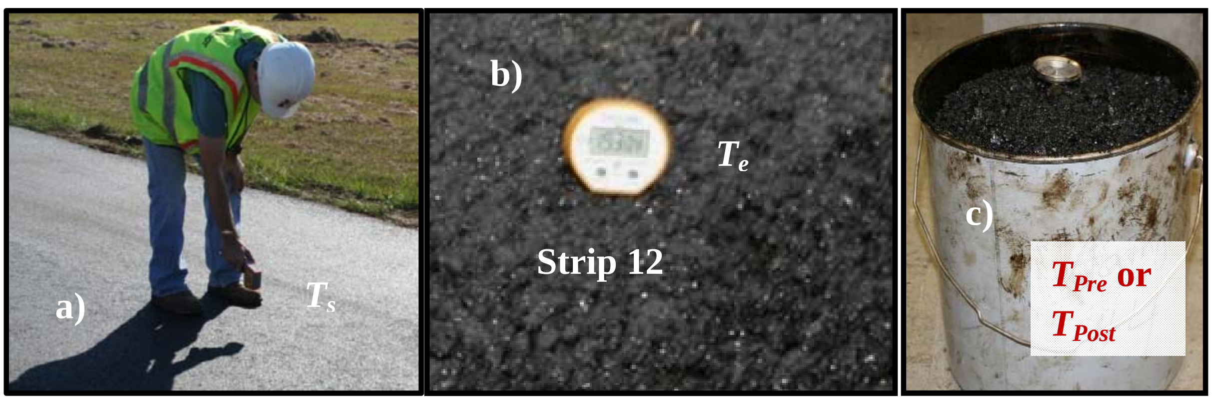


Figure 3.13. In Place Asphalt Temperature Measurement

A digital thermometer was also placed a few centimeters into test strips 11 and 12 in zone 1. Periodic temperature measurements were taken with this device (Figure 3.13b). These measurements were referred to as embedded temperature (T_e).

Rochester asphalt thermometers (24 cm long) were used to measure temperature in buckets full of sampled asphalt mix. Immediately after sampling, thermometers were pushed fully into the bucket and once equilibrated, the temperature was recorded. Temperatures recorded in buckets of pre-haul and post-haul mix were referred to as T_{Pre} and T_{Post}, respectively. Figure 3.13c shows an example of this temperature measurement approach. T_{Pre} was taken as the mixing or plant production temperature. For all trucks but the one instrumented, T_{Pre} would have been taken approximately 5 minutes after loading. For the instrumented truck, T_{Pre} would have been approximately 15 minutes after loading and was usually just before the trucks left the plant.

3.5.4 Superpave Gyratory Compactor

Two Pine Model AFG1A Superpave Gyratory Compactors (SGCs) were used alongside three calibrated molds for most compaction that were set to a 1.25 degree external angle (Figure 3.14). Loose mix was sampled during production and 150 mm diameter specimens were compacted at the asphalt production facility. A few specimens were compacted at a laboratory other than the production facility using a Pine AFGC125XA.

Figure 3.14. Superpave Gyratory Compactor

3.5.5 Linear Asphalt Compactor

Slabs that were approximately 8 cm thick were compacted in the Linear Asphalt Compactor (LAC), which was located approximately 40 km from the asphalt production facility (Figure 3.15). Asphalt was sampled from the silo with a front end loader and metal buckets were subsequently filled that were transported to the LAC while wrapped in blankets. Once in the laboratory with the LAC, the buckets were placed in an oven at 146 C and held until they were compacted. Slabs were compacted over a period of several hours, with the time from exiting the silo recorded. Compaction parameters were a 138 C temperature, 18 passes, and a 2,413 kPa system pressure. This compactor is not nearly as common as the SGC; additional operational details are provided in Doyle and Howard (2011), while Howard et al. (2012) provides an evaluation of the compactor.

Figure 3.15. Linear Asphalt Compactor

3.5.6 Asphalt Pavement Analyzer and Hamburg Loaded Wheel Testing

Asphalt Pavement Analyzer (APA) testing was performed to 8,000 cycles at a 64 C test temperature, with a 445 N vertical load, and with a hose contacting the specimen with an internal pressure of 690 kPa. Hamburg Loaded Wheel Testing (HLWT) was performed to 20,000 passes at a 50 C test temperature, with a 705 N vertical load, and with solid metal wheels contacting the specimen. Two specimens that were either SGC or field compacted (FC) with similar air voids were placed in one track for APA and HLWT. Both tests are performed in the equipment shown in Figure 3.16 by changing out the hose rack and solid wheels. Specimens that were thinner than the target specimen dimensions (63.5 mm for HLWT and 75 mm for APA) were tested with Plaster of Paris under the specimens so they would conform properly to the molds. Specimens that were thicker than the target dimensions were trimmed on the bottom so they would conform to the target dimensions. APA cores should be at least 50 mm tall according to AASHTO T340.

Figure 3.16. APA and HLWT Test Equipment

3.5.7 PURWheel Laboratory Wheel Testing

The original PURWheel was developed in the 1990's at Purdue University. It was donated to Mississippi State University in 2007, where it was renovated, modified, and placed into operation alongside the protocols described in Howard et al. (2010). The renovations and modifications relative to the original equipment and protocols are also found in Howard et al. (2010). Two protocols were used: 1) PURWheel-dry, dry specimen tested in 64 C air; and 2) PURWheel-wet, specimen tested submerged in 64 C water.

LAC slabs were tested in the PURWheel, alongside slabs sawn from the field compacted test strips (wheel tracking was parallel to the direction of compaction). Tested slabs are approximately 30 cm square. Laboratory specimens are 7.6 $\pm$ 0.6 cm thick, and 73% of the field specimens tested were within this tolerance. The thinnest field specimen tested was 6.2 cm, with 21% of the specimens tested being thinner than 7.0 cm. Specimens thicker than 8.2 cm occurred 2% of the time, with the thickest specimen tested being 8.5 cm.

Figure 3.17 shows key PURWheel components. Figure 3.17a shows an overall view of the hood/tank assembly, control box, and computer powered by *Hawk* software. Two independently controlled wheel carriages mounted with 4-ply pneumatic tires load the slabs for 20,000 passes. Figure 3.17b is a slab plastered into the mold and ready to be tracked by one of the wheel carriages. Figure 3.17c is an expanded view of a carriage with the pneumatic wheel retracted after a completed test. The tire inflation pressure is 862 kPa, the wheel load is 1,750 N, and the contact pressure at the beginning of the test is ≈ 630 kPa.

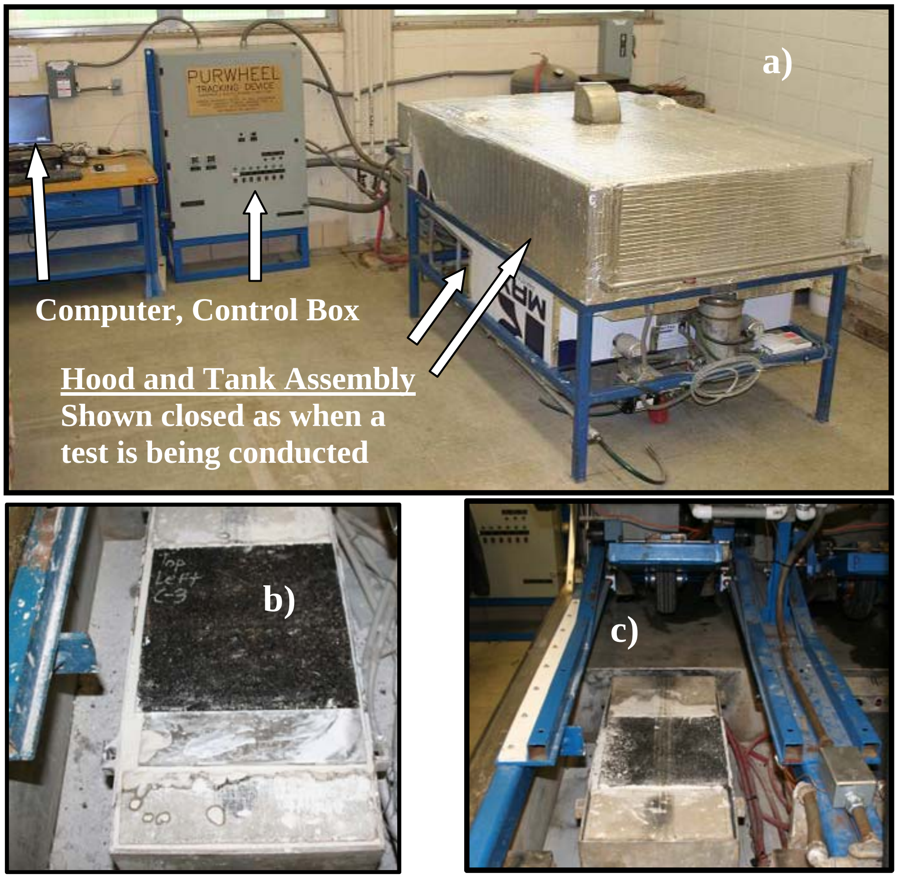


Figure 3.17. PURWheel Laboratory Wheel Tracker

3.5.8 Workability Testing

Workability was measured using a prototype known as the Asphalt Workability Device (AWD) that was developed at the University of Massachusetts Dartmouth (Figure 3.18). The AWD operates on torque measurement principles established in Marvillet and Bougalt (1979) and Gudimettla et al. (2003). The AWD rotates loose asphalt mixture at a constant speed (15 rpm for this study) in a bucket and separately records the resultant torque exerted on a pug mill style paddle shaft embedded into the mixture. Concurrently the surface and internal temperatures of the mixture are recorded. As the mixture cools in ambient conditions, the torque exerted on the shaft increases thereby giving an indication of the workability of the mixture at different temperatures.

Based on the raw torque versus temperature data collected from the AWD, a best fit exponential model is fit to the data and plotted versus the actual temperature range where the torque data was collected. The mixtures tested were placed in an oven set at 149 C until they were loose enough to split to the appropriate test size. The split specimen was then heated to 149 C, placed in the AWD, and the test commenced (testing terminated at approximately 88 C). Mixtures with lower torque values are considered more workable.

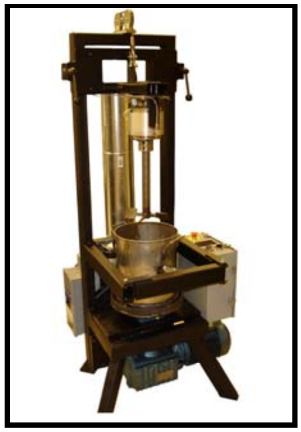


Figure 3.18. Asphalt Workability Device (AWD)

3.5.9 Asphalt Volumetrics and Binder Grading

A series of tests were performed to measure asphalt volumetrics and to grade asphalt binders. Some of these tests were performed by groups other than MSU, and when noteworthy the laboratory who performed the test was indicated. If not indicated, the test was likely performed by MSU, or it was not relevant where the test was performed.

Bulk specific gravity of compacted asphalt mixes (G_{mb}) was measured on each core and SGC specimen using AASHTO T331 (CoreLok®) and AASHTO T166 (SSD). Generally speaking, the CoreDry® was used to dry cores prior to testing. All cores were verified to be at constant mass (i.e. no moisture) prior to measuring G_{mb}.

Maximum mixture specific gravity (G_{mm}) was performed according to AASHTO T209. Several laboratories performed G_{mm} tests as it is a critical parameter. MSU performed T209-10 under vacuum and mechanical agitation (Method A), with just over 1,500 g of mix placed into the container to be agitated. Paragon Technical Services, Inc. (PTSi) measured G_{mm} with AASHTO T 209-11 (Method B). The dry back approach (T209-10 Section 15) was also used in some cases to determine if water was entering the aggregate pores.

Air voids (V_a) were calculated using several combinations of G_{mb} and G_{mm}. The four most often used combinations were: $V_{a\text{-}T166\text{-}Pre}$ (G_{mb} measured using T166 and G_{mm} measured on mix sampled from truck prior to hauling); $V_{a\text{-}T166\text{-}Post}$ (G_{mb} measured using T166 and G_{mm} measured on mix sampled from paver after hauling); $V_{a\text{-}T331\text{-}Pre}$ (G_{mb} measured using T331 and G_{mm} measured on mix sampled from truck prior to hauling); $V_{a\text{-}T331\text{-}Post}$ (G_{mb} measured using T331 and G_{mm} measured on mix sampled from paver after hauling). Other combinations or special designations were used on a limited basis, but they follow the same general form and are explained as needed when used.

Asphalt content (AC) (AASHTO T164), Asphalt extraction (AASHTO T164), asphalt recovery (AASHTO R59), binder grading (AASHTO R29), and extracted gradation (AASHTO T30) were performed by PTSi. The extraction solvent was 85% toluene and 15% ethanol, and approximately 450 grams of binder were recovered from each test strip. Five solvent washes were performed the majority of the time. Burns Cooley Dennis, Inc. (BCD) measured AC by extraction using AASHTO T164 (or ASTM D2172 Method A) using trichloroethylene (TCE). BCD and PTSi also measured AC using the ignition method (AASHTO T308 or ASTM D6307). Neither laboratory applied a correction factor to the measured value. APAC Mississippi, Inc. measured AC using the nuclear approach (AASHTO T287) with a Troxler model 3241-C gage.

3.6 Mixture Production and Placement

The thirteen test strips were produced on four days (Figure 3.19), and enough mixture was produced and placed in a silo to allow all trucks to be loaded consecutively. Mix was in the silo for an estimated 30 minutes before loading; some of the mix would have been in the process of production during this time. Each morning, mix was initially produced that was not used in test strips to allow the plant to reach equilibrium and ensure the proper binder was fully incorporated into the mixture. For Strip P, 100 tons of mix were produced prior to making the material tested, and 50 tons were produced for strips 1 to 12 prior to making the material tested. Test Strip P (for preliminary strip) was produced with the plant's normal binder source (B1 from Hunt Refining Company) so no modifications were needed. Strip P served to collect preliminary information (e.g. offsets for in place density measurement) and to provide a platform for placing the remaining test strips. To accommodate use of B2 (test strips 1 to 8) and B3 (test strips 9 to 12), the following steps were taken.

Just before test strips 1 to 8 were scheduled for placement, one of the plant's binder tanks was emptied. The morning when test strips 1 to 4 were placed, one tanker of B2 was delivered and placed into this tanker. The tanker contained enough binder for test strips 1 to 8 and the extra 50 tons produced prior to that used for test strips. Test strips 1 to 4 were placed November 1 and test strips 5 to 8 were placed November 2 so binder from another tank was used for normal plant operations on November 1, but after all test strip mixture was made on November 2, all remaining B2 was used to empty the tank and allow one tanker of B3 to be delivered the morning of November 3 to complete all testing.

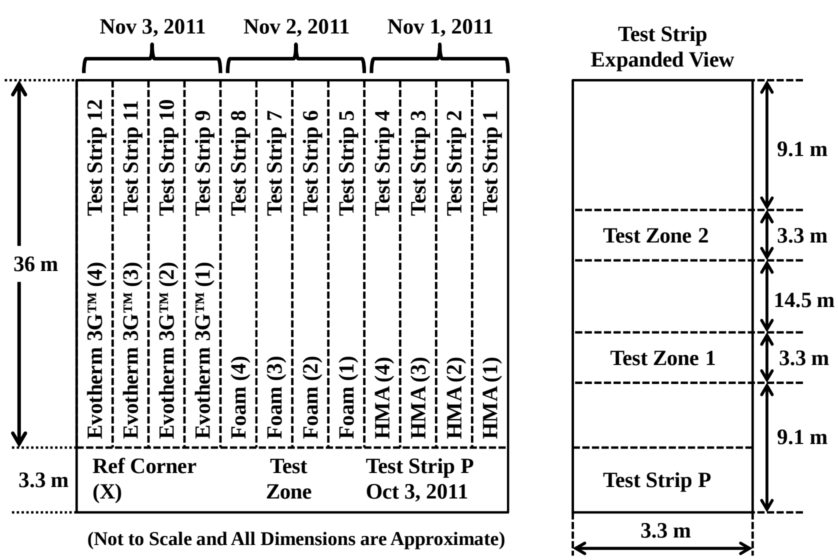


Figure 3.19. Test Strip Placement Configuration

Trucks were loaded at the APAC Columbus facility (all trucks were loaded within ten minutes of each other), and once all trucks had been loaded they traveled in a convoy. The trucks headed to US Hwy 82 (approximately 5.7 km from plant) and then headed eastbound from the Main Street-Downtown Columbus exit to the Stokes RD-New Hope exit. The distance between these exits on US Hwy 82 is 15 km. The trucks traveled 89 to 105 km/hr (55 to 65 mph) while on US Hwy 82. All trucks traveled in a loop between the two exits on US Hwy 82 (one loop would be approximately 31 km including exiting to turn around) until sufficient time had passed for one of the trucks to be compacted. At this time, all trucks came to the plant, one truck was unloaded and the asphalt compacted, and the remaining trucks resumed the aforementioned route. The instrumented truck was the first truck loaded and the last truck compacted.

The time when a truck was loaded and pulled out from the plant silo was defined as t_L. The time a truck had been loaded was defined as t_{Load}. For example, strip P was loaded at t_L of 8:02 AM (t_{Load} equals 0 min at this point), and at 9:02 AM t_{Load} was 60 min. The time when the truck left the plant was defined t_{Lp}, and the time when mix was first dumped into the paver was defined t_{bp}, or the time to begin paving. The haul time (t_h) was defined as t_{bp} minus t_L. The time when the strip was fully placed but no compaction had occurred was defined t_{ep}.

Figure 3.19 shows parking lot once placed. Each of the thirteen test strips was produced with one truck load of mix weighing approximately 24 tons. The target thickness for each test strip was 6.3 to 7.5 cm (5 to 6 times the NMAS), which is thicker than MDOT allows for this mixture in traditional applications (5 times the NMAS) as outlined in MDOT Supplement to Special Provision No. 907-401-2 dated June 2009. In an emergency environment a thicker layer might be needed to carry the relatively heavy emergency vehicles into the disaster area and being able to pave in a single lift is more time efficient.

Figure 3.20 illustrates the roller pattern used to compact the test strips. The roller pattern was developed to allow the amount of compaction received to be quantified with reasonable certainty and to have compaction occurring within the test zones while the rollers moved parallel to the traffic direction. The coring and sawing pattern presented later in this chapter was developed alongside the Figure 3.20 roller pattern and the Figure 3.12 in place density measurement pattern to ensure reasonable and uniform test strip comparisons.

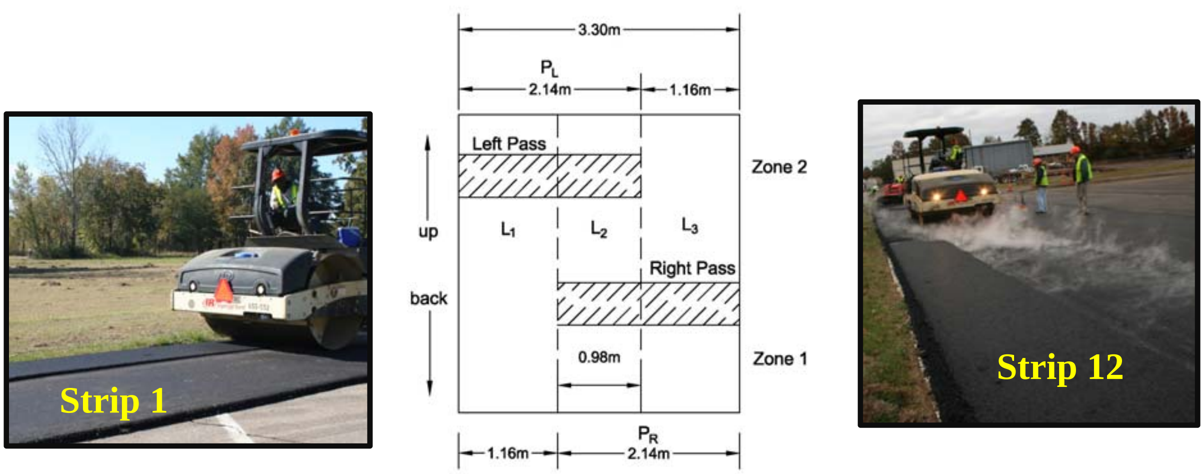


Figure 3.20. Roller Pattern Used For Compaction (All Dimensions Are Approximate)

The 2.14 m wide roller always traveled down one side of the strip (left side or right side as shown in Figure 3.20); the roller never traveled down the middle of the strip (i.e. with material on either side of the roller not being compacted). The roller moved up or back on the left or right side of the strip. When the roller made a pass on the left side of the strip (i.e. when the 1.16 m wide area of the strip not being compacted was to the right of the roller) this was referred to as a left pass (P_L). When the roller made a pass on the right side of the strip (i.e. when the 1.16 m wide area of the strip not being compacted was to the left of the roller) this was referred to as a right pass (P_R). One left pass and one right pass were defined as a full pass (P_F), which was by definition for this project when the entire strip had been crossed one time (due to the pattern one full pass crossed a 0.98 m wide area in the center of the strip twice). For example, if the roller went up on the left (one P_L) and back on the right (one P_R) this would be the equivalent of one full pass (P_F) as the entire strip was passed over by the roller one time and the 0.98 m overlap zone was passed over twice.

Each left pass (P_L) or right pass (P_R) was either all vibratory, all static, or static in Zone 1 and vibratory in Zone 2. The 14.5 m distance between Zone 1 and Zone 2 (Figure 3.19) was ample to transition from static to vibration of the 6 m long roller and still ensure vibration was occurring at the Figure 3.12 in place density measurement locations. In each test zone, the total number of passes (N_F) was recorded (i.e. the total number of full passes (P_F) that occurred), alongside the number of vibratory passes (N_V) and static passes (N_S). N_V plus N_S equals N_T; each test zone in a test strip had the same N_T, but differing N_V and N_S values to assess the relative effect of static and vibratory compaction on these mixes.

Test Zone 1 received a fixed number of vibratory roller passes ($N_V = 3$) and a variable number of static passes ($N_S = 3$ to 5). Test Zone 2 received a variable number of vibratory roller passes ($N_V = 4$ to 6) and a fixed number of static passes ($N_S = 2$). Compaction was monitored in Test Zone 2 as discussed in Section 3.5.2 and compaction continued until the gages indicated traditional density requirements had been met or that the mix could not be further densified. The decision to end compaction was made entirely using information in Zone 2; information used was a combination of PQI results, nuclear density results, and subjective evaluation made by individuals at the test site. Occasionally a strip was backrolled with one static pass two to four hours after the original compaction.

The DD-138 HFA roller, when in vibratory mode, used setting 6 (vibration frequency of 2,800 vpm and a 183.1 kN [41,170 lb] centrifugal force). These settings were based on experience of the paving contractor as they would be during a disaster response situation. Typical mainline work using the Figure 3.1 mix and the DD-138 HFA compactor consists of 5 passes (4 vibratory, 1 static). Both drums were vibrating for vibratory passes.

Figure 3.21 is an aerial view of the parking lot at the conclusion of paving. Each test strip is numbered the same as in Figure 3.19. Test zones 1 and 2 were painted on each strip prior to core and slab removal; zone 1 was painted orange and zone 2 was painted white.

Figure 3.21. Aerial View of Fully Paved Parking Lot

3.7 Loose Mixture Sampling

Loose plant mixed asphalt was sampled from either the truck or plant silo prior to hauling (referred to hereafter as pre-haul or pre) and also out of the paver after being hauled (referred to hereafter as post-haul or post). Several buckets were needed from strip P so a front end loader sampled material from the silo pre-haul. One bucket of mix was taken from strip P post-haul. One or two buckets were taken per strip for strips 1 to 12 pre-haul, and five buckets were taken from each of these strips post-haul. Some of the materials sampled were compacted prior to cooling in the *SGC* or *LAC*, while the remaining material was cooled and used for other testing (e.g. G_{mm}, binder grading, extracted aggregate gradation).

3.8 Laboratory Compacted Asphalt Mixture

A total of one hundred-seventy eight *SGC* specimens and six *LAC* specimens were produced during this project as described in the following sections.

3.8.1 Laboratory Mixed and Laboratory Compacted Asphalt Mixture

Laboratory specimens were prepared prior to field testing using oven dried project aggregates separated into multiple sieve sizes and re-combined to the mix design gradation using binders B2 and B3. *HMA* specimens were mixed at 160 C, short term aged for 4 hr at the compaction temperature of 146 C, and compacted to height in the *SGC* to produce desired air void levels. *Additive* specimens were mixed at 129 C, short term aged for 4 hr at the compaction temperature of 118 C, and compacted to height in the *SGC* to produce desired air void levels. Mixing was performed at 129 C to assess the effect of mixing temperature for reference purposes. Twenty-four *SGC* specimens were prepared with laboratory mixed (*LM*) material; half were for APA testing and the other half were for HLWT testing.

3.8.2 Plant Mixed and Laboratory Compacted Asphalt Mixture

3.8.2.1 Strip P Plant Mixed and Laboratory Compacted Asphalt Mixture

Thirty-four *SGC* specimens were produced from pre-haul plant mixed (*PM*) strip P material. Two of these specimens were compacted to 65 gyrations (110 to 120 mm height) for mix design verification. The remaining thirty-two specimens were compacted to varying air void levels. Half of these specimens were tested in the APA and the other half in the HLWT to evaluate the effect of air voids on test results.

Six *LAC* slabs were also produced from strip P pre-haul material; each slab was sawn in half to produce two approximately 30 cm square specimens. The pre-haul mix was stored in an oven at 146 C and slabs were compacted beginning at t_{Load} of 150 minutes and ending at t_{Load} of 540 minutes. Slabs were reasonably uniformly spaced within this time interval.

3.8.2.2 Strips 1 to 12 Plant Mixed and Laboratory Compacted Asphalt Mixture

One hundred-twenty *SGC* specimens (ten per strip) were compacted from strips 1 to 12. Two specimens were compacted per strip from pre-haul mix to 110 to 120 mm height using 65 gyrations for mix design verification. Eight specimens were compacted per strip using post-haul material targeting 6 to 8% air voids by compacting to a prescribed height; four of these specimens were for APA testing, and the other four were for HLWT testing.

Half of the post-haul specimens (four) were made with each *SGC* available. The required specimen mass was first batched from a bucket of post-haul mix, and thereafter the material was placed in an oven set to the temperature recorded from on a thermometer placed in the bucket immediately after sampling from the paver. After a few minutes of heating to re-gain temperature lost during batching, the specimens were compacted.

3.9 Coring and Sawing Plant Produced and Field Compacted Asphalt

Seven hundred-fifty four cores and over two-hundred slabs were taken from the thirteen test strips. All cores were used to evaluate in place density of the test strips, while some of these cores were also used to evaluate mixture stability in the APA and HLWT. Slabs were taken for two purposes: 1) PURWheel testing; and 2) to serve as residual material in the event more testing was desired.

3.9.1 Coring and Sawing Test Strip P

Twelve cores and sixteen slabs each approximately 30 cm square were taken from the strip P field compacted (*FC*) material (Figure 3.22). Two cores were taken from each location where in place density was measured according to Section 3.5.2. These locations were marked with yellow chalk and were identified as shown in Figure 3.22 (e.g. 2-3).

Figure 3.22 also illustrates the process of sawing and removing the field compacted slabs (the same general approach was also used for strips 1 to 12). Each strip P slab was marked with P-FS (interpreted strip P field compacted slab), a slab number, and an arrow indicating the direction of compaction. A removal tool was fabricated for driving under a slab after sawing to break the tack coat bond between layers and to fully support the slab during removal. When removing a slab, the slab removal tool was inclined slightly to ensure the front edge could get underneath the slab, and the tool was struck modestly (high impact blows were avoided) on the circular portion just below the handle.

Cuts were first made in the transverse direction. Approximately 30 cm was measured from the pavement edge at two locations, and a straightedge used to mark the first longitudinal cut to remove the excess material that was not tested. The straight edge remaining after removing the excess material was used to mark the next row of slabs, and thereafter the second longitudinal cut was made. Prior to slab removal, each slab in the row was marked with the slab number and an arrow indicating the compaction direction. The row of slabs was then removed, and the process of removing rows of slabs was repeated until all slabs were obtained.

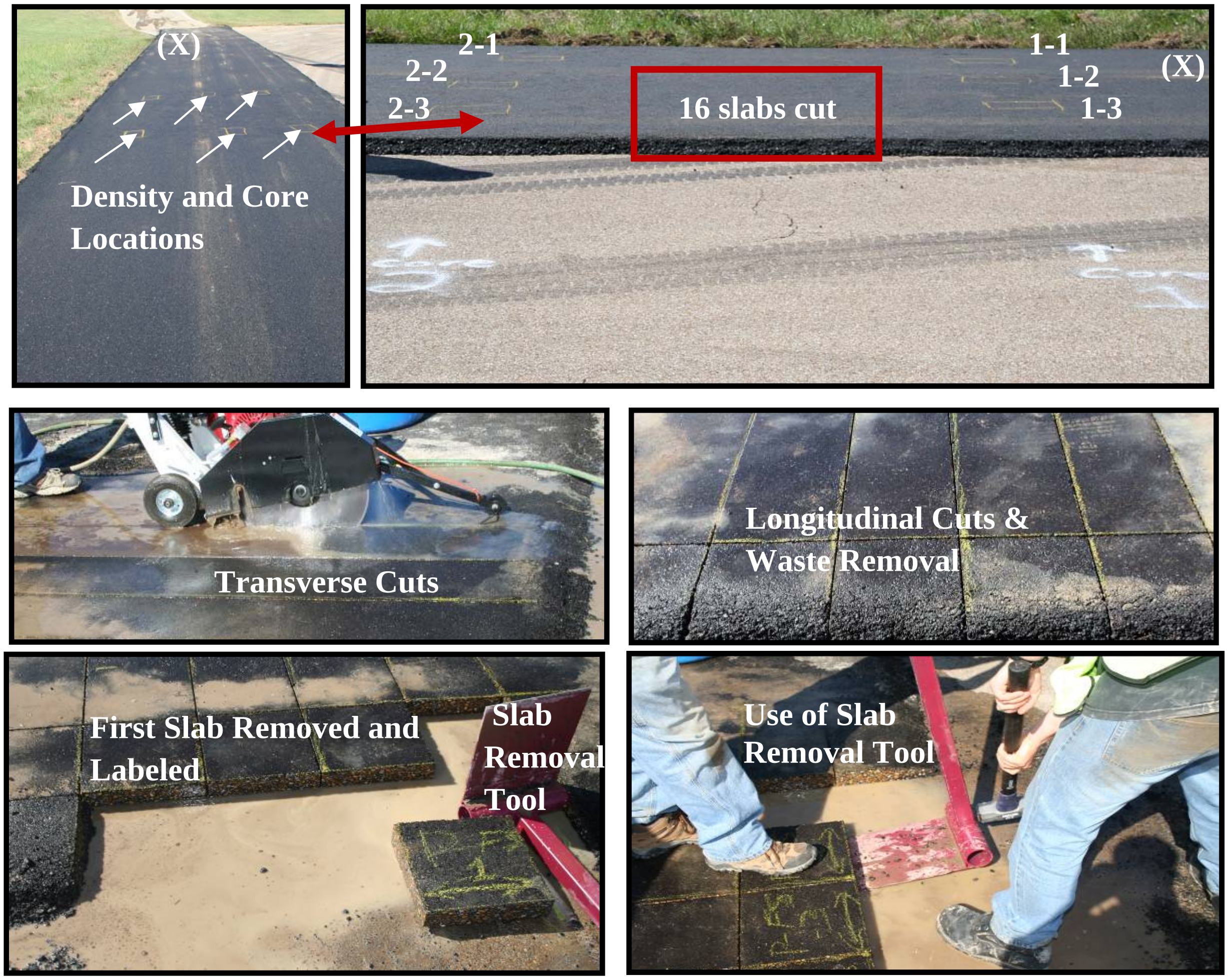


Figure 3.22. Strip P Core and Slab Removal Layout

3.9.2 Coring and Sawing Test Zones in Strips 1 to 12

Figure 3.23 illustrates the process used to mark and remove 15 cm diameter core specimens (twenty-nine to thirty-four cores were taken in each of the twenty-four test zones). Each core was marked with a strip, zone, and core number. For example, S1-Z2-C3 is a core taken from strip 1 in test zone 2 and is labeled as core 3 in the pattern described in the following paragraph. Cores were removed with a tool fabricated by welding a used core bit to a metal pipe and removing a semi-circular area sawn off one side of the bit. The tool allowed cores to be dislodged without applying considerable forces to the core while breaking the bond between the tack coat and supporting material.

Strips were marked so that all cores were taken away from the edges, which were marked with paint swirls (Figure 3.23). An area approximately 240 cm wide (transverse direction or direction perpendicular to compaction) and 120 cm long (longitudinal direction or direction parallel to compaction) was then marked in a grid pattern that was used to consistently saw cores between test strips. Within this 240 by 120 cm area, fifteen cores were sawn; the grid was marked so that twenty 30 by 30 cm areas were available with 15 cm spaces between them in the 240 cm direction and no space between them in the 120 cm direction. The amount of space between the painted edges and the 30 cm square areas where cores were taken varied, generally speaking, from 5 to 15 cm.

Fifteen of these twenty 30 cm square areas were numbered 1 to 15 beginning nearest the reference corner (X) and having three rows from left to right (1 to 5, 6 to 10, and 11 to 15, respectively) so that core 15 was the farthest from the reference corner. A fourth row of five 30 cm square areas were left un-numbered (they were used to replace a numbered core in the event of damage during sawing or removal). Cores taken from the fourth row were noted with an "a" next to the core location; e.g. C6a would denote a core taken from the fourth row to replace C6. The remaining fifteen cores (C 16 to C30) were taken in the same pattern as C1 to C15 after leaving a 90 cm space in the longitudinal direction, which was the space where slabs were sawn as described in the remainder of this section.

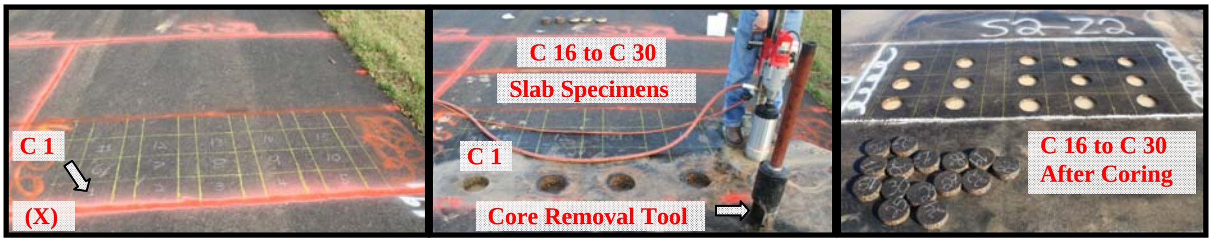


Figure 3.23. Marking and Removing Cores

Figure 3.24 illustrates the sawing and removal pattern within the 90 cm longitudinal direction space allocated for slabs. Sawing and removal practices followed the approach presented in Figure 3.22. Each 30 cm square slab was marked with a strip and slab number, alongside an arrow indicating the direction of compaction. For example, S1-5 is a slab taken from strip 1 and is slab 5 in the Figure 3.24 pattern. The area for cutting slabs was marked using the edge boundaries from coring that are shown in Figure 3.23. The outer marks between the painted edges indicating waste material and the first 30 cm square area used for core removal were also used as the outer marks for slab removal, making the length in the transverse direction available for sawing slabs variable.

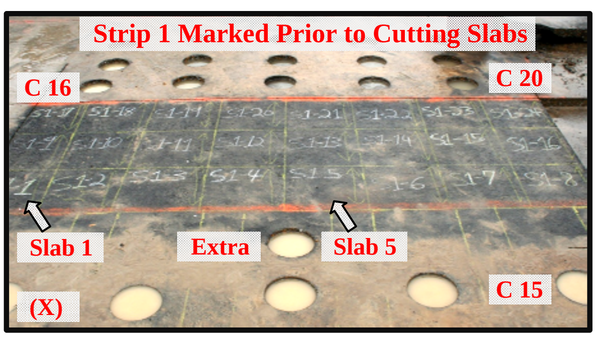


Figure 3.24. Sawing and Removal of Slabs

Up to twenty-four slabs were obtained per test strip and all slabs were taken from zone 1. A relatively small number of slabs were damaged during removal that were discarded, and for some strips only twenty-one slabs were taken due to the amount of space between painted edges that was discussed earlier in this section. It is estimated that 250 slabs were taken. An accurate count of slabs taken was not necessary since the number taken was approximately double that which was required for the project. The manner in which slabs were removed, however, made it essentially the same effort to take all the useable slabs within the 90 cm zone as opposed to only some of them so the researchers took all the slabs in the event they could meet an unforeseen need later in the project.

Cores from test zones 1 and 2 were removed November 9th through the 19th, and slabs were removed November 21 to December 2. The thickness (t) of each core and slab removed was measured with a caliper and recorded. This value was used to determine t/NMAS, which is often considered when evaluating compactability of asphalt pavements. Temperatures were generally well above freezing from placement through specimen removal. On a few occasions, temperature recorded by a local weather outlet dipped slightly below freezing for a brief period in the night, but average temperatures on those days were well above freezing which would have kept the ground and pavement temperatures elevated. Figure 3.25 shows the final condition of each test zone.

Figure 3.25. Test Zones at the Conclusion of Specimen Removal

3.9.3 Coring Strip 1 to 12 Density Locations

Twelve cores were removed (one per strip at density measurement location L2 as seen in Figure 3.12). These cores were removed December 2, 2011. The purpose of these cores was to compare final in place density measured by the two in situ density devices to values measured on cores at the same location. These cores were not cut for approximately one month to allow re-testing by the in situ density devices some time after initial testing. Since no traffic was allowed on the locations, the in situ density shouldn't have changed.

CHAPTER 4 – MIX TEMPERATURES DURING TRANSPORT

4.1 Overview of Mix Temperatures During Transport

A key component of the research was to develop an instrumentation system capable of measuring temperature within an asphalt truck at multiple locations in real time. Full scale measurement of this nature has not been successfully performed to the knowledge of the authors. A complete system had to be designed, built, verified, and used at full scale. This chapter provides information that would allow others to build similar systems.

Generally speaking, two metal probes were pushed into the asphalt mix on the vehicle's passenger side that were subsequently opened to expose a series of thermocouples directly to the asphalt inside the truck. The probes reach approximately half way into the truck and allow temperature measurement from the center of the mix to the edge of the truck bed. Temperature was monitored in the truck cab in real time by a data acquisition system. A member of the research team rode in the truck and provided updates to the rest of the research team who remained at the parking lot.

4.2 Materials Used to Fabricate Instrumentation

Table 4.1 provides all major parts needed to build the data acquisition system used on the trucks. ID's 1 to 4 are the data acquisition system, which is shown in Figure 4.1 inside the box fabricated to allow placement in the truck cab floor during testing. All wires were securely fastened to minimize problems with connections loosening during testing. ID's 5 to 7 were tools purchased to allow the remaining components to be assembled. ID's 8 to 14 were used to build the quick connections between the trunk line cable (ID 15) and the probe (Figure 4.2) inserted into the asphalt (ID 17) fitted with thermocouples (ID 16).

Table 4.1. Major Components of Instrumented Truck System

ID	Supplier[a]	Model	Description
1	Dell	E5520	Notebook computer controlling data acquisition
2	NI[a]	LabView™	Software used to write data acquisition code
3	NI	NI 9211	Thermocouple analog input module
4	NI	NI 9172	CompaqDaq chassis
5	Omega	MTC-IT	Insertion tool for connector pins and sockets
6	Omega	MTC-RT	Removal tool for connector pins and sockets
7	Omega	MTC-CT	Crimping tool
8	Omega	MTC-24-MC	In line male cord connector 24 cavities
9	Omega	MTC-24-FC	In line female cord connector 24 cavities
10	Omega	MTC-24-SHL	Backshell cable clamp (connection support)
11	Omega	HPC-CU-P	Hollow pins (male) Copper (+) stamped
12	Omega	HPC-CO-P	Hollow pins (male) Constantan (-) stamped
13	Omega	MTC-CU-S	Sockets (female) Copper (+) machined
14	Omega	MTC-CO-S	Sockets (female) Constantan (-) machined
15	Omega	8TX20PP	8 Pair Thermocouple Extension Cable (12.2 m long)
16	Omega	TC-TT-T-20-72	Bead type-T PFA insulated thermocouples (182.9 cm leads)
17	Gilson	GP-200	D-tube sampler used as thermocouple probe

a: NI = National Instruments™ Omega = Omega Engineering, Inc. Gilson = Gilson Company, Inc.
Note that HPC-CU-S and HPC-CO-S sockets were not durable enough to allow repeated connections.

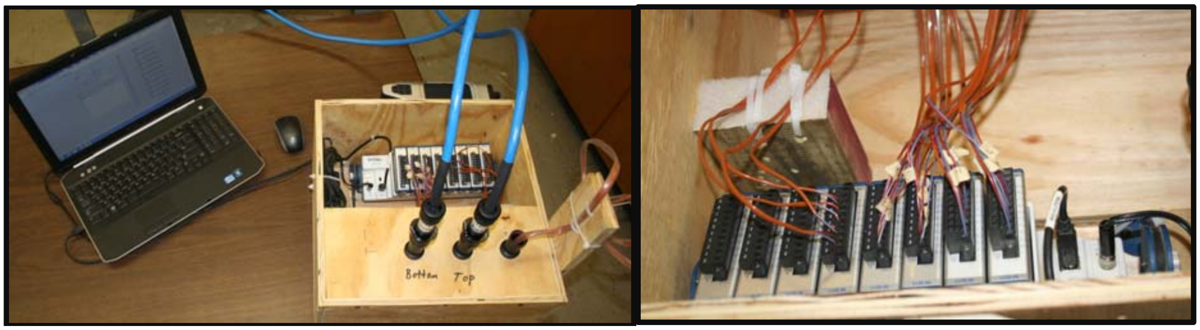


Figure 4.1. Photos of Data Acquisition System (ID’s 1 to 4)

Figure 4.2. Photos of Probe Inserted into Trucks (ID 17)

4.3 Laboratory Evaluation of Instrumented Probe

The most important parameter associated with the full scale instrumentation approach was whether accurate temperature measurements could be made using the probe as the carrier for bead thermocouples. Laboratory testing was performed with thermocouples wired directly to the data acquisition system (i.e. no trunk line or connectors), and the probe cavity was not filled with silicone as it ultimately was during field testing. The purpose of the experiments was to determine if the approach of instrumenting a hollow probe with thermocouples, pushing the probe into an asphalt truck, opening the probe to expose the thermocouples directly to asphalt, and recording data over an extended period was feasible.

