

Development and Implementation of a Mechanistic and Empirical Pavement Design Guide (MEPDG) for Rigid Pavements

ANNUAL REPORT FOR FY 2010

ODOT SPR ITEM NUMBER 2208

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October, 2010

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1.0 Overview

This document is an update of the progress of the research on ODOT project 2208

“Development and Implementation of a Mechanistic and Empirical Pavement Design Guide

(MEPDG) for Rigid Pavements”. This report summarizes the work that was completed at

Oklahoma State University between October 1st, 2009 and September 30th, 2010. The focus of

this project is on assisting ODOT in implementing the MEPDG into their rigid pavement design

practices. It was decided to best accomplish this goal by completing the following tasks:

- A. Review of the inputs to the MEPDG and determine the sensitivity on the final design values.
- B. Investigate base material practices for concrete pavements through a literature review and survey of experiences from others.
- C. Increase the quantity of weather sites in Oklahoma that provide environmental inputs for the MEPDG.
- D. Examine different curing methods for rigid pavement construction and their impact on the early age curling and warping of continuous reinforced concrete pavements
- E. Provide regional material input parameters that can be used in the MEPDG for the design of rigid pavements

In the first phase of the project task A and C was completed. Please refer to the report from

phase I for more information. During this period task B was completed and so is presented in

this report. Progress will also be given on tasks D and E.

1.1 Background of the MEPDG

The MEPDG is design software that was developed by Applied Research Associates (ARA) through several funding projects from the National Cooperative Highway Research Program (NCHRP). The goal of the software is to provide a new design methodology for concrete and asphalt pavements based on the latest failure mechanisms in combination with empirical data from the performance of pavements in the field. Before the release of the MEPDG it was common for designers to use a version of the AASHTO design guide. This design method has seen several different iterations that vary from hand methods that use nomo-graphs to simple software interfaces. The AASHTO design guide is based on empirical performance of several miles of test track in Ottawa, Illinois from 1958 to 1960. This testing is commonly called the AASHO road test. For this testing pavements were continuously loaded with trucks over a period of little more than 2 years.

While the AASHTO design guide has served designers well the following criticisms were made by the MEPDG documentation of the AASHO road test (ARA 2004):

1) Modern traffic levels have increased by 10 to 20 times the levels since the time of the AASHO road test. Because only a limited amount of data could be obtained from the original test, extrapolation of the damage observed in the AASHO road test was needed to determine the long-term performance of the pavements. While some extrapolation was deemed reasonable to determine the performance of pavements in the 1950s; however, this extrapolation would need to be taken to the extreme to meet modern traffic levels.

2) Environmental loading is thought to be an important component in the design of concrete pavements. Since the AASHO road test was only limited to pavements in Ottawa, Illinois and to a short period of little more than 2 years this key component cannot be modeled.

- 3) A limited number of construction materials were used in the construction of the test track.
For example only one type of hot mix asphalt subgrade and only one concrete mixture was used.
- 4) The vehicle weights used for the test are now out dated.
- 5) The drainage system for the pavement has not been considered in the test.
- 6) Pavement rehabilitation procedures were not considered by AASHTO design guide.

The MEPDG has done its best to try and take as many of these variables as possible into account. The creators of the MEPDG feel that these short comings can be overcome if one is able to fundamentally define the performance of a pavement through the use of the latest mathematical models in combination with the measurement of the actual performance of pavements with a significant number of differences in climate, loading, and construction materials. These empirical observations are imperative to help the mathematical expressions to become meaningful and useful.

2.0 Investigate base material practices for concrete pavements through a literature review and survey of experiences from others

2.1 Introduction

The use of sub-grade drainage systems, in the form of permeable bases and/or the incorporation of edge drains, has over the last few decades been considered an option for improving the long term performance of concrete pavements. The effectiveness of these features as a means of draining water and consequently extending the life of a roadway is still unclear. A major problem with this subject is that past investigations have not been able to monitor the performance of these drainage systems from their installation throughout the lifetime of the pavement to monitor their performance. Also, most projects have focused on

only the effectiveness of subsurface drainage systems without monitoring the structural effect of these systems over the life of the pavement surface.

For this task the research team has investigated the current literature over permeable bases and subsurface drainage for concrete pavements. Each document is summarized in a section. The summary was written to provide a pavement designer with as many useful findings as possible.

The most significant findings from this review was from “Pavement Subsurface Drainage Design Reference Manual” (FHWA-NHI-08-030). This document states the designs of the subgrade, drainable base, separator layer, and water removal system should be completed as a system. Guidance is provided to help the designer best to choose these systems for different cases and materials. This allows the designer flexibility to adjust their subgrade design based on other materials.

Next the findings from several research projects are summarized. These include National Cooperative Highway Research Program’s (NCHRP) Project 1-34, “Performance of Pavement Subsurface Drainage” and studies from several other DOTs.

2.2 Pavement Subsurface Drainage Design Reference Manual (FHWA-NHI-08-030)

2.2.1 Introduction

The objective of this report is to summarize the current design and construction methods recommended by the FHWA regarding subsurface drainage systems that incorporate permeable bases. Although permeable bases are concentrated on in this report, the construction of an effective drainage system depends on the success of all components within the system. For this reason, specifications and discussion for bases, sub-bases, sub-grade and water removal

components are provided as well. Each of these components have multiple design options thus after a summary of each component, comprehensive drainage system options will be provided. The basis for this report is the Pavement Subsurface Drainage Design Reference Manual (FHWA-NHI-08-030) which was a supplemental document for a short course given by FHWA and NHI. The document represents the culmination of experiences from Departments of Transportation across the nation as well as research project findings, thus the specifications and methods described are founded in the success and failures of drainage systems across the country. For this reason, this manual was used as a reference for all statements made herein.

2.2.2 Subsurface Drainage Systems

The main objective of any subsurface drainage system is to remove excess water from the pavement structure, then deflect the water to an appropriate location. The need for such a system is evident in the list of distresses due to excessive or prolonged moisture. Table 2.1 summarizes the distresses found in PCC pavements due to moisture. The prevention of distresses and thus the extension of a pavement's service life is the goal of every drainage design, but there are many ways that a systems function or an individual components function may be compromised. The specifications given herein reflect current measures to prevent the failure of each component but it is critical to understand the mechanisms that the design specifications are trying to prevent. The structure of this report is such that these mechanisms are explained so that design engineers may understand why components have certain limitations and when to turn to another viable drainage design. Although there are multiple means by which excess water may enter the pavement, drainage systems are only intended and designed to handle surface infiltration, the flow of water through the top of the pavement. The implications of this statement will be discussed further in the design section of the report. A

standard profile for a roadway with a subsurface drainage design system includes a pavement layer, a drainable base, a separator layer and/or longitudinal drainage, and the subgrade. Figure 2.1 below displays a typical profile for a PCC pavement with a basic drainage system.

Table 2.1 - Moisture Related Distresses (adapted from Carpenter et. Al 1979)

Type	Distress Manifestation	Moisture Problem	Climatic Problem	Material Problem	Load-Associated?	Structural Defect Begins in		
						PCC	Subbase	Subgrade
Surface Defects	Spalling	Possible	Freeze-thaw Cycles	Mortar	No	Yes	No	No
	Scaling	Yes	Freeze-thaw Cycles	Chemical Influence	No	Yes Finishing	No	No
	D-Cracking	Yes	Freeze-thaw Cycles	Aggregate Expansion	No	Yes	No	No
	Crazing	No	No	Rich Mortar	No	Yes Weak Surface	No	No
Surface Deformation	Blow-up	No	Temperature	Thermal Properties	No	Yes	No	No
	Pumping and Erosion	Yes	Moisture	Inadequate Strength	Yes	No	Yes	Yes
	Faulting	Yes	Moisture-Suction	Erosion-Settlement	Yes	No	Yes	Yes
	Curling/Warping	Yes	Moisture & Temperature	Moisture and Temperature Differentials	No	Yes	No	No
Cracking	Corner	Yes	Moisture	Cracking follows Erosion	Yes	No	Yes	Yes
	Diagonal Transverse Longitudinal	Yes	Moisture	Follows Erosion	Yes	No	Yes	Yes
	Punchout (CRCP)	Yes	Moisture	Deformation Follows Cracking	Yes	No	Yes	Yes

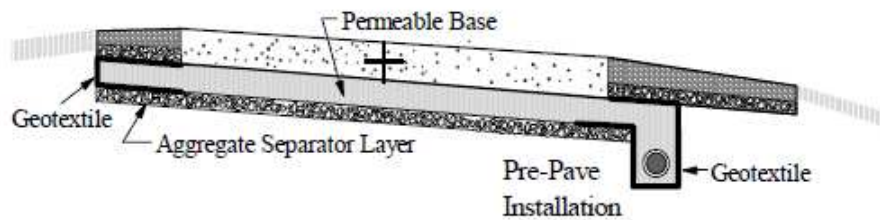


Figure 2.1 - Typical cross-section of subsurface drainage system (FHWA 1994)

Each component serves a distinct purpose in the overall function of removing water from the structure. The role of the permeable base is to allow excess water in the pavement structure to quickly drain. It achieves this role by balancing a high permeability, which allows for quick draining, and a reasonable stability for supporting the pavement structure and construction traffic. This balance of stability and permeability define the two different approaches to constructing permeable bases. Unstabilized permeable bases use a dense-graded aggregate to achieve stability through aggregate interlock but lack a high permeability due to their gradation. Stabilized permeable bases, which are generally more open-graded, are treated with asphalt or cement to add stability to an otherwise unstable gradation. The open gradation offers a high permeability. The role of the separator layer or sub-base is to direct water that passes through the permeable base away from the substructure as well as prevent the intermixing of layers. A very dense-graded aggregate layer with a low permeability and/or a geotextile layer are typically used to accomplish this. Removing water via the separator layer is accomplished by either daylighting the layer or using longitudinal drains with there being multiple viable options for drain schemes. The subgrade is to serve as a solid foundation for the structure above, as are all layers that constitute the pavement structure and substructure. Figure 2.2 below visualizes the entire process described above.

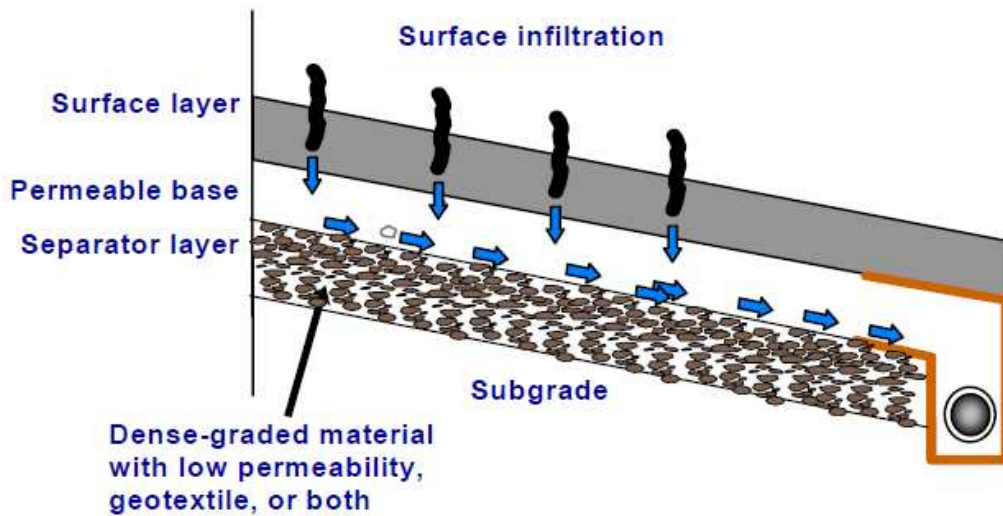


Figure 2.2 - Concept of drainage provided by separator layer (FHWA-NHI-08-030)

Over the years, a number of factors have been discovered that can limit the effectiveness of a subsurface drainage system. Each system component is susceptible to loss of performance given certain conditions. FHWA has reflected these conditions in their recommendations for the design and construction of each component. Following is a summary of recommendations for each component type along with an explanation behind the recommendation.

2.2.3 Permeable Bases

Creating a base layer that is permeable enough to quickly drain excess water from the pavement while maintaining the structural integrity to support construction traffic before paving and the load of the pavement after has proven to be a precarious balance. Many of the specifications to follow are to ensure that this balance is maintained.

2.2.3.1 Material Specifications

Beginning with aggregate qualities, FHWA specifies a number of characteristics including class, texture, soundness, sand fraction and wear that must be met to ensure the strength and stability of the base. These specifications along with the testing procedures needed to verify them are provided in Table 2.2. These specifications are to apply for both unstabilized and stabilized permeable bases. The gradation of the aggregate available for use in the permeable base is a key factor for determining if an unstabilized permeable base is sufficient for a design. Typically a more open-graded aggregate is needed to ensure an adequate permeability, but if the gradation is too open then the base will lack stability, thus the need for stabilization through asphalt or cement treatment. Many states typically use AASHTO No. 57 and 67 aggregate because the gradations reflect stability and a high permeability although No. 57 has a low coefficient of uniformity which implies low stability and strength. For this reason, No. 57 aggregate is a good choice for a stabilized permeable base.

Some useful tables have been included from the FHWA report. Table 2.3 summarizes gradations commonly used for unstabilized bases. Tables 2.4 and 2.5 summarize gradations used for asphalt treated and cement treated permeable bases respectively. It should also be noted that if the road being constructed is to handle a heavy traffic load, a stabilized permeable base is recommended due to the added strength and stability.

