Evaluation of High Absorptive Materials to Improve Internal Curing of Low Permeability Concrete

Norbert Delatte, Eric Mack, and John Cleary

for the
Ohio Department of Transportation
Office of Research and Development

State Job Number *134227*

March 2007

Technical Report Documentation Page

1. Report No	2. Government Accession No.	3. Recipient Catalog No.
FHWA/OH-2007/06		

4. Title and Subtitle	5. Report Date
Evaluation of High Absorptive Materials to Improve Internal Curing of Low Permeability Concrete	March 2007
	6. Performing Organization Code
7. Authors Norbert Delatte, Eric Mack, and John Cleary	**8. Performing Organization Report No.**
9. Performing Organization Name and Address Cleveland State University Department of Civil and Environmental Engineering Stillwell Hall Cleveland, Ohio 44115	**10. Work Unit No.**
	11. Contract or Grant No. 134227
12. Sponsoring Agency Name and Address **Ohio Department of Transportation** **1980 West Broad Street** **Columbus, Ohio 43223**	**13. Type of Report and Period Covered** Final report, 1 April 2005 – 31 March 2007
	14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

Early age cracking of bridge decks is a national problem, and may substantially reduce service lives and increase maintenance costs. Cracking occurs when the tensile stress exceeds the tensile strength of the concrete. This is a time-dependent phenomenon, since both the stress and strength change at early ages. Moisture loss increases stress (with increasing shrinkage) and impairs strength gain. Internal curing is one method that has been suggested to reduce early age bridge deck cracking, particularly of concretes with low water to cementitious materials (w/cm) ratios.

Many state highway agencies have implemented high performance concrete (HPC) for bridge decks. The low permeability of HPC is used to protect reinforcing steel and prevent corrosion. However, if the concrete cracks, then the protection may be greatly diminished. Transverse cracks due to concrete shrinkage allow water and corrosive chemicals to quickly reach the reinforcing steel causing corrosion and shortening the lifespan of the bridge deck. Reducing shrinkage cracking has been the focus of recent research into mitigation strategies.

One unintended consequence of the use of high performance concrete may be early-age cracking. Field studies have shown that, in some cases, high performance concrete bridge decks have cracked less than a year after placement. The use of internal curing to reduce autogenous shrinkage was investigated in this study. One method of internal curing was through the use of coarse aggregates with high absorption capacities. Another method discussed is the use of a partial replacement of the fine aggregate with a structural lightweight aggregate with a very high absorption capacity.

Bridge deck cracking is also affect by the nominal maximum size coarse aggregate. The effect on shrinkage with increasing size is discussed. ODOT's District 12, located in Northeastern Ohio, found in an investigation of 116 HPC bridge decks placed between 1994 and 2001 that bridges with little or no cracking used coarse aggregate with an absorption > 1 %, while 75 % of bridges with unacceptable cracking used coarse aggregate with absorption < 1 %. This report discusses the laboratory investigation of the field results to determine the better ways to prevent bridge deck cracking – internal curing or paste reduction by using an aggregate blend. The laboratory investigation found that the strongest effect on cracking was due to the replacement of a small maximum size coarse aggregate with an optimized coarse aggregate gradation. Increasing the coarse aggregate absorption level from < 1% to > 1% had a less dramatic effect. The use of LWA for internal curing to the low absorption coarse aggregate also had a less dramatic effect. Other classes of structural and paving concrete were also discussed, in addition to the HP concrete.

There were numerous benefits of internal curing for high performance concrete. The cracking tendencies were reduced. Concrete mixtures that did not have the lightweight fine aggregate cracked in less time. Specimens that contained the lightweight fine aggregate were far superior when dealing with shrinkage. Concrete strengths were also improved with LWA.

17. Key Words	18. Distribution Statement
Concrete, high performance, durability, bridge decks, structures, pavements, aggregates, lightweight aggregate, drying shrinkage, autogenous shrinkage	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161

19. Security Classif (of this report)	20. Security Classif. (of this page)	21. No of Pages	22. Price
Unclassified	**Unclassified**	142	

Form DOT F 1700.7 (8-72)

Evaluation of High Absorptive Materials to Improve Internal Curing of Low Permeability Concrete

State Job Number 134227

FINAL REPORT

Prepared in cooperation with the Ohio Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration

By

Cleveland State University
University Transportation Center for Work Zone Safety and Efficiency
and
Department of Civil and Environmental Engineering
Stillwell Hall
Cleveland, Ohio

March 2007

Research Team: Norbert Delatte (Principal Investigator), Eric Mack, and John Cleary (Research Assistants)

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Ohio Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

ABSTRACT

Early age cracking of bridge decks is a national problem, and may substantially reduce service lives and increase maintenance costs. Cracking occurs when the tensile stress exceeds the tensile strength of the concrete. This is a time-dependent phenomenon, since both the stress and strength change at early ages. Moisture loss increases stress (with increasing shrinkage) and impairs strength gain. Internal curing is one method that has been suggested to reduce early age bridge deck cracking, particularly of concretes with low water to cementitious materials (w/cm) ratios.

Many state highway agencies have implemented high performance concrete (HPC) for bridge decks. The low permeability of HPC is used to protect reinforcing steel and prevent corrosion. However, if the concrete cracks, then the protection may be greatly diminished. Transverse cracks due to concrete shrinkage allow water and corrosive chemicals to quickly reach the reinforcing steel causing corrosion and shortening the lifespan of the bridge deck. Reducing shrinkage cracking has been the focus of recent research into mitigation strategies.

One unintended consequence of the use of high performance concrete may be early-age cracking. Field studies have shown that, in some cases, high performance concrete bridge decks have cracked less than a year after placement. The use of internal curing to reduce autogenous shrinkage was investigated in this study. One method of internal curing was through the use of coarse aggregates with high absorption capacities. Another method discussed is the use of a partial replacement of the fine aggregate with a structural lightweight aggregate with a very high absorption capacity.

Bridge deck cracking is also affect by the nominal maximum size coarse aggregate. The effect on shrinkage with increasing size is discussed. ODOT's District 12, located in Northeastern Ohio, found in an investigation of 116 HPC bridge decks placed between 1994 and 2001 that bridges with little or no cracking used coarse aggregate with an absorption > 1 %, while 75 % of bridges with unacceptable cracking used coarse aggregate with absorption < 1 %. This report discusses the laboratory investigation of the field results to determine the better ways to prevent bridge deck cracking – internal curing or paste reduction by using an aggregate blend. The laboratory investigation found that the strongest effect on cracking was due to the replacement of a small maximum size coarse aggregate with an optimized coarse aggregate gradation. Increasing the coarse aggregate absorption level from < 1% to > 1% had a less dramatic effect. The use of LWA for internal curing to the low absorption coarse aggregate also had a less dramatic effect. Other classes of structural and paving concrete were also discussed, in addition to the HP concrete.

There were numerous benefits of internal curing for high performance concrete. The cracking tendencies were reduced. Concrete mixtures that did not have the lightweight fine aggregate cracked in less time. Specimens that contained the lightweight fine aggregate were far superior when dealing with shrinkage. Concrete strengths were also improved with LWA.

EXECUTIVE SUMMARY

Problem

Early age cracking of bridge decks is a national problem, and may substantially reduce service lives and increase maintenance costs. Cracking occurs when the tensile stress exceeds the tensile strength of the concrete. This is a time-dependent phenomenon, since both the stress and strength change at early ages. Moisture loss increases stress (with increasing shrinkage) and impairs strength gain. Internal curing is one method that has been suggested to reduce early age bridge deck cracking, particularly of concretes with low water to cementitious materials (w/cm) ratios.

Objectives

The purpose of this research was to develop methods to economically produce more durable and crack resistant high performance concrete using internal curing. Observations by ODOT District 12 indicated that there is a relationship between the amount of cracking on the bridge decks and the absorption of the coarse aggregate. District 12 developed a specification requirement for HPC concrete that requires aggregates to have 1% or greater absorption.
This research project was carried out to evaluate and validate District 12 findings and, if true, establish other alternate methods of providing internal moisture for curing. One alternative is the use of highly absorptive products in small quantities such as fine lightweight aggregate (LWA).

Description

A literature survey was carried out concerning internal curing and early age cracking, particularly of bridge decks. The research was divided into four phases: concrete mixtures using traditional ODOT materials and mixture designs, concrete mixtures using high absorption fine LWA, concrete mixtures using coarse aggregate with a larger nominal size in a blended mixture, and field testing.

For each concrete mixture, the following tests were performed:

- Fresh concrete properties – slump, air, and unit weight
- Hardened concrete properties – compressive, flexural, and splitting tensile, performed at 7 and 28 days, plus 56 and 90 days for high performance concrete
- Unrestrained shrinkage (bar) tests, sealed and unsealed – measurements taken up to 90 days
- Restrained shrinkage/cracking tendency (ring) tests – measurements taken up to 90 days.

The ring test is used to determine the time of cracking of a concrete specimen due to drying and autogenous shrinkage against the restraint of the steel ring. Tests were carried out until cracking or for a maximum of 90 days.

Crushed limestone coarse aggregates with three different absorption levels were tested, along with gravel aggregate at a single absorption level. Concrete mixtures that were very susceptible to early cracking were modified by the inclusion of small amounts of fine lightweight aggregate.

Conclusions & Recommendations

The strongest effect on cracking was due to the replacement of a small maximum size coarse aggregate (#8) with an aggregate blend of #8 and #57. ODOT's current HP mixture uses only #8 aggregate. No matter what the level of absorption, the shrinkage was dramatically reduced with a blended mixture. Only one of eight specimens made with an aggregate blend cracked before 90 days elapsed. Increasing the coarse aggregate absorption level from low to medium had a less dramatic effect, as did the introduction of LWA for internal curing to the low absorption coarse aggregate. Internal curing enhanced the early as well as the ultimate strength of the concrete. Compressive strengths increased by up to twenty percent when fine LWA was used.

Implementation Potential

Based on this research, there are two possible ways to substantially reduce bridge deck cracking. These methods may also be applied to other transportation concrete applications, particularly with low w/cm ratios.

The first way is by replacing small maximum size coarse aggregate (#8) with a larger size aggregate (e.g., # 57) or a blend of sizes. Since most producers have these materials readily available, the added cost should be small. Since larger coarse aggregate allows a reduction in cementitious material content, it would in fact be possible to reduce the cost of the concrete. This would require modifications to the current ODOT high performance (HP) specifications.

The second way is by internal curing, replacing a portion of the fine aggregate with fine LWA. Strength of the concrete is also increased. This method, however, may cost more than the first. Producers do not currently keep fine LWA on hand, and there may be costs associated with handling an additional material. The costs may, however, be outweighed by the performance benefits. Internal curing may also be used to improve the durability and performance of concrete pavements.

Economic benefits of implementation include substantial reduction in maintenance costs for concrete structures and pavements in both the long and short term. ODOT District 12 field observations have documented early cracking of bridge decks that is likely to add up to considerable maintenance expenditures over the projected life of the bridges, starting a few years after construction.

Further research of concrete with internal curing would be beneficial to document field performance and to investigate the effect on freeze-thaw durability, wear resistance, and permeability. Use of internal curing may also help reduce differential drying shrinkage of concrete pavements slabs, thus helping to reduce the magnitude of locked in warping (observed during the LTPP program). Internal curing is also likely to be beneficial for improving thin concrete overlays on bridge decks or pavements.

TABLE OF CONTENTS

LIST OF TABLES

FIGURES

EQUATIONS

INTRODUCTION

Background

The importance of curing concrete is well known. “Concrete must be properly cured if its optimum properties are to be developed. An adequate supply of moisture is necessary to ensure that hydration is sufficient to reduce the porosity to a level such that the desired strength and durability can be attained and to minimize volume changes in the concrete due to shrinkage. Concrete structures rarely fail because the specified design strength is not attained… loss of potential durability in the long term due to inadequate curing is a more widespread and insidious problem since the maintenance-free service life is reduced.” (Mindess et al., 2003, p. 287).

Cracking occurs when the tensile stress exceeds the tensile strength of the concrete. This is a time-dependent phenomenon, since both the stress and strength change over time, particularly at early ages. Moisture loss increases stress (with increasing shrinkage) and impairs strength gain.

This is illustrated in Figure 1. In the figure on the left, at time of cracking t_c the tensile stress (dashed line) exceeds the tensile strength of the concrete (solid line) and a crack develops. In the figure on the right, either the tensile stress is reduced, or the tensile strength is increased, or both, and the crack does not form. Tensile stress is reduced with proper curing, and tensile strength is increased, so curing is essential for preventing concrete cracking. Curing shifts the strength development curve upward and the stress development curve downward so that the tensile stress never exceeds the tensile strength of the concrete.

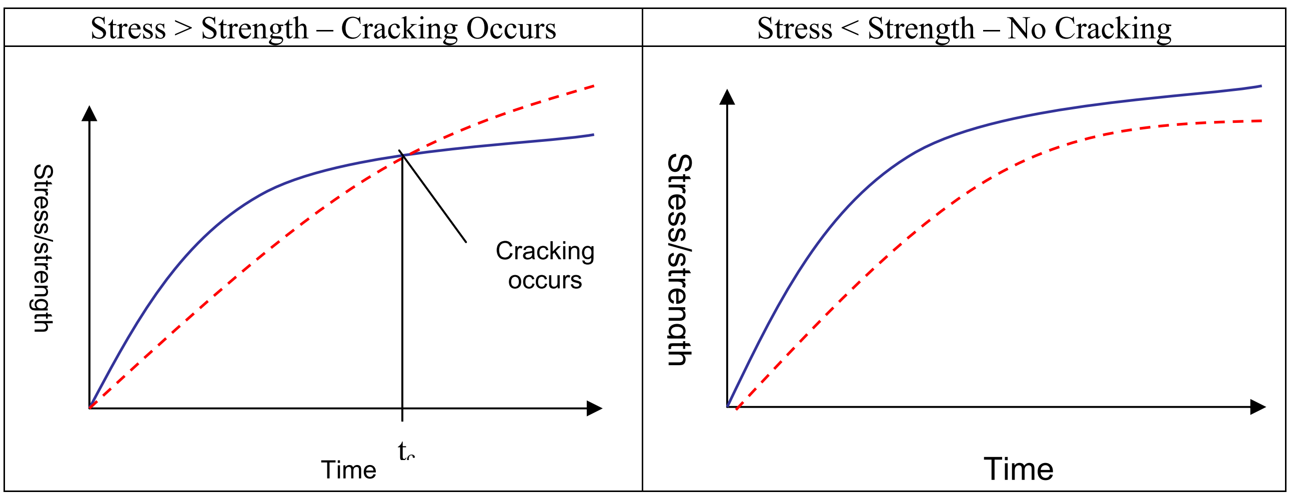


Figure 1: Cracking Tendency

This problem is of particular significance with high performance and high strength concretes. The water-cementitious ratio (w/cm) is defined by the American Concrete Institute (ACI) as “the ratio of the mass of water, exclusive only of that absorbed by the aggregates, to the mass of cementitious material in concrete, stated as a decimal and abbreviated as w/c or w/cm” (ACI 116R 2000 p. 72). If only cement is used, this is called the water-cement (w/c) ratio. “At the low w/cm ratios used for high-strength concrete, complete hydration cannot take place.” (Mindess et al., 2003, p. 535). Concrete with w/cm below 0.42 will self-dessicate unless water is added during the curing period. This leads to bulk, or autogenous shrinkage, with potential for internal microcracking (Mindess et al., 2003, p. 85). Autogenous shrinkage may be aggravated by the use of silica fume (Mindess et al., 2003, p. 429). ACI 116 Cement and Concrete Terminology defines "volume change, autogenous—change in volume produced by continued hydration of cement, exclusive of effects of applied load and change in either thermal condition or moisture content." (ACI 116R 2000 p. 71).

As discussed in the background and literature review section, internal curing has been proposed to improve the strength gain and reduce early age shrinkage of high performance and high strength concrete. This can be provided by an absorptive coarse aggregate, an absorptive lightweight fine aggregate, or some other material such as superabsorbent polymer particles (SAP).

ODOT problem statement

The following problem statement description was provided by ODOT for this project:

Properties of aggregates used in concrete affect the concrete quality, however aggregates are overall, inherently inconsistent. District 12's evaluation of bridge decks cracking indicates that there is a relationship between the amount of cracking on the bridge decks and the absorption of the coarse aggregate. They have set a specification requirement for HPC concrete that requires aggregates to have 1% or greater absorptions. This concept is attributed to the lack of "internal curing" of the concrete; autogenous cracking; and the ability of a higher absortive aggregate to provide some internal curing moisture to help lower the cracking.

As ODOT aggregate sources are 80% above 1% absorption the statewide condition for cracking should be only 20% if this autogenous – internal curing issue is true but currently reported statewide cracking percentages do not validate this number.

This research is to evaluate and validate District 12 findings and, if true, establish other alternate methods of providing internal moisture for curing. Possible other alternatives are using highly absorptive products in small quantities such as lightweight sand. Providing a solution such as this will allow the use of aggregates that are currently excluded by the District 12 solution yet are high quality in many other aspects. This will result in a steady state of acceptable conditions as targeted in Strategic Initiative 3.

Objectives of the Study

The primary objective of this study was to develop methods to economically produce more durable high strength and high performance concrete using internal curing. The objective was supported by the following goals:

- Document the benefit of internal curing for concrete properties
- Evaluate alternative methods to promote internal curing – e.g. coarse aggregate with absorption > 1 %, lightweight fine aggregate, and other materials such as polymers
- Develop a draft model specification for implementing internal curing by ODOT and other agencies
- Develop at least two classes of concrete with internal curing
    - Using coarse aggregate with absorption > 1 %
    - Using a lightweight structural fine aggregate (LWA) partial replacement
    - Consider a third class using superabsorbent polymer particles (SAP)

- Develop a field testing plan to demonstrate and evaluate the benefits of implementing internal curing in field construction

Other methods were investigated for reducing or eliminating concrete cracking, particularly of bridge decks.

Scope

The scope of this research was divided into two areas: internal curing for high performance bridge decks, and internal curing for other classes of concrete used in transportation. Those two areas were further broken down into two divisions: normal weight aggregate concrete (with varying levels of coarse aggregate absorption) and concrete with partial fine lightweight aggregate replacement. The influence of aggregate size and gradation was also considered.

Potential Benefits

Successful development of internal curing would help eliminate cracking in current HPC and other concrete mixtures. Internal curing could be used by other states and FHWA, as well as by ODOT. Successful implementation of internal curing would lead to a decrease in deck cracking. This would, in turn, lead to longer lives for bridge decks and other structural components, and a corresponding decrease in life cycle costs. This would be the main tangible benefit of this research.

Internal curing may also be used to improve the durability and performance of concrete pavements. Use of internal curing may also help reduce differential drying shrinkage of concrete pavements slabs, thus helping to reduce the magnitude of locked in curling (observed during the LTPP program). The research program was broadly structured so as to address applications of internal curing in addition to bridge decks. In particular, internal curing will be beneficial for improving thin concrete overlays on bridge decks or pavements.

Economic benefits will include substantial reduction in maintenance costs for concrete structures and pavements in both the long and short term. The ODOT District 12 report (Crowl and Sutak, 2002) documents early cracking of bridge decks that is likely to add up to considerable maintenance expenditures over the projected life of the bridges, starting a few years after construction.

Organization of this Report

This report consists of 10 chapters, starting with this Introduction. The second chapter is the Background and Literature Review, followed by ODOT Field Observations and Survey Results. The laboratory investigation is documented in three chapters, Experimental Design, Materials, and Experimental Results. Field Testing is discussed in a separate chapter. The three concluding chapters are Discussion, Implementation Recommendations, and Conclusions.

BACKGROUND AND LITERATURE REVIEW

This literature review addresses three issues: properties of concrete used in transportation, particularly those covered by ODOT specifications; the interaction between shrinkage, deformation, and crackiclass ng; and strategies to reduce shrinkage and cracking.

Where concrete bridge decks crack, the average chloride ion concentration can exceed the corrosion threshold for conventional reinforcement within a single year (Lindquist et al. 2006). Therefore, preventing cracking is essential for providing a long life for bridges and other structures and facilities. In 1988, it was observed that the value of concrete infrastructure in the U.S. was valued at $ 6 trillion, and thus improvements in concrete durability represented a multi-billion dollar opportunity to preserve facilities and reduce future expenditures (Concrete Durability 1988).

Transportation Concrete (ODOT Specifications)

Concrete used in transportation includes bridge substructures and superstructures, including bridge decks, pavements, and other applications. State agencies have developed a variety of specifications and proportioning options for different uses.

Proportioning options for ODOT concrete are provided in section 499 of the *Construction and Material Specifications* (State of Ohio 2005). The classes include HP, or high performance concrete, class C, which is used for pavement, and class S which may be used for structures. Classes C and HP may also be used for structures. These classes, along with some of the options used, are shown in Table 1. Option 2 for class C and S, which is not shown, reduces the cement content by 50 lb/yd^3 (30 kg/m^3) and requires the use of a water reducing admixture. The cement reduction and admixture are also used for Option 3. ODOT also specifies a class F concrete, which was not investigated in this study.

Table 1: ODOT Concrete Classes and Applications

Concrete Class	Cementitious Materials	Coarse aggregate size	Applications
HP 1	Cement, fly ash	# 8	Structures
HP 2	Cement, GGBFS	# 8	Structures
HP 3	Cement, fly ash, silica fume	# 8	Superstructures, including bridge decks
HP 4	Cement, GGBFS, silica fume	# 8	Superstructures, including bridge decks
S	Cement	# 57 or # 67	Structures
S Option 1	Cement, fly ash	# 57 or # 67	Structures
S Option 3	Cement, GGBFS	# 57 or # 67	Structures
C	Cement	# 7, # 78, # 8, # 57 or # 67	Pavements, structures
C Option 1	Cement, fly ash	# 7, # 78, # 8, # 57 or # 67	Pavements, structures
C Option 3	Cement, GGBFS	# 7, # 78, # 8, # 57 or # 67	Pavements, structures

This research focused primarily on class HP, because that was the class of concrete used for the bridge decks where the problems were observed in ODOT District 12. Class S and class C Option 1 were also tested, based on survey responses from ODOT districts.

High Strength Concrete/High Performance Concrete

The definition of high strength concretes has been changing over the course of the last eighty years. In the 1920s concrete compressive strength above 3,000 psi (21 MPa) was considered high strength concrete. In the 1950s 5,000 psi (34 MPa) was considered high compressive strength. During the 1960s, high strength concrete of up to 7,500 psi (52 MPa) in compression was being used in commercial construction.

In recent years compressive high strength concrete has reached 20,000 psi (138 MPa) in buildings. High-strength concrete is beneficial in construction because columns can support more weight and as a result be made thinner than normal strength concrete columns. Spans for prestressed concrete bridge girders may also be increased.

In the 1980's the production of high-strength concrete (HSC) became more common. Wider HSC application increased the use of higher cement contents, supplementary cementing materials (such as silica fume, fly ash and ground granulated blast furnace slag) and lower water-binder ratios as a result of the extensive use of superplastizers (Hoff, 2002).

The American Concrete Institute (ACI) defines high-performance concrete (HPC) as "concrete meeting special combinations of performance and uniformity requirements that cannot always be achieved routinely when using conventional constituents and normal mixing, placing and curing practices" (ACI 116R 2000, p. 17). These requirements may include ease of placement, compaction without segregation, early age strength, long-term mechanical properties, permeability, density, heat of hydration, toughness, volume stability, and long life in severe requirements (Russell, 1999).

While a high-strength concrete (HSC) is always a high-performance concrete (HPC), a HPC is not always a HSC. ACI defines a high-strength concrete as concrete as having a specified compressive strength for design of 6,000 psi (41 MPa) or higher. HPC is typically made with appropriate materials including superplasticizers, fly ash, blast furnace slags and silica fumes. These materials should be combined according to a mix design that is properly mixed, transported, handled, placed, and cured to give necessary performance in properties of concrete, including compressive strength, low permeability, high density, and resistance to environmental degradation.

With the development of HPC, the focus of its usage in comparison to HSC is based more on performance characteristics than on compressive strength. In the age of long service life expectations of concrete structures, the dimensional stability as well as the durability of concrete have become very important properties. By reducing the permeability of concrete, durability problems such as chloride ion transport, freezing and thawing damage, sulphate attack, corrosion of steel reinforcement, and acid attack can be controlled (Czarnecki and Kroman, 2005).

High performance concrete has low permeability. Low permeability of concrete is attained by lowering w/c ratio, by using sufficient cement, and by the addition of pozzolans such as silica fume, fly ash or slag. New technology regarding Portland cement and admixtures has helped facilitate high early strength concrete, yet experience and research indicate that HSC may be prone to higher early age cracking and shrinkage, thus allowing access of aggressive ions that in turn lead to serious durability problems. HPC mixture proportions, w/cm ratio and the incorporation of pozzolanic materials as well as chemical admixtures should provide advantageous performance characteristics for durability and strength of HPC construction (Czarnecki and Kroman, 2005). HPC has the most useful applications in concrete technology today when its special properties are give full consideration in design and construction, and fully developed (Harmon, 2005).

In recent years, a number of innovations have been developed to help improve the properties of high performance low permeability concrete. One of those innovations has decreased the amount of water due to the addition of super-plasticizer. Super-plasticizers are linear polymers containing sulfuric acid groups attached to the polymer backbone at regular intervals. In the 1980's the production of high-strength concrete (HSC) became more common and, to accomplish it, came the use of higher cement contents, supplementary cementing materials such as silica fume, fly ash and blast furnace slag cement and lower w/cm ratios as a result of the extensive use of super-plastizers (Hoff 2002). For very high-strength concrete silica fume is a necessity, but it will increase the cost of the concrete.

Other Classes of Transportation Concrete

In Ohio, Class C and Class S concretes are used in addition to Class HP (State of Ohio 2005). These may be used for pavements, substructures, and other elements of infrastructure. These classes differ from ODOT Class HP concrete in several important respects – generally w/cm ratios are higher, and larger coarse aggregates are often used. As a result, they would be expected to be less susceptible to drying and autogeneous shrinkage, and thus less prone to cracking. Class C and Class S concrete mixtures were evaluated in this study to determine if they would also benefit from internal curing.

In Dallas, Texas, concrete with internal curing has been used for residential paving. Intermediate sized lightweight aggregate with 3/8 inch to number 4 particle size (9.5 to 2.36 mm) was used as a substitution for about 200 lb/yd^3 (120 kg/m^3) of fine aggregate and 300 lb/yd^3 (180 kg/m^3) of coarse aggregate. Intermediate sized particles provide internal curing and improve the overall aggregate gradation. Over two years, about 550,000 yd^3 (420,000 m^3) of this concrete has been placed, nearly half on a single large project. Strength was improved, and the projects have exhibited very little cracking. A typical increase in compressive strength is about 1,000 psi (7 MPa) (Villareal and Crocker 2007).

Properties of Concrete

Properties of concrete may be divided into fresh and hardened properties. Fresh properties (e.g. slump and air content) are tested at delivery and placement, and the load of concrete may be rejected if they are not met. Hardened properties, such as strength and stiffness, are tested after hardening and curing. Compressive strength is routinely tested for quality control, and other properties may be evaluated for research.

The hardened properties of concrete are extremely important for the long term durability and performance of concrete structures. These properties include tensile and compressive strength, water-cement (or cementitious materials) ratio, cracking tendencies, relative humidity, potential of tensile forces (stress buildup), and modulus of elasticity. Properties are affected by mixture parameters such as binder contents and use of supplemental cementitious materials such as silica fume, fly ash, ground granulated blast furnace slag (Lam, p.1 2005).

Properties of concrete are of interest both at early ages and in the long term. While the early age properties are important, long-term properties such as shrinkage and permeability play significant roles (Lam, p.1 2005). A typical example is the distinct difference between fly ash and slag cement. When fly ash is added to concrete, the early age strengths are generally higher than concrete mixes with slag cement. Eventually, the strengths of concrete mixes with slag cement will surpass those with fly ash.

Shrinkage, Deformation, and Cracking

Shortly after placement, concrete may be susceptible to plastic shrinkage and plastic shrinkage cracking unless it is properly protected. Once it begins to harden, deformations occur due to drying shrinkage, autogenous shrinkage, thermal effects, and creep. Loading also causes deformations, but early age cracking may occur in the absence of external loads.

If the deformations are restrained, as in reinforced bridge decks, cracking occurs if the applied stress due to restraint of deformation exceeds the strength of the concrete. The splitting tensile strength is the best measure of resistance to cracking due to restrained deformation. The various sources of deformation in concrete are discussed in this section, with special attention to HPC.

Drying Shrinkage

Drying shrinkage is caused by the reduction of moisture from the cement paste components, producing a corresponding decrease in volume. Long-term drying shrinkage has typically been what is addressed in the literature and considered in structural design and detaling (Hoff, 2002). Joints in pavements and slabs are used to help control cracking. Drying shrinkage occurs in concrete that is exposed to the environment.

As a result, a difference in the RH between the environment and concrete water is transferred from the concrete to the environment. If concrete dries and shrinks and then is wetted again, even in the first days, it will not return to its original dimensions (Lam, p.13 2005).

Drying shrinkage is affected by paste, concrete, and environmental parameters. Paste parameters include the porosity and age of paste, both of which are affected by the w/c or w/cm ratio and the age of the paste. Other paste properties include curing temperature, cement composition, moisture content, and admixtures. Concrete parameters include aggregate stiffness and aggregate/cement content, as well as the volume-surface ratio and the thickness of the element. Environmental parameters are relative humidity and time and rate of drying, which are controlled by curing the concrete (Mindess et al. 2002, p. 420).

Internal restraint can lead to micro-cracking of cement-based materials. There are two internal restraint mechanisms of drying shrinkage micro-cracking. The first mechanism is self-restraining of the material. This is a result of the non-uniform shrinkage of the material caused by a moisture gradient that develops during drying. Important factors are the size and geometry of the specimen; in other words, the ratio of the surface area to the volume (Shiotani et al. 2002).

In plain cement paste specimens, this type of restraint causes tensile stresses near the drying surface and compressive stresses in the core of the specimen. When the tensile stresses exceed the tensile strength of the materials, micro-cracks with a limited penetration depth develop (Shiotani et al., 2002).

The second type of internal restraint in cement-based composites is provided by the presence of stiff aggregate particles. If the restraint caused by an aggregate particle in a shrinking matrix is large enough, radial and bond cracks are formed around the aggregate particle (Shiotani et al., 2002). This type of micro-cracking may sometimes take place in the core of the material.

An experimental program conducted by Rutgers University and funded by the New Jersey Department of Transportation entitled "Development of HPC Mix Designs for Transportation Structures in New Jersey" sought to find the drying shrinkage and compressive creep of HPC/HSC. Creep and shrinkage are important for high rise structures and long-span bridge structures. The research was undertaken to reduce the creep-induced sagging and shrinkage cracking that have been observed in concrete structures. Since HSC and HPC contain different admixtures than NSC and/or greater quantities, and admixtures can greatly affect the behavior of concrete's mechanical properties, it was essential that the creep and shrinkage of HSC and HPC be studied (Suksawang et al., 2005).

Materials including silica fume, slag and fly ash have been used in the United States to improve HPC and HSC. These pozzolanic materials help make HSC/HPC denser and more resistant to chemical attacks, and alter the mechanical properties that affect creep and shrinkage. Even though there have been significant discoveries in concrete technology, creep and shrinkage predictions are still typically based on the properties of NSC. The study at Rutgers investigated the effect of pozzolanic materials on the shrinkage and creep behaviors of HPC/HSC to help develop specifications for the New Jersey Department of Transportation. Eight mixtures were cast and tested. Three mixtures contained varying percentages of silica fume, three had varying percentages of Class F fly ash, and two had different combinations of silica fume and fly ash (Suksawang et al., 2005).

The creep and shrinkage prediction model most commonly used in the United States is the ACI 209 model. ACI 209 is a general purpose model and does not incorporate any adjustments for the strength of the concrete (Suksawang et al., 2005). The ACI 209 model uses the following variables: 1) relative humidity, 2) specimen size, 3) type of curing method used, and 4) the age at the end of the curing time.

The research found that because the silica fume has a high demand for water, drying shrinkage increases as the content of silica fume in the concrete increases. However, fly ash has a lower water demand and therefore drying shrinkage is lessened as the fly ash content increases. Silica fume concrete has a lower capillary void content than fly ash concrete and therefore silica fume has a lower creep than fly ash (Suksawang et al., 2005). Creep allows stresses to relax and reduces cracking tendencies.

Autogenous Shrinkage

Autogenous Shrinkage is defined as a concrete volume change that occurs without moisture transfer to the environment. At early ages (the first few hours), before the concrete has formed a hardened skeleton, autogenous shrinkage is the result of only chemical shrinkage. At later ages (> 1 day), the autogenous shrinkage can also result from self-desiccation, since the hardened skeleton resist the chemical shrinkage. Self-desiccation is the localized drying resulting from a decreasing relative humidity (Hoff, 2002). The American Concrete Institute defines relative humidity as the as ratio of the quantity of water vapor actually present to the amount present in a saturated atmosphere at a given temperature; expressed as a percentage. Autogenous shrinkage is very similar to drying shrinkage, but with no loss of moisture. Autogenous shrinkage begins at the interior of the concrete, unlike drying shrinkage.

As early as 1948, it was shown that autogenous shrinkage due to self-desiccation occurs when the w/cm is below 0.42, as all the mixing water is consumed. Other investigations noted that the w/cm limit for autogenous shrinkage can vary between 0.36 and 0.48 depending on the cement type. When the w/cm is much lower than 0.42, and the cement can no longer gain curing water, the cement seeks extra water from the internal pores and thus lowers the relative humidity.

The autogenous shrinkage behavior of lightweight aggregate concrete typically is quite different than that of the NWC. Its time-dependent behavior depends on the initial moisture states of the lightweight aggregate, the size of the aggregate used, and the amount of aggregate used. The key is getting enough additional moisture into the concrete by use of moisture preconditioned LWA for hydration of the binder to continue (Hoff, 2002).