Testing was performed on a small asphalt quantity relative to a truck. If problematic behaviors do not show up with a small quantity of asphalt there is little reason to suspect they would occur with a much larger quantity. Laboratory tests were performed in metal buckets with an 18,900 cm^3 volume that was filled with asphalt. A 19 mm NMAS field mixed asphalt (79% limestone and 4.3% PG 67-22 binder and G_{mm} of 2.553) was used for the experiments. Estimated air voids of the mix in the bucket was 23%, as the mix mass was 37.2 kg for all experiments. Three items were of interest during the laboratory experiments:

1. Can the same temperatures be measured in the same mix by the same type of thermocouple with and without the probe?
2. Can the potential of the probe transferring heat to the mix be negated when the probe is warmer at another location than it is where the measurement is being taken? In other words, can the higher temperature of the probe near the middle of the mix be isolated from the measurements of interest near the edges of the mix?
3. Can the potential of the probe transferring heat from the mix be negated when the probe is cooler at another location than it is where the measurement is being taken? In other words, can the lower temperature of the probe near the edge of the truck, and especially the portion outside the truck be isolated from the measurements of interest at distances farther into the mix?

Figure 4.3 summarizes the two experiments performed; control experiment and probe experiment. Each experiment used one bucket of asphalt. Both buckets were filled with heated asphalt, covered with a metal lid, and allowed to cool to room temperature. The instrumented buckets were placed into a Cascade TEK Model TFO-28 oven ($\pm$ 2 C temperature uniformity at 150 C and 0.79 m^3 capacity), with the thermocouples attached to the data acquisition system.

The control experiment incorporated two bead thermocouples attached to a small wooden rod that allowed both thermocouples to be placed in the center of the bucket (radially and vertically). Half of the asphalt was placed, the thermocouples were inserted, and the rest of the asphalt was placed. The average reading of these thermocouples is denoted *Control* hereafter.

The probe experiment incorporated three bead thermocouples: 1) *Reference TC*-placed near the upper end of the probe in a manner where the measuring junction contacted the probe; 2) *Flat TC*-placed at the center of the bucket (radially and vertically) with the cable laid flat so that the measuring junction contacted the probe; 3) *Curved TC*-placed at the center of the bucket (radially and vertically) by curving the thermocouple cable in a manner where the measuring junction was not touching the probe (the measuring junction was a few millimeters above the probe). The probe tip was touching the bottom of the bucket. *Flat* and

Curved thermocouples were within a few millimeters of each other, and are shown in a close up view in Figure 4.3. The probe (with thermocouples mounted inside) was placed in the bucket while closed, then the bucket was filled with asphalt. A pre-fabricated bucket lid with a hole and a support bracket was used to keep the probe aligned during testing. Once the support bracket was in place, the probe was opened to expose the bead thermocouples to the asphalt (Figure 4.3 shows a close up photograph of the probe opened where the bead thermocouple cables can be seen entering the asphalt mixture).

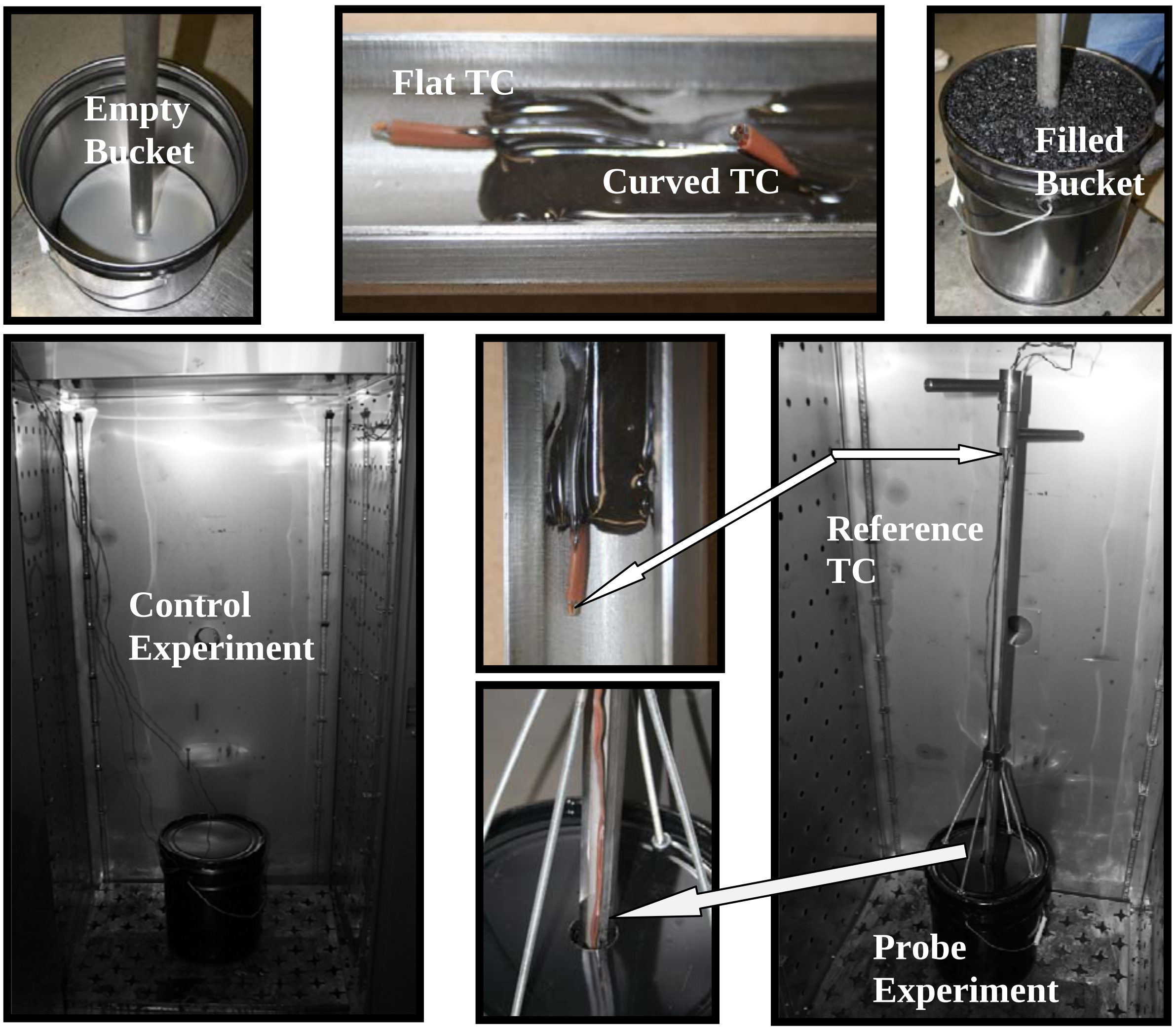


Figure 4.3. Photographs of Laboratory Temperature Experiments

Both buckets were initially at room temperature sitting in the oven that was shut off. The oven door was closed, the oven temperature was set to 166 C, and the material sat in that environment over night to ensure all material was essentially the same temperature and that temperature gradients within the asphalt were minimized. Some temperature gradient will always exist in an asphalt mass of this size and the temperature within the asphalt is usually slightly cooler than the oven temperature. For these experiments, the measured temperatures were 1 to 3 C below the oven setting when the buckets were removed from the oven, sat in the laboratory floor (a cloth was placed over the hole in the probe bucket lid), and allowed to cool. Temperatures were recorded beginning with the buckets in the room temperature oven through heating and until the asphalt cooled to approximately 80 C.

Results of the laboratory temperature experiments are plotted in Figure 4.4. There is no practical difference between the control and probe experiments indicating the same temperatures can be measured in the same mix with and without the probe. Figure 4.4b shows the probe transferring heat to the mix as the reference thermocouple is much warmer than the mix. There is some difference between the flat thermocouple configuration and the curved thermocouple configuration, as the flat configuration is noticeably higher than the control while the curved configuration is practically the same as the control. In the curved configuration, the temperature measurements appear to have been isolated from the probe. Figure 4.4c shows a small influence of the probe on the flat configuration while cooling, but the difference dissipates rather quickly until there is no practical difference in any of the thermocouple configurations while cooling (Figure 4.4d). Test results indicated that the measurements of interest can be isolated from the probe, and that the curved thermocouple configuration is the most appropriate choice for full scale testing. The probe with the curved thermocouple configuration was used for full scale testing as a result of the laboratory tests.

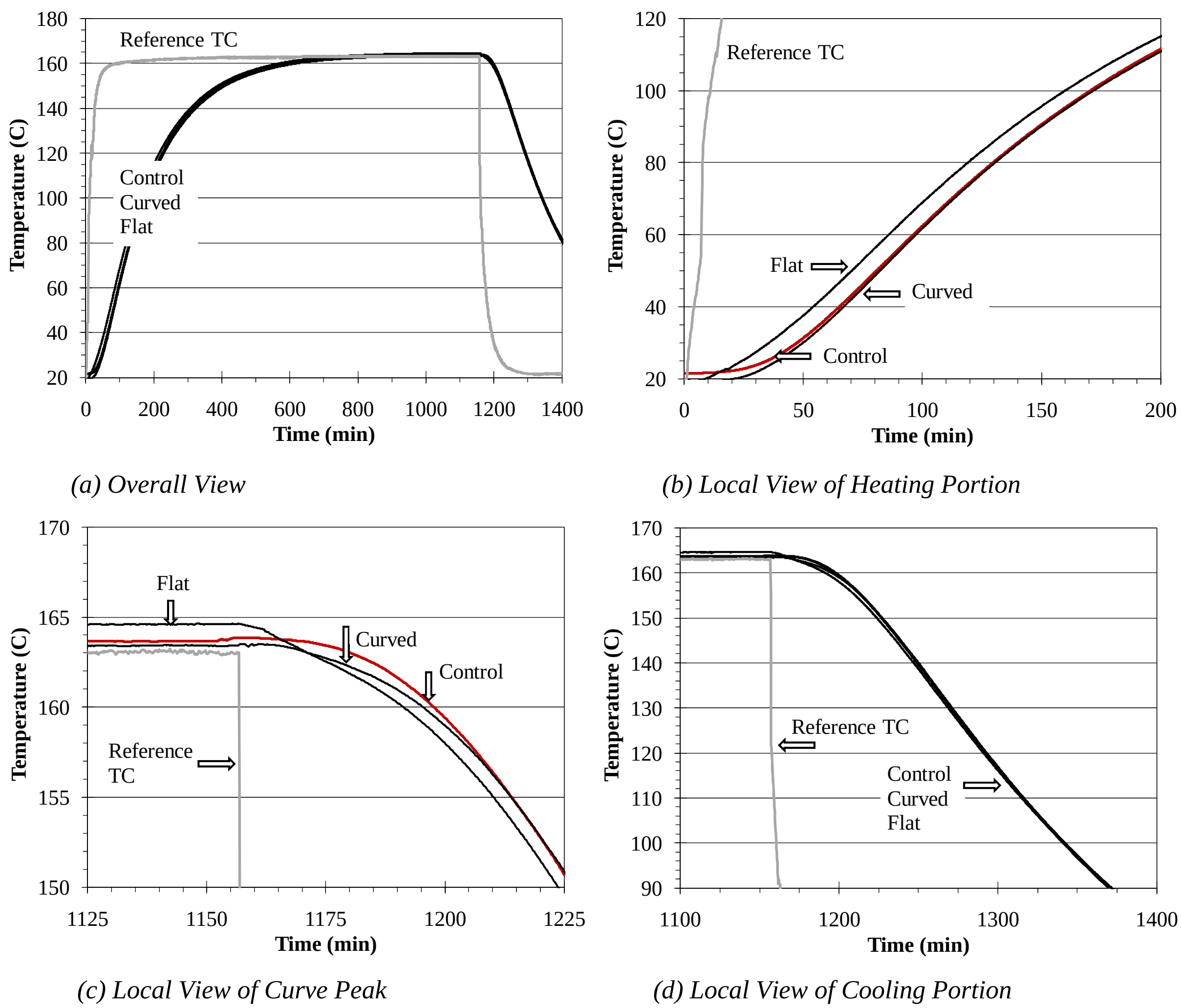


Figure 4.4. Results of Laboratory Temperature Experiments

4.4 Instrumentation Fabrication

The Section 4.2 parts were used to assemble the instrumentation components. The Figure 4.1 data acquisition box and thermocouple bundles (discussed later in this section) were equipped with male quick connectors, and the trunk line (blue cable) was equipped with female quick connectors. The connectors had to be fabricated, and manufacturer instructions were largely followed during fabrication. Each thermocouple had a copper wire and a constantan wire, making it necessary for the path to the data acquisition system to be made from these two materials. The thermocouple, trunk line, pins, and sockets were a continuous series of either copper or constantan that were wired directly to the data acquisition modules.

The male quick connects require a 4.4 to 5.2 mm stripped wire. Figure 4.5a shows the end of the wire for the male end of a quick connector that has been stripped and separated. To make the connection on the male wire end, a male pin is crimped onto the wire (Figure 4.5b). There are tight tolerances on the strip length because the insulation must enter the end of the tip, but the wire must extend until it can be seen in the pins peep hole (Figure 4.5c). Once all of the pins and wires have been crimped and the tabs removed, the pins are inserted into the male quick connector. The quick connector is lubricated with a silicone oil and the wire is started into the quick connector. The MTC-IT tool is placed against the back of the tip and the wire is placed into the channel on the tool. The wire is then pushed into the quick connect until a click is heard. All wires are placed using the aforementioned steps, and Figure 4.5d shows a completed male connector, which has 16 pins (8 copper, 8 constantan) and accommodates eight thermocouple measurements.

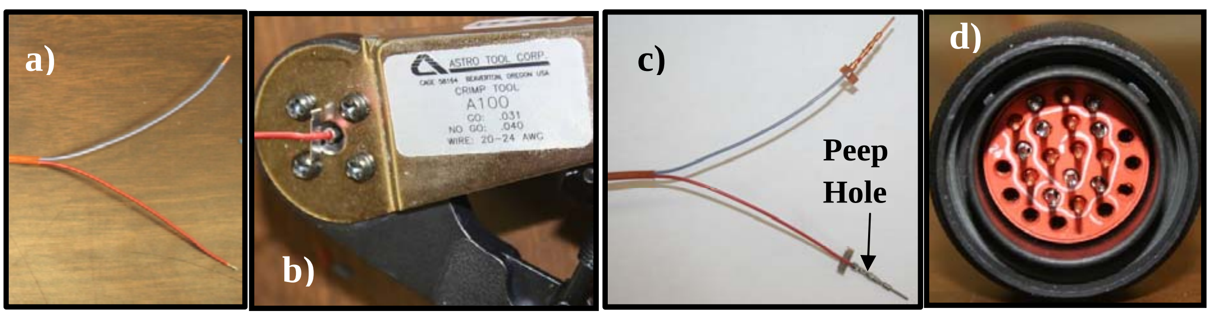


Figure 4.5. Male Connector Fabrication

Figure 4.6 shows the wiring component (referred to as chassis bundle) connecting the trunk line to the data acquisition box (Figure 4.1 has two of these components). Each wire was covered with heat shrink tubing for insulation and protection. The wires were separated into two groups of four thermocouples as the NI 9211 modules each accommodate four thermocouples. Wires were traced using labeling on the trunk wires and connectors.

Figure 4.6. Male Connector (Trunk Line) and Stripped Ends (NI 9211 Modules)

To create a female connector, the sixteen (8 copper, 8 constantan) numbered wires in the trunk line were separated, stripped, and sockets placed on each wire (Figure 4.7a). Two ground wires were present in the trunk line, but they were not used. The trunk line wires with sockets were placed into the female connector; Figure 4.7b shows the wires prior to insertion and Figure 4.7c shows the connector with the sockets in place. Heat shrink tubing and the backshell cable clamp (ID 10) were then placed (Figure 4.7d) to protect the ends of the trunk line from damage due to twisting and pulling.

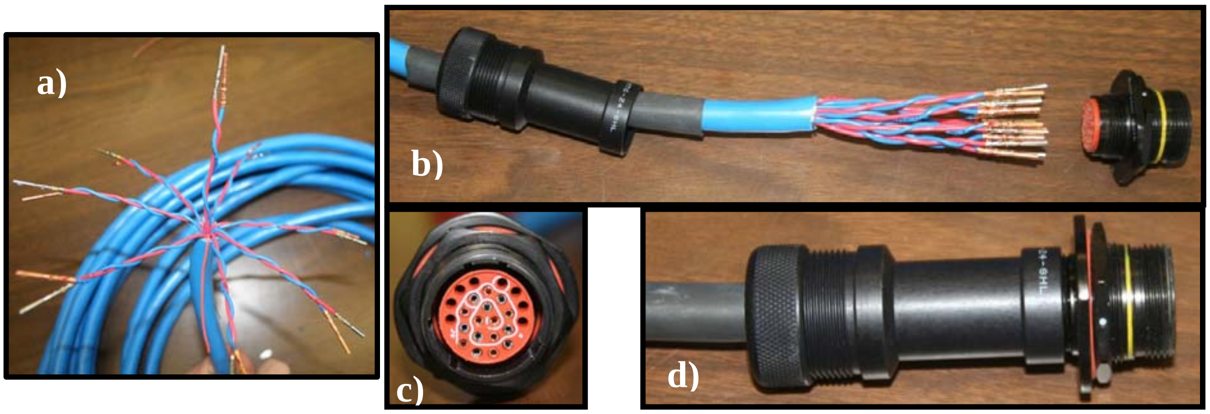


Figure 4.7. Female Connector Fabrication For Trunk Lines

Thermocouple bundles were made to fit into the Figure 4.2 probe and allow measurement at four locations along its length as shown in Figure 4.8. Two thermocouples were placed at each location for redundancy, making the total number of thermocouples per probe equal to eight. Thermocouple bundles used male quick connectors (Figure 4.3) and attached directly to the trunk line. To create a thermocouple bundle, the bead thermocouples (ID 16) were trimmed to lengths of 35.6, 73.7, 111.8, and 149.9 cm for placement in locations L1 to L4, respectively, while allowing all thermocouples to fit into the make quick connector. All eight thermocouples were then bundled together with heat shrink tubing for insulation, protection, and ease of loading into the probes. Figure 4.6 also shows a thermocouple bundle with male connector that is coiled for easier visibility.

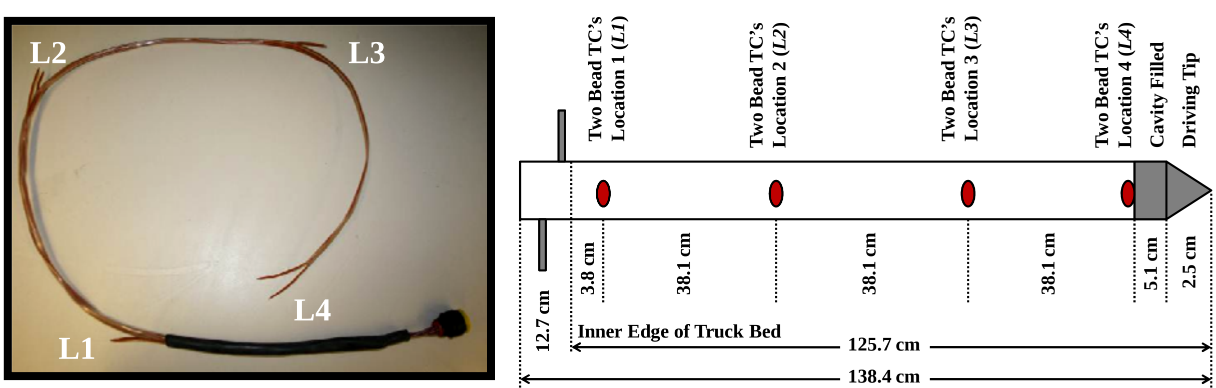


Figure 4.8. Thermocouple Probe Locations and Bundle

Spots were etched inside the probes to facilitate thermocouple alignment and subsequent re-alignment as the thermocouple bundles were, generally speaking, replaced after each use. Thermocouples were aligned within approximately 1 cm of the etched locations. JB Kwikweld (referred to hereafter as JB) was used to attach the eight thermocouples (two per location) to the metal probe in the curved configuration discussed in Section 4.3. Figure 4.9a is an example of JB locations, and Figure 4.9b shows two thermocouples curved and awaiting JB treatment. RTV silicone (referred to hereafter as RTV) was used to fill the remaining air space to minimize air movement in the probe as it was learned during preliminary field testing (discussed later in the chapter) that errors could otherwise result. RTV was also used to fill all remaining space in the hole at the end of the probe where wires exited the probe; Figure 4.9d shows the inside of the hole and Figure 4.9e shows the outside of the hole. Figure 4.9c is a close up view of a location showing the thermocouple measuring junctions covered with a small piece of heat shrink tubing that was shrunk to the point it fully covered the junction but could still be easily removed by hand once the JB and RTV were in place and had solidified. The tubing was placed to protect the measurement junction during fabrication; Figure 4.2b shows thermocouples without the tubing with the measurement junction exposed.

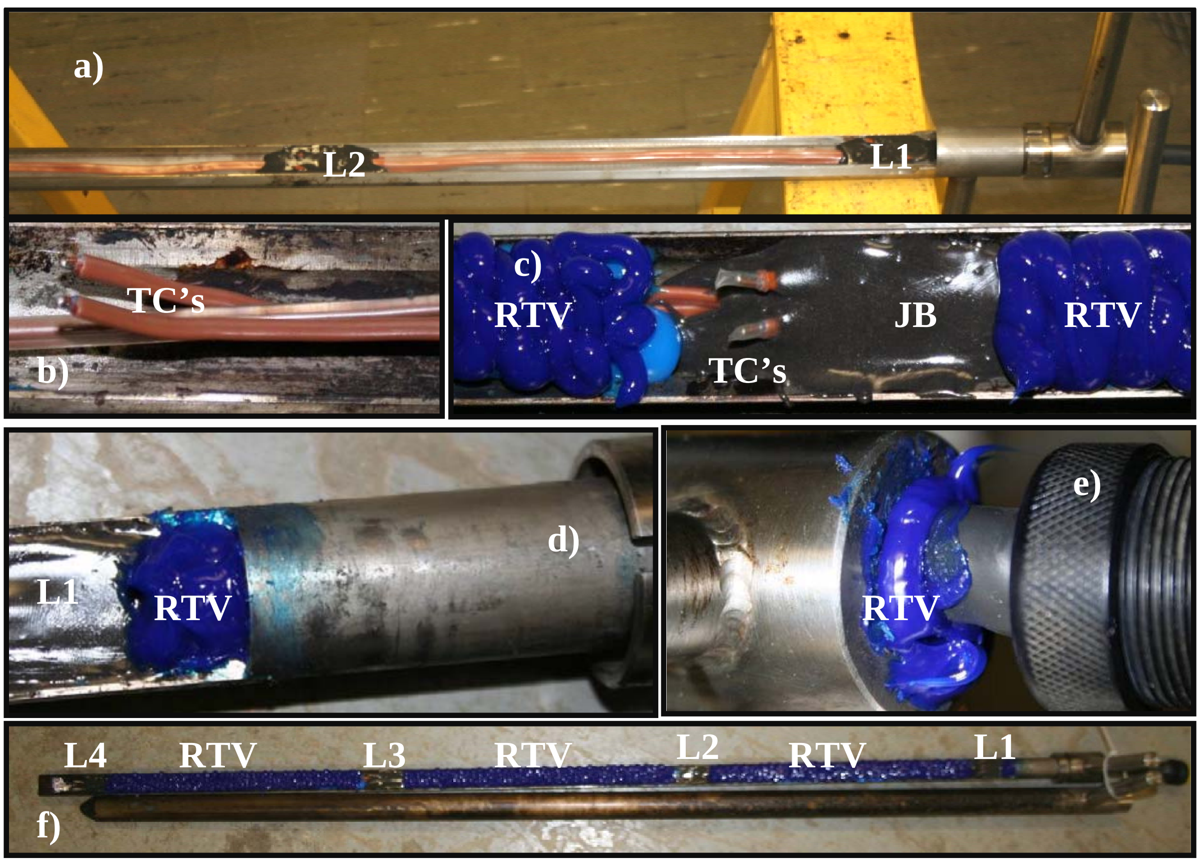


Figure 4.9. Placing Thermocouple Bundles Inside Probe

Thermocouple bundles were removed by first scraping the silicone from the probe. The JB areas were removed by applying heat to the back side of the probe, which facilitated the JB bond to loosen upon metal expansion which allowed the material to be dislodged from the probe. These activities were performed outdoors while research personnel wore protective eyewear, gloves, vapor masks, and similar to protect them from any fumes that might be present. To remove a thermocouple bundle and replace it with a new thermocouple bundle took one operator approximately two hours.

4.5 Instrumentation Verification

Once the instrumentation system was fabricated (i.e. chassis bundle, trunk line, and thermocouple bundle), it was verified by placing the thermocouples in a calibrated oven and verifying temperatures recorded by the data acquisition system matched the oven settings within the tolerance of the oven. The Cascade TEK Model TFO-28 oven used in Section 4.3 was also used to verify the instrumentation. Externally calibrated thermometers were also placed in the oven as another check. Once temperatures were successfully recorded with all wiring and connectors in place, proper labeling and similar quality control operations were performed prior to taking the system to the field for full scale testing.

4.6 Instrumenting the Asphalt Truck

Twenty thermocouples were placed on the asphalt truck shown in Figure 3.7 for each day of testing. Two probes (eight thermocouples each) were placed into the asphalt, and four thermocouples measured air temperature just above the passenger door mirror (Figure 4.10). These measurements are denoted (T_{ta}), or temperature of the air surrounding the truck. Two of these thermocouples were exposed to air and sunlight (circled in Figure 4.10), while the other two were enclosed in a metal box (indicated with an X in Figure 4.10).

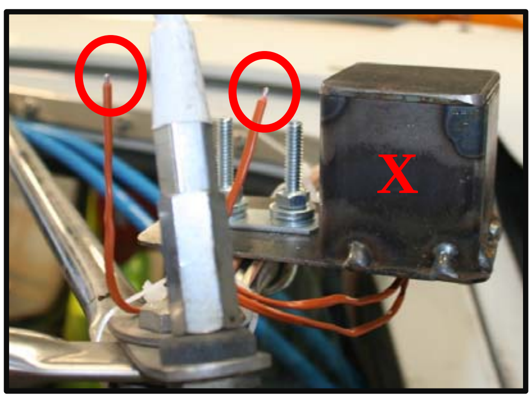


Figure 4.10. Asphalt Truck Air Temperature Measurement

The instrumented truck bed (Figure 3.7) was 7.32 m long, 2.44 m wide and 1.22 m tall. Probes were installed lengthwise 3.35 m from the end of the bed closest the cab, or 0.31 m from the lengthwise midpoint. This location was the fifth depression on the side of the truck (installation was easier and probe reached farther into asphalt) and was reasonably close to the middle of the truck lengthwise. Widthwise, the probes measured temperature from 3.8 to 118.1 cm from the inner edge of the truck bed. For a 2.44 m wide bed, the center is 122 cm from the inner edge of the truck bed and places *L4* (Figure 4.8) 3.9 cm from the center of the asphalt in the width direction. Since temperature gradients are symmetrical widthwise, this configuration measures temperature throughout the width of the truck for practical purposes. The center of the bottom probe (or probe 1) was 30.5 cm from the bottom of the truck bed, while the center of the top probe (or probe 2) was 76.2 cm from the bottom of the truck bed.

Figure 4.11 shows relevant details associated with inserting the probes into the truck bed. Figure 4.11a shows probe 1 open and a tape measure with the distance to the bottom of the truck bed. Figure 4.11b shows a close up view of probe 1 where the etched mark where thermocouples for *L1* (Figure 4.8) will eventually measure temperature. Also shown are the bolts welded to the inside of the truck that allow the alignment guide (Figure 4.11c) to be bolted to the outside of the truck. The alignment guide allows the probe to be pushed into the truck horizontally (or nearly horizontally). The alignment guide also has a notch that was eventually used to tie the probe open and keep it from moving in any direction during transport. A third benefit of the alignment guide was it provided support for the outer portion of the probe and for the trunk line. The alignment guide fit over the 3.8 cm diameter hole that was cut in the side of the truck to allow the probe to be inserted into the mix with very little clearance to minimize localized temperature disturbances. The wall thickness where the hole was cut was approximately 3 mm. Figure 4.11d shows probe 1 fully inserted into the side of the truck.

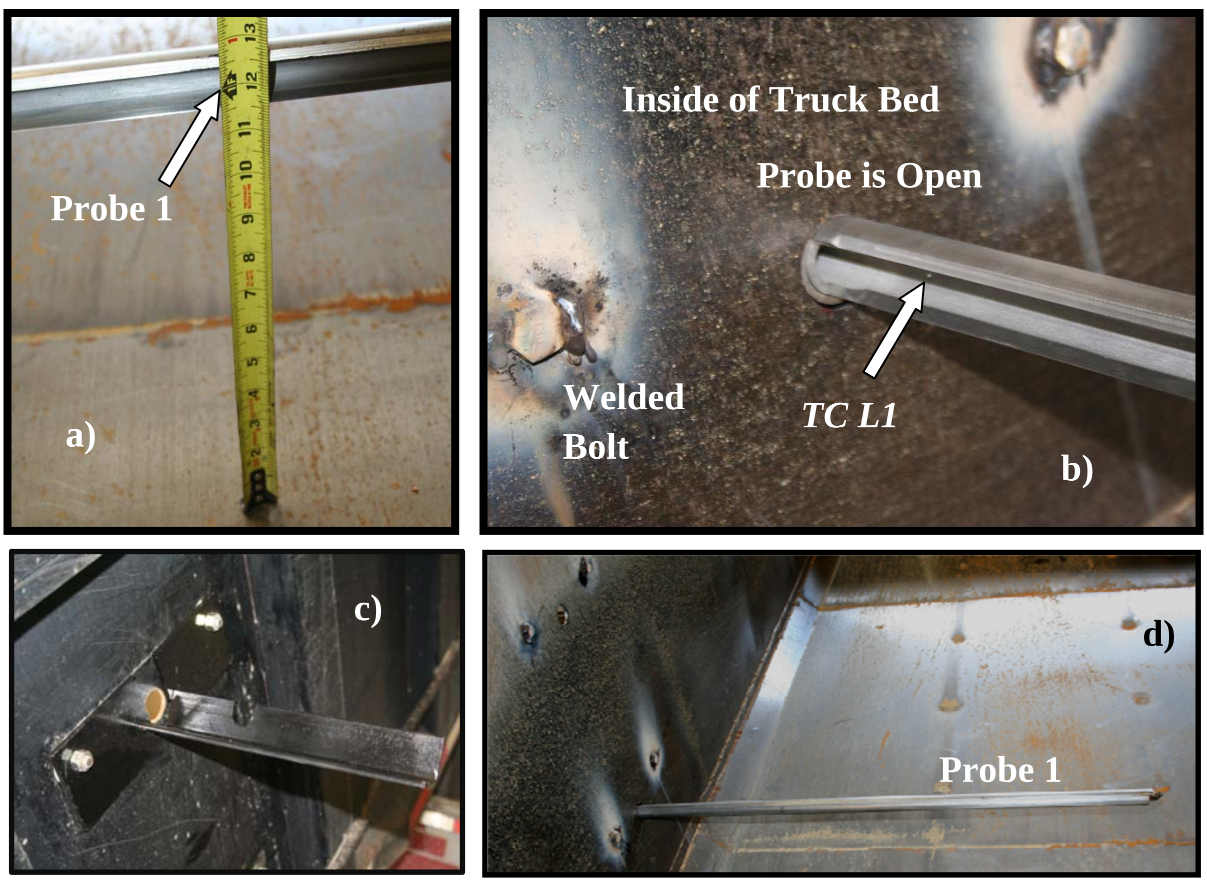


Figure 4.11. Probe Alignment in Trucks

The pre-fabricated box holding the NI chassis and modules (Figure 4.1) was placed in the floor of the truck cab, while the notebook computer was monitored by a passenger in the truck (Figure 4.12a shows the data acquisition box with one of the two trunk lines connected). The chassis was powered by the trucks outlet, while the computer was powered by spare batteries. The trunk lines (two were present for strips 1 to 12) were ran out the passenger door window, behind the exhaust, along the centerline between the truck and trailer, and then along the bed to the probe resting in the Figure 4.11c alignment guide. Figures 4.12b and 4.12c illustrate the overall path of the trunk lines. The trunk lines were connected to the truck and trailer using plastic wire ties, and slack was placed to allow the vehicle to turn. Plastic wire ties were selected since they are easy to remove with clippers. It took less than one hour for two people to install the data acquisition system, trunk lines, and alignment guides. These steps were performed before asphalt production.

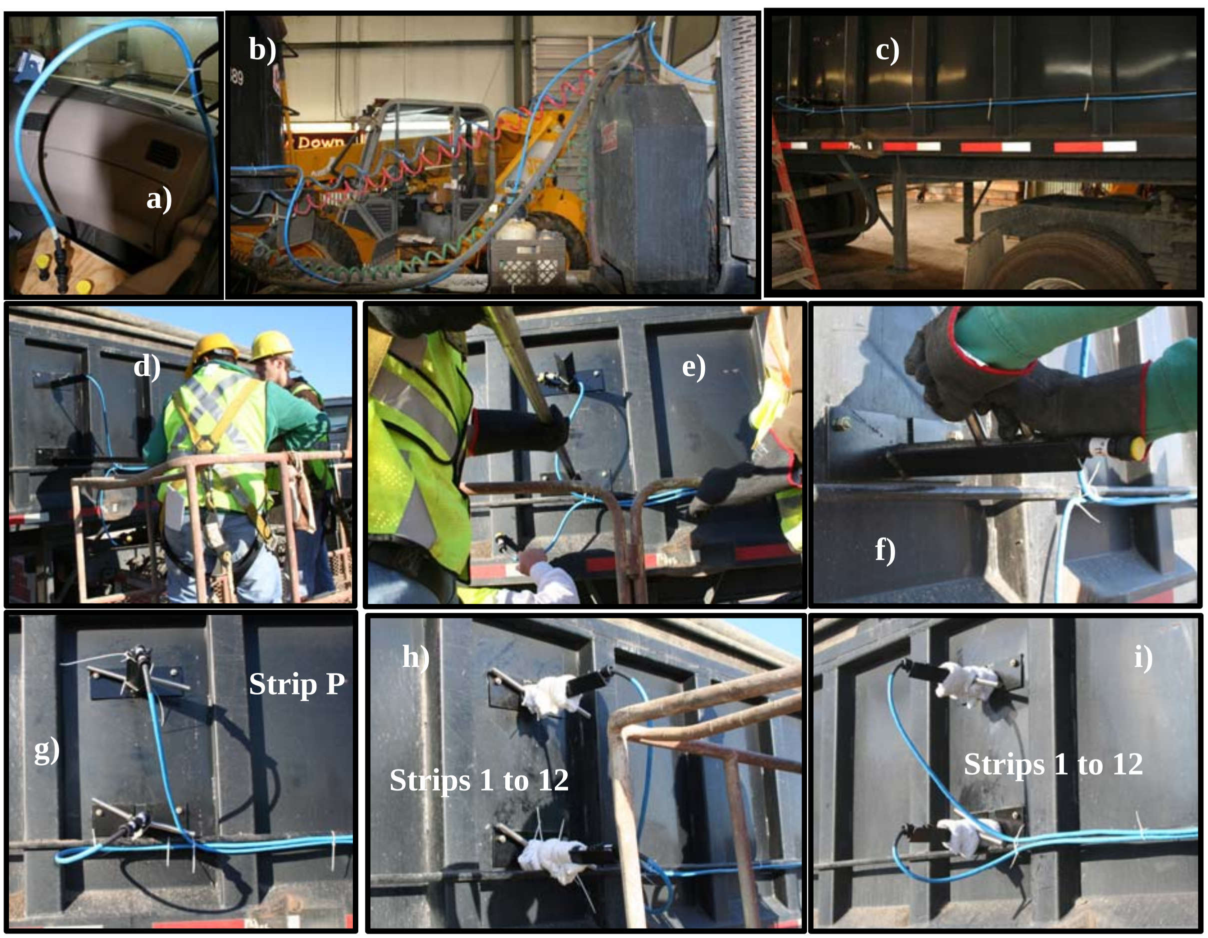


Figure 4.12. Placing Instrumentation System on Asphalt Truck

After the asphalt was produced and loaded in the truck, a hydraulic lift was used as a platform to allow two people to push the probes into the asphalt, while one or two additional personnel assisted on the ground. Figures 4.12d to 4.12f show the general steps used to install the probes, which were to push the probes along the alignment guide into the asphalt, connect the trunk line, open the probe to expose the thermocouples to the asphalt, use wire ties to secure the probe and cable to the alignment guide, and for strips 1 to 12 to wrap a pre-fabricated cloth with foam insulation around the opening and probe end to minimize air flow and heat loss. Figure 4.12g shows the strip P approach without the protective covering, which was shown to be problematic as discussed in the next section. Figures 4.12h and 4.12i show the probes wrapped with a cloth containing foam insulation, which was successful. It took 7 to 11 minutes to install the probes with the data acquisition and trunk line already in place.

4.7 Full Scale Test Results

Three sets of full scale test results were collected. The data collected on strip P and Hwy 389 were preliminary data used to make minor modifications to the data acquisition protocols. The data collected during strips 1 to 12 was the only data of interest for analysis. Strip P was placed October 3, 2011; Hwy 389 was placed October 14, 2011; and strips 1 to 12 were placed November 1 to 3, 2011.

One modification for each data set was the method used to pre-heat probes prior to pushing them into the trucks. Test strip P inserted the probes into a pile of hot asphalt (150 C+) for 30 to 60 minutes, but left the ends exposed (Figure 4.13a). This approach worked reasonably well, but the probe continued to heat for the first several minutes after insertion into the truck. On Hwy 389, hand held torches were used to heat the probes while sitting on metal frames. A small laboratory investigation revealed the probes could achieve on the order of 138 C from seven minutes of heating by measuring temperatures with the thermocouples inside the probes. The Hwy 389 data set revealed that heating with torches was not as uniform as the strip P approach. The approach ultimately used on strips 1 to 12 was to heat the probes in a pile of asphalt but to cover the ends with towels to minimize heat loss out the ends (Figure 4.13b).

Figure 4.13. Pre-Heating Probes With Plant Mixed Asphalt

Probes were removed manually by one or two people standing on a hydraulic lift while the probes were still open. Removal was more difficult than insertion, but it was feasible in well under five minutes. Upon removal, the interior probe components were in essentially the same condition as when inserted, except that asphalt was adhered to them to a modest extent (Figure 4.14).

Figure 4.14 Probes Removed Manually Post Haul

4.7.1 Preliminary Testing

Strip P testing did not fill the probe with RTV, and the ends of the probes outside the truck bed were not wrapped with a cloth filled with insulation. Otherwise the system was the same as described in Sections 4.4 and 4.6. Data collected in this manner appeared to be compromised by outside influences; an example is shown in Figure 4.15 (distance from inner edge of truck bed for each measurement location is shown in parenthesis). For all measurements but *L4*, temperature dropped while the truck was traveling at full speed down the highway and increased when the truck turned around or when it was traveling at slower speeds. The expectation is that air from outside the truck was infiltrating the probe and affecting temperature measurements, especially since *L4* did not appear to be affected.

The pre-haul asphalt temperature (T_{Pre}) measured as discussed in Section 3.5.3 was approximately 4 C less than the highest temperatures measured at *L4*. In that pre-haul material had been exposed to ambient air (14 to 15 C) while sitting at the top of the truck for approximately 15 min prior to sampling, these readings are in reasonable agreement and indicate *L4* was working properly and was not compromised. The overall conclusion from strip P was thermocouple readings *L1* to *L3* were compromised and that an improved system was needed. The improvements were to fill the probe with RTV silicone and to insulate the probes outside of the truck bed by wrapping them with insulation.

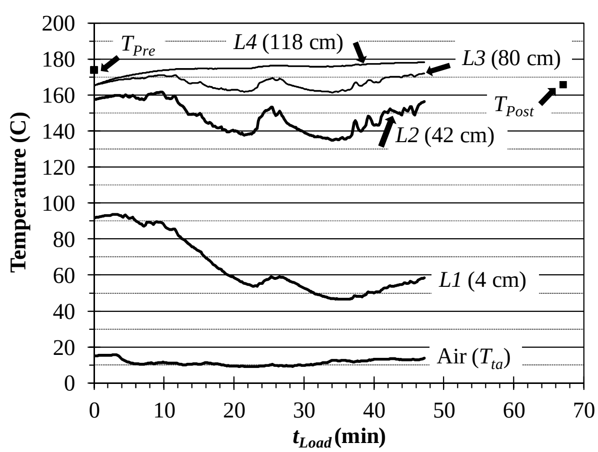


Figure 4.15. Asphalt Temperature Measurements from Strip P

Figure 4.16 plots all four air temperature measurements taken as shown in Figure 4.10 during strip P testing. All four measurements were practically the same. Data from these four measurements were averaged for strips 1 to 12 based on these findings.

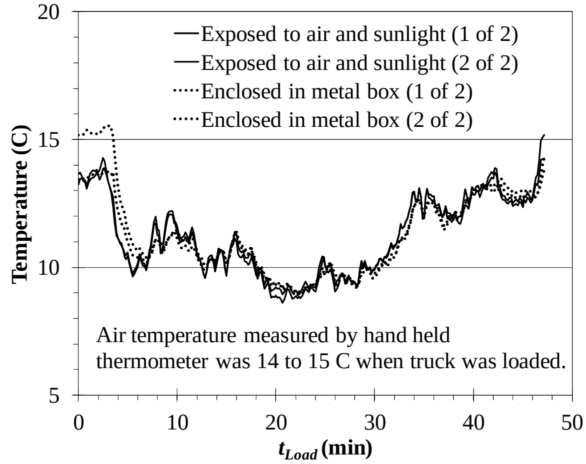


Figure 4.16. Air Temperature Measurements for Strip P

Prior to use on the twelve test strips, the modified instrumentation approach was investigated on a load of mix hauled to a local construction project on Hwy 389 near Starkville, MS. The haul had a few instances where the truck speed reduced, and the insulated probe was not susceptible to speed changes as it had been in test strip P. The modified approach was then deemed suitable for the twelve test strips. Torch heating was used for this experiment and it did not appear to uniformly heat the probes.

4.7.2 Test Strips 1 to 12

Trucks were aligned visually under the loading silos to maximize consistency between temperature measurements between test days. The largest of the three drops into the trucks was the middle drop to ensure all of both probes were covered with asphalt. Figure 4.17 shows the middle of the trucks where the probes were inserted.

Note: *Additive* was loaded too early for photos; probes were covered (visual evaluation).