Table 2.2 - Recommended physical property requirements for aggregates used in permeable bases
(FHWA-NHI-08-030)

Specification	Requirement	Test Method
Aggregate classification	Aggregates for the permeable base should at least meet the requirements for a class B aggregate in accordance with AASHTO specifications.	AASHTO M 283-83, <i>Coarse Aggregate for Highway and Airport Construction</i>
Aggregate wear	Maximum L.A. abrasion wear should be 40 to 45 percent.	AASHTO T 96-87, <i>Resistance to Abrasion of Small-Size Coarse Aggregates by Use of the Los Angeles Machine</i>
Aggregate texture	Use only crushed aggregate material with at least two mechanically fractured faces, as determined by the material retained on the 4.75-mm (#4) sieve.	ASTM D 5821-95, <i>Determining Percentage of Fractured Faces in Coarse Aggregates</i>
Aggregate soundness	For pavements subjected to freeze-thaw, permeable base aggregates soundness loss should not exceed 12 percent, as determined by the sodium sulfate soundness test, or 18 percent, as determined by the magnesium sulfate test.	AASHTO T 104 (ASTM C 88), <i>Soundness of Aggregate by the Use of Sodium Sulfate or Magnesium Sulfate</i>
Fraction of natural sand	Not more than a maximum of 15 percent natural sand by weight is recommended in aggregate mixtures for permeable bases.	ASTM C 125-88, <i>Standard Terminology Relating to Concrete and Concrete Aggregates</i>

Table 2.3 - Unstabilized aggregate permeable base gradations. (FHWA 1994)

Gradation	Percent Passing Sieve Size											
	50 mm (2 in)	37.5 mm (1½ in)	25.4 mm (1 in)	19.0 mm (¾ in)	12.5 mm (½ in)	9.52 mm (⅜ in)	4.75 mm (#4)	2.36 mm (#8)	1.18 mm (#16)	0.425 mm (#40)	0.30 mm (#50)	0.075 mm (#200)
AASHTO #57		100	95-100		25-60		0-10	0-5				
AASHTO #67			100	90-100		20-55	0-10	0-5				
Iowa			100					10-35			0-15	0-6
Minnesota			100	65-100		35-70	20-45			2-10		0-3
New Jersey		100	95-100		60-80		40-55	5-25	0-12		0-5	
Pennsylvania	100			52-100		33-65	8-40		0-12			0-5

Table 2.4 - Asphalt-stabilized permeable base. (NAPA 1991)

State	Percent Passing Sieve Size					
	25 mm (1 in)	19 mm (¾ in)	9.5 mm (3/8 in)	4.75 mm (#4)	2.36 mm (#8)	0.075 mm (#200)
California	100	90-100	20-45	0-10		0-2
Florida	100	90-100	20-45	0-10	0-5	0-2
Illinois	90-100	84-100	40-60	0-12		
Kansas	100	90-100	20-45	0-10	0-5	0-2
Ohio	95-100			25-60	0-10	
Texas	100	95-100	20-45	0-15	0-5	2-4
Wisconsin	95-100	80-95	25-50	35-60	20-45	3-10
Wyoming	90-100	20-50		20-50	10-30	0-4

Table 2.5 - Cement-stabilized permeable base (FHWA 1994)

State	Percent Passing Sieve Size						
	37.5 mm (1½ in)	25.4 mm (1 in)	19 mm (¾ in)	12.5 mm (½ in)	9.5 mm (3/8 mm)	4.75 mm (#4)	2.36 mm (#8)
California	100	88-100	X + 15*		X + 15*	0-16	0-6
Virginia		100		25-60		0-10	0-5
Wisconsin		100	90-100		20-55	0-10	0-5

* The value X should be proposed by the contractor.

If it is determined that a stabilized permeable base is required, it is necessary to ensure that the asphalt or cement treatments used will create a stable and workable mixture without hindering the base's ability to drain water quickly. The intention is to coat aggregates and allow contact points between them to "cement". Recommendations suggest the treatments meet certain specifications including content, workability, permeability and grade. These recommendations are displayed in Tables 2.6 and 2.8, while Table 2.7 lists the asphalt treatments used in various states. Generally, asphalt or cement content in a mixture is a function of the gradation of the aggregate used because more open-graded aggregates have less surface area and thus need less treatment to coat them. Most agencies use 1.5 to 2.5 percent

asphalt content by weight. For cement treated bases, an application rate of 8 lb/ft³ to 10.5 lb/ft³ is recommended.

Table 2.6 - Recommended asphalt cement stabilizer properties (FHWA-NHI-08-030)

Specification	Requirement	Test Method
Asphalt cement content	Asphalt content must ensure that aggregates are well coated. Minimum recommended asphalt content is 2 percent by weight. Final asphalt content should be determined according to mix gradation and film thickness on coarse aggregates.	ASTM D 2489, <i>Test Method for Degree of Particle Coating of Bituminous-Aggregate Mixtures</i>
Asphalt cement grade	Recommended grade is AC-30 or stiffer.	ASTM D 946, <i>Penetration-Graded Asphalt Cement for Use in Pavement Construction</i> ASTM D 3381, <i>Viscosity-Graded Asphalt Cement for Use in Pavement Construction</i>
Anti-stripping	See figure 6.3.	Lottman Test (Lottman 1978) Modified Lottman Test (Tunnicliff and Root 1984) ASTM D 1075-81, <i>Test Method for Effect of Water on Compressive Strength of Compacted Bituminous Mixtures</i>
Anti-stripping agents	Aggregates exhibiting hydrophilic characteristics can be counteracted with 0.5 to 1 percent lime.	NCHRP Report No. 274 (Tunnicliff et al. 1984)
Permeability	Minimum mix permeability is 300 m/day (1000 ft/day).	AASHTO T 3637, <i>Permeability of Bituminous Mixtures</i>

Table 2.7 – Asphalt stabilized permeable base mix material specifications (NAPA 1991)

State	Anti-Strip Test	% Asphalt	Asphalt Grade	Permeability Test
California	Yes	3.0	AR-8000/AC-40	No
Florida	No	2.5	AC-30	No
Georgia	Yes	3 – 3.5	AC-30	Constant Head
Illinois	No	2 – 3	AC-20	No
Indiana	No	3 – 4.5	AC-20	AASHTO T-215
Kansas	No	2.0	AC-10	Constant Head
Kentucky	No	1.5 – 2.5	AC-20	No
Maryland	Yes	2.5 – 3.5	AC-20	AASHTO T-167
Ohio	-	2.0	AC-20	No
Pennsylvania	No	1.5 – 3.5	AC-20	No
Tennessee	Yes	2.5 – 7.0	AC-20/AC-30	No
Texas	-	2.0 – 4.0	AC-10/AC-20	No
Virginia	Yes	2.5 – 3.5	AC-30	Constant Head
Washington	Yes	1.5 – 2.5	AR-4000	No
Wisconsin	No	1.5 – 2.5	60/70-200/300 pen	Falling Head
Wyoming	Yes	-	AC-20	Yes

Table 2.8 - Recommended Portland cement stabilizer properties (FHWA-NHI-08-030)

Specification	Requirement	Test Method
Portland cement content	Portland cement content selected must ensure that aggregate are well coated. An application rate of 130 to 170 kg/m ³ (8.1 to 10.6 lb/ft ³) is recommended.	ASTM C 1078, <i>Determining Cement Content of Freshly Mixed Concrete</i>
Water-cement ratio	Recommended water-cement ratio to ensure strength and workability is 0.3 to 0.5.	ACI <i>Manual of Concrete Practice</i> , Part 1, American Concrete Institute. ASTM C 1079, <i>Determining Water Content of Freshly Mixed Concrete</i>
Workability	Mix slump should range from 25 to 75 mm (1 to 3 inches)	ASTM C 143, <i>Test Method for Slump of Portland Cement Concrete</i>
Cleanliness	Use only clean aggregates.	ASTM C 33, <i>Concrete Aggregates</i>
Permeability	Minimum mix permeability is 300 m/day (1000 ft/day).	

2.2.3.2 Construction Specifications

As equally important as the material specifications for a drainable base are certain minimum requirements for the construction of the layer. Again these specifications were formulated to ensure that the drainable base is permeable enough to allow a coefficient of permeability of at least 1000 ft/day but stable enough to support the designed traffic flow for the roadway. Generally speaking if the permeable base is able to withstand construction traffic without showing signs of pumping or rutting, it should be stable enough to support the design traffic for the roadway.

FHWA recommends that the drainable base should be at least 4 inches thick to overcome any construction variances and provide an adequate hydraulic conduit to transmit water to an edge drain.

There are also concerns when compacting the permeable base during construction, including crushing of the aggregate, intermixing of sub layers, and general overcompaction. With this in mind, suggestions have been made based on successful compaction techniques used by other DOT's. For untreated permeable bases, most agencies specify one to three passes of a 5 to 10 metric ton steel roller. The maximum lift thickness should not exceed 6 inches. Vibratory rollers should be used with extreme caution with all base types because of intermixing concerns. It was found that in order to prevent intermixing, a geotextile separator layer should be used underneath the permeable base to prevent pumping and mixing. For permeable asphalt treated bases, the most common problem reported occurred when the base material was rolled at elevated temperatures. Although compaction procedures are the same as those listed above, in most cases compaction was deferred until the next day or treated with water before hand. Cement treated permeable bases present problems with workability during the paving process. Typically a paver should follow a spreading machine, but in the event that the workability of the mixture causes issues with the paver, a subgrade paver should be used in its place. In

order to prevent further problems a mix should be designed with a minimum water to cement ratio of 0.45 and the minimum cement content which still meets other strength requirements. For compaction, the Wisconsin and Illinois DOT's have had success with attaching vibratory pans to the subgrade planer.

2.2.4 Separator Layer

The separator layer is the layer of soil, rock or fabric below the permeable base meant to deflect water which passes through the permeable base away from the subgrade. It also serves as a stable platform for construction of the permeable base and pavement layers as well as a means for preventing fines in the subgrade from intermixing with the permeable base. In order to satisfy these requirements, the separator layer must be stable enough to support construction traffic and have a low enough permeability to prevent the flow of water or the pumping of fines.

The three basic types of separator layers are untreated aggregate, treated aggregate and a combination of a geotextile with an aggregate layer. The reference manual provides the designer with a table (T7.1) to help in selecting the most appropriate type of separator layer. The key criteria for selection are rainfall, design traffic and subgrade properties.

2.2.4.1 Material Specifications

After a separator layer type has been selected, the design process begins with determining proper material properties. In the case of an untreated aggregate separator layer, the reference manual provides a typical gradation suitable for the design purposes mentioned earlier. This gradation is shown in Table 2.9 below. However, the given gradation must be adjusted to account for the gradation of the permeable base above and the subgrade below. This is to ensure that intermixing at the interfaces is

limited. Equations 7.1 through 7.4 in the reference manual detail the nature of these changes to the separator layer.

Table 2.9 - Aggregate separator layer gradation (FHWA-1994a)

Sieve Size, mm	U.S. Customary Sieve Size	Percent Passing
37.5	1½ in	100
19	¾ in	95 – 100
4.75	#4	50 – 80
0.425	#40	20 – 35
0.075	#200	5 – 12

Aggregate gradation for a treated separator layer should be similar to that of an untreated layer. This is because the deflection of water and the prevention of intermixing of layers are the primary purposes for the separator layer. An asphalt or cement treatment should be used to increase stability but this should not be a trade off for permeability. Asphalt and cement contents should be determined similarly to typical treated subbases. Guidelines given previously for treatment contents and properties with permeable bases may be used as a guideline.

The use of a geotextile layer alone should only be used in cases where an aggregate layer is not possible. This could be possible where height requirements are a concern such as under bridges. Otherwise, a geotextile layer should be used in conjunction with a subbase. In these circumstances the geotextile layer only serves as protection from the intermixing of layers. The role of water deflection is then deferred to the subbase or subgrade layer. This means that the permeability should be high enough to allow passage of water without impediment. A typical failure generally results from the clogging of the geotextile, resulting in backup of water within the permeable base. In order to prevent this, calculations must be done to determine the apparent opening size of the pores in the fabric as well as the gradient ratio. These values are based on the properties of the layers above and below the geotextile. More

information on the procedures for design of the geotextile can be found in section 7 of the reference manual, in particular Figure 7.7 which displays a flow chart for design criteria. Also page 7-19 in the reference manual provides detailed design guidelines.

2.2.4.2 Construction Specifications

In addition to material specifications, there are also certain construction procedures that must be followed. Among these for an aggregate separator layer are a minimum thickness of 4 inches and a density of 95 percent of the modified Proctor maximum density. The aggregate layer should also show no signs of distress prior to the placement of the permeable base. Treated bases should be constructed in accordance with typical asphalt or cement treated base procedures.

2.2.5 Drainage

After the water has been deflected to the sides of the substructure by the separator layer, a system must be in place to continue water flow away from the substructure. This can be accomplished by either daylighting the permeable base or by using a system of longitudinal edge drains. Typically daylighting is sufficient for normal design flows, but in areas of high annual rainfall, edge drains may need to be considered. Also, the geometry of the roadway may require the use of edge drains to ensure water does not puddle inside of the substructure.

The function of the drainage system is certainly critical to the success of a permeable base in protecting the roadway from moisture related distresses. In fact, most of the issues reported from state DOT's originate with failure of the drainage system, either from lack of proper maintenance or poor construction techniques. The reference manual provides a detailed account of these collective experiences and bases its guidelines on this knowledge.

There are many types of edge drains including pipes, prefabricated geocomposites called fin drains, and aggregate trench drains. It is recommended that only pipe edgedrains be used. This is because both fin drains and aggregate trenches are unable to be maintained and flushed. As stated earlier, maintenance is the key to drainages systems working properly over time so only pipe edge drains will be discussed.

Figure 2.3 displays a typical profile of a pipe edge drain embedded in an aggregate trench.

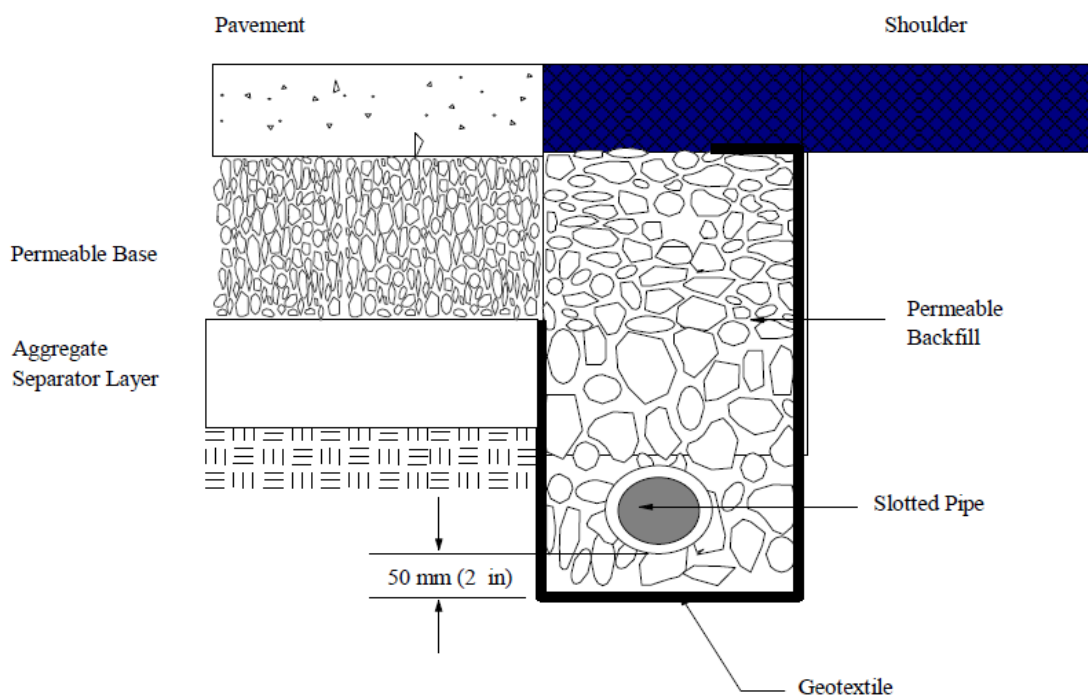


Figure 2.3 - Permeable base section with longitudinal edge drains. (FHWA-NHI-08-030)

It is recommended that the design of drainage components is based on the Time to Drain method. More information on this approach and its governing equations can be found in Section 8 of the reference manual.