Autogenous deformation and change of the relative humidity (RH-change) within concrete have been observed for over a century. However, only within the last decade have these phenomena received widespread attention. The reason for this is that autogenous deformation and autogenous RH-change are phenomena of special importance within high-strength (high-performance) concrete technology, and a significant utilization of these concretes did not take place until the early 1980s (Hoff 2002). Routine use of concrete with w/cm $\leq$ 0.42 is relatively recent.

Prior to this, the earliest description of autogenous shrinkage dates back to 1900 when a concrete research pioneer, Le Chatelier, described self-desiccation and created a registry system for the properties of cement (Hansen and Jensen, 2000). In this description, Le Chatelier stated that it is important to differentiate between the absolute volume, or chemical shrinkage and the apparent volume, or autogenous deformation, of a hardening cement (Hansen and Jensen, 2000).

Some years later, in 1927, reported measurements of autogenous RH-change for cement with w/c-ratios of 0.24-0.36 showed that the concrete may have internal relative humidity of 90% after one month of hardening. The following year, in 1928, the researchers Neville and Jones gave a description of a tool used to measure the volumetric deformations of cement mixtures during sealed hardening at a constant temperature (Hansen and Jensen, 2000).

In 1934, Lynam was the first to use the term autogenous shrinkage, defined as, shrinkage that is not due to thermal causes or to a loss of moisture in the air. By 1940, experimental results had been published on autogenous deformation. After five years of hardening, the magnitude of autogenous shrinkage was observed in the range of 50-100 μ strain. These numbers were reasonably small compared to thermal deformation and drying shrinkage, and for that reason little attention was given to the concept in concrete research and practice for many years. In the 1940s and 1950s, it became possible to describe and perform calculations in theory on the phases of hardening cement. Through these calculations, researchers found that at sufficiently high w/c-ratios self-desiccation would not take place (Hansen and Jensen 2000).

One problem with HPC mixtures has been an increased tendency to undergo early-age cracking. While this cracking may or may not reduce the compressive strengths of concretes, it compromises their durability. The occurrence of early-age cracking is one that is complicated and depends on many factors, including autogenous strains, autogenous stresses, drying, stress relaxation, structural detailing, structural execution and thermal effects (Geiker et al. 2003).

ACI 308 (1992) notes that for moist curing pavements and other slabs on the ground at temperatures above 40^{o}F (5^{o}C), "the recommended minimum period of maintenance of moisture and temperature for all procedures is 7 days or the time necessary to attain 70% of the specified compressive or flexural strength, whichever period is less." The specification does not address maintaining relative humidity, which is crucial to prevent autogenous shrinkage.

Thermal Deformation

Thermal effects also cause deformation of concrete. The deformation is equal to the product of the thermal coefficient α, the temperature change ΔT, and the length of the member. The thermal coefficient α of concrete depends primarily on the thermal coefficient of the aggregate used for the concrete. Limestone generally has a much lower thermal coefficient than gravel, so the thermal deformations and stresses are smaller in concrete made with limestone than those made with gravel.

Creep

Creep is defined by ACI 116 (2000) as "time-dependent deformation due to sustained load." It also relaxes stress under constant deformation. Creep is affected by the same parameters as drying shrinkage, in much the same way. It is also affected by the applied stress and duration of the load (Mindess et al. 2002, p. 420). In a bridge deck subject to drying and autogenous shrinkage, creep will have the tendency to reduce stresses over time. Both creep and drying shrinkage are addressed in ACI 209 (2005).

Total Deformation and Strain

In the absence of externally applied load or prestressing, the total deformation due to all of these effects is:

Equation 1: Total Deformation and Strain

$$\delta_{total} = \varepsilon_{total} L \text{ and } \varepsilon_{total} = \varepsilon_{sh} \pm \alpha \Delta T - \varepsilon_{cr}$$

Where δ_{total} = total strain;
L = length
ε_{total} = total strain;
ε_{sh} = shrinkage strain, due to both drying and autogenous shrinkage;
α = coefficient of thermal expansion;
ΔT = change in temperature; and
ε_{cr} = creep strain

Positive values are tensile. Creep is assumed to subtract from total strain and therefore relax stress.

Cracking Due to Restrained Shrinkage

Cracking of concrete depends on several variables. The degree of restraint plays a significant role. If concrete is prevented from shrinking freely in structures, residual tensile stresses develop which, if high enough, can result in cracking. This cracking is particularly troublesome at early-ages of concrete while it is building strength slowly. Figure 1 shows how stress and strength development affect cracking. Models have been proposed to predict the age of (or potential for) cracking based on comparing the time dependent tensile strength of concrete with the residual stress level that develops as a result of restrained shrinkage (Weiss et al., 2000).

If the deformation is restrained, the total strain (Equation 1) may be used to calculate the stress σ built up:

Equation 2: Stress Due to Restrained Shrinkage

$$\sigma = \varepsilon_{total} E$$

Where E = modulus of elasticity of the concrete.

A ring shrinkage test may be used to assess the cracking risk due to restrained shrinkage. The restrained ring-test has become widely used test for assessing the potential shrinkage cracking of concrete mixtures. The ring test is inexpensive, easy to carry out, and provides sufficient restraint. Both AASHTO and ASTM have published standards for ring shrinkage tests of concrete, AASHTO PP34-99 (1999) and ASTM C1581 (2004).

The ring shrinkage test measures the strain build up against a steel ring. The steel ring restrains the concrete and prevents it from shrinking freely so that it develops tensile stresses (Hossain and Weiss 2004). In the study completed by Czarnecki and Kroman (2005), tests were conducted using AASHTO provisional standard PP34-99, Standard Practice for Estimating the Cracking Tendency of Concrete, using ring-type specimens to reproduce restrained shrinkage cracking of a bridge deck. The concrete exposed to fifty percent RH had the highest level of shrinkage regardless of the type of mix for both the cement contents and the aggregate sizes (Czarnecki and Kroman 2005).

While previous studies provide interesting results and present a useful background for the proposed study, there remains one key item missing from the body of knowledge on internal curing. Although there are some limited shrinkage tests, there are no comprehensive tests for cracking tendency or autogenous shrinkage. The only broadly based investigation of cracking tendency remains the field observations of the ODOT District 12 report (Crowl and Sutak, 2002). This study is reviewed in detail in the next section of this report.

In the study documented in this report, the concrete rings were cast around the outside of a steel ring of specific dimensions. The inside of the steel rings was instrumented with four strain gauges placed at the quarter points around the ring. The strains were recorded every thirty minutes by a data acquisition system with the top surface of the concrete sealed and drying on the exposed sides of the concrete rings at fifty percent relative humidity.

Strategies to Reduce Shrinkage and Cracking

A variety of strategies have been developed to help control shrinkage of concrete, particularly shrinkage of HPC and HSC. Drying shrinkage is controlled, in large part, by traditional curing methods.

Particular mitigation strategies to help control autogenous shrinkage include control of the cement particle size distribution, the addition of saturated lightweight fine aggregates, the new concept of water entrained concrete, the use of a controlled permeability framework, the addition of shrinkage-reducing admixtures (more commonly used to control drying shrinkage), and the modification of the mineralogical composition of the cement (Bentz and Jensen, 2004). Mindess et al. (2002) defines the absorption capacity to be the maximum amount of water the aggregate can absorb. It is calculated from the difference in weight between the surface saturated dry and oven dried states.

The basic characteristics of cement paste that help control autogenous shrinkage include the surface tension of the pore solution, the geometry of the pore network, the kinetics of the cementitious reactions, and the visco-elastic response of the developing solid framework.

Traditional Methods for Curing Concrete

Traditional methods of curing concrete include water added and water retained methods. Water added curing uses water ponded on the surface, or saturated covering such as mats or burlap. This type of curing is common for bridge decks. Pavements are usually cured with sprayed curing membrane forming compounds, a water retained method.

These may not be adequate for HPC and HSC with very low w/cm ratios. As the outside of the concrete cures, it rapidly becomes too impermeable for external supply of water (Weber and Reinhardt, 2003). As a result, the only water available is within the concrete, leading to self-dessication and autogenous shrinkage. External water can not penetrate into low permeable concrete.

During air exposure of high strength concrete water evaporation was observed, which, in turn resulted in a reduction of compressive strength and the manifestation of microcracks. Therefore, the use of wet curing, even for an extended period of time was not a reliable method to definitely improve the properties of HSC (Weber and Reinhardt, 2003). Typically, after seven days of moist curing, most contractors spray the concrete with a curing compound.

Internal Curing

In the early 1990s, internal curing was proposed for HSC. The basic idea behind the use of LWA is substituting a portion of the original aggregates from the mix design with pre-wetted LWA (Weber and Reinhardt, 2003).

In order to achieve the intended design properties for concrete use it is important to properly cure the concrete (Weber and Reinhardt, 2003*)*. Internal curing refers to the process by which the hydration of cement is aided by the availability of additional internal water that is not part of the mixing water. The additional internal water for internal curing is most often provided by using small amounts of saturated, fine, LWA or superabsorbent polymer particles in the concrete (Bentz et al. 2005). Benefits of internal curing include increased hydration and strength development, increased durability, reduced permeability, and reduced autogenous shrinkage and cracking.

The impact of internal curing begins immediately with the initial hydration of the cement, with strength benefits that can typically be observed at ages as early as two days (Bentz et al., 2005). Internal curing is particularly advantageous in concrete mixtures with a low water to cementitious materials ratio (w/cm). In concretes that have a low w/cm ratio, the permeability of the concrete quickly becomes too low to allow the effective transfer of water from the external surface to inside of the concrete. When implementing internal curing of concrete it is possible to ensure that there will be continued availability of sufficient amounts of moisture even deep within the concrete mass (Lam, 2005).

Roberts (2004) notes:

- Concrete properties that are improved by internal curing include early age and ultimate compressive, flexural, and tensile strength as well as durability. Autogenous cracking and permeability are both reduced. Specific tests results at a w/cm ratio of 0.434 include a 25 % reduction in 84 day permeability (coulombs), a 14 % increase in 3 day flexural strength, a 10 % increase in 28 day compressive strength, a 6 % increase in 28 day tensile strength, and a 2 % increase in durability factor.

- Cement particle hydration at lower w/cm ratios is improved by providing water through a source such as lightweight sand produced from structural grade expanded shale
- The Chesapeake Bay Bridge, built in 1952, used non-air-entrained concrete with structural lightweight coarse and fine aggregates and is still in good condition.
- 1,500 pounds per cubic yard (890 kg/m^3) of normal weight limestone coarse aggregate with an absorption of 1 % (as used by ODOT district 12) supplies 15 pounds per cubic yard (8.9 kg/m^3) of water to the curing concrete. Similarly, 100 pounds per cubic yard (59.3 kg/m^3) of fine LWA replacement with 15 % absorption supplies 15 pounds per cubic yard (8.9 kg/m^3) of water.

In addition to the Roberts (2004) presentation, the following unpublished papers and reports were reviewed in the course of this research:

- Hoff, George C, *The Use of Lightweight Fines for the Internal Curing of Concrete,* Hoff Consulting LLC, Prepared for Northeast Solite Corporation, August 30, 2002. This is a comprehensive report of world wide research on internal curing.
- Roberts, John W., *The 2004 Practice and Potential of Internal Curing of Concrete Using Lightweight Sand,* presented at Advances in Concrete Through Science and Engineering, Northwestern University, Evanston, Illinois, March 22 – 24, 2004
- Roberts, John W., *Improving Concrete Pavements Through Internal Curing,* presented at the Open Session American Concrete Institute meeting in Detroit, Michigan, April 23, 2002.
- Roberts, John W., and McWhorter, James F., Jr., *Internal Curing,* Northeast Solite Corporation. This paper summarizes results of tests conducted in 2001 – 2001 on lightweight aggregate in New York and Kentucky.
- Three laboratory test reports prepared by PSI, Inc. for Northeast Solite.

At the American Concrete Institute Fall 2002 convention in Phoenix, Arizona, two dozen presentations were made on autogenous deformation of concrete and high performance structural lightweight concrete (Ries and Holm, 2004, Jensen et al., 2004). Three papers specifically addressed internal curing:

- Geiker at al., (2004) compared two types of internal curing – saturated lightweight fine aggregate and addition of superabsorbent polymer particles (SAP). They found that the use of these materials reduced autogenous shrinkage.
- Hammer et al. (2004) noted that the efficiency of lightweight aggregate (LWA) for internal curing depends on amount of water in LWA, particle spacing factor for LWA, and LWA pore structure. The ability of the LWA to release water is of prime importance.
- de Jesus Cano Barrita et al., (2004) investigated internal curing for self-consolidating concrete as well as concrete with silica fume and found it to be useful.

The most interesting recent development reported in this work is the use of superabsorbent polymer particles (SAP) as a third alternative to moderately absorptive coarse aggregate or expanded shale structural lightweight fine aggregate partial replacement. However, at least for the immediate future, SAP may be more expensive. SAP may eventually become economical – 1 pound (454 grams) of a polymer that absorbs 15 times its weight in water would provide internal curing equivalent to 1,500 pounds per cubic yard (890 kg/yd^3) of coarse aggregate with 1 % absorption or 100 pounds per cubic yard (59.3 kg/m^3) of structural LWA with 15 % absorption. However, Lam (2005) found that SAP was very difficult to mix into the concrete, even in the laboratory.

This research evaluated two methods of internal curing. The first was the use of a coarse aggregate with an intermediate level of absorption. The second was the addition of lightweight fine aggregate to the concrete, as discussed below.

Internal Curing Using Lightweight Aggregates

The effectiveness of lightweight aggregate (LWA) as an internal curing agent depends primarily on three factors: 1) The amount of water in the LWA, 2) the LWA particle spacing factor and 3) the LWA pore structure. Theoretical models exist that describe the interaction of the 3 factors, but more experimental work is needed to fully understand their relationship (Hammer et al. 2003).

The optimum amount of LWA used to attain internal curing is a function of the type of LWA used, the amount and size of that LWA, the degree of moisture preconditioning the LWA receives, the water-binder ratio that exists at mixing, the type and amount of binders used in the concrete mix, and the extent and amount of external moist curing afforded to the concrete element (Hoff, 2002). Bentz et al. (2005) proposed an equation to estimate the amount of LWA needed for internal curing of any given concrete mixture:

Equation 3: Lightweight Aggregate Quantity

$$M_{LWA} = \frac{C_f \times CS \times \alpha_{\max}}{S \times \phi_{LWA}}$$

where:

M_{LWA}	=	mass of (dry) fine LWA needed per unit volume of concrete (kg/m^3 or lb/yd^3);
C_f	=	cement factor (content) for concrete mixture (kg/m^3 or lb/yd^3);
CS	=	chemical shrinkage of cement (g of water/g of cement or lb/lb);
α_{max}	=	maximum expected degree of hydration of cement;
S	=	degree of saturation of aggregate (0 to 1); and
ϕ_{LWA}	=	absorption of lightweight aggregate (kg water/kg dry LWA or (lb/lb).

During hydration of the cement, a system of capillary pores is formed in the cement paste. The radii of these pores are smaller than the pores of the LWA. As soon as the internal relative humidity (RH) decreases (due to hydration and drying), a humidity gradient develops. With the LWA acting as a water reservoir, the pores of the cement paste absorb the water from the LWA by capillary suction (Hoff, 2002). In theory, the capillary forces of the cement paste are at high enough levels to soak up the water from the LWA grain and move it to the drier cement paste, where a reaction with the un-hydrated cement can occur (Weber and Reinhardt, 2003).

The un-hydrated cement particles from the cement paste now have more free-water available for hydration. The new hydration products grow in the pores of the cement paste, thus causing them to get smaller. The capillary suction, which is the inverse to the square of the pore radius, becomes larger as the radius becomes smaller (Weber and Reinhardt, 2003). As a direct result, this capillary suction process enables the pores to continue to absorb water from the LWA. This continues until all the water from the LWA has been transported to the cement paste (Hoff, 2002). This progression of the reduction in size of the capillary pores along with capillary suction of water from the LWA creates a pressure difference for water transport from the LWA to the cement paste. The water will stop being moved from the LWA once the RH in the LWA grain and in the cement paste that has hardened are in balance (Weber and Reinhardt, 2003).

On the macroscopic level, the humidity gradient on the surface of concrete members exposed to the environment should be given consideration. The lower the RH of the environment for the structure, the steeper the gradient between the surface layer and the more interior of the concrete structure (Weber and Reinhardt, 2003). Due to water evaporation this gradient increases. At the surface layer, the moisture from the LWA will be transported to the cement paste faster than in the interior of the element. Because the water from the LWA is chemically bound, the structure on the surface is denser, reducing water evaporation. As a result, the process of diffusion becomes slower.

At the surface of the concrete, an additional humidity gradient occurs due to evaporation from the concrete surface. This accelerates the appearance of the localized humidity gradient. The water from the LWA near the surface is then used up faster than in the interior of the concrete, thus causing the near-surface layer of the concrete to become denser in a shorter period of time. This helps reduce the amount of water that would normally evaporate and contributes to improve internal curing of the concrete. It also leads to a reduction in or no stresses due to drying, and helps eliminate surface cracking. This is, of course, important for bridge deck cracking. Typically, the addition of LWA in concrete reduces modulus of elasticity. Reductions in the modulus of elasticity of the concrete can be beneficial in reducing cracking (Hoff 2002).

Coarse Aggregate Size and Gradation

"Aggregates generally occupy 70 to 80 percent of the volume of concrete and therefore can be expected to have an important influence on its properties" (Mindess et al. 2003: 121). Aggregate is not simply an inert filler in concrete, and its properties deserve careful consideration. Grading of an aggregate is determined by a sieve analysis, where the mass of an aggregate sample retained on each of a number of standard sieves is recorded. Two key parameters are the maximum aggregate size and the shape of the gradation curve.

Absorption and surface moisture are of significance for calculating water that aggregate will add to or subtract from paste, and are used in mixture proportioning. Specific gravity is used to establish weight-volume relationships, also for mixture proportioning. Unit weight differs from specific gravity in that it includes not only the volume of the particles but the volume of the space between them when they are densely packed (Mindess et al. 2003: 133 – 140).

Use of a continuously or densely graded aggregate will also reduce paste requirements, since the smaller aggregate fills gaps in the larger aggregate. Uniformly graded or gap graded aggregates require more paste.

"Aggregate grading research for soils, base, asphalt, and other applications has proven that the best performance is derived from that blend of equi-dimensional particles that are well-graded from coarsest to finest. Optimum combined aggregate grading is important for portland cement concrete because it minimizes the need for the all-important second mix component—the paste—and has a significant effect on the air-void structure in the paste. The paste volume should be no more than is necessary to provide lubrication during placement and bind the inert aggregate particles together to resist the forces that will affect the mass during its service life… Gap grading (especially at the No. 4 and 8 sieves) and excessive fine sand and cementitious materials content were found to cause problems. Corrections to fill gaps in the aggregate grading led to significant reductions in water, improvements in mobility and finishability, and increases in strength." (Shilstone and Shilstone 2002: 81).

Therefore, use of the largest maximum size of coarse aggregate practicable reduces the paste requirements of concrete. ODOT specifications allow both larger # 57 and smaller # 8 coarse aggregates as alternates in some mixtures, and only # 8 in HPC. Use of # 57, or a blend of # 57 and # 8, is likely to reduce cracking tendencies.

Previous ODOT research found that coarse aggregates up to grade 357, or a 2 inch (50 mm) nominal maximum size, could be used in concrete structures and pavements. It was found that there were no adverse effects on compressive strength, and economy could be improved. This research did not address class HP concrete (Ioannides and Mills 2006a, Ioannides et al. 2006b).

ODOT FIELD OBSERVATIONS AND SURVEY RESULTS

Introduction

According to the National Bridge Inventory, there are more than 27,900 bridge structures in the state of Ohio. More than 7,260 (26 %) of Ohio's bridges were classified as being either structurally deficient or functionally obsolete in the 2000 calendar year (Fitch et al., 2002).

The Ohio Department of Transportation has used high performance concrete (HPC) to build concrete bridge decks for more than 14 construction seasons in District 12 (Crowl and Sutak, 2002). District 12 is responsible for planning, designing and maintaining the network of transportation in Cuyahoga, Geauga and Lake Counties in the Northeast part of Ohio.

Northeastern Ohio, and particularly the ODOT District 12 and greater Cleveland area, has a large number of bridges located in a harsh winter environment. Large amounts of road salt and other chemicals are used on bridge decks in an attempt to keep them clear for traffic. These chemicals may accelerate the corrosion of reinforcing steel and the deterioration of bridge decks. Thus, it is important to prevent or reduce bridge deck cracking.

During the 2001 construction season, ODOT District 12 personnel observed extensive cracking of eleven of fourteen new HPC bridge decks. The bridge decks were made with class HP3 and HP4 concrete, which used cement, silica fume, and either fly ash (HP3) or ground granulated blast furnace (GGBF) slag (HP4). All coarse aggregates were crushed limestone. The transverse cracks were regularly spaced and showed signs of leaching. An analysis of six projects is provided in Table 2. This preliminary analysis concluded (Crowl and Sutak 2002):

- Two bridge decks from the 2001 season without transverse cracks used coarse aggregate with an absorption of 1.39 or 1.52 percent.
- Four bridge decks from the 2001 season with substantial transverse cracking used coarse aggregate with an absorption of 0.41 percent.

Table 2: Comparison of Bridge Decks With and Without Cracking, 2001 Construction Season

Project	Location	Coarse Aggregate Absorption (%)
281(99)	WB I-480 over Rockside	0.41
197(00)	EB Fairmount over I-271	0.41
528(00)	Dover Center, Cahoon, and Canterberry	0.41
107(01)	Wagar, Northview, and W159th	0.41
Uncracked Bridge Decks		
480(99)	NB and SB I-271 over Tinkers Creek	1.39
157(01)	Highland Road over I-271	1.52

A survey of the use of high performance concrete in bridge decks was carried out over four years. The study evaluated fourteen bridge decks on six different construction projects, along with three HPC bridge decks that had been included in previous surveys. Since 1994, bridge decks in the state of Ohio have been inspected for their structural integrity. Table 3 shows the bridge deck cracking problems in District 12 discovered through those inspections.

Table 3: HPC Deck Cracking by Year

Year	Number of Bridge Decks	Number of Bridges with Cracking
1994	10	0
1995	8	0
1996	12	1
1997	14	5
1998	22	10
1999	13	10
2000	17	13
2001	13	10

The transverse cracking spacing was consistent for each bridge; but varied from bridge to bridge. These transverse cracks appeared within three months after construction. Table 3 illustrates the rise in frequency of cracking in high performance concrete mixtures used in bridge decks from 1994 until 2001. These alarming statistics raised concerns about durability of the HPC. Engineers at District 12 investigated possible reasons for this cracking occurrence.

ODOT District 12 Investigation

In order to gather data on the background and extent of the problem, an investigation was conducted on the bridge decks constructed with HPC between 1994 and 2001. The study covered 116 HPC bridge decks, and produced the following findings (Crowl and Sutak, 2002):

- Over the 1994 – 2001 seasons, all 64 bridge decks with minimal or no cracking used coarse aggregate with absorption > 1 %. However, 75 % of the 52 bridge decks with severe cracking used coarse aggregate with absorption < 1 %. Bridge decks constructed in 1994 and 1995 did not crack – the early age cracking first began to appear in 1996.
- The use of synthetic fibers did not seem to prevent cracking.

One possibility considered was poor construction techniques. This theory was ruled out because the same problem was found with several contractors. Procedures for bridge deck curing were believed to be adequate.

The investigators examined the individual concrete batch tickets, provided to ODOT. They contained information about the supplier of the cement, other cementitious materials, and fine and coarse aggregate. It was noted that concrete made with coarse aggregates with low absorption tended to crack more often than that with coarse aggregates with a higher absorption.

The deck of the SR44 northbound bridge over I-90 was replaced in 2000. The deck developed transverse cracks at five-foot (1.5 m) intervals. The cracking is shown in Figure 2. Table 4 shows the mix quantities and absorption capacities. The absorption capacity for the coarse aggregates was less than one percent.

For the River Road bridge the absorption capacity of the aggregates was significantly higher, as shown in Table 5. There was no cracking in this bridge deck, which is shown in Figure 3. The two decks were placed in consecutive summers. The absorption capacity of the River Road bridge concrete aggregate was greater than two percent. Fourteen similar cases were found. It was hypothesized that this difference in absorption capacity was the key to the difference in cracking.

Figure 2: SR44 northbound over I-90 (Crowl and Sutak 2002)

Table 4: SR44 northbound over I-90 Project Mix Design (Crowl and Sutak 2002)

High Performance Mix #4 Modified		**w/c = 0.42 % Air = 7.0**	
Material	Specific Gravity	Absorption Capacity	lbs/yd^3 (kg/m^3)
GGBF Slag	2.90	N/A	170 (101)
Natural Sand	2.61	1.04	1275 (756)
Number 8 Stone	2.76	**0.81**	370 (219)
Number 57 Stone	2.76	**0.35**	1390 (825)
Type 1 Cement	3.15	N/A	430 (255)
Water	1	N/A	252 (149)

Table 5: River Rd. over I-90 Mix Design (Crowl and Sutak 2002)

High Performance Mix #4 Modified		**w/c = 0.42 % Air = 7.0**	
Material	Specific Gravity	Absorption Capacity	lbs/CY (kg/m^3)
GGBF Slag	2.90	N/A	170 (101)
Natural Sand	2.61	0.99	1275 (756)
Number 8 Stone	2.58	**3.06**	370 (219)
Number 57 Stone	2.59	**2.37**	1390 (825)
Type 1 Cement	3.15	N/A	430 (255)
Water	1	N/A	252 (149)

Figure 3: River Rd. over I-90, with no deck cracking (Crowl and Sutak 2002)

Phased Construction

Emery Road over I-271 provided an interesting case study. The bridge deck was placed in two phases. The same general contractor and the same concrete supplier were used. One phase of the bridge deck cracked, while the other did not crack. It was determined that half way through the summer season, the concrete plant changed the source of supply of coarse aggregate. The fine aggregate remained the same throughout the season. The comparison is shown in
Figure 4.

The case study strongly suggested that the aggregate source was related to the cracking. Every component of the mixture design remained constant throughout the process except the type of silica fume in phase 1. The mixture designs for phase 1 and phase 2 are shown in Table 6 and Table 7. The main difference between the aggregates was the absorption level.

Table 6: Phase 1 Mixture Design Emery Road Over I-271

High Performance Mix #4 Modified		**w/c = 0.42 % Air = 7.0**	
Material	Specific Gravity	Absorption Capacity	lbs/CY (kg/m^3)
GGBF Slag	2.92	N/A	170 (101)
Natural Sand	2.62	1.16	1275 (756)
Number 8 Stone	2.54	1.93	370 (219)
Number 57 Stone	2.54	1.52	1390 (825)
Type 1 Cement	3.15	N/A	400 (237)
Micro-Silica	2.20	N/A	30 (18)
Water	1	N/A	252 (149)

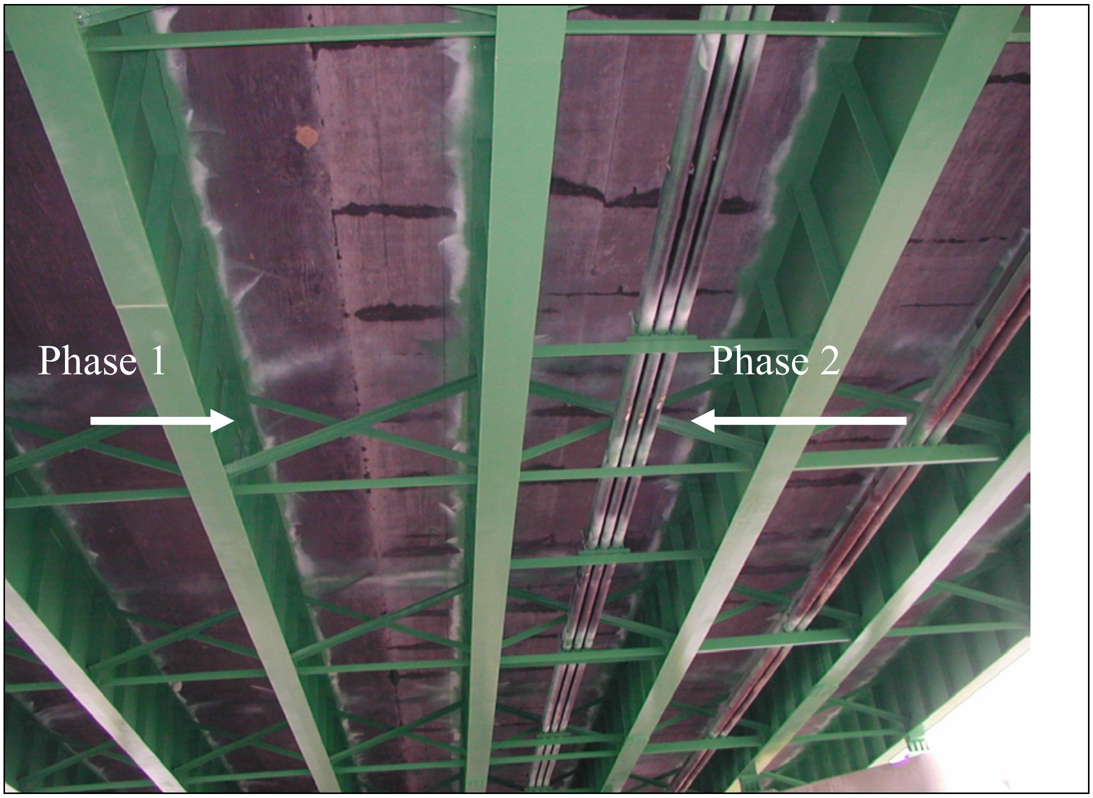


Figure 4: Emery Road over I-271 (Crowl and Sutak 2002)

Table 7: Phase 2 Mixture Design Emery Road Over I-271

High Performance Mix #4 Modified		**w/c = 0.42 % Air = 7.0**	
Material	Specific Gravity	Absorption Capacity	Lbs/CY (kg/m^3)
GGBF Slag	2.92	N/A	170 (101)
Natural Sand	2.62	1.16	1275 (756)
Number 8 Stone	2.72	1.38	370 (219)
Number 57 Stone	2.80	0.39	1390 (825)
Type 1 Cement	3.15	N/A	400 (237)
Micro-Silica	2.20	N/A	30 (18)
Water	1	N/A	252 (149)

Conclusions from the Crowl and Sutak Report

A review of prior years of construction indicated that the absorption of the course aggregates for bridges with transverse cracking was generally below one percent. In bridge decks where one phase cracked while the other phase did not, a change in the mixture design and the coarse aggregate used showed a coarse aggregate absorption value greater than one percent in the phase without cracking, and below one percent in the phase with cracking (Crowl and Sutak, 2002). The report determined that there was a correlation between aggregate absorption capacity and cracking tendency.

The report on the performance of 116 bridges to documented a number of interesting findings (Crowl and Sutak, 2002):

- All bridge decks used high strength concrete (ODOT HP specification)
- Two bridge decks without transverse cracks used coarse aggregate with an absorption of 1.39 or 1.52 percent
- Four bridge decks with substantial transverse cracking used coarse aggregate with an absorption of 0.41 percent
- In a verification survey of 116 bridge decks, all 64 with minimal or no cracking used coarse aggregate with absorption > 1 %. 75 % of the 52 bridges with severe cracking used coarse aggregate with absorption < 1 %.
- On the basis of these findings, District 12 developed a Modified High Performance Concrete Note to require the use of coarse aggregate with absorption > 1 %

The factors considered included mix design, weather conditions, the contractor's workmanship, curing, structure type, and method of construction. The investigation concluded that "by far the most consistent difference between the two groups is in the value for percent absorption in the coarse aggregate used in the HPC mixes" (Crowl and Sutak 2002*,* p. 14).

Although the current specification for HP3 and HP4 concrete uses # 8 coarse aggregate (3/8 inch or 9.5 mm maximum size), several of the mixtures in the study used a blend of # 8 and # 57 coarse aggregate (1 inch or 25 mm maximum size). Similar cracking trends were observed with both sizes of coarse aggregate.

ODOT District 12 also had a number of concrete mixtures tested by Master Builders, Inc. (now BASF admixtures) in Cleveland in order to evaluate permeability and shrinkage. They suggested a high performance concrete mixture using grade # 57 coarse aggregate blended with # 8, as opposed to # 8 alone, and using ground granulated blast furnace slag (GGBFS). Once ODOT District 12 observed extensive bridge deck cracking, four changes were made to the bridge deck concrete. The changes were:

- Use of a blend of # 8 and # 57 aggregates rather than only #8
- Limits on the absorption capacity of the coarse aggregate to ≥ 1 %
- A slight increase in the w/cm ratio from 0.40 to 0.42
- A decrease in total cementitious materials of approximately 10 %

The research program attempted to isolate the effects of these changes on cracking tendency. Therefore, HP3 Blend 2 was also tested, which only changed the aggregate gradation and kept the w/cm ratio and total cementitious materials the same as the ODOT HP3 and HP4 specification. For purposes of this research, low absorption was defined as less than 1 %, medium as 1 – 2 %, and high as greater than 2 %. All coarse aggregates used in these tests were crushed limestone.

A study sponsored by the Kansas DOT found that crack density of bridges built during the 1990s was greater than that of those built during the 1980s. They attributed the increase to changes in material properties and construction practices over the past 20 years (Lindquist et al. 2006).

Results of Survey of Ohio Department of Transportation District Practices

Researchers at Cleveland State University surveyed representatives from all twelve districts within the state's transportation network. The purpose of the survey was to review each district's practices related to HPC and other classes of concrete. The survey was carried out by telephone and by electronic mail. The survey questions were:

1. Have you experienced bridge deck cracking with the HP3 and HP4 mixtures? Have you used other HP mixtures that have been prone or resistant to cracking?
2. Have you encountered cracking problems with class S structural concrete?
3. Have you encountered cracking problems with class C pavement concrete?
4. Have you used Supplemental Specification 848 for bridge deck overlays? If so, have there been any cracking problems? Micro-silica, latex modified, or superplasticized?
5. What aggregates do you use most for bridge deck/structural concrete? Limestone or gravel, or other? Source?
6. Have you used any HP mixtures with gravels?