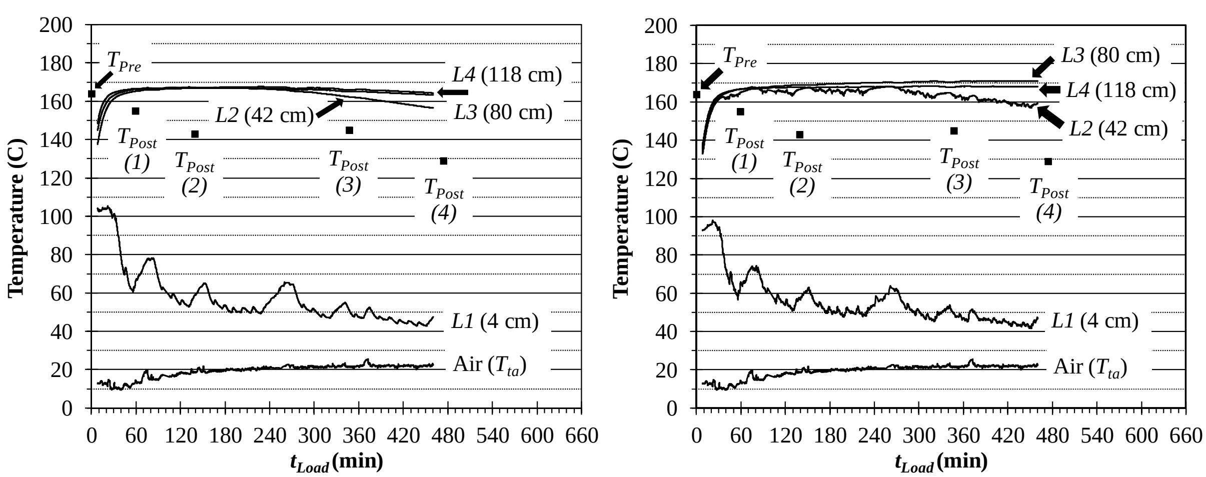
Figure 4.17. Photos Verifying Probes Were Covered During Transport

Strip 1 to 12 test results are provided in Figures 4.18 to 4.20. Temperatures measured within the probes at each Figure 4.8 location are shown alongside pre-haul (T_{Pre}), post-haul (T_{Post}), and air (T_{ta}) temperatures. Values shown in parenthesis beside thermocouple location numbers are the distance (cm) from the inner edge of the truck bed to the measurement location. In terms of percentages, *L1* to *L4* are 3, 34, 66, and 97% of the way to the middle of the truck in the 122 cm direction taking the inner edge of the truck bed as a reference. T_{Pre}

was taken as the average of strips 1 to 3 for Figure 4.18, the average of strips 5 to 7 for Figure 4.19, and the average of strips 9 to 11 for Figure 4.20. Strips 4, 8, and 12 were not used since they were not sampled for approximately 10 minutes after the other strips and were always cooler than the values from the other strips (See Table 5.1). The values shown in Figures 4.18 to 4.20 better represent the initial temperature of the asphalt not directly on the surface of the truck, which is the value of interest for comparison purposes. T_{Post} was shown for each strip (strip number was shown in parenthesis), and was plotted with $t_{Load} = t_h$ since the asphalt sample from the paver was taken at t_h for practical purposes.

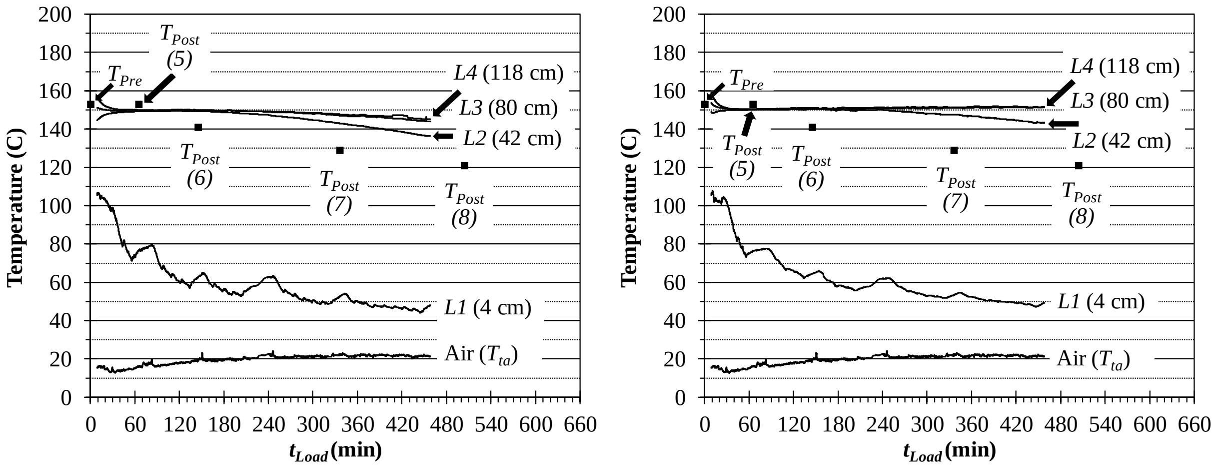
The Figure 4.18 to 4.20 data was processed relative to that collected as follows. Brief noise spikes, likely from a loose connection, were manually removed and temperatures during these periods were interpolated. An example of a filtered noise spike is a temperature reading instantly increasing 20 C and maintaining this value for a 30 second period before returning to the continuous curve. In some cases, data acquisition would cease and the operator would re-start the system (period where data was not being collected would be well under 5 minutes). In these instances, missed data was interpolated. Many of the instances where data collection ceased were the operator copying the files to prevent meaningful data loss before resuming data acquisition. Brief periods (e.g. 1 minute) without temperature data was of no concern as the overall temperature behavior was of interest and changes in the temperature curves occurred relatively slowly. In cases where both thermocouples at a given location were not damaged and recorded continuous measurements, the values were averaged and plotted in Figures 4.18 to 4.20. In some cases, one of the thermocouples was damaged and the plot reflects one temperature measurement. When both thermocouples were functioning, temperature measurements between them were in very reasonable agreement.

It took on the order of 10 minutes to fully insert the probes once the truck was loaded with asphalt, and once inserted it took on the order of 20 additional minutes for the probes to reach equilibrium. The data plotted in Figures 4.18 to 4.20 before t_{Load} of 30 minutes has no physical meaning as it is clear the probes are coming to equilibrium. Inserting the probes into pre-heated asphalt reduced the time to equilibrium, but as seen in Figures 4.18 to 4.20 approximately 20 minutes was still required. For purposes of this study meaningful data collected beginning 30 minutes after the truck was loaded is easily sufficient.



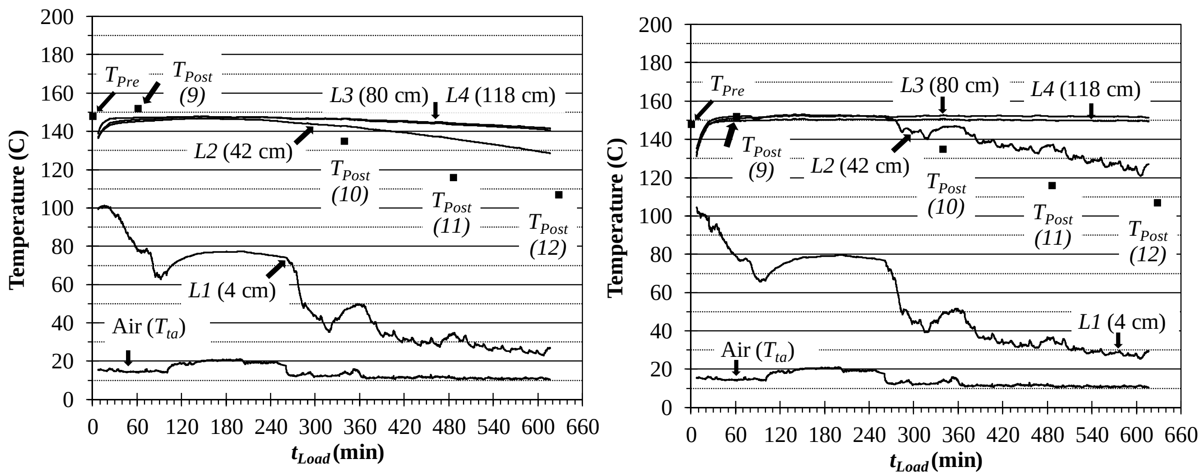
a) Probe 1 (30.5 cm from bottom of truck bed) *b) Probe 2 (76.2 cm from bottom of truck bed)*

Figure 4.18. Strip 1 to 4 (*HMA*) Temperatures During Hauling



a) Probe 1 (30.5 cm from bottom of truck bed) *b) Probe 2 (76.2 cm from bottom of truck bed)*

Figure 4.19. Strip 5 to 8 (*Foam*) Temperatures During Hauling



a) Probe 1 (30.5 cm from bottom of truck bed) *b) Probe 2 (76.2 cm from bottom of truck bed)*

Figure 4.20. Strip 9 to 12 (*Additive*) Temperatures During Hauling

Strips 1 to 4 were handled in the most favorable conditions for long haul distances; highest mixing temperature (164 C) and mean air temperature over entire haul of 19.3 C. Strips 5 to 8 were handled in the next most favorable conditions for long haul distances; next to highest mixing temperature (153 C) and mean air temperature over entire haul of 19.5 C. Strips 9 to 12 were handled in the least favorable conditions for long haul distances; lowest mixing temperature (148 C) and mean air temperature over entire haul of 14.3 C (this value includes period where parked in shed; temperature was as low as 10.3 C).

Temperatures at *L1* were erratic and dropped rapidly in all cases. The general trend was decreasing temperature, but temperature fluctuated due to, for example, sunlight exposure on the side of the truck where the probes were located. Due to rainfall, the instrumented truck was parked in a shed on November 3 (strips 9 to 12 shown in Figure 4.20) from approximately 8:05 AM to 10:45 AM (t_{Load} of approximately 100 to 260 min). During this period, temperature at *L1* increased in a progressive and smooth fashion, then maintained a fairly steady reading until the trucks commenced their normal route where wind was present and resumed the cooling at *L1*. This period where the trucks were parked would be an advantage in terms of cooling not to have wind blowing over the truck bed, but strips 9 to 12 were at a fair temperature disadvantage with air temperature (T_{ta}) being noticeably less than for strips 1 to 8, which were practically identical).

L2 measurements were very informative; arguably the most informative of any taken. Temperature at *L2* began to decrease in the 240 to 300 min (4 to 5 hr) window for Figures 4.18 to 4.20. This observation indicates that for the conditions encountered, all cooling before a four hour haul time occurred in the outer third of the truck bed.

Table 4.2 summarizes Figures 4.18 to 4.20. Measurements from Figures 4.18 to 4.20 were averaged over the time periods shown to provide a concise summary of all data collected from both probes on all three test days. The time periods used bracketed placement of strips 1 to 3, 5 to 7, and 9 to 11, while using the last five minutes of data collected to represent strips 4, 8, and 12 since the probes were disconnected before placing these strips.

Table 4.2. Summary of Temperatures During Hauling for all Test Strips

					Values shown are (Probe 1/Probe 2)			
Strip	**t_{Load} (min)**	**t_h (min)**	**T_{ta} (C)**	**T_{Pre} (C)**	***L1***	***L2***	***L3***	***L4***
1	44 to 74	59	14	164	68/65	165/165	166/167	167/167
2	124 to 154	139	19	164	59/57	167/166	167/169	167/167
3	332 to 362	347	22	164	51/50	162/164	166/171	165/168
4	455 to 460	474	22	164	46/46	157/159	164/171	163/168
5	50 to 80	65	16	153	76/76	149/150	150/151	150/150
6	130 to 160	145	19	153	62/64	149/150	150/151	150/150
7	321 to 351	336	22	153	52/53	143/147	147/151	148/151
8	453 to 458	504	21	153	47/49	136/143	144/152	145/151
9	46 to 76	61	15	148	80/80	145/152	146/151	147/150
10	324 to 354	339	14	148	45/48	143/146	146/152	147/150
11	471 to 501	486	11	148	32/35	136/136	144/152	144/150
12	612 to 617	628	11	148	27/29	129/127	141/151	142/149

-- The average T_{Pre} used in Figures 4.18 to 4.20 was also used in this table.
-- T_{ta} and *L1* to *L4* are average temperature measured over t_{Load} time period shown.
--*L3* measurements for strips 1 to 4 appear to drift upward a few degrees and may not be fully reliable.

The average pre-haul temperature independently measured with hand held thermometers (T_{Pre}) was 1 to 3 degrees lower than values for strip 1, 2 to 4 degrees higher than values for strip 5, and 3 degrees higher to 4 degrees lower than values for strip 9. Strip 1, 5, and 9 data was used for ease of visual comparison as it is shown in Table 4.2. The same observations can be drawn with data in Figures 4.18 to 4.20, but is more difficult to see at the same precision as in Table 4.2. The key observation is that the probe temperatures were within 4 degrees of independently measured thermometer temperatures during early haul times before the mixture cooled, which supports the quality of the probe measured data. Table 5.1 shows individual T_{Pre} measurements vary 3 to 8 degrees for measurements taken within five minutes after loading. In summary, the probe measurements appear to be as reliable as well accepted measurements with thermometers.

Probe 1 (probe nearest the bottom of the truck bed) typically cools more than probe 2. The maximum difference observed in the two probes, though, was only 10 C and occurred just over 10 hours after loading with strips 9 to 12 (*Additive*). Air temperature during later portions of the *Additive* haul were 11 C. The largest temperature drop for any *L2* to *L4* measurement was: 7 C for *HMA* (strips 1 to 4) at 7.6 hr; 17 C for *Foam* (strips 5 to 8) at 7.6 hr; 21 C for *Additive* (strips 9 to 12) at 10.3 hr. These values are not meant for between test day comparisons but are shown to illustrate the significant amount of time the majority of the asphalt in a truck can retain heat. The contrast to these measurements are those at *L1* (very close to edge of truck bed) where temperatures had cooled over 100 C for all three test days.

To represent the majority of the mix in the truck yet account for the cooler material near the edges, the laboratory short term aging protocol for long haul distances should hold mixes at a temperature a modest amount below their mixing temperature (15 to 20 C is suggested) for simulation of haul distances of 360 min (6 hr) or less. This material should be cooled to the desired compaction temperature over a period of a few minutes (transferring the material to a second oven set to a low temperature (e.g. 60 C) or turning off the oven set 15 to 20 C below the mixing temperature is suggested as a beginning point). A consistent temperature over the first 360 min is more representative of the behaviors measured than the approach used in phase 1 (Figure 2.3) where mix was progressively cooling. For simulation of haul distances over 360 min, the oven temperature could be dropped somewhat at 360 min to represent the cooling that occurred in the majority of the truck during this period. After this much short term aging, though, it is probably acceptable to keep the oven 15 to 20 C below the mixing temperature and cool to the desired compaction temperature over a slightly longer time than haul distances of 360 min or less. As has been stated previously, haul distances exceeding 6 hr are very unlikely (only very severe or rare conditions would require them), though if needed this project shows they are feasible.

The full-scale data collected shows the phase 1 approach dropped mixture temperature too quickly (effects could be less short term aging and less absorption). Figure 4.21 plots the full-scale data from Figure 4.18b to pertinent data from phase 1 plotted in Figure 2.3. Figure 4.18 was selected over Figures 4.19 or 4.20 since its mixing temperature was the same as the HMA used by Diefenderfer et al. (2007). The data shows that for early temperatures representing traditional construction the phase 1 approach was very reasonable (an expected result as significant amounts of data are available), but for conditions representative of long haul distances for emergency construction temperatures were too low at the conclusion of phase 1 STAP short-term aging. While Figure 4.21 strongly indicates the phase 1 laboratory short-term aging protocols were not fully representative of full-scale, the phase 1 mixture testing is still a reasonable approximation of emergency paving behavior.

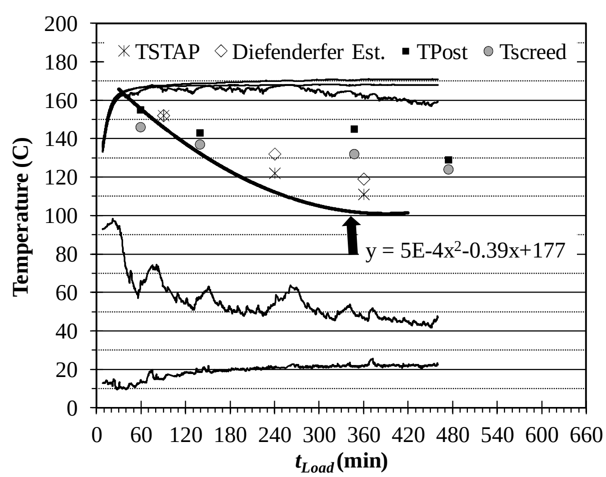


Figure 4.21. Comparison of *HMA* Temperatures During Hauling to Phase 1 STAPs

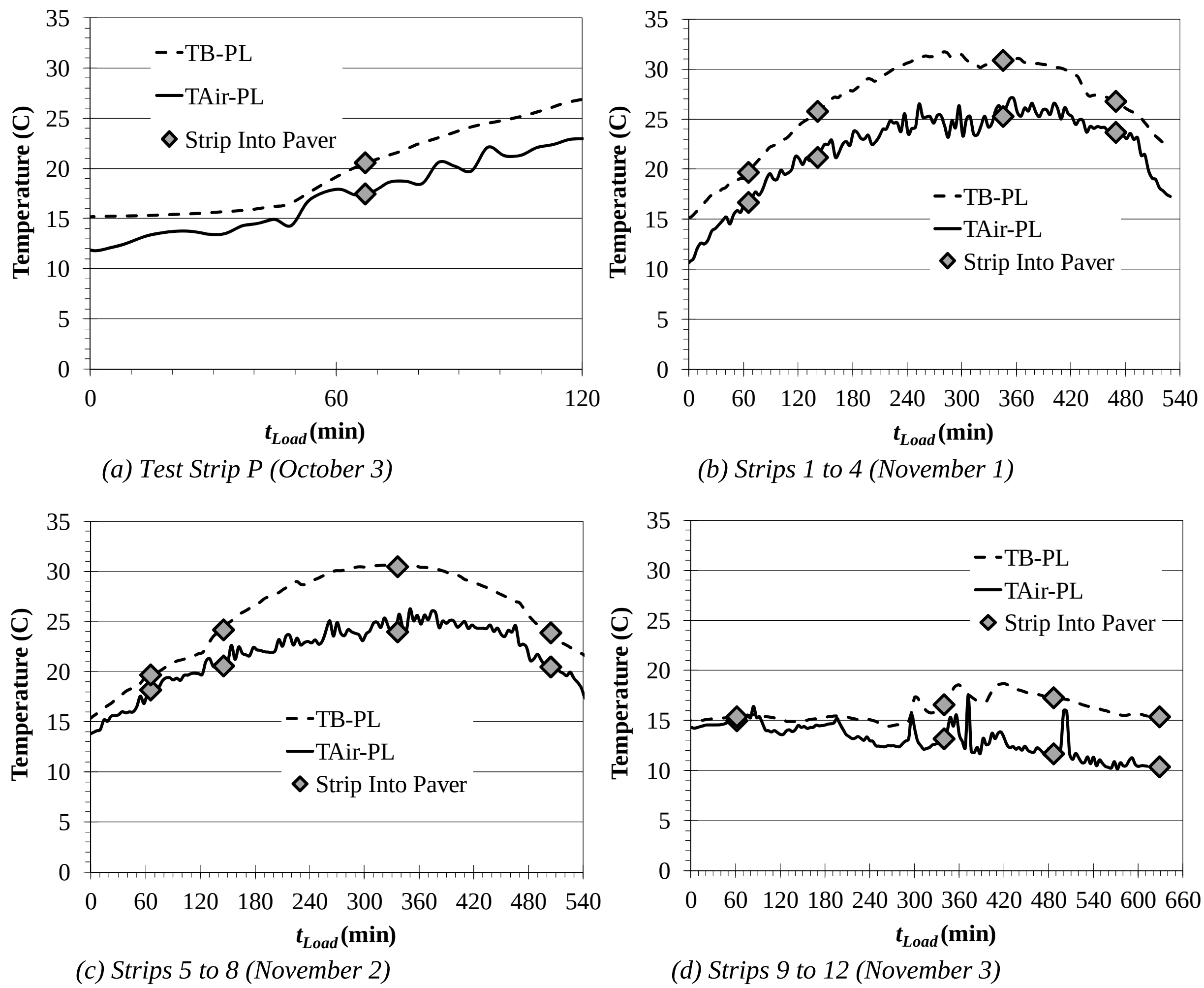
CHAPTER 5 – CONSTRUCTION RESULTS

5.1 Overview of Test Section Construction

This chapter provides pertinent construction information from placement of the thirteen test strips. Information provided in this chapter includes site conditions during paving, general observations from paving the test strips, and results of paving the test strips. In place air voids measured from coring are utilized to a minor extent in this chapter; they are presented in detail in Chapter 6.

5.2 Site Conditions During Paving

Figure 5.1 plots air and existing surface temperatures measured as shown in Figure 3.11. Truck loading time (t_{Load}) was averaged for all test days except strip P, making the *x*-axis accurate to $\pm$ 5 min, which is sufficient. Markers are shown on the air and pavement curves to indicate parking lot conditions at the time mix was dumped into the paver.



(a) Test Strip P (October 3)

(b) Strips 1 to 4 (November 1)

(c) Strips 5 to 8 (November 2)

(d) Strips 9 to 12 (November 3)

Figure 5.1. Air and Existing Parking Lot Surface Temperatures

Figure 5.2 plots relative humidity (RH) for each test day. Truck loading time (t_{Load}) was averaged for all test days, making the *x*-axis accurate to $\pm$ 5 min, which is sufficient. November 1 (Strips 1 to 4) was practically identical to November 2 (Strips 9 to 12). November 3 (Strips 9 to 12) was more humid. Figure 5.3 shows this behavior visually; strip 1 represents conditions for strips 1 to 8 and strip 9 represents conditions for strips 9 to 12.

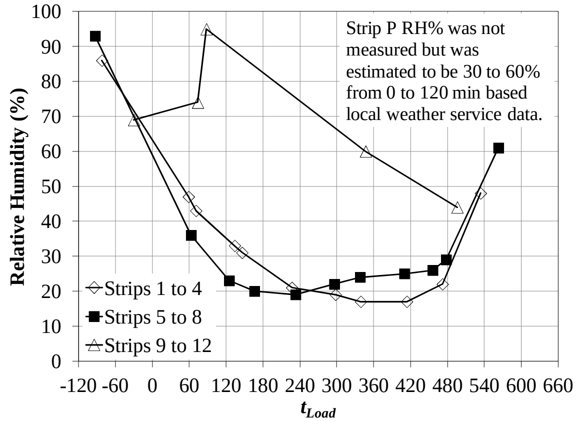


Figure 5.2. Relative Humidity Test Results

Figure 5.3. Representative Site Conditions During Paving

Overall, the weather on October 3, November 1, and November 2 was clear and sunny with slight breezes occasionally occurring throughout the day. All three days experienced very similar conditions that were favorable for paving. November 3 was very different from the other three days. Wind speeds were estimated at 24 to 32 km/hr (15 to 20 mph) when strip 9 was placed. Wind speeds remained fairly high throughout most parts of the day. It was overcast when strip 9 was placed, and it had started to sprinkle lightly by the end of strip 9 compaction. A few minutes after strip 9 was compacted, heavy rain commenced at approximately 8:30 AM, and the rain was essentially over by 9:00 AM. Light and occasional rain continued into the late morning. Three of the four trucks were parked indoors from approximately 8:05 to 10:45 AM (t_{Load} of approximately 100 to 260 min). Strip 10 was placed in a mostly overcast condition with a few brief rays of sunshine. It was fully overcast when strips 11 and 12 were placed.

Strip P and strips 1 to 8 were placed in much more favorable conditions than strips 9 to 12. Figure 5.4 shows the temperature and RH conditions for each test strip that were plotted throughout each test day in Figures 5.1 and 5.2. The lower temperatures and higher winds make it apparent strips 9 to 12 were compacted in more difficult conditions.

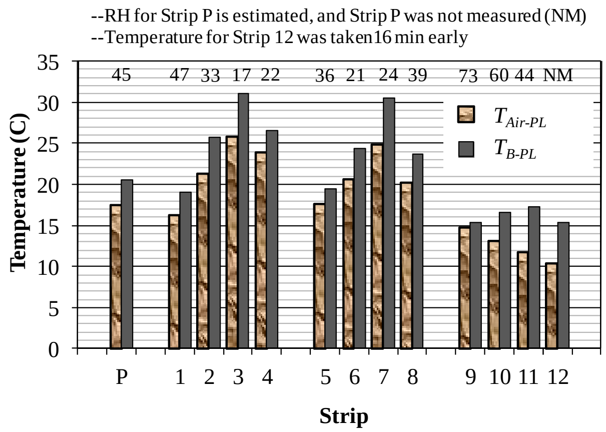


Figure 5.4. Test Strip Site Condition Comparisons

5.3 Paving Observations for Long Haul Distances

While placing strips 1 and 2 it was observed the mat was spreading laterally a noticeable amount (the screed width was set at approximately 3.13 m). The amount of lateral spread that was occurring was potentially problematic in terms of having enough space remaining for strip 12, and as a result corrective action was taken. After strip 2 was placed, approximately 0.5 m was trimmed along the entire length of the strip. For strips 3 to 12, a cut-off shoe was placed in the paver to help control strip widths. The mix appeared to show some tenderness signs, but no efforts were made to document the behavior in a quantifiable manner. Average compacted mat widths were: 3.29 m for strips 1 to 4 (0.5 m was added to the width of strip 2 for calculations), 3.35 m for strips 5 to 8, and 3.45 m for strips 9 to 12.

The roller pattern used was selected to provide consistency for the compaction energy applied to the asphalt mat. The roller pattern was not intended to produce a smooth surface, nor was it intended to prevent mat tears in the transverse direction. In some test strips (e.g. strip 4 in Figure 5.5), a noticeable tear occurred but this would not be expected in the field as the roller operator could adjust their pattern and not produce a stress concentration at a single location with each roller pass.

Figure 5.5. Example Mat Tear At Edge of Roller

Strips 3, 4, 7, 8, 10, 11, and 12 were hauled distances longer than are part of typical paving practices. For long haul distances removing the mix from the truck was a potential concern. Figure 5.6 shows that while the truck beds had to be raised considerable distances, the mix came out of the truck. Generally speaking, the mix broke loose and slid to the back all at once (strip 12 in Figure 5.6 is an example). After the strips were placed, drivers were able to remove all material from the truck beds from the cab without use of manual labor inside the bed (strip 3 in Figure 5.6 is an example). A noticeable amount of material remained in the trucks, but some of this material was extra material hauled in the event it was needed for the test strip. Material came out of all twelve strips without use of manual labor.

Strip 3 **Strip 3** **Strip 12 Pre-Movement** **Strip 12 Post-Movement**

Figure 5.6. Removal of Mix From Trucks for Longer Haul Times

After the long hauls, the truck beds were cool enough to comfortably rest a hand on the bed. In some instances (e.g. strip 4 in Figure 5.7) the mix near the edges was cool enough a hand could comfortably rest on the mix. There were several conglomerates when strips 4 and 8 were placed, and as a result there was just enough material to pave the test strip. The paver clogged enough with strip 7 that the paving train had to be stopped. The most likely reason was the screed was only heated 15 minutes prior to the arrival of this strip, which may not have been sufficient (typically it was heated 20 to 30 min). Strip 12 had major conglomerates (examples are shown in Figure 5.7) and they remained in the paver after the strip was placed (this could reduce efficiency in a disaster environment).

Figure 5.7. Conglomerates of Mix at Longer Haul Times

Figures 5.8 to 5.10 provide visual evaluations of each of the twenty-four test zones that were performed a few days after paving. Surface texture became coarser in strips 1 to 4 as the haul time increased. Strips 5 to 8 exhibited the same general behavior except there was more of a surface texture disparity between strips 5 and 6 relative to 7 and 8. Strips 9 to 12 were opposite as the surface texture was better in strips 11 and 12 than in strips 9 and 10.

For emergency applications, surface characteristics of any test zone in any strip was more than adequate for response, recovery, or similar temporary applications. This is noteworthy because no special efforts were made in this regard; the focus of the test strips was to evaluate compactability, load stability, and resistance to moisture damage. These behaviors are investigated in the reminder of this report.

Note: Mat was fairly uneven and torn in places outside the tested area for Strip 4 Zone 2 (S4-Z2)

Figure 5.8. Visual Evaluation of Strips 1 to 4 (*HMA*) Post Construction

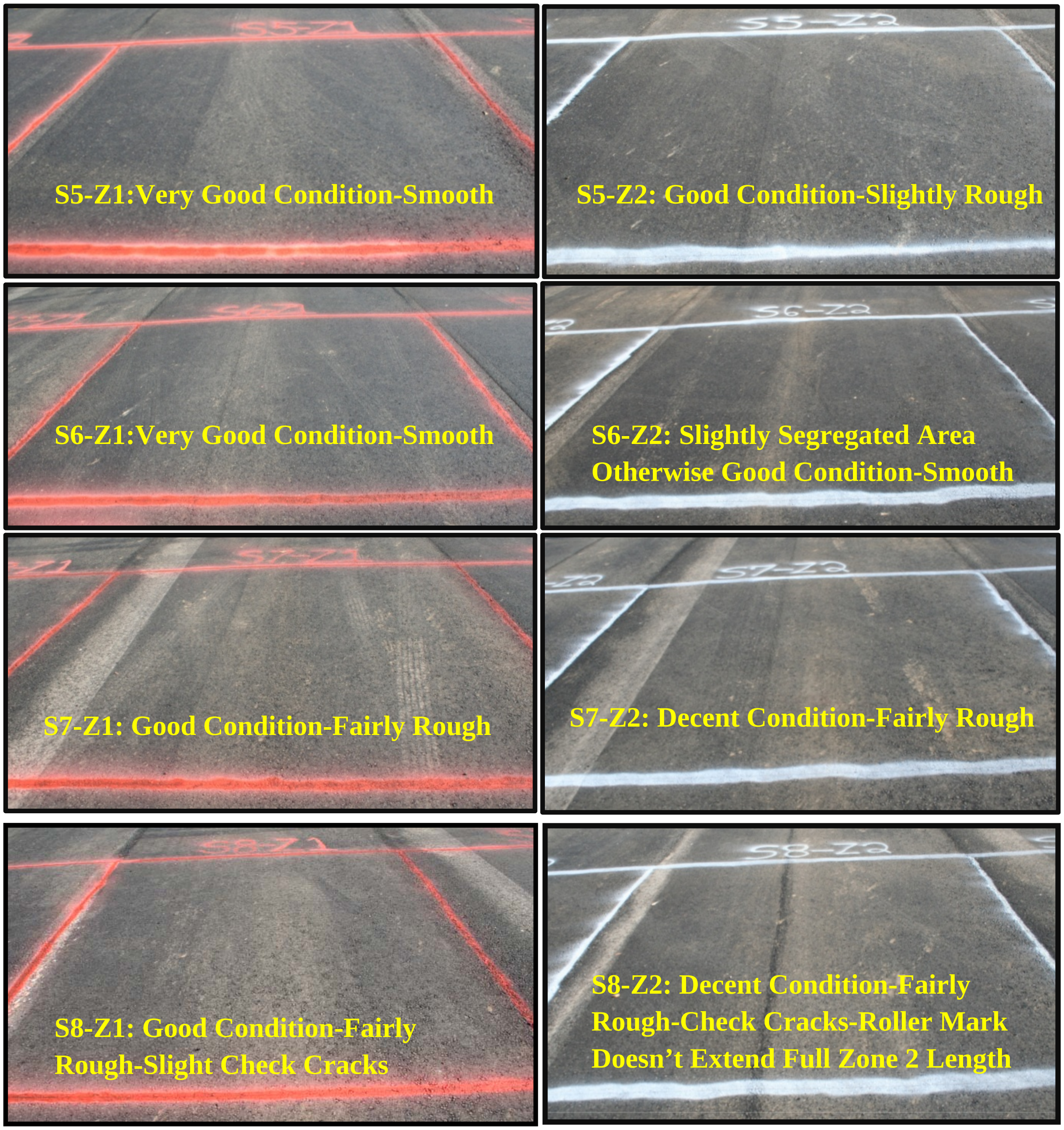


Figure 5.9. Visual Evaluation of Strips 5 to 8 (*Foam*) Post Construction

Note: Screed temperatures did not seem to be fully adjusted to cooler day, at least during paving strips 9 and 10. At least some of the roughness observed in strips 9 and 10 is likely due to screed.

Figure 5.10. Visual Evaluation of Strips 9 to 12 (*Additive*) Post Construction

5.4 Paving Test Results

Table 5.1 summarizes test results pertinent to field construction. An immediate observation are the very long haul times that each mix type (*HMA*, *Foam*, and *Additive*) experienced while still being able to pave a test strip with no major complications. The data in Table 5.1 is very supportive of asphalt concrete's ability to facilitate disaster recovery in a variety of manners. The remainder of this section evaluates the temperature and in place density data collected, largely for use in conjunction with the in place air voids measured on cores and discussed in Section 6.7.

Table 5.1. Construction Test Results

											Zone 1				Zone 2				
Strip	Tons[a]	t_L	t_{Lp}	t_{bp}	t_{ep}	t_h (min)	T_{Pre} (C)	T_{Post} (C)	T_{Screed} (C)	N_T	N_V	N_S	t/NMAS	N_V	N_S	t/NMAS	V_{a-N} (%)	V_{a-P} (%)
P	23.44	8:02 AM	8:17 AM	9:09 AM	9:14 AM	67	174	166	143	4	---	---	---	4	0	7.1	---	---
1	23.65	9:07 AM	9:20 AM	10:06 AM	10:23 AM	59	166	155	146	6	3	3	5.6	4	2	5.5	10.2	7.1
2	23.46	9:03 AM	9:20 AM	11:22 AM	11:37 AM	139	161	143	137	7	3	4	5.2	5	2	4.4	9.3	7.5
3	25.11	9:00 AM	9:20 AM	2:47 PM	3:00 PM	347	166	145	132	7	3	4	6.2	5	2	5.1	8.8	7.1
4	23.57	8:56 AM	9:20 AM	4:50 PM	5:10 PM	474	154	129	124	8	3	5	5.7	6	2	5.7	9.4	6.3
5	23.40	9:07 AM	9:18 AM	10:12 AM	10:27 AM	65	154	153	132	7	3	4	5.5	5	2	5.2	5.7	5.2
6	23.43	9:04 AM	9:18 AM	11:29 AM	11:44 AM	145	149	141	131	7	3	4	6.1	5	2	6.3	6.9	5.3
7	25.01	9:02 AM	9:18 AM	2:38 PM	3:04 PM	336	157	129	118	8	3	5	6.2	6	2	6.0	7.0	7.3
8	23.55	8:59 AM	9:18 AM	5:23 PM	5:44 PM	504	143	121	106	7	3	4	5.7	5	2	5.8	10.4	6.3
9	23.32	6:33 AM	6:52 AM	7:34 AM	7:48 AM	61	146	152[b]	137	7	3	4	5.9	5	2	5.2	9.0	5.7
10	24.11	6:30 AM	6:52 AM	12:09 PM	12:33 PM	339	149	135	105	8	3	5	5.4	6	2	5.5	9.9	4.7
11	23.48	6:27 AM	6:52 AM	2:33 PM	2:47 PM	486	149	116	116	6	3	3	5.6	4	2	5.0	9.6	6.2
12	23.31	6:24 AM	6:52 AM	4:52 PM	5:09 PM	628	143	107	93	6	3	3	6.3	4	2	5.1	9.8	6.8

a: Tons of mix per truck were from truck tickets. Average of 23.8 tons per truck, with total tonnage of 309.
b: Value believed to be erroneous.

5.4.1 Temperature Test Results

Figure 5.11 plots temperature behind the screed measured with an infrared device (T_{Screed}) to values measured post-haul in sample buckets with thermometers (T_{Post}). Readings correlated to one another fairly well, with T_{Screed} values being approximately 90% of T_{Post} values. Note that T_{Screed} is equal to T_s when t_{Screed} is equal to 0, and these data points are also plotted in Figure 5.12.

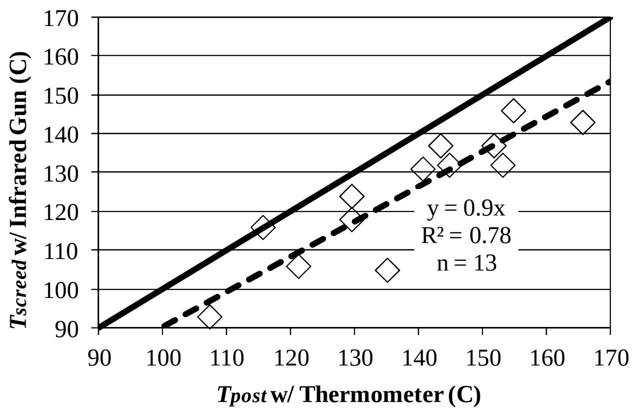


Figure 5.11. Comparison of Temperature Behind Paver Screed and in Sample Bucket

Figure 5.12 shows asphalt surface temperatures during paving (T_s) beginning when the screed passed over the location monitored (t_{Screed} of 0). Temperature was recorded once at t_{Screed} of 0 and again every time the roller passed by the location monitored (P_L or P_R in Figure 3.20), so the data shows the delay between placement and compaction, as well as the duration of time where compaction was performed. Table 5.2 was created using the data presented in Figure 5.12, alongside estimates of mid depth temperature (T_M) performed as described in the following paragraph. Table 5.2 is sorted according to haul time.

Table 5.2. Compaction Temperature Summary

t_h (hr)	Strip	t_{Screed} (min)	T_S (C)	T_M (C) Estimated
1.0 to 1.1	P	15 to 35	110 to 90	119 to 97
	1	7 to 25	110 to 90	119 to 97
	5	5 to 19	110 to 90	119 to 97
	9	5 to 18	110 to 90	132 to 108
2.3 to 2.4	2	5 to 21	120 to 90	130 to 97
	6	5 to 18	110 to 85	119 to 92
5.6 to 5.8	3	11 to 32	100 to 80	108 to 86
	7	8 to 24	95 to 75	103 to 81
	10	7 to 50	80 to 55	96 to 66
7.9 to 8.4	4	15 to 31	95 to 80	103 to 86
	8	8 to 22	85 to 70	92 to 76
	11	5 to 20	90 to 70	108 to 84
10.5	12	16 to 30	75 to 65	90 to 78

-- T_S range shown is where most of the compaction occurred.
-- t_{Screed} range shown is time interval where most of the compaction occurred.

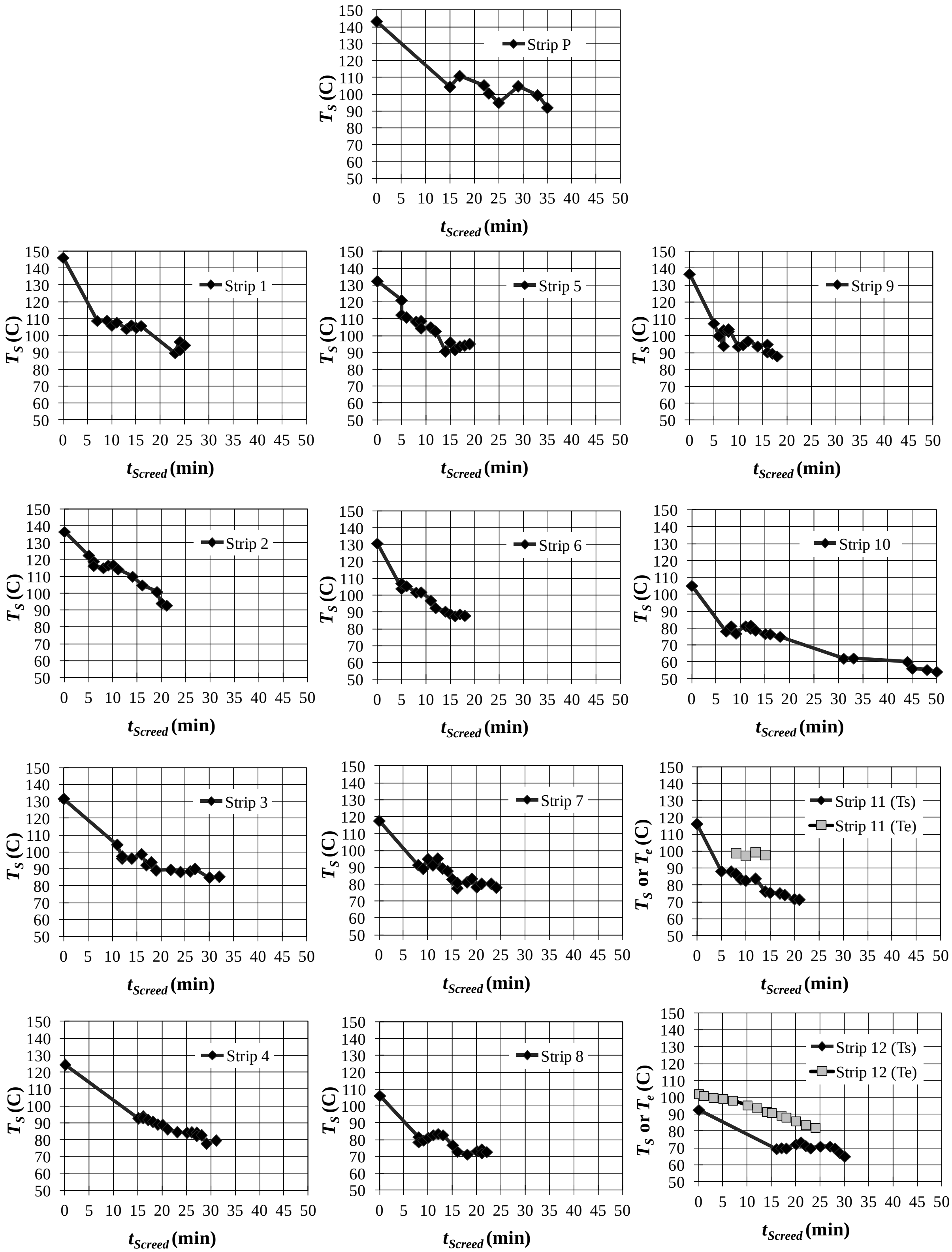


Figure 5.12. Asphalt Mat Temperatures During Paving

Embedded thermometer readings (T_e) taken during placement of strips 11 and 12 were on average 20% higher than T_S readings with a 6% standard deviation (Figure 5.12). This is higher than the 8% increase of T_S to T_M predicted by Figure 2.4 (T_e and T_M measure temperature near the middle of the asphalt layer in a different manner). The difference between Figures 5.12 and 2.4 is reasonable considering the data in Figure 2.4, generally speaking, was in more favorable weather conditions that would not have as much of a temperature gradient from the mat surface to the interior. In that strip P, and strips 1 to 8 were placed in favorable conditions, T_M was estimated from T_S by multiplying by 1.08, whereas strips 9 to 12 were multiplied by 1.20 to estimate T_M. Estimated T_M values are provided in Table 5.2.

Mix temperatures were used to account for all environmental conditions during the paving process. A cessation temperature of 80 C was used to subjectively evaluate paving conditions for all test strips. Mix tenderness appeared to occur (at least to some extent) as briefly discussed in Section 5.3, but since no quantifiable information was available it was not considered in the following assessments. Williams et al. (2011) provides a fairly detailed literature review on the subject of mix tenderness.