2.2.5.1 Material Specifications

A typical design of an edge drain system will include longitudinal pipes, outlet pipes, an aggregate trench and typically some degree of geotextile wrapping around the trench. As with other components, the design starts with material specifications. The reference manual suggests that all longitudinal pipes be made of flexible, corrugated polyethylene (AASHTO M252) or PVC (AASHTO M278). It is noted that if an asphalt treatment is to be used that excessive temperatures may cause problems in the edge drains. In this case, PVC electric conduit like EPC 40 or 80 is suggested. Outlet pipes are exposed to much higher stresses from construction traffic and for this reason are recommended to have solid walls as opposed to the corrugated walls of longitudinal pipes. The backfill used around the longitudinal drains in the aggregate trench must be at least as permeable as the base material and is typically the same gradation as the base material. Also D_{85} of the backfill should be 1.2 times larger than the openings in of the corrugated pipe to prevent clogging of the longitudinal drains.

2.2.5.2 Construction Specifications

The minimum diameter for the longitudinal pipes should be 4 inches. This is not for flow reasons as this rarely controls sizing of the pipe, but to ensure maintenance techniques are effective. The drainage trench should be deep enough so that the top of each pipe may be at least 2 inches below the bottom of the permeable base as well as 2 inches of bedding below the pipe. For the minimum 4 inch diameter pipe, this puts the trench depth at 8 inches. During construction, a minimum of 6 inches of cover is required to protect the pipes during compaction. Figure 2.4 below provides information on specifics regarding outlet pipes and ditches. Figure 2.5 provides details for headwall construction. There are numerous other specifications given in Section 8 of the reference manual concerning construction of drainage systems involving longitudinal pipes. Please refer to this section for more information regarding specifics of geotextile placement, trench depths, construction overview, etc.

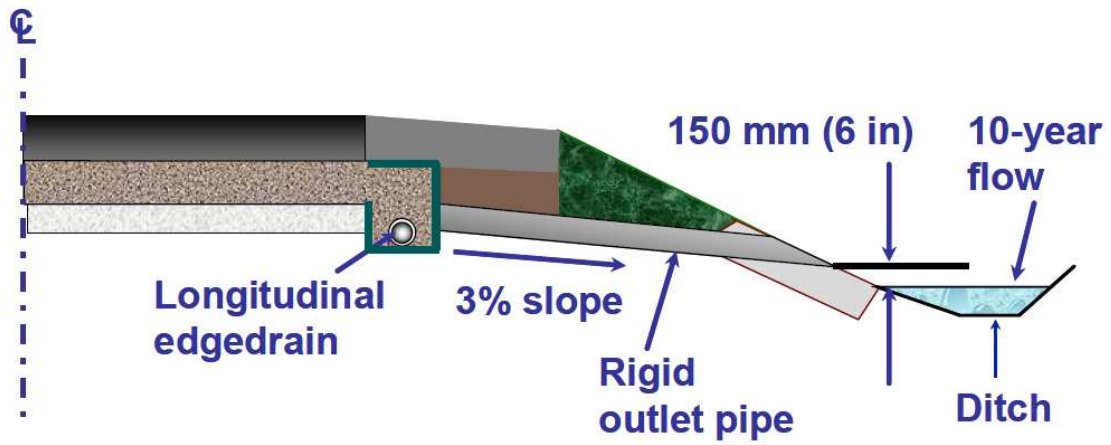


Figure 2.4 - Recommended design for edge drain outlet (FHWA-NHI-08-030)

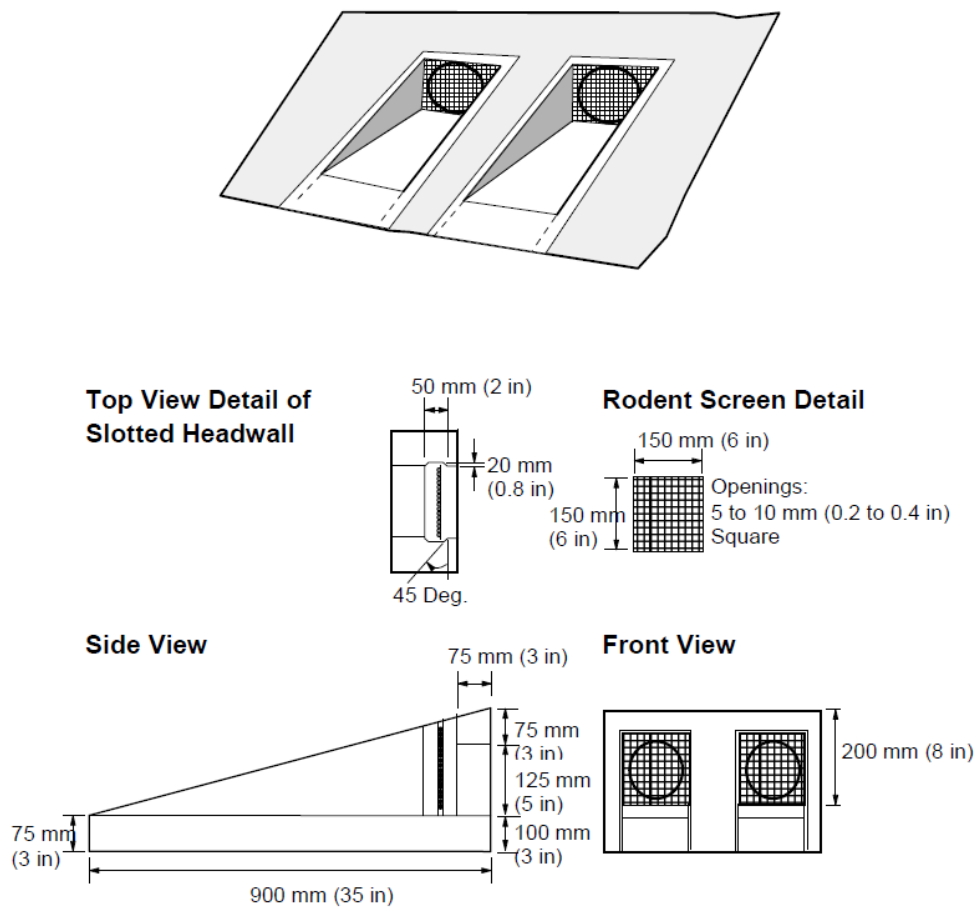


Figure 2.5 - Recommended headwall design for dual outlet system (FHWA-NHI-08-030)

2.2.6 Summary Guidelines

Overall the choices made on what option to use for a drainable base, separator layer and water removal system are often governed by a single characteristic of the site conditions – often subgrade quality. This is because the recommendations made above for each individual option often tie together into comprehensive design schemes.

For instance, a weak subgrade may require that a treated separator layer is used. A treated permeable base is then necessary to accommodate the separator layer. Daylighting the permeable base to remove water is now an acceptable option for the dominant portion of the roadway. This example demonstrates how the choices made for one component leads to choices for other components due to compatibility recommendations made above.

With this in mind, if a particular component option is desired, such as an untreated permeable base of a certain gradation, designers may follow compatibility requirements to design the remaining components. This example may require a geotextile separator layer to prevent intermixing of layers, longitudinal edgedrains, and a stabilized subgrade if the subgrade has insufficient strength.

2.3 “An Evaluation of IDOT’s Current Underdrain Systems”; Illinois Department of Transportation, 1995

Illinois as a state has been using underdrain systems since the 1970’s. In 1995 IDOT evaluated the effectiveness of pipe and mat underdrains. Both have been heavily used in the state and it was unclear if one performed better than the other. The experiment consisted of unearthing a section of shoulder at

52 locations which had underdrain systems. The removed sections were then examined for damage and later tested in a lab for flow rates. The results of which provide valuable recommendations for agencies considering either. The recommendations include:

- Discontinuing the use of polypropylene products because they tend to collect fines and lose functionality
- Discontinuing the use of two manufacturers drainage mats as they are prone to structural damage and loss of functionality; drainage mats are typically plastic with circular openings that are placed below the pavement surface and act as a highly permeable layer
- Revised maintenance procedures to ensure screens are in place at all drain outlets and that mowing occurs as close to the outlets as possible

2.4 “Evaluation and Analysis of Highway Pavement Drainage”; Kentucky Transportation Center, 2003

The Kentucky Transportation Center conducted an analysis of drainage system performance by utilizing finite element models to investigate various pavement designs incorporating subsurface drainage components. The models assumed a steady state saturated flow. In these models the drainage system components and pavement materials and conditions were varied. The project was interested in not only determining the effectiveness of drainage systems but what factors affect the inflow of water into the pavement layers. The results of this modeling led to the following conclusions:

- Pavement geometry affects surface drainage but not subsurface drainage
- Cracks in the pavement increase the inflow of water into subsurface layers and thus the need for subsurface drainage features
- For widening projects, longitudinal drains should be placed at the interface of new and old layers to shorten the drainage path

- A surface drainage layer with low permeability should have underlying layers with increasing permeability to ease the movement of subsurface water while still maintaining structural integrity

2.5 “Comparison of Pavement Drainage Systems”; MnROAD, 1995

This project detailed the effectiveness of drainage in four test sections. These sections were designed with varying subsurface drainage features including one control section without subsurface drainage designs. The remaining three sections utilized longitudinal drains and/or permeable asphalt treated layers. Reflectometers were placed in the constructed layers of the sections so that saturation and flow could be measured at the time of construction and after rain events. The conclusions for this experiment include:

- Although all sections demonstrated the ability to drain subsurface water, sections with a permeable asphalt treated layer drained the most volume of water, typically within two hours
- About 40% of all rainfall penetrates the pavement surface
- Sealing longitudinal and transverse joints provided protection from inflow for roughly two weeks before typical inflow resumed
- The project recommended that measurements continue to be made and that structural performance of the surface pavement be monitored.

2.6 NCHRP Project 1-34

In this section a summary of the methodology, results, and recommendations of all four phases is provided. Project 1-34, phase A was the first attempt for the NCHRP at characterizing the performance of subsurface drainage systems. This project was completed in 1998. Once complete, the NCHRP financed phase B to critically review the original project as well as establish a blue print for long-term evaluation. This plan was enacted upon in phase C of the project through the Special Pavement Study-2

(SPS-2). Phase C ran from 1998 until 2002. The final installment, phase D, continued analyses of the SPS-2 sections in addition to focusing on testing the long term functionality of the drainage systems. This phase included data through 2005.

2.6.1 Project 1-34 A and B

NCHRP Project 1-34A and B provided an initial look at the performance of subgrade drainage systems in use at that time. The bulk of observations made were from databases provided by the Federal Highway Administration (FHWA) and from visual distress surveys performed on rigid pavements found throughout the country. Data collected through these methods was then used to create mechanistic-empirical models. Visual surveys were completed for each section in which a verbal description was provided on the condition of the roadway. Rutting, fatigue cracking, and the condition of the drain outlets were the predominant comments for the surveys. Additionally, information was collected on the age, repair history, and traffic volume seen at each location. Table 2.10 provides a summary of the test sections surveyed. States represented in this phase include: Kansas, Minnesota, North Carolina, Pennsylvania, Wisconsin, Illinois, Oklahoma and Ontario. For JPCP sections, three of the nine locations investigated had drained and undrained sections at the same location; for JRCP and CRCP sections each had one location with both a drained and undrained sections.

Table 2.10 – Summary of pavement sections investigated in NCHRP 1-34A and B.

Pavement Type	JPCP	JRCP	CRCP
sections with permeable base	11	4	10
sections with edge drains	19	5	12
Number of Locations	9	3	4

The findings for this project were reported by identifying the pavement type and then summarizing visual distress surveys. Ideally, direct comparisons between undrained sections and drained sections were made and the performance could be evaluated. However this was rarely possible. Most observations revealed that small, if any, statistical differences existed between drained and undrained sections if the undrained sections “were properly designed”. Of the observations where statistical evidence existed, conclusions made by the research team include:

- The number of deteriorated cracks in JRCP was lower for permeable bases
- Cement-treated permeable bases (CTB) should not be used in conjunction with CRCP due to excessive bonding
- Concrete sections with permeable bases averaged less than half the amount of deteriorated joints than that of sections with dense-graded bases
- Permeable bases are easily penetrated by fines
- Edge drain outlets must be well maintained to function properly due to vegetation overgrowth and other means of clogging such as rodent nests

Upon completion of this phase, the research team noted that any conclusions presented were with the limited amount of data available and should be investigated with a larger sample group along with more numerical methods as opposed to subjective visual reports. It was with this knowledge that Project 1-34B was able to create a plan that would eliminate many of the shortcomings of Project 1-34A. Among their suggestions for future research were longer analysis of sections, direct comparisons between drained and undrained sections at the same location to eliminate doubt about climactic and geological variables, and more advanced analytical techniques (coring, deflection data, roughness measurements, video inspection of edge drains).

2.6.2 Project 1-34 C

With the recommendations of Project 1-34B and the inclusion of SPS-2 sections, Project 1-34C undertook a long term evaluation of rigid pavements with subsurface drainage features. The SPS-2 experiment was designed to assess the influence of concrete width and thickness, flexural strength, base type, sub drainage, climate and traffic level.

Some specifics of the investigation is given below:

- Fourteen locations were investigated to represent varied climactic variables (rain and temperature)
- Each location has 12 sections of varying width, flexural strength and thickness – 4 drained and 8 undrained
- Every drained section (asphalt treated permeable base) has two control sections – 1 dense graded aggregate and 1 lean concrete base

The focus of this project was on the structural performance of the trial sections, thus parameters reflecting the structural integrity were measured throughout the project. Parameters of interest include transverse and longitudinal cracking, faulting, rutting and the International Roughness Index (IRI). IRI

calculations were made by averaging the IRI value of each wheel path at the time of construction and periodically throughout the phase. The conclusions for this project were framed around the statistical differences in these parameters by comparing numerical values of undrained sections versus drained sections.

The team found that for transverse and longitudinal cracking as well as faulting, the control sections tended to deteriorate first, although in most cases the differences were statistically insignificant. In the case of IRI, they determined that the quality of drainage was not a factor.

Recommendations from the 1-34C team include:

- Adding deflection data to the list of structural integrity parameters used above (cracking, faulting, rutting and IRI)
- Testing the capabilities of the drainage systems in place by measuring flow rates
- Determining the effect of filter fabrics on flow rates
- Adding the data from the SPS-2 to the most recently completed database

The team found that the largest shortcoming of their findings was differentiating what effects were due to base type and which were due to drainage capabilities.

2.6.3 Project 1-34 D

With the recommendations of Project 1-34C, the methodology of Project 1-34D would continue to collect structural data such as cracking, faulting, rutting and IRI values in addition to the collection of deflection data, the measurement of flow rates through the drainage systems and the use of video equipment to survey the condition of drainage pipes below the ground surface. Flow rates through the sections were tested by coring and removing a hole in the pavement surface, then running water

through the hole. The outlets were then monitored to measure flow through them. The sections analyzed in this project include not only the SPS-2 sites but also related data from the Minnesota Road Research Project and the Wisconsin Department of Transportation. This allowed for analysis to be done for the sections of 10 years or more.