The twelve district representatives provided insight into specific trends within each district. Researchers investigated whether each district experienced bridge deck cracking when using these two particular mixtures. District representative responses indicated that seventy-five percent of the districts have experienced cracking. Thirty-three percent of the districts have used HP with modifications. Table 8 shows district-by-district usage of HP #3 and HP #4 and its susceptibility to cracking.

Table 8: Bridge Deck Cracking Reported

District	Yes	No
1	X	
2	X	
3	X	
4	X	
5	X	
6		X
7	X	
8		X
9		X
10	X	
11	X	
12	X	

Seventy five percent (9 of 12) districts reported bridge deck cracking. This does not imply that the percentage of bridge decks cracked in Ohio approaches 75 %, only that cracking has occurred nearly throughout the state at some level.

All districts used limestone aggregate for bridge decks. Limestone is the most commonly used aggregate in the state of Ohio because of its availability. Limestone is easily shipped from Canadian quarries across the Great Lakes to Ohio. However, seventeen percent have used gravels along with the limestone as their primary aggregate. Many of these districts have resorted to using limestone with absorption capacities greater than one percent to avoid cracking. This is a direct result of District 12's findings.

In the state of Ohio, research has shown that Class C concrete pavement has seen cracking in recent years. District responses to this cracking problem were most often a result of mid slab cracking, however, this may not be due to the concrete shrinkage problems. Thirty-three percent of the districts surveyed reported a problem with mid slab cracking. Mid slab cracking may occur if the joints of pavements are too far apart.

The survey questioned the districts about cracking problems regarding Class S structural concrete. Twenty-five percent of the state's districts reported cracking problems with Class S concrete. Some of their responses indicated that those districts are using high performance concrete for substructures instead of Class S concrete.

The problem addressed in District 12's report was only noted in bridge decks, but could also possibly be observed in pavements and structural concrete. Bridge deck cracks are easy to see from the underside after the concrete is placed and cured. On the other hand, pavements and substructures cannot be examined as readily. The cracks that occur in bridge decks may also be present in pavements and structures, given the fact that they use the same aggregate sources and are prone to similar environmental factors.

There are many different ways to specify Class C concrete under Ohio Department of Transportation guidelines. Districts most frequently use Class C concrete with options one and three. Options one and three contain fly ash or ground granulated blast furnace slag. Another alternative is Class C option two. Option two contains lower cement content and a water-reducing admixture. The options for Class C concrete choice allow for districts to use readily available materials in pavements.

EXPERIMENTAL DESIGN

The experimental research was divided into four phases: concrete mixtures using traditional ODOT materials and mixture designs, concrete mixtures using high absorption fine lightweight aggregate, concrete mixtures using coarse aggregate with a larger nominal size in a blended mixture, and field testing.

Limestone is commonly used for high performance concrete by ODOT. Therefore, most of the HP mixtures tested used limestone coarse aggregate. Some of the HP mixtures were also tested with gravels. For purposes of this research, low absorption was defined as less than 1 %, medium as 1 – 2 %, and high as greater than 2 %.

All concrete mixing, with the exception of the field testing, was performed in the Concrete Research Laboratory in the Fenn College of Engineering at Cleveland State University. The field samples were obtained from the respective mixture sites and transported to Cleveland State University, where specimens were prepared.

For each concrete mixture, the following tests were performed, as indicated in Table 9:

- Fresh concrete properties – slump, air, and unit weight
- Hardened concrete properties – compressive, flexural, and splitting tensile strength, performed at 7 and 28 days, plus 56 and 90 days for high performance concrete. Modulus of elasticity was also measured.
- Unrestrained shrinkage (bar) tests, sealed and unsealed – measurements taken up to 90 days
- Restrained shrinkage/cracking tendency (ring) tests – measurements taken up to 90 days

Table 9: Test Program

Test	Protocols and methods	Number of tests
Fresh Concrete Properties		
Workability	ASTM C 143	1
Air content	ASTM C 231	1
Unit weight	ASTM C 138	1
Hardened Concrete Properties		
Compressive strength	ASTM C 39, 4 x 8 in cylinders	2 each age, total 4
Flexural strength	ASTM C 78, 6 x 6 x 20 in beams	1 each age, total 2
Splitting tensile strength	ASTM C 496, 4 x 8 in cylinders	2 each age, total 4
Modulus of elasticity	ASTM C 496, 4 x 8 in cylinders	2 each age, total 4
Shrinkage and Cracking Tendency		
Unrestrained, unsealed shrinkage	ASTM C 157	2
Unrestrained, sealed shrinkage	ASTM C 1090	2
Restrained shrinkage ring tests	AASHTO PP34-99	2

Concrete Mixture Designs

The Ohio Department of Transportation (ODOT) currently uses five high performance concrete mixture designs. These are designated HP #1, HP #2, HP #3, HP #4 and HP #4 Modified. Typically HP #1 and HP #2 are only used in the substructures of bridges. HP #3, HP

#4 and HP #4 Modified are used in the superstructure of the bridges and were the focus of this research. HP #4 Modified was developed and is used by ODOT District 12.

ODOT uses four Class C concrete mixture designs designated Class C, Class C Option 1, Class C Option 2 and Class C Option 3. ODOT also provides four Class S concrete mixtures designated Class S, Class S Option 1, Class S Option 2 and Class S Option 3. Based on the interviews with the ODOT districts, Class S and Class C Option 1 were selected for testing. The coarse aggregate used for each was # 57. Table 10 through Table 16 provide the general mixture designs for this experimental program, in quantities per cubic yard and cubic meter.

Table 10: HP#3 Mixture Design

Aggregate Type	Fine Aggregate lb (kg)	Lightweight Fine Aggregate lb (kg)	#8 Coarse Aggregate lb (kg)	Cement Content lb (kg)	Class C Fly Ash lb (kg)	Microsilica lb (kg)	Maximum w/cm
Gravel	1340 (795)	-	1460 (866)	480 (285)	150 (89)	30 (18)	0.40
Limestone	1350 (801)	-	1480 (878)	480 (285)	150 (89)	30 (18)	0.40
Limestone w/LWA	1012 (600)	227 (135)	1480 (878)	480 (285)	150 (89)	30 (18)	0.40

Table 11: HP#4 Mixture Design

Aggregate Type	Fine Aggregate lb (kg)	Lightweight Fine Aggregate lb (kg)	#8 Coarse Aggregate lb (kg)	Cement Content lb (kg)	GGBF Slag lb (kg)	Microsilica lb (kg)	Maximum w/cm
Gravel	1370 (813)	-	1475 (875)	440 (261)	190 (113)	30 (18)	0.40
Limestone	1370 (813)	-	1490 (884)	440 (261)	190 (113)	30 (18)	0.40
Limestone w/LWA	1041 (618)	227 (135)	1490 (884)	440 (261)	190 (113)	30 (18)	0.40

Table 12: ODOT HP #4 Modified Mixture Design

Aggregate Type	Fine Aggregate lb (kg)	Lightweight Fine Aggregate lb (kg)	#8 Coarse Aggregate lb (kg)	#57 Coarse Aggregate lb (kg)	Cement Content lb (kg)	GGBF Slag lb (kg)	Microsilica lb (kg)	Maximum w/cm
Gravel	1245 (739)	-	360 (214)	1335 (792)	400 (237)	170 (101)	30 (18)	0.42
Limestone	1245 (739)	-	360 (214)	1335 (792)	400 (237)	170 (101)	30 (18)	0.42
Limestone w/LWA	946 (561)	206 (122)	360 (214)	1335 (792)	400 (237)	170 (101)	30 (18)	0.42

Table 13: HP #3 Blended Mixture Design

Aggregate Type	Fine Aggregate lb (kg)	Lightweight Fine Aggregate lb (kg)	#8 Coarse Aggregate lb (kg)	#57 Coarse Aggregate lb (kg)	Cement Content lb (kg)	Class C Fly Ash lb (kg)	Microsilica lb (kg)	Maximum w/cm
Gravel	1350 (801)	-	297 (176)	1184 (702)	480 (285)	150 (89)	30 (18)	0.40
Limestone	1350 (801)	-	297 (176)	1184 (702)	480 (285)	150 (89)	30 (18)	0.40
Limestone w/LWA	1012 (600)	227 (135)	297 (176)	1184 (702)	480 (285)	150 (89)	30 (18)	0.40

Table 14: HP #4 Blended Mixture Design

Aggregate Type	Fine Aggregate lb (kg)	Lightweight Fine Aggregate lb (kg)	#8 Coarse Aggregate lb (kg)	#57 Coarse Aggregate lb (kg)	Cement Content lb (kg)	GGBF Slag lb (kg)	Microsilica lb (kg)	Maximum w/cm
Gravel	1370 (813)	-	299 (177)	1192 (707)	440 (261)	190 (113)	30 (18)	0.40
Limestone	1370 (813)	-	299 (177)	1192 (707)	440 (261)	190 (113)	30 (18)	0.40
Limestone w/LWA	1041 (618)	227 (135)	299 (177)	1192 (707)	440 (261)	190 (113)	30 (18)	0.40

Table 15: ODOT Class C Option 1 Mixture Design

Aggregate Type	Fine Aggregate lb (kg)	Lightweight Fine Aggregate lb (kg)	#57 Coarse Aggregate lb (kg)	Cement Content lb (kg)	Class F Fly Ash lb (kg)	Maximum w/cm
Gravel	1140 (676)	-	1700 (1009)	510 (303)	90 (53)	0.50
Limestone	1260 (748)	-	1595 (946)	510 (303)	90 (53)	0.50
Limestone w/LWA	961 (570)	206 (121)	1595 (946)	510 (303)	90 (53)	0.50

Table 16: ODOT Class S Mixture Design

Aggregate Type	Fine Aggregate lb (kg)	#57 Coarse Aggregate lb (kg)	Cement Content lb (kg)	Maximum w/cm
Gravel	1070 (635)	1660 (985)	715 (424)	0.44
Limestone	1240 (736)	1510 (896)	715 (424)	0.44

Some testing also investigated overlay concrete with silica fume, because thin overlays are often susceptible to cracking. Proportions for the overlay concrete are shown in Table 17. The overlay concrete also includes one pound per cubic yard (0.593 kg/m^3) of 1 ¼ inch (32 mm) long polypropylene fibers.

Table 17: Silica Fume Overlay Concrete Mixture Design

Aggregate	Fine Aggregate	Coarse Aggregate	Cement	Silica fume	W:CM
Type	Weight lb (kg)	Weight lb (kg)	Weight lb (kg)	Weight lb (kg)	Ratio Max
Limestone	1410 (836)	1450 (860)	600 (356)	50 (30)	.40
Gravel	1410 (836)	1450 (860)	600 (356)	50 (30)	.40

Concrete Mixing and Sample Preparation

In addition to the strength testing discussed below, all mixtures were tested for fresh properties to ensure that they complied with the ODOT specification. The fresh concrete properties of a mixture can be important and were recorded for each mixture.

The slump was tested following ASTM C143: Slump of Hydraulic-Cement Concrete. ODOT specifies a maximum slump of 4 inches (100 mm) for Class C and Class S concrete and a maximum slump of 8 inches (200 mm) for high performance mixtures (ODOT 2005). The unit weight of each mixture was also tested following ASTM C138: Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete.

The air content of the fresh concrete was the most important fresh concrete test performed and was tested following ASTM C 231: Air Content of Freshly Mixed Concrete by the Pressure Method. ODOT specifies an air content of 6 ± 2%for Class C Option 1 and Class S mixtures and an air content of 7 ± 2%for high performance mixtures. Test methods are shown in Table 9. Results are reported in Appendix A.

The cylinders and two of the concrete prisms were cured in a lime bath following ASTM C 511. The restrained shrinkage rings were kept in a temperature controlled environment at 73.5 ± 3.5^{o} F (23 ± 2.0^{o} C) and 50 ± 5 % relative humidity. The other two concrete prisms were kept with the ring specimens at 50% relative humidity.

The experimental program consisted of the following tests: ASTM C1581: Determining Age at Cracking and Induced Tensile Stress Characteristics of Mortar and Concrete under Restrained Shrinkage, ASTM C157: Length Change of Hardened Hydraulic-Cement Mortar and Concrete, ASTM C39: Compressive Strength of Cylindrical Concrete Specimens, ASTM C496: Splitting Tensile Strength of Cylindrical Concrete Specimens, ASTM C469: Static Modulus of Elasticity of Concrete in Compression and ASTM C78: Flexural Strength of Concrete. Test methods are shown in Table 9.

Strength Testing

Specimens were tested for compressive, splitting tensile, and flexural strength at various ages.

Compressive Strength

The concrete compressive strength was tested at 7, 28, 56, and 90 days following ASTM C 39. The concrete specimens were 4 inch diameter by 8 inch length (100 x 200 mm) cylinders and were cured inside plastic molds for the first 24 hours. After 24 hours the cylindrical specimens were removed from the molds and placed into a lime bath kept at 73.5 ± 3.5^{o} F (23 ± 2.0^{o} C). The concrete specimens were kept in the lime bath until the time of testing. Specimens were molded following ASTM C192 and stored in a moist cure tank following ASTM C511. Three specimens were tested at each age following ASTM C39 on a hydraulic loading machine.

The compressive strength (f'_c) was determined by dividing the ultimate applied load by the cross-sectional area of the cylinder. The compressive strength and the type of fracture of each cylinder were recorded, and the average compressive strength was reported.

Splitting Tensile Strength

The splitting tensile strength test was performed to indirectly determine the tensile strength of 4 in by 8 in (100 x 200 mm) concrete cylinders following ASTM C 496. The curing specification of the cylinders was identical to that of the specimens in compression. The splitting tensile test was performed on concrete at 7, 28, 56, and 90 days.

The specimens were tested on a hydraulic loading machine following ASTM C496. The splitting tensile strength was determined from Equation 4 (ASTM C496):

Equation 4: Splitting Tensile Strength

$$T = 2P/\pi ld$$

Where:

T = splitting tensile strength (psi) (kPa)
P = maximum applied load (lbf) (kN)
l = length (in) (m)
d = diameter (in) (m)

For this research, the length was equal to 8 inches (200 mm) and the diameter was equal to 4 inches (100 mm). Therefore, the splitting tensile strength was equal to the maximum applied load divided by 50.2655 in U.S. units (64.4 in SI). For each age, one specimen was tested for splitting tensile strength and recorded.

Flexural Strength/ Modulus of Rupture

Flexural strength (modulus of rupture) was tested using 6 x 6 x 20 inch (152 x 152 x 508 mm) beams, following ASTM C 78 for third point (four point) loading. Specimens were tested at 7, 28, and 56 days. Some mixtures also had specimens tested at 1 and 3 days. Five specimens were molded following ASTM C192. Specimens were stored in a water tank following ASTM C511. Beams were tested following ASTM C78 at one day in third point loading in a portable beam tester and at all subsequent times on a hydraulic press. The modulus of rupture (R) was calculated from Equation 5 (ASTM C78):

Equation 5: Modulus of Rupture

$$R = PL/bd^2$$

Where:

R = modulus of rupture (psi) (MPa)
P = maximum applied load (lbf) (N)
L = span length (in) (mm)
b = average width of specimen at fracture (in) (mm)
d = average depth of specimen at fracture (in) (mm)

In our research the span length was equal to 18 inches (457 mm) and the width and depth were both 6 inches (152 mm). For this set up the modulus of rupture was simply equal to the total applied load divided by twelve in U.S. units (1.29×10^{-4} in SI).

Modulus of Elasticity

The static modulus of elasticity was measured according ASTM C 469. An aluminum yoke was used to connect the dial gage to the cylinder specimen. One cylinder was tested at each interval of 7, 28, 56, and 90 days.

The modulus was determined using one specimen following ASTM C 469. The modulus was calculated using Equation 6 (ASTM C 649):

Equation 6: Secant Modulus of Elasticity

$$E = (S_2 - S_1)/(\varepsilon_2 - 0.000050)$$

Where:

E = secant modulus of elasticity (psi) (MPa)
S_2 = stress corresponding to 40% of the ultimate load (psi) (MPa)
S_1 = stress corresponding to a longitudinal strain, ε_1, of 50 millionths (psi) (MPa)
ε_2 = longitudinal strain produced by stress S_2

Shrinkage

The following shrinkage and deformation properties of concrete were measured:

- Unrestrained shrinkage of unsealed specimens, subject to both drying and autogenous shrinkage,
- Unrestrained shrinkage of sealed specimens, subject to autogenous shrinkage only, and
- Restrained shrinkage of ring specimens

Unrestrained Shrinkage of Unsealed Specimens

Unrestrained shrinkage tests were performed on unsealed 3 by 3 by 10 inch (76.2 by 76.2 by 254 mm) cast specimens using ASTM C 157. The concrete specimens had a 5/8 inch (16 mm) stud cast into each end, so the gauge length measured was 11 ¼ inches (286 mm). The concrete prism mold is a 3 in by 3 in by 10 in (76.2 by 76.2 by 254 mm) cast steel mold. Specimens were made and covered in plastic for 24 hours.

After removing the specimens from the molds, two of the specimens were coated with paraffin wax on all four sides while the other two specimens were left unsealed. Measurements were taken at 1, 7, 28, 56, and 90 days. Two sets of specimens were made. One set was kept in the water baths, and the other set was kept at 50 % humidity in the room with the restrained ring specimens. The concrete prisms in the lime bath were kept at $73.5 \pm 3.5\,^{o}$F ($23 \pm 2\,^{o}$C).

The change in length was determined from Equation 7 (ASTM C157):

Equation 7: Percent Shrinkage

$$\Delta L_x = (CRD - \text{initial } CRD)/G \times 100$$

Where:

ΔL_x = length change of specimen (%)

CRD = difference between the comparator reading of the specimen and the reference bar

G = gage length (10 in) (254 mm)

The length comparator readings were recorded and the percent length change was reported at the specified times for each mixture.

Unrestrained Shrinkage of Sealed Specimens

ASTM C 1090 (2001), height change of protected cylinders to isolate autogenous shrinkage from drying shrinkage and other effects, is written for 3 by 6 inch (76.2 x 152 mm) cylinders. The specimens are sealed to prevent moisture loss or gain, and thus drying shrinkage is not a factor and any length change is due to autogenous volume change only. This test may be modified to use the same apparatus as for unsealed unrestrained shrinkage. For this research, the specimens were sealed with paraffin and tested in the same manner as the unsealed specimens, as discussed above.

Restrained Shrinkage (Ring Specimens)

A number of tests are discussed in the literature for restrained shrinkage. These include ring tests, uniaxial tests on specimens with either two anchored ends or one anchored and one free movable ends, restrained plate specimen tests, and bottom-restrained linear specimens. The ring test seems to have found the widest acceptance.

The restrained shrinkage of ring specimens has been standardized by AASHTO and ASTM and is explained in Voigt et al. (2004). This test evaluates concrete mixtures against two performance criteria:

- Age of concrete at detection of first crack
- Maximum width of crack after certain curing time

The ring test is shown in Figure 5 and is conducted as follows: "For the casting of the specimens, a tubular cardboard mold was placed next to the steel ring, forming a gap for placing the fresh concrete. The cardboard ring was removed 24 h after casting and the top surface of the concrete ring was sealed with silicone rubber. The prepared specimens… were then placed on turntables in an environmental chamber to ensure controlled and repeatable curing conditions at a temperature of 22 °C and a relative humidity of 50 %. The rings were checked every 24 h for visible cracks and the age of occurrence of each crack was recorded. The width of each crack was measured using a microscope at 42 days after casting…" (pp. 234 – 235, Voigt et al. 2004).

The restrained shrinkage ring rested on a 24 inch by 24 inch, 5/8 inch thick (0.6 x 0.6 x 0.016 m) plywood sheet coated with an epoxy to prevent bonding. The wood was then covered with Mylar plastic to keep the concrete from bonding to the base. A 13 inch (330 mm) outside diameter steel tube acted as an internal form as well as a restraint. The steel tube had a thickness of ½ (13 mm). Two independent strain gages were mounted 180^o apart on the inside of the steel tube (Type CEA 06 125 UW 120 ohms from Vishay Intertechnology Inc.). The gages measure the strain of the steel due to the shrinkage of the concrete poured around the outside of the steel. The steel rings were 6 inches (152 mm) high. The outer form for the concrete ring was a ¼ inch thick (6.4 mm) cardboard form tube (sonotube) with an inside diameter of either 16 or 18 inches

(406 or 457 mm). The 16 inch (406 mm) outer form conformed to ASTM C 1581, but could only be used for concrete mixes with maximum nominal size coarse aggregates less then ½ inch (13 mm). A modification had to be made for HP#4 blended mixes because of the addition of #57 coarse aggregate. The restrained shrinkage setup is illustrated in Table 18 and Table 19 and Figure 5.

Figure 5: Instrumented Ring Test

The concrete was cast into the form as shown in Table 18 and Table 19. The specimens were moist cured with wet burlap and plastic for 24 hours. After 24 hours, the plastic and burlap were removed. A thin layer of paraffin wax was then placed on the top exposed surface. The outer forms were removed and the specimens were kept in a temperature controlled environment at 73.5 ± 3.5^{o} F ($23 \pm 2\ ^{o}$C) and 50 ± 5 % relative humidity. Strain measurements were recorded as soon as specimens were poured. All restraints on the internal steel ring were removed after the concrete was placed. A data logger system was used to record the strain. Measurements were taken at ½ hour intervals. The test data concluded at 90 days or when the specimen cracked, if prior to 90 days.

Table 18: Dimension for HP#3 and HP#4

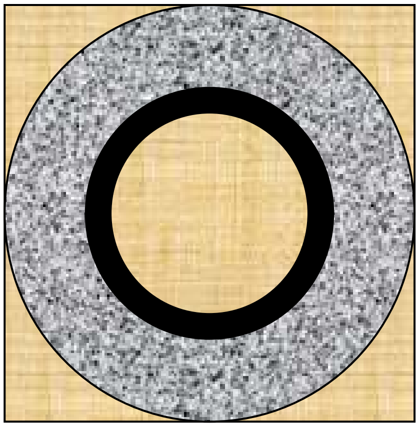
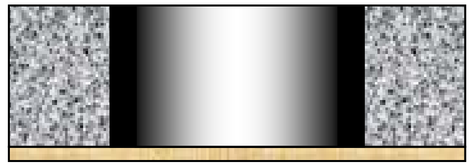
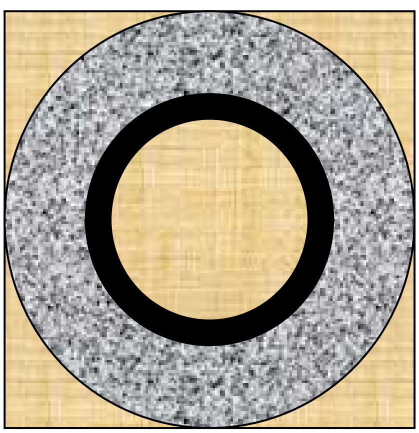
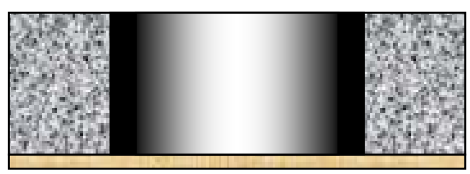
Plan view	Side view
	
Dimensions	**inches (mm)**
Specimen outer diameter	16 (406)
Steel ring outer diameter	13 (330)
Steel ring inner diameter	12 (305)
Specimen height	6 (152)

Table 19: Dimensions for HP#4 blended, Class S, and Class C

Plan view	Side view
	
Dimensions	**inches (mm)**
Specimen outer diameter	18 (457)
Steel ring outer diameter	13 (330)
Steel ring inner diameter	12 (305)
Specimen height	6 (152)

MATERIALS

This chapter discusses the materials used to produce the concrete tested. All materials were on the ODOT approved list except the lightweight aggregate, which is not commonly used at present by the agency.

Cementitious Materials

Cementitious materials used in this research included Type I cement, ground granulated blast furnace slag (GGBFS), fly ash, and silica fume (also called microsilica).

Cement

The cement used for this experiment was Type I Portland cement from St. Marys Cement in Ontario. The cement meets ASTM C150. The cement composition and physical properties of the cement are shown in Table 20. These results were provided by St. Marys.

Table 20: Cement properties

Typical Oxide Composition	St. Marys Type I cement (%)
CaO	61.81
SiO_2	20.45
Al_2O_3	4.55
Fe_2O_3	2.77
MgO	3.15
SO_3	3.76
Total Alkali as Na_2O	0.65
Physical Properties	**St. Marys Type I cement**
Fineness: Blaine (m^2/kg)	381
Autoclave Expansion (%)	0.379
Percent Retained in #325 sieve	4.0

Ground Granulated Blast Furnace Slag

The Ground Granulated Blast Furnace (GGBF) slag used for this experiment was CemPlus from St. Marys Cement in Ontario. The GGBF slag meets ASTM C 989 Grade 100 for Type S GGBF slag. The GGBF slag composition and physical properties are shown in Table 21. These results were provided by St. Marys.

Fly Ash

The fly ash used for this study was class C from ISG. The source of the class C fly ash was the First Energy plant in Ashtabula, Ohio. This fly ash meets both ASTM C 311 and C 114.

Silica Fume

The silica fume used for this study was Axim Catexol S.F.-D of Middlebranch, Ohio. Axim meets ASTM C 1240 for silica fume used in concrete and mortar.

Table 21: GGBF Slag Properties

Typical Oxide Composition	St. Marys GGBF Slag (%)
Sulphide	0.59
SO_3	1.49
Total Alkali as Na_2O	0..88
Physical Properties	**St. Marys GGBF Slag**
Fineness: Blaine (m^2/kg)	524
Autoclave Expansion (%)	0.1
Percent Retained in #325 sieve	0.6

Normal Weight Coarse Aggregates

The normal weight aggregates used in the concrete mixes varied. Limestone aggregates with three different absorption levels were tested. Both #8 and #57 coarse aggregate were sampled for the test program. #8 coarse aggregate has a maximum nominal size of 3/8 inch (9.5 mm). #57 coarse aggregate has a maximum nominal size of 1 inch (25 mm). Natural river gravel coarse aggregate was also used for part of this testing.

Limestone

Three different crushed limestone aggregates were used in this research. The limestone aggregates tested are frequently used in ODOT District 12 and are on the ODOT list of approved aggregates. They are representative of similar materials used statewide.

The low absorption aggregate was Cedarville from Michigan Limestone. Cedarville has an absorption of 0.35% for the #57 aggregate and 0.70% for the #8 aggregate. Rogers City limestone, also from Michigan Limestone, was the medium absorption aggregate. Rogers City, commonly referred to as Calcite, has an absorption of 1.38% for the #57 aggregate and 1.74% for the #8 aggregate. The Cedarville and Calcite aggregates were supplied by the Ontario Stone Corporation. The high absorption aggregate used for this research was Marblehead limestone from Lafarge Corporation. This aggregate has an absorption of 3.03% for the #57 aggregate and 3.77% for the #8 aggregate and was supplied by Osborn Concrete and Stone.

Each mixture contained aggregates from one source, because concrete suppliers do not typically mix coarse aggregate suppliers. The sieve analyses for these aggregates are provided in Appendix B. Table 22 lists the absorption, bulk dry density and SSD specific gravities for all coarse and fine aggregates used in this research. Limestones are shown with “LS.”

Natural River Gravel Coarse Aggregate

The natural river gravel coarse aggregate used for this research was supplied by Martin-Marietta Materials from their Fairborn facility. Fairborn gravels have an absorption level of 1.51% for the #57 aggregate and 1.73 % for the #8 aggregate. Both aggregates were considered medium absorption aggregates for the purpose of this research. It was determined from surveys conducted in each ODOT district that the use of natural gravels is not wide spread on ODOT projects, and only one type was used in this research. This aggregate meets the ODOT specifications and is on the ODOT approved list of aggregates. Both #57 and #8 coarse aggregate was used in this research. The sieve analysis for this aggregate is provided in Appendix B. Gravels are shown in Table 22 with the designation “GR.”

Table 22: Aggregate absorptions and densities

Aggregate Designation	Size of Aggregate	Absorption (%)	Bulk Specific Gravity lb/ft^3 (kg/m^3)	SSD Specific Gravity lb/ft^3 (kg/m^3)
Cedarville LS	#8	.7	2.732 (43.762)	2.752 (44.083)
Cedarville LS	#57	.35	2.795 (44.772)	2.805 (44.932)
Rogers City LS	#8	1.74	2.515 (40.286)	2.559 (40.991)
Rogers City LS	#57	1.38	2.529 (40.511)	2.565 (41.087)
Marblehead LS	#8	3.77	2.471 (39.582)	2.565 (41.087)
Marblehead LS	#57	3.03	2.493 (39.934)	2.569 (41.151)
Fairborn GR	#8	1.73	2.625 (42.048)	2.671 (42.785)
Fairborn GR	#57	1.51	2.645 (42.369)	2.686 (43.026)
Shalersville sand	Natural Sand	1.12	2.576 (41.264)	2.605 (41.728)
Hydrocure 500	Light Weight Fine Aggregate	21.2	-	1.800 (28.833)

Normal Weight Fine Aggregate

The fine aggregate used in this research was Shalersville natural sand from Lafarge Corporation. This aggregate has an absorption of 1.12 % and was supplied by Osborn Concrete and Stone. Shalersville is frequently used for ODOT projects in District 12.

Structural Lightweight Fine Aggregate

The structural lightweight aggregate used in this research was HydroCure 500 supplied by Northeast Solite. HydroCure 500 is a lightweight aggregate with a high absorption capacity (21.2%) and is similar to natural fine aggregate in shape and gradation. Sieve analysis for both the natural fine aggregate and HydroCure 500 can be found in Appendix B. HydroCure 500 is manufactured from natural materials. The material is expanded through heating in a kiln. This expansion greatly increases the porosity of the material, leading to its light weight and high absorption capacity.

Chemical Admixtures

Two chemical admixtures were used in this research – a high range water reducer and an air entraining admixture. Both admixtures were supplied by BASF/Master Builders. The high range water reducer was Glenium 3030 NS. Glenium meets ASTM C 494 requirements for Type A, water-reducing, and Type F, high-range water-reducing, admixtures. The air entrainment admixture was MB-AE 90. MB-AE 90 meets the requirements of ASTM C 260. The dosages used for each mixture can be found in the concrete mix design worksheets (Appendix A).

EXPERIMENTAL RESULTS

The results shown in this chapter represent the average of three specimens from the same mixture design and curing conditions for compressive strength and one specimen each for splitting tensile strength, modulus of rupture, modulus of elasticity and shrinkage specimens.

Observations of Fresh Concrete

The air content, unit weight and workability of each concrete mixture was tested and recorded. The weight of the batch and amount and type of chemical admixtures was also recorded. A summary of all results is provided in Appendix A. All mixtures were within ODOT specifications for fresh concrete properties.

While mixing the concrete, no difficulties were encountered with using lightweight fine aggregate. The amount of LWA used in the mixtures was small enough that a traditional pressure air meter could be used for testing. The blended mixtures with #57 and #8 aggregate seemed to be easier to finish than the mixtures with only #8 aggregate.

Compressive Strength

The compressive strength of the concrete samples was determined following ASTM C 39. Cylinders were 4 by 8 inch (100 by 200 mm) and tested at 7, 28, 56 and 90 days. The specimens were removed from the molds 24 hours after initial set and cured in a water storage tank in accordance with ASTM C 192. The Ohio Department of Transportation specifics a 28 day compressive strength of 4,500 psi (31 MPa) for High Performance and Class S concrete and a 28 day compressive strength of 4,000 psi (28 MPa) for Class C concrete (ODOT Spec 2005). With the exception of Class C Option 1 Low Absorption all of the concrete samples exceeded the compressive strength requirements. Table 23 contains a summary of the average compressive strengths.

Figure 6 through Figure 10 depict the average compressive strengths for HP #3, HP #4 and HP #4 Modified. The graphs indicate that ODOT's minimum 28 day strength of 4,500 psi (31 MPa) is exceeded by all of the High Performance mixes at 7 days, with the exception of HP #4 Blended Gravels and HP #4 Modified Gravels. These mixtures met the ODOT specification by 28 days.

For HP #3 and HP #4, the low absorption stone has the lowest compressive strengths, followed by the medium, then the high. Also, when a replacement of the fine aggregate with Hydrocure 500 was made the compressive strength was increased after 28 days. The general trend observed in the preceding mixes did not translate to the blended High Performance mixes. In HP #3 Blended, HP #4 Blended and HP #4 Modified the medium absorption coarse aggregate had the highest strength, followed by the high absorption, then the low absorption coarse aggregate.

The results for the Class S mixtures are shown in Figure 11. The mixtures with the high and medium absorption aggregate met the ODOT 28 day specification at 7 days, and the mixture with the low absorption aggregate met the specification at 28 days.

As mentioned earlier, the 28 day compressive strength of Class C Option 1 with the low absorption stone did not meet the ODOT specification of 4,000 psi (28 MPa). However, when a partial replacement of the fine aggregate with Hydrocure 500 was made, the 28 day compressive strength was above the ODOT minimum specification. As can be seen in Figure 12, Class C Option 1 concrete made with all other aggregates met the specification.