Compaction began 5 to 16 minutes after material exited the screed, and with exception of strip 10 was complete within 35 minutes after material exited the screed. Beginning to end of compaction was 13 to 21 minutes for all strips but 10, where compaction lasted 43 minutes. Five of the eight roller passes for strip 10 occurred within the first 11 minutes of compaction, while the remaining three passes occurred over a 19 minute period after a 13 minute compaction pause.

For the conditions of this study, Brown et al. (2009) provides time estimates of 18 to 23 minutes for material to cool from 120 to 80 C. Eight of the twelve strips had behind the screed temperatures exceeding 120 C. Compaction was complete within 25 minutes of material exiting the screed for all strips but P, 3, 4, 10, and 12.

Compaction conditions were similar and favorable for strips P, 1, 5, and 9, which were hauled 1.0 to 1.1 hr. Compaction conditions were also similar and favorable for strips 2 and 6 (2.3 to 2.4 hr haul times). Direct comparison of air voids within these sections at a given haul time seems reasonable based on compaction conditions. Compaction conditions were not meaningfully different for the 1.0 to 1.1 hr or the 2.3 to 2.4 hr haul times.

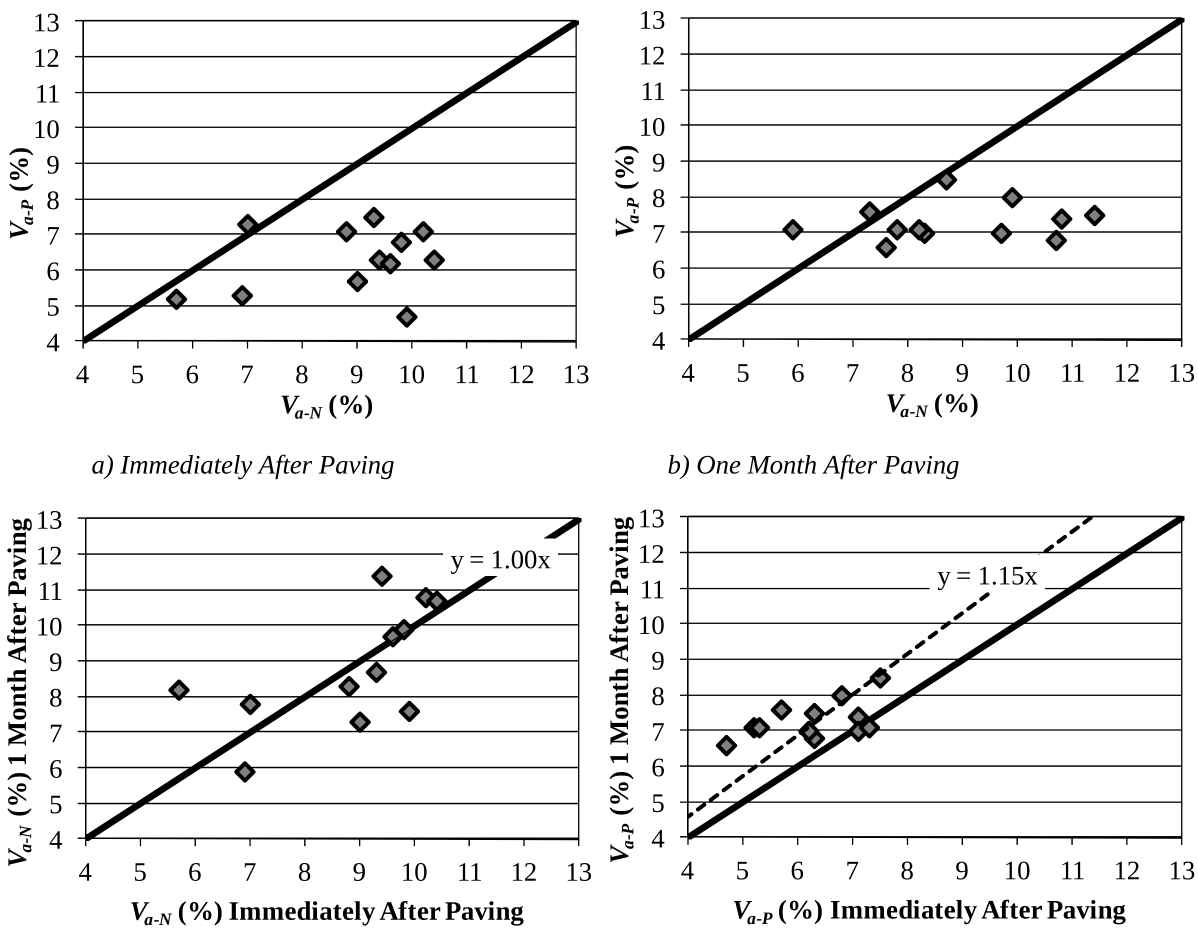
Compaction conditions for strips 3 and 7 were marginally favorable as temperatures were generally above 80 C for the duration of compaction. Temperatures were, however, noticeably cooler than strips hauled 2.4 hr or less. Compaction conditions for strip 10 were less favorable than strips 3 and 7 as discussed later in this section. Direct comparisons between strips 3 and 7 seem reasonable, though comparing strip 3 or 7 to strip 10 should consider the difference in compaction conditions, at least subjectively.

Compaction conditions for strip 4 were generally equivalent to strip 3, which were marginally favorable. Strip 8 was hauled an additional 0.5 hr relative to strip 4, and its compaction conditions were somewhat less favorable than strip 4. Strip 11 compaction conditions were less than favorable. Comparisons between strips 4, 8, and 11 should all consider differences in compaction conditions, at least subjectively.

With exception of strip 9, all *Additive* sections (10, 11, and 12) were compacted in less than favorable conditions. Surface temperature was 80 to 90 C at the beginning of compaction for strips 10 and 11. The estimated mid-depth temperature was more favorable at 96 to 108 C, but with cool underlying layer (16 to 17 C) and air (12 to 13 C) temperatures (see Figure 5.4) there would have been formidable temperature gradients in the asphalt mat. Strip 12 was compacted in very unfavorable conditions. There was a 16 minute duration between the screed and the first roller pass, and by the first roller pass the material was nearing the 80 C cessation temperature in the middle of the mat.

5.4.2 In-Place Density Test Results

Two in-place density devices were used to monitor compaction and they were used to make on site decisions regarding the amount of compaction to perform. Readings from strip P were taken and used to provide an offset for strips 1 to 12. These devices were used at the three locations described in Section 3.5.2 (L_1, L_2, L_3), and cores were taken at L_2 within each section after all in place density testing was complete. Table 5.1 provides in place density results converted to air voids for both gages (V_{a-N} and V_{a-P}). Figure 5.13 compares PQI and nuclear gage readings immediately after paving and again one month after paving for the same locations (no traffic was allowed on the locations during this time and one location was included per test strip).



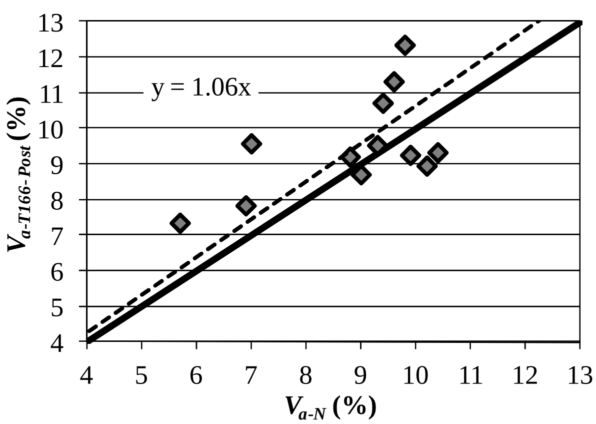
a) Immediately After Paving *b) One Month After Paving*

c) Stability of Nuclear Gage *d) Stability of PQI*

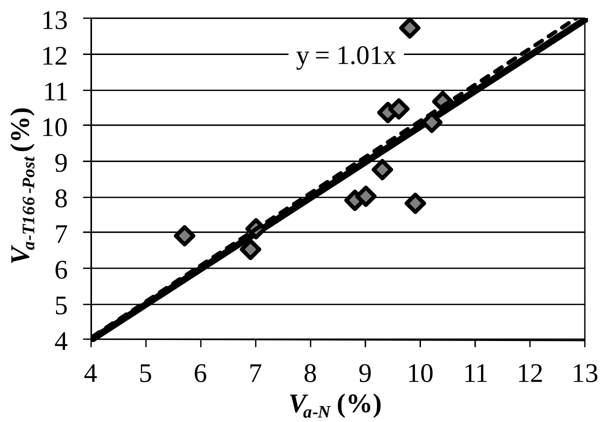
Figure 5.13. Comparison of PQI and Nuclear Gage In Place Air Voids

The nuclear gage predicted higher air voids immediately after paving and one month after paving. The air voids predicted by the nuclear gage were stable overall between the original readings and readings taken one month later (Figure 5.13c), as evidenced by a trendline with a slope of 1.00 (a fair amount of scatter was present indicating some inconsistency of individual readings). The air voids predicted by the PQI were not stable between the original readings and readings taken one month later (Figure 5.13d), as evidenced by a trendline with a slope of 1.15 (gage indicated air voids increased by 15% on average in absence of traffic over a one month period, which is at best unlikely).

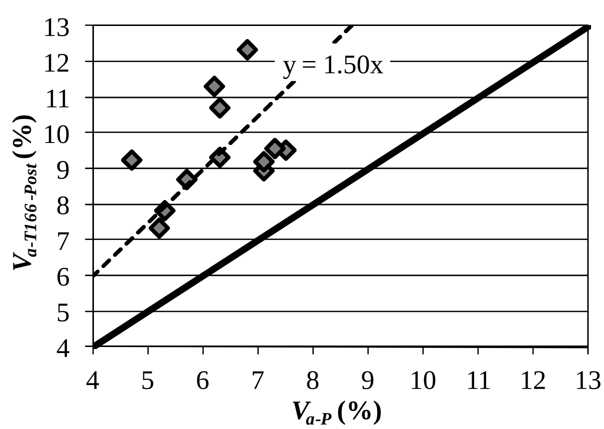
Figure 5.13 indicated at least one gage did not adequately represent in place air voids of the strips. Figure 5.14 clearly shows that the nuclear gage better predicted in place air voids than the PQI. Slopes of the nuclear gage relative to measured air voids in the test zone and for the individual density measurement locations were fairly close to the trendline with slopes of 1.01 and 1.06. An equality plot of L_2 and zone 2 (omitted for brevity) has a slope of 0.95 (plot of the y-axis of Figure 5.14a and 5.14b), indicating the locations used to measure density were reasonable representations of the test zone and that the only item of pertinence was the in place measurement method. The data in Figure 5.14 strongly suggests the nuclear gage data should have been relied upon more than the PQI data during paving.



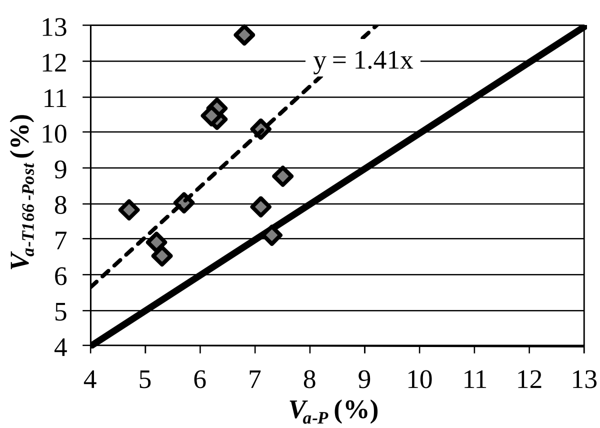
a) Nuclear Gage to Zone 2 Air Voids



b) Nuclear Gage to L2 Air Voids



c) PQI to Zone 2 Air Voids

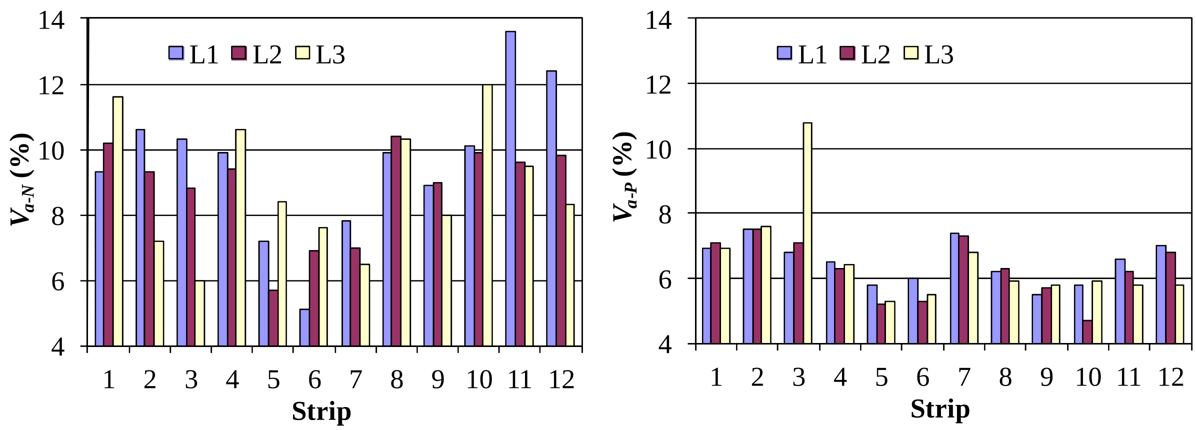


d) PQI to L2 Air Voids

Figure 5.14. Comparison of In Place and Core Measured Air Voids

PQI readings showed in place air voids were 7.5% or less in zone 2 for all test strips. Figure 5.14 and data provided in Chapter 6 clearly indicate this was not the case. On site decision making would have been improved if only the nuclear gage and subjective evaluation had been used.

Density measurement locations L_1, L_2, and L_3 represent the outer regions of the mat where compaction only occurred while the roller was traveling in one direction (i.e. L_1 and L_3), and also represent the inner region of the mat where compaction occurred while the roller was traveling in either direction (i.e. L_2). Figure 5.15 plots PQI and nuclear gage readings at the conclusion of paving according to test location. PQI readings should be questioned, but were provided since they were measured during the project. There does not appear to be a pattern to air voids as a function of mat position in the direction perpendicular to rolling, and L_2 does not appear more dense than L_1 or L_3 on a consistent basis. Average in place air voids from all 12 strips from nuclear gage readings were 9.6, 8.8, and 8.8% for L_1, L_2, and L_3, respectively. Average in place air voids from all 12 strips from PQI readings were 6.5, 6.3, and 6.5% for L_1, L_2, and L_3, respectively. As a result, the roller pattern was not considered in Chapter 6 when discussing or comparing air voids.



a) Nuclear Gage *b) PQI*

Figure 5.15. PQI and Nuclear Gage Readings by Location at Conclusion of Paving

CHAPTER 6 – MIXTURE TEST RESULTS

6.1 Overview of Mixture Test Results

Mixture test results performed on laboratory and field prepared materials are provided in this chapter. Several different tests were performed to evaluate the test strip properties. The data presented in this chapter compliments the construction data provided in Chapter 5.

6.2 G_{mm} Test Results

Figure 6.1 plots G_{mm} results for strip P where the dry back method was not used. Specimens were sampled pre-haul and allowed to cool immediately (t_{Load} of 15 min), while other specimens sampled pre-haul were placed in a 143 C oven and conditioned for different amounts of time on 60 minute intervals. An additional sample was taken post-haul (t_{Load} of 70 min), and all twenty data points were used to develop Figure 6.1. The strip P mix was made an estimated 30 minutes prior to loading so, t_{Load} of 50 to 70 minutes would reasonably represent Mississippi protocol short term aging of 90 minutes. G_{mm} is approximately 2.399 based on Figure 6.1 near a 90 minute age, which is 0.021 higher than the mix design G_{mm} of 2.378 (both used the B1 binder source). The average G_{mm} from the two samples taken from the paver was 2.405, which is 0.027 higher than the mix design. APAC Columbus measured G_{mm} to be 2.378 on a sample taken pre-haul; this measurement matched the mix design.

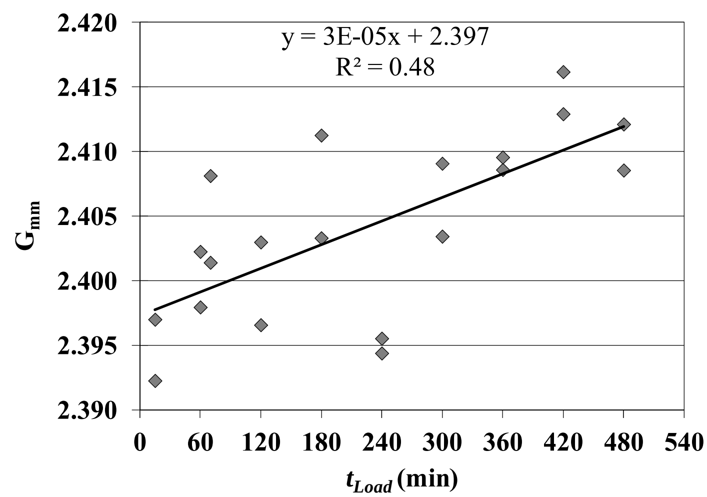


Figure 6.1. Test Strip P G_{mm} Test Results (Binder B1)

Table 6.1 provides G_{mm} results for test strips 1 to 12 where the dry back method was not used. The MSU laboratory performed six tests pre-haul (Pre) and six additional tests post-haul (Post) on samples taken during construction (144 total tests). Thirty-two additional samples were also tested by five other laboratories with varying amounts of replication; PTSi performed the second highest number of tests at thirteen. Of these thirty-two tests, all but five were within 0.019 of the mean value measured by MSU (values are italic in Table 6.1).

Using 0.019 as a comparer is debatable, as two different methods (T209 Methods A and B) were used to measure G_{mm} and 0.019 is the d2s multi laboratory precision of T209-05 (this revision of T209 did not provide different d2s limits for mechanical agitation (Method A) and manual agitation (Method B)). T209-10 increased the d2s limits and separated Method A (d2s of 0.024) from Method B (d2s of 0.029). Four of the five values that did not meet d2s at 0.019 were acceptable at 0.024. The d2s limit can be viewed as an acceptable range of two results; note that comparing the average of six tests to one individual test is not fully in line with the intention of d2s multi laboratory precision limits but for purposes of this research it is believed to be reasonable (likely somewhat conservative) by the authors.

Table 6.1. G_{mm} Test Results for Strips 1 to 12

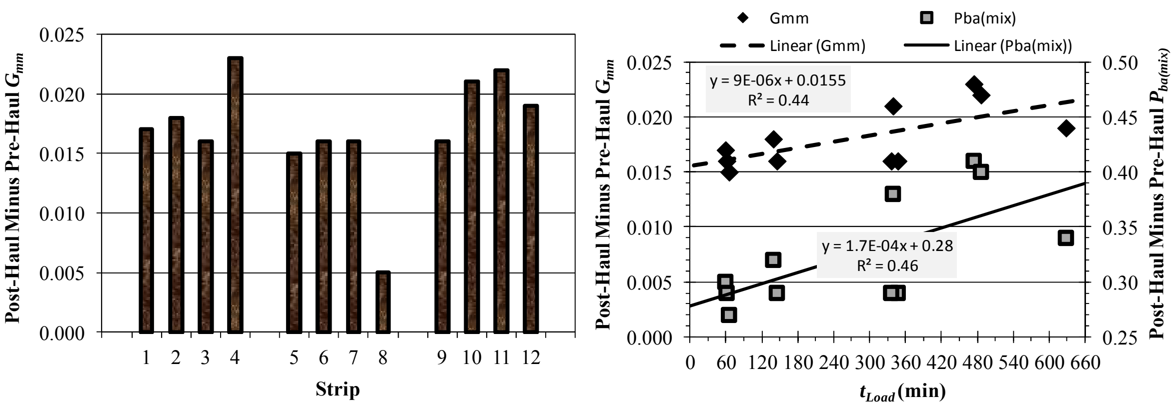
	MSU Test Results							Other Laboratory Results				
Strip	**1**	**2**	**3**	**4**	**5**	**6**	***Avg***	**1**	**2**	**3**	**4**	**5**
1-Pre	2.395	2.391	2.398	2.395	2.396	2.396	***2.395***	2.387[A]	2.392[B]			
1-Post	2.411	2.415	2.416	2.407	2.411	2.410	***2.412***	2.406[B]	2.410[M]	2.424[P]	2.414[P]	*2.381*[P]
2-Pre	2.406	2.399	2.409	2.404	2.412	2.400	***2.405***					
2-Post	2.428	2.431	2.425	2.417	2.413	2.424	***2.423***					
3-Pre	2.406	2.401	2.400	2.402	2.407	2.411	***2.405***					
3-Post	2.425	2.422	2.425	2.417	2.418	2.420	***2.421***					
4-Pre	2.396	2.397	2.409	2.403	2.409	2.398	***2.402***					
4-Post	2.425	2.420	2.429	2.440	2.423	2.412	***2.425***	*2.404*[P]	2.432[P]			
5-Pre	2.388	2.398	2.396	2.394	2.389	2.392	***2.393***	2.379[A]				
5-Post	2.412	2.403	2.409	2.411	2.406	2.406	***2.408***	2.403[B]	2.415[E]	*2.431*[P]	2.413[P]	
6-Pre	2.398	2.391	2.395	2.392	2.399	2.392	***2.395***	2.400[E]				
6-Post	2.409	2.407	2.415	2.413	2.416	2.407	***2.411***	2.417[E]				
7-Pre	2.398	2.394	2.400	2.391	2.392	2.393	***2.395***					
7-Post	2.410	2.413	2.407	2.407	2.417	2.413	***2.411***	2.418[E]				
8-Pre	2.411	2.408	2.403	2.412	2.405	2.396	***2.406***					
8-Post	2.414	2.417	2.409	2.404	2.412	2.411	***2.411***	2.417[E]	*2.388*[P]	2.419[P]		
9-Pre	2.380	2.387	2.385	2.385	2.380	2.383	***2.383***	2.377[A]				
9-Post	2.401	2.394	2.403	2.397	2.402	2.399	***2.399***	2.395[B]	2.398[E]	2.401[P]	2.403[P]	
10-Pre	2.384	2.387	2.396	2.386	2.390	2.382	***2.388***	2.397[E]				
10-Post	2.411	2.422	2.402	2.398	2.412	2.408	***2.409***	2.414[E]				
11-Pre	2.391	2.384	2.394	2.383	2.392	2.388	***2.389***					
11-Post	2.409	2.409	2.413	2.416	2.408	2.410	***2.411***	2.413[E]				
12-Pre	2.399	2.393	2.400	2.388	2.386	2.396	***2.394***					
12-Post	2.406	2.412	2.411	2.411	2.418	2.419	***2.413***	2.419[E]	2.396[M]	*2.391*[P]	2.423[P]	

--PTSi lab mixed sample with binder B2 mixed at 160 C had a G_{mm} of 2.415 after a 90 minute short term age.
--Strips 1 to 8 used binder B2 and strips 9 to 12 used binder B3.
--A = Test conducted by APAC Columbus (truck where sample was taken was not recorded, but was pre-haul).
--B = Test conducted by Burns Cooley Dennis, Inc.
--E = Test conducted by MeadWestVaco.
--M = Test conducted by MDOT central laboratory.
--P = Test conducted by PTSi.

The d2s discussion in the previous paragraph indicates that the strip 1-Post value of 2.381 measured by PTSi is likely incorrect due to a bad sample, equipment malfunction, or similar. PTSi ran two additional samples of strip 1-Post and obtained values in line with the other laboratories further increasing the likelihood 2.381 was an incorrect value. More importantly, comparing MSU test results with those of the other five laboratories in the general context of d2s limits indicates the MSU test results are reasonable.

Additional testing was performed using the dry back method to determine if the Table 6.1 values need to be reduced due to moisture being pulled into the asphalt mixture during testing. MSU and PTSi performed eleven tests using the dry back method and the G_{mm} was, on average, 0.005 less when using the dry back method. For the range of average G_{mb} (presented later in the report) and G_{mm} (Table 6.1) values considered in this test program, the air voids would reduce 0.2% for all data presented when rounded to one decimal place. This is not a meaningful change considering the goals of this research, and as a result the average MSU test results (shown in bold and italics in Table 6.1) were used for all air void calculations in the remainder of this report.

Figure 6.2a plots the change in G_{mm} that occurred in the Table 6.1 data for each test strip. The change in G_{mm} that occurred for strip 8 is immediately observed to be lower than the other eleven strips. Upon examination of Table 6.1, the pre-haul value of 2.406 appears higher than the pre-haul values of strips 5 to 7 (2.393 to 2.395) though it is well within 0.019 when compared to strips 5 to 7 which were made the same day. All pre-haul samples were taken from one location in each truck and the six tests conducted did not deviate an appreciable amount. As a result, no additional information was available to evaluate the strip 8 pre-haul G_{mm}, but it should be noted that it is possible it is approximately 0.01 too high (reducing the strip 8 G_{mm} from 2.406 to 2.396 would reduce the air voids for the range of G_{mb} values in this research by 0.4% when rounded to one decimal place).



a) Data by Test Strip *b) Data as a Function of t_{Load}*

Figure 6.2. Haul Distance Effects on G_{mm} and $P_{ba(mix)}$

Figure 6.2b plots change in G_{mm} and change in absorbed asphalt on a mix mass basis ($P_{ba(mix)}$) that occurred during hauling (i.e. post-haul value minus pre-haul value). Data from strip 8 was not plotted in Figure 6.2b since it was noticeably out of line with the other eleven test strips. G_{mm} and $P_{ba(mix)}$ were not plotted directly as a function of haul time (as G_{mm} was plotted in Figure 6.1) since each day of production had a somewhat different pre-haul G_{mm} which could skew the data and prevent the effects of interest from being observed. $P_{ba(mix)}$ was determined by calculating the effective specific gravity (G_{se}) and effective binder content (P_{be}) using the total asphalt content (P_b), aggregate bulk specific gravity (G_{sb}), and asphalt binder specific gravity (G_b) provided on the mix design alongside measured G_{mm} values.

Figure 6.2b shows G_{mm} and $P_{ba(mix)}$ increasing with haul time, but the rate of increase is not substantial. Figure 6.2a and 6.2b indicate that most of the asphalt absorption occurred within an hour after the mix was loaded into trucks (t_{Load}). Between t_{Load} of 60 and 660 min, the Figure 6.2b trendline equation predicts 0.1% asphalt absorption on a mix mass basis. If emergency responders were so inclined, the data suggests that adding 0.1 to 0.2% virgin binder to the mix design value would have easily compensated for the additional haul distances in terms of asphalt absorption. The authors are of the position that up to 0.2% asphalt addition to the mix design should be allowed for long haul distances in emergency situations (the small amount of additional asphalt would help field compaction and should have a minimal effect on mixture stability as a considerable portion of the added asphalt is expected to be absorbed during the long haul).

Normal MDOT operations are to obtain a G_{mm} sample from the truck (pre-haul) and to age this sample in the contractor's laboratory the day the mix is placed. Mix is not sampled from the paver. Sections 5.4.2 and 5.6 of MDOT (2006) state that oven temperatures during aging are 5 to 10 C higher than the compaction temperature and that the mix should be heated 30 to 120 min prior to G_{mm} testing.

For this project, pre-haul mix was sampled in metal buckets and as needed temporarily placed in ovens while materials were batched to make SGC specimens and for G_{mm} testing. Time and temperature records were not kept, but it is estimated these materials would have been hot slightly less than 30 minutes. Overall, pre-haul G_{mm} values should be reasonably close to those from MDOT protocols (the mix was designed for use with MDOT protocols), and as a result they are used as the primary reference in this report.

6.3 Asphalt Content Test Results

Asphalt content test results are provided in Table 6.2. Extraction asphalt contents were lower than Ignition or Nuclear asphalt contents, but this is not uncommon with high absorption aggregates as removing all asphalt from the aggregate pores can be difficult. The average asphalt contents measured for strips 1 to 4, 5 to 8, and 9 to 12 with ignition and nuclear methods were 5.5%, 5.4%, and 5.6%, which are all within 0.2% of the mix design and within normal field production operational tolerances. Table 6.2 suggests there were no meaningful differences between asphalt contents relative to the mix design or between days of mix production; extraction data also supports this position.

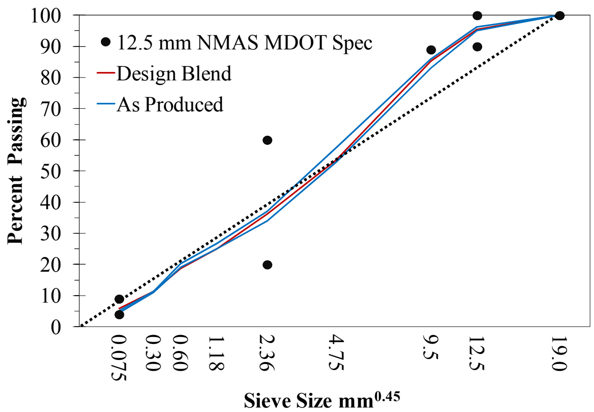
Table 6.2. Percent Asphalt Content Test Results

Strip	PTSi (Extraction)	BCD (Extraction)	PTSi (Ignition)	BCD (Ignition)	APAC (Nuclear)
P	4.4	---	5.5	---	5.4
1 to 4	4.9	4.7	5.3	5.8	5.4
5 to 8	4.9	5.0	5.3	5.1	5.7
9 to 12	4.8	4.9	5.5	5.7	5.6

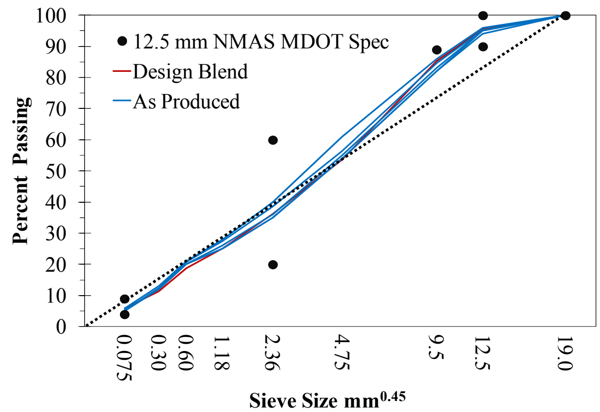
Note: Mix design asphalt content was 5.4%.

6.4 Gradation Test Results

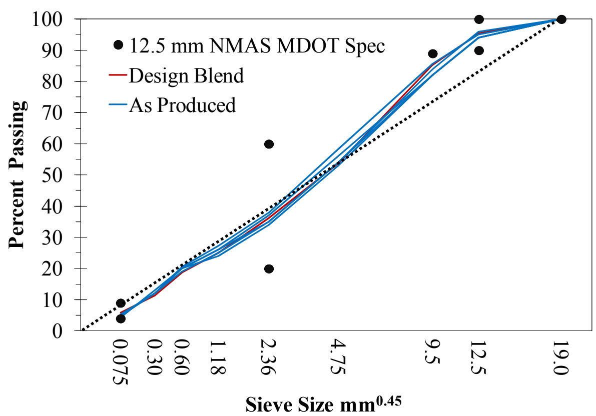
Figure 6.3 plots gradation test results. Generally speaking, the as produced mixes fell below the maximum density line after the 4.75 mm sieve and followed the design blend. There are slight humps in the curves around the 0.6 mm (No. 30) sieve; humps in the finer portion of a gradation can be a sign of potential for tenderness in some cases. The 2.36 mm sieve ranged from 31 to 40% passing (design value was 36%), which is reasonable consistency between test sections. Some differences were observed at the 0.075 mm (No. 200) sieve, but they were not major differences. Table 6.3 summarizes percent passing the 0.075 mm sieve (i.e. fines). Generally speaking, fines contents were within 0.5% of each other for the different test days and they were 0.5 to 1.0% below the design fines content of 5.9%. For purposes of comparison, the gradation was considered the same between all test strips. Fines contents slightly lower than the design value, however, could have a modest effect on compactability of all mixes. Cooley and Williams (2009) reported a small change in air voids with change in fines ($P_{0.075}$) content; higher fines produced lower air voids. With this relationship, all in place air voids reported in this document would be slightly lower if the design fines content had been achieved.



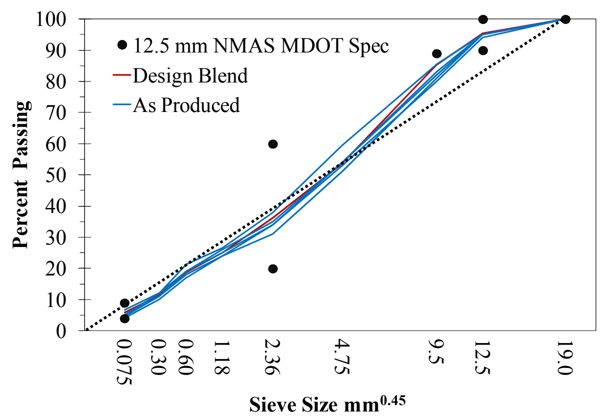
a) Strip P



b) Strips 1 to 4



c) Strips 5 to 8



d) Strips 9 to 12

Figure 6.3. Gradation Test Results

Table 6.3. Fines Content Summaries

Strip	Range (%)	Average (%)
P	4.5 to 5.2	4.9
1 to 4	5.0 to 5.8	5.3
5 to 8	4.3 to 5.3	4.9
9 to 12	4.1 to 6.7 (*5.2*)	5.1 (*4.7*)

--6.7 could be outlier, values in parenthesis are with 6.7 removed.
--design fines content was 5.9%.

6.5 Binder Grading Test Results

Table 6.4 provides binder grade results. SuperPave™ performance grade characteristics were determined in accordance with Table 1 provided in AASHTO M320. Binder grades were performed for all thirteen test strips from mix sampled at the paver (post-haul), and binder sampled from the delivery tanker was also graded prior to use. Strip P was graded for information purposes only. The analysis in the remainder of this section focuses on the *HMA*, *Foam*, and *Additive* grades obtained on strips 1 to 12 (i.e. binders B2 and B3).

Table 6.4. Binder Grade Results

Strip P	O DSR G*/sinδ kPa(°C)	RTFO DSR G*/sinδ kPa(°C)	PAV DSR G*sinδ kPa(°C)	BBR Stiffness MPa(°C)	BBR m-Value (---)	True Grade (°C)
PG67-22 (B1)	1.9(64)	2.8(70)*	4.2(22)	114(-12)	0.331(-12)	69.2-26.0
Strip P-1.1 hr haul	---	2.4(82)	4.2(22)	85(-12)	0.338(-12)	82.8-26.2
HMA						
PG67-22 (B2)	1.4(64)	4.2(64)*	3.8(25)	152(-12)	0.306(-12)	66.9-22.8
Strip 1-1.0 hr haul	-	3.4(76)	5.1(25)	173(-12)	0.288(-12)	79.7-20.0
Strip 2-2.3 hr haul	-	4.2(76)	5.2(25)	159(-12)	0.292(-12)	81.3-20.9
Strip 3-5.8 hr haul	-	4.2(76)	5.2(25)	164(-12)	0.292(-12)	81.3-20.8
Strip 4-7.9 hr haul	-	4.3(76)	5.3(25)	160(-12)	0.291(-12)	81.7-20.1
Foam						
PG67-22 (B2)	1.4(64)	4.6(64)*	3.8(25)	152(-12)	0.306(-12)	66.9-22.8
Strip 5-1.1 hr haul	-	3.1(76)	4.4(25)	154(-12)	0.298(-12)	78.8-21.7
Strip 6-2.4 hr haul	-	3.3(76)	4.0(25)	127(-12)	0.309(-12)	79.2-23.3
Strip 7-5.6 hr haul	-	3.5(76)	3.8(25)	120(-12)	0.305(-12)	79.9-22.8
Strip 8-8.4 hr haul	-	3.7(76)	5.2(25)	162(-12)	0.295(-12)	80.1-21.1
Additive						
PG67-22 (B3)	1.7(64)	4.6(64)*	3.8(25)	152(-12)	0.306(-12)	69.5-22.6
Strip 9-1.0 hr haul	-	2.8(76)	5.0(25)	169(-12)	0.297(-12)	78.1-21.6
Strip 10-5.7 hr haul	-	4.1(76)	4.3(25)	131(-12)	0.312(-12)	81.1-23.7
Strip 11-8.1 hr haul	-	4.3(76)	3.7(25)	106(-12)	0.318(-12)	81.5-24.0
Strip 12-10.5 hr haul	-	3.5(82)	6.0(25)	198(-12)	0.281(-12)	85.9-18.8

* These values were obtained from RTFO aged binders, all other values in the RTFO column are on binders as extracted from mixtures which were not RTFO aged.

-- The B1 sample graded was obtained a few weeks after the 1.1 hr haul, i.e. the binders came from different tanker loads.

The supplied binder has a continuous high grade of up to 69.5 which is almost one grade higher than the LTPPBind recommended grade of PG64. Haul times up to 8.4 hrs for all three mixtures produced true grades ranging from a high temperature grade limit of 81.7 to a low temperature grade limit of -24.0. The overall stiffness increase after one hour of hauling was approximately equivalent to 1.5 times the RTFO stiffness of the base binder, with stiffness increasing to a maximum approximate equivalent stiffness of 1.8 times the RTFO stiffness up to 8.4 hrs of haul time. The noted increase in binder stiffness due to haul time duration is considered to be comparable to normal binder stiffening during hot mix production and placement.

Haul times up to 8.4 hrs did not reveal deleterious binder performance effects. The data presented indicates that even for non-emergency conditions transport and placement of asphalt within an eight hour time period would have no detrimental effects on in place mixture performance. The *Foam* and *Additive* mixtures exhibited modestly better low temperature properties than the *HMA* (on the order of 2 ^{o}C lower true grades). The *Additive* hauled 10.5 hrs exhibited noticeably less desirable binder grade properties than the 8.1 hrs haul as the high temperature grade increased 4.4 ^{o}C and the low temperature grade decreased 5.2 ^{o}C. For emergency paving this is not problematic, but for permanent conditions this is not desirable. Overall, binder grade results revealed no problems for emergency paving.

6.6 Workability Test Results

Figure 6.4 plots workability results. Mixtures exhibiting lower torque values are considered more workable, which is an indication of how the materials can be handled and compacted. Pre-haul material was tested for each mix type (*HMA*, *Foam*, and *Additive*), and post-haul material was tested for strips 3, 7, and 10 since their haul times were practically identical. Measured values are plotted for each pre-haul and post-haul case, as is the change in workability (i.e. post-haul minus pre-haul at each temperature) that occurred for each mix type.

The change in torque was noticeably less for *Additive* (approximately 3 N-m compared to 5 N-m), while *HMA* and *Foam* torque changes were essentially the same (approximately 5 N-m). Indications are that the *Additive* mixture is generally more workable than the *HMA* and *Foam* mixtures after a long haul. From these results it appears that introduction of additive technology marginally improved the workability for long haul distances, but the workability did not decrease drastically for any of the mixes tested. Loss of workability does not appear to be a major concern for long haul distances based on the data presented herein.

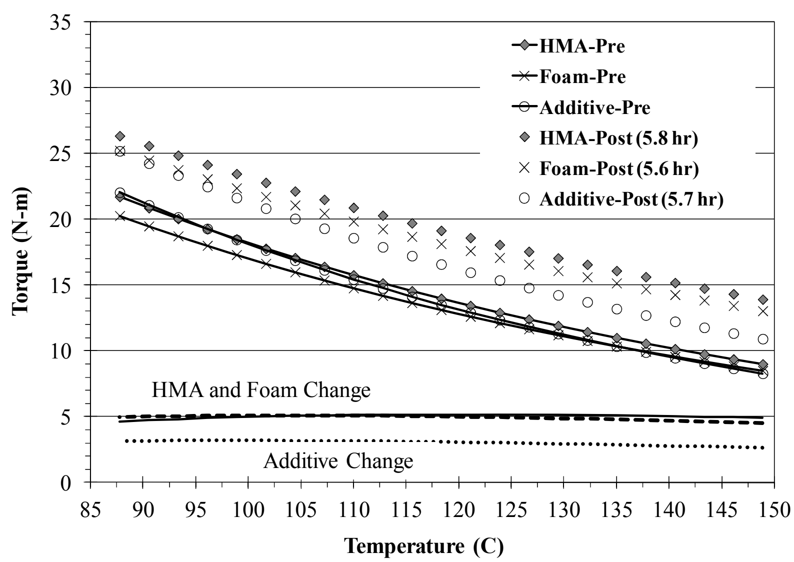


Figure 6.4. AWD Workability Test Results

6.7 In Place Air Voids and Mix Design Verification Test Results

Mix design verification test results are provided in Table 6.5, with data combined per day of mix production. Strip P air voids were noticeably higher than strips 1 to 12. Asphalt content and gradation data presented in sections 6.3 and 6.4 do not explain the differences. All testing was performed on one sample, and test replication was lower than for strips 1 to 12 where four samples were pulled (one per truck).

Strip 5 to 8 air voids were noticeably lower than strips 1 to 4 or strips 9 to 12. Lower mix design verification air voids typically translate to lower in place air voids. One of the strip 5 T166 values was 2.9%, and both of the strip 7 T166 values were 2.5%. The highest air void level measured with T166 in strips 1 to 12 was 4.2%. Asphalt content and gradation data presented in sections 6.3 and 6.4 do not explain the differences. Overall the mix design verification results for strips 1 to 12 were within a generally accepted range of 3 to 5%, but strips 5 to 8 were on the very low end of this range.

Table 6.5. Mix Design Verification Test Results (Compacted to 65 Gyrations)

	AASHTO T331			AASHTO T166		
Strip	**Avg**	**Stdev**	***n***	**Avg**	**Stdev**	***n***
P	6.0	---	2	5.1	---	3
1 to 4	3.9	0.39	8	3.6	0.40	10
5 to 8	3.3	0.29	8	3.0	0.28	10
9 to 12	3.8	0.21	8	3.5	0.17	10

-- Seven of the T166 tests were from APAC, and the remaining tests were from MSU. Values between MSU and APAC were in agreement except strip P, where T166 values were 4.5 APAC and 5.4 (twice) from MSU.

-- Air voids were calculated with pre-haul G_{mm} values.

Table 6.6 summarizes in place air voids for all thirteen test strips. In place air voids were calculated with pre-haul and post-haul G_{mm} values alongside G_{mb} values measured with T331 and T166. Four in place air void calculations resulted from the two G_{mm} and the two G_{mb} calculation methods, which was discussed in more detail in Section 3.5.9. Measuring G_{mb} with T166 and measuring G_{mm} on pre-haul material aligns most closely with typical MDOT practice (see Section 6.2), and as a result the majority of the discussion was made regarding $V_{a\text{-}T166\text{-}Pre}$ data.