Highlights from the findings of this project are found below:

- Edge drains may never function fully or at all due to the nature of some subgrade soils. Water may be more conducive to flowing downwards through the soil as opposed to laterally through the edge drains
- Deflection data suggests that deformation in a section is related to the stiffness of the base material rather than the quality of drainage.
- Load transfer values for undrained sections are no worse than drained, permeable base sections
- IRI values, initial and final, are predominantly due to base stiffness
- In terms of faulting, sections with undrained lean concrete bases or permeable asphalt-treated base are slightly better than sections with dense graded aggregate bases
- In terms of cracking, lean concrete bases (LCB) performed the worst, followed by dense aggregate bases and then permeable asphalt treated bases; more than 60% of the LCB sections had some cracking while only 30% of the aggregate-base and PATB sections had only nominal cracking
- Edge drain pipes were sometimes crushed during construction
- Outlets that received little maintenance would become overgrown and lose functionality

Overall, the research team concluded that performance of the test sections was related more with the stiffness of the base material rather than the drainage capabilities of the base. This however should not

deter agencies from considering subgrade drainages systems. Project 1-34D makes these final recommendations to agencies:

- Consider climate and soil taxonomy as to identify sites that are at risk for excessive moisture and poor natural drainage
- Review visual distress surveys from local roads for signs of poor drainage (pumping, potholes, etc.)

Due to the conclusion that stiffness of the base layers contribute more to structural performance than drainage, it is recommended that designers consider a stiffer base layer, although a base layer that is too stiff such as LCB showed increased cracking in a significant number of sections investigated.

2.7 Conclusions

The majority of the reports reviewed suggested that roadways with subgrade drainage systems tend to perform closely with that of their undrained counterparts in terms of structural integrity. Furthermore, with subgrade drainage systems proper construction procedures and periodic maintenance of drainage outlets must be taken into consideration to ensure the effectiveness of these systems. Areas with high annual precipitation or soils with low permeability appear to be good candidates for these drainage features.

3.0 Examine different curing methods for rigid pavement construction and their impact on the early age curling and warping of continuous reinforced concrete pavements

Currently, the MEPDG requires that the user input the curing methodology to be used in the construction of a pavement. From analysis completed in this report it can be shown that this input variable is significant in determining the design thickness of CRCP. The MEPDG only allows the user two choices between curing type. They are either wet mat or spray on cure. Although the wet mat cure has

been indicated to be the most effective curing technique, it is the least economical to be implemented in the field. The goal of this task is to find if the properties of a concrete pavement receiving a wet mat cure can be provided by a method that is more economical.

After further investigation of the MEPDG it has been determined that for design purposes of CRCP that the assumed difference between a wet mat and spray cure is the amount of initial curling and warping of the pavement due to differentials in shrinkage, moisture, and zero stress temperature. This initial deformation of the pavement is detrimental as it causes the pavement to lose its initial support from the sub base and therefore increase the stresses in the pavement from subsequent external loads.

3.1 Progress on this Task

Through conversations with several Oklahoma paving contractors it has been determined that there would be significant interest in either reducing the design thickness of a pavement or extending the pavements life through the use of a curing method that is more efficient than a spray on cure. During these conversations several different possibilities were mentioned including using burlap, watering the pavement at regular intervals, or the use of a more robust or multiple application of a curing compound.

In this project the research team's aim is to determine a baseline for the curling and warping of these different curing methods and then determine their effectiveness with small laboratory paste specimens stored at 73° F and 40% relative humidity. Next these curing techniques will be investigated in specimens that are 6" x 8" x 8' in length and use concrete. This size is chosen to simulate a strip of concrete pavement. Again these specimens will be stored at 73° and 40% RH to compare the early age warping of the different curing methods. It is then planned to take this research to a larger scale on either the OSU campus or at an actual job site to construct a full scale pavement and monitor the early age curling and warping.

3.2 Paste Beam Experiments

3.2.1 Materials and mixture proportions

The Portland cement used in these tests was a Type I according to ASTM C 150. Curing techniques are achieved by using wet burlaps and plastic for wet curing techniques as well as curing compounds from W.R.Meadows. These curing compounds are summarized in Table 3.1.

Table 3.1 - Curing compound properties

Type	Properties	
2245	poly-alpha-methylstyrene-based	white-pigmented
1200	resin-based	white-pigmented
1600	wax-based	white-pigmented

The paste mixtures in this experiment had a water to cement ratio (w/cm) of 0.42. The total volume of paste that was prepared for each experiment was 142.2 in³.

3.2.2 Sample preparation and methods

The paste mixtures were prepared according to ASTM C 350. Three paste beams with dimensions of 39.4"x2.4"x0.5" were made from each mixture. A summary of these specimens are shown in Fig. 3.2.

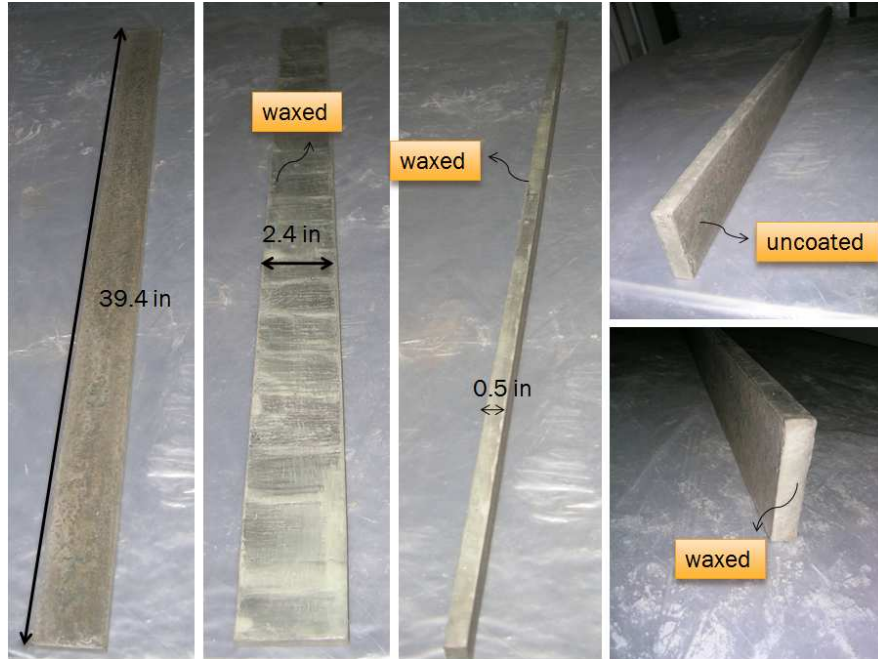


Figure 3.2 - A waxed paste beam on all side except the finished surface

These specimens are all stored in an environmental chamber at 73° F and 40% relative humidity. Figure 3.3 shows the specimens with different curing methods placed on their sides in the chamber room.



Figure 3.3 - Specimens inside the black plastic bag, Samples of curing compound method,

Wet cured samples inside the plastic are all stored on the side in the chamber room.

All specimens were cured using wet burlap for 24 hours before demolding. Next they were demolded, weighed, sealed, reweighed and stored on their sides. Control, wet cured, and curing compound

samples are all sealed with wax in all sides except the top side which will be exposed to curing techniques; alternative control samples are sealed using aluminum tape to verify the performance of the wax.

Next, the performances of different curing techniques were investigated. Some samples were kept in wet burlap for 1, 3, 7, and 14 days of additional curing after demolding and waxing. The curing was then removed and the specimens were subjected to the drying environment. In addition to wet curing several specimens were cured in a sealed plastic bag for 1 and 3 days after waxing all sides except the top. This curing was similar to the wet burlap cure, but no external moisture was added to the specimens during the curing process in the bag.

The effectiveness of curing compounds was also investigated. For this testing these curing compounds were not applied until 5 hours after casting. Three different applications were applied to the specimens. In each case the manufacturers recommended dosage was used 163 ft²/gal (4m²/L) and one about 50% lower and 150% higher was used. The curing compound has been applied in a single layer with a Chapin 5797 flat nozzle with pump pressure of 40 psi as suggested in the manufacturer's literature. To modify the application of the compounds three different nozzle distances were used with the same cart velocity. For this purpose a cart has been designed to move the nozzle in the long dimension of the beam easily to maintain a uniform coverage as much as possible.

The coverage rate on samples cured by the curing compound can be determined by the following equation.1:

$$\text{Eq.1} \quad v = \frac{\text{coeff.} \times F}{C \times w}$$

Where:

v = cart speed (ft. /sec),

coeff. = 0.13636,

F = flow rate (gal/min) per nozzle,

C = desired coverage (gal/ft²), and

w = nozzle spacing (in.).

The following picture in Fig. 3.4 shows the schematic view of the spray coverage calculated by Eq.1. This equation is commonly suggested by the curing compound manufacturers to determine the correct coverage rate for an experiment.

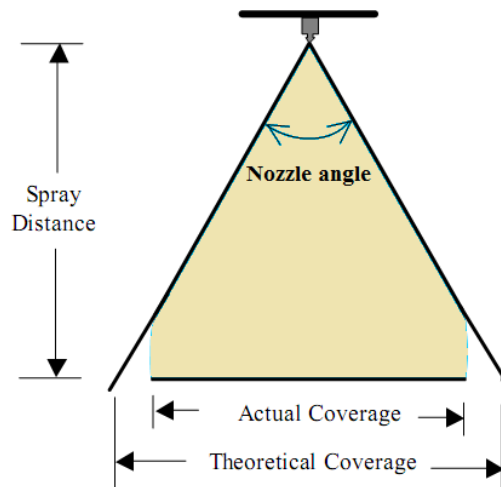


Figure 3.4 - Spray coverage

The flow rate, cart speed, desired coverage, and spray distance used in this study to spray the curing compounds computed by the equation. A summary of the variables during spraying are presented in Table 3.2.

Table 3.2 - 50%, 100%, 150% application rates calculated for a constant cart speed

	Application rates (%)		
	50	100	150
F (gal/min)	0.5	0.5	0.5
C (gal/ft ²)	0.003067	0.006135	0.009202
Length (in)	40	40	40
Overlap (%)	30	30	30
Spray Distance (in)	20.85	10.40	6.95
Width (in)	24.48	12.21	8.16
V (ft. /sec)	1.33	1.33	1.33
Coverage time (sec)	2.5	2.5	2.5

To check the uniformity of the coverage some practice tests have been done; the application coverage at different locations have been investigated by using some plates with given areas. Before and after spraying, plates should be weighed; using the area of the plate and the density of the curing compound the thickness of the layer will be calculated. A summary of the amount of curing compound applied and the variance for each test is shown in Fig. 3.5.

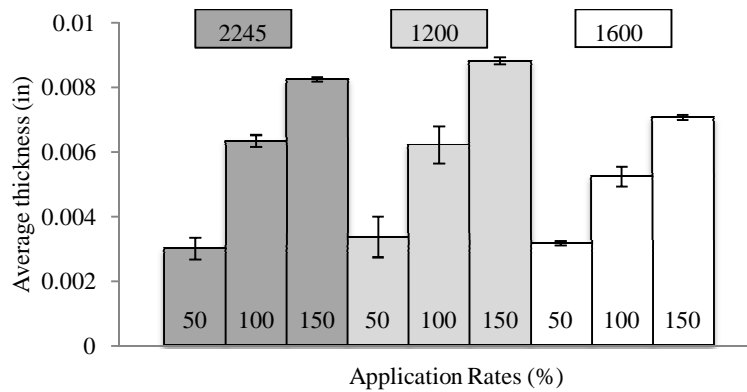


Figure 3.5 - Thicknesses of the curing compounds on the surface of the paste bars

After the curing compound has been applied at 5 hours after mixing has begun the samples are placed in the chamber for curing. Twenty four hours after mixing the specimens are then removed from the chamber and demolded. Next all of the specimens' sides were covered with wax except for the side with the curing compound. The specimen is then stored in the environmental chamber on its side.

3.2.3 Test procedure and measurement

In this experiment the finished surface is not sealed with a wax and water is allowed to evaporate. This differential loss of moisture in the specimen will cause a moisture and shrinkage gradient in the specimen. This gradient causes differential strain to occur and therefore a curling of the specimen. This test is advantageous as the moisture loss is quick and the gradients can be quite large. This leads to a significant deformation of the specimens that is easy to measure. To insure that the curling measurements are only the result of this differential in strain caused by the drying the beams are stored on their sides.

To measure the curling two ends of the specimen are attached to a flat aluminum plate with the uncoated surface facing the plate. The distance between the aluminum plate and the specimen is

measured at regular locations along the length with a caliper that is accurate to 0.0005" as shown in Fig. 3.6. Some typical data is shown in Fig. 3.9. The curling of the beam is symmetric and is maximum at the midpoint. This maximum deflection at different time intervals is used to determine the ability to minimize curling by different curing methods. The weight of the sample is also measured with time. This measurement provides information about the moisture content of the specimen.



Figure 3.6 - Measuring the curling height using a caliper

3.2.4 Results

The results from the comparison between the coating materials wax and aluminum tape are shown in Fig. 3.8.