Table 23: Compressive strength of concrete cylinders

Mix Identification	Compressive Strength							
	7 Day		28 Day		56 Day		90 Day	
	psi	MPa	psi	MPa	psi	MPa	psi	MPa
HP #3 High	6486	45	8293	57	8950	62	8848	61
HP #3 High Re-mix	5605	39	7854	54	7963	55	8148	56
HP #3 Medium	5906	41	7903	54	9244	64	8825	61
HP #3 Medium w/LWA	6353	44	7581	52	9084	63	-	-
HP #3 Low	5259	36	7562	52	7245	50	7465	51
HP #3 Low w/LWA	6272	43	8151	56	10041	69	10373	72
HP #3 Gravels	6127	42	8461	58	-	-	-	-
HP #3 Gravels w/LWA	4646	32	7193	50	-	-	-	-
HP #3 Blended High	6461	45	8487	59	8853	61	9082	63
HP #3 Blended Medium	7703	53	10023	69	10931	75	10570	73
HP #3 Blended Low	5749	40	7427	51	8421	58	8833	61
HP #4 High	6874	47	8052	56	9165	63	8733	60
HP #4 High Re-mix	6511	45	9568	66	9462	65	9439	65
HP #4 Medium	6275	43	8552	59	9453	65	9454	65
HP #4 Low	5880	41	8515	59	8976	62	8685	60
HP #4 Low w/ LWA	5972	41	8740	60	9617	66	10248	71
HP #4 Gravels	6585	45	8402	58	8502	59	9014	62
HP #4 Blended High	5863	40	7873	54	8619	59	8714	60
HP #4 Blended Medium	6290	43	8567	59	9324	64	9012	62
HP #4 Blended Low	4975	34	6641	46	7010	48	7540	52
HP #4 Blended Gravels	4204	29	5750	40	5770	40	5876	41
HP #4 Modified High	6385	44	8105	56	8379	58	8607	59
HP #4 Modified Medium	6197	43	8380	58	8797	61	9447	65
HP #4 Modified Medium w/LWA	6491	45	8395	58	8865	61	9512	66
HP #4 Modified Low	5090	35	7395	51	7381	51	7326	51
HP #4 Modified Low w/LWA	4987	34	8243	57	8763	60	8583	59
HP #4 Modified Gravels	3952	27	5557	38	5851	40	5909	41
Class S High	4554	31	5929	41	6539	45	6705	46
Class S Medium	5338	37	6532	45	7195	50	7216	50
Class S Low	3844	27	4815	33	5080	35	5119	35
Class C Option 1 High	3860	27	4630	32	4787	33	5271	36
Class C Option 1 Medium	3982	27	5075	35	5544	38	5963	41
Class C Option 1 Low	2807	19	3590	25	4190	29	-	-
Class C Option 1 Low w/LWA	3178	22	4184	29	4790	33	5425	37
Microsilica Overlay High	5188	36	6513	45	7090	49	7428	51
Microsilica Overlay High with Fibers	4806	33	6483	45	7360	51	7465	51
Microsilica Overlay High w/LWA	7331	51	9863	68	10375	72	9811	68

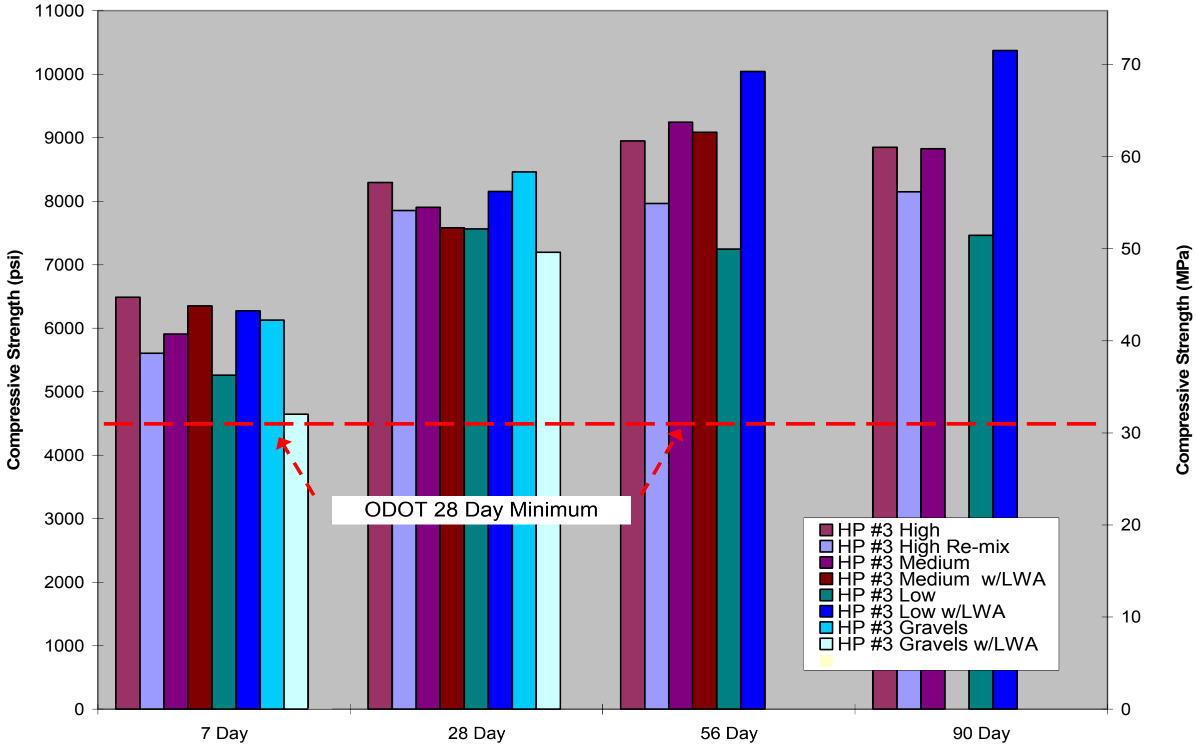


Figure 6: HP #3 Compressive Strengths

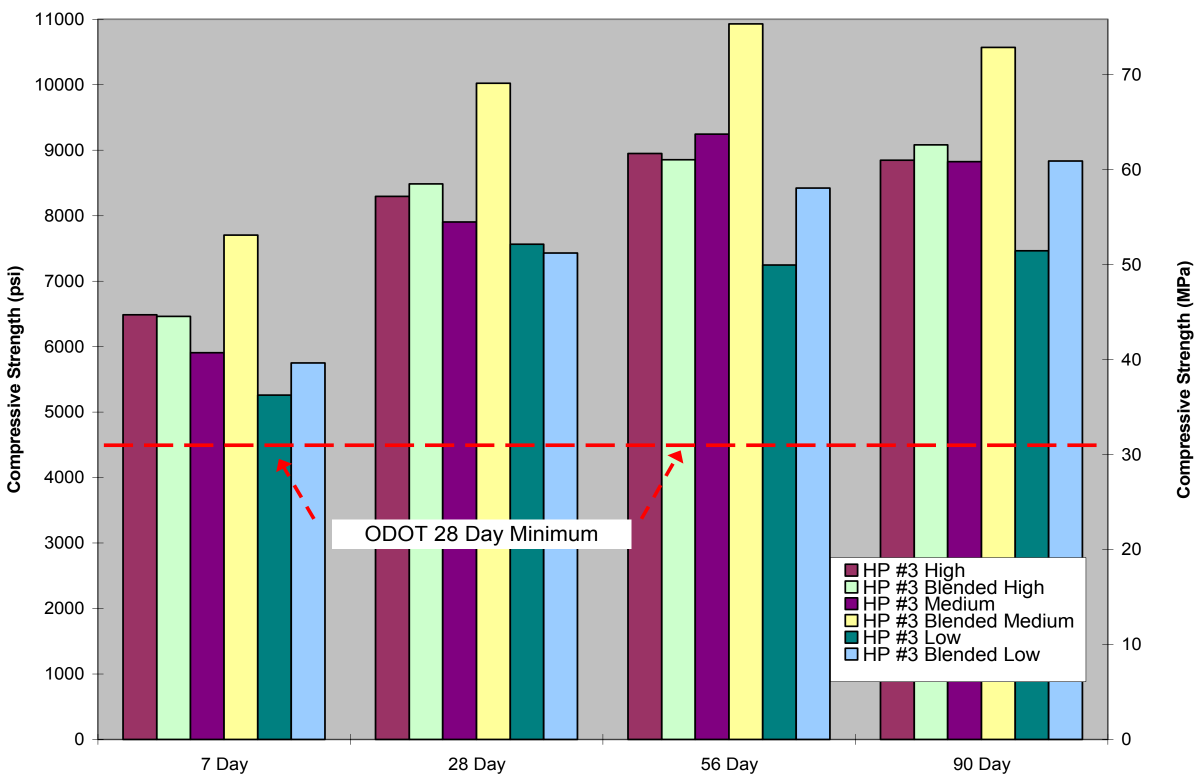


Figure 7: HP #3 Blended Compressive Strengths

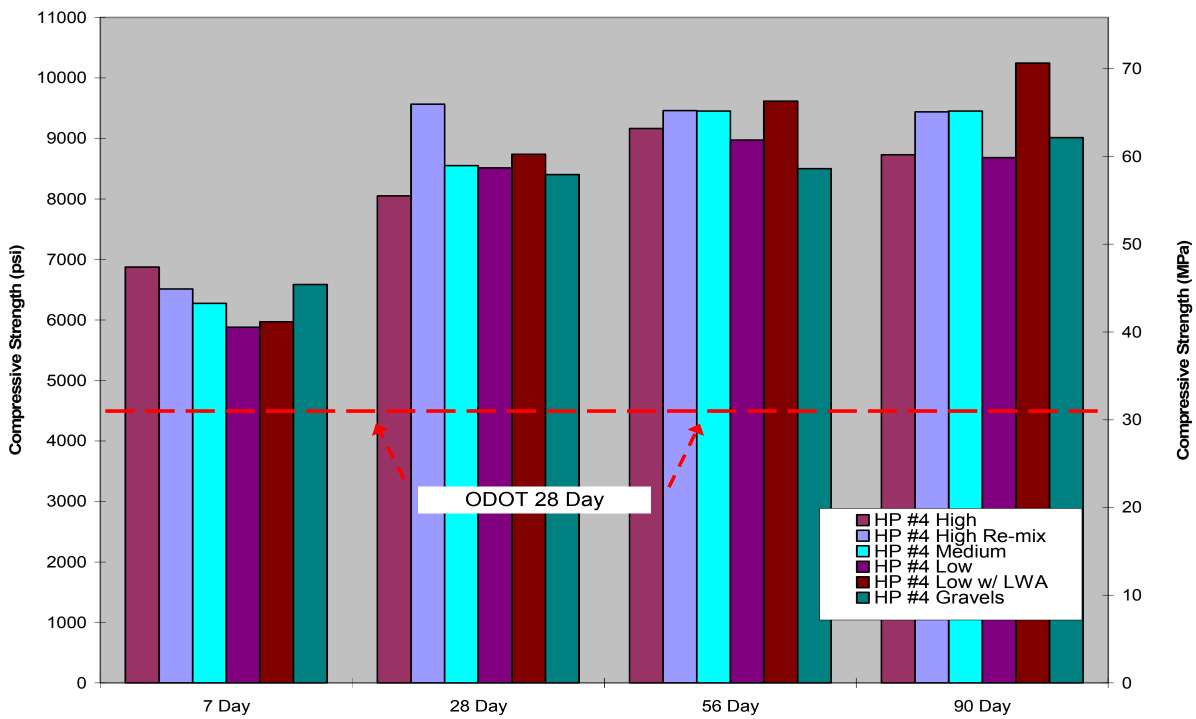


Figure 8: HP #4 Compressive Strengths

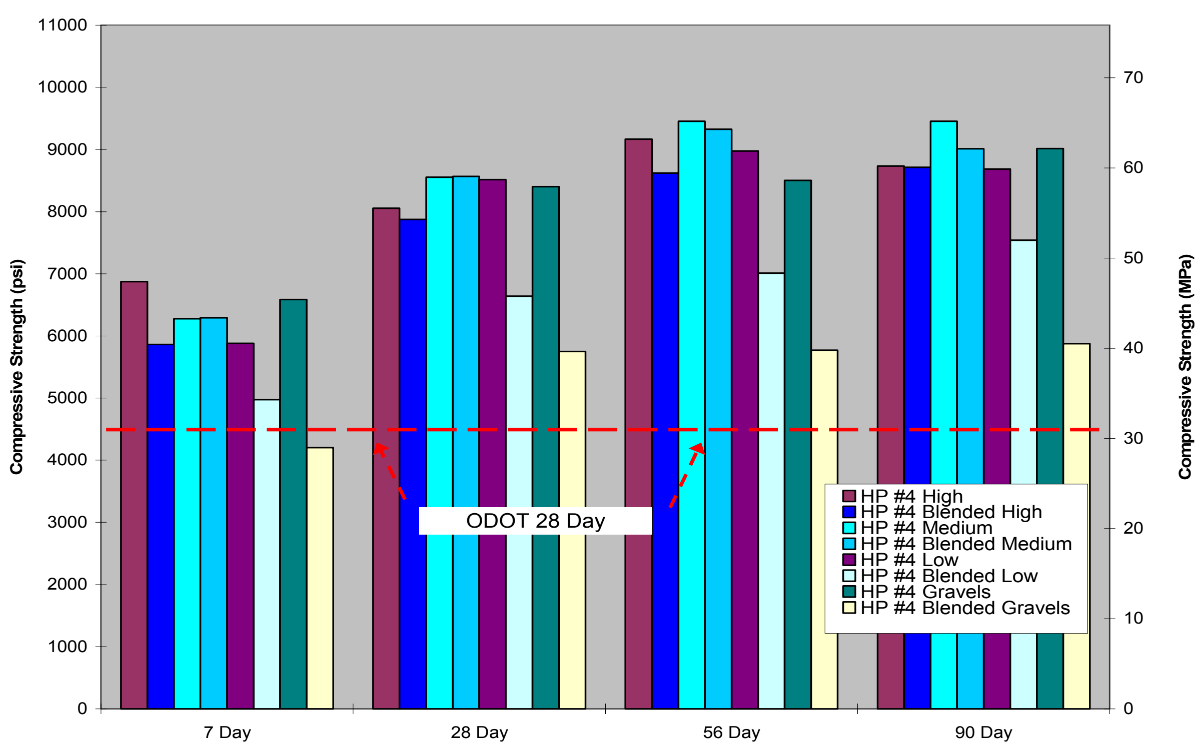


Figure 9: HP #4 Blended Compressive Strengths

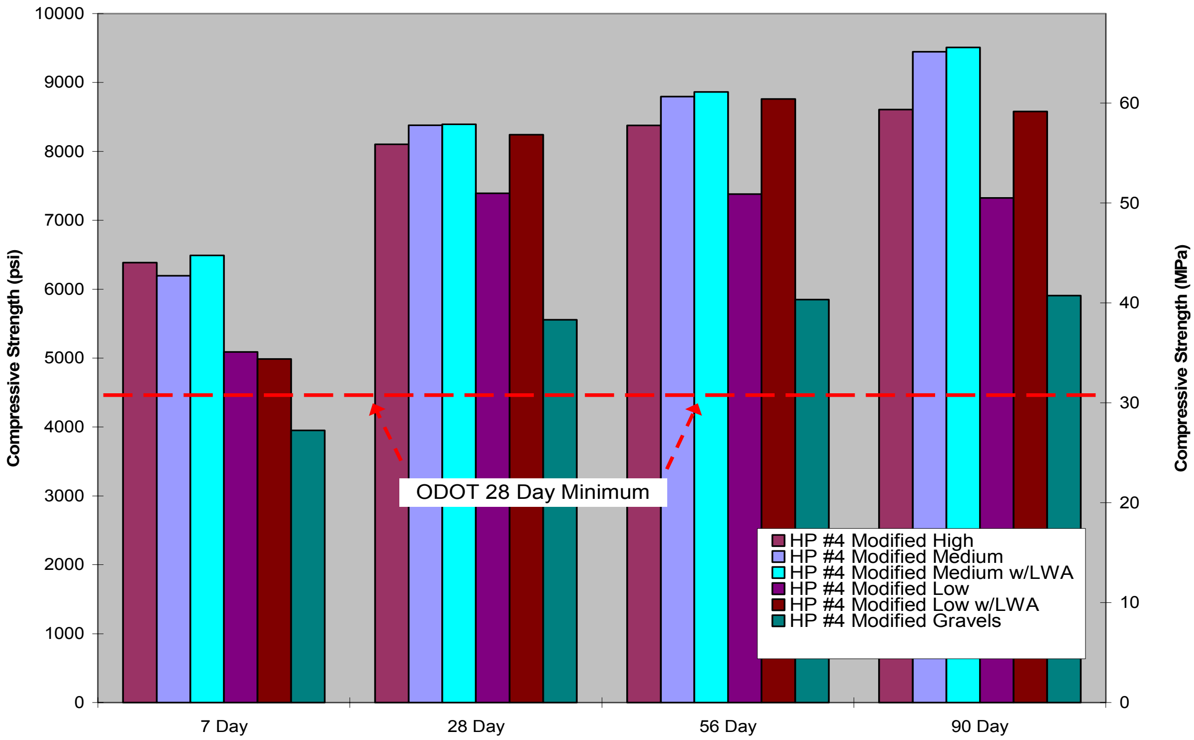


Figure 10: HP #4 Modified Compressive Strengths

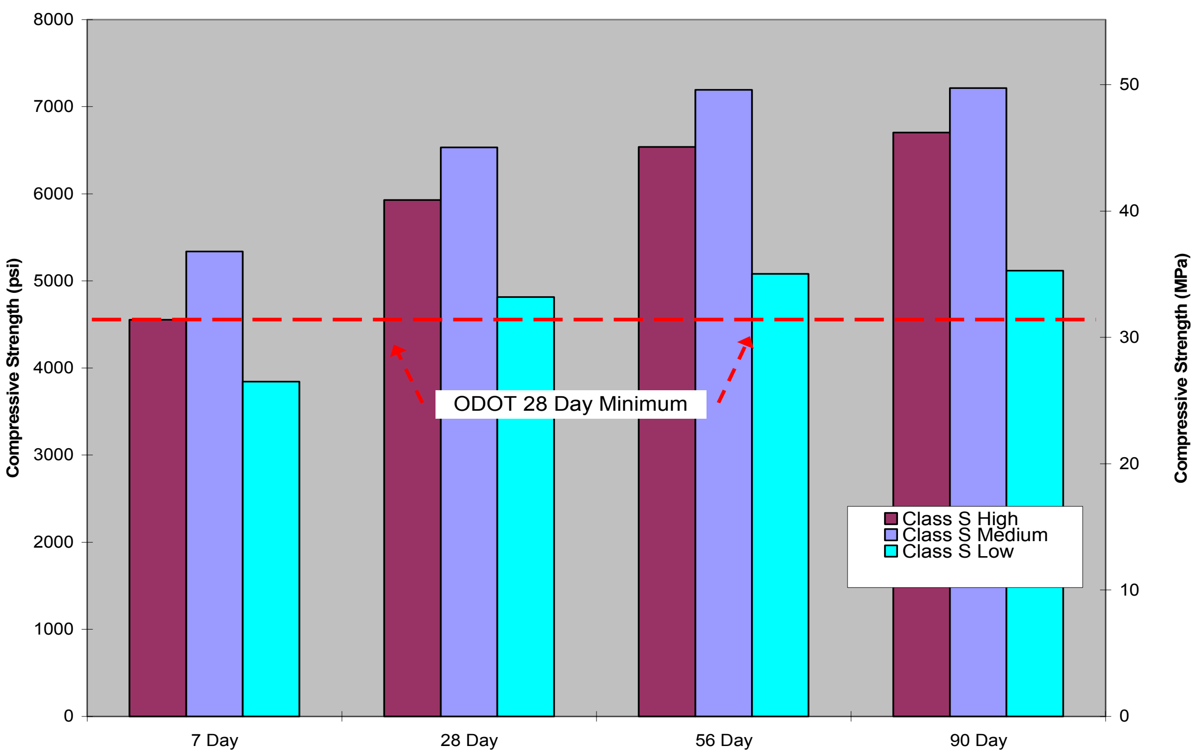


Figure 11: Class S Compressive Strengths

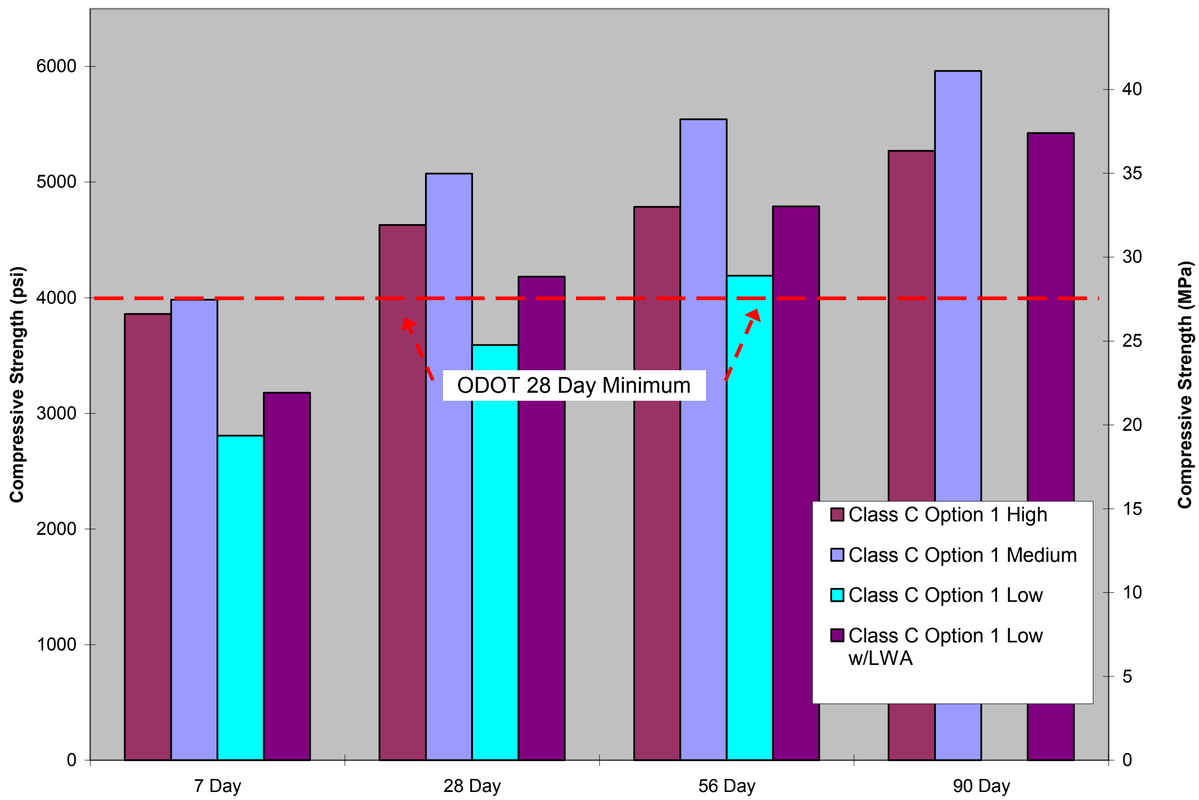


Figure 12: Class C Option 1 Compressive Strengths

Modulus of Rupture

Beam specimens were molded to determine the modulus of rupture (MOR). The specimens had a cross section of six by six inches and a length of twenty inches (152 x 152 x 508 mm). The modulus of rupture was determined at 1, 7, 28, and 56 days following ASTM C 78 using third-point loading on a hydraulic testing machine. Table 24 contains a summary of the MOR data.

Figure 13 through Figure 17 depict all of the high performance flexural strengths measured. From these graphs, it can be seen that for every mix with a 3 day beam test, the modulus of rupture was above 600 psi (4.14 MPa). For every mix without a 3 day beam test, the 7 day modulus of rupture was above 600 psi (4.14 MPa). The Ohio Department of Transportation (ODOT) specifies that if high early strength concrete is required the 3 day modulus of rupture must be above 600 psi (4.14 MPa) (ODOT Spec 2005). It can also be observed that most of the mixtures had 28 day flexural strengths above 900 psi (6.21 MPa), with some mixes exceeding 1200 psi (8.27 MPa) by 56 days.

Figure 18 shows the results for the modulus of rupture for the Class S concrete mixes. It can be seen that all of the mixes also had modulus of rupture above 600 psi (4.14 MPa) at 3 days and above 700 psi (4.83 MPa) by 7 days. The Class C Option 1 pavement mixes depicted in Figure 19 had a lower modulus of rupture, with two of the 3 day breaks above 600 psi (4.14

MPa) and the other two above 400 psi (2.76 MPa). All of the mixes reached 700 psi (4.83 MPa) by 28 or 56 days.

Table 24: Modulus of Rupture

Mix Identification	Modulus of Rupture									
	1 Day		3 Day		7 Day		28 Day		56 Day	
	psi	MPa	psi	MPa	psi	MPa	psi	MPa	psi	MPa
HP #3 High	-	-	-	-	825	5.69	1017	7.01	917	6.32
HP #3 Medium	633	4.36	938	6.47	929	6.41	1063	7.33	1158	7.98
HP #3 Low	-	-	-	-	867	5.98	900	6.21	1300	8.96
HP #3 Medium w/LWA	753	5.19	858	5.92	875	6.03	992	6.84	904	6.23
HP #3 Low w/LWA	627	4.32	-	-	783	5.40	1113	7.67	1271	8.76
HP #3 Gravels	527	3.63	417	2.88	817	5.63	1000	6.89	921	6.35
HP #3 Gravels w/LWA	573	3.95	646	4.45	758	5.23	875	6.03	950	6.55
HP #3 Blended High	653	4.50	846	5.83	883	6.09	1013	6.98	892	6.15
HP #3 Blended Medium	658	4.54	871	6.00	883	6.09	1079	7.44	892	6.15
HP #3 Blended Low	660	4.55	900	6.21	900	6.21	1142	7.87	1042	7.18
HP #3 Blended Low w/LWA	600	4.14	708	4.88	763	5.26	896	6.18	829	5.72
HP #4 High	683	4.71	854	5.89	938	6.47	-	-	1067	7.36
HP #4 Medium	717	4.94	696	4.80	958	6.61	964	6.65	-	-
HP #4 Low	-	-	-	-	833	5.74	967	6.67	1250	8.62
HP #4 Low w/ LWA	747	5.15	850	5.86	1056	7.28	1129	7.78	1292	8.91
HP #4 Gravels	633	4.36	800	5.52	950	6.55	1242	8.56	1083	7.47
HP #4 Blended High	500	3.45	-	-	904	6.23	996	6.87	946	6.52
HP #4 Blended Medium	627	4.32	-	-	763	5.26	1000	6.89	921	6.35
HP #4 Blended Low	633	4.37	775	5.34	892	6.15	975	6.72	1242	8.56
HP #4 Blended Low w/ LWA	400	2.76	542	3.74	754	5.20	1042	7.18	971	6.69
HP #4 Blended Gravels	600	4.14	721	4.97	867	5.98	950	6.55	1142	7.87
HP #4 Modified High	-	-	-	-	933	6.43	1125	7.76	1146	7.90
HP #4 Modified Medium	653	4.50	-	-	904	6.23	1125	7.76	1100	7.58
HP #4 Modified Medium w/LWA	583	4.02	833	5.74	833	5.74	775	5.34	992	6.84
HP #4 Modified Low	-	-	800	5.52	1008	6.95	1250	8.62	979	6.75
HP #4 Modified Low w/LWA	507	3.50	-	-	800	5.52	896	6.18	1167	8.05
HP #4 Modified Gravels	567	3.91	700	4.83	825	5.69	958	6.61	1083	7.47
Class S High	590	4.07	733	5.05	779	5.37	733	5.05	750	5.17
Class S Medium	733	5.05	750	5.17	879	6.06	800	5.52	879	6.06
Class S Low	650	4.48	638	4.40	796	5.49	950	6.55	1075	7.41
Class C Option 1 High	553	3.81	647	4.46	630	4.34	950	6.55	775	5.34
Class C Option 1 Medium	517	3.56	753	5.19	604	4.16	667	4.60	875	6.03
Class C Option 1 Low	400	2.76	533	3.67	613	4.23	788	5.43	-	-
Class C Option 1 Low w/LWA	450	3.10	467	3.22	563	3.88	542	3.74	825	5.69
Microsilica Overlay High	567	3.91	767	5.29	783	5.40	1096	7.56	888	6.12
Microsilica Overlay High with Fibers	583	4.02	800	5.52	813	5.61	904	6.23	833	5.74
Microsilica Overlay High w/LWA	683	4.71	900	6.21	875	6.03	1021	7.04	-	-