With regard to emergency paving, air void levels for all test strips and test zones were adequate. Wheel tracking presented in the remainder of this chapter verifies adequacy of all test strips for emergency paving. In place air voids ($V_{a\text{-}T166\text{-}Pre}$) ranged from 6.8 to 11.6%. The goal of the research team was to be unable to compact the last test strip of a given type (i.e. strips 4, 8, and 12) to an adequate level. Data presented in Table 6.6 indicates this goal was partially but not fully met as the last strip of each test day could be compacted to 9.1 to 10.0% air voids for either zone 1 or zone 2. This is extremely encouraging for emergency paving as the haul times for strips 4, 8, and especially 12 are longer than anticipated for even an emergency event. Several state departments of transportation allow air voids less than 10% to remain in place (a pay penalty is typically applied for higher air voids, but the public is allowed to travel on the pavements for a permanent application).

Table 6.6. In Place Air Voids Test Results Measured on Cores

			AASHTO T 331						AASHTO T 166					
			G_{mb}		$V_{a\text{-}T331\text{-}Pre}$ (%)		$V_{a\text{-}T331\text{-}Post}$ (%)		G_{mb}		$V_{a\text{-}T166\text{-}Pre}$ (%)		$V_{a\text{-}T166\text{-}Post}$ (%)	
Zone	**Strip**	***n***	**Avg**	**Stdev**	**Avg**	**Stdev**	**Avg**	**Stdev**	**Avg**	**Stdev**	**Avg**	**Stdev**	**Avg**	**Stdev**
---	P	12	2.145	0.016	10.4	0.65	10.8	0.65	2.155	0.007	10.0	0.28	10.4	0.28
1	1	31	2.179	0.019	9.0	0.81	9.7	0.80	2.201	0.013	8.1	0.54	8.8	0.53
	2	30	2.161	0.021	10.2	0.86	10.8	0.86	2.185	0.018	9.2	0.76	9.8	0.75
	3	30	2.148	0.026	10.7	1.09	11.3	1.08	2.176	0.026	9.5	1.09	10.1	1.09
	4	30	2.083	0.031	13.3	1.30	14.1	1.29	2.133	0.024	11.2	1.01	12.0	1.00
	5	30	2.197	0.019	8.2	0.77	8.8	0.77	2.220	0.015	7.2	0.62	7.8	0.62
	6	30	2.193	0.025	8.4	1.05	9.0	1.04	2.215	0.019	7.5	0.79	8.1	0.79
	7	31	2.147	0.022	10.4	0.93	10.9	0.92	2.172	0.013	9.3	0.54	9.9	0.54
	8	31	2.141	0.031	11.0	1.30	11.2	1.30	2.163	0.024	10.1	0.98	10.3	0.98
	9	30	2.131	0.038	10.6	1.61	11.2	1.60	2.161	0.029	9.3	1.22	9.9	1.21
	10	30	2.101	0.037	12.0	1.53	12.8	1.52	2.142	0.027	10.3	1.12	11.1	1.11
	11	30	2.111	0.034	11.7	1.44	12.5	1.43	2.149	0.026	10.0	1.09	10.9	1.08
	12	34	2.125	0.025	11.2	1.03	11.9	1.02	2.156	0.026	10.0	1.09	10.7	1.09
2	1	29[a]	2.170	0.016	9.4	0.67	10.0	0.67	2.196	0.014	8.3	0.58	9.0	0.57
	2	31	2.169	0.020	9.8	0.82	10.5	0.81	2.192	0.017	8.8	0.72	9.5	0.72
	3	30	2.177	0.037	9.5	1.56	10.1	1.55	2.198	0.030	8.6	1.24	9.2	1.23
	4	29[a]	2.133	0.028	11.2	1.18	12.0	1.16	2.165	0.022	9.9	0.92	10.7	0.91
	5	30	2.211	0.012	7.6	0.52	8.2	0.52	2.231	0.010	6.8	0.41	7.3	0.41
	6	30	2.203	0.020	8.0	0.82	8.6	0.82	2.222	0.016	7.2	0.67	7.8	0.67
	7	30	2.166	0.025	9.6	1.02	10.2	1.02	2.180	0.027	9.0	1.13	9.6	1.13
	8	30	2.169	0.037	9.8	1.53	10.0	1.53	2.186	0.029	9.1	1.20	9.3	1.19
	9	30	2.172	0.032	8.9	1.36	9.5	1.35	2.190	0.026	8.1	1.08	8.7	1.07
	10	29[a]	2.161	0.036	9.5	1.53	10.3	1.51	2.186	0.027	8.4	1.14	9.2	1.13
	11	31	2.093	0.033	12.4	1.39	13.2	1.38	2.138	0.025	10.5	1.06	11.3	1.05
	12	34	2.071	0.046	13.5	1.92	14.2	1.91	2.115	0.031	11.6	1.28	12.3	1.27

a: one of the specimens was damaged in the laboratory and was discarded.

It is very important to interpret the field compaction data in this report in the context that the research team intentionally did not make several adjustments to field compaction patterns that would have very likely resulted in lower in place air voids. The rationale was to simulate a disaster environment and pre-select one general pattern and use it throughout the test program with no adjustments other than the number of passes performed. For an actual project, adjustments would occur in the very early stages of emergency paving that would very likely improve density relative to the first attempt for a given mix and haul distance. Relative to the options available, compaction was performed within a fairly narrow range.

Compaction only occurs when there is sufficient confinement to prevent particles from moving laterally. Lateral movement can occur with steel wheel rollers, and subsequent reduction in density can occur from additional compaction (an example plot is provided by Brown et al. 2009). Pneumatic tire rollers are not prone to this behavior, which is one reason they are useful for providing additional density after use of a vibratory steel wheel roller. In that the mixes placed for this project experienced some lateral movement under steel wheel rollers, use of a pneumatic roller would have very likely reduced in place air voids.

Compactor roller patterns are usually developed for the mix and construction practices used since selecting the wrong force, rolling too fast, and making too many vibratory passes can be problematic. Rolling speed was not measured in this project, and the vibration settings were constant. The number of vibratory passes varied between zone 1 and zone 2. One of the motivations for varying the number of vibratory roller passes between zones 1 and 2 was to observe how the different mixes (*HMA*, *Foam*, and *Additive*) responded to compaction after different haul times. Strips 1 to 10 either responded favorably to additional vibratory passes (i.e. air voids decreased) or they were not meaningfully affected by additional vibratory passes (i.e. air voids were essentially the same). Interestingly, strips 11 and 12 responded negatively to additional vibratory passes (i.e. air voids increased). Strips 11 and 12 would likely have benefitted from roller pattern modifications; strip 12 in particular where additional vibration increased air voids by an average value of 1.6%.

Individual performance of *HMA* (strips 1 to 4), *Foam* (strips 5 to 8), and *Additive* (strips 9 to 12) mixes is discussed in the remainder of this section. The discussion is focused largely on emergency paving, though there are several interesting observations related to permanent paving that are not discussed in any detail. Individual performance was discussed in the context of the test zone with the lowest air voids since adjustments would be made very quickly during paving to lower air voids, and for purposes of this discussion the potential of the mix to be compacted was of primary interest.

Figure 6.5a plots the lowest air void level measured for each test strip (e.g. strip 1 air voids were 8.1% for zone 1 and 8.3% for zone 2 and 8.1% was plotted). In an overall sense air voids increased with haul time for *HMA*, *Foam*, and *Additive* as would be expected. Values of t/NMAS were between 5 and 6 for nine of the twelve strips. Strip P had a very high t/NMAS value of 7.1, which appears to have affected density relative to the other strips. As discussed during review of literature, t/NMAS values greater than 5 may begin to negatively affect compaction. This project targeted t/NMAS values modestly outside conventional levels to evaluate a worst case type of scenario as during a disaster paving multiple lifts would be avoided if possible. It is conceivable that density of all strips would have dropped slightly if, for example, compaction occurred between t/NMAS of 4 to 5. The one data point in the 4 to 5 range, however, was strip 2 and its density was actually slightly higher than strip 3.

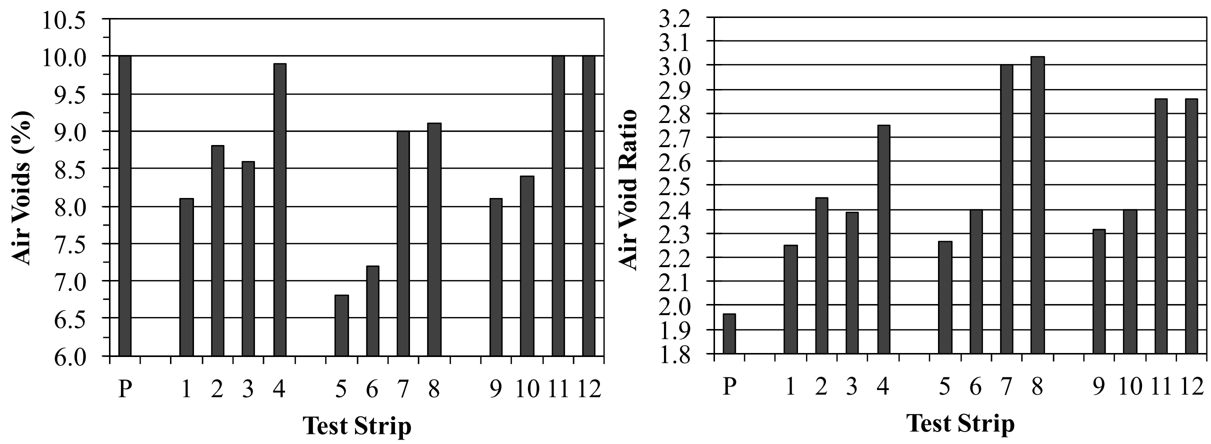


Figure 6.5. Test Strip Air Void Comparisons

If Figure 6.5a is viewed absent other information, it appears *Foam* was the best performer in terms of compaction. Figure 6.5b, however, provides additional information that accounts for the Table 6.5 mix design verification test results. Air void ratio is the in place air voids represented in Figure 6.5a divided by the mix design verification air void results in Table 6.5.

A general guideline is laboratory mix design verification air voids double in place for customary levels of compactive effort (i.e. an SGC compacted specimen with 4% air voids can be compacted to 8% air voids in place under typical construction conditions without excessive compactive effort). This guideline should in no way be taken as absolute, or even highly accurate, but it does highlight the importance of considering mix design verification test results when comparing in place air voids of one mix to another. Lower mix design verification air voids usually translate to lower in place air voids with all other factors remaining constant. Equation 2.2 (Cooley and Williams 2009) refer laboratory compacted specimens during production as $V_{a(P)}$ and their sensitivity analysis reveals in place air voids can change just over 2% due to changes in mix composition.

Strip 1 (*HMA*), strip 5 (*Foam*), and strip 9 (*Additive*) have practically identical air void ratios. Strip 1 and strip 9 have similar mix design verification results, and in place air voids were also the same. It is possible that subtle mix volumetric differences accounted for the difference in strips 1 and 9 versus strip 5, in which case compactability of all three strips was essentially the same, or it is also possible that the *Foam* improved compaction and accounted for the differences.

Construction conditions being less favorable for *Additive* strips 10 to 12 (Chapter 5) should be considered when viewing the data in this section. Strip 12 was hauled 10.5 hr in adverse weather, compacted at a t/NMAS of 6.3, and still achieved 10% air voids. For haul distances of 8 hr or less, there is no compelling case to use any mix type (*HMA*, *Foam*, or *Additive*) over another in terms of in place air voids, though *Foam* would be a logical choice in Mississippi since many asphalt plants are equipped with the necessary equipment. The only product the research team felt comfortable taking to the 10 hr (+) range was the *Additive* mix. Its compaction ability at these haul times was fairly remarkable, especially considering it was not different than *HMA* at conventional haul times.

6.8 APA Test Results

Figure 6.6 plots plant mixed (*PM*) strip P and lab mixed (*LM*) APA test results. Temperatures during mixing are shown in parenthesis for *LM* specimens. Some specimens were compacted in the Superpave Gyratory Compactor (SGC), while others were field compacted (*FC*). PM-SGC specimens were used to estimate the effect of air voids on rut depth over a wide range of voids. Rut depths doubled from 2.1 mm at 3% voids to 4.2 mm at approximately 10% voids, and were 5 mm at 12% voids. Rut depths of 5 mm or less up to 12% air voids is a stable mix. Specimens mixed at 160 C in the laboratory had comparable rut depths to the *PM* specimens that were mixed at 174 C. When the mixing temperature was reduced in the laboratory to 129 C, rut depths increased 3 mm on average. Field compacted (*FC*) specimens were not as rut resistant as SGC compacted specimens. Two of the three *FC* specimen data points shown in Figure 6.6 were tested just a week after placement. One of the FC data points was stored in ambient laboratory conditions for approximately 7.5 months while exposed to laboratory air but not exposed to moisture or sunlight. The rut depth of the specimen stored for some time was within 0.3 mm of one of the data points tested a few days after placement. This test was conducted since specimens were stored several months in some cases prior to rut testing due to the volume of testing performed for this project. The data presented in Figure 6.6 indicates rut depths were representative of those soon after construction.

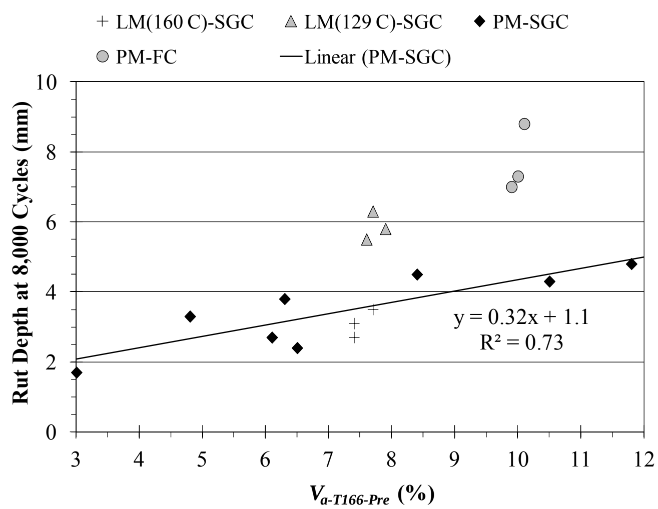


Figure 6.6. Strip P and Lab Mixed APA Test Results

Figure 6.7 plots APA test results for strips 1 to 12 versus $V_{a\text{-}T166\text{-}Pre}$. Twelve field cores were tested per strip by combining all cores from test zones 1 and 2, sorting air voids in ascending order, and picking six air void levels that encompassed the strip. Two cores from each air void level were tested as a pair in one track of the APA to produce one data point. Generally speaking, the air void levels selected were: lowest air voids, air voids above the lowest values but below the mean of either zone 1 or zone 2, mean values of zones 1 and 2, air voids above the mean values of zones 1 and 2, but below the highest levels, highest air voids measured. Rut depth was the value measured at 8,000 cycles, while rut rate was calculated as the mm change in rutting per 1,000 cycles between 2,000 and 8,000 cycles.

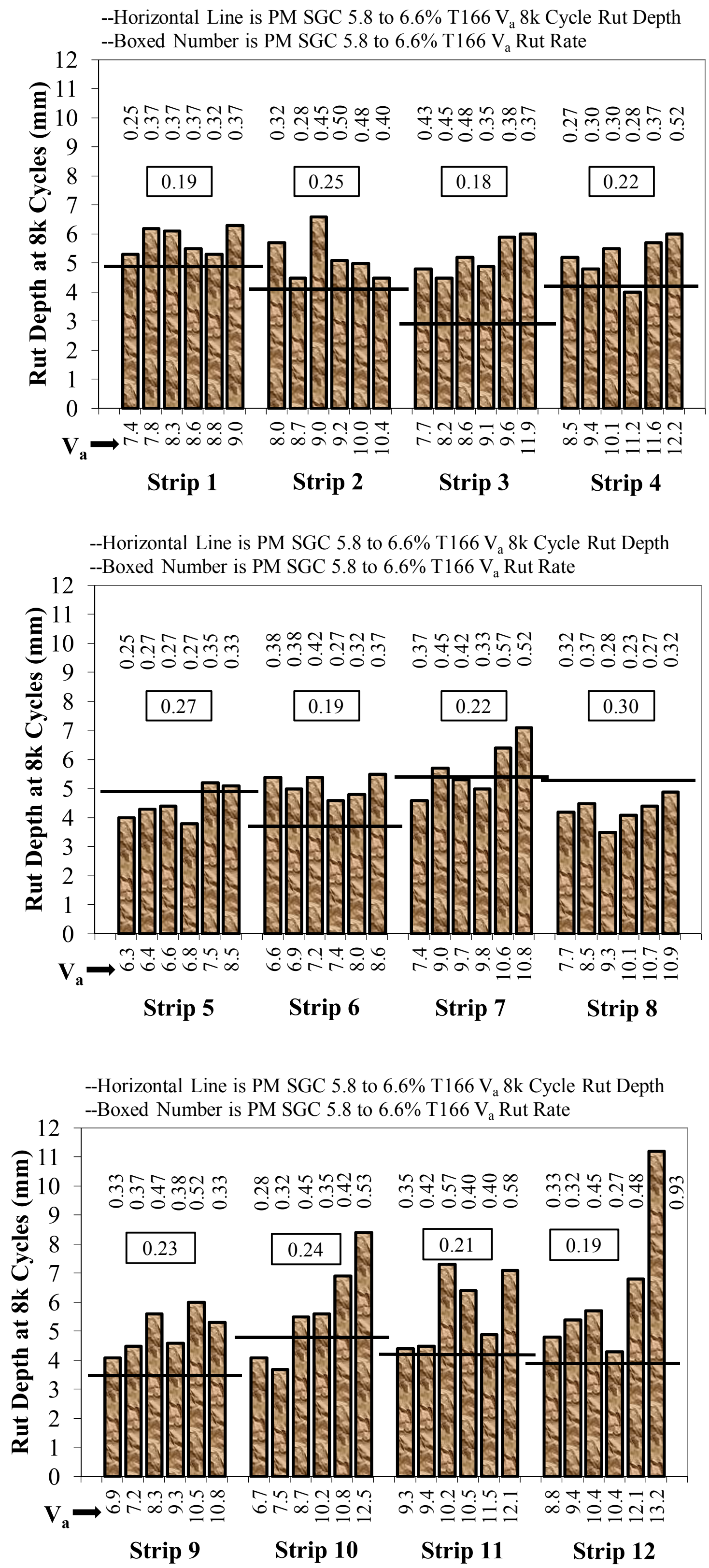


Figure 6.7. APA Rut Test Results for Strips 1 to 12

Rut depths for all strips were easily acceptable for emergency construction. Even if the 8 mm criteria often used for permanent construction is applied to air voids in place, which it usually isn't, only two tests have values above 8 mm (strip 10 has one value at 8.4 mm and strip 12 has one value at 11.2 mm). The highest average rut depth in place for any strip was 6.4 mm for strip 12. There were no meaningful differences between *HMA*, *Foam*, and *Additive* rut depths in place. When the rut depths from each section were averaged, *HMA* values were 5.2 to 5.8 mm for strips 1 to 4, *Foam* values were 4.3 to 5.7 mm for strips 5 to 8, and *Additive* values were 5.0 to 6.4 mm for strips 9 to 12. Rut depths did not appear to be noticeably affected by air voids.

There was no meaningful correlation between average field compacted rut depth or rut rate and laboratory compacted rut depth or rut rate for each test strip. On average, rut depths were 1 mm higher for field compacted specimens relative to lab compacted specimens, and rut rates were 0.16 mm/1000 cycles higher for field compacted specimens. Air voids were higher for field compacted specimens.

6.9 HLWT Test Results

Figure 6.8 plots plant mixed (*PM*) strip P and lab mixed (*LM*) HLWT test results. Temperatures during mixing are shown in parenthesis for *LM* specimens. Some specimens were compacted in the Superpave Gyratory Compactor (SGC), while others were field compacted (*FC*). PM-SGC specimens were used to estimate the effect of air voids on rut depth over a wide range of voids. Rut depths doubled from 2.7 mm at 3% voids to 5.4 mm at approximately 10.5% voids, and were 5.9 mm at 12% voids. HLWT rut depths of 6 mm or less up to 12% air voids after 20,000 passes is a stable mix.

The most stringent criteria mentioned in Chapter 2 was 4 mm of rutting or less after 20,000 passes. This criteria would be applied on specimens with air voids much lower than 12%. An air void level of 6.8% corresponded to 4 mm of rutting, for *PM*-SGC specimens, which is an air void level more representative of typical wheel tracking test conditions. Specimens mixed at 160 C in the laboratory had lower rut depths than the *PM* specimens that were mixed at 174 C (2.4 mm less on average). When the mixing temperature was reduced in the laboratory to 129 C, rut depths increased 4 mm on average.

Field compacted (*FC*) specimens were not as rut resistant as SGC compacted specimens. Two of the four *FC* specimen data points shown in Figure 6.8 were tested approximately three weeks after placement (rut depths of 10.3 and 12.1 mm). The other two *FC* data points were collected from specimens stored in ambient laboratory conditions for 7.5 to 9.5 months while exposed to laboratory air but not exposed to moisture or sunlight (rut depths of 7.4 and 7.7 mm). Rut depth was, on average, 3.6 mm less for the specimen stored for some time compared to those tested approximately three weeks after placement. This test was conducted since specimens were stored several months in some cases prior to HLWT testing due to the volume of testing performed for this project.

The data presented in Figure 6.8 indicates rut depths may have been higher if specimens would have been tested immediately. HLWT findings differed from APA findings in this regard as APA rut testing several months later was within the range of values tested soon after construction. It should be noted that with such a limited amount of data, these findings are not conclusive.

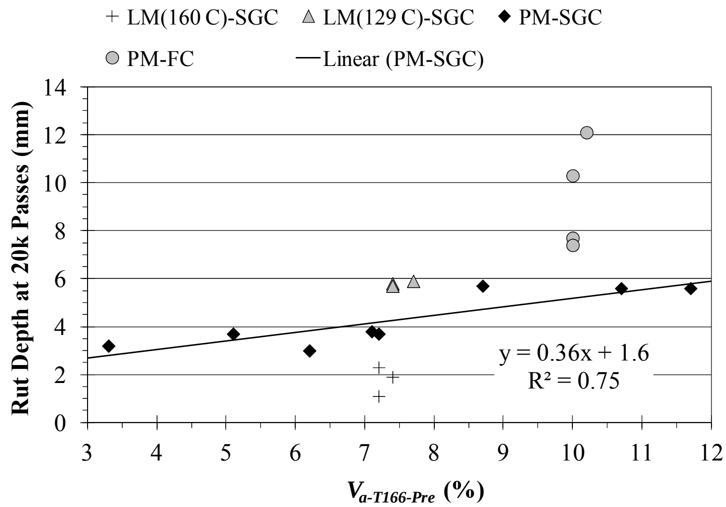


Figure 6.8. Test Strip P and Lab Mixed HLWT Results

Figure 6.9 plots HLWT test results for strips 1 to 12 versus $V_{a\text{-}T166\text{-}Pre}$. Twelve field cores were tested per strip by combining all cores from test zones 1 and 2, sorting air voids in ascending order, and picking six air void levels that encompassed the strip. Two cores from each air void level were tested as a pair in one track of the HLWT to produce one data point. Air voids were selected in the same manner as with APA specimens. The rut depth reported was at 20,000 passes unless otherwise noted. Stripping inflection point (SIP) is a term often associated with the HLWT, which is used as an assessment of stripping potential. No specimens tested in this report had a SIP, which is positive in terms of mix performance.

HLWT rut depths for all strips appear to be acceptable for emergency construction. Of the four criteria presented in Chapter 2, 12.5 mm of rutting prior to 7,500 passes seems the most reasonable for emergency paving since it is the least stringent. Even less stringent criteria might also be acceptable (especially for field compacted materials tested at in place air voids), but for the purposes of this report 12.5 mm of rutting at 7,500 passes was used. None of the specimens tested for strips 1 to 4 (*HMA*) or 5 to 8 (*Foam*) rutted 12.5 mm at 20,000 passes, so their performance is easily acceptable for emergency paving based on the aforementioned criteria. Rut depths averaged for each section ranged from 7.9 to 8.9 mm for *HMA* and 8.4 to 10.2 mm for *Foam*. Nine of the twenty-four sets of *Additive* specimens (38%) experienced 12.5 mm or more rutting. Two of these nine sets failed to reach 7,500 passes prior to 12.5 mm of rutting; strip 10 at 12.3% air voids rutted 12.5 mm at 7,400 passes and strip 12 at 14.0% air voids rutted 12.5 mm at 3,900 passes. While this level of HLWT performance is probably adequate for emergency construction, it is below *Foam* and *HMA* performance.

Rut depths measured on SGC specimens made from mix taken from the paver (value shown in Figure 6.9) were 4.7 to 6.1 mm for *HMA*, 4.7 to 6.5 mm for *Foam*, and 4.1 to 6.5 mm for *Additive*. SGC results were practically identical between mix types. Typically HLWT criteria are applied to laboratory compacted specimens, and as seen all mixes were easily within the criteria for SGC compacted materials. On average, rut depths were 3 mm higher for field compacted specimens relative to lab compacted specimens (only strips 1 to 8 were used in this assessment since no specimens terminated prior to 20,000 passes). Air voids were higher for field compacted specimens.

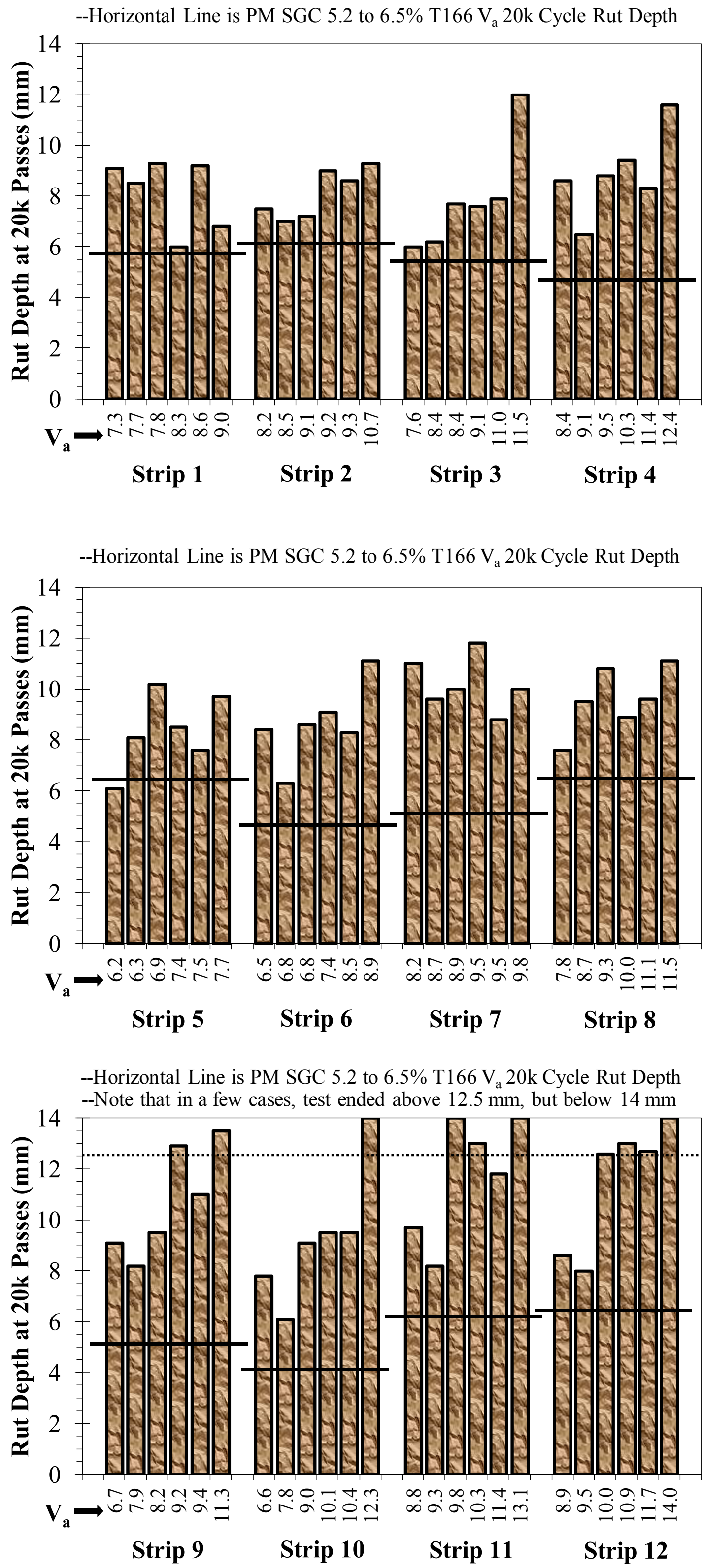


Figure 6.9. HLWT Rut Test Results for Strips 1 to 12

6.10 PURWheel Test Results

Table 6.7 summarizes PURWheel test results, and Appendix A provides more detailed information about each individual test. The strip P laboratory compacted (*LC*) tests were performed without replication, and they are denoted with *LC* in parenthesis. The remaining tests were performed on field compacted (*FC*) specimens, and two replicates were performed for each *FC* specimen. Field compacted specimens are denoted with FC in parenthesis. A total of 58 PURWheel tests were performed.

PURWheel tests were performed, generally speaking, between mid December of 2011 to mid June of 2012. Most replicates were tested within a few days of each other, though strip 2 was a notable exception as replicate tests were performed two to three months apart. The data from strip 2 was therefore used to assess the effects of specimen storage in ambient laboratory conditions for a period of time while exposed to laboratory air but not exposed to moisture or sunlight. Limited APA testing indicated little to no difference of specimen storage time, while limited HLWT testing indicated there might be some differences due to specimen storage time (specimen damage decreased with increased storage time). Strip 2 dry testing of specimens two months apart were in good agreement, and strip 2 wet testing of specimens three months apart was in reasonable agreement considering that more discrepancy between replicate tests was found for some specimens tested days apart (e.g. strip 3). PURWheel testing agreed with APA testing in very general terms. Overall, there was no compelling evidence that specimen storage time had a meaningful effect on results, and the issue was not considered any further.

Table 6.7 is interpreted as follows. $P_{12.5}$ is the number of passes required to achieve 12.5 mm of rutting and is provided in thousands of passes (i.e. 6.5k is 6,500 passes). $P_{12.5}$ is recorded for the wet and dry test and the ratio of wet to dry values expressed as a percentage is denoted $P_{12.5(R)}$. A $P_{12.5(R)}$ value of 100 indicates no moisture effects after 12.5 mm of rutting, while values less than 100 indicate some effect of moisture. Rutting rate (RR_{PW}) was determined by taking the difference in rut depths at 4,000 and 16,000 passes and dividing by the number of cycles (6,000) between these readings. The result was a rutting rate expressed as mm of rutting per 1,000 cycles for direct comparison to APA results.

Rut depths (RD_{PW}) at 4,000 (i.e. 4k), 5,000 (5k), 16,000 (16k) and 20,000 (20k) passes were recorded for both tests. In some instances specimens failed prior to 20,000 passes (or data was otherwise not collected). In these instances dashes were placed in Table 6.7. When a mix replicate did not rut 12.5 mm (in either the wet or dry test), a value of 20,000 (or 20k) was used for $P_{12.5}$. CR is the cohesion ratio, defined as the ratio of wet to dry rut depths at 5,000 passes and values greater than 1.0 indicate moisture effects. The difference in rut depths between wet and dry tests at 5,000 passes is denoted Δ5k with units of mm.

Stripping inflection point (SIP) was defined as the intersection of the creep region and stripping region. The Appendix A figures show SIP graphically, which has units of passes. When a mix did not have a stripping inflection point in a wet test, a value of 20k was used for SIP.

Table 6.7. PURWheel Test Results

		Dry Protocol (PURWheel-dry)						Wet Protocol (PURWheel-wet)						Combined Results		
	App. A			RD_{PW} by Passes (mm)						RD_{PW} by Passes (mm)						
Strip	Table(s)	$P_{12.5}$	RR_{PW}	4k	5k	16k	20k	$P_{12.5}$	SIP	4k	5k	16k	20k	CR	$P_{12.5(R)}$	Δ5k
P (LC)[a]	A.1	20k	0.28	3.1	3.4	4.8	5.6	6.5k	6.0k	7.2	10.2	---	---	3.0	33	6.8
P (LC)[b]	A.2	20k	0.33	3.1	3.2	5.1	5.8	18.7k	17.5k	3.0	3.4	7.5	12.4	1.1	94	0.2
P (LC)[c]	A.3	20k	0.18	2.4	2.4	3.5	3.4	19.2k	16.5k	4.5	4.9	9.8	13.2	2.0	98	2.5
P (FC)	A.4, A.17	9.2k	1.32	8.7	10.0	16.6	18.4	2.1k	---	---	---	---	---	---	23	---
1 (FC)	A.5, A.18	20k	0.57	5.0	5.5	8.4	9.2	8.5k	9.3k	9.0	10.7	---	---	1.9	43	5.2
2 (FC)	A.6, A.19	20k	0.55	4.4	4.9	7.7	8.4	12.7k	10.6k	5.1	5.7	---	---	1.2	64	0.8
3 (FC)	A.7, A.20	20k	0.70	4.5	5.0	8.7	9.4	11.7k	20k	10.0	12.8	---	---	2.6	59	7.8
4 (FC)	A.8, A.21	20k	0.62	5.3	5.8	9.0	9.7	3.2k	---	---	---	---	---	---	16	---
5 (FC)	A.9, A.22	14.4k	1.20	7.6	8.4	14.8	16.5	4.5k	4.5k	---	---	---	---	---	31	---
6 (FC)	A.10, A.23	14.8k	1.13	6.8	7.7	13.6	15.1	6.6k	7.8k	8.9	10.3	---	---	1.3	45	2.6
7 (FC)	A.11, A.24	8.2k	1.53	9.0	10.0	18.2	20.5	4.1k	4.9k	---	---	---	---	---	50	---
8 (FC)	A.12, A.25	11.2k	1.10	7.9	8.8	14.5	16.1	6.7k	7.9k	9.3	10.7	---	---	1.2	60	1.9
9 (FC)	A.13, A.26	19.3k	0.88	6.1	6.6	11.4	12.3	10.6	17.5k	7.8	9.0	---	---	1.4	55	2.4
10 (FC)	A.14, A.27	10.9k	1.50	7.0	8.0	16.0	17.8	3.4k	---	---	---	---	---	---	31	---
11 (FC)	A.15, A.28	17.3k	0.85	6.2	6.9	11.3	12.7	5.2k	5.5k	11.7	12.8	---	---	1.9	30	5.9
12 (FC)	A.16, A.29	16.5k	0.93	6.4	7.1	12.0	12.7	9.5k	14.4k	7.8	8.6	---	---	1.2	58	1.5

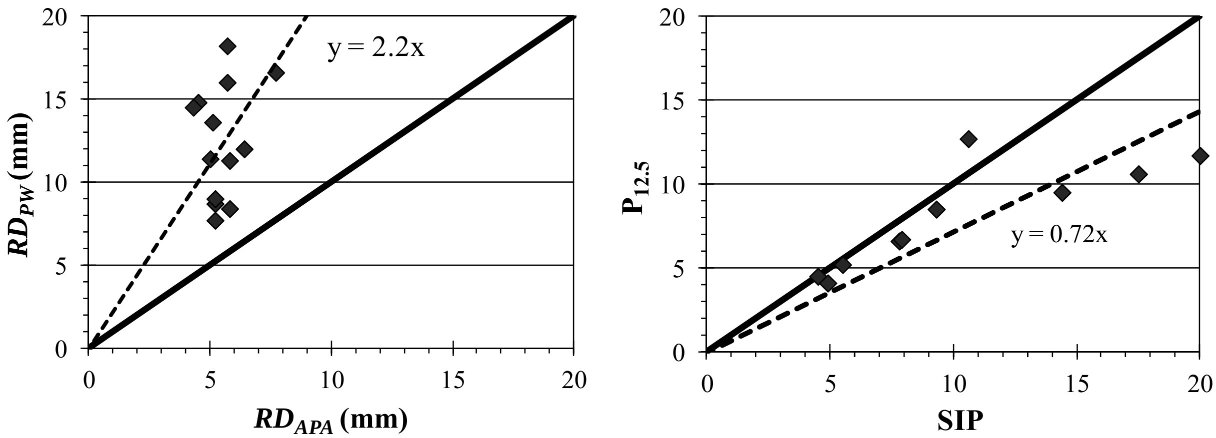
Note: Air voids of these specimens were not measured but correspond to zone 1 data provided in Table 6.6.

a: dry protocol test was after being heated 240 min, and wet protocol test was after being heated 150 min.

b: dry protocol test was after being heated 305 min, and wet protocol test was after being heated 420 min.

c: dry protocol test was after being heated 540 min, and wet protocol test was after being heated 480 min.

For this report, dry rutting was evaluated mostly with rut depths at 16,000 passes (8,000 cycles) and $P_{12.5}$. Figure 6.10a compares rut depths measured by the APA (Figure 6.7 values were averaged for each strip) to PURWheel rut depths at 8,000 cycles. Rut depths measured by the APA were denoted RD_{APA}. PURWheel rut depths were 2.2 times higher than APA rut depths based on a linear trendline with zero intercept, but there was noticeable scatter. APA rut depths were in a narrow band compared to PURWheel rut depths.



a) Dry Protocol Data *b) Wet Protocol Data*

Figure 6.10. Comparison of APA and PURWheel Data

Using only APA data (Section 6.8) led to the assessment that *HMA*, *Foam*, and *Additive* rut depths were not meaningfully different. PURWheel data does not lead to the same assessment. *HMA* rut depths were the best, with values ranging from 7.7 to 9.0 mm (average value for all strips was 8.5 mm). *Additive* rut depths were next lowest, with values ranging from 11.3 to 16.0 mm (average value for all strips was 12.7 mm). *Foam* rut depths were the worst, with values ranging from 13.6 to 18.2 mm (average value for all strips was 15.3 mm). None of these values are necessarily problematic for emergency paving, but the trend is different than the more conventionally used APA test result. *HMA* specimens lasted the full 20,000 passes without a 12.5 mm rut, *Additive* specimens lasted 16,000 passes without a 12.5 mm rut, and *Foam* specimens lasted 12,000 passes without a 12.5 mm rut.

Figure 6.10b compares $P_{12.5}$ and SIP, and shows that overall, stripping inflection points occurred at later pass intervals than 12.5 mm of rutting. In that the lowest $P_{12.5}$ value was 3,200 passes, it is unlikely that stripping would occur during a temporary disaster recovery application. *HMA* took 9,000 passes (overall) to achieve 12.5 mm of rutting in the wet protocol test, *Additive* took 7,200 passes, and *Foam* took 5,500 passes. Interestingly, each mix was able to carry approximately 45% of the passes to 12.5 mm in a wet condition as they could in a dry condition. The same conclusion is reached for practical purposes if the $P_{12.5(R)}$ values for each test strip are averaged.

For purposes of emergency paving, PURWheel testing so nearly simulating loaded truck tires is a strong verification of the feasibility of the concept presented in this report. While there are several potentially interesting items for permanent application discussion that could use the PURWheel data presented in this report, they should not take away from the key point that emergency paving with asphalt is feasible, even for haul distances of several hours. An emergency pavement compacted to even modest levels should last at least a few thousand passes, even in hot and wet conditions.

CHAPTER 7 - SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

7.1 Summary

The primary objective of this report was to perform full-scale testing of hot-mixed and warm-compacted asphalt for disaster recovery purposes (e.g. respond to hurricane damage). After disasters such as hurricanes, power is often out for large distances, which limits the use of conventional construction approaches. Use of hot-mixed and warm-compacted asphalt hauled from a considerable distance (i.e. a location with power and functioning infrastructure) should drastically reduce recovery time by increasing efficiency of all activities associated with response and recovery.

This report was phase 2 of the emergency paving effort. Phase 1 was a laboratory study that ended with two primary questions: 1) can hot-mixed and warm-compacted asphalt be delivered to a location of interest at a temperature of 105 C or higher; and 2) can hot-mixed and warm-compacted asphalt be compacted to 11 to 14% air voids after a very long haul distance? The answer to both questions was yes.

Phase 2 consisted of producing asphalt concrete at a full-scale facility, loading the material into trucks, hauling the material for different amounts of time, and compacting the material into test strips on a parking lot. The process was monitored from production, to transport, to paving, to compacted material properties. Approximately 175 laboratory compacted specimens were tested, alongside approximately 750 field cores and over 100 field sawn slabs.

One coarse graded 12.5 mm Superpave mixture with PG67-22 binder was evaluated. This mix was produced with no binder modifications, and it was also produced with foamed asphalt. The third and final method was to modify the PG67-22 binder with 0.5% of M1 Evotherm 3G™. Plant mixing temperatures were targeted at 160 C for all material produced.

A data acquisition approach was developed that allowed instrumented probes to be inserted into a loaded truck of asphalt concrete and measure temperature from near the edge of the truck bed to near the middle of the mix. The approach was successful and appeared to be as reliable as well accepted thermometer measurements, yet it was able to continuously measure temperatures during hauling. Data collected with the probes provided several fairly unique observations with regard to mixture behavior during transport.

The researchers obtained feedback from end users throughout the project. For example, feedback obtained from MDOT early in the project was that an emergency paving material would be useful. Additionally, some of the authors of this report could be end users as well. As of the completion date for this report, four technical presentations without an associated paper have been given, one peer reviewed conference paper (and presentation) has been published, and one peer reviewed journal article has been accepted for publication. These documents encompass phase 1 and phase 2 emergency paving efforts. Feasibility of the approach was discussed with prospective stakeholders and end users at many of the aforementioned venues. Future activities to interface with potential end users and ensure capability gaps and operational requirements are addressed are planned for the future.

7.2 Conclusions

Several technical oriented items are documented in the report that could be listed as conclusions, but they were largely omitted from this chapter so that the most important points could be reinforced. The overall conclusion of this research is that emergency paving where haul distances of several hours are required is feasible. Haul distances of 8 hr or less were concluded to be implementable as no formidable problems were observed for any of the mixes tested.

Asphalt concrete could be hauled 1.0 to 10.5 hr and be placed with a paver. The mix was subsequently compacted to 6.8 to 11.6% air voids based on AASHTO T166. Testing including workability, binder grading, wheel tracking, and moisture damage revealed no formidable problems for emergency paving. An emergency pavement compacted to even modest levels should last at least a few thousand truck passes.

For haul distances of 8 hr or less, there was no compelling case to use any mix type (traditional hot mixed asphalt, foamed asphalt, or Evotherm 3G™ modified asphalt) over another in terms of in place air voids. Foamed asphalt, though, would be a logical choice in Mississippi since many asphalt plants have the necessary equipment and there is no meaningful cost associated with foamed versus non-foamed asphalt when the equipment is already present. The foamed asphalt would not be expected to be worse than non-foamed (i.e. traditional hot mix) asphalt for compaction, which makes it a logical choice since costs are comparable. Evotherm 3G™ modified asphalt was the only product the research team felt comfortable taking to 10 hr (+) haul times. It's compaction ability at these haul times was fairly remarkable, especially considering it was not different than traditional hot mixed asphalt at conventional haul times.