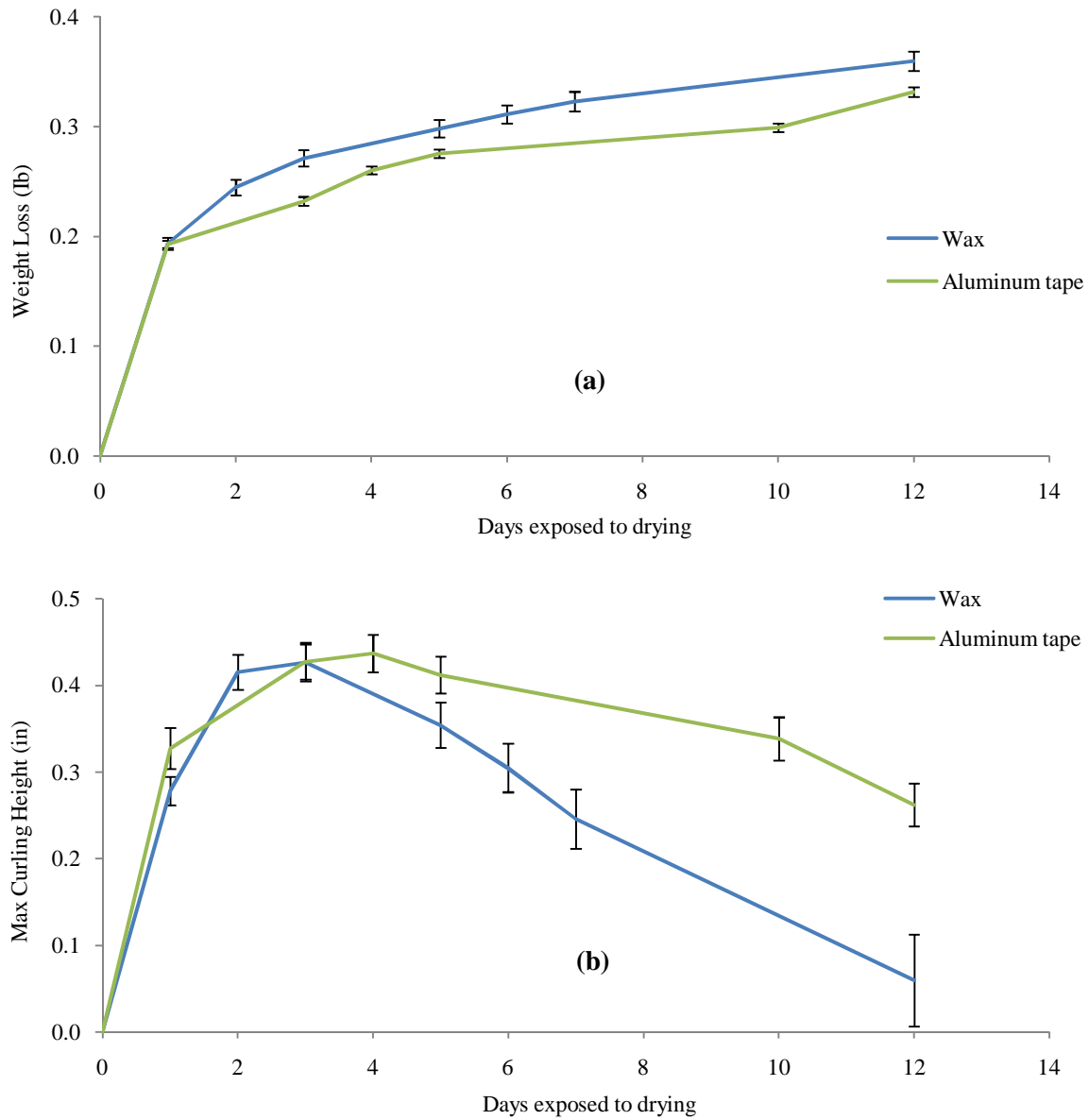


Figure 3.8 - Comparison between coatings' performance in retaining moisture content during the early age (a) Moisture loss vs. early age; (b) curling height at the middle over time

Figure 3.9 shows the typical curves gained by measuring the deflection of the beam at different locations over time. The maximum curling caused by drying shrinkage is at the middle of the bar.

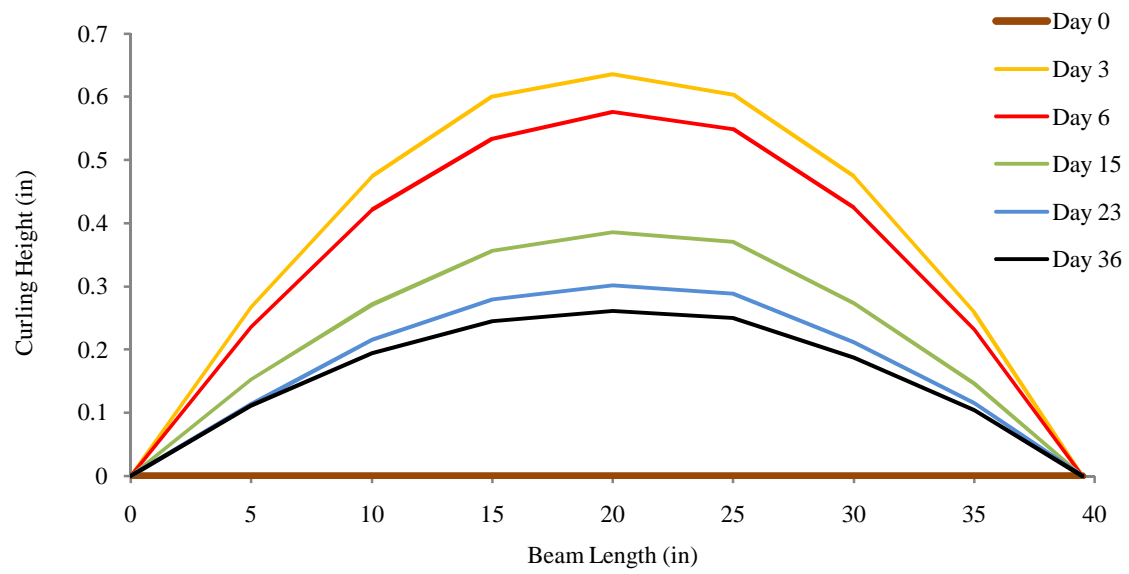
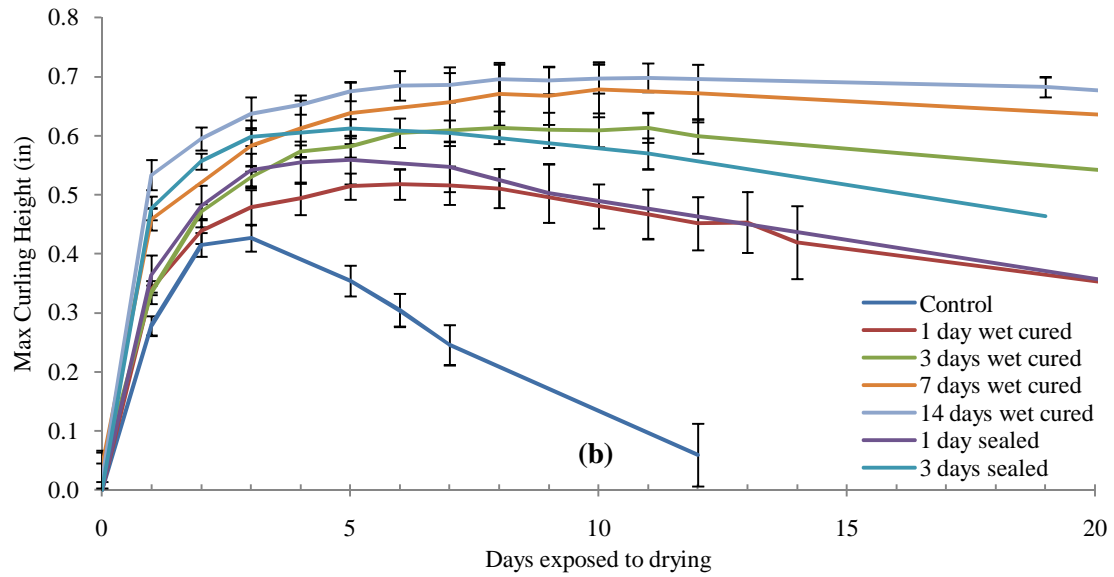
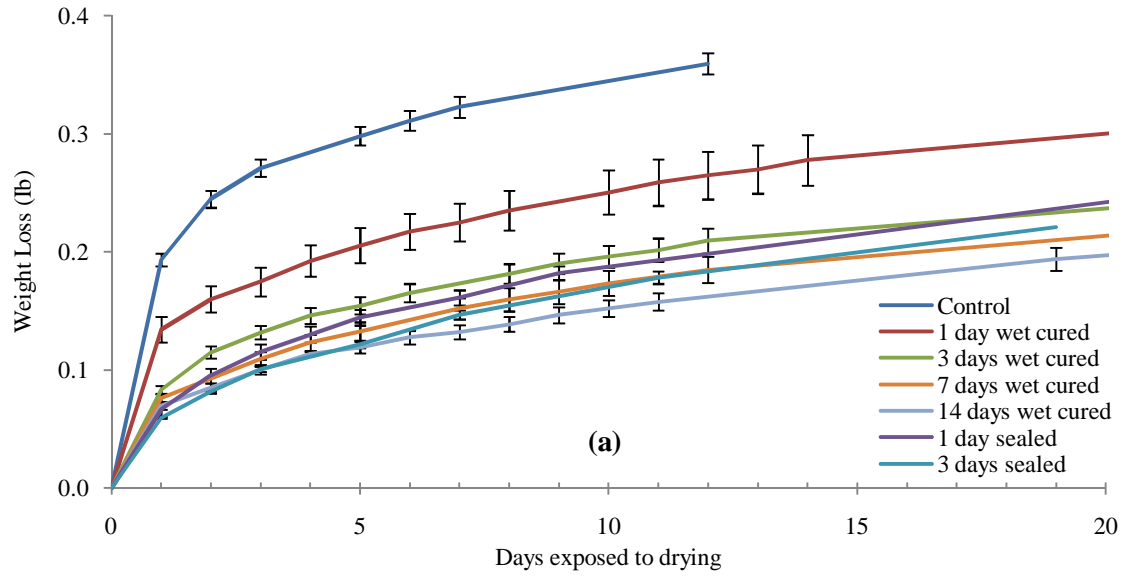


Figure 3.9 - Curling height of a beam mixed with $w/cm=0.5$

The moisture loss and the max curling height of samples cured with wet burlap for 1 and 3 days, sealed with the plastic bag for 1 and 3 days, and the control specimen are shown in Fig. 3.10.



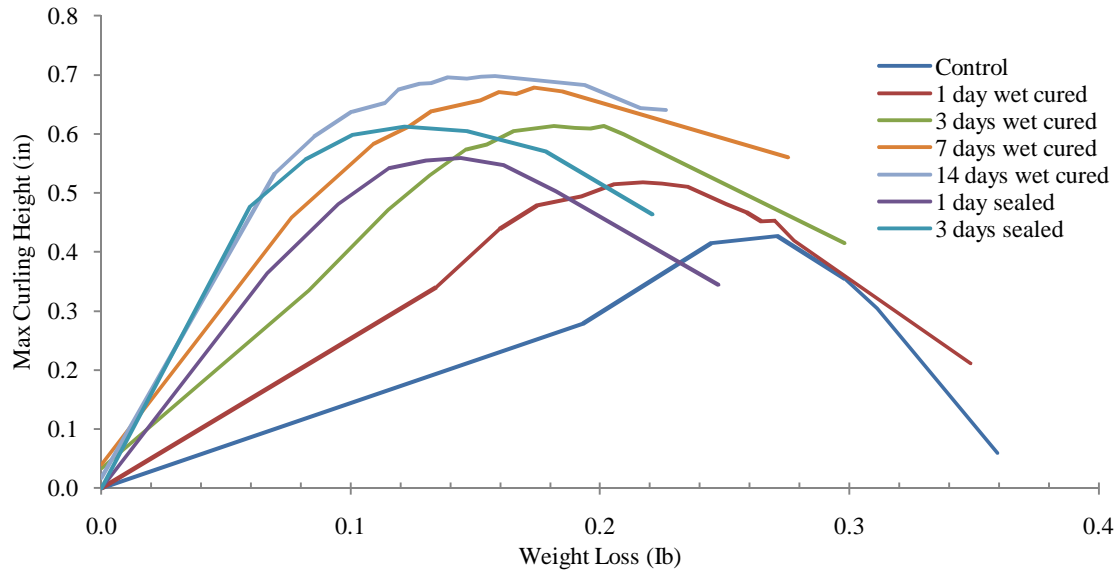


Figure 3.10 - Weight loss over the age of the specimen (a), maximum curling height over the time (b), and Maximum curling of the specimen in comparison with its weight loss (c)

The following graphs in the Fig. 3.11 are showing the effect of W.R. Meadows curing compound 2245 in different application rates on the moisture content and curling of the paste beams:

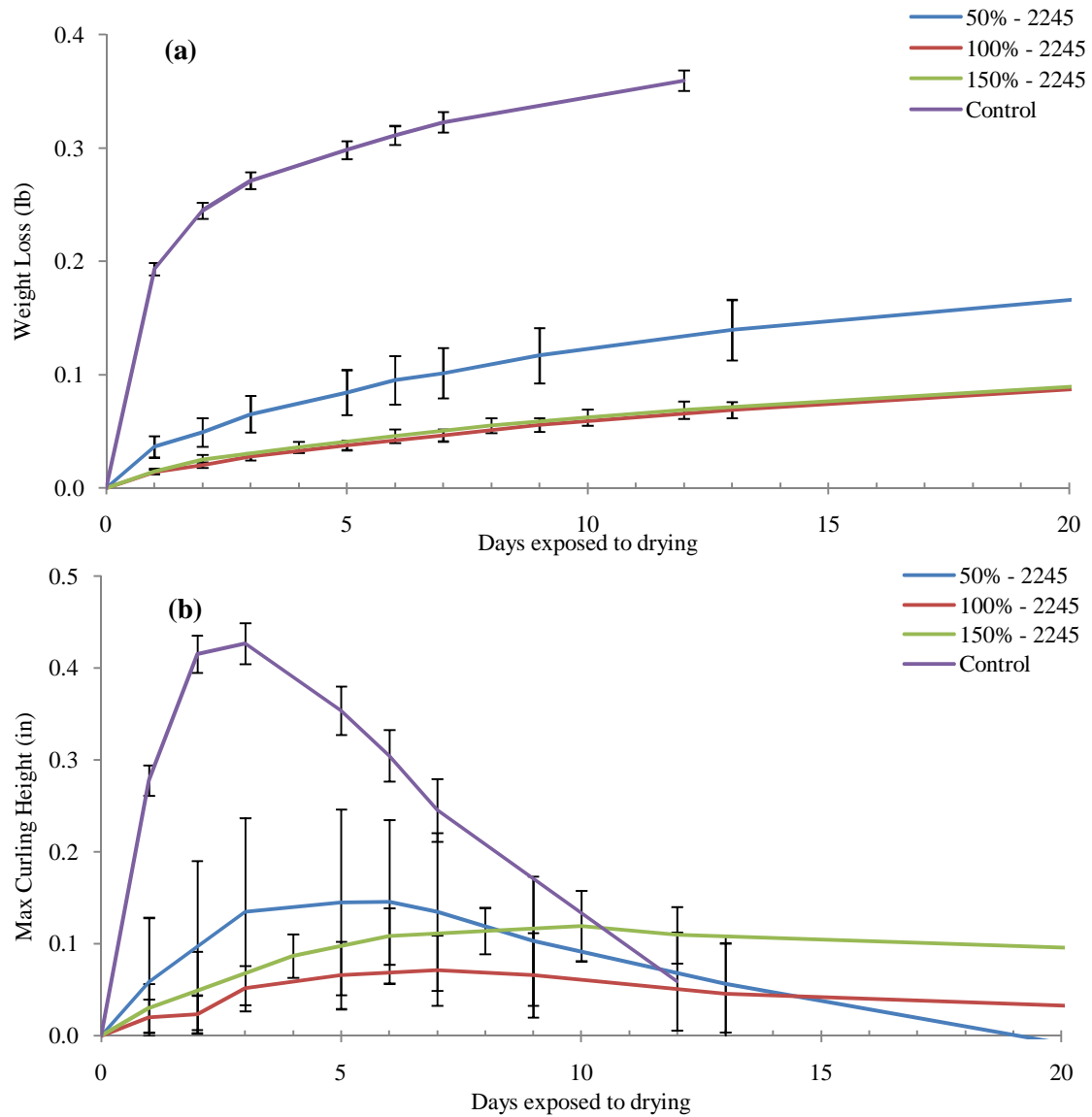


Figure 3.11 - (a) Weight loss of the samples cured with 2245 over the early ages and (b) their max curling over this time

Figure 3.12 shows the results of 50%, 100%, and 150% application rates of 1200 pigmented:

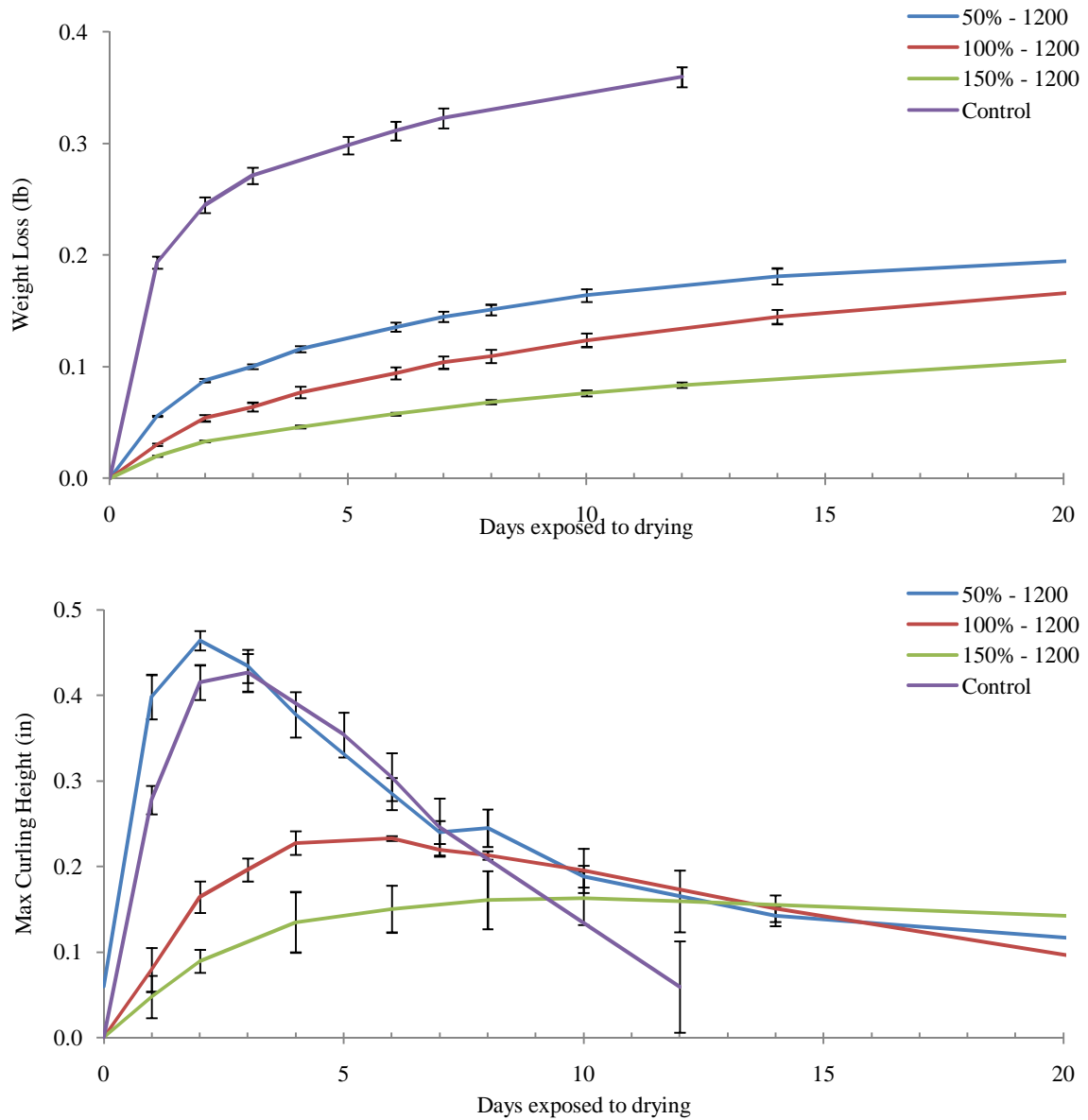


Figure 3.12 - (a) Weight loss of the specimens cured with 1200 over the early ages and (b) their max curling over this time

W.R. Meadows curing compound 1600 results are shown in the graphs presented in the fig. 3.13:

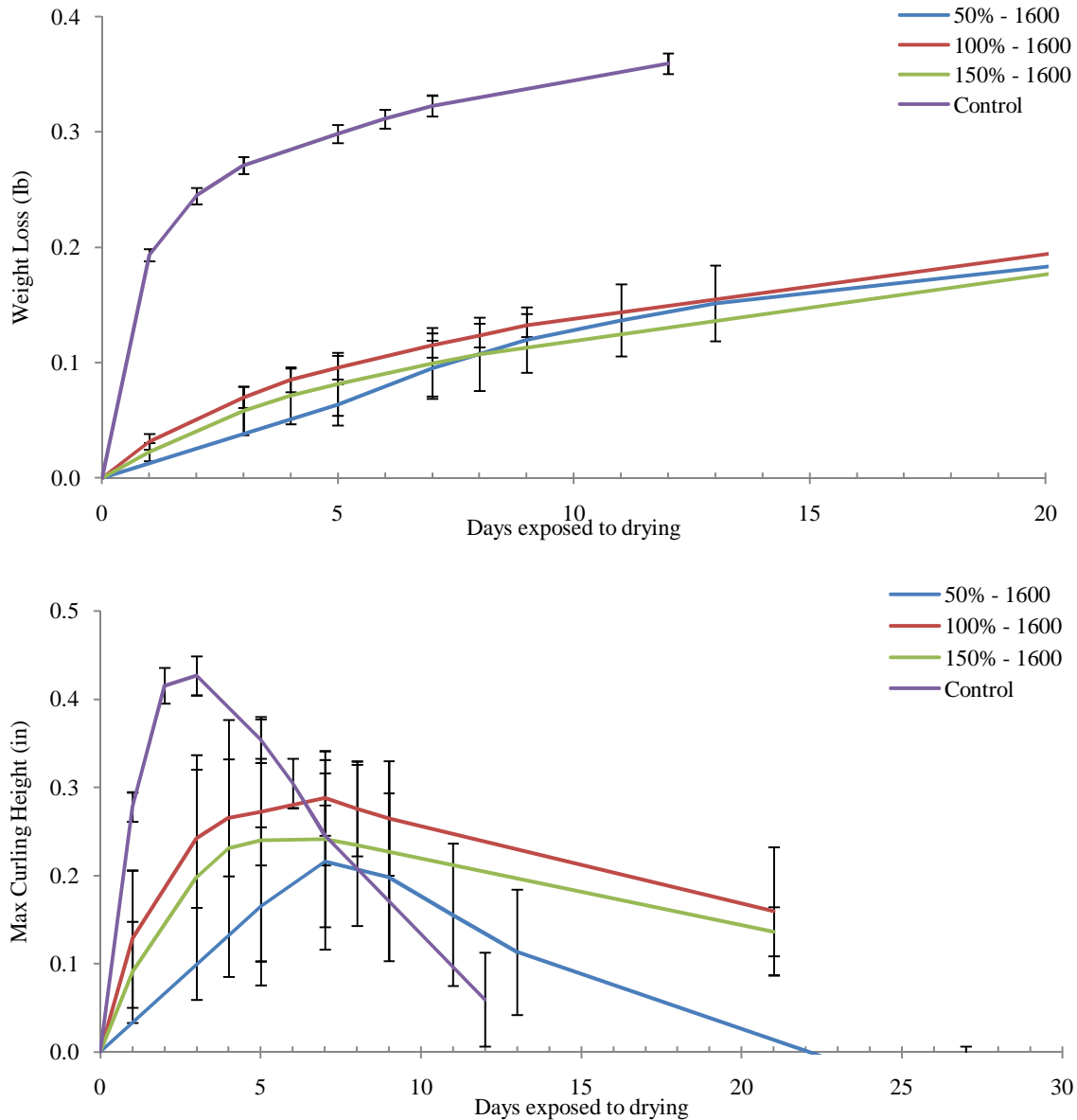


Figure 3.13 - (a) Weight loss of the specimens cured with 1600 over the early ages and (b) their max curling over this time

3.2.5 Discussions

Although the work is not yet complete the following observations can be drawn from the data. Figure 3.8 shows that there is no considerable difference between the aluminum tape and wax performance; the test was done due to a concern about the wax consistency and capability in retaining moisture

content, and finding an easier and more economical way to seal the sample. Therefore the wax was selected to seal the samples.

When comparing the wet curing technique and plastic bag method in Fig. 3.10 the weight loss of the control specimen is larger and the wet cured samples for 14 days and the sealed specimens for 3 days have the least moisture loss. Despite the moisture loss being similar between the 3 day sealed cure and the 14 day wet cure the measured curling was not as the 14 day wet cured showed the most curling and the 3 day sealed cure showed the least. This suggests that by adding the external moisture with the wet cure method that once this water is lost a greater internal moisture gradient is formed and therefore there is an increase in the observed amount of curling.

The following can be said for the curing compounds. For all of the coatings as the coverage increased so did the performance of the compound. The 1600 curing compound had the worst performance of all of the coatings. This is likely due to imperfections in the surface of the compound. More work is being done to compare the performance of the coatings.

3.3 Concrete Beam Experiments

3.3.1 Materials

The cement used in this test is a Type I according to ASTM C 150. The mixtures also used local dolomitic limestone and a river sand. The fly ash used for the testing is a ASTM C 618 class C ash. The air entraining agent is a wood rosin. At the sides of the concrete beams a synthetic moisture barrier was used as a form liner. This allowed a moisture barrier at all sides of the specimens except at the finished surface.

3.3.2 Mixture Proportions and Methods

In this experiment a w/cm of 0.41 was used. A concrete mixture was used that was typical of Oklahoma concrete pavement mixtures. The mixture procedure is as below:

- Mixing sand and rock and half or less water for 3 min.
- Add cement/fly ash and rest of the water and mix for 3 min.
- Scrape for 2 min.
- Mix another 3 min and add AEA.
- Measure the slump (ASTM C 143), unit weight and the air content (ASTM C 138).

3.3.3 Sample preparation, Casting and Curing

In this experiment the concrete beams 7.5'×6"×8" with all sides sealed with a type of synthetic moisture barrier except the top surface for each specimen are used, Fig 3.14. These specimens are all stored on their side in an environmental chamber at 73° F temperature and 40% relative humidity. After placing and vibrating the concrete in 3 different layers the top surface was finished flat. The top of the beam was then finished with a burlap drag and finally tined using a comb equipped with steel to make grooves 1/16" wide, 1/4" deep, and with a center to center spacing of 1" between the tines. The sample will be demolded very carefully after 5 hours. Then it should be sealed at the edges by wax or glue.

The samples are then cured different ways. One curing method used wet burlap and then covered with a plastic tarp. With another method no curing was used. In future work the research team plans to look at a number of other curing techniques.

After the curing is completed the specimen is stored on its side on top of wood dowels. After the beam is turned it was fixed at one edge with a C-clamp, steel plates, and bearing pads.



Figure 3.14 - A concrete beam specimen that is being stored on its side.

3.3.4 Test Procedure and Measurement

The relative humidity will be measured at the middle of the beam at 4 different distances 0.5", 1", 3", and 5" from the finished surface using the DS1923 Hygrochron Temperature/Humidity Logger iButton sensors. The sensors should be put in 4 different holes that are cast into the side of the beam as shown in Fig 3.15. The holes are closed after inserting the sensors and the data is measured once a month. A typical set of the data is shown in Fig. 3.16.



Figure 3.15 - Covered holes by tape, deflection gauge at mid-length held by a magnet, and moisture barrier overlaps

The deflected shape of the beam is measured at 3 different locations 45", 67.5", and 89" from the supported plate. Also the surface strain of the beam was measured with surface mounted demac points. These points are glued to the surface of the beam after it is placed on its side and are able to measure strain. By both of these methods measure the same parameters only in different ways. This will be a useful double check on the performance of the curling measurements.

3.3.5 Results

A typical set of data is shown in Fig. 3.17, 3.18, and 3.19 for a beam. The plot in Fig. 3.17 shows the change in relative humidity within the depth of the beam over time. Figure 3.18 shows the curling height or the deflected shape of the beam over time. The data is shown two different ways. Figure 3.18a shows the deflected shape of the beam and Fig. 3.18b shows a plot of the individual measurements of the LVDT over time. Figure 3.19 shows the surface strain of the beam that is measured over time. The same plots are shown in Figs. 3.20 and 3.21 for a beam that was stored on its side. A comparison of the maximum deflections for beams that are stored with the finished surface on

the side and one with the finished surface facing up. This plot is shown in Fig. 3.22.

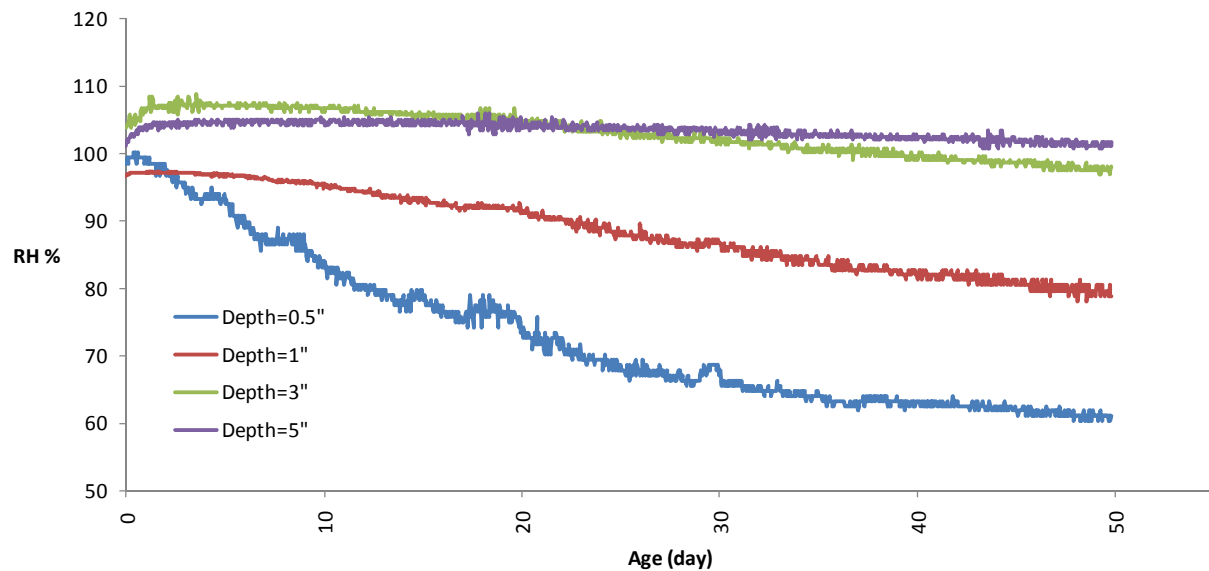


Figure 3.16 – A plot of the relative humidity change inside of the beam with time.