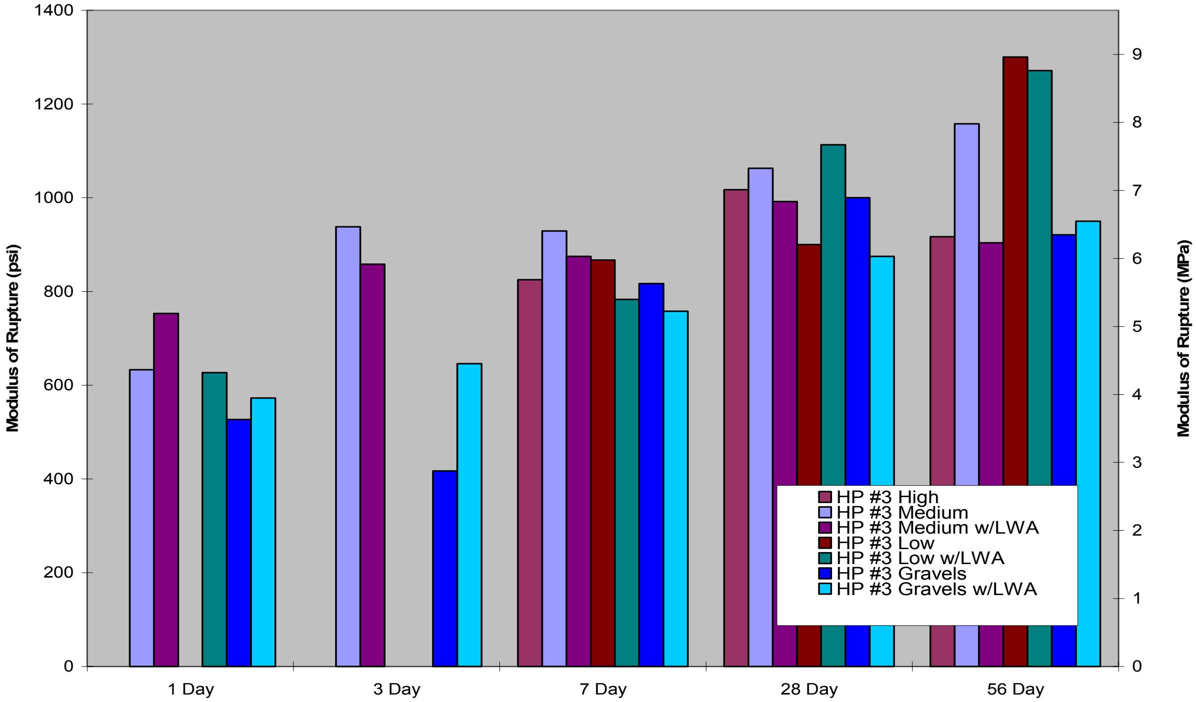


Figure 13: HP #3 Modulus of Rupture

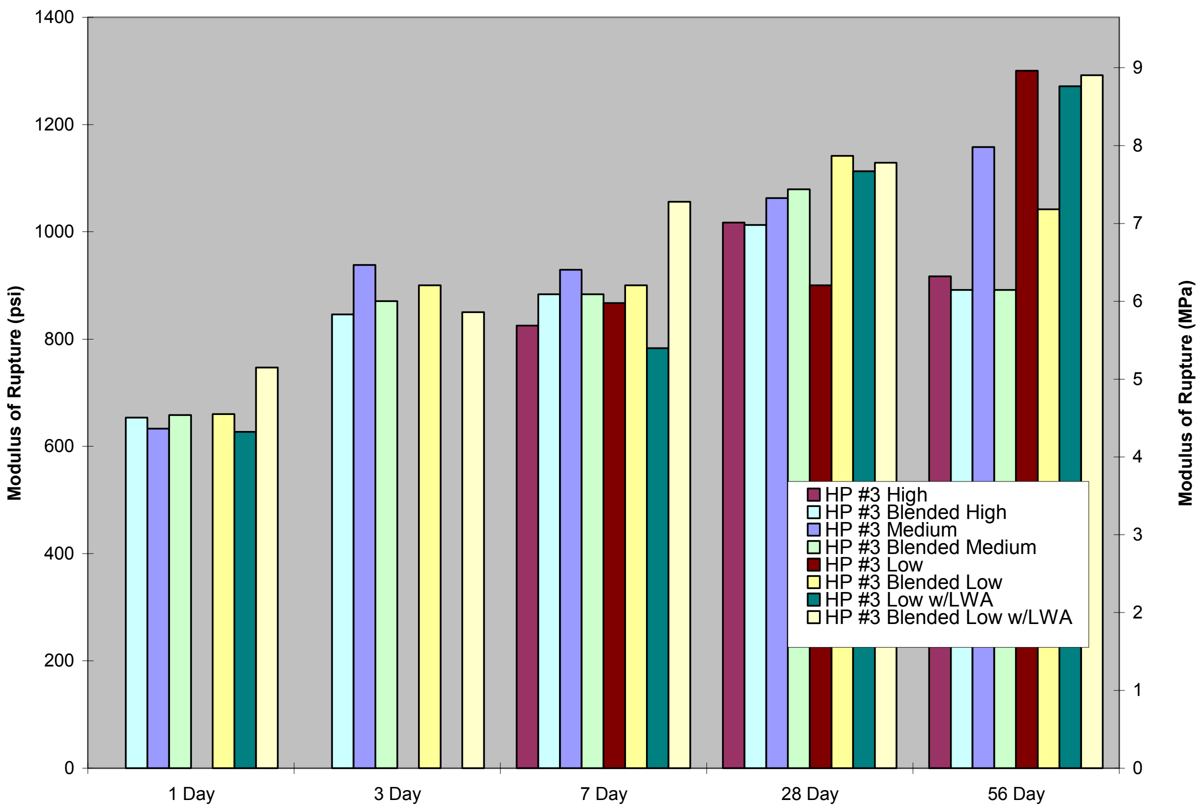


Figure 14: HP #3 Blended Modulus of Rupture

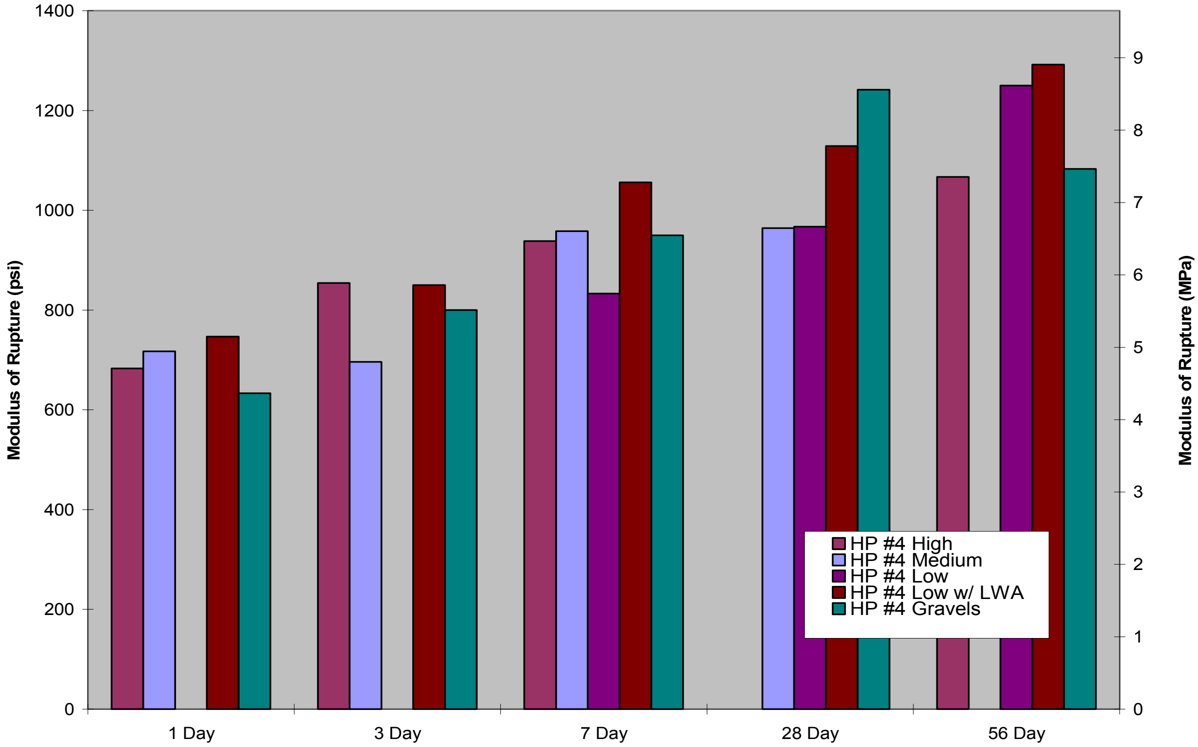


Figure 15: HP #4 Modulus of Rupture

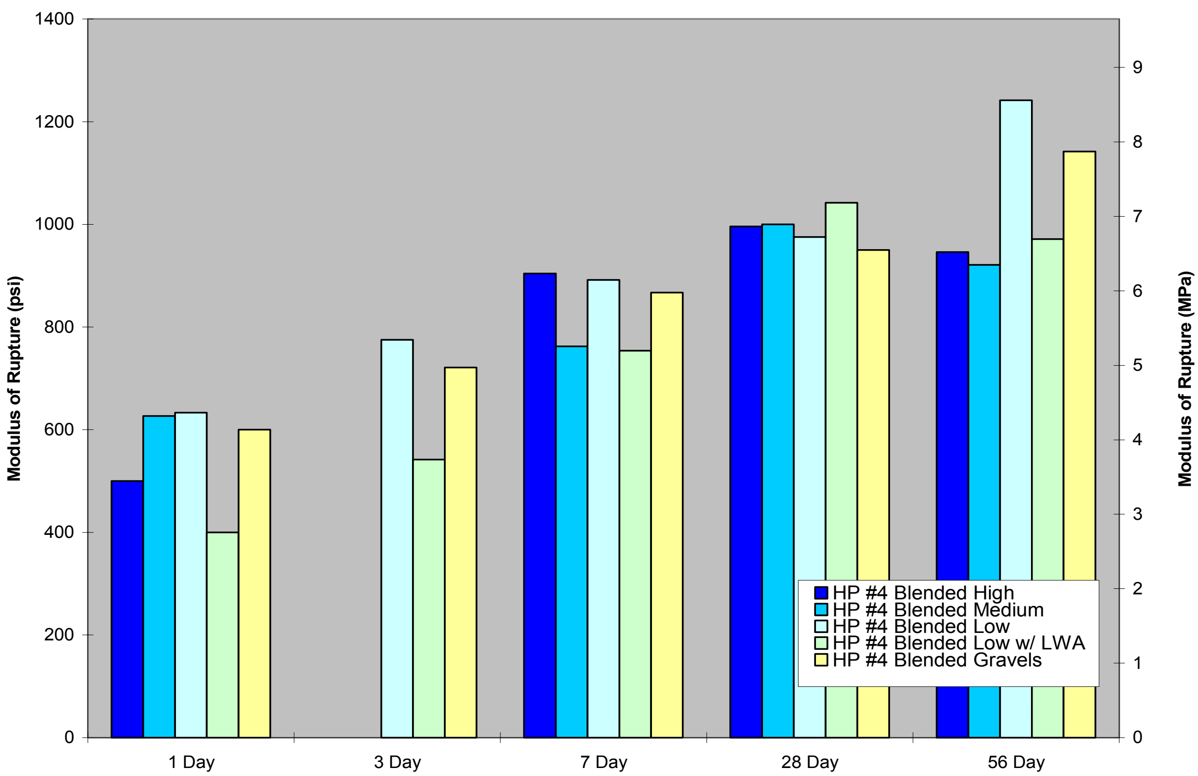


Figure 16: HP #4 Blended Modulus of Rupture

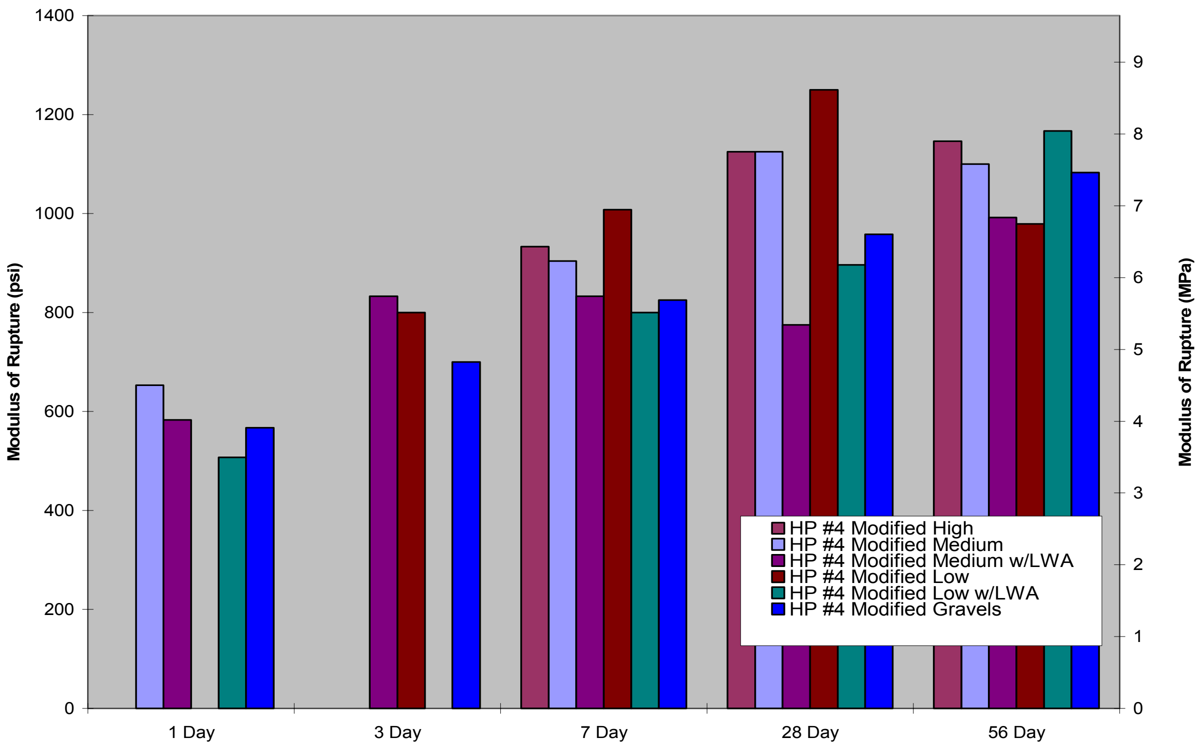


Figure 17: HP #4 Modified Modulus of Rupture

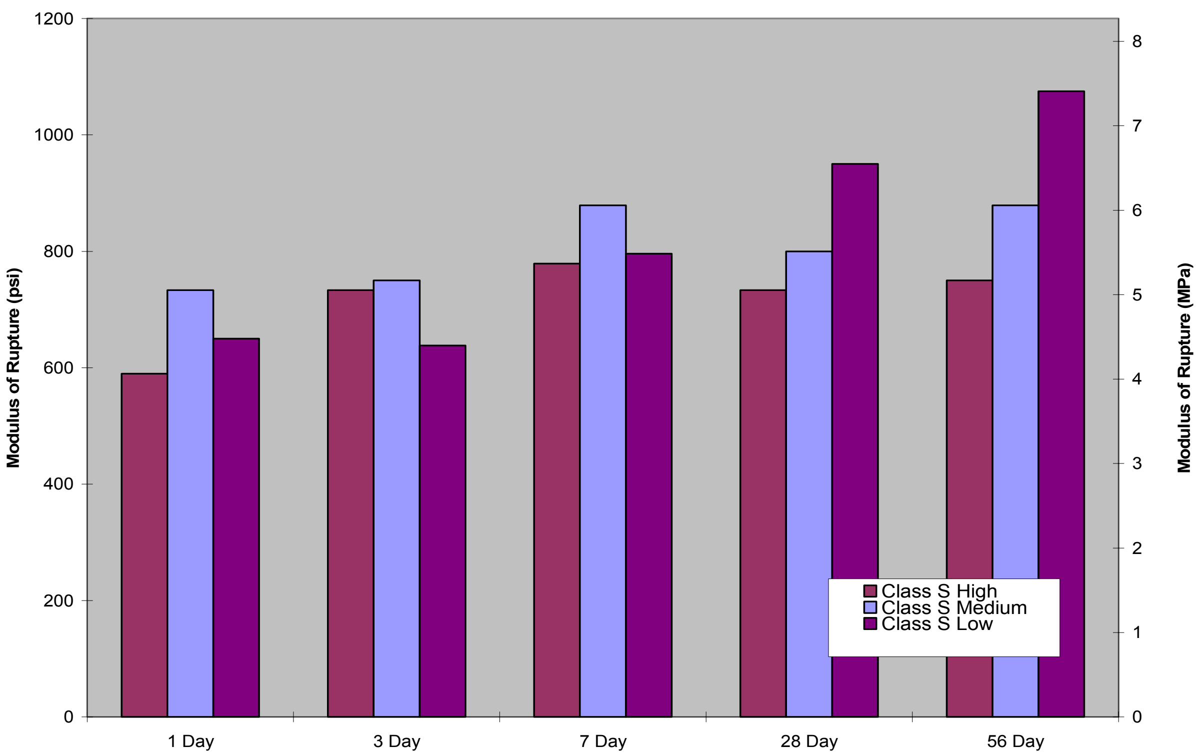


Figure 18: Class S Modulus of Rupture

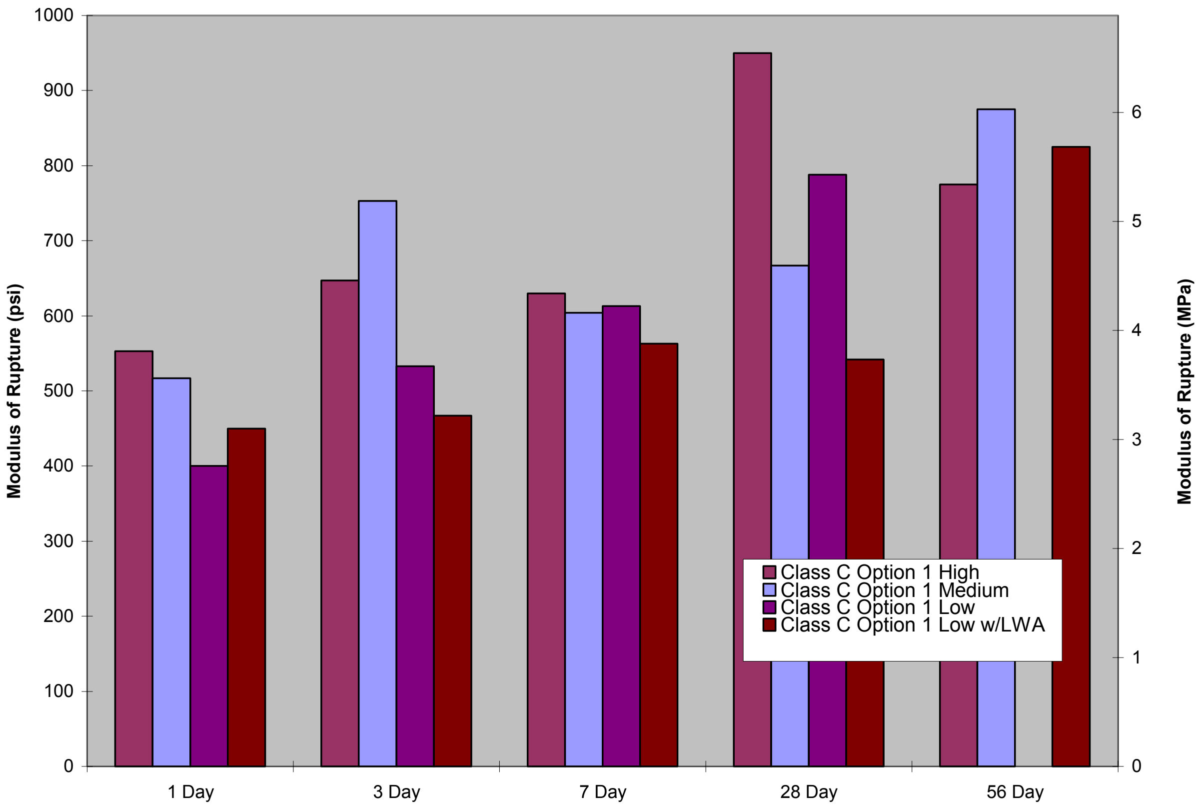


Figure 19: Class C Option 1 Modulus of Rupture

Splitting Tensile Strength

The splitting tensile strength of the concrete specimens was determined at 7, 28, 56, and 90 days, in accordance with ASTM C 496. The specimens were molded at the same time and in the same manner as the compressive strength specimens. Table 25 contains a summary of the splitting tensile strengths. ODOT does not specify a required splitting tensile strength. In this research, it was measured as an indicator of resistance to cracking.

Figure 20 through Figure 26 outline the splitting tensile strength for HP #3, HP #4, HP #4 Modified, Class S and Class C Option 1, respectively. The figures indicate the variability of the splitting tensile strengths. It can be seen in some cases that the splitting tensile strength was increased when Hydrocure 500 was introduced, but the variability of the samples makes the generalization of trends difficult. The figures do indicate, however, that all of the High Performance samples had a splitting tensile strength above 400 psi (2.76 MPa) by 7 days.

Table 25: Splitting Tensile Strength

Mix Identification	Splitting Tensile Strength							
	7 Day		28 Day		56 Day		90 Day	
	psi	MPa	psi	MPa	psi	MPa	psi	Mpa
HP #3 High	448	3.09	518	3.57	761	5.25	587	4.05
HP #3 High Remix	493	3.40	692	4.77	507	3.50	565	3.90
HP #3 Medium	442	3.05	589	4.06	599	4.13	676	4.66
HP #3 Medium w/LWA	573	3.95	757	5.22	664	4.58	756	5.21
HP #3 Low	515	3.55	537	3.70	583	4.02	828	5.71
HP #3 Low w/LWA	512	3.53	492	3.39	491	3.39	796	5.49
HP #3 Gravels	507	3.50	610	4.21	-	-	-	-
HP #3 Gravels w/LWA	557	3.84	776	5.35	761	5.25	-	-
HP #3 Blended High	463	3.19	926	6.38	557	3.84	680	4.69
HP #3 Blended Medium	666	4.59	710	4.90	831	5.73	820	5.65
HP #3 Blended Low	766	5.28	684	4.72	641	4.42	676	4.66
HP #4 High	557	3.84	501	3.45	532	3.67	625	4.31
HP #4 High Remix	450	3.10	-	-	652	4.50	649	4.47
HP #4 Medium	617	4.25	672	4.63	583	4.02	684	4.72
HP #4 Low	497	3.43	602	4.15	659	4.54	666	4.59
HP #4 Low w/ LWA	433	2.99	602	4.15	722	4.98	836	5.76
HP #4 Gravels	656	4.52	781	5.38	755	5.21	846	5.83
HP #4 Blended High	578	3.99	672	4.63	589	4.06	590	4.07
HP #4 Blended Medium	549	3.79	605	4.17	545	3.76	625	4.31
HP #4 Blended Low	477	3.29	573	3.95	688	4.74	766	5.28
HP #4 Blend Gravels	499	3.44	564	3.89	540	3.72	557	3.84
HP #4 Modified High	519	3.58	507	3.50	694	4.78	609	4.20
HP #4 Modified Medium	495	3.41	537	3.70	545	3.76	756	5.21
HP #4 Modified Medium w/LWA	615	4.24	494	3.41	873	6.02	857	5.91
HP #4 Modified Low	488	3.36	625	4.31	716	4.94	617	4.25
HP #4 Modified Low w/LWA	475	3.28	850	5.86	704	4.85	796	5.49
HP #4 Modified Gravels	507	3.50	547	3.77	519	3.58	499	3.44
Class S High	513	3.54	425	2.93	660	4.55	670	4.62
Class S Medium	434	2.99	781	5.38	570	3.93	561	3.87
Class S Low	371	2.56	489	3.37	529	3.65	681	4.70
Class C Option 1 High	379	2.61	660	4.55	456	3.14	658	4.54
Class C Option 1 Medium	463	3.19	421	2.90	591	4.07	716	4.94
Class C Option 1 Low	320	2.21	283	1.95	490	3.38	423	2.92
Class C Option 1 Low w/LWA	418	2.88	443	3.05	569	3.92	-	-
Microsilica Overlay High	393	2.71	482	3.32	564	3.89	547	3.77
Microsilica Overlay High with Fibers	458	3.16	497	3.43	509	3.51	-	-
Microsilica Overlay High w/LWA	597	4.12	617	4.25	589	4.06	584	4.03