7.3 Recommendations

The overall recommendation from this research is to use hot-mixed and warm-compacted asphalt concrete as an emergency paving material for disaster recovery applications. The approach has passed laboratory and full-scale testing. A few more specific recommendations are listed below.

- Develop a logistical plan where asphalt producers who are agreeable to performing emergency paving are identified with their state asphalt association. If a disaster occurs, state asphalt associations could be contacted, and in turn contact the appropriate producers. Key asphalt plants could be placed on alert if a somewhat predictable event such as a hurricane were likely to hit an area close enough they could respond. Proper planning could make it where emergency paving began hours after the disaster event ceased, which is considerably faster than any process in effect in present day. A pre-negotiated financial arrangement would also likely be needed.
- Avoid using mixes with a history of tenderness for emergency paving if other mixes are available.
- Fine graded mixes are often more easily compacted than coarse graded mixes, so if a fine graded mix is available, it should be considered for very long haul distances to support disaster recovery. A coarse graded mix was used in this project since they are more common.

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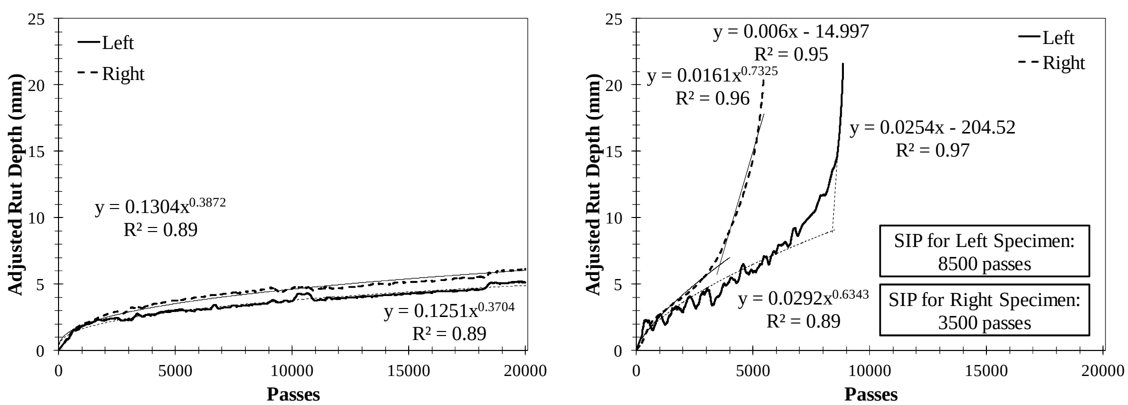
APPENDIX A – PURWHEEL RAW TEST DATA

Table A.1. PURWheel Test Results for P-LS-1 and P-LS-2

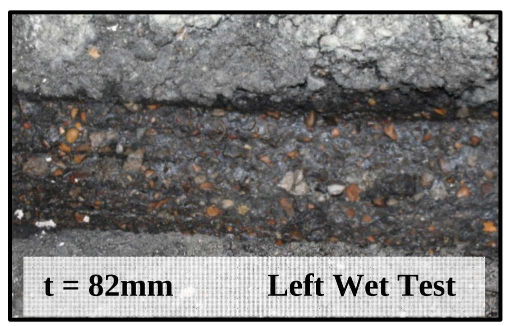
Dry Test (Air Voids 11.0%), Global Test ID 107				Wet Test (Air Voids 11.0%), Global Test ID 105			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.6	250	0.6	250	1.3	250	0.6
500	1.1	500	1.2	500	2.3	500	1.6
1000	1.8	1000	1.9	1000	2.7	1000	2.8
2000	2.3	2000	2.7	2000	2.9	2000	4.0
4000	2.7	4000	3.5	4000	5.3	4000	9.0
8000	3.4	8000	4.5	8000	11.7	5466	20.8**(---)**[1]
12000	3.9	12000	4.7	8860	21.6 **(---)**[1]	---	---
16000	4.4	16000	5.2	---	---	---	---
20000	5.1 **(6.2)**[1]	20000	6.1 **(7.9)**[1]	---	---	---	---

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C   Tire Pressure: 862 kPa   Wheel Load: 178.6 kg



a) Dry Test (t_{Load}=240 min)   *b) Wet Test (t_{Load}=150 min)*

Figure A.1. PURWheel Test Results for P-LS-1 and P-LS-2



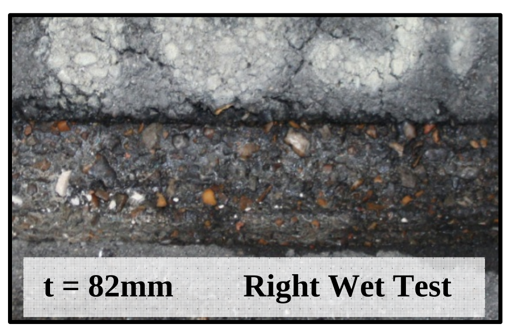
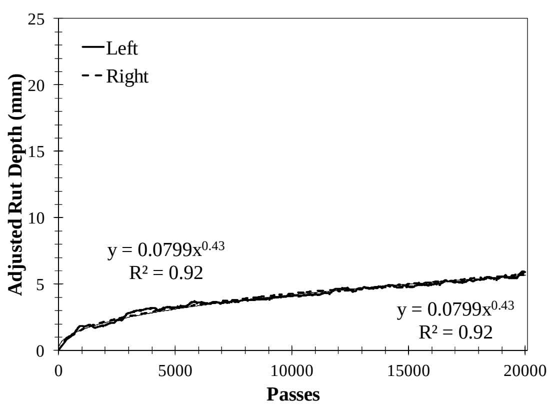


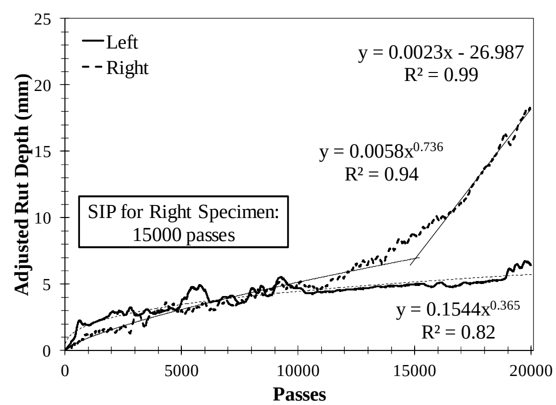
Table A.2. PURWheel Test Results for P-LS-3 and P-LS-4

Dry Test (Air Voids 11.0%), Global Test ID 108				Wet Test(Air Voids 11.7%), Global Test ID 143			
Left Specimen (mm)		Right Specimen (mm)		Left Specimen (mm)		Right Specimen (mm)	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.6	250	0.7	250	0.6	250	0.3
500	1.0	500	1.1	500	1.7	500	0.5
1000	1.8	1000	1.6	1000	1.9	1000	1.2
2000	1.9	2000	2.2	2000	2.9	2000	1.6
4000	3.2	4000	2.9	4000	2.9	4000	3.0
8000	3.8	8000	3.9	8000	4.7	8000	4.2
12000	4.6	12000	4.5	12000	4.6	12000	5.6
16000	5.0	16000	5.1	16000	5.1	16000	9.9
20000	5.9 **(7.3)**[1]	20000	5.7 **(9.3)**[1]	20000	6.4**(6.3)**[1]	20000	18.4**(---)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Dry Test (t_{Load}=305 min)



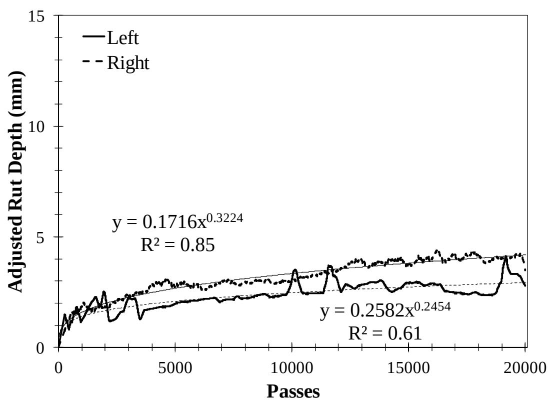
b) Wet Test (t_{Load}=420 min)

Figure A.2. PURWheel Test Results for P-LS-3 and P-LS-4

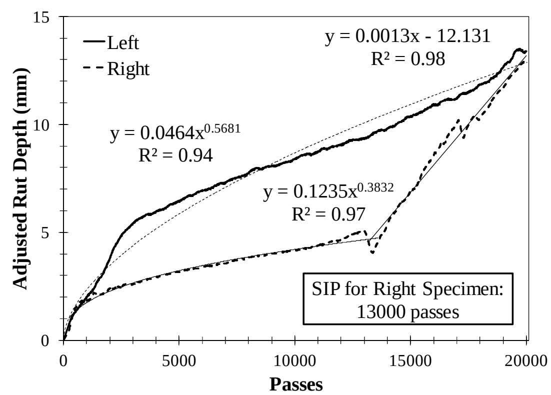
Table A.3. PURWheel Test Results for P-LS-5 and P-LS-6

Dry Test (Air Voids 12.4%), Global Test ID 144				Wet Test(Air Voids 11.3%), Global Test ID 106			
Left Specimen (mm)		Right Specimen (mm)		Left Specimen (mm)		Right Specimen (mm)	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.5	250	0.8	250	0.8	250	0.5
500	1.1	500	1.4	500	1.3	500	1.5
1000	1.2	1000	1.8	1000	2.0	1000	1.8
2000	2.3	2000	1.9	2000	3.8	2000	2.4
4000	1.8	4000	2.9	4000	6.0	4000	2.9
8000	2.2	8000	3.0	8000	7.8	8000	3.8
12000	2.8	12000	3.5	12000	9.1	12000	4.7
16000	2.9	16000	4.0	16000	10.9	16000	8.6
20000	2.8**(4.7)**[1]	20000	4.0**(6.4)**[1]	20000	13.4 **(13.1)**[1]	20000	13.0 **(13.7)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

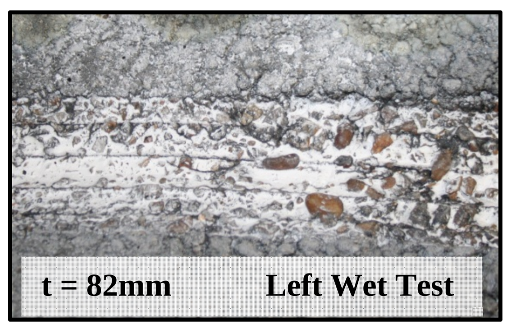


a) Dry Test (t_{Load}=540 min)



b) Wet Test (t_{Load}=480 min)

Figure A.3. PURWheel Test Results for P-LS-5 and P-LS-6



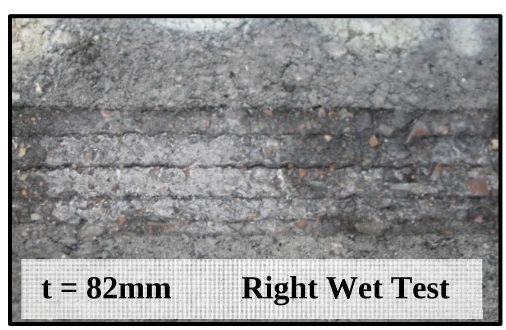
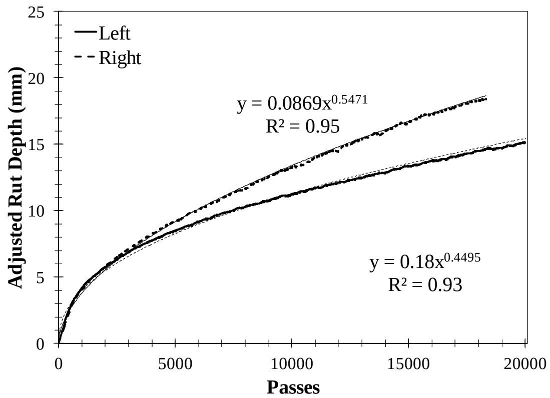


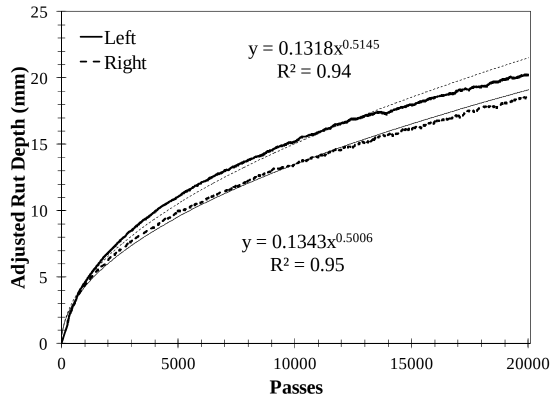
Table A.4. PURWheel Dry Test Results for Strip P (Field Compacted)

Replicate 1, Global Test ID 112				Replicate 2, Global Test ID 113			
Left Specimen (mm)		Right Specimen (mm)		Left Specimen (mm)		Right Specimen (mm)	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.5	250	1.3	250	1.5	250	1.4
500	2.7	500	2.6	500	2.8	500	2.8
1000	4.2	1000	4.0	1000	4.6	1000	4.4
2000	5.8	2000	5.8	2000	6.8	2000	6.3
4000	7.8	4000	8.3	4000	9.9	4000	8.8
8000	10.3	8000	11.7	8000	13.8	8000	12.2
12000	12.1	12000	14.5	12000	16.6	12000	14.6
16000	13.8	16000	17.2	16000	18.6	16000	16.7
20000	15.1 **(15.2)**[1]	18310	18.4 **(18.7)**[1]	20000	20.2 **(19.6)**[1]	20000	18.6 **(18.6)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

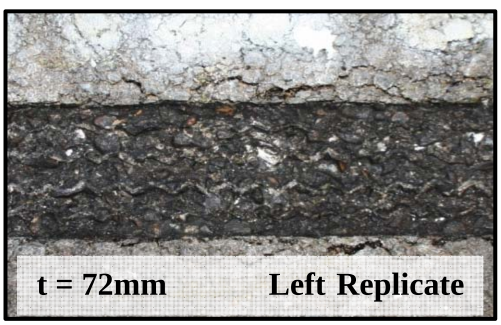


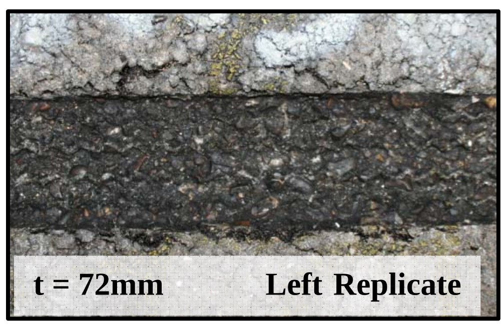
a) Replicate 1

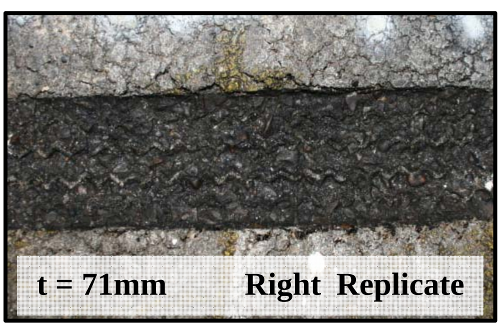


b) Replicate 2

Figure A.4. PURWheel Dry Test Results for Strip P (Field Compacted)







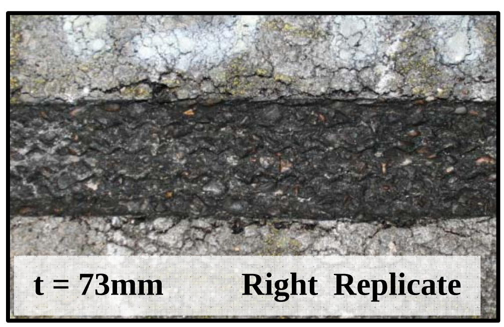
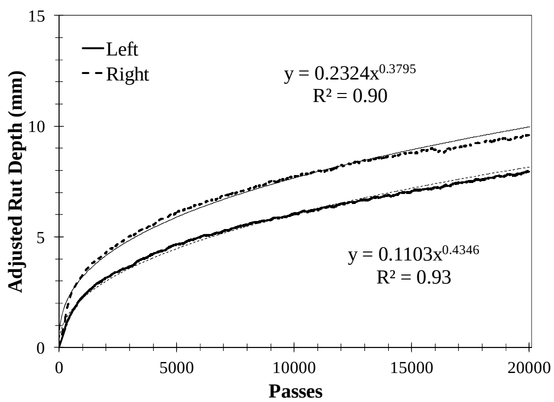


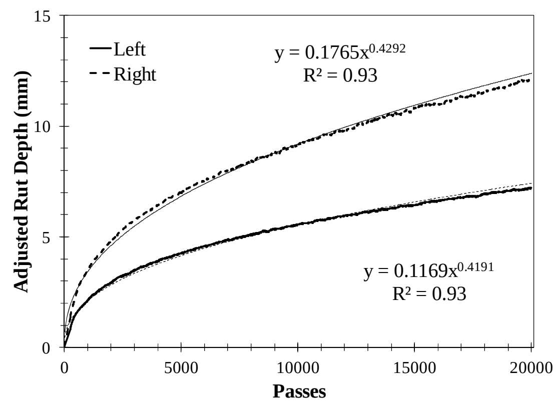
Table A.5. PURWheel Dry Test Results for Strip 1

Replicate 1, Global Test ID 114				Replicate 2, Global Test ID 115			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.9	250	1.3	250	0.8	250	1.4
500	1.5	500	2.4	500	1.5	500	2.4
1000	2.3	1000	3.3	1000	2.1	1000	3.5
2000	3.2	2000	4.3	2000	2.9	2000	4.8
4000	4.2	4000	5.5	4000	3.9	4000	6.4
8000	5.6	8000	7.1	8000	5.1	8000	8.4
12000	6.5	12000	8.2	12000	6.0	12000	9.8
16000	7.2	16000	8.9	16000	6.6	16000	11.0
20000	7.9 **(8.8)**[1]	20000	9.6 **(12.0)**[1]	20000	7.2 **(8.3)**[1]	20000	12.1 **(13.8)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

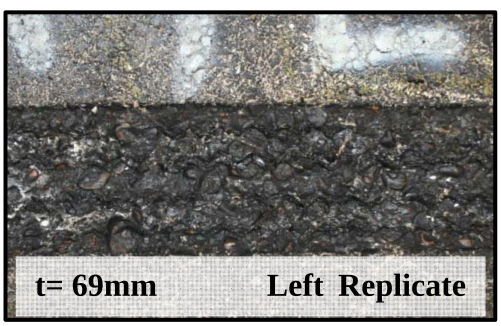


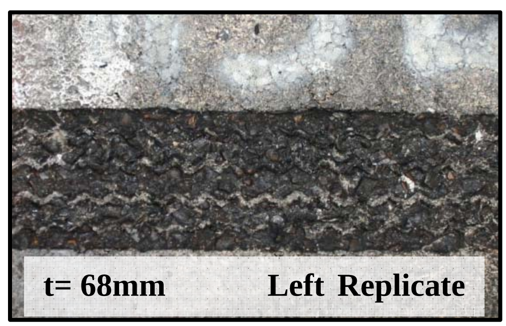
a) Replicate 1

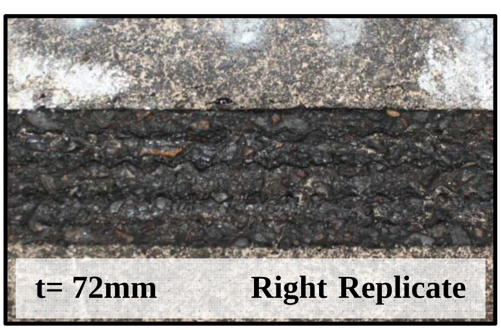


b) Replicate 2

Figure A.5. PURWheel Dry Test Results for Strip 1







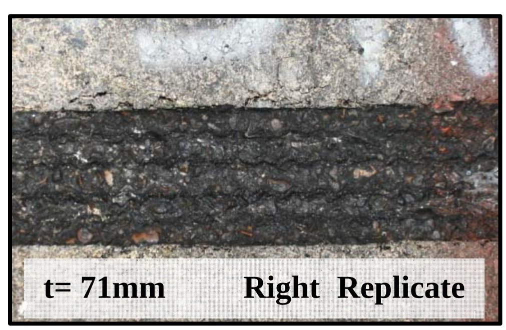
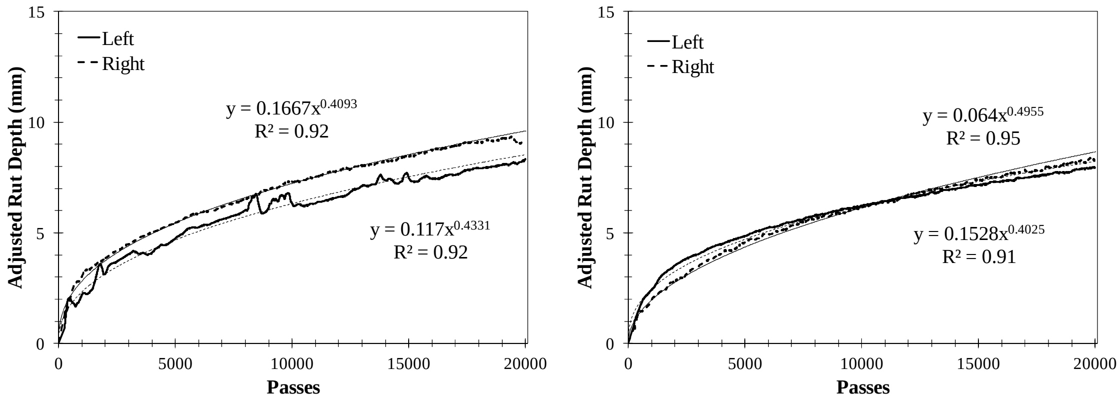


Table A.6. PURWheel Dry Test Results for Strip 2

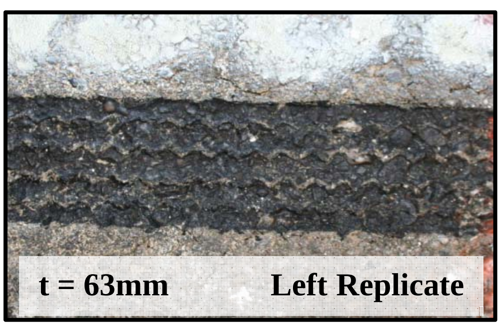
Replicate 1, Global Test ID 138				Replicate 2, Global Test ID 154			
Left Specimen (mm)		Right Specimen (mm)		Left Specimen (mm)		Right Specimen (mm)	
Pass	Adj. Rut	Pass	Adj. Rut	Pass	Adj. Rut	Pass	Adj. Rut
250	0.7	250	1.3	250	0.9	250	0.8
500	2.0	500	2.1	500	1.7	500	1.4
1000	2.2	1000	3.0	1000	2.4	1000	2.0
2000	3.1	2000	3.9	2000	3.5	2000	2.8
4000	4.1	4000	5.0	4000	4.5	4000	4.1
8000	6.0	8000	6.5	8000	5.8	8000	5.6
12000	6.6	12000	7.8	12000	6.6	12000	6.7
16000	7.4	16000	8.7	16000	7.3	16000	7.5
20000	8.3 **(10.4)**[1]	20000	9.2 **(12.6)**[1]	20000	7.9 **(9.5)**[1]	20000	8.3 **(13.3)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Replicate 1 *b) Replicate 2*

Figure A.6. PURWheel Dry Test Results for Strip 2



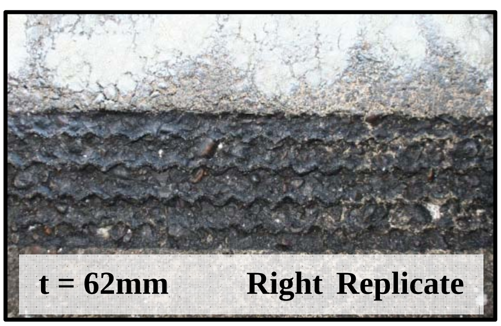
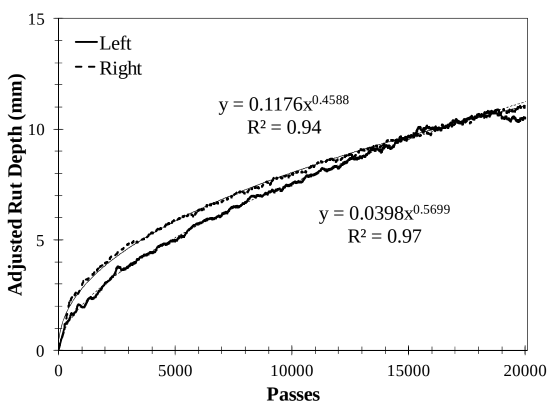


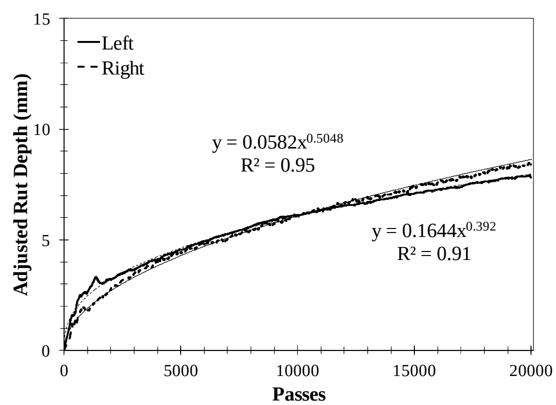
Table A.7. PURWheel Dry Test Results for Strip 3

Replicate 1, Global Test ID 141				Replicate 2, Global Test ID 142			
Left Specimen (mm)		Right Specimen (mm)		Left Specimen (mm)		Right Specimen (mm)	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.1	250	1.0	250	1.2	250	0.6
500	1.5	500	2.2	500	1.8	500	1.3
1000	2.0	1000	3.0	1000	2.6	1000	1.8
2000	3.1	2000	3.9	2000	3.2	2000	2.8
4000	4.4	4000	5.3	4000	4.1	4000	4.0
8000	6.7	8000	7.1	8000	5.6	8000	5.4
12000	8.2	12000	8.6	12000	6.5	12000	6.7
16000	10.0	16000	9.8	16000	7.3	16000	7.6
20000	10.5 **(17.4)**[1]	20000	11.0 **(15.0)**[1]	20000	7.8 **(10.1)**[1]	20000	8.4 **(11.6)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C   Tire Pressure: 862 kPa   Wheel Load: 178.6 kg

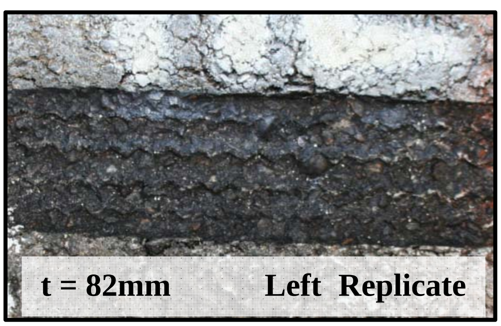


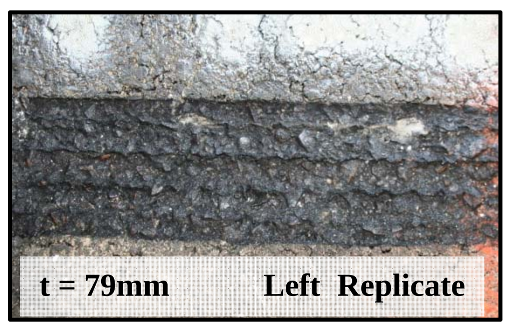
a) Replicate 1

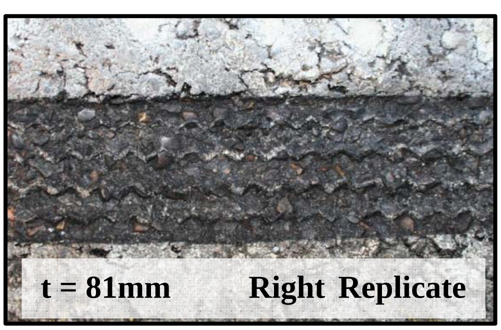


b) Replicate 2

Figure A.7. PURWheel Dry Test Results for Strip 3







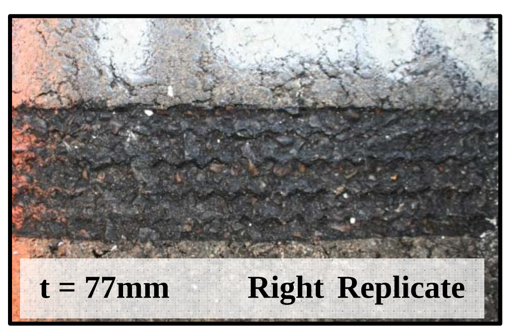
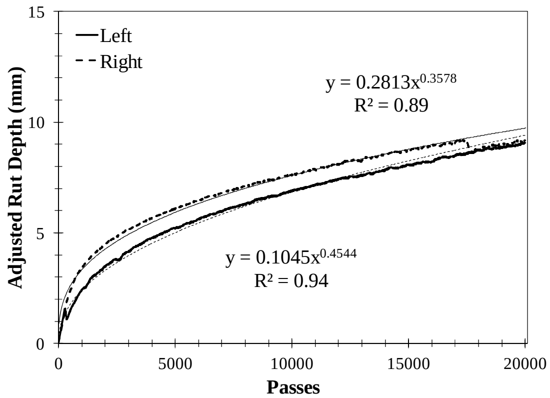


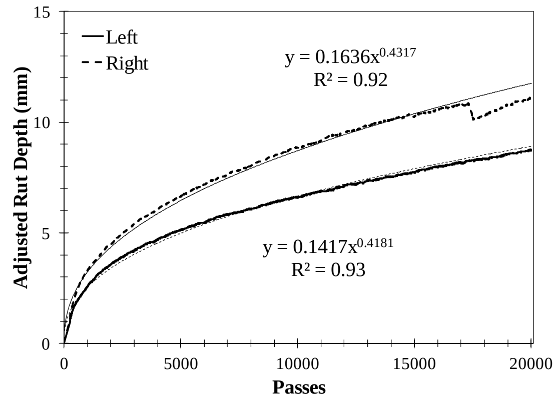
Table A.8. PURWheel Dry Test Results for Strip 4

Replicate 1, Global Test ID 120				Replicate 2, Global Test ID 121			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.6	250	1.4	250	1.0	250	1.2
500	1.5	500	2.4	500	1.8	500	2.2
1000	2.4	1000	3.4	1000	2.6	1000	3.3
2000	3.5	2000	4.5	2000	3.6	2000	4.5
4000	4.8	4000	5.7	4000	4.7	4000	6.1
8000	6.3	8000	7.1	8000	6.1	8000	8.0
12000	7.4	12000	8.1	12000	7.1	12000	9.5
16000	8.2	16000	9.0	16000	8.0	16000	10.6
20000	9.1 **(9.7)**[1]	20000	9.7 **(11.6)**[1]	20000	8.7 **(10.1)**[1]	20000	11.1 **(12.7)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C   Tire Pressure: 862 kPa   Wheel Load: 178.6 kg

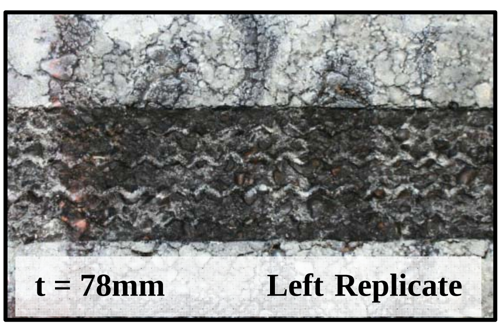


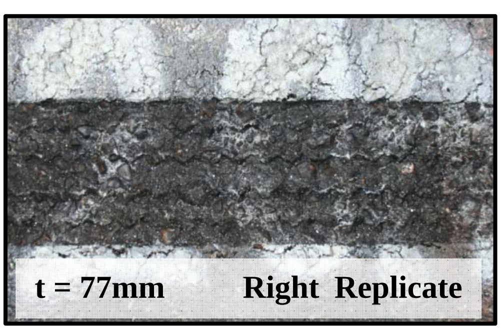
a) Replicate 1



b) Replicate 2

Figure A.8. PURWheel Dry Test Results for Strip 4





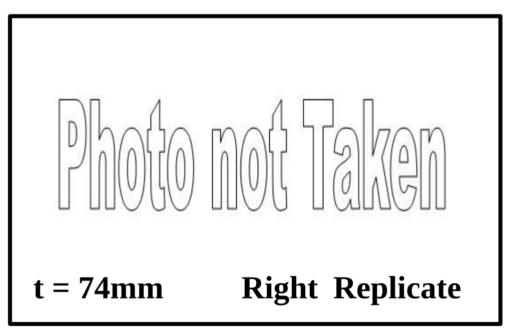
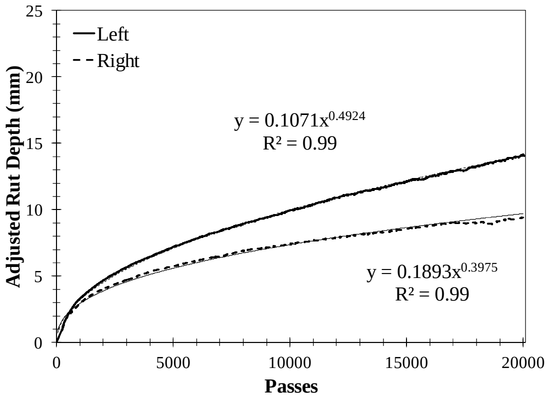


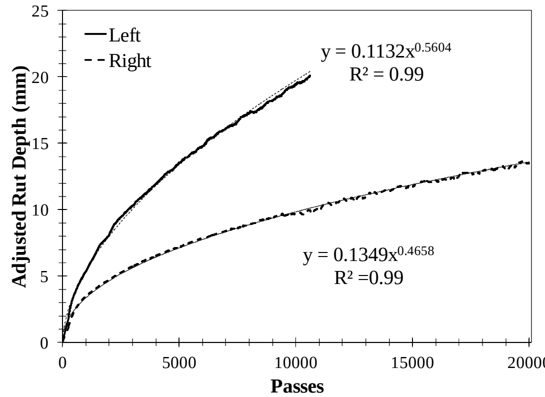
Table A.9. PURWheel Dry Test Results for Strip 5

Replicate 1, Global Test ID 122				Replicate 2, Global Test ID 123			
Left Specimen (mm)		Right Specimen (mm)		Left Specimen (mm)		Right Specimen (mm)	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.1	250	1.0	250	1.9	250	1.2
500	2.1	500	2.1	500	3.6	500	2.3
1000	3.3	1000	3.0	1000	5.4	1000	3.5
2000	4.7	2000	4.0	2000	8.1	2000	4.7
4000	6.5	4000	5.3	4000	12.0	4000	6.5
8000	8.9	8000	6.9	8000	17.3	8000	8.9
12000	10.9	12000	7.9	10824	18.4 **(18.1)**[1]	12000	10.8
16000	12.5	16000	8.8	---	---	16000	12.2
20000	14.0 **(13.3)**[1]	20000	9.4 **(11.3)**[1]	---	---	20000	13.5 **(14.8)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

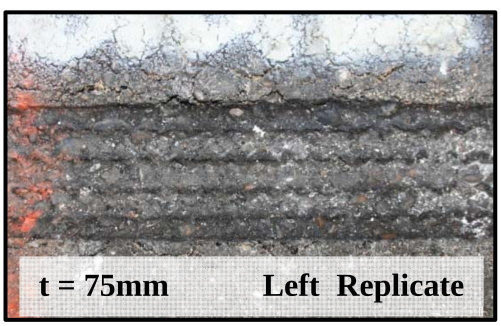


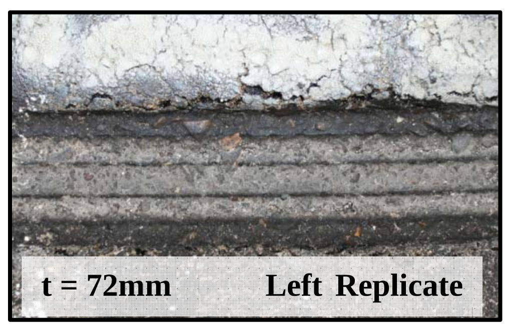
a) Replicate 1

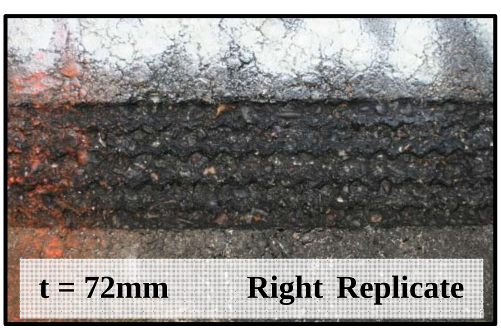


b) Replicate 2

Figure A.9. PURWheel Dry Test Results for Strip 5







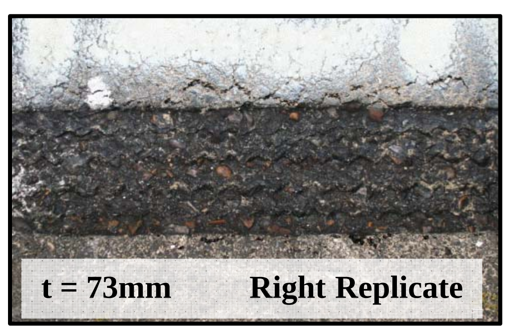
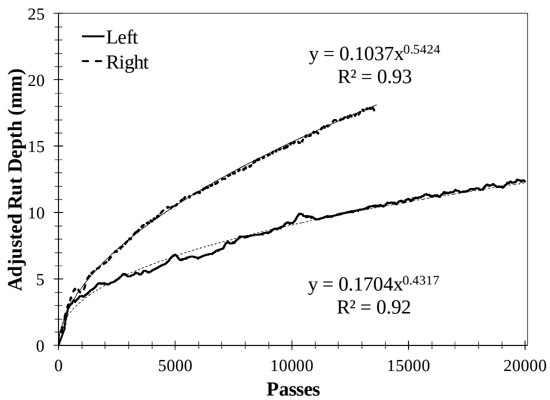


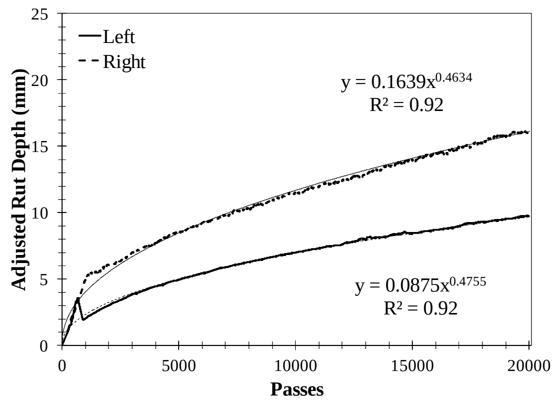
Table A.10. PURWheel Dry Test Results for Strip 6

Replicate 1, Global Test ID 147				Replicate 2, Global Test ID 148			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.3	250	2.1	250	1.1	250	1.1
500	3.0	500	3.5	500	2.6	500	2.5
1000	3.7	1000	4.0	1000	2.0	1000	5.0
2000	4.6	2000	6.2	2000	3.0	2000	6.0
4000	5.6	4000	9.4	4000	4.4	4000	7.7
8000	8.2	8000	13.4	8000	6.3	8000	10.3
12000	9.9	12000	17.0	12000	7.6	12000	12.4
16000	11.3	13608	9.4 **(11.3)**[1]	16000	8.7	16000	14.5
20000	12.4 **(14.3)**[1]	---	---	20000	9.7 **(12.8)**[1]	20000	16.1 **(17.5)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

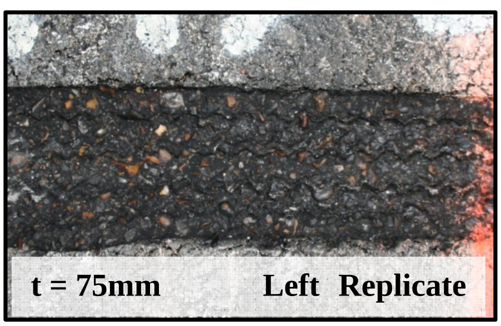


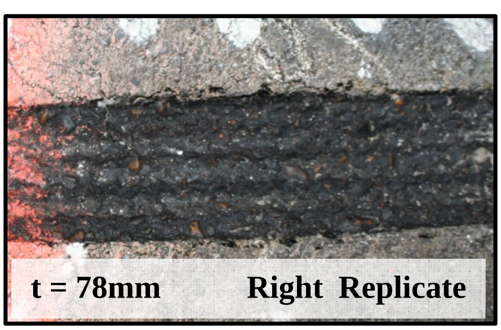
a) Replicate 1



b) Replicate 2

Figure A.10. PURWheel Dry Test Results for Strip 6





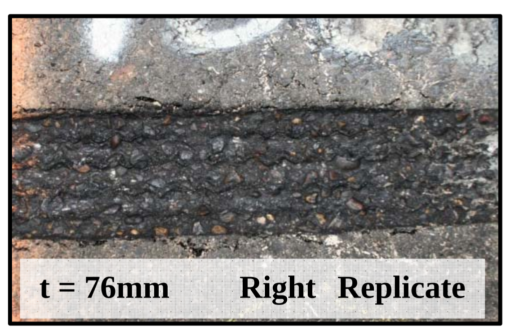
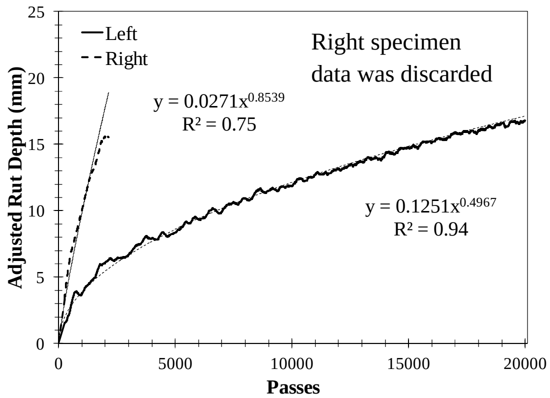


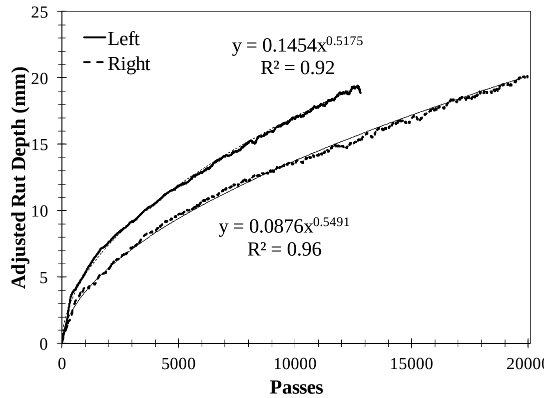
Table A.11. PURWheel Dry Test Results for Strip 7