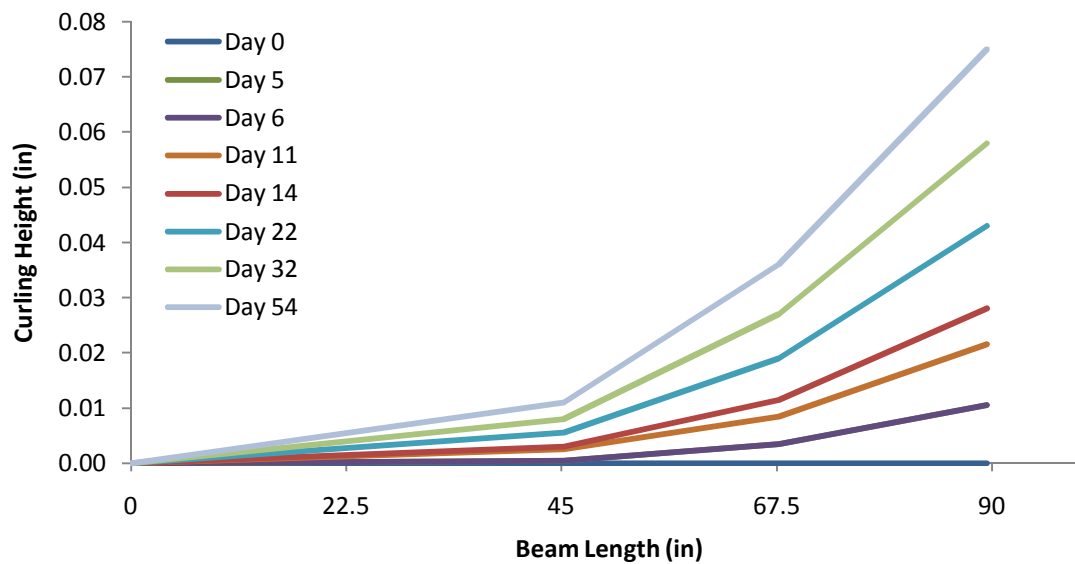


Figure 3.17a - Beam curling over the entire length for a beam stored on its side.

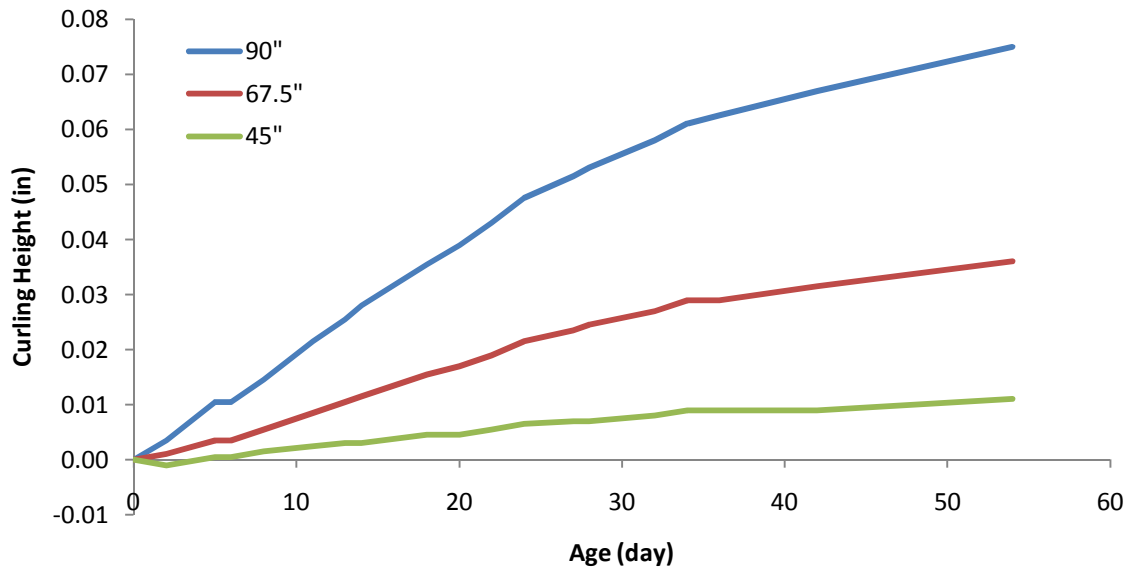


Figure 3.17b - Beam curling at different locations during the time for a beam stored on its side.

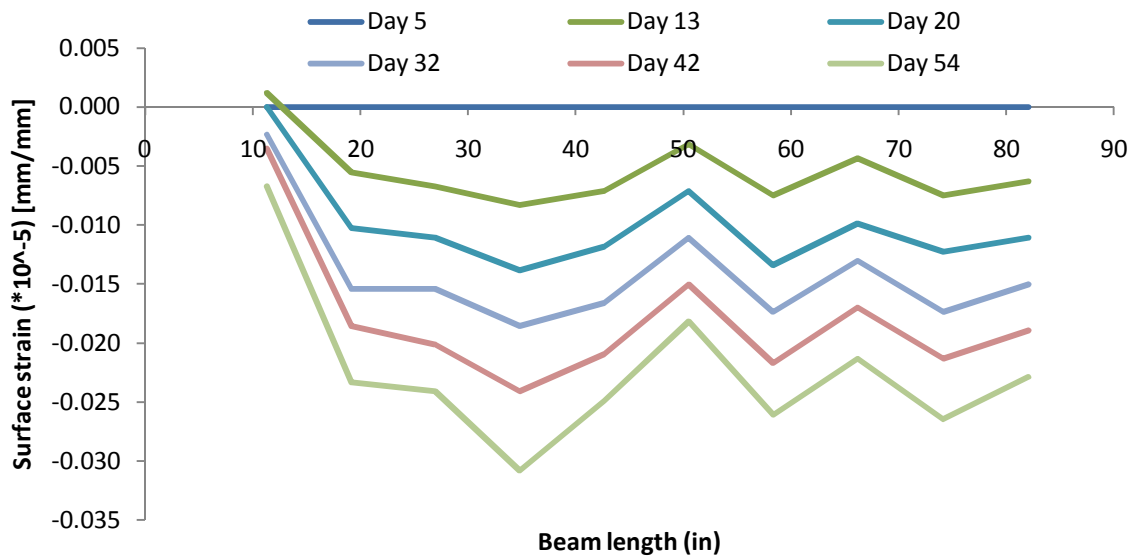


Figure 3.18 - The surface strain of the beam over time for a beam stored on its side.

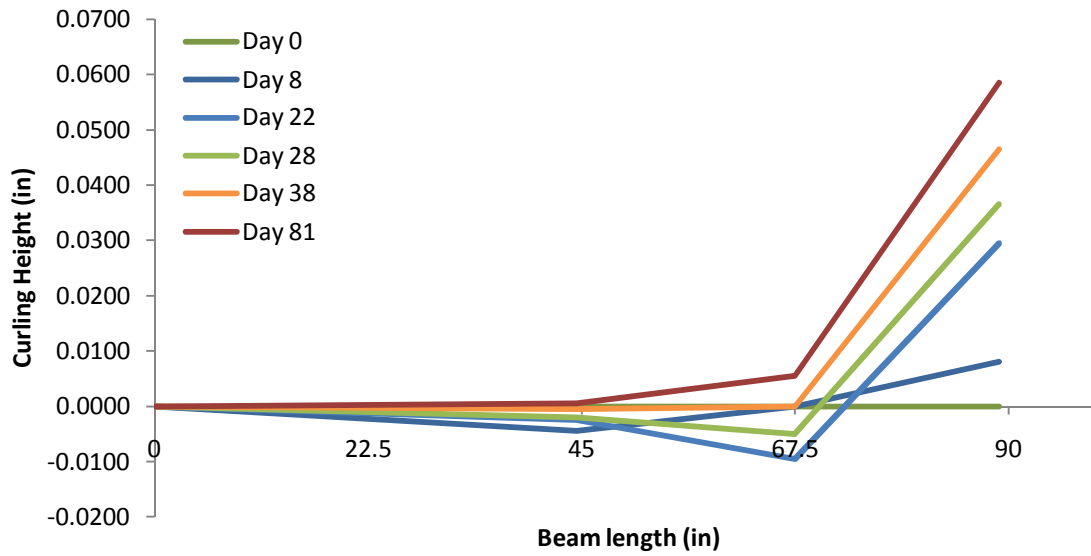


Figure 3.19a - Beam curling over the entire length after the curing process for a beam stored with its finished surface facing up.

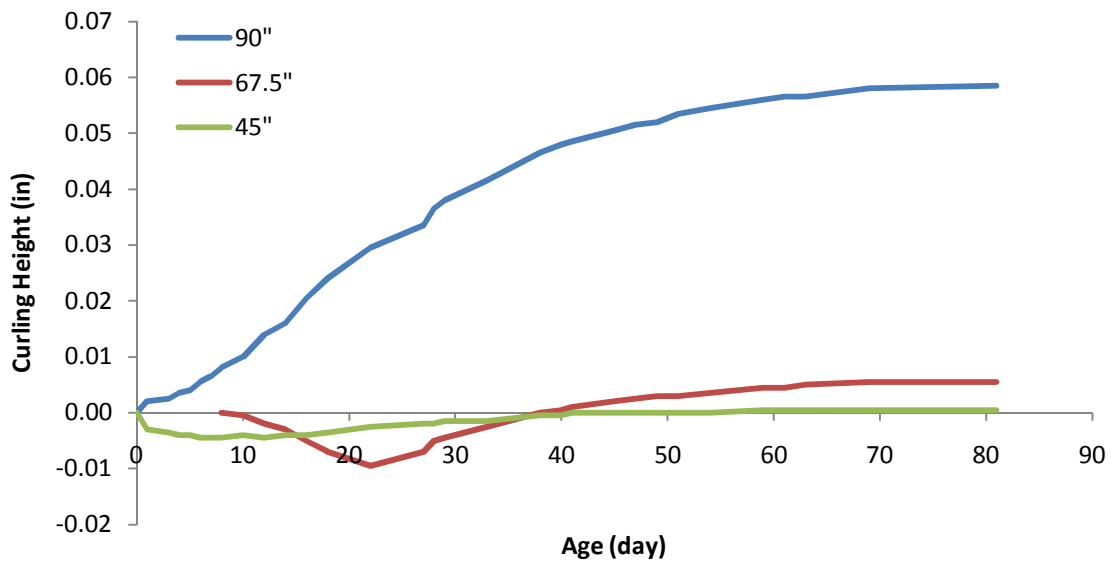


Figure 3.19b - Beam curling at different locations during the time for a beam with its finished surface facing up.

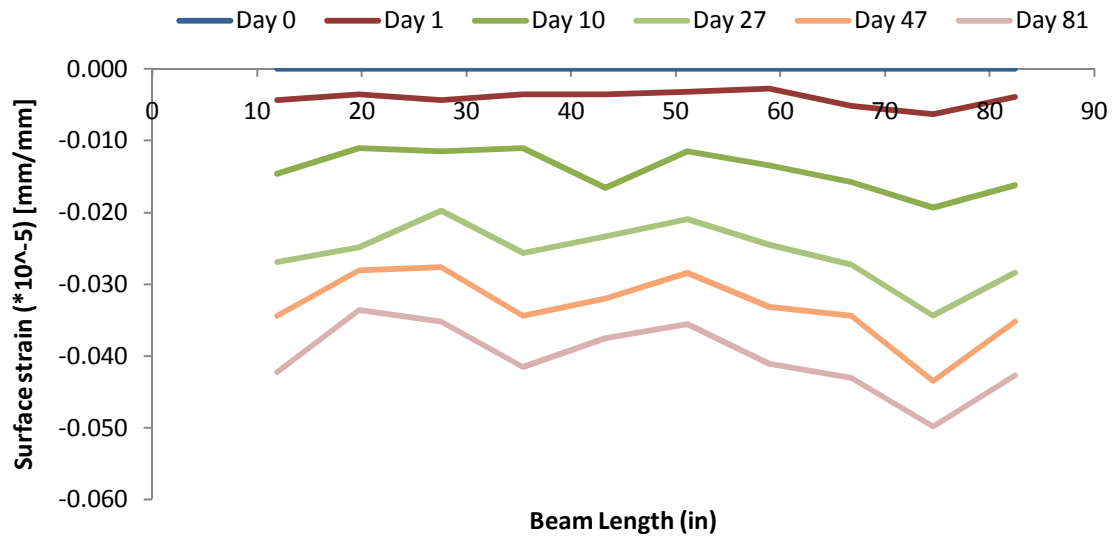


Figure 3.20 – The surface strain of the beam over its length for a beam with its finished surface facing up.

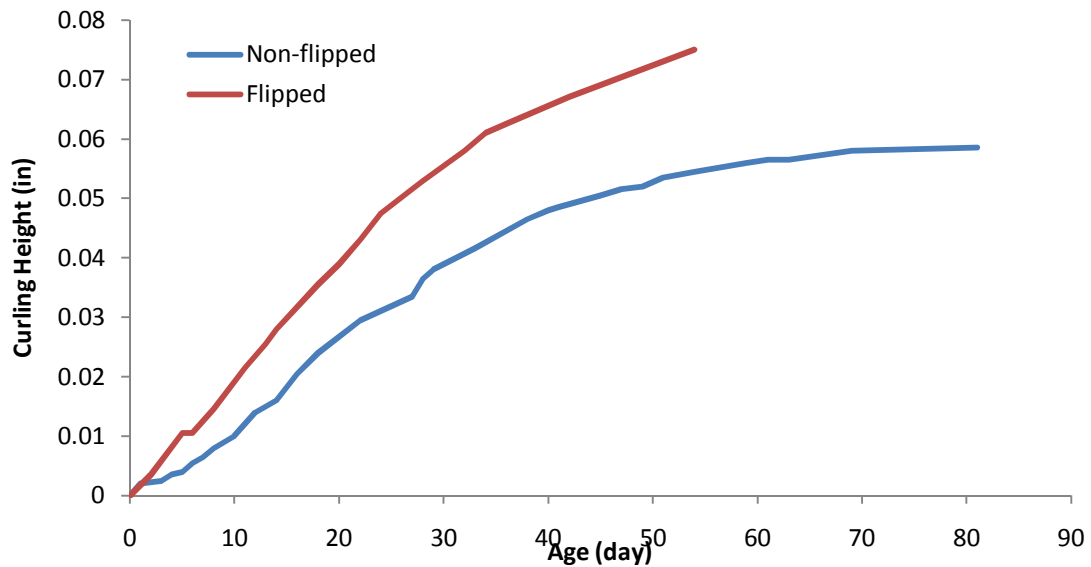


Figure 3.21 – The maximum curling of the flipped and non-flipped beams during the time

A comparison was also made for concrete beams that have been wet cured for 3 days and one that did not cure.