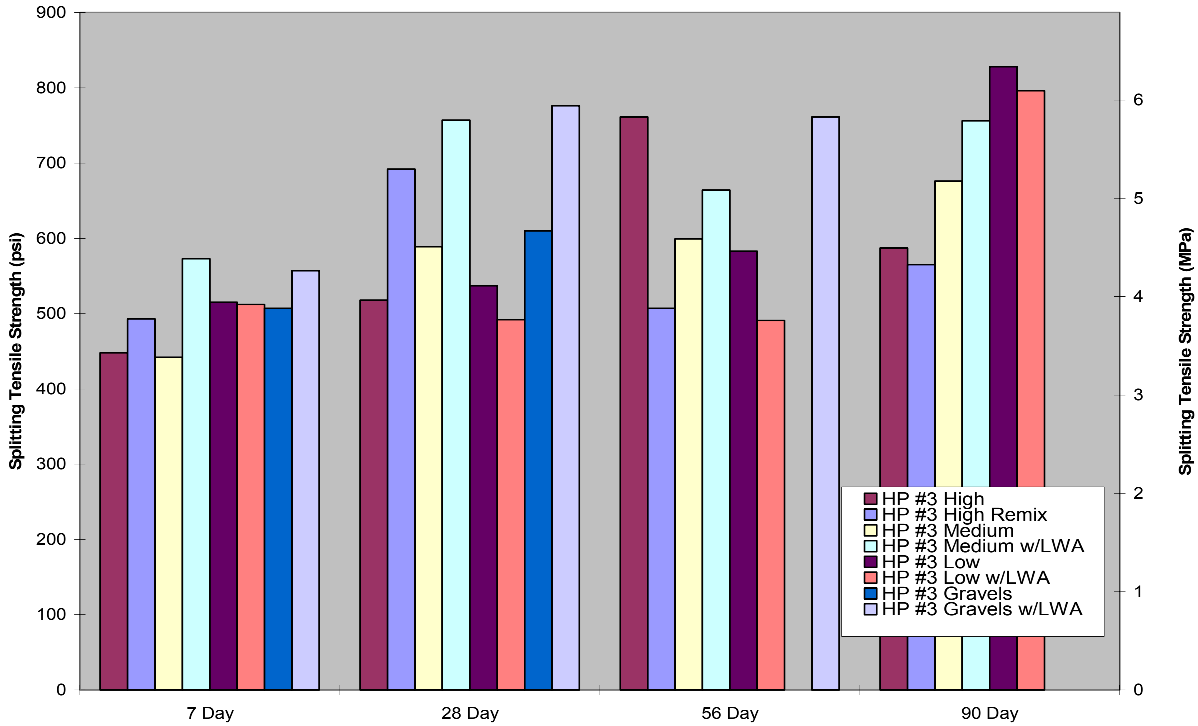


Figure 20: HP #3 Splitting Tensile Strength

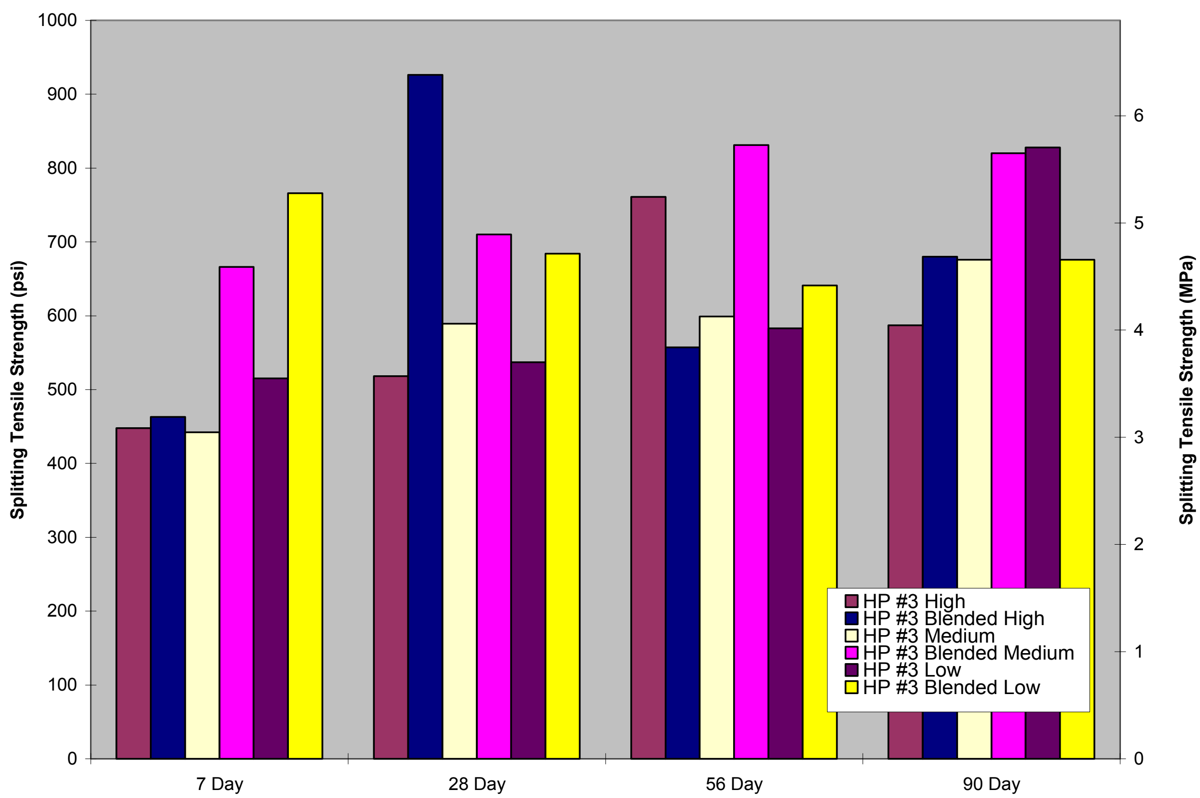


Figure 21: HP #3 Blended Splitting Tensile Strength

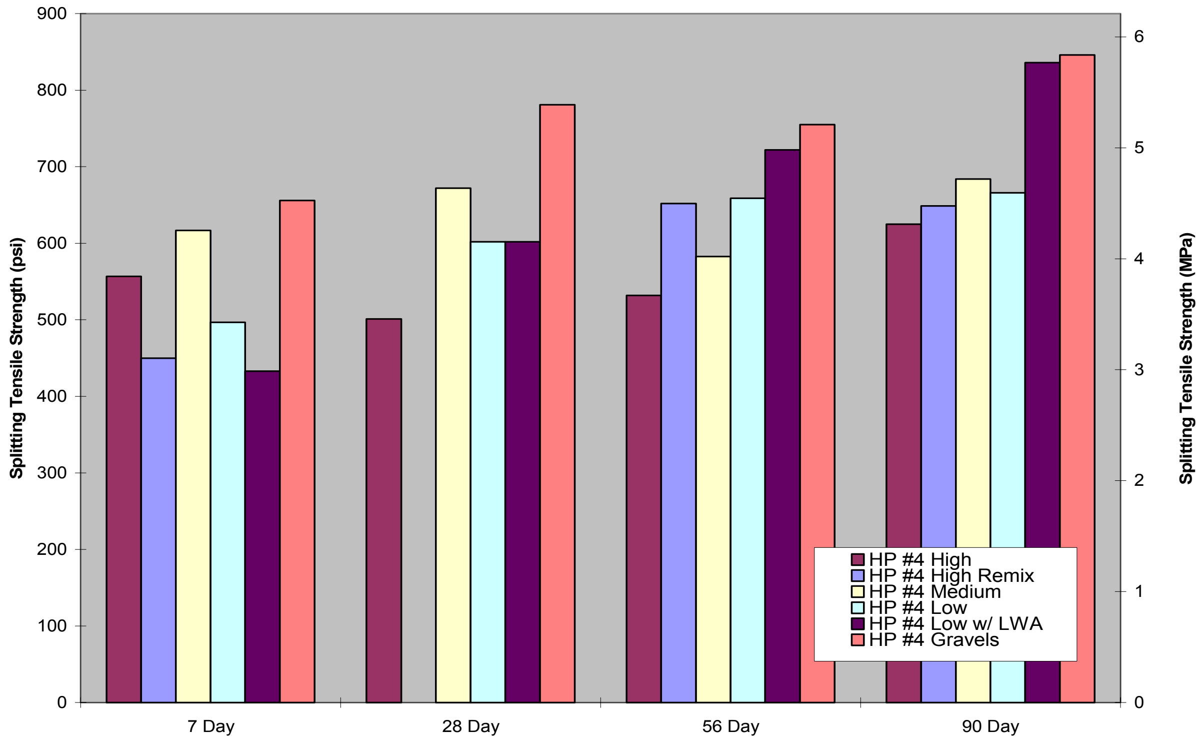


Figure 22: HP #4 Splitting Tensile Strength

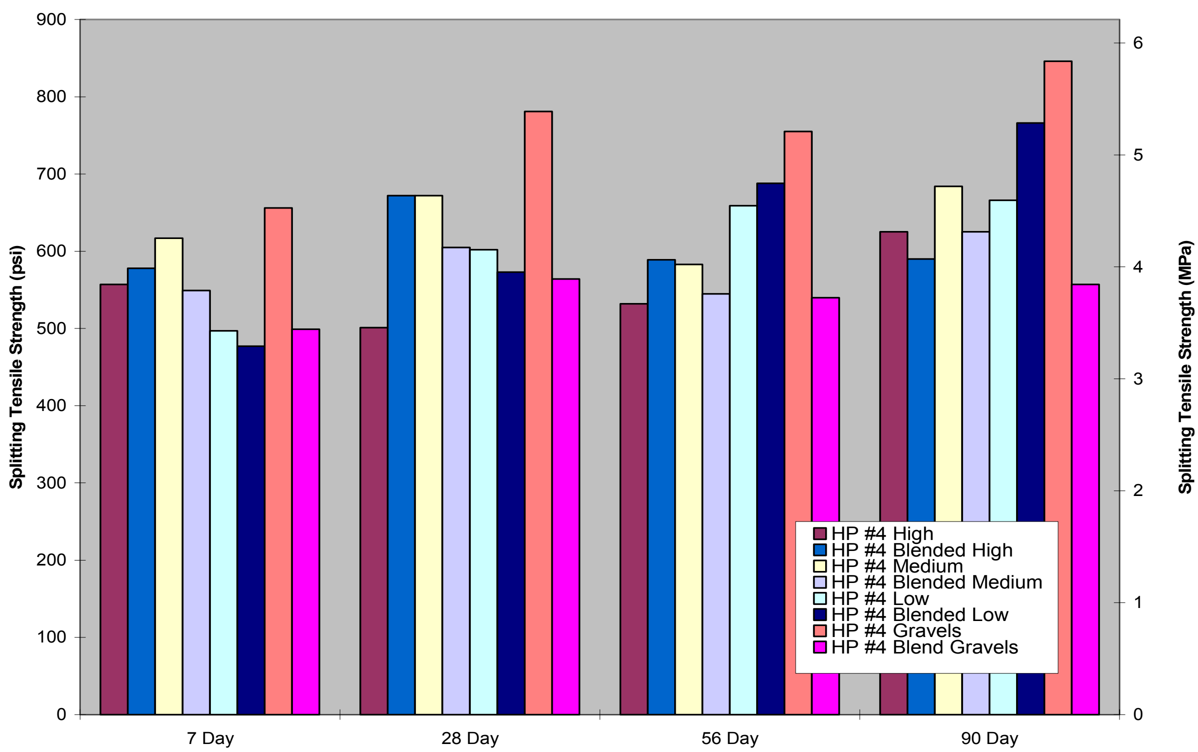


Figure 23: HP #4 Blended Splitting Tensile Strength

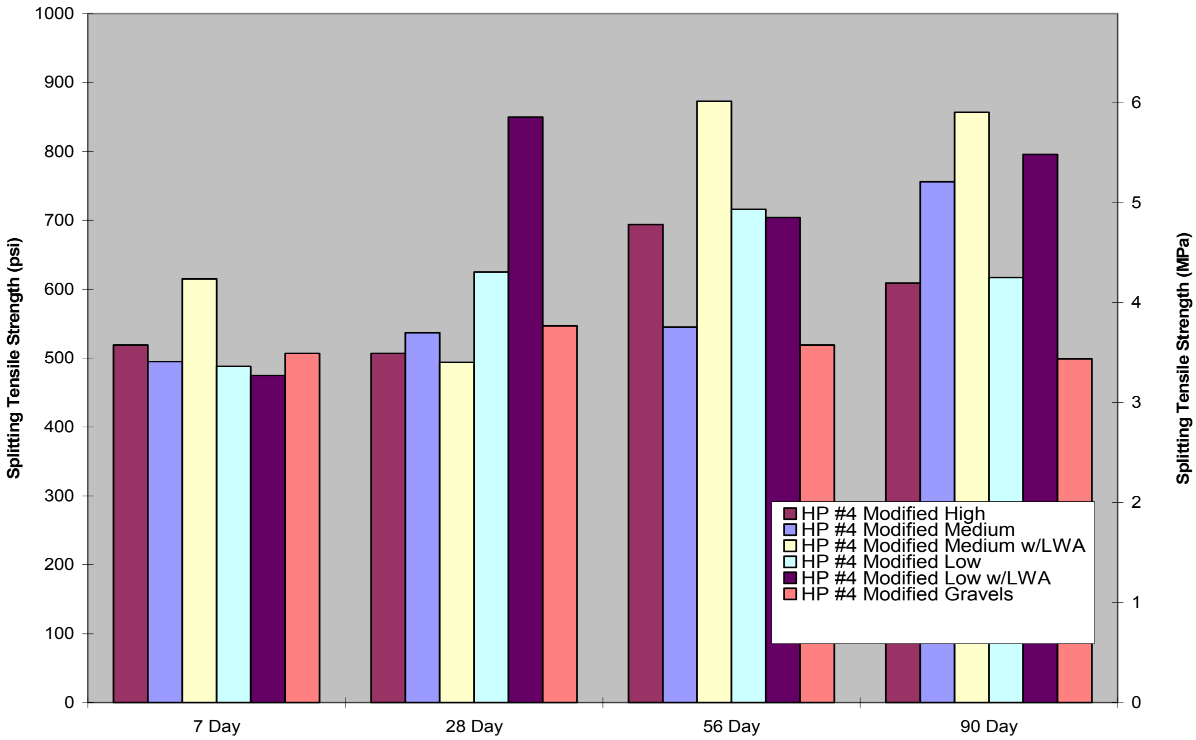


Figure 24: HP #4 Modified Splitting Tensile Strength

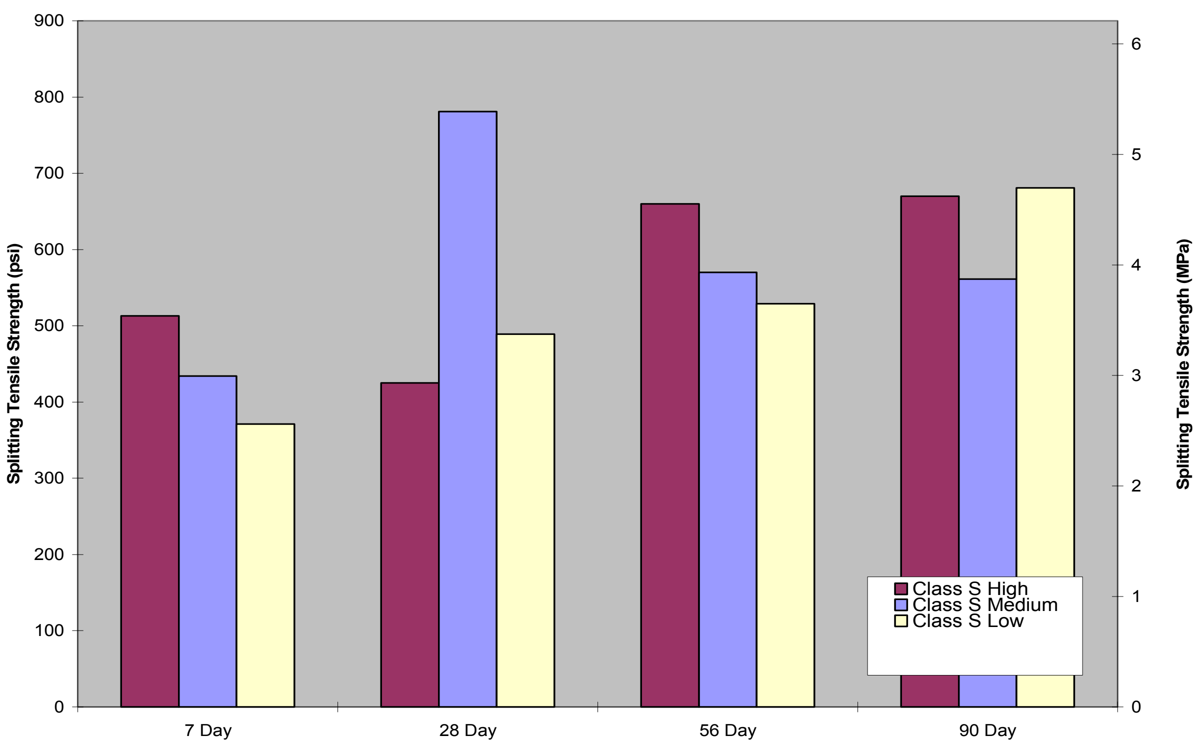


Figure 25: Class S Splitting Tensile Strength

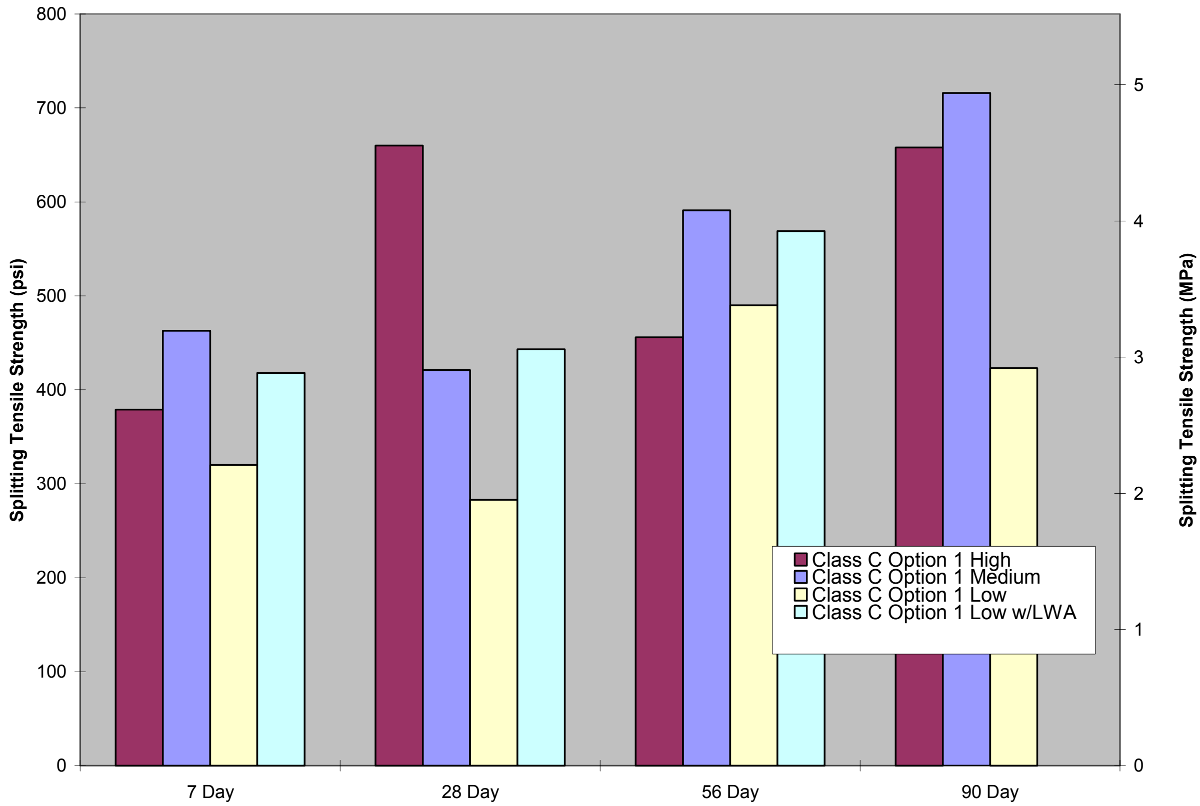


Figure 26: Class C Option 1 Splitting Tensile Strength

Static Modulus of Elasticity

The static modulus of elasticity was determined at 7, 28, 56, and 90 days using ASTM C 469. The results of the modulus of elasticity testing are shown in Table 26. Figure 27 through Figure 33 display the results for each mixture. Some variability in the modulus is apparent, but the values seem reasonable. The modulus of elasticity is relevant to cracking because for a given strain, a higher modulus produces a higher tensile stress.

Table 26: Static Modulus of Elasticity

Mix Identification	Modulus of Elasticity							
	7 Day		28 Day		56 Day		90 Day	
	10^6 psi	MPa	10^6 psi	MPa	10^6 psi	MPa	10^6 psi	MPa
HP #3 High	3.68	25373	3.94	27165	4.30	29647	3.87	26683
HP #3 High Remix	3.79	26131	3.75	25855	4.30	29647	3.74	25786
HP #3 Medium	3.82	26338	3.99	27510	4.69	32336	4.48	30889
HP #3 Medium w/LWA	3.63	25028	4.46	30751	4.66	32130	4.44	30613
HP #3 Low	-	12617	4.52	31164	4.17	28751	4.30	29647
HP #3 Low w/LWA	3.59	24752	4.36	30061	5.01	34543	5.18	35715
HP #3 Gravels	4.61	31785	4.66	32130	4.60	31716	-	-
HP #3 Gravels w/LWA	4.19	28889	4.13	28475	4.70	32405	-	-
HP #3 Blended High	3.82	26338	4.66	32130	4.84	33371	4.78	32957
HP #3 Blended Medium	5.07	34956	5.41	37301	5.59	38542	5.34	36818
HP #3 Blended Low	4.23	29165	5.01	34543	-	-	5.94	40955
HP #4 High	2.99	20615	3.82	26338	4.25	29303	4.83	33302
HP #4 High Remix	3.26	22477	4.40	30337	3.82	26338	4.30	29647
HP #4 Medium	3.89	26821	4.74	32681	4.30	29647	4.63	31923
HP #4 Low	2.86	19719	4.30	29647	5.16	35577	4.69	32336
HP #4 Low w/ LWA	3.27	22546	4.42	30475	5.16	35577	5.23	36060
HP #4 Gravels	4.33	29854	4.10	28269	4.45	30682	4.88	33646
HP #4 Blended High	4.97	34267	4.40	30337	4.58	31578	4.47	30820
HP #4 Blended Medium	4.44	30613	5.29	36473	5.09	35094	4.58	31578
HP #4 Blended Low	3.93	27096	4.69	32336	4.91	33853	4.54	31302
HP #4 Blended Gravels	3.57	24614	3.46	23856	4.10	28269	3.85	26545
HP #4 Modified High	3.94	27165	4.30	29647	3.67	25304	4.14	28544
HP #4 Modified Medium	3.82	26338	4.72	32543	4.69	32336	4.99	34405
HP #4 Modified Medium w/LWA	4.38	30199	4.37	30130	5.16	35577	5.13	35370
HP #4 Modified Low	3.87	26683	4.82	33233	4.83	33302	4.56	31440
HP #4 Modified Low w/LWA	3.37	23235	4.30	29647	5.05	34819	4.81	33164
HP #4 Mod. Gravels	3.86	26614	3.55	24476	4.54	31302	4.10	28269
Class S High	3.71	25580	4.21	29027	3.72	25648	4.33	29854
Class S Medium	4.67	32199	4.47	30820	4.41	30406	4.91	33853
Class S Low	3.59	24752	4.91	33853	4.52	31164	5.40	37232
Class C Option 1 High	3.20	22063	-	-	3.68	25373	4.33	29854
Class C Option 1 Medium	3.91	26958	4.02	27717	4.50	31026	4.96	34198
Class C Option 1 Low	3.37	23235	3.44	23718	3.37	23235	-	-
Class C Option 1 Low w/LWA	3.74	25786	4.20	28958	4.75	32750	3.91	26979
Microsilica Overlay High	2.96	20408	3.40	23442	3.65	25166	4.08	28131
Microsilica Overlay High with Fibers	3.15	21718	4.18	28820	4.06	27993	4.24	29234
Microsilica Overlay High w/LWA	2.96	20408	3.48	23994	4.02	27717	4.37	30130

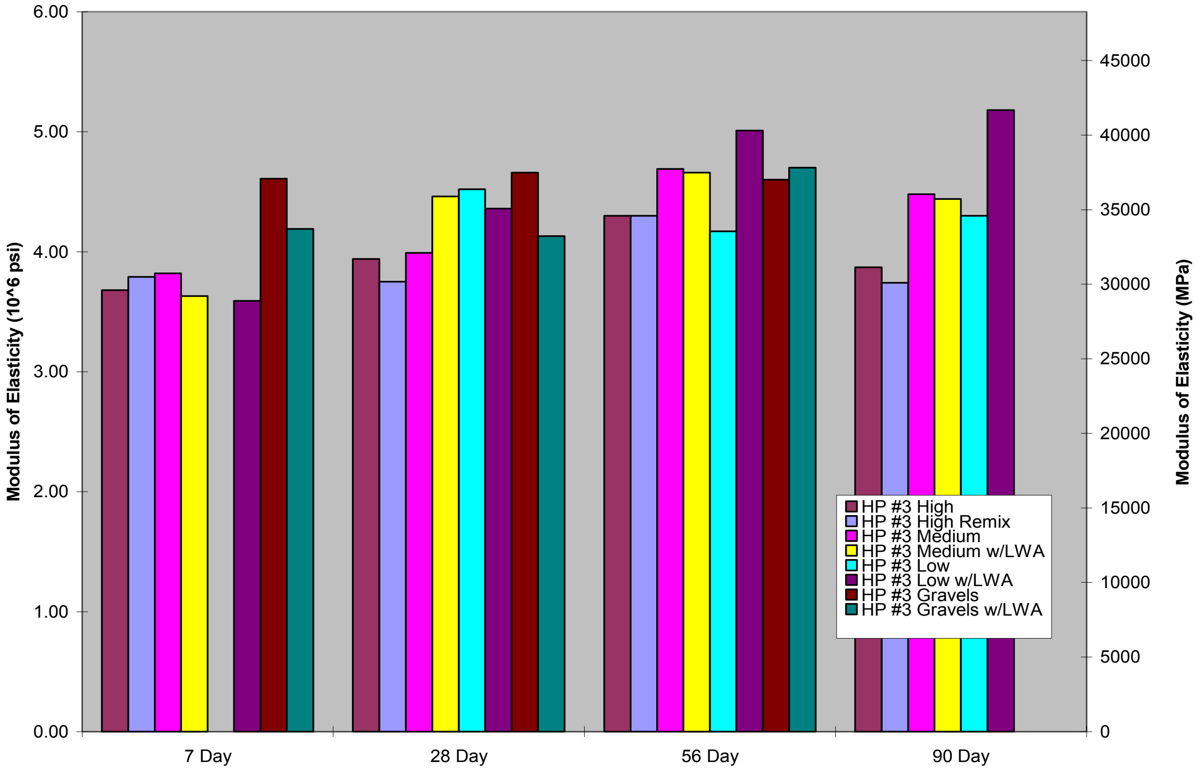


Figure 27: HP #3 Static Modulus of Elasticity

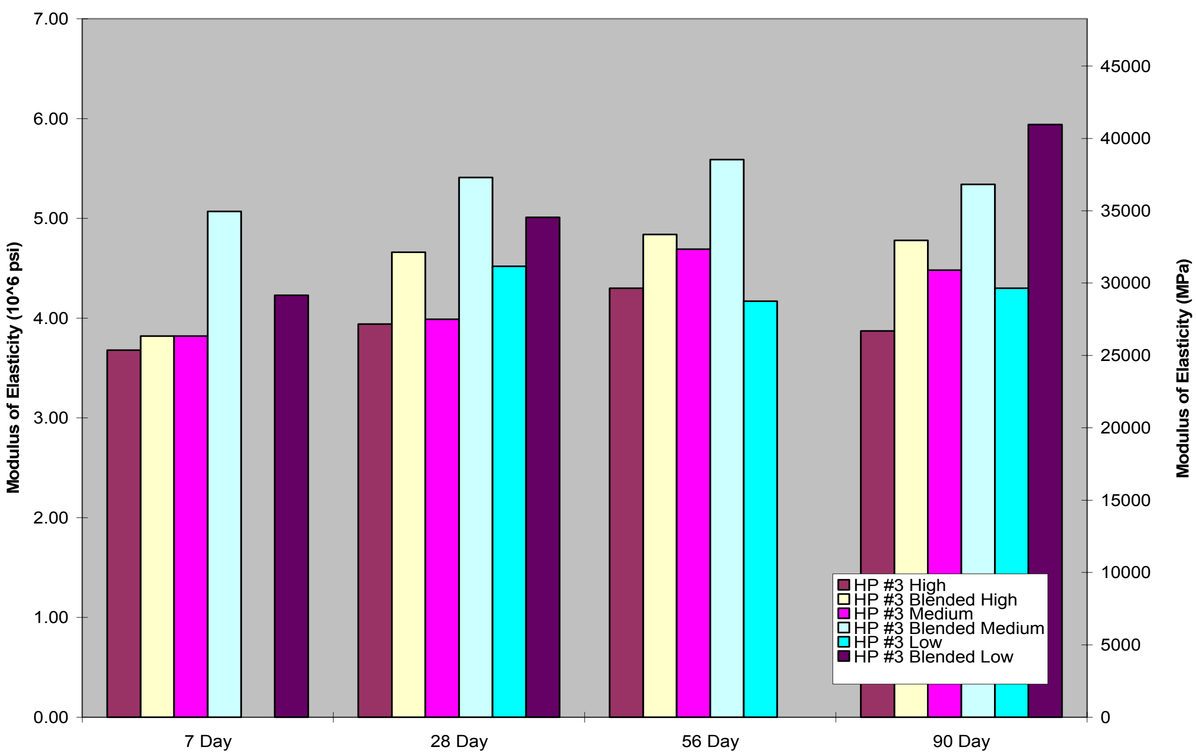


Figure 28: HP #3 Blended Static Modulus of Elasticity

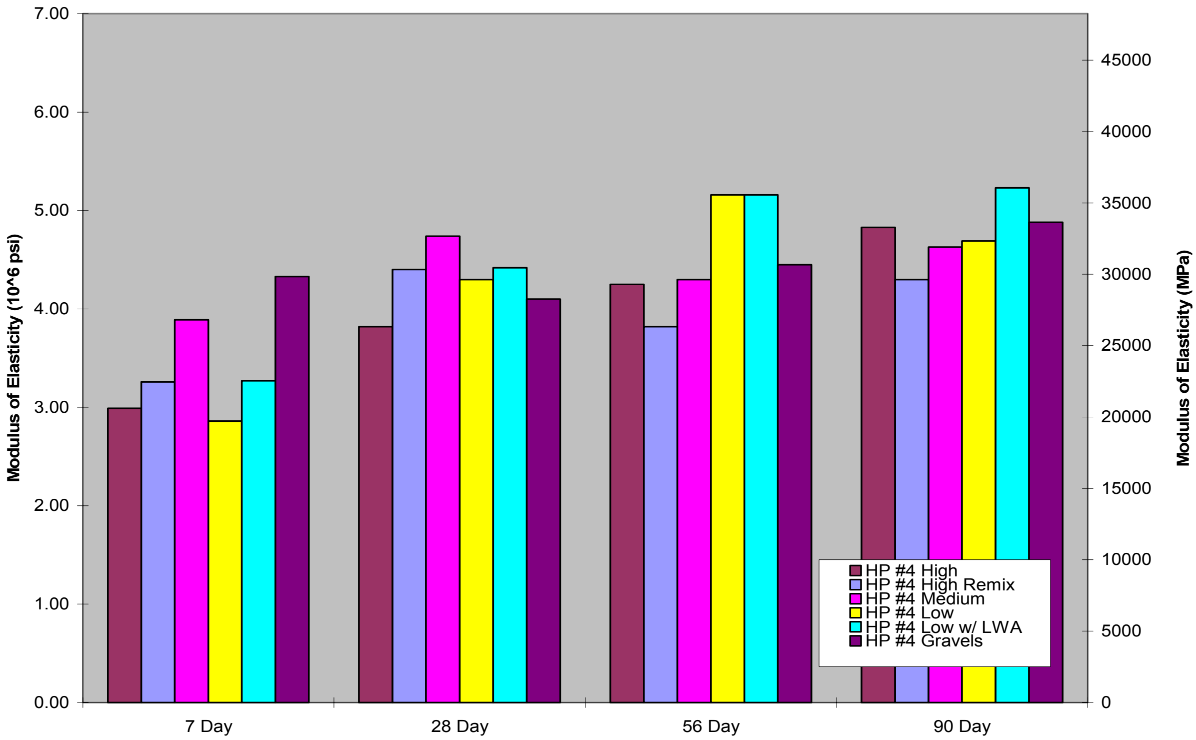


Figure 29: HP #4 Static Modulus of Elasticity

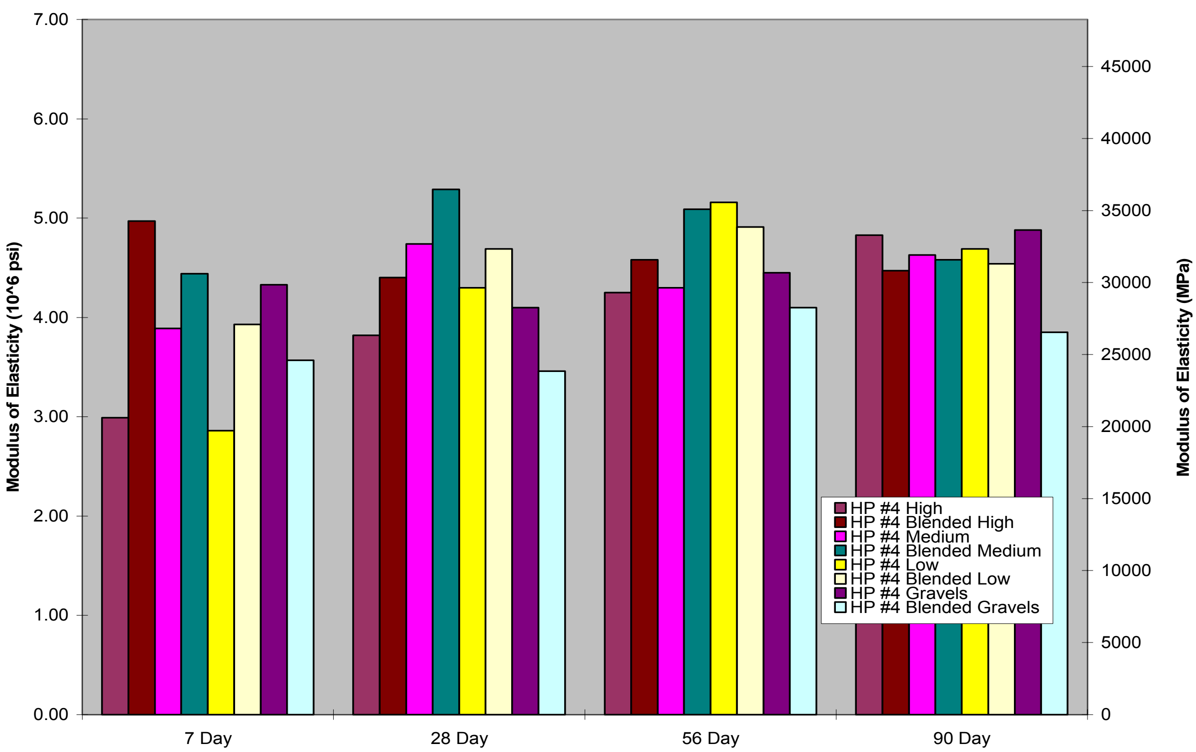


Figure 30: HP #4 Blended Static Modulus of Elasticity

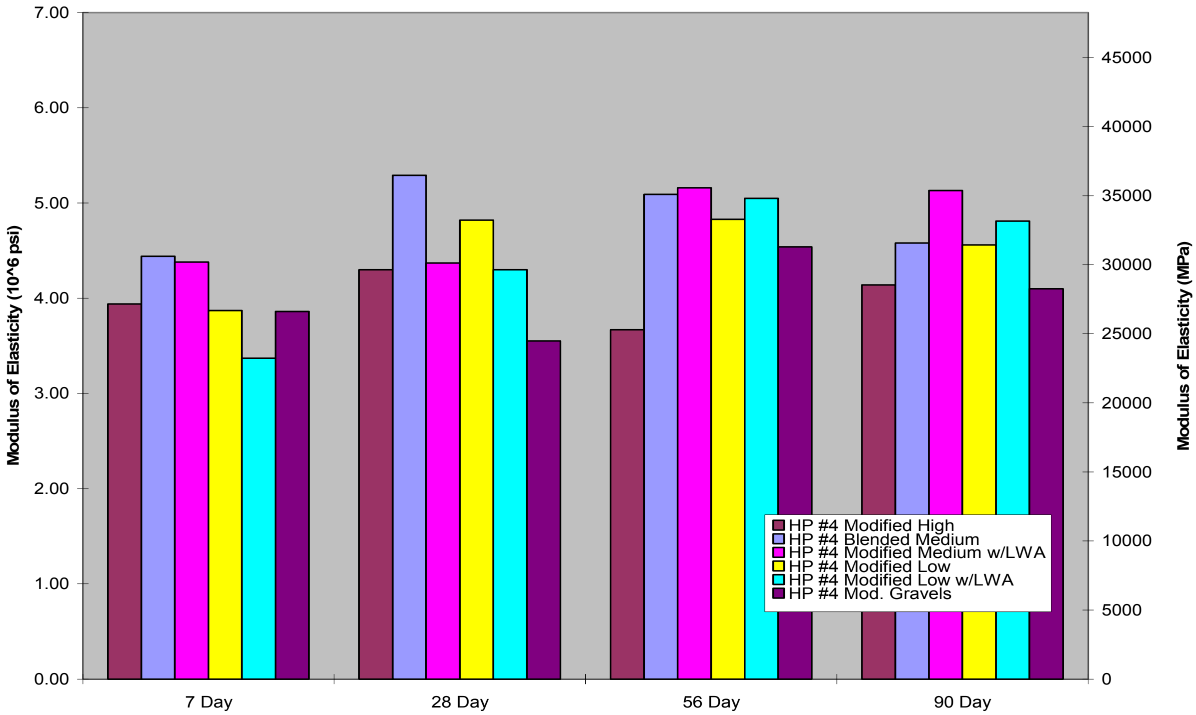


Figure 31: HP #4 Modified Static Modulus of Elasticity

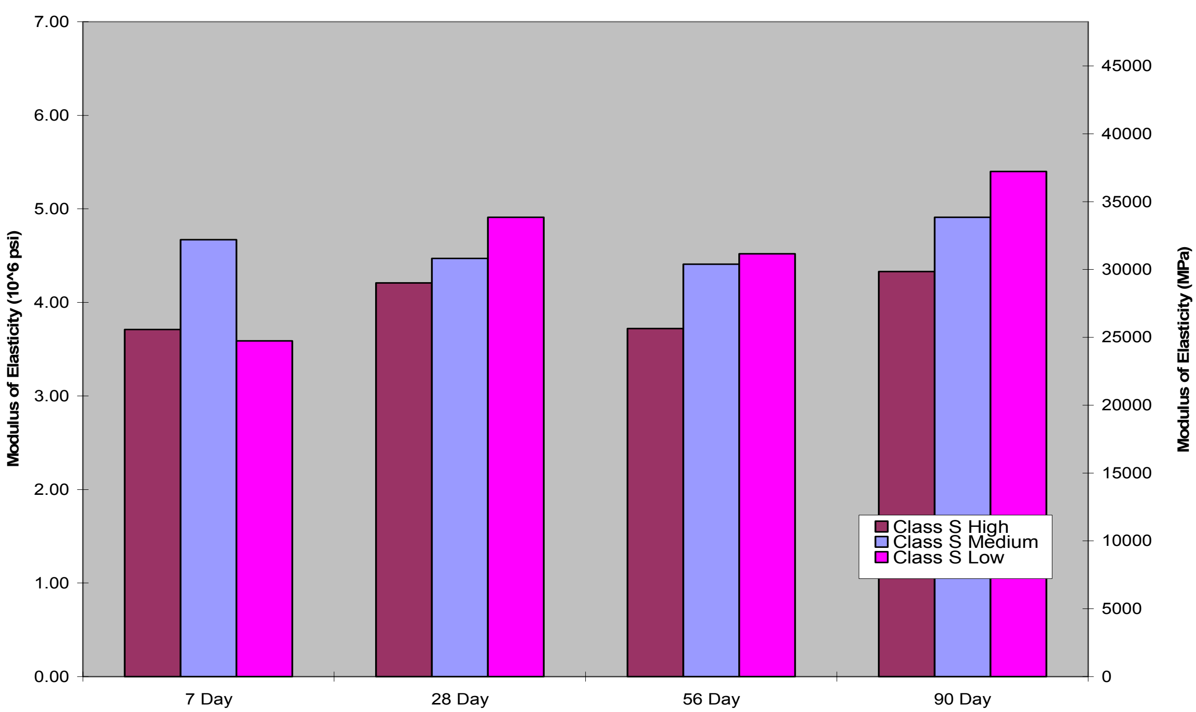


Figure 32: Class S Static Modulus of Elasticity

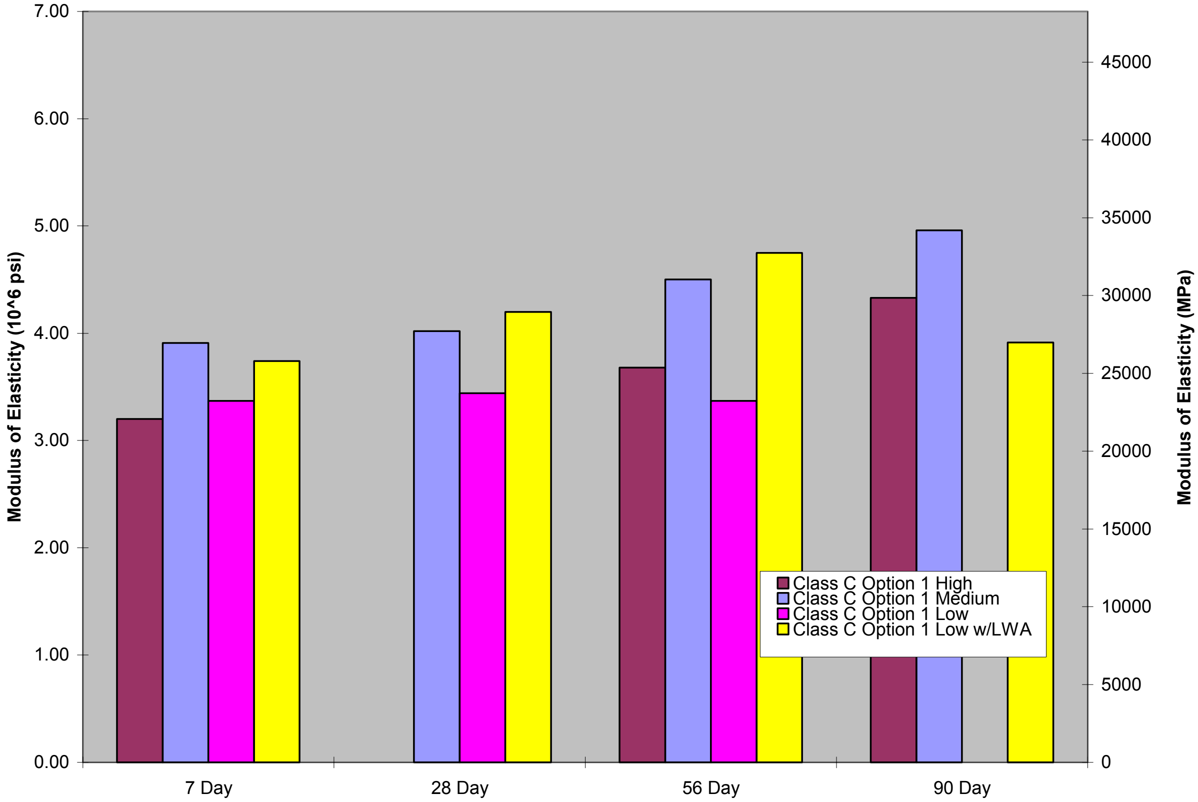


Figure 33: Class C Option 1 Static Modulus of Elasticity

Unrestrained Shrinkage

The unrestrained shrinkage for the concrete specimens was determined following ASTM C 157. Shrinkage readings were started approximately 24 hours from time of set and were recorded after 7, 28, 56, and 90 days. Four concrete prisms were prepared from each mixture, two of which were sealed with paraffin wax. One sealed and one unsealed specimen each was placed in the moist cure room at a temperature of 73.5^{o} F ± 3.5^{o} F (23 ± 2^{o} C) and 50% ± 5% relative humidity. The other two samples were placed in the lime bath.

The specimens that were placed in the lime bath experienced very little shrinkage, as can be seen in Table 27 and Table 28. The sealed High Performance specimens had a maximum shrinkage of 0.015% or 150 microstrain. The unsealed High Performance specimens had a maximum shrinkage of 0.021% or 210 microstrain.

The concrete samples that were placed in the moist cure room experienced larger amounts of shrinkage than the samples in the lime bath. These values are shown in Table 29 and Table 30. Typically the sealed samples shrank less than the unsealed samples. The maximum shrinkage in the sealed samples was 800 microstrain for one sample with most falling below 550 microstrain. The unsealed samples experienced a maximum shrinkage of 650 microstrain.