Replicate 1, Global Test ID 151				Replicate 2, Global Test ID 152			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.6	250	3.4	250	2.2	250	1.5
500	2.6	500	6.6	500	3.9	500	2.8
1000	3.7	1000	10.1	1000	5.3	1000	4.1
2000	6.1	2000	15.6	2000	7.6	2000	5.6
4000	7.9	2136	15.5 **(---)**[1]	4000	10.5	4000	8.5
8000	10.9	**Data was discarded**		8000	15.1	8000	12.2
12000	13.2	---	---	12000	18.8	12000	14.8
16000	15.2	---	---	12808	18.8 **(21.9)**[1]	16000	17.6
20000	16.8 **(19.4)**[1]	---	---	---	---	20000	20.1 **(21.0)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

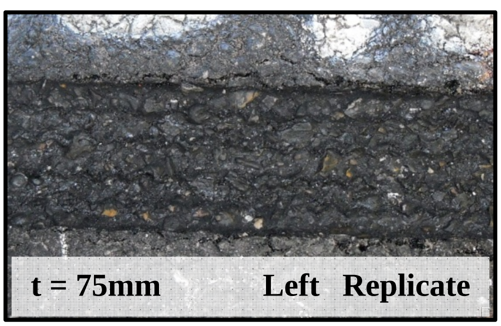


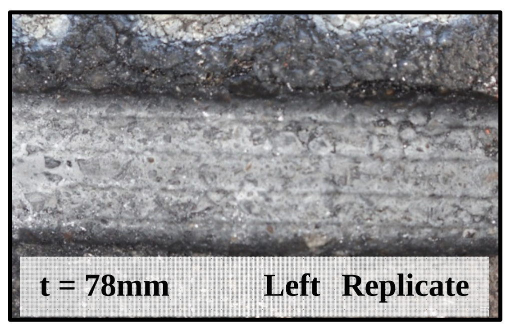
a) Replicate 1

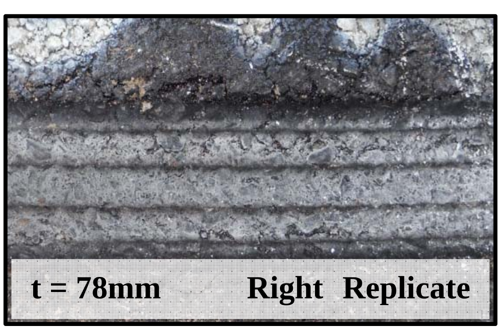


b) Replicate 2

Figure A.11. PURWheel Dry Test Results for Strip 7







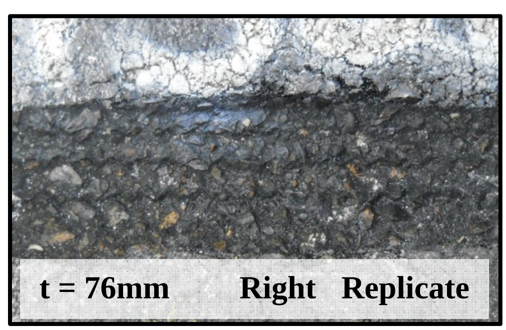
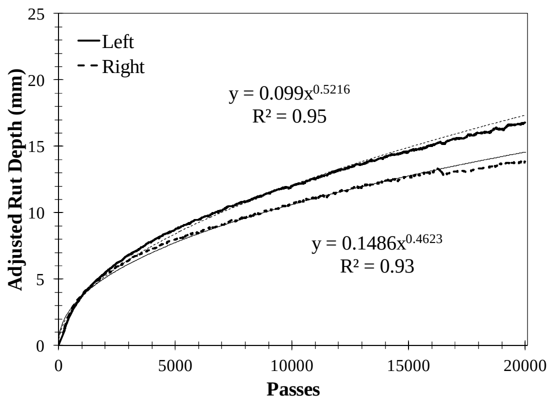


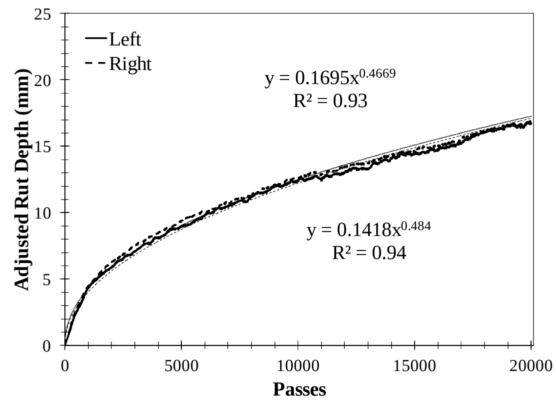
Table A.12. PURWheel Dry Test Results for Strip 8

Replicate 1, Global Test ID 126				Replicate 2, Global Test ID 127			
Left Specimen (mm)		Right Specimen (mm)		Left Specimen (mm)		Right Specimen (mm)	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.2	250	1.3	250	1.4	250	1.5
500	2.3	500	2.5	500	2.5	500	2.8
1000	3.7	1000	3.8	1000	4.4	1000	4.5
2000	5.5	2000	5.4	2000	5.9	2000	6.3
4000	7.8	4000	7.3	4000	8.1	4000	8.5
8000	10.8	8000	9.6	8000	11.2	8000	11.4
12000	13.2	12000	11.5	12000	13.1	12000	13.5
16000	15.1	16000	13.1	16000	14.8	16000	15.0
20000	16.8 **(16.6)**[1]	20000	13.9 **(14.4)**[1]	20000	16.7 **(16.9)**[1]	20000	16.8 **(16.3)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

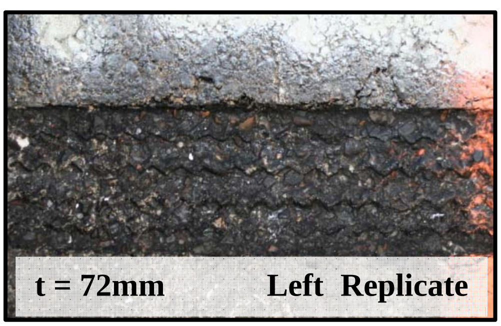


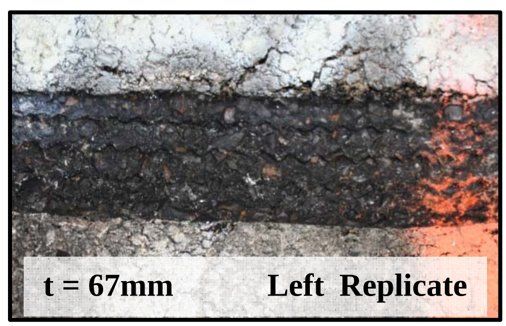
a) Replicate 1

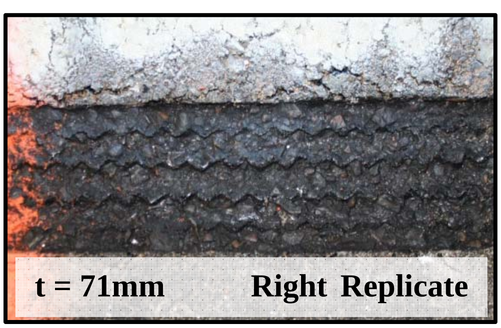


b) Replicate 2

Figure A.12. PURWheel Dry Test Results for Strip 8







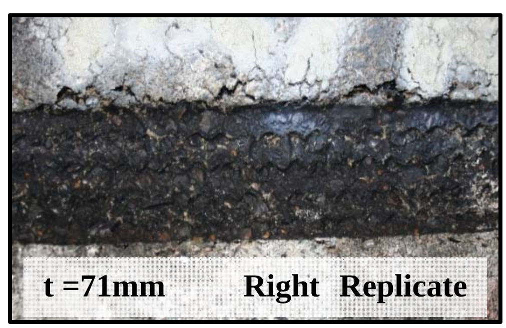
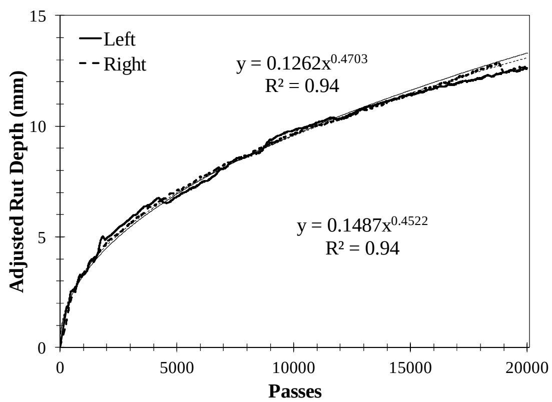


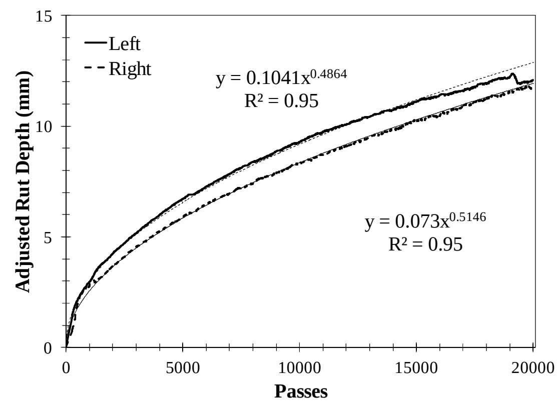
Table A.13. PURWheel Dry Test Results for Strip 9

Replicate 1, Global Test ID 130				Replicate 2, Global Test ID 131			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.7	250	1.1	250	1.4	250	0.8
500	2.5	500	2.3	500	2.2	500	2.0
1000	3.3	1000	3.4	1000	3.0	1000	2.8
2000	4.9	2000	4.8	2000	4.3	2000	3.7
4000	6.6	4000	6.4	4000	6.0	4000	5.3
8000	8.7	8000	8.7	8000	8.4	8000	7.4
12000	10.4	12000	10.3	12000	10.1	12000	9.1
16000	11.7	16000	11.8	16000	11.4	16000	10.5
20000	12.6 **(13.7)**[1]	20000	12.7 **(14.9)**[1]	20000	12.1 **(15.6)**[1]	20000	11.8 **(12.6)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

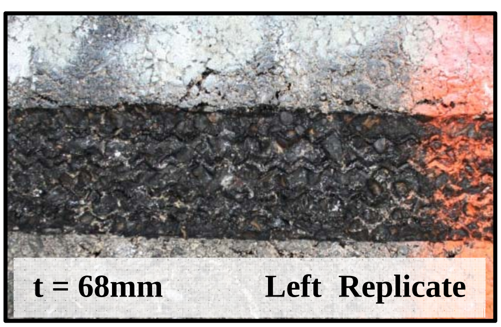


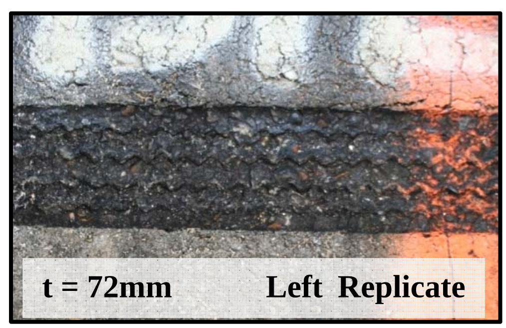
a) Replicate 1

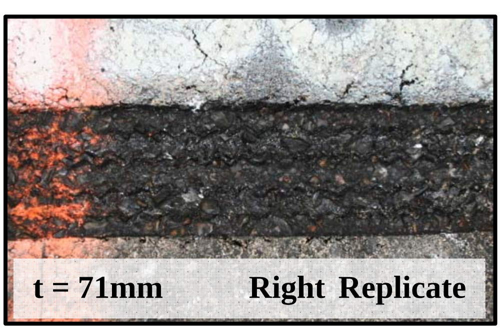


b) Replicate 2

Figure A.13. PURWheel Dry Test Results for Strip 9







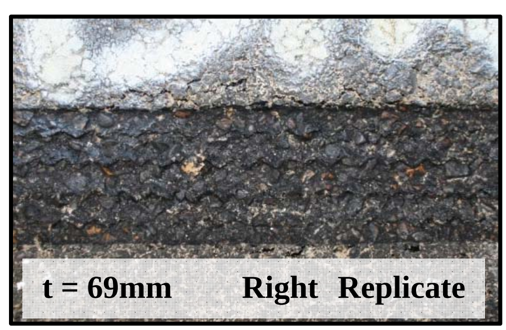
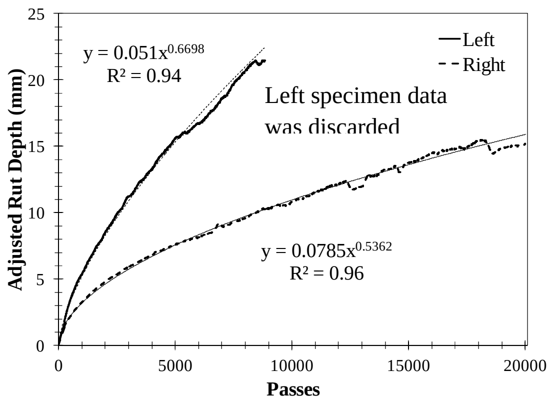


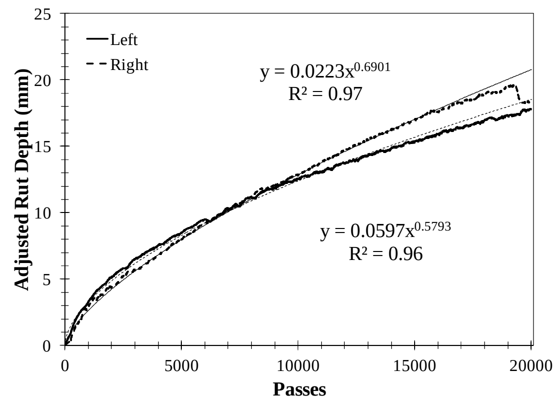
Table A.14. PURWheel Dry Test Results for Strip 10

Replicate 1, Global Test ID 156				Replicate 2, Global Test ID 157			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.8	250	1.4	250	1.0	250	0.4
500	3.4	500	2.2	500	2.1	500	1.5
1000	5.4	1000	3.3	1000	3.3	1000	3.0
2000	8.5	2000	4.8	2000	5.2	2000	4.4
4000	13.3	4000	6.8	4000	7.5	4000	6.8
8000	20.7	8000	9.6	8000	11.1	8000	11.2
8840	21.5 **(20.8)**[1]	12000	12.2	12000	13.7	12000	14.7
Data was discarded		16000	14.4	16000	15.9	16000	17.6
---	---	20000	15.9 **(16.8)**[1]	20000	17.8 **(19.5)**[1]	20000	19.6 **(20.1)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

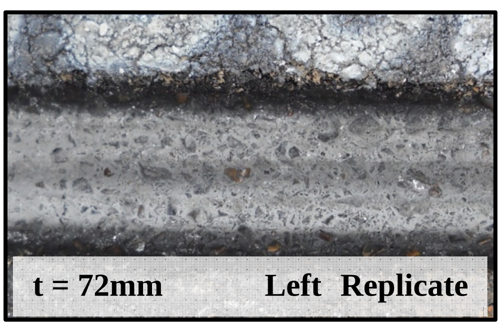


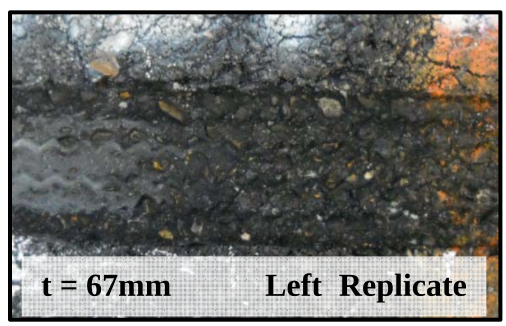
a) Replicate 1

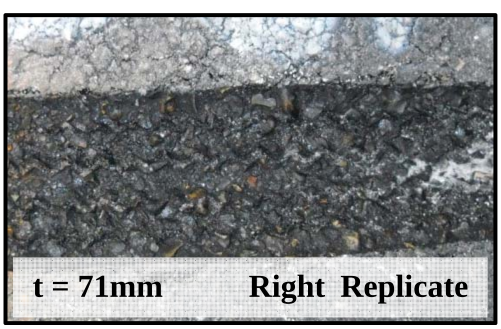


b) Replicate 2

Figure A.14. PURWheel Dry Test Results for Strip 10







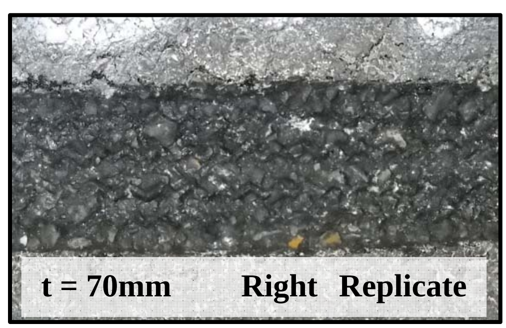
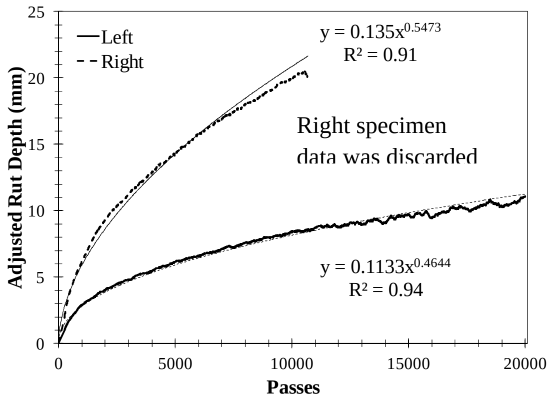


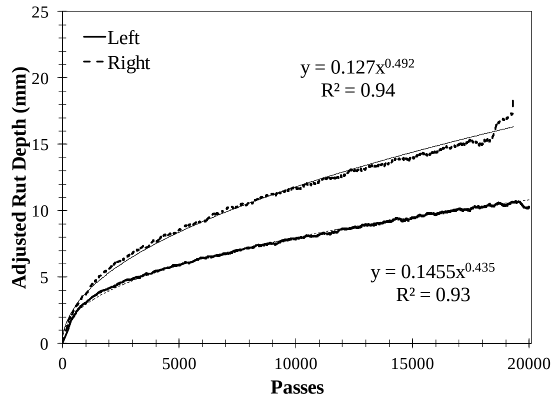
Table A.15. PURWheel Dry Test Results for Strip 11

Replicate 1, Global Test ID 164				Replicate 2, Global Test ID 165			
Left Specimen (mm)		Right Specimen (mm)		Left Specimen (mm)		Right Specimen (mm)	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.0	250	2.0	250	1.1	250	1.7
500	1.9	500	4.0	500	2.1	500	2.6
1000	2.9	1000	6.2	1000	3.1	1000	3.7
2000	4.0	2000	9.3	2000	4.2	2000	5.6
4000	5.5	4000	12.9	4000	5.5	4000	7.7
8000	7.6	8000	18.1	8000	7.2	8000	10.6
12000	8.8	10672	20.1 **(20.7)**[1]	12000	8.6	12000	12.7
16000	9.5	**Data was discarded**		16000	9.7	16000	14.7
20000	11.1 **(13.5)**[1]	---	---	20000	10.3 **(12.6)**[1]	19312	16.6 **(18.2)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C　　Tire Pressure: 862 kPa　　Wheel Load: 178.6 kg

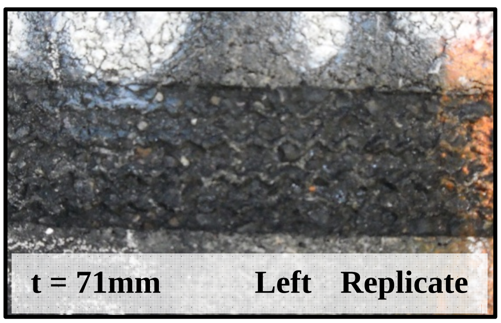


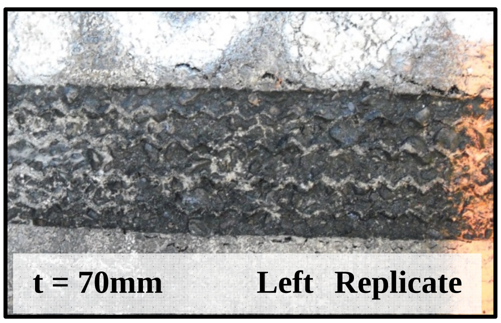
a) Replicate 1

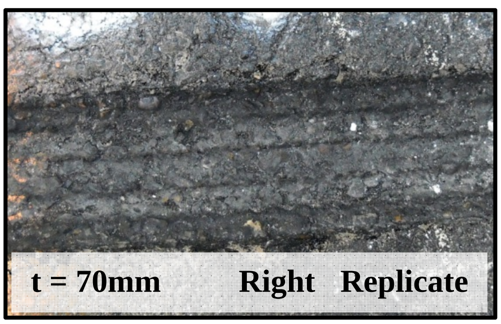


b) Replicate 2

Figure A.15. PURWheel Dry Test Results for Strip 11







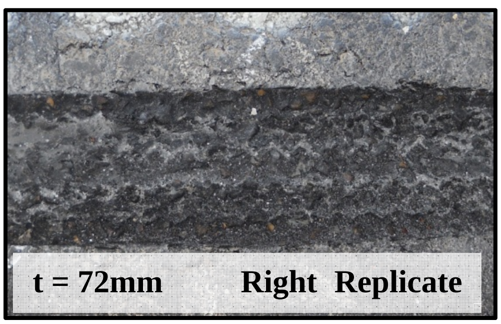
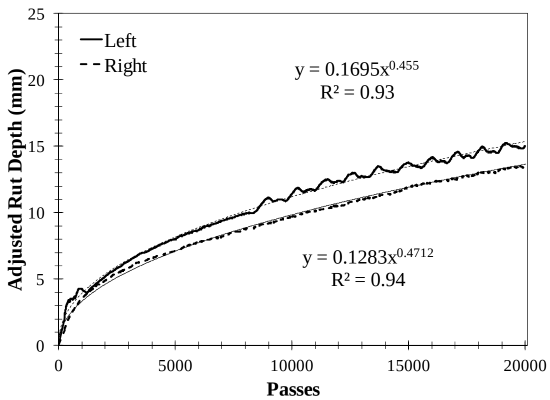


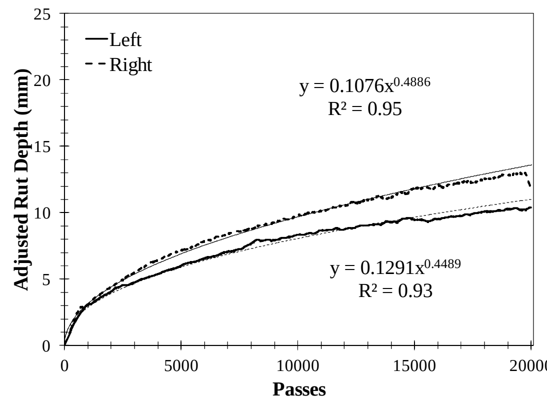
Table A.16. PURWheel Dry Test Results for Strip 12

Replicate 1, Global Test ID 134				**Replicate 2, Global Test ID 135**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	2.1	250	1.2	250	1.0	250	1.2
500	3.5	500	2.3	500	1.9	500	2.4
1000	4.3	1000	3.6	1000	3.1	1000	3.0
2000	5.2	2000	4.9	2000	4.1	2000	4.4
4000	7.3	4000	6.6	4000	5.4	4000	6.4
8000	9.9	8000	8.8	8000	7.7	8000	8.8
12000	12.4	12000	10.5	12000	8.8	12000	10.5
16000	14.2	16000	12.3	16000	9.5	16000	12.1
20000	15.0 **(16.2)**[1]	20000	13.3 **(13.7)**[1]	20000	10.4 **(11.7)**[1]	20000	12.0 **(15.9)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C   Tire Pressure: 862 kPa   Wheel Load: 178.6 kg

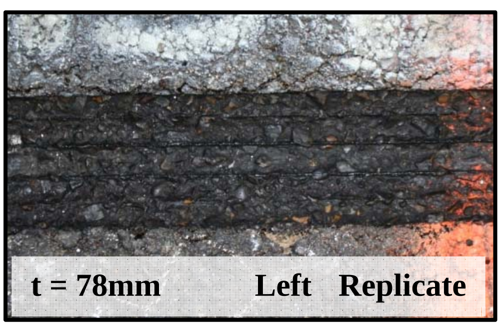


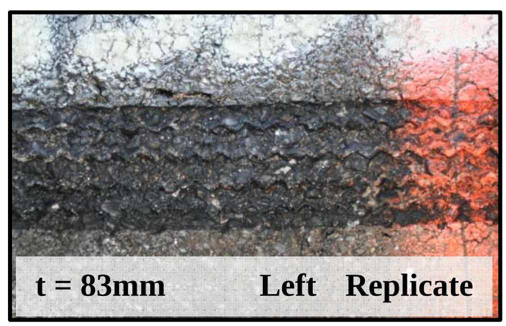
a) Replicate 1

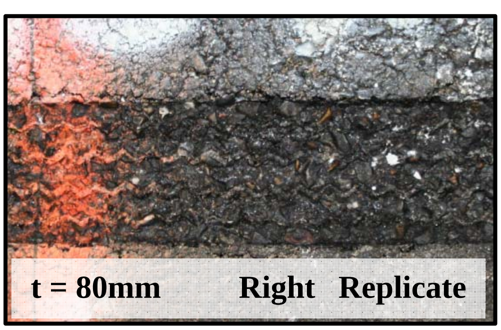


b) Replicate 2

Figure A.16. PURWheel Dry Test Results for Strip 12







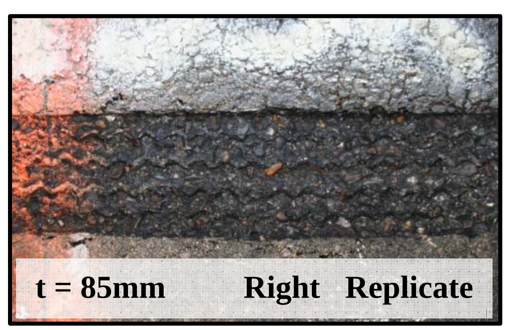
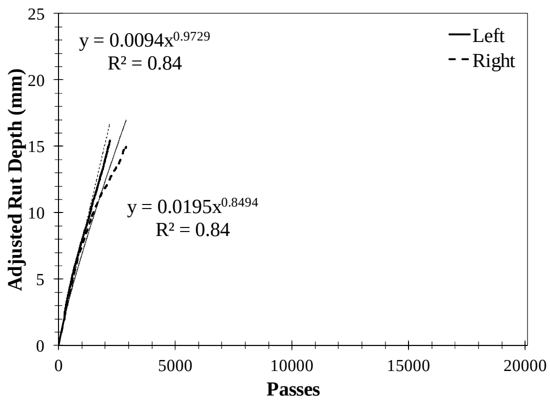


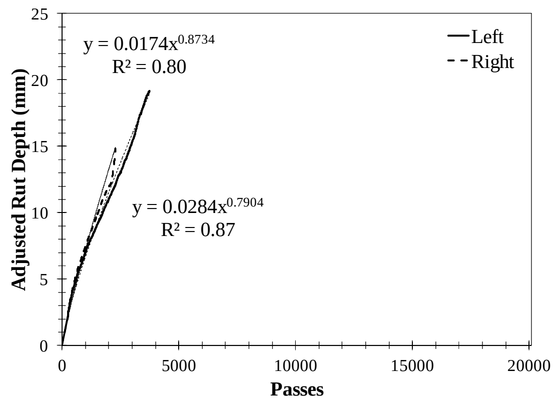
Table A.17. PURWheel Wet Test Results for Strip P (Field Compacted)

Replicate 1, Global Test ID 109				Replicate 2, Global Test ID 110			
Left Specimen (mm)		Right Specimen (mm)		Left Specimen (mm)		Right Specimen (mm)	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	2.3	250	2.1	250	2.3	250	2.3
500	4.5	500	4.2	500	4.4	500	4.7
1000	7.9	1000	7.5	1000	7.0	1000	7.5
2000	14.1	2000	11.9	2000	11.0	2000	11.9
2196	15.4 **(---)**[1]	2886	15.0 **(14.2)**[1]	3734	19.2 **(18.6)**[1]	2284	14.8 **(21.1)**[1]
---	---	---	---	---	---	---	---
---	---	---	---	---	---	---	---
---	---	---	---	---	---	---	---
---	---	---	---	---	---	---	---

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C   Tire Pressure: 862 kPa   Wheel Load: 178.6 kg

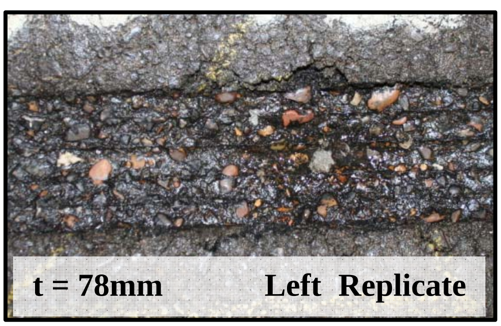


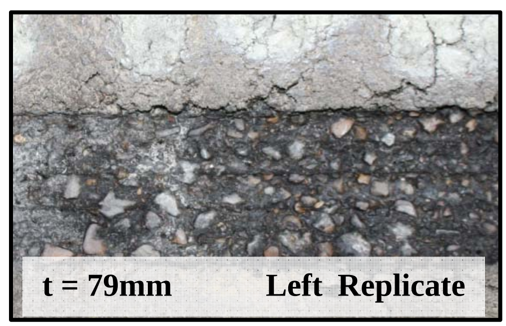
a) Replicate 1

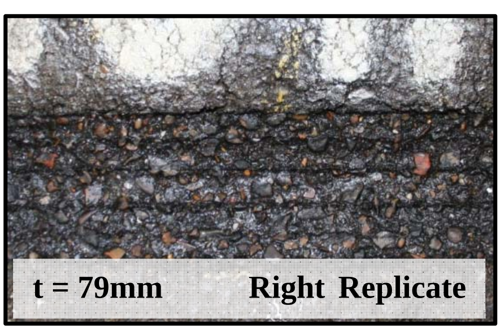


b) Replicate 2

Figure A.17. PURWheel Wet Test Results for Strip P (Field Compacted)







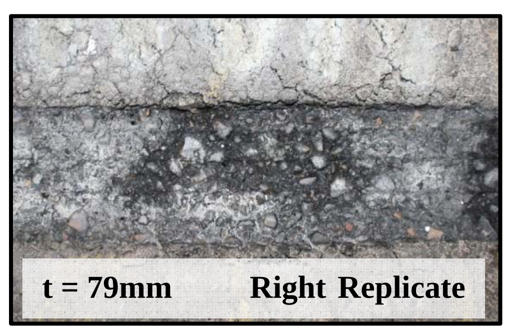
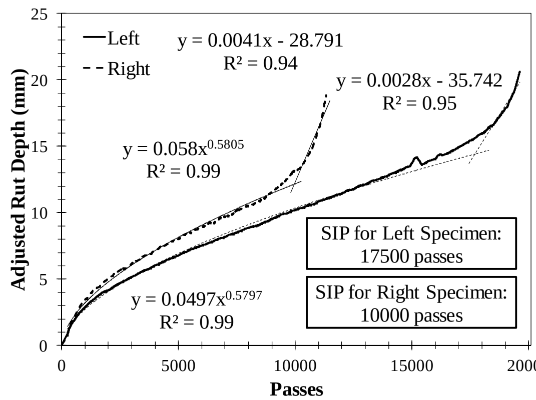


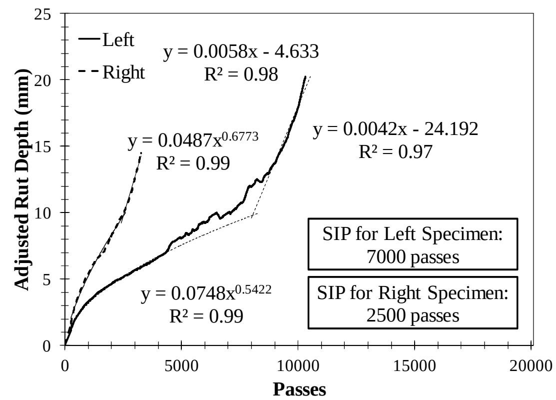
Table A.18. PURWheel Wet Test Results for Strip 1

Replicate 1, Global Test ID 111				Replicate 2, Global Test ID 116			
Left Specimen (mm)		Right Specimen (mm)		Left Specimen (mm)		Right Specimen (mm)	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.9	250	0.8	250	1.2	250	1.7
500	1.8	500	2.1	500	2.2	500	3.4
1000	2.9	1000	3.5	1000	3.3	1000	5.4
2000	4.2	2000	5.0	2000	4.6	2000	8.3
4000	6.0	4000	7.1	4000	6.7	3276	14.5 **(17.1)**[1]
8000	8.9	8000	10.5	8000	12.0	---	---
12000	11.5	11332	18.8 **(19.3)**[1]	10324	20.3 **(---)**[1]	---	---
16000	14.1	---	---	---	---	---	---
19616	20.6 **(18.6)**[1]	---	---	---	---	---	---

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

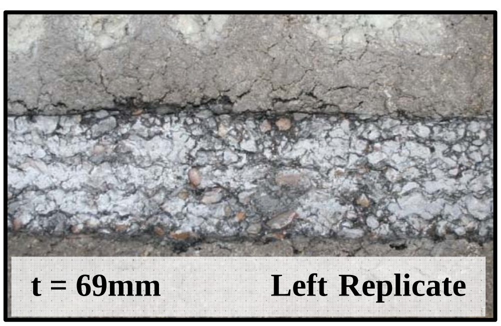


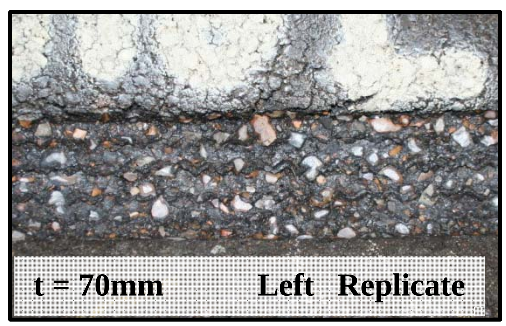
a) Replicate 1

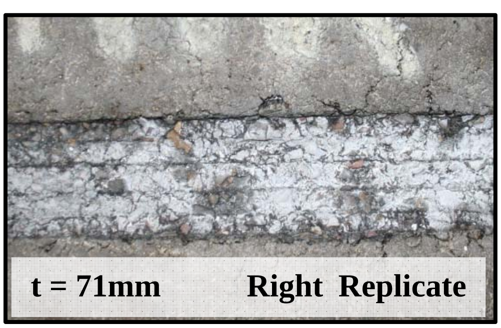


b) Replicate 2

Figure A.18. PURWheel Wet Test Results for Strip 1







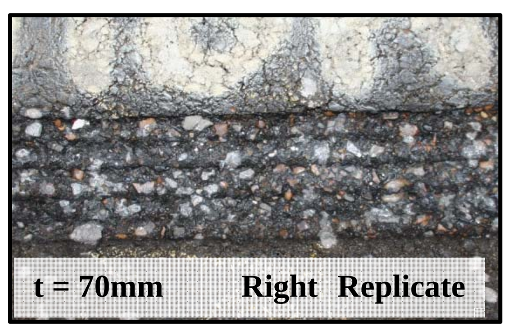
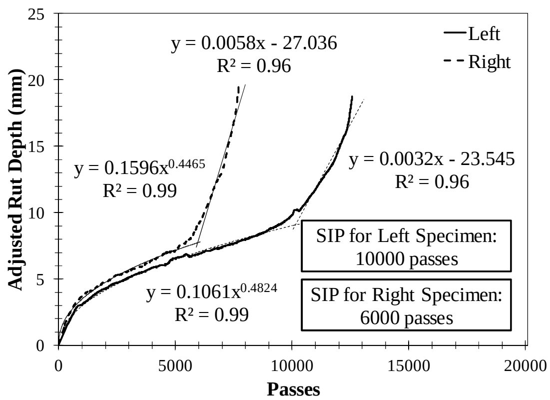


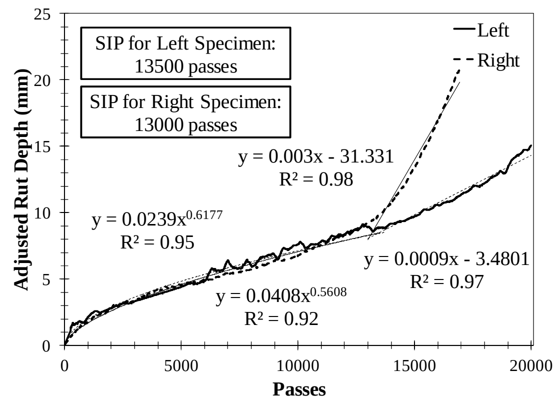
Table A.19. PURWheel Wet Test Results for Strip 2

Replicate 1, Global Test ID 117				Replicate 2, Global Test ID 140			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.1	250	1.5	250	1.2	250	0.7
500	2.0	500	2.5	500	1.7	500	1.1
1000	3.1	1000	3.7	1000	2.1	1000	1.8
2000	4.3	2000	4.8	2000	2.8	2000	2.7
4000	5.8	4000	6.4	4000	3.9	4000	4.2
8000	8.0	7742	20.0 **(---)**[1]	8000	6.0	8000	5.7
12000	14.8	---	---	12000	8.4	12000	8.3
12580	18.8 **(---)**[1]	---	---	16000	10.6	16000	16.9
---	---	---	---	20000	15.1 **(16.7)**[1]	16938	20.8 **(---)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

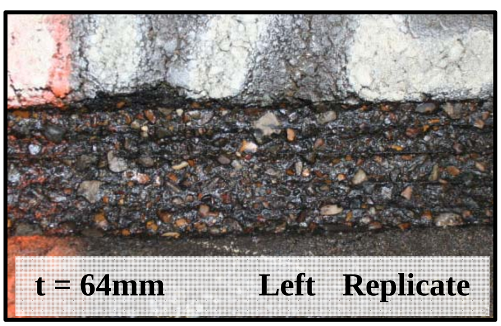


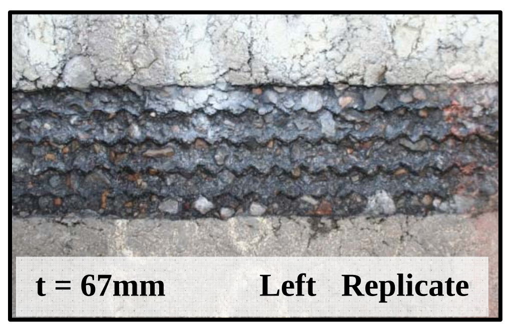
a) Replicate 1

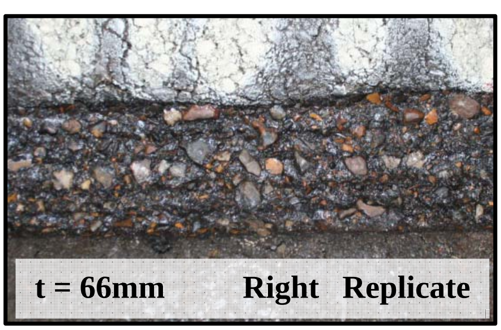


b) Replicate 2

Figure A.19. PURWheel Wet Test Results for Strip 2







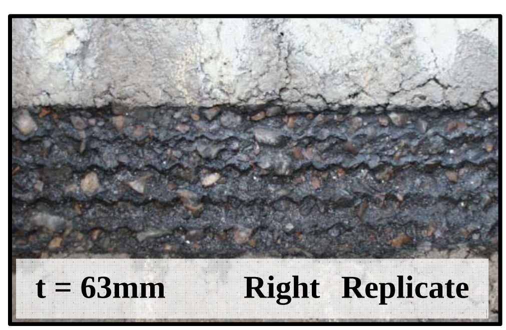
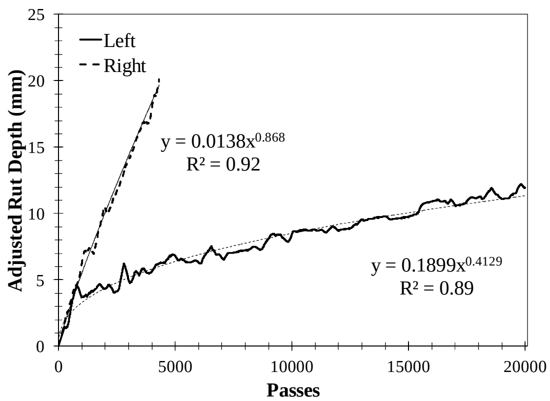


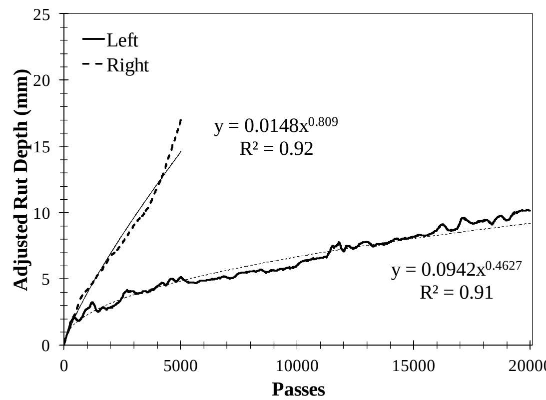
Table A.20. PURWheel Wet Test Results for Strip 3

Replicate 1, Global Test ID 145				Replicate 2, Global Test ID 146			
Left Specimen (mm)		Right Specimen (mm)		Left Specimen (mm)		Right Specimen (mm)	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.5	250	1.8	250	1.6	250	1.4
500	2.6	500	3.3	500	2.0	500	2.5
1000	3.7	1000	6.4	1000	2.7	1000	4.2
2000	4.4	2000	10.4	2000	2.9	2000	6.7
4000	5.6	4000	18.0	4000	4.4	4000	11.9
8000	7.2	4308	20.1 **(---)**[1]	8000	5.6	5024	17.0 **(---)**[1]
12000	8.7	---	---	12000	7.1	---	---
16000	10.9	---	---	16000	8.7	---	---
20000	11.9 **(12.8)**[1]	---	---	20000	10.2 **(12.9)**[1]	---	---