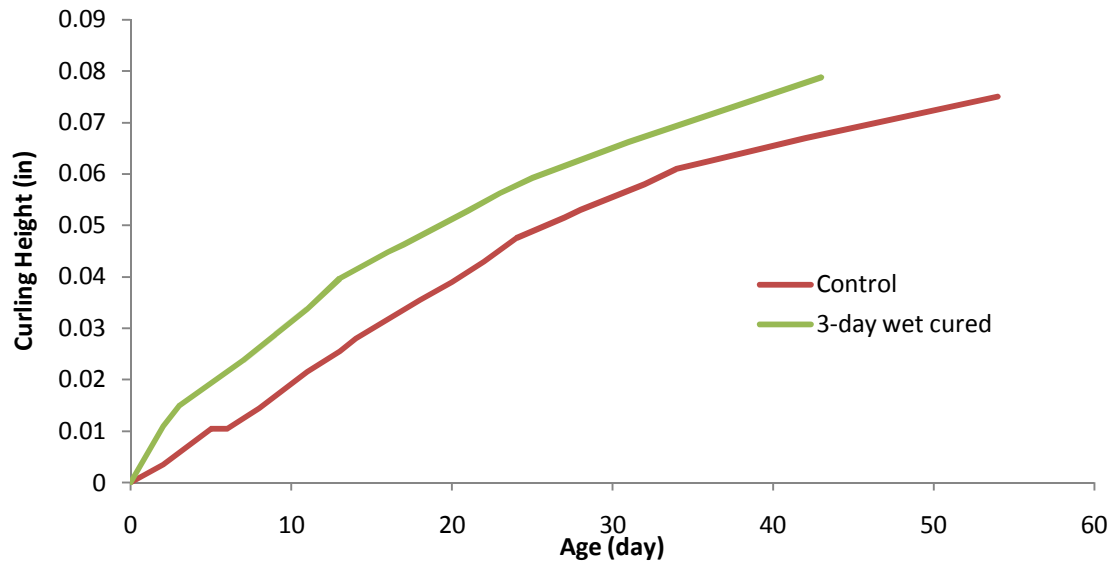


Figure 3.22 – The maximum curling of the control and 3-day-wet-cured beams for beams stored on their side.

3.3.6 Discussion

The research team has decided to only do testing with specimens on their side as the self weight of the beam makes it difficult to analyze. The reason it is so difficult to analyze is that as the beam tries to curl upwards the self weight pulls the beam downwards and brings it back in contact with the support below. Since the research team is trying to quantify and compare the curling of the beams it will make the testing easier to just evaluate beams on their sides. The team feels that once it is understood how the beams curl then calculations can be used to model how gravity loads will impact it.

It can be seen in Fig. 3.22 that a beam that was wet cured for 3 days showed more curling than a beam that was not cured. The research team is still working towards understanding why this is occurring.

However, similar results were obtained from the paste beams and so this phenomenon seems to be repeatable between the two test specimens.

3.4 Future Work

Significant work was accomplished during this period to establish the methods to be used to investigate different curing practices and their impact on curling. In order to better understand this phenomenon more work is being done to quantify why this is occurring. In addition work is also being completed to start using curing compounds to investigate their performance in the concrete beam tests. Also combinations of the use of curing compounds and wet curing will be investigated. In addition investigations will be made for the reason why different curing compounds are performing different for the same coverage rate. The research team feels that this is at least partially dependant on the imperfections that are present in the curing compound when it is applied. New methods of coverage will be investigated that could provide insight into the performance of different curing compounds. The research team feels that this will make a significant impact in the effectiveness in curing for future Oklahoma concrete pavements.

4.0 Provide regional material input parameters that can be used in the MEPDG for the design of rigid pavements

The MEPDG user manuals have suggested that more accurate pavement designs can be determined if accurate material input values can be obtained for local materials. Through the sensitivity analysis contained in this report and through discussions with ODOT it has been determined that the following parameters should be further investigated for Oklahoma paving concrete mixtures:

1. Concrete Shrinkage
2. Coefficient of thermal expansion (CTE)
3. Strength testing

4.1 Progress on this Task

4.1.1 Concrete Shrinkage

There are a significant number of concrete mixture shrinkage parameters that must be characterized for the MEPDG. These include: maximum shrinkage, reversible shrinkage, and time to develop 50% of the maximum shrinkage. These parameters will be investigated with the AASHTO T160 “Standard Method of Test for Length Change of Hardened Hydraulic Cement Mortar and Concrete” with the only modification being that a relative humidity of 40% is used instead of 50%. This change was made as the MEPDG design manual suggests that by using 40% instead of 50% that a more accurate estimate of the maximum shrinkage will be obtained for the concrete specimen with less testing time (ARA 2004).

It is well documented that the shrinkage of a concrete mixture depends largely on the percentage of the mixture that is paste, the water content of the mixture, and the types of cementitious materials in the mixture. A standard paving mixture for the state of Oklahoma has been determined through discussions

with ODOT and review of typical Oklahoma concrete paving mixtures. These mixtures consist of 0.42 water to cementitious ratio (w/cm) with total cementitious content of 564 lbs and a 20% replacement with fly ash. The aggregate is proportioned to have approximately 60% coarse aggregate and 40% fine aggregate. Approximately five percent air is entrained in each mixture. This mixture produces a paste content of 25% in the mixture. This mixture will be systematically altered by substituting different mixture components including four different cements, four different fly ashes, and three different paste contents to examine their impact on the shrinkage parameters measured by the MEPDG.

These tests were started during phase I of the project but a failure in the environmental chamber has required these mixtures to be recast and reinvestigated. Because of the delay additional time may be required at the end of the project to complete the testing.

4.1.2 CTE Testing

In the sensitivity analysis completed for this research project it was determined that the design of CRCP is very sensitive to the CTE value of the concrete. In order to get a better understanding of these values for typical Oklahoma concrete paving mixtures the research team plans on evaluating how different components of a mixture impact the CTE. This will be done by measuring the CTE of a standard concrete mixture to find a baseline for the mixture for the raw materials. Next different parameters of the mixture are proposed to be varied and the resulting impact on the CTE to be measured. By combining the influence from the different aggregate sources and the impact of the paste on the CTE then one should be able to create a tool that is able to estimate the CTE for a mixture based on the mixture ingredients and proportions for typical ODOT mixtures. It is anticipated that the following variables will be investigated: cement content, cement replacement with supplementary cementitious materials (SCMs), water to cement ratio, and aggregate gradation.

From discussions with ODOT the following seven coarse aggregate and 3 fine aggregate pits will be investigated in this project. These pits were chosen as they cover a significant geographic area and mineralogy of the state. A summary of the pits is provided in Table 7.

Table 4-1 - Summary of the aggregates to be investigated for this project.

	Owner	Town	Rock Type	Absorption (%)	SG (SSD)	Unit Weight (lb/ft ³)
Coarse Ag.	Dolese	Davis	Limestone	0.89	2.67	168.1
	Hanson	Davis	Rhyolite	0.64	2.71	165.6
	Dolese	Coleman	Dolomitic Limestone	0.55	2.77	173.1
	Martin Marietta	Sawyer	Sandstone	2.02	2.52	156.1
	Dolese	Hartshorne	Limestone	1.13	2.62	169.2
	Pryor Stone	Pryor	Sandy Limestone	1.61	2.63	162
	Quapaw	Drumwright	Dolomitic Limestone	0.92	2.81	147
Fine Ag.	Dolese	Dover	River Sand		2.6	110
	Southwester State Sand	Snyder	Manufactured Sand		2.7	108

* Aggregate properties from ODOT Materials Division

All of the concrete mixtures for this testing has been completed and the specimens are being prepared for CTE testing.

During this quarter two CTE frames were constructed, along with a water bath and a closed loop measuring system that should be able to automatically run the tests. Calibrations of the system are being completed and testing of these specimens should begin soon.

4.1.3 Strength Testing

The strength of a mixture depends on the cementing materials in the mixture, the water to cement ratio, and the bond between the aggregates and the paste. To evaluate these parameters 4" x 8" compression cylinders (AASHTO T 22 "Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimens") and flexural beams (AASHTO T 97 "Standard Method of Test for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)") will be prepared for a number of the

mixtures prepared for the CTE and shrinkage testing and tested for strength at 7, 14, 28, and 90 days of hydration. This testing will not only provide ODOT with a range of results for the common mixtures used in the state but will also provide a correlation between the two measurements. Based on the results it was found that the relationship between the compressive and flexural strength of the specimens closely followed equation 1.

Equation 1
$$\sigma_{flexural} = 9.5\sqrt{f'c}$$

The relation is shown in Fig. 4.3 and is a similar value as to what is suggested in the MEPDG design manual.

Table 4.2 – Mixture designs

Mixture #	Mixture Description	Slump (in)	Unit wt (lb)	Air content (%)
1	5.5 sacks Lafarge	0.75	147.4	5.5
2	5.5 sacks Lafarge 20% red rock	1	146.0	6.1
3	5.5 sacks Lafarge 20% Muskogee	1.5	151.4	6.7
4	5.5 sacks Holcim 20% red rock	1	154.2	5.3
5	5.5 sacks Buzzi 20% red rock	1.25	152.6	6.25
6	5.5 sacks Lafarge 20% GRDA	1.25	152.5	6

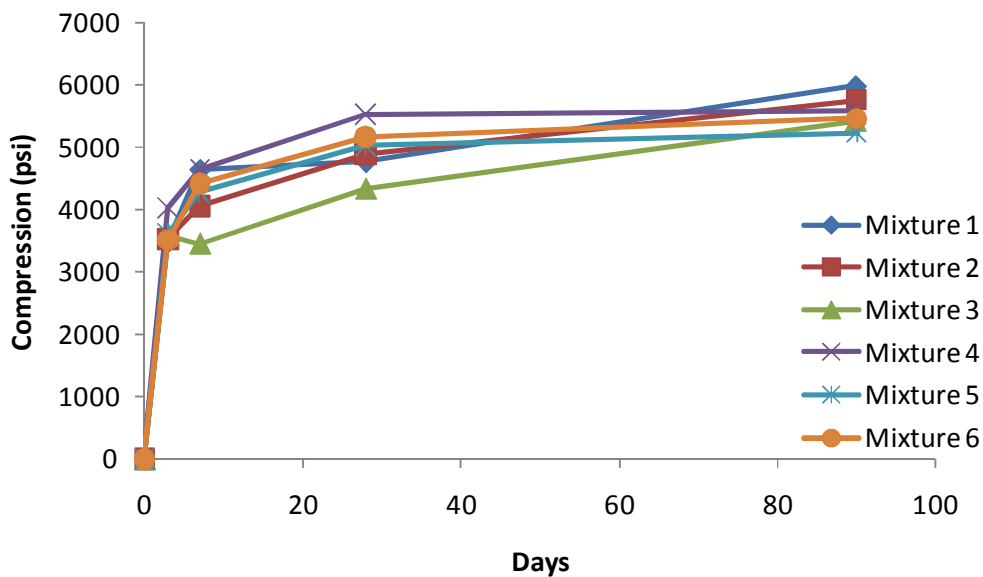


Figure 4.1 – Compressive strength versus time for the different mixtures.

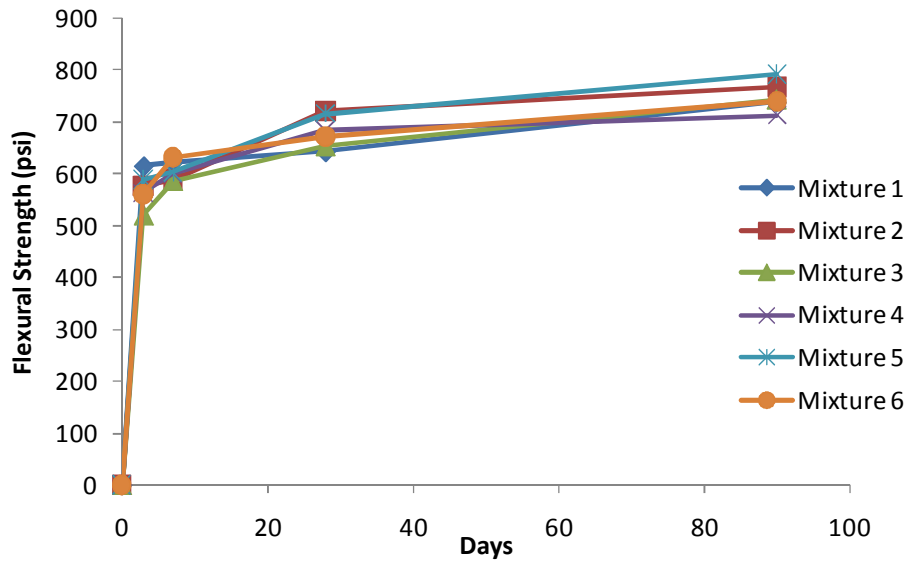


Figure 4.2 – Flexural strength versus time for the different mixtures.

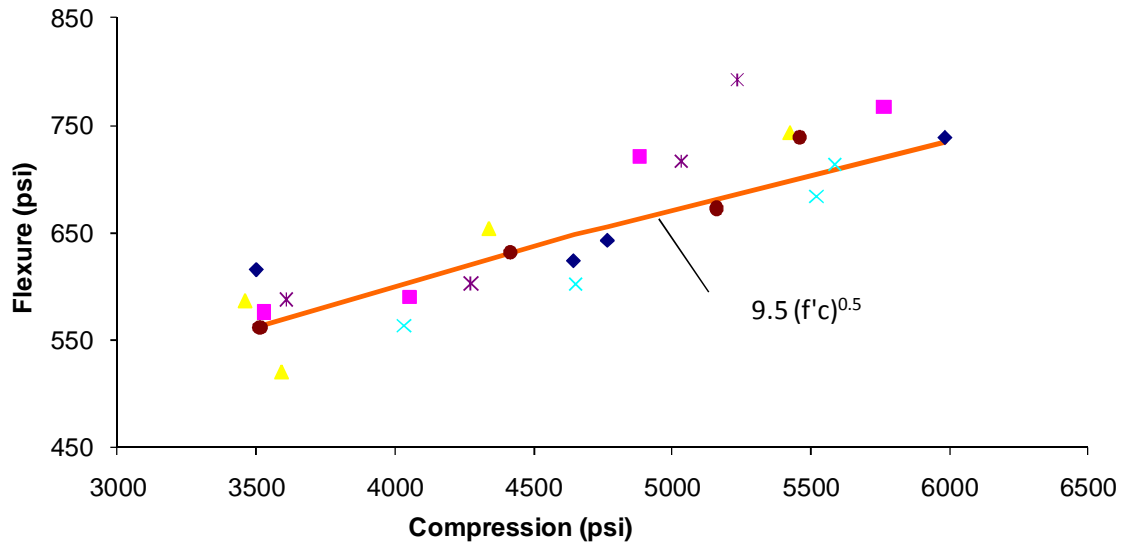


Figure 4.3 – A comparison between flexural and compressive strength for the mixtures.

7.0 Conclusion

This report has provided a summary of the work completed to date on ODOT project 2208

“Development and Implementation of a Mechanistic and Empirical Pavement Design Guide (MEPDG) for Rigid Pavements”. This document contains completed work for Task B and provides updates on the others. This project is currently on time, within budget, and has met the promised milestones for the first year. There is a chance that additional time may be needed to complete the shrinkage measurements in the project. Continued communication will be used between the research team and ODOT to determine if this is needed. Work is underway for phase 3 of the project.

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