Table 27: Percent Shrinkage of Sealed Samples in Lime Bath

Mix Identification	Percent Length Change			
	7 Day	28 Day	56 Day	90 Day
HP #3 High	0.000%	0.001%	0.013%	0.010%
HP #3 High Remix	0.003%	0.001%	0.001%	0.002%
HP #3 Medium	0.002%	0.001%	0.000%	0.000%
HP #3 Medium Remix	0.002%	0.001%	0.004%	-
HP #3 Med w/LWA	0.001%	0.002%	0.005%	0.002%
HP #3 Low	0.012%	0.003%	0.001%	0.000%
HP #3 Low w/LWA	0.002%	0.000%	0.001%	0.003%
HP #3 Gravels	0.002%	0.001%	-	-
HP #3 Gravels w/ LWA	0.003%	0.000%	-	-
HP #3 Blended High	0.004%	0.004%	0.004%	0.006%
HP #3 Blended Medium	0.004%	0.007%	0.004%	0.006%
HP #3 Blended Low	0.001%	0.002%	0.000%	0.001%
HP #4 High	0.002%	0.006%	0.001%	0.000%
HP #4 Medium	0.004%	0.003%	0.004%	0.001%
HP #4 Low	0.003%	0.010%	0.007%	0.007%
HP #4 Low w/ LWA	0.003%	0.009%	0.008%	0.007%
HP #4 Gravels	0.000%	0.001%	0.003%	0.005%
HP #4 Blended High	0.005%	0.001%	0.006%	0.005%
HP #4 Blended Medium	0.000%	0.004%	0.002%	0.003%
HP #4 Blended Low	0.003%	0.001%	0.005%	0.002%
HP #4 Blended Gravels	0.003%	0.002%	0.000%	0.004%
HP #4 Modified High	0.006%	0.012%	0.011%	0.008%
HP #4 Modified Medium	0.000%	0.001%	0.000%	0.004%
HP #4 Modified Medium w/LWA	0.020%	0.015%	0.014%	0.013%
HP #4 Modified Low	0.004%	0.006%	0.000%	0.000%
HP #4 Modified Low w/LWA	0.002%	0.004%	0.009%	0.006%
HP #4 Modified Gravels	0.002%	0.003%	0.001%	0.002%
Class C Option 1 High	0.014%	0.005%	0.012%	0.015%
Class C Option 1 Medium	0.011%	0.005%	-	0.003%
Class C Option 1 Low	0.002%	0.014%	0.004%	0.003%
Class C Option 1 Low w/LWA	0.005%	0.000%	0.009%	0.010%
Class S High	0.001%	0.004%	0.008%	0.008%
Class S Medium	0.002%	0.000%	0.003%	0.003%
Class S Low	0.005%	0.001%	0.001%	0.001%
Microsilica Overlay High w/Fibers	0.001%	0.001%	0.004%	0.007%
Microsilica Overlay High	0.040%	0.003%	0.007%	0.005%
Microsilica Overlay High w/LWA	0.005%	0.010%	-	0.001%

Table 28: Percent Shrinkage of Unsealed Samples in Lime Bath

Mix Identification	Percent Length Change			
	7 Day	28 Day	56 Day	90 Day
HP #3 High	0.003%	0.002%	0.003%	0.001%
HP #3 High Remix	0.006%	0.001%	0.003%	0.005%
HP #3 Medium	0.006%	0.003%	0.005%	0.008%
HP #3 Medium Remix	0.003%	0.001%	0.003%	-
HP #3 Medium w/LWA	0.004%	0.005%	0.000%	-
HP #3 Low	0.008%	0.008%	0.001%	0.006%
HP #3 Low w/LWA	0.008%	0.009%	0.012%	0.017%
HP #3 Gravels	0.004%	0.001%	-	-
HP #3 Gravels w/LWA	0.002%	0.006%	-	-
HP #3 Blended High	0.000%	0.001%	0.001%	0.000%
HP #3 Blended Medium	0.000%	0.002%	0.001%	0.002%
HP #3 Blended Low	0.003%	0.000%	0.002%	0.000%
HP #4 High	0.009%	0.012%	0.003%	0.003%
HP #4 Medium	0.007%	0.003%	0.005%	0.013%
HP #4 Low	0.008%	0.015%	0.014%	0.020%
HP #4 Low w/ LWA	0.012%	0.021%	0.019%	0.017%
HP #4 Gravels	0.005%	0.003%	0.006%	0.000%
HP #4 Blended High	0.002%	0.002%	0.002%	0.003%
HP #4 Blended Medium	0.001%	0.007%	0.003%	0.006%
HP #4 Blended Low	0.006%	0.004%	0.006%	0.004%
HP #4 Blended Gravels	0.002%	0.006%	0.003%	0.008%
HP #4 Modified High	0.001%	0.002%	0.003%	0.001%
HP #4 Modified Medium	0.001%	0.001%	0.001%	0.005%
HP #4 Modified Medium w/LWA	0.030%	0.020%	0.018%	0.016%
HP #4 Modified Low	0.004%	0.003%	0.004%	0.012%
HP #4 Modified Low w/LWA	0.004%	0.005%	0.010%	0.014%
HP #4 Modified Gravels	0.005%	0.005%	0.005%	0.006%
Class C Option 1 High	0.016%	0.014%	-	-
Class C Option 1 Medium	0.004%	0.015%	-	0.014%
Class C Option 1 Low	0.009%	0.006%	0.007%	0.013%
Class C Option 1 Low w/LWA	0.003%	0.003%	0.006%	0.006%
Class S High	0.001%	0.006%	0.009%	0.012%
Class S Medium	0.003%	0.006%	0.008%	0.002%
Class S Low	0.001%	0.002%	0.004%	0.005%
Microsilica Overlay High w/Fibers	0.005%	0.007%	0.103%	0.006%
Microsilica Overlay High	0.040%	0.032%	0.090%	0.002%
Microsilica Overlay High w/LWA	0.002%	0.007%	-	0.006%

Table 29: Percent Shrinkage of Sealed Samples in Moist Cure Room

Mix Identification	Percent Length Change			
	7 Day	28 Day	56 Day	90 Day
HP #3 High	0.013%	0.032%	0.054%	0.054%
HP #3 High Remix	0.017%	0.046%	0.052%	0.049%
HP #3 Medium	0.010%	0.027%	0.036%	0.037%
HP #3 Medium Remix	0.012%	0.025%	0.035%	-
HP #3 Medium w/LWA	0.007%	0.021%	0.024%	0.021%
HP #3 Low	0.016%	0.024%	0.035%	0.043%
HP #3 Low w/LWA	0.008%	0.017%	0.019%	0.023%
HP #3 Gravels	0.020%	0.045%	0.040%	-
HP #3 Gravels w/LWA	0.011%	0.028%	-	-
HP #3 Blended High	0.011%	0.022%	0.029%	0.034%
HP #3 Blended Medium	0.006%	0.019%	0.023%	0.035%
HP #3 Blended Low	0.021%	0.033%	0.039%	0.043%
HP #4 High	0.091%	0.080%	0.081%	0.074%
HP #4 Medium	0.008%	0.018%	0.024%	0.022%
HP #4 Low	0.010%	0.016%	0.026%	0.041%
HP #4 Low w/ LWA	0.008%	0.016%	0.020%	0.031%
HP #4 Gravels	0.011%	0.020%	0.030%	0.034%
HP #4 Blended High	0.013%	0.017%	0.022%	0.025%
HP #4 Blended Medium	0.009%	0.013%	0.020%	0.024%
HP #4 Blended Low	0.013%	0.028%	0.037%	0.042%
HP #4 Blended Gravels	0.011%	0.025%	0.036%	0.037%
HP #4 Modified High	0.013%	0.016%	0.035%	0.037%
HP #4 Modified Medium	0.008%	0.013%	0.018%	0.016%
HP #4 Modified Medium w/LWA	0.020%	0.020%	0.018%	0.027%
HP #4 Modified Low	0.008%	0.021%	0.030%	0.029%
HP #4 Modified Low w/LWA	0.011%	0.013%	0.015%	0.055%
HP #4 Modified Gravels	0.009%	0.018%	0.024%	0.028%
Class C Option 1 High	0.002%	0.015%	0.020%	0.026%
Class C Option 1 Medium	0.015%	0.017%	-	0.029%
Class C Option 1 Low	0.001%	0.040%	0.046%	0.047%
Class C Option 1 Low w/LWA	0.011%	0.025%	0.022%	0.029%
Class S High	0.010%	0.014%	0.022%	0.027%
Class S Medium	0.015%	0.025%	0.037%	0.044%
Class S Low	0.024%	0.033%	0.041%	0.048%
Microsilica Overlay High w/Fibers	0.011%	0.025%	0.024%	0.025%
Microsilica Overlay High	0.028%	0.087%	0.028%	0.022%
Microsilica Overlay High w/LWA	0.004%	0.020%	-	0.016%

Table 30: Percent Shrinkage of Unsealed Samples in Moist Cure Room

Mix Identification	Percent Length Change			
	7 Day	28 Day	56 Day	90 Day
HP #3 High	0.036%	0.059%	0.076%	0.076%
HP #3 High Remix	0.033%	0.061%	0.065%	0.061%
HP #3 Medium	0.004%	0.023%	0.028%	0.023%
HP #3 Medium Remix	0.028%	0.044%	0.048%	-
HP #3 Medium w/LWA	0.027%	0.054%	0.059%	0.074%
HP #3 Low	0.025%	0.034%	0.049%	0.060%
HP #3 Low w/LWA	0.024%	0.044%	0.047%	0.052%
HP #3 Gravels	0.073%	0.048%	0.059%	-
HP #3 Gravels w/ LWA	0.026%	0.049%	-	-
HP #3 Blended High	0.030%	0.046%	0.051%	0.054%
HP #3 Blended Medium	0.022%	0.044%	0.047%	0.059%
HP #3 Blended Low	0.065%	0.052%	0.047%	0.044%
HP #4 High	0.032%	0.059%	0.047%	0.062%
HP #4 Medium	0.024%	0.045%	0.053%	0.049%
HP #4 Low	0.029%	0.036%	0.052%	0.056%
HP #4 Low w/ LWA	0.017%	0.028%	0.043%	0.051%
HP #4 Gravels	0.029%	0.046%	0.060%	0.059%
HP #4 Blended High	0.030%	0.047%	0.054%	0.060%
HP #4 Blended Medium	0.023%	0.033%	0.042%	0.048%
HP #4 Blended Low	0.028%	0.045%	0.056%	0.062%
HP #4 Blended Gravels	0.027%	0.047%	0.056%	0.057%
HP #4 Modified High	0.033%	0.044%	0.052%	0.060%
HP #4 Modified Medium	0.020%	0.036%	0.042%	0.037%
HP #4 Modified Medium w/LWA	0.018%	0.040%	0.042%	0.049%
HP #4 Modified Low	0.024%	0.037%	0.065%	0.140%
HP #4 Modified Low w/LWA	0.024%	0.029%	0.034%	0.044%
HP #4 Modified Gravels	0.025%	0.048%	0.051%	0.048%
Class C Option 1 High	0.011%	0.033%	0.035%	0.061%
Class C Option 1 Medium	0.019%	0.032%	-	0.037%
Class C Option 1 Low	0.000%	0.056%	0.055%	0.045%
Class C Option 1 Low w/LWA	0.035%	0.055%	0.052%	0.059%
Class S High	0.024%	0.022%	0.043%	0.068%
Class S Medium	0.025%	0.039%	0.051%	0.059%
Class S Low	0.032%	0.044%	0.058%	0.063%
Microsilica Overlay High w/Fibers	0.032%	0.045%	0.039%	0.057%
Microsilica Overlay High	0.028%	0.051%	0.074%	0.062%
Microsilica Overlay High w/LWA	0.030%	0.059%	-	0.059%

Restrained Shrinkage

The restrained shrinkage test was carried out following ASTM C-1581 for the mixes using a #8 coarse aggregate. A modified version was used to test the mixtures with #57 course aggregate. This test method allowed the researchers to determine the time to crack, the stress rate at cracking and the maximum tensile stress at cracking. The procedure developed by See et al. (2002) was used.

For each mixture, two restrained shrinkage rings were molded. Each ring was instrumented with two strain gages, and the information from each gage was averaged. A data logging system was used to record the output from the strain gages at half hour intervals. This data was then transferred to a spreadsheet and plotted as strain versus time. A typical plot is shown in Figure 34.

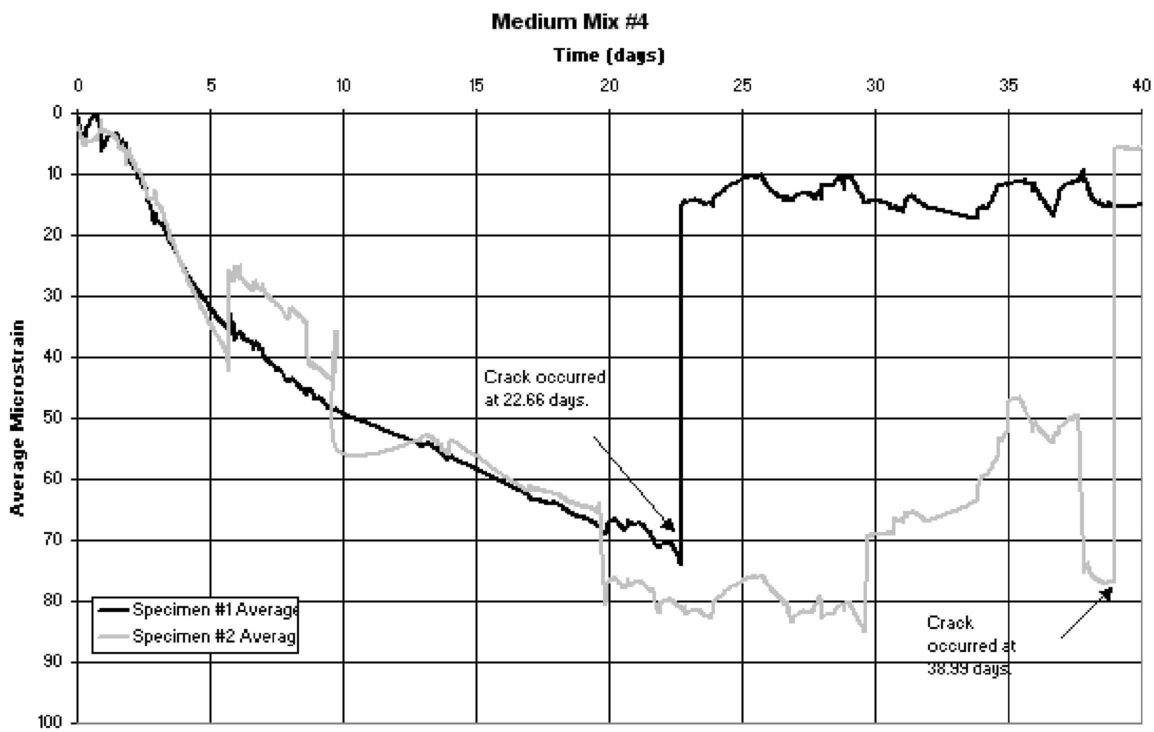


Figure 34: Typical strain versus time to cracking from ring test

The time to crack and the maximum strain in the steel ring was determined from the respective plots. The stress rate at cracking was determined from Equation 8 (ASTM C 1581):

Equation 8: Strain Rate Factor

$$S(t) = \frac{G|\alpha|}{2\sqrt{t}}$$

Where: α = strain rate factor
G = parameter governed by the setup of the ring
t = time to cracking

The strain rate factor (α) is determined from a plot of the square root of time verses the average strain. A typical plot is shown in Figure 35.

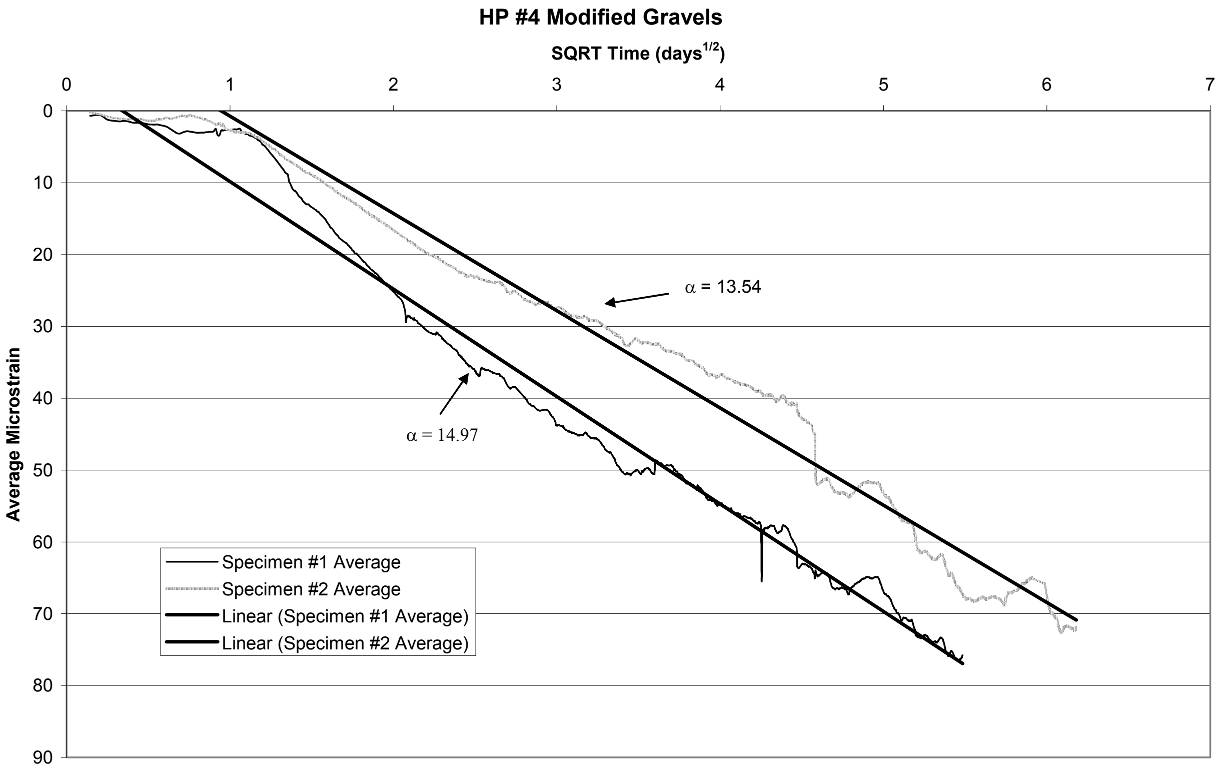


Figure 35: Typical determination of factor α

The value of the parameter G is a function of the stiffness of the steel and of the respective thicknesses of the steel and concrete, and can be determined from Equation 9 (See et al., 2002):

Equation 9: G governed by ring setup

$$G = \frac{E_{st} r_{ic} h_{st}}{r_{is} h_c}$$

Where: E_{st} = Modulus of Elasticity of the Steel (29 x 10^6 psi) (200 GPa)
r_{ic} = Internal radii of the concrete (6 ½ inches) (165 mm)
r_{is} = Internal radii of the steel (6 inches) (152 mm)
h_{st} = Thickness of the steel (½ inch) (13 mm)
h_c = Thickness of the concrete (1 ½ and 2 ½ inches) (38 and 64 mm)

The value of G for the mixtures with the #8 coarse aggregate was determined to be 10.47 x 10^6 psi (74.3 GPa). The mixtures with the #57 coarse aggregate was determined to have a value of 6.28 x 10^6 psi (44.1 GPa) for G. The reason for the difference in the value of G was due to the thicker ring of concrete (2.5 inches, 64 mm) for the mixtures that contained the number fifty-seven coarse aggregate. The parameter G was used to determine the maximum induced tensile stress in the concrete specimen. This tensile stress was determined from Equation 10 (ASTM C 1581):

Equation 10: Maximum Induced Tensile Stress

$$\sigma_{\max} = G\left|\varepsilon_{\max}\right|$$

Where: ε_{max} = maximum strain of the steel at time of cracking.

Table 31 and Table 32 provide a summary of the results for the high performance mixtures, and Table 33 provides a summary for the Class S and Class C mixtures. The data indicate that the blended high performance mixtures did not crack in most cases. The difference in time to crack for the 1.5 inch and 2.5 inch (38 and 64 mm) concrete rings will be discussed in the next chapter.

The potential for cracking is classified in two ways in ASTM C 1581. The first is by the time to crack, and the second is by the average stress rate at cracking. Table 34 contains a summary of criteria from ASTM C 1581. Most of the blended high performance mixtures fell within the low potential for cracking classification using the average stress rate criteria. The only exception to this was the HP #4 Blended mixture with gravels. It was found that gravels performed poorly and were susceptible to cracking. A majority of the high performance mixtures with #8 coarse aggregate fell within and below the Moderate-Low classification.

Table 31: Restrained Shrinkage Ring Data for HP #3 Mixtures

Mixture	Concrete Ring Thickness in (mm)	Time To Cracking days	Crack Width in (mm)	Steel Ring Strain at Cracking (micro-strain)	Stress Rate at Cracking S psi/day(kPa/d)	Maximum Tensile Stress psi (MPa)
HP #3 High	1.5 (38)	7.89	0.040(1.016)	37.32	25.5 (176)	391 (2.70)
HP #3 High	1.5 (38)	10.97	0.040(1.016)	52.74	33.7 (232)	552 (3.81)
HP #3 High Remix	1.5 (38)	55.90	0.016(0.406)	96.17	7.7 (53)	1007(6.94)
HP #3 High Remix	1.5 (38)	90.00	No Crack	82.48	3.2 (22)	864 (5.96)
HP #3 High Comparison	1.5 (38)	21.72	0.016(0.406)	199.72	60.5 (417)	2092(14.4)
HP #3 High Comparison	1.5 (38)	22.47	0.020(0.508)	91.88	26.8 (185)	962 (6.63)
HP #3 High Comparison	2.5 (64)	90.00	No Crack	-	-	-
HP #3 High Comparison	2.5 (64)	16.00	0.020(0.508)	83.35	22.1 (152)	524 (3.61)
HP #3 Medium	1.5 (38)	36.76	0.010(0.254)	92.53	14.0 (97)	969 (6.68)
HP #3 Medium	1.5 (38)	90.00	No Crack	135.72	2.8 (19)	1421 (9.8)
HP #3 Medium Comparison	1.5 (38)	90.00	No Crack	11.43	-	120 (0.83)
HP #3 Medium Comparison	1.5 (38)	90.00	No Crack	48.27	-	505 (3.48)
HP #3 Medium Comparison	2.5 (64)	90.00	No Crack	82.51	5.0 (34)	864 (5.96)
HP #3 Medium Comparison	2.5 (64)	90.00	No Crack	74.40	4.1 (28)	779 (5.37)
HP #3 Medium w/LWA	1.5 (38)	90.00	No Crack	-	-	-
HP #3 Medium w/LWA	1.5 (38)	86.27	0.016(0.406)	87.73	4.7 (32)	919 (6.34)
HP #3 Medium w/LWA	2.5 (64)	40.16	0.025(0.635)	71.89	6.6 (46)	452 (3.12)
HP #3 Medium w/LWA	2.5 (64)	90.00	No Crack	108.10	4.7 (32)	679 (4.68)
HP #3 Low	1.5 (38)	37.03	0.010(0.254)	72.79	14.6 (101)	762 (5.25)
HP #3 Low	1.5 (38)	33.99	0.010(0.254)	92.59	12.8 (88)	970 (6.69)
HP #3 Low w/ LWA	1.5 (38)	24.45	0.016(0.406)	62.44	9.0 (62)	654 (4.51)
HP #3 Low w/ LWA	1.5 (38)	81.40	0.013(0.330)	72.80	9.2 (63)	762 (5.25)
HP #3 Low w/ LWA	2.5 (64)	90.00	No Crack	128.98	3.8 (26)	810 (5.58)
HP #3 Low w/ LWA	2.5 (64)	90.00	No Crack	118.54	3.7 (25)	745 (5.14)
HP #3 Gravels	1.5 (38)	20.49	0.016(0.406)	51.73	15.9 (110)	542 (3.74)
HP #3 Gravels	1.5 (38)	51.00	0.030(0.762)	-	-	-
HP #3 Gravels	1.5 (38)	90.00	No Crack	107.40	3.0 (21)	675 (4.65)
HP #3 Gravels	1.5 (38)	59.00	0.025(0.635)	93.93	4.3 (30)	590 (4.07)
HP #3 Gravels w/LWA	1.5 (38)	>56	0.030(0.762)	-	-	-
HP #3 Gravels w/LWA	1.5 (38)	>56	0.025(0.635)	-	-	-
HP #3 Gravels w/LWA	2.5 (64)	90.00	No Crack	-	-	-
HP #3 Gravels w/LWA	2.5 (64)	36.00	0.016(0.406)	-	-	-

Table 32: Restrained Shrinkage Ring Data for HP #4 and HP Blended Mixtures

Mixture	Concrete Ring Thickness in (mm)	Time To Cracking days	Crack Width in (mm)	Steel Ring Strain at Cracking (micro-strain)	Stress Rate at Cracking S psi/day(kPa/d)	Maximum Tensile Stress psi (MPa)
HP #4 High	1.5 (38)	12.96	0.050(1.27)	54.11	14.6 (101)	567 (3.91)
HP #4 High	1.5 (38)	12.01	0.050(1.27)	33.47	16.2 (112)	351 (2.42)
HP #4 High Remix	1.5 (38)	12.33	0.030(0.762)	66.28	38.2 (263)	694 (4.78)
HP #4 High Remix	1.5 (38)	11.31	0.030(0.762)	11.31	40.0 (276)	691 (4.76)
HP #4 High Remix	2.5 (64)	15.06	0.050(1.27)	49.82	12.2 (84)	313 (2.16)
HP #4 High Remix	2.5 (64)	21.00	0.040(1.016)	66.58	12.2 (84)	418 (2.88)
HP #4 Medium	1.5 (38)	22.68	0.010(0.254)	73.95	19.4 (134)	774 (5.34)
HP #4 Medium	1.5 (38)	38.99	0.013(0.330)	76.72	10.5 (72)	803 (5.54)
HP #4 Low	1.5 (38)	22.67	0.040(1.016)	68.07	13.7 (94)	713 (4.92)
HP #4 Low	1.5 (38)	37.40	0.010(0.254)	93.13	11.4 (79)	975 (6.72)
HP #4 Low w/ LWA	1.5 (38)	18.61	0.020(0.508)	66.46	23.0 (159)	696 (4.80)
HP #4 Low w/ LWA	1.5 (38)	60.90	0.016(0.406)	78.39	6.2 (43)	821 (5.66)
HP #4 Gravels	1.5 (38)	32.72	0.003(0.076)	19.75	3.6 (25)	207 (1.43)
HP #4 Gravels	1.5 (38)	14.05	0.025(0.635)	47.13	22.5 (155)	494 (3.41)
HP #4 Modified High	2.5 (64)	90.00	No Crack	42.42	1.6 (11)	267 (1.84)
HP #4 Modified High	2.5 (64)	50.81	0.035(0.889)	35.65	2.4 (17)	224 (1.54)
HP #4 Modified Med	2.5 (64)	90.00	No Crack	59.87	-	376 (2.59)
HP #4 Modified Med	2.5 (64)	90.00	No Crack	66.05	2.7 (19)	415 (2.86)
HP #4 Modified Medium w/LWA	2.5 (64)	90.00	No Crack	-	-	-
HP #4 Modified Medium w/LWA	2.5 (64)	90.00	No Crack	45.76	1.5 (10)	288 (1.99)
HP #4 Modified Low	2.5 (64)	90.00	No Crack	73.17	2.5 (17)	460 (3.17)
HP #4 Modified Low	2.5 (64)	90.00	No Crack	112.10	4.0 (28)	704 (4.85)
HP #4 Modified Low w/ LWA	2.5 (64)	90.00	No Crack	82.09	2.6 (18)	516 (3.56)
HP #4 Mod Low w/LWA	2.5 (64)	90.00	No Crack	80.96	2.6 (18)	509 (3.51)
HP #4 Mod Gravels	2.5 (64)	30.07	0.011(0.276)	75.77	8.6 (59)	476 (3.28)
HP #4 Mod Gravels	2.5 (64)	38.18	0.003(0.076)	71.83	6.9 (48)	451 (3.11)
HP #3 Blended High	2.5 (64)	90.00	No Crack	71.53	2.4 (17)	449 (3.10)
HP #3 Blended High	2.5 (64)	90.00	No Crack	88.98	2.9 (20)	559 (3.85)
HP #3 Blended Med	2.5 (64)	90.00	No Crack	119.38	3.9 (27)	750 (5.17)
HP #3 Blended Med	2.5 (64)	90.00	No Crack	76.19	1.9 (13)	479 (3.30)
HP #3 Blended Low	2.5 (64)	90.00	No Crack	108.58	3.0 (21)	682 (4.7)
HP #3 Blended Low	2.5 (64)	90.00	No Crack	82.03	2.2 (15)	515 (3.55)
HP #4 Blended High	2.5 (64)	57.27	0.025(0.635)	106.67	6.4 (44)	670 (4.62)
HP #4 Blended High	2.5 (64)	77.88	0.030(0.762)	99.81	5.4 (37)	627 (4.32)
HP #4 Blended Med	2.5 (64)	74.80	0.020(0.508)	125.76	4.8 (33)	790 (5.45)
HP #4 Blended Med	2.5 (64)	76.91	0.011(0.279)	63.72	2.6 (18)	400 (2.76)
HP #4 Blended Low	2.5 (64)	90.00	No Crack	109.38	3.5 (24)	687 (4.74)
HP #4 Blended Low	2.5 (64)	83.00	0.016(0.406)	-	-	-
HP #4 Blend Gravels	2.5 (64)	19.97	0.007(0.178)	93.76	17.9 (123)	589 (4.06)
HP #4 Blend Gravels	2.5 (64)	19.85	0.040(1.016)	76.78	14.2 (98)	482 (3.32)

Table 33: Restrained Shrinkage Ring Data for Class S and Class C Mixtures

Mixture	Concrete Ring Thickness in (mm)	Time To Cracking days	Crack Width in (mm)	Steel Ring Strain at Cracking (micro-strain)	Stress Rate at Cracking S psi/day(kPa/d)	Maximum Tensile Stress psi (MPa)
Class S High	2.5 (64)	69.42	0.020(0.508)	46.21	2.6 (18)	670 (4.62)
Class S High	2.5 (64)	90.00	No Crack	98.20	3.6 (25)	617 (4.25)
Class S Medium	2.5 (64)	59.88	0.016(0.406)	81.02	4.3 (30)	509 (3.51)
Class S Medium	2.5 (64)	35.79	0.025(0.635)	77.14	7.4 (51)	485 (3.34)
Class S Low	2.5 (64)	90.00	No Crack	85.43	2.1 (14)	537 (3.70)
Class S Low	2.5 (64)	57.03	0.013(0.330)	63.72	3.3 (23)	400 (2.76)
Class C Option 1 High	2.5 (64)	104.68	No Crack	84.84	2.9 (20)	533 (3.67)
Class C Option 1 High	2.5 (64)	104.68	No Crack	85.49	3.1 (21)	537 (3.70)
Class C Option 1 Medium	2.5 (64)	87.95	No Crack	-	-	-
Class C Option 1 Medium	2.5 (64)	87.95	No Crack	15.51	0.6 (4)	97 (0.67)
Class C Option 1 Low	2.5 (64)	103.84	No Crack	144.43	7.4 (51)	908 (6.26)
Class C Option 1 Low	2.5 (64)	103.84	No Crack	100.88	2.8 (19)	634 (4.37)
Class C Option 1 Low w/ LWA	2.5 (64)	95.93	No Crack	2.77	-	17 (0.12)
Class C Option 1 Low w/ LWA	2.5 (64)	95.93	No Crack	-	-	-
Microsilica Overlay High w/ Fibers	2.5 (64)	29.20	0.020(0.508)	139.24	22.5 (155)	1458 (10.05)
Microsilica Overlay High w/ Fibers	2.5 (64)	90.00	No Crack	73.62	3.2 (22)	771 (5.32)
Microsilica Overlay High	2.5 (64)	51.23	0.016(0.406)	82.51	7.5 (52)	864 (5.96)
Microsilica Overlay High	2.5 (64)	39.53	0.025(0.635)	45.82	10.3 (71)	480 (3.31)
Microsilica Overlay High w/ LWA	2.5 (64)	18.15	No Crack	60.44	22.7 (157)	633 (4.36)
Microsilica Overlay High w/ LWA	2.5 (64)	18.90	0.013(0.330)	36.16	11.5 (79)	379 (2.61)

Table 34: ASTM C 1581 Potential for Cracking

Net Time-to-Cracking, t_{cr} (days)	Average Stress Rate S (psi/day)	Potential for Cracking
$0 < t_{cr} \leq 7$	$S \geq 50$	High
$7 < t_{cr} \leq 14$	$25 \leq S < 50$	Moderate-High
$14 < t_{cr} \leq 28$	$15 \leq S < 25$	Moderate-Low
$t_{cr} > 28$	$S < 15$	Low

FIELD TESTING

In order to verify the laboratory results, a small field testing program was carried out. The field testing was coordinated with Dale Crowl of ODOT District 12. The samples from bridge deck and pavement projects were obtained from the concrete batch plants. The samples were then transported back to the concrete laboratory at Cleveland State University, where the fresh concrete tests were performed and samples were prepared.

For each mixture, two ring specimens were prepared along with two beam specimens, four concrete prisms and twenty cylinders. The beams were broken at three and twenty-eight days and the other tests were performed as usual. The results of the testing were then compared to the results from the same mixtures made in the laboratory and the results were recorded. A total of twenty cylinders were molded for each sample collected. A summary of the tests performed and the standards followed is provided in Table 35.

Table 35: Tests Performed and Standards Followed

Fresh Concrete Properties	
Workability	ASTM C 143
Air Content	ASTM C 231
Unit Weight	ASTM C 138
Hardened Concrete Properties	
Compressive Strength	ASTM C 39
Splitting Tensile Strength	ASTM C 496
Modulus of Elasticity	ASTM C 469
Modulus of Rupture	ASTM C 78
Shrinkage and Cracking Tendency	
Unrestrained Shrinkage Bars	ASTM C 157
Restrained Shrinkage Rings	AASHTO PP34-99

Observations During Field Testing

The first field test was conducted on Wednesday June 28th 2006 at approximately 6:00 am. A sample of concrete was obtained for the deck of the West 53rd street Bridge over the Norfolk Southern Railroad tracks. The concrete was supplied by Cuyahoga Concrete and was ODOT's HP #4 Modified mixture. The aggregate used by Cuyahoga was Marblehead limestone coarse aggregate. Marblehead is considered a high absorption aggregate.

The second field test was a concrete sample from the deck for the West 143rd street Bridge over Interstate 71 and was conducted on Thursday June 29th 2006 at 10:00 pm. This deck also used ODOT's HP #4 Modified mixture supplied by Tech Ready Mix. The aggregate used was Calcite limestone coarse aggregate. Calcite is a medium absorption aggregate.

The third field test was performed at 8:00 am on Wednesday July 19th 2006. The concrete was ODOT's Class C Option 1 mixture and was supplied by Tech Ready Mix. This mixture also used Calcite limestone coarse aggregate. The sample was for a slip form paving job at West 14th and Holmden. Table 36 documents the weather conditions during each field test.

Table 36: Temperature and Humidity Information

Field Test	#1	#2	#2	#3
Location	W 53rd Street	W 143rd Street	W 143rd Street	W 14th Street
Date	6/28/06	6/29/06	6/30/06	7/19/06
Time of Pour	6:00 AM	10:00 PM	-	8:00 AM
Maximum Temperature	80° F (27°C)	74° F (23°C)	78° F (26°C)	86° F (30°C)
Time	3:29 PM	5:35 PM	5:37 PM	2:06 PM
Minimum Temperature	62° F (17°C)	59° F (15°C)	57° F (14°C)	64° F (18°C)
Time	11:59 PM	6:19 AM	4:52 AM	4:48 AM
Average Temperature	72° F (22°C)	67° F (19°C)	68° F (20°C)	75° F (24°C)
Maximum Humidity	93%	84%	87%	87%
Time	1:00 AM	7:00 AM	6:00 AM	4:00 AM
Minimum Humidity	42%	46%	39%	43%
Time	3:00 PM	5:00 PM	3:00 PM	12:00 PM
Average Humidity	68%	65%	63%	65%
Highest Wind Speed	32 MPH (51 km/h)	17 MPH (27 km/h)	15 MPH (24 km/h)	17 MPH (27 km/h)
Average Wind Speed	7.6 MPH (12 km/h)	9.3 MPH (15 km/h)	5.8 MPH (9.3 km/h)	5.6 MPH (9 km/h)

Field Test Results

The hardened concrete properties were tested in the laboratory, and the results are provided in Table 37 through Table 41. The results from the same mixtures prepared in the laboratory have also been included.

The results for modulus of rupture of the bridge decks were lower than the respective lab specimens in both instances. The pavement mixture, however, showed higher values for the field specimens. The compressive strengths and the splitting tensile strengths of the field and lab specimens were comparable, with some variations. The modulus of elasticity for each test was also comparable, with no clear trends noted.

The unrestrained shrinkage specimens followed the expected trends, with samples in the moist cure room experiencing more shrinkage than the specimens in the lime bath. There were differences, however in the amount of shrinkage between the field specimens and the lab specimens.

Table 37: Modulus of Rupture for Field Testing

Mix Identification	Modulus of Rupture			
	7 Day		28 Day	
	psi	MPa	psi	MPa
Field Test #1: HP #4 Modified High	667	4.60	850	5.86
Lab Test: HP #4 Modified High	933	6.43	1125	7.76
Field Test #2: HP #4 Modified Medium	825	5.69	767	5.29
Lab Test: HP #4 Modified Medium	904	6.23	1125	7.76
Field Test #3: Class C Option 1 Medium	613	4.23	775	5.34
Lab Test: Class C Option 1 Medium	604	4.16	667	4.60

Table 38: Compressive Strength for Field Data

Mix Identification	Compressive Strength							
	7 Day		28 Day		56 Day		90 Day	
	psi	MPa	psi	MPa	psi	MPa	psi	MPa
Field Test #1: HP #4 Modified High	5243	36.15	7395	50.99	7451	51.37	7311	50.41
Lab Test: HP #4 Modified High	6385	44.02	8105	55.88	8379	57.77	8607	59.34
Field Test #2: HP #4 Modified Medium	6718	46.32	9025	62.23	9569	65.98	9400	64.81
Lab Test: HP #4 Modified Medium	6197	42.73	8380	57.78	8797	60.65	9447	65.13
Field Test #3: Class C Option 1 Medium	3729	25.71	4816	33.21	5590	38.54	5968	41.15
Lab Test: Class C Option 1 Medium	3982	27.45	5075	34.99	5544	38.22	5963	41.11

Table 39: Splitting Tensile Strength for Field Data

Mix Identification	Splitting Tensile Strength							
	7 Day		28 Day		56 Day		90 Day	
	psi	MPa	psi	MPa	psi	MPa	psi	MPa
Field Test #1: HP #4 Modified High	645	4.45	610	4.21	611	4.21	500	3.45
Lab Test: HP #4 Modified High	519	3.58	507	3.50	694	4.78	609	4.20
Field Test #2: HP #4 Modified Medium	742	5.12	704	4.85	595	4.10	657	4.53
Lab Test: HP #4 Modified Medium	495	3.41	537	3.70	545	3.76	756	5.21
Field Test #3: Class C Option 1 Medium	477	3.29	567	3.91	635	4.38	592	4.08
Lab Test: Class C Option 1 Medium	463	3.19	421	2.90	591	4.07	716	4.94

Table 40: Static Modulus of Elasticity for Field Data

Mix Identification	Modulus of Elasticity							
	7 Day		28 Day		56 Day		90 Day	
	10^6 psi	MPa	10^6 psi	MPa	10^6 psi	MPa	10^6 psi	MPa
Field Test #1: HP #4 Modified High	3.66	25235	3.93	27096	4.02	27717	3.31	22822
Lab Test: HP #4 Modified High	3.94	27165	4.30	29647	3.67	25304	4.14	28544
Field Test #2: HP #4 Modified Medium	4.73	32612	5.16	35577	5.35	36887	4.94	34060
Lab Test: HP #4 Modified Medium	3.82	26338	4.72	32543	4.69	32336	4.99	34405
Field Test #3: Class C Option 1 Medium	3.93	27096	4.25	29303	4.37	30130	4.33	29854
Lab Test: Class C Option 1 Medium	3.91	26958	4.02	27717	4.50	31026	4.96	34198

Table 41: Unrestrained Shrinkage for Field Specimens

Mix Identification	Percent Length Change			
	7 Day	28 Day	56 Day	90 Day
Sealed Sample in Moist Cure Room				
Field Test #1: HP #4 Modified High	0.006%	0.006%	0.010%	0.013%
Lab Test: HP #4 Modified High	0.013%	0.016%	0.035%	0.037%
Field Test #2: HP #4 Modified Medium	0.016%	0.028%	0.039%	0.040%
Lab Test: HP #4 Modified Medium	0.008%	0.013%	0.018%	0.016%
Field Test #3: Class C Option 1 Medium	0.089%	0.069%	-	0.041%
Lab Test: Class C Option 1 Medium	0.015%	0.017%	-	0.029%
Unsealed Sample in Moist Cure Room				
Field Test #1: HP #4 Modified High	0.029%	0.084%	0.153%	0.159%
Lab Test: HP #4 Modified High	0.033%	0.044%	0.052%	0.060%
Field Test #2: HP #4 Modified Medium	0.058%	0.044%	0.035%	0.032%
Lab Test: HP #4 Modified Medium	0.020%	0.036%	0.042%	0.037%
Field Test #3: Class C Option 1 Medium	0.122%	0.054%	-	0.036%
Lab Test: Class C Option 1 Medium	0.019%	0.032%	-	0.037%
Sealed Sample in Lime Bath				
Field Test #1: HP #4 Modified High	0.004%	0.000%	0.002%	0.002%
Lab Test: HP #4 Modified High	0.006%	0.012%	0.011%	0.008%
Field Test #2: HP #4 Modified Medium	0.002%	0.001%	0.002%	0.001%
Lab Test: HP #4 Modified Medium	0.000%	0.001%	0.000%	0.004%
Field Test #3: Class C Option 1 Medium	0.000%	0.000%	-	0.001%
Lab Test: Class C Option 1 Medium	0.011%	0.005%	-	0.003%
Unsealed Sample in Lime Bath				
Field Test #1: HP #4 Modified High	0.001%	0.011%	0.007%	0.007%
Lab Test: HP #4 Modified High	0.001%	0.002%	0.003%	0.001%
Field Test #2: HP #4 Modified Medium	0.003%	0.001%	0.004%	0.002%
Lab Test: HP #4 Modified Medium	0.001%	0.001%	0.001%	0.005%
Field Test #3: Class C Option 1 Medium	0.007%	0.004%	-	0.001%
Lab Test: Class C Option 1 Medium	0.004%	0.015%	-	0.014%

DISCUSSION

Concrete Strengths

As stated previously, all of the specimens tested in compression exceeded the Ohio Department of Transportation requirement of 4,500 psi (31 MPa) after seven days. ODOT's requirement of 4,500 psi (31 MPa) is for twenty-eight day strength. The addition of lightweight fine aggregate increased the compressive strength of the specimens at every age. In theory, lightweight fine aggregate has the capacity to hold more moisture in its pores than traditional fine aggregate. This additional moisture would slowly be released to the cement particles helping to better hydrate the cement and increase the compressive strength. The partial replacement of lightweight fine aggregate increased the compressive strength by approximately 1,000 psi (6.9 MPa).

Splitting tensile strength had more variability than the compressive strength. Mindess et al. (2002) cite the following equation to correlate concrete compressive strength to splitting tensile strength:

Equation 11: Splitting tensile strength estimated by compressive strength

$$f_{sp} = 4.34(f_c')^{0.55} \text{ (psi)}$$
$$f_{sp} = 0.305(f_c')^{0.55} \text{ (MPa)}$$

When applying Equation 11 to seven day strength concrete, it fit the data well. At later ages, the equation did not fit as well. By ninety days, the measured splitting tensile cylinders had greater values than predicted. This relationship is shown in Figure 36 for the HP mixtures.

Unrestrained Shrinkage

The addition of LWA decreased the amount of unrestrained shrinkage. Bisschop (2002) concluded that the aggregate size had no effect on the drying shrinkage of concrete. The drying shrinkage is very similar between blended aggregate and straight #8 aggregate mixtures. The mixtures with LWA had slightly less drying shrinkage than the mixtures with only #8 coarse aggregates. As was predicted, the sealed specimens also had less shrinkage than unsealed specimens. The sealed specimens isolate autogenous shrinkage from drying shrinkage.

Restrained Shrinkage

Based on the criteria given in Table 34, the cracking potential was categorized by each concrete mixture. The categories are "high", "moderate–high", "moderate–low", "low", and "very low" and are shown in Table 42. These categories are based upon the work of See (2002).

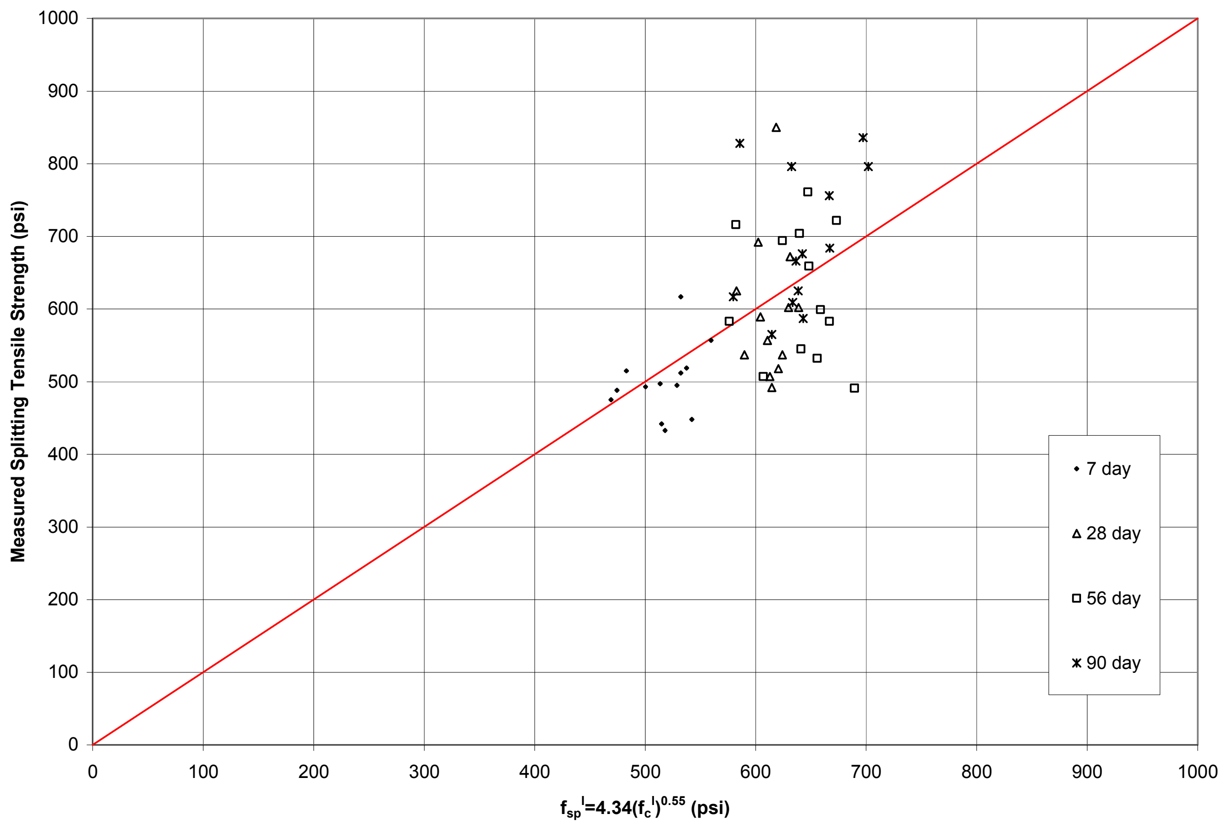


Figure 36: Measured vs. predicted splitting tensile strength (HP mixtures)

Table 42: Cracking potential classification, based on stress rate at cracking (See 2002)

Net Time to Cracking, t_{cr}, (day)	Stress Rate at Cracking, S, psi/day (kPa/day)	Potential for Cracking
$0<t_{cr}\leq 7$	$S\geq 49.3$ ($S\geq 339.9$)	High
$7<t_{cr}\leq 14$	$24.65\leq S<49.3$ ($170.0\leq S<339.9$)	Moderate - High
$14<t_{cr}\leq 28$	$14.5\leq S<24.65$ ($100.0\leq S<170.0$)	Moderate – Low
$t_{cr}\geq 28$	$S<14.$ ($S<100.0$)	Low

See's tests ended at twenty-eight days. The tests documented in this report continued for ninety days. Many of these specimens cracked between twenty-eight and ninety days. However, these concrete mixtures may not necessarily crack in the field, under different curing conditions.

An additional category was created to try to better represent the data points that did not crack after ninety days. The "very low" classification was based upon the Net Time to Cracking, t_{cr}, greater than fifty-six days and Stress Rate at Cracking, S, less than 9 psi/day (62 kPa/day). Figure 37 displays the new cracking criteria, along with the data collected during this research project for HP mixtures. The classifications for the HP mixtures are shown in Table 43.

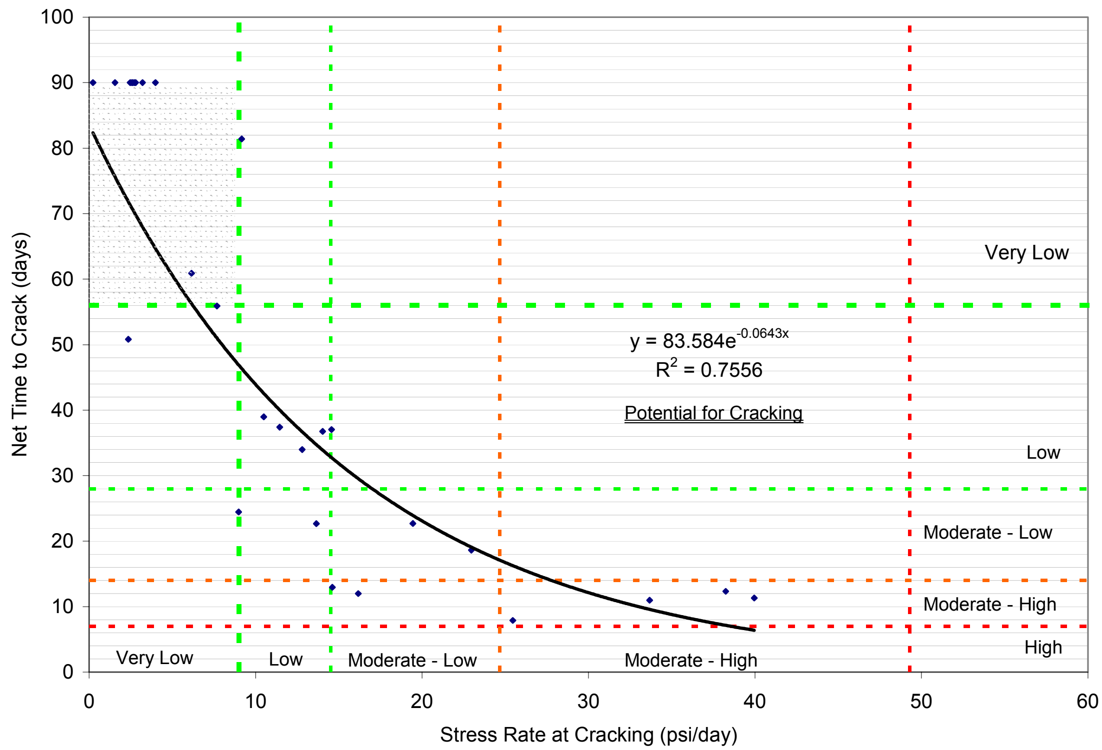


Figure 37: Net time to cracking versus stress rate at cracking

Ring specimens made with blended concrete mixtures tended to be much more resistant to cracking than high performance concrete mixtures with only #8 aggregate. Blended mixtures received either "low" or "very low" categorization.

Lightweight fine aggregate improved the resistance to cracking with low absorption aggregates. In most cases, the lightweight mixtures improved the concrete by one categorization level. Many of these made with lightweight aggregate were classified as "very low."

Comparison of Ring Geometry

The thickness of the concrete ring in the experiment was varied out of necessity. ASTM C 1581 indicates that a 16 inch (406 mm) outside diameter ring should be used to mold a specimen and has a maximum nominal size coarse aggregate of 0.5 inches (13 mm). For the mixtures with #8 coarse aggregate, 3/8 inch (9.5 mm) nominal maximum size, the 16 inch (406 mm) outside diameter ring worked, but mixtures containing #57 coarse aggregate, 1 inch (25 mm) nominal maximum size, a larger outside diameter was required. An 18 inch (457 mm) mold was chosen for these mixtures. The time to cracking is dependent on specimen geometry, and a correction factor was determined from experimental and theoretical analysis.

If it is assumed that the ratio of the maximum induced tensile stress to the maximum strain in the steel ring is constant for different ring geometries with the same mixture, then the parameter G can be used to determine a correction factor. It was determined that the larger (18 inch, 457 mm) rings should take approximately 1.67 times longer to crack than the smaller rings (16 inch, 406 mm).

Table 43: Summary of restrained shrinkage ring test classification for HP mixtures

Mix ID	Time to Cracking, t_{cr} (days)	Stress Rate at Cracking, S psi/day (kPa/day)
HP #3 High Specimen #1	Moderate - High	Moderate - High
HP #3 High Specimen #2	Moderate - High	Moderate - High
HP #3 High Re-mix Specimen #1	Low	Very Low
HP #3 High Re-mix Specimen #2	Very Low	Very Low
HP #4 High Specimen #1	Moderate - High	Moderate - Low
HP #4 High Specimen #2	Moderate - High	Moderate - Low
HP #4 High Re-mix Specimen #1	Moderate - High	Moderate - High
HP #4 High Re-mix Specimen #2	Moderate - High	Moderate - High
HP Blend High Specimen #1	Very Low	Very Low
HP Blend High Specimen #2	Low	Very Low
HP #3 Medium Specimen #1	Low	Low
HP #3 Medium Specimen #2	Very Low	Very Low
HP #4 Medium Specimen #1	Moderate - Low	Moderate - Low
HP #4 Medium Specimen #2	Low	Low
HP Blend Medium Specimen #1	Very Low	Very Low
HP Blend Medium Specimen #2	Very Low	Very Low
HP #3 Low Specimen #1	Low	Moderate - Low
HP #3 Low Specimen #2	Low	Low
HP #3 Low with LWA Specimen #1	Moderate - Low	Very Low
HP #3 Low with LWA Specimen #2	Very Low	Low
HP #4 Low Specimen #1	Moderate - Low	Low
HP #4 Low Specimen #2	Low	Low
HP #4 Low with LWA Specimen #1	Moderate - Low	Moderate - Low
HP #4 Low with LWA Specimen #2	Very Low	Very Low
HP Blend Low Specimen #1	Very Low	Very Low
HP Blend Low Specimen #2	Very Low	Very Low
HP Blend Low with LWA Specimen #1	Very Low	Very Low
HP Blend Low with LWA Specimen #2	Very Low	Very Low

Two sets of specimens were cast to determine the correction factor experimentally. Both sets contained two 16 inch (406 mm) rings and two 18 inch (457 mm) rings cast at the same time from the same mixture. The results of the tests are provided in Table 44. The HP #3 Low w/LWA mixture did not crack for the 18 inch (457 mm) specimen, and therefore could not be used to calculate the correction factor. For the HP #4 High mixture the 16 inch (406 mm) ring cracked at an average of 11.82 days, and the 18 inch (457 mm) ring cracked at an average of 18.03 days. This indicates that the larger specimen took 1.53 times longer to crack.

Other researchers have also investigated the role of specimen geometry and time to crack. Weiss et al (2000) determined that for the same mixture a specimen with a concrete thickness of 1.181 inches (30 mm) and a ratio of outside radius to inside radius of 1.2 took approximately 8 days to crack. Another specimen with a concrete thickness of 2.953 inches (75 mm) and a ratio of 1.5 took approximately 11.7 days to crack. Finally a specimen with a concrete thickness of 5.905 inches (150 mm) and a ratio of 2.0 cracked at approximately 23.33 days. A plot of the ratio of outside radius to inside radius vs. time to cracking is shown in Figure 38.

Table 44: Comparison Between 16 and 18 inch (406 and 457 mm) Rings

Mixture	Concrete Ring Thickness in (mm)	Ro/Ri	Time To Cracking (days)
HP #3 Low w/ LWA Specimen #1	1.5 (38)	1.23	24.45
HP #3 Low w/ LWA Specimen #2	1.5 (38)	1.23	81.40
HP #3 Low w/ LWA Specimen #1	2.5 (64)	1.38	90.00
HP #3 Low w/ LWA Specimen #2	2.5 (64)	1.38	90.00
HP #4 High Specimen #1	1.5 (38)	1.23	12.33
HP #4 High Specimen #2	1.5 (38)	1.23	11.31
HP #4 High Specimen #1	2.5 (64)	1.38	15.06
HP #4 High Specimen #2	2.5 (64)	1.38	21.00

The equation for a curve that has been fit to this data is:

Equation 12: Time to crack versus ratio of ring radii

$$^{Ro}/_{Ri} = -0.0025\, t^2 + 0.13\, t + 0.3188$$

Where, t = time to crack

This equation was developed from the data plotted in Figure 38. From Equation 3 it was determined that if the mixture used in this experiment with a 16 inch (406 mm) ring and a ratio of outside radius to inside radius of 1.23, the time to crack would be approximately 8.36 days. If the mixture was used on an 18 inch (457 mm) ring with a ratio of outside radius to inside radius of 1.38 was used the ring would crack in approximately 10.20 days. This equation indicates that the larger ring would take 1.22 times longer to crack.

The theoretical determination of a correction factor for the 16 inch (406 mm) and 18 inch (457 mm) rings was determined to be 1.67, the correction factor from our experiments was 1.53 and the correction factor determined from Weiss et al (2000) was 1.22.

The ASTM standard classifies any mixture that does not crack after 28 days to have a low potential for cracking if the concrete specimen has thickness of 1.5 inches (38 mm). For our research, all specimens were tested until they cracked or until 90 days had elapsed. The correction factors indicate that this is long enough to determine if the mixtures have a low potential for cracking with a 2.5 inch (64 mm) concrete thickness. For direct comparisons of time to crack for different mixtures more experiments should be run, but if the ASTM classification is to be determined for a low potential then the current data are sufficient.

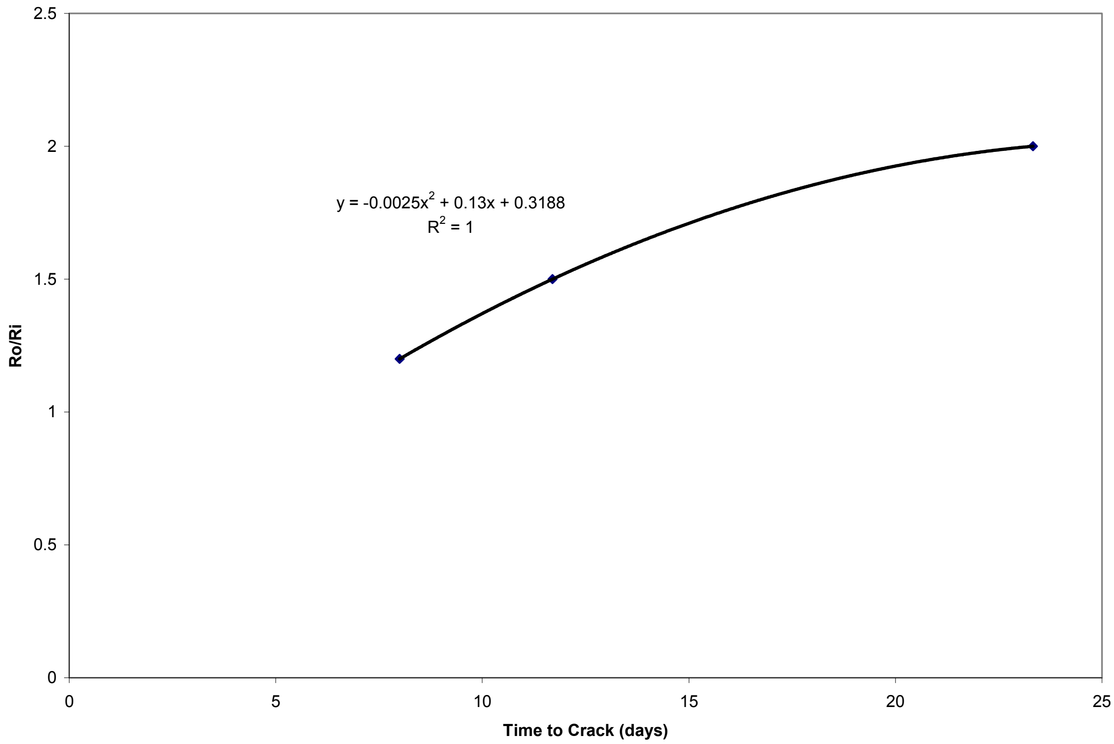


Figure 38: Ro/Ri vs. Time to Cracking

Fly Ash, Ground Granulated Blast Furnace Slag, and Silica Fume

The concrete mixtures used in this research contained cement plus either fly ash and silica fume, or ground granulated blast furnace slag (GGBFS) and silica fume. The amount of silica fume remained constant for both mixtures. At early ages mixtures containing fly ash had higher compressive strengths than GGBFS. At later ages mixtures containing the GGBFS had met and surpassed the compressive strength of those with fly ash, as expected. There seemed to be no consistent difference in cracking tendencies between fly ash and GGBFS.

Modulus of Elasticity

Typically, the modulus of elasticity for lightweight aggregate concrete is lower than normal weight aggregate concrete. This research found no substantial difference between the two. As part of the research, attempts were made to correlate the modulus of elasticity to the American Concrete Institute code equation. ACI code equation for the modulus of elasticity is (ACI 318 2005):

Equation 13: E_c, Modulus of Elasticity

$$E_c = 33w^{1.5}\sqrt{f'_c} = 57{,}000\sqrt{f'_c}$$
$$E_c = 0.043w^{1.5}\sqrt{f'_c} \quad \text{(MPa)}$$

Where: f'_c = Ultimate compressive strength at 28 days in psi (MPa) w = weight of concrete in lbs/ft ^{3} (kg/m^3)

Figure 39 shows the predicted modulus of elasticity versus the measured modulus of elasticity after 28 days. The predicted modulus of elasticity was higher than the measured modulus in every case. Equation 13 considers the compressive strength of the concrete as well as the density. The small amount of LWA used did not greatly affect the density.

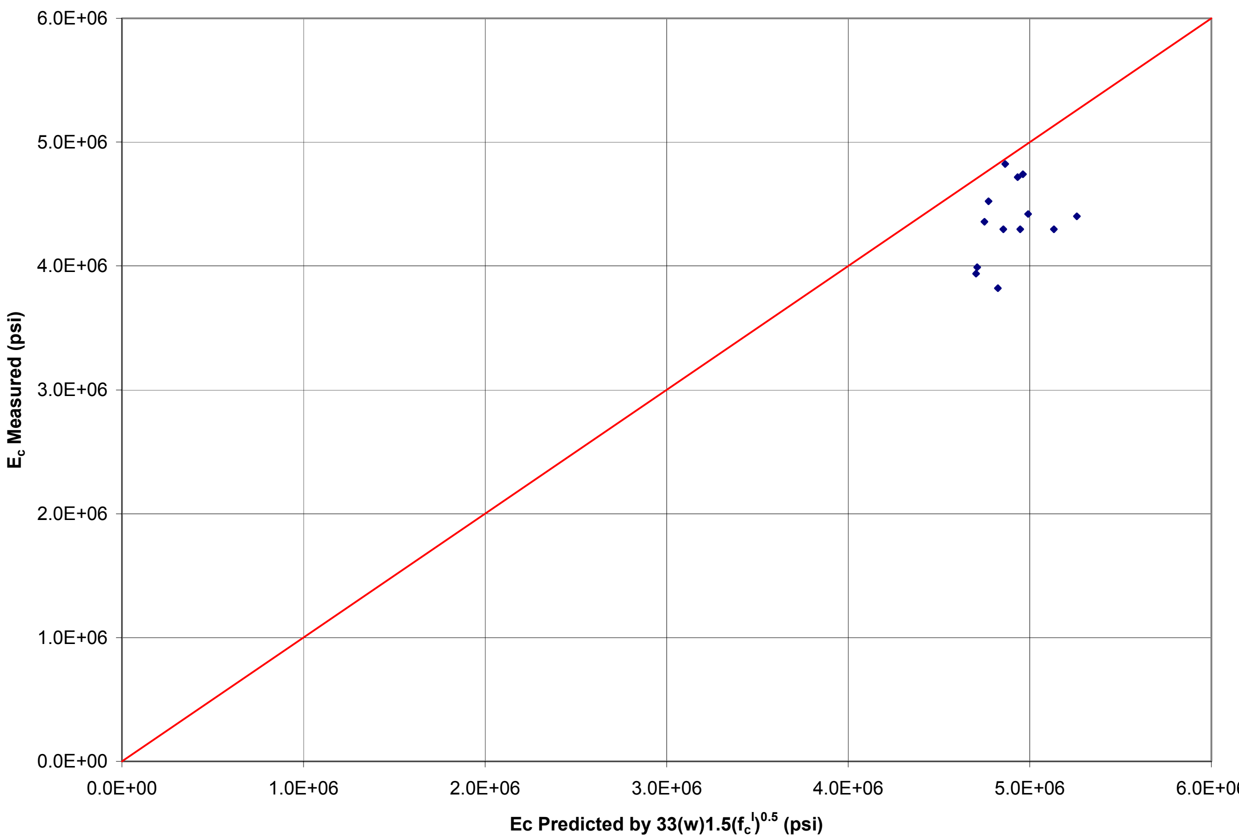


Figure 39: E_c Measured vs. E_c Predicted
Note – 1 million psi = 6.9 GPa

Effect of Aggregate Absorption Level

The original research hypothesis, based on the ODOT District 12 field observations, was that the low absorption coarse aggregates would have the greatest risk of cracking, and that the medium and high absorption aggregates would have a lesser risk. A summary of the results for the HP #8 coarse aggregates is shown in the first two lines of Table 45.

The low absorption aggregate has a higher cracking risk than the medium absorption aggregate, but the high absorption risk has the highest. In the ODOT field observations, the high absorption aggregate was not well represented. It should also be noted that the low absorption aggregate HP # 8 was the only mixture that did not have any runouts – no samples survived for 90 days, whereas some of the samples of the medium and high absorption aggregates were still uncracked at the 90 day mark.

Table 45: Comparison Between Low, Medium, and High Absorption Aggregates (HP)

Mixture	Minimum time	Maximum time	Average	Standard deviation
HP low absorption	23	37	32.8	6.9
HP medium absorption	23	90	47.1	29.5
HP high absorption	11	90	26.7	30
HP low absorption with LWA	19	81	46.3	29.9
HP blend*	51 (33)	90 (59)	85 (55.6)	13.9 (9.1)

* with larger ring – divide by 1.53 to compare to smaller ring, value in parentheses

Effect of Inclusion of Lightweight Aggregate

The mixtures with LWA generally had higher strength than those without, particularly at early ages. The average increase in compressive strength at 7 and 28 days was approximately 6.4 %. This indicated that the LWA aided the strength development of HP concrete with low w/cm ratios.

Comparing the HP low absorption with and without LWA (line 1 versus line 4 in Table 45), the inclusion of the LWA increases the average time to crack and makes the HP low absorption with LWA roughly equivalent to the HP medium absorption. Table 31 and Table 32 indicate that adding LWA to HP medium absorption seems to make that mixture more crack resistant.

Effect of Aggregate Maximum Size and Gradation

The effect of increasing the maximum size of the aggregate from #8 to # 57 is complicated somewhat in that the majority of the testing was carried out with different size rings. However, it is clear from Table 32 that very few of the blended aggregate specimens cracked before 90 days. Comparing line 5 to the other lines of Table 45 shows that, even with adjusting for ring size, the blended mixtures were clearly the most crack resistant. This is most likely due to the reduced amount of paste used with larger maximum aggregate sizes and more optimized gradations.

Comparison Between Gravel and Limestone

The results of testing samples made with gravel aggregate were surprising. As may be seen in Table 31 and Table 32, #8 gravel mixtures cracked at 14 – 33 days and blended gravels cracked at 20 – 38 days, which when divided by 1.53 adjusts to 13 – 25 days. There is a considerably higher risk of cracking with gravels than with limestones of similar absorption capacity.

Table 25 indicates that the mixtures made with gravel often have a lower splitting tensile strength than comparable mixtures made with limestone. Modulus of elasticity values (Table 26) are similar or higher. Unrestrained shrinkage values (Table 27 through Table 30) are also similar or higher. Therefore, with similar or higher stress and lower strength than limestone aggregate mixtures, the cracking tendency for gravels is higher.

Class S and Class C Option 1 Concrete

Cracking data for the Class S and Class C Option 1 mixtures is summarized in Table 33. None of the Class C Option 1 mixtures cracked. The Class S mixtures cracked no earlier than 36 days (adjusted to 24 by dividing by 1.53), and were therefore moderate-low to low potential for cracking. Much of this can be attributed to the use of # 57 aggregate, and thus less paste than is used with # 8 aggregate.

Class S mixtures all exceeded a compressive strength of 4,500 psi (31 MPa) at 28 days. All Class C Option 1 mixtures exceeded 3,500 psi (24 MPa) at 28 days. The inclusion of LWA in the Class C Option 1 did not have a significant effect on cracking resistance, which was already very high, but did improve strength.

IMPLEMENTATION RECOMMENDATIONS

Research results are of little use unless they can be put into practice. Many excellent research studies have had little impact on engineering practice, generally because it may be very difficult for practitioners to extract the usable findings.

To date, preliminary results from this research have been presented at the Ohio Transportation Engineering Conference (OTEC) (October 2006) and the annual meeting of the Transportation Research Board (TRB) (January 2007). These presentations were limited to the HP portion of the research. Additional submissions will be made to OTEC and TRB following the completion of the study and approval of the final report. These will incorporate the additional findings from Class C, Class S, and overlay concrete.

A presentation has been scheduled for the fall 2007 meeting of the American Concrete Institute (ACI). ACI Committee members have vast experience in all aspects of concrete paving, and can provide useful feedback to research team members. Committee reports and other documents synthesize the state of the practice and are specifically intended for use by practicing engineers. FAA, FHWA, USACE, and other agencies participate in committee deliberations and often make use of ACI products. By presenting the results of this project at ACI committee meetings and paper sessions, the research team can rapidly and widely disseminate the information to practitioners.

Field Testing Plan

The laboratory results are promising, but need to be verified in the field. The limited field testing above has so far been in agreement with the laboratory testing. It would be useful to undertake field testing throughout the state of Ohio at a variety of project sites, using the test plan outlined in Table 35. Field observations, such as those documented in Table 36, will also be important. The testing of