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

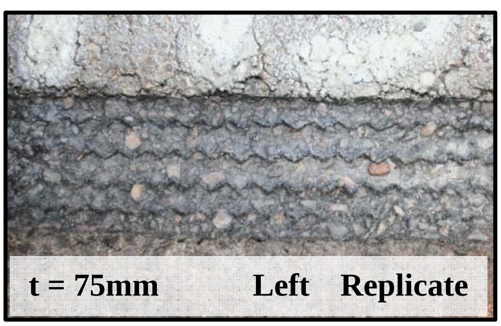


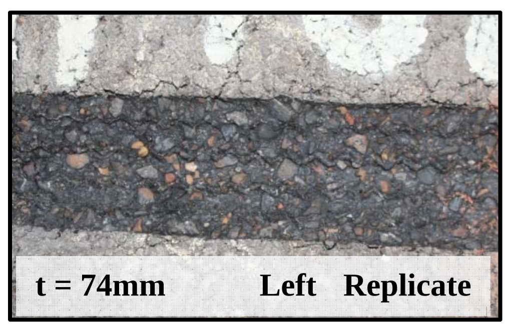
a) Replicate 1

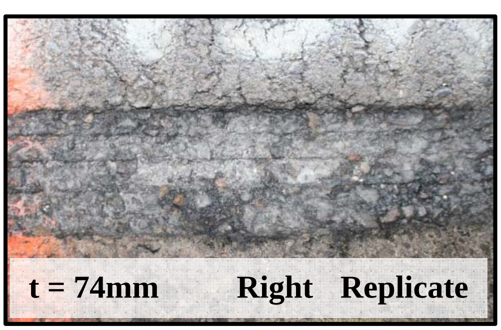


b) Replicate 2

Figure A.20. PURWheel Wet Test Results for Strip 3







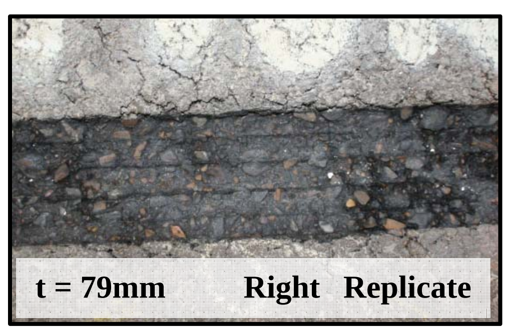
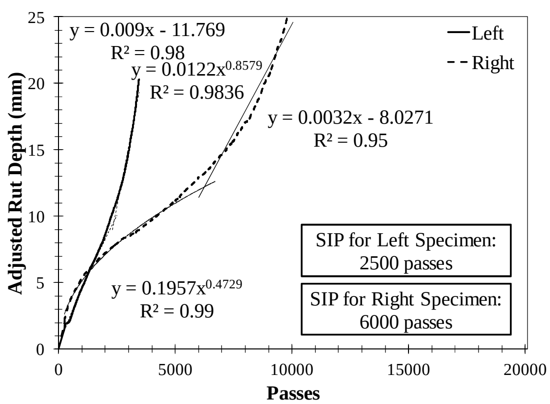


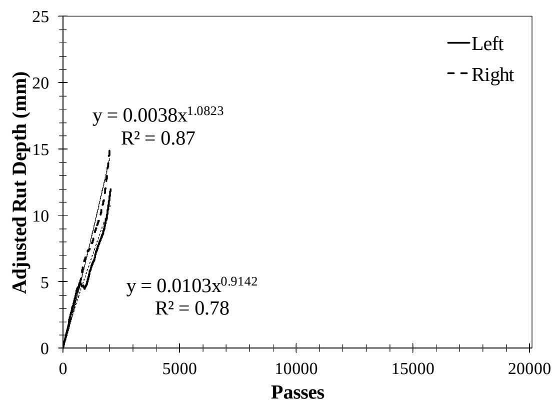
Table A.21. PURWheel Wet Test Results for Strip 4

Replicate 1, Global Test ID 118				Replicate 2, Global Test ID 119			
Left Specimen (mm)		Right Specimen (mm)		Left Specimen (mm)		Right Specimen (mm)	
Pass	Adj. Rut	Pass	Adj. Rut	Pass	Adj. Rut	Pass	Adj. Rut
250	1.6	250	2.1	250	2.0	250	1.9
500	2.3	500	3.7	500	3.6	500	3.6
1000	4.6	1000	5.3	1000	4.8	1000	7.0
2000	8.6	2000	7.2	2000	11.5	1994	15.0 **(21.0)**[1]
3448	20.3 **(20.5)**[1]	4000	9.8	2038	12.0 **(17.2)**[1]	---	---
---	---	8000	17.1	---	---	---	---
---	---	10060	27.6 **(22.7)**[1]	---	---	---	---
---	---	---	---	---	---	---	---
---	---	---	---	---	---	---	---

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

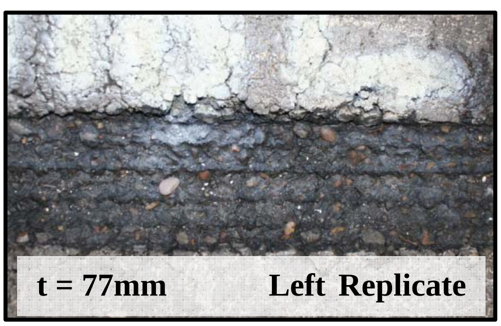


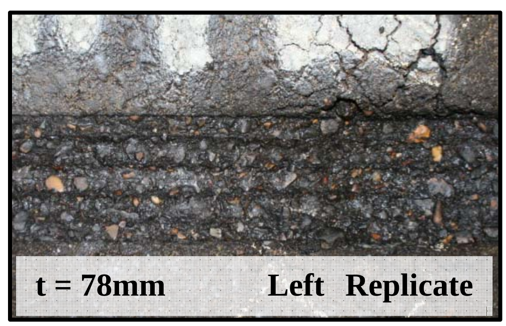
a) Replicate 1

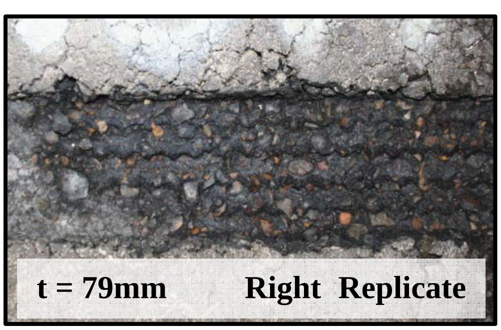


b) Replicate 2

Figure A.21. PURWheel Wet Test Results for Strip 4







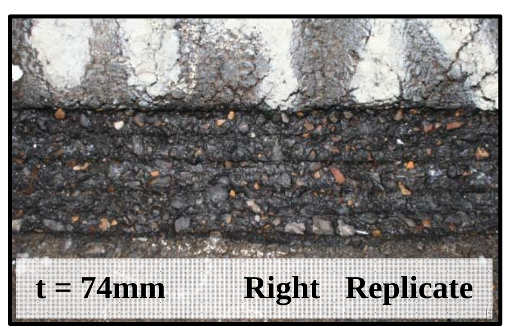
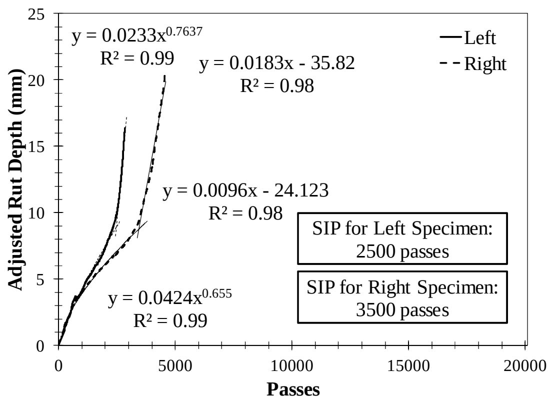


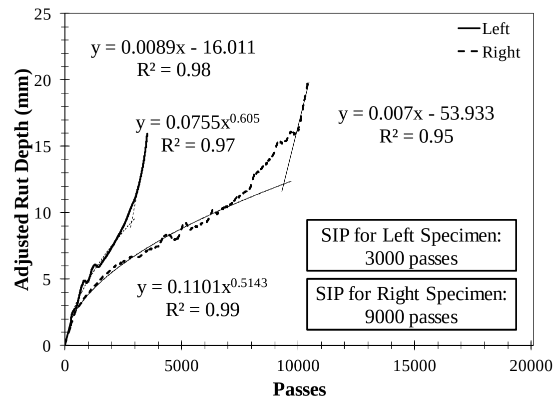
Table A.22. PURWheel Wet Test Results for Strip 5

Replicate 1, Global Test ID 124				Replicate 2, Global Test ID 125			
Left Specimen (mm)		Right Specimen (mm)		Left Specimen (mm)		Right Specimen (mm)	
Pass	Adj. Rut	Pass	Adj. Rut	Pass	Adj. Rut	Pass	Adj. Rut
250	1.2	250	1.5	250	1.9	250	1.5
500	2.4	500	2.4	500	2.9	500	2.7
1000	4.1	1000	4.0	1000	4.9	1000	3.9
2000	7.4	2000	6.1	2000	7.4	2000	5.7
2840	16.4 **(25.6)**[1]	4000	13.4	3528	15.9 **(17.9)**[1]	4000	7.5
---	---	4554	20.4 **(21.2)**[1]	---	---	8000	12.2
---	---	---	---	---	---	10462	13.5 **(14.8)**[1]
---	---	---	---	---	---	---	---
---	---	---	---	---	---	---	---

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

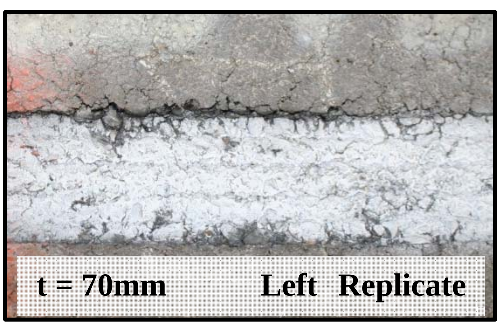


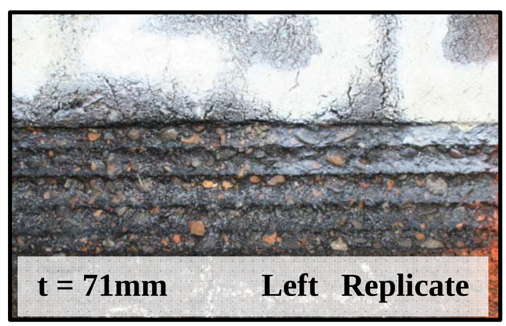
a) Replicate 1

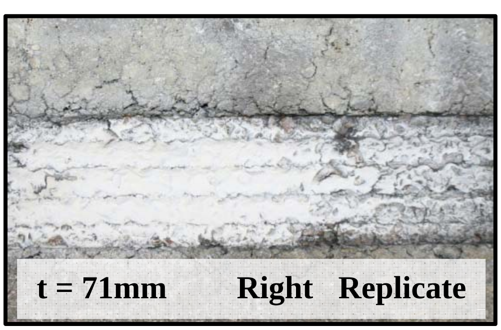


b) Replicate 2

Figure A.22. PURWheel Wet Test Results for Strip 5







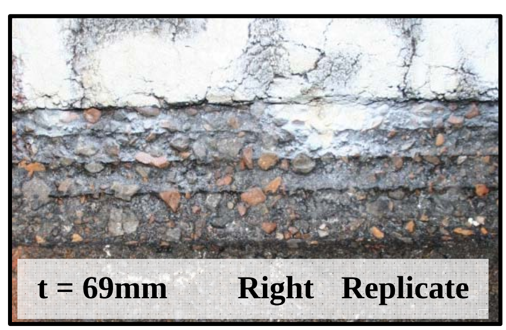
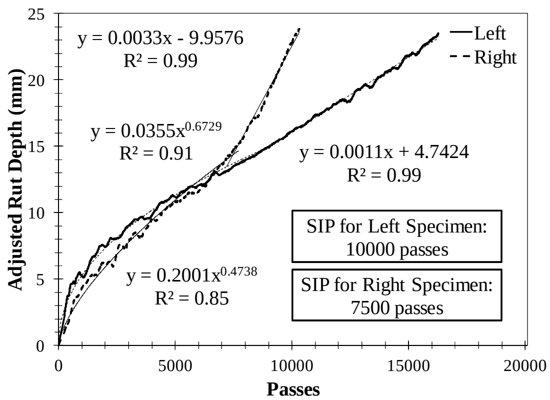


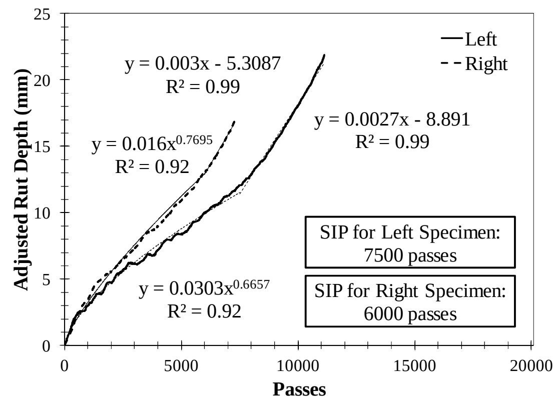
Table A.23. PURWheel Wet Test Results for Strip 6

Replicate 1, Global Test ID 149				Replicate 2, Global Test ID 150			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	2.3	250	1.1	250	1.2	250	0.9
500	4.5	500	2.3	500	2.2	500	2.1
1000	5.2	1000	4.4	1000	3.0	1000	3.4
2000	7.4	2000	6.2	2000	4.7	2000	5.6
4000	9.8	4000	9.5	4000	7.2	4000	9.0
8000	14.0	8000	16.0	8000	12.9	7310	17.1 **(18.1)**[1]
12000	18.3	10308	23.8 **(22.6)**[1]	11140	21.9 **(22.5)**[1]	---	---
16000	23.0	---	---	---	---	---	---
20000	23.5 **(21.1)**[1]	---	---	---	---	---	---

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C   Tire Pressure: 862 kPa   Wheel Load: 178.6 kg

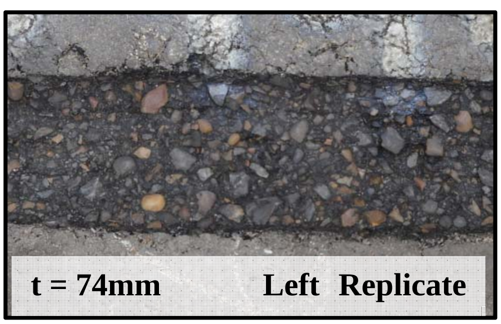


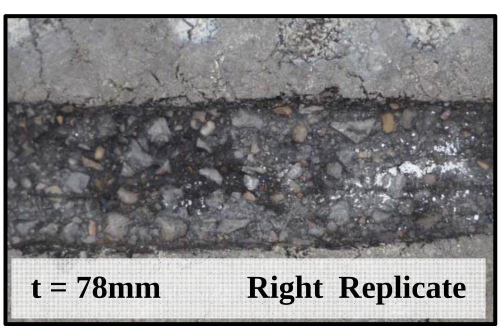
a) Replicate 1



b) Replicate 2

Figure A.23. PURWheel Wet Test Results for Strip 6





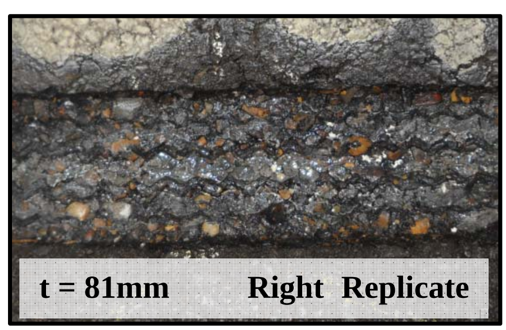
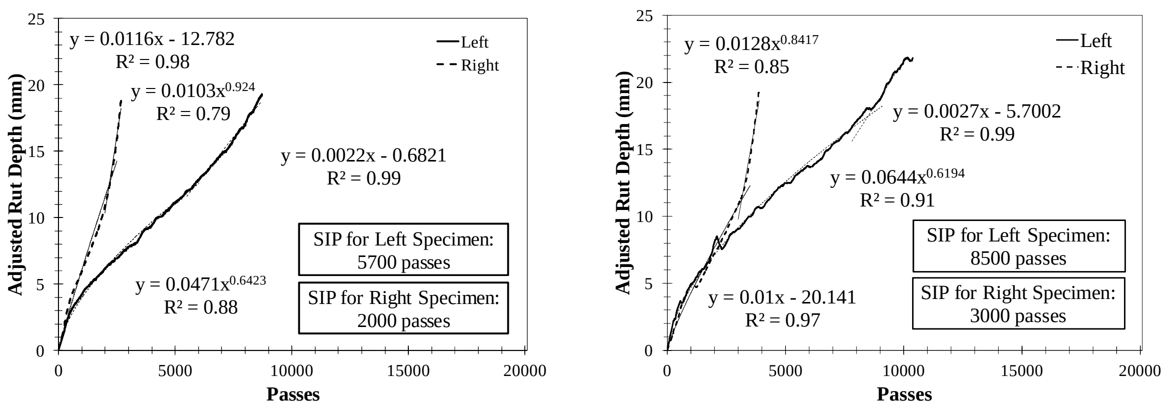


Table A.24. PURWheel Wet Test Results for Strip 7

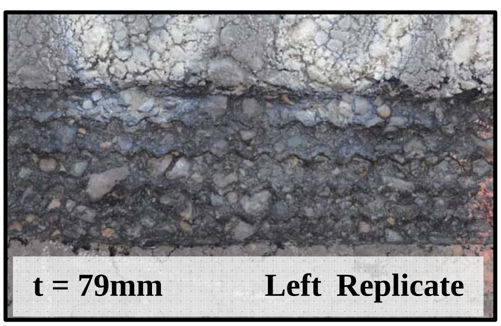
Replicate 1, Global Test ID 168				Replicate 2, Global Test ID 153 Left & 169 Right			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.5	250	1.9	250	2.1	250	1.3
500	2.9	500	3.9	500	3.4	500	2.7
1000	4.2	1000	6.0	1000	4.9	1000	4.9
2000	6.2	2000	10.9	2000	8.1	2000	7.2
4000	9.4	2684	19.0 **(---)**[1]	4000	10.6	3888	19.6 **(20.8)**[1]
8000	17.0	---	---	8000	17.1	---	---
8726	19.3 **(19.8)**[1]	---	---	10382	21.8 **(20.0)**[1]	---	---
---	---	---	---	---	---	---	---
---	---	---	---	---	---	---	---

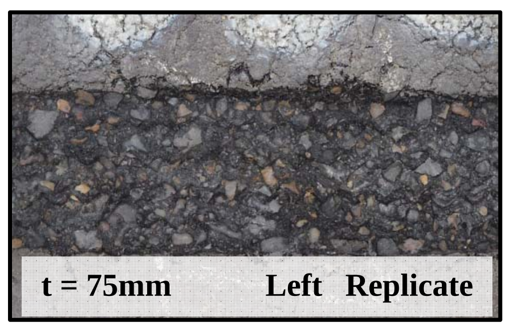
1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C   Tire Pressure: 862 kPa   Wheel Load: 178.6 kg

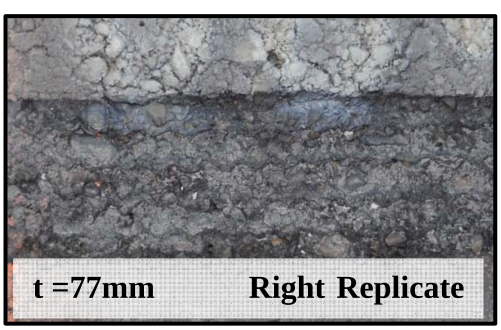


a) Replicate 1   *b) Replicate 2*

Figure A.24. PURWheel Wet Test Results for Strip 7







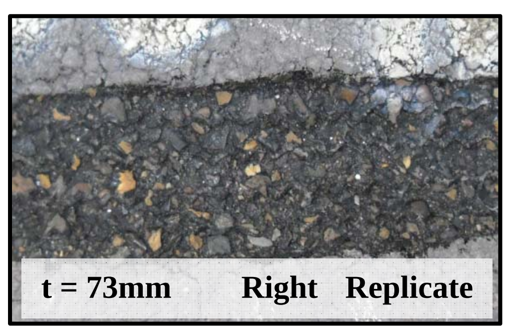
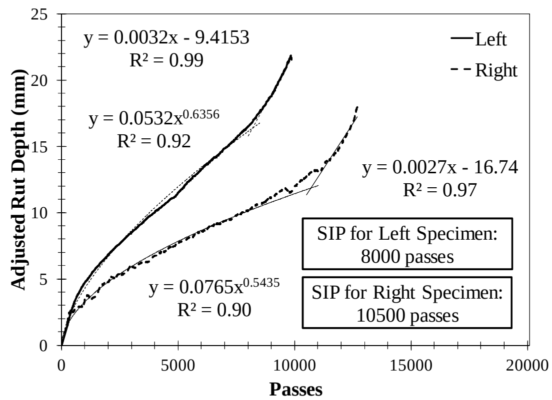


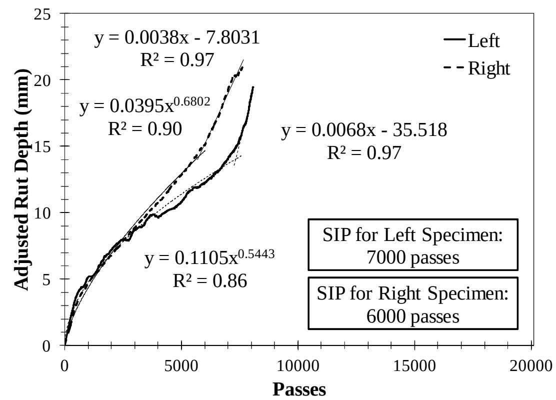
Table A.25. PURWheel Wet Test Results for Strip 8

Replicate 1, Global Test ID 128				Replicate 2, Global Test ID 129			
Left Specimen (mm)		Right Specimen (mm)		Left Specimen (mm)		Right Specimen (mm)	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.5	250	1.6	250	2.1	250	1.5
500	3.0	500	2.5	500	3.7	500	3.1
1000	4.7	1000	3.4	1000	5.1	1000	4.7
2000	6.8	2000	4.7	2000	7.2	2000	6.8
4000	10.0	4000	6.7	4000	9.7	4000	10.8
8000	16.5	8000	10.1	8000	18.9	7668	21.2 **(20.2)**[1]
9874	21.5 **(18.7)**[1]	12000	15.1	8076	19.5 **(21.1)**[1]	---	---
---	---	12700	18.1 **(20.3)**[1]	---	---	---	---
---	---	---	---	---	---	---	---

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

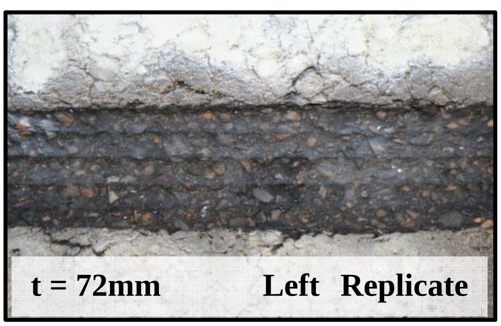


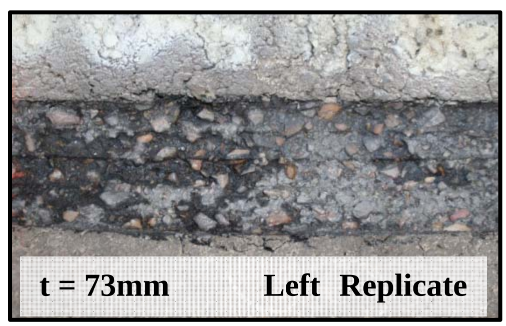
a) Replicate 1

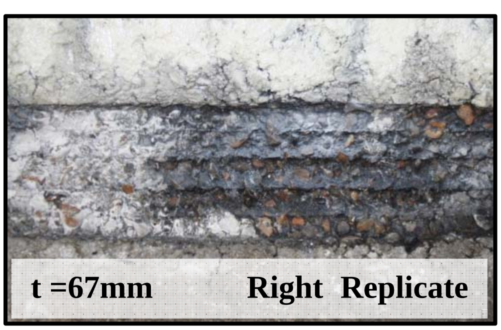


b) Replicate 2

Figure A.25. PURWheel Wet Test Results for Strip 8







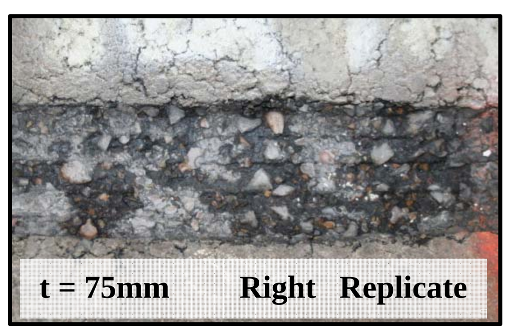
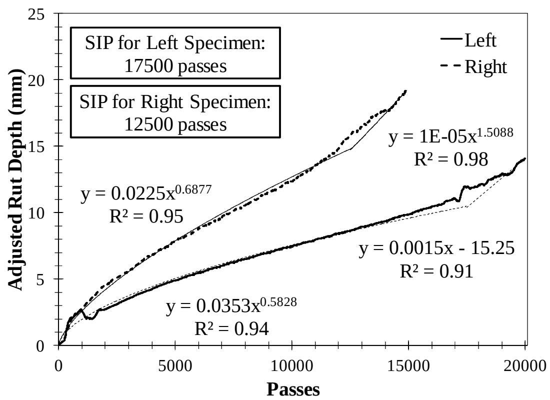


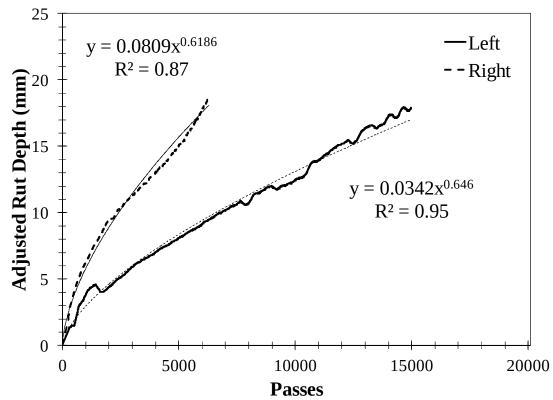
Table A.26. PURWheel Wet Test Results for Strip 9

Replicate 1, Global Test ID 132				Replicate 2, Global Test ID 133			
Left Specimen (mm)		Right Specimen (mm)		Left Specimen (mm)		Right Specimen (mm)	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.4	250	0.5	250	1.1	250	2.0
500	2.1	500	1.7	500	1.5	500	4.0
1000	2.6	1000	2.8	1000	3.8	1000	6.3
2000	2.7	2000	4.6	2000	4.4	2000	9.4
4000	4.2	4000	6.8	4000	7.0	4000	13.0
8000	6.5	8000	10.5	8000	10.7	6286	18.7 **(21.7)**[1]
12000	8.4	12000	14.8	12000	15.2	---	---
16000	10.4	14928	19.4 **(---)**[1]	14972	17.9 **(19.7)**[1]	---	---
20000	14.1 **(16.9)**[1]	---	---	---	---	---	---

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

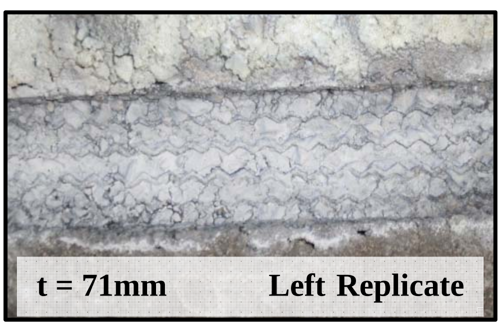


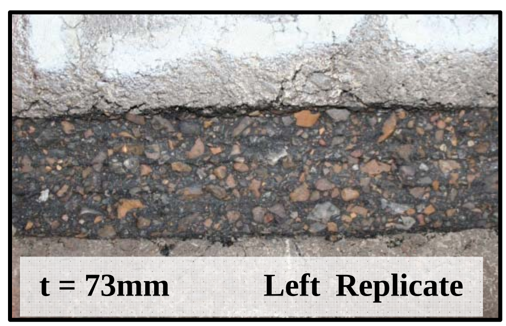
a) Replicate 1

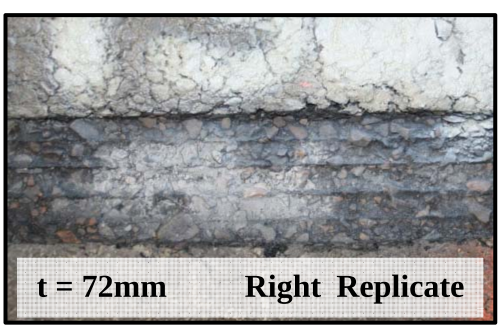


b) Replicate 2

Figure A.26. PURWheel Wet Test Results for Strip 9







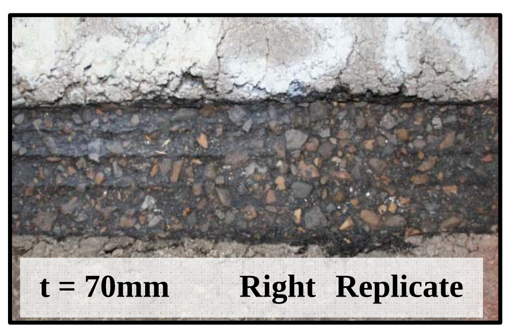
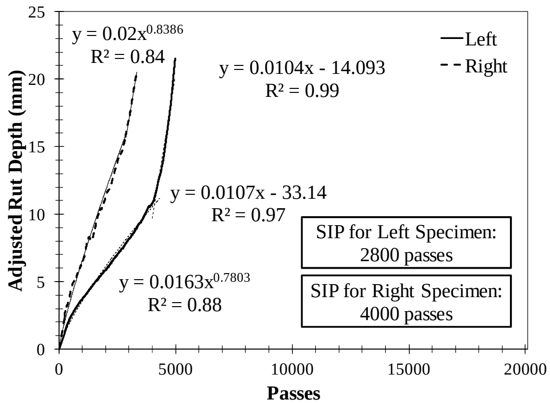


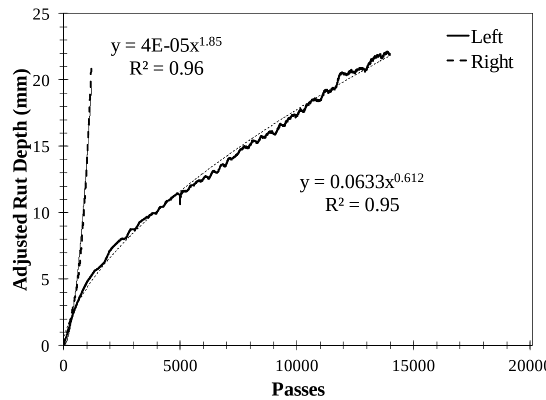
Table A.27. PURWheel Wet Test Results for Strip 10

Replicate 1, Global Test ID 158				Replicate 2, Global Test ID 163			
Left Specimen (mm)		Right Specimen (mm)		Left Specimen (mm)		Right Specimen (mm)	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.2	250	2.5	250	1.5	250	1.6
500	2.4	500	4.3	500	2.8	500	3.8
1000	3.7	1000	6.4	1000	4.8	1000	13.8
2000	5.9	2000	11.3	2000	7.2	1192	21.4 **(---)**[1]
4000	10.8	3330	20.7 **(23.1)**[1]	4000	10.0	---	---
4978	21.5 **(21.3)**[1]	---	---	8000	15.1	---	---
---	---	---	---	12000	20.5	---	---
---	---	---	---	13986	17.8 **(19.5)**[1]	---	---
---	---	---	---	---	---	---	---

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

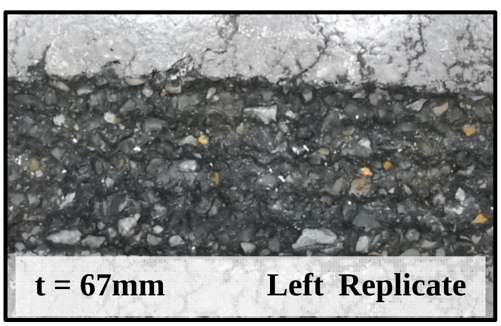


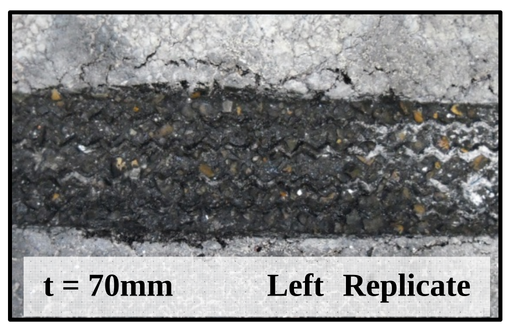
a) Replicate 1

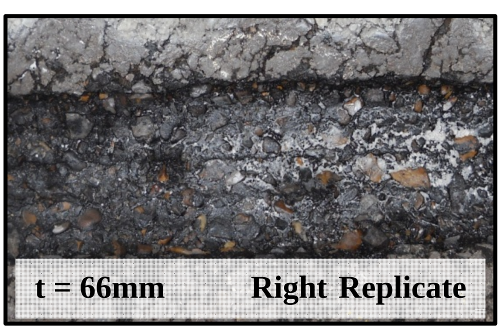


b) Replicate 2

Figure A.27. PURWheel Wet Test Results for Strip 10







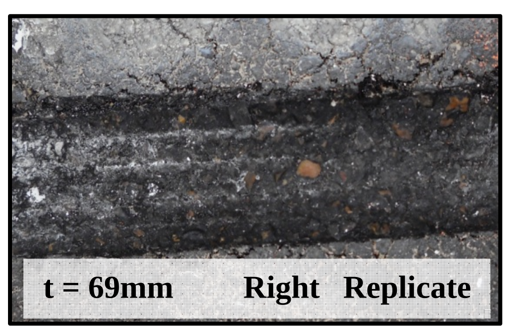
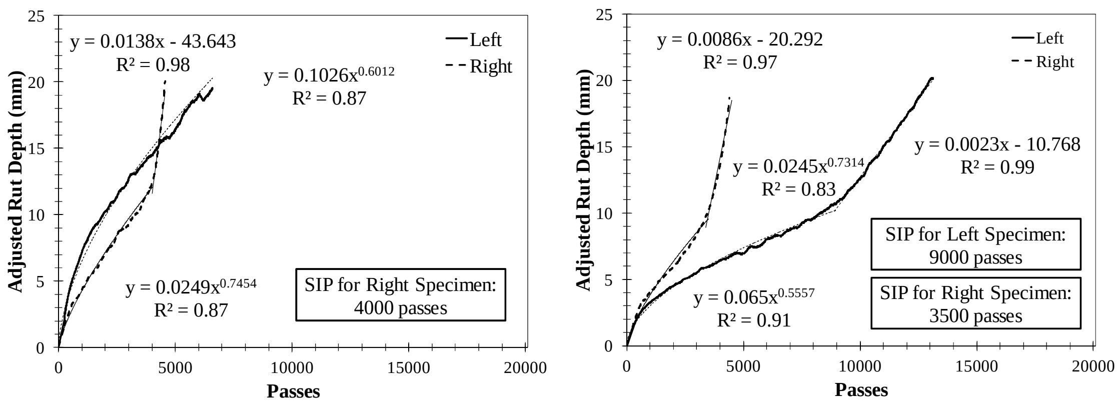


Table A.28. PURWheel Wet Test Results for Strip 11

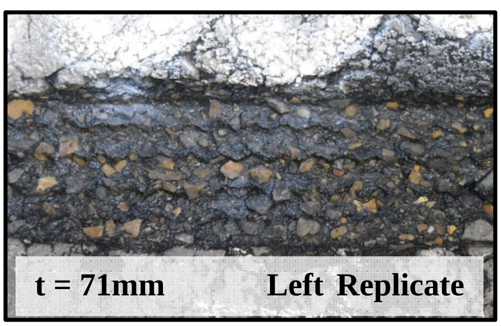
Replicate 1, Global Test ID 166				Replicate 2, Global Test ID 167			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	2.3	250	1.7	250	1.2	250	1.4
500	4.6	500	2.9	500	2.2	500	2.7
1000	7.2	1000	4.4	1000	3.3	1000	4.0
2000	10.3	2000	7.1	2000	4.5	2000	5.9
4000	14.5	4000	12.3	4000	6.4	4000	13.6
6590	19.5 **(20.6)**[1]	4558	20.1 **(---)**[1]	8000	9.6	4412	18.7 **(---)**[1]
---	---	---	---	12000	17.3	---	---
---	---	---	---	13124	20.2 **(19.5)**[1]	---	---
---	---	---	---	---	---	---	---

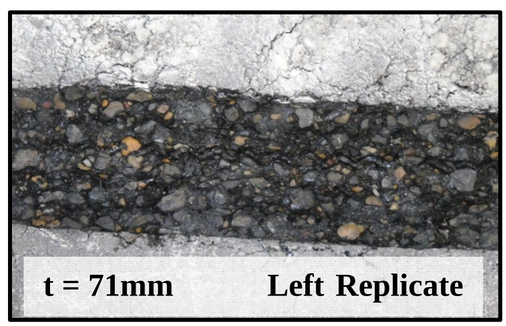
1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

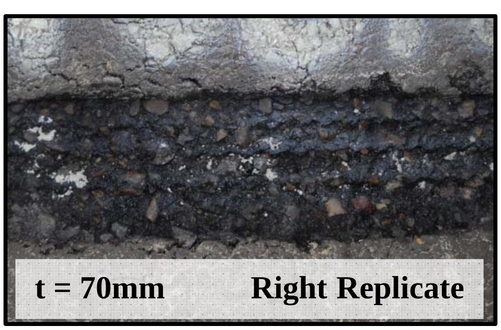


a) Replicate 1 *b) Replicate 2*

Figure A.28. PURWheel Wet Test Results for Strip 11







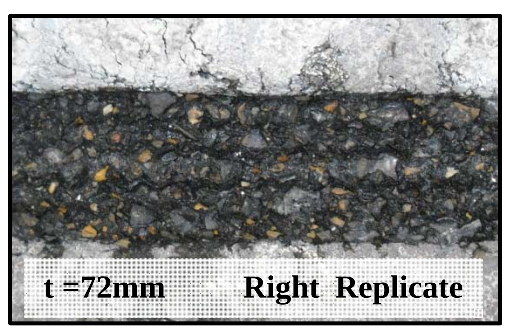
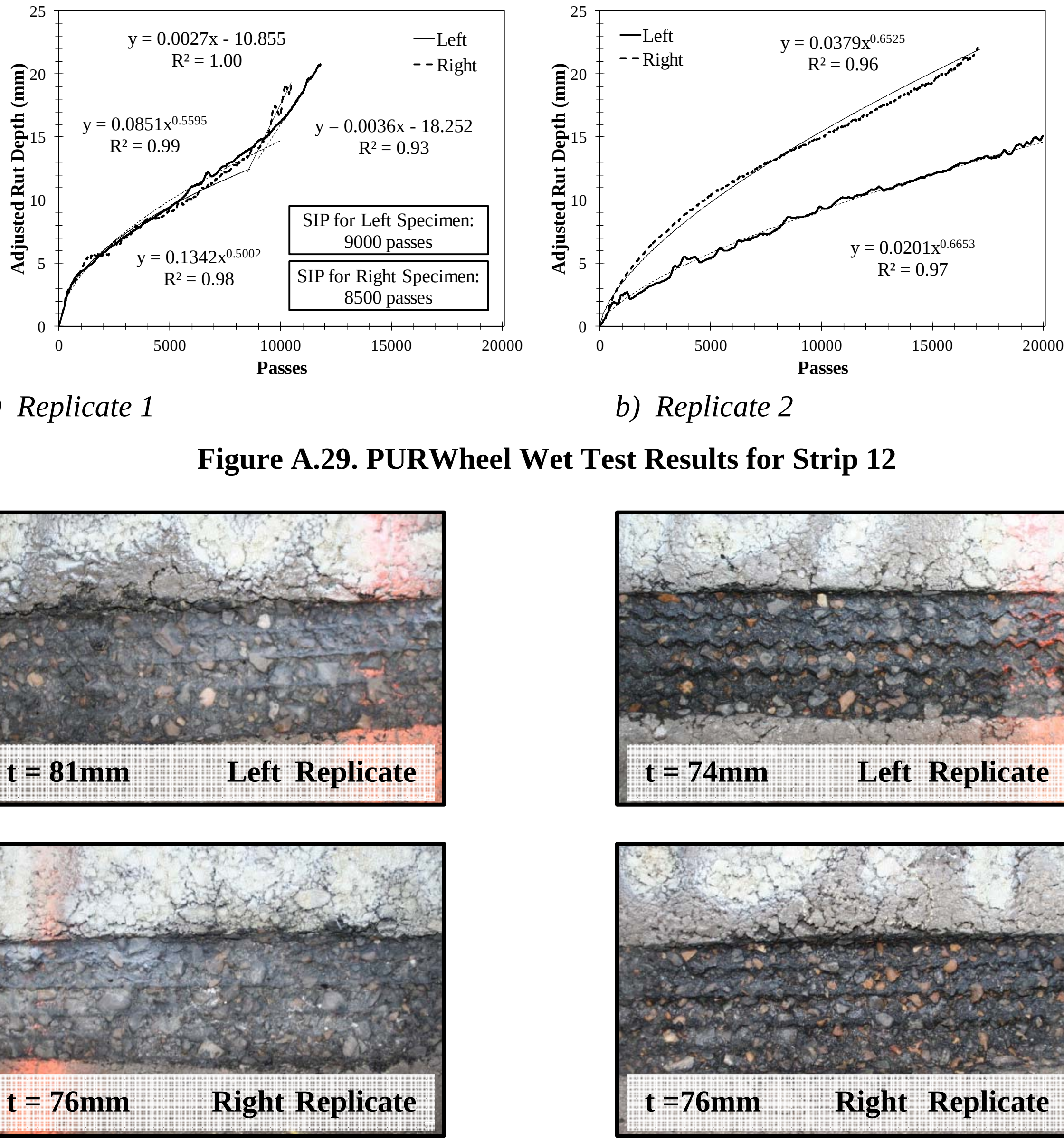


Table A.29. PURWheel Wet Test Results for Strip 12

Replicate 1, Global Test ID 136				Replicate 2, Global Test ID 137			
Left Specimen (mm)		Right Specimen (mm)		Left Specimen (mm)		Right Specimen (mm)	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.7	250	1.8	250	0.7	250	0.7
500	3.0	500	3.0	500	1.7	500	1.9
1000	4.3	1000	4.6	1000	2.5	1000	3.6
2000	5.8	2000	5.7	2000	2.9	2000	5.8
4000	8.3	4000	8.5	4000	5.3	4000	9.1
8000	13.3	8000	12.8	8000	7.7	8000	13.3
11810	20.8 **(23.7)**[1]	10474	19.1 **(19.4)**[1]	12000	10.5	12000	16.7
---	---	---	---	16000	12.7	16000	20.4
---	---	---	---	20000	15.1 **(17.3)**[1]	17100	22.0 **(21.2)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Replicate 1 *b) Replicate 2*

Figure A.29. PURWheel Wet Test Results for Strip 12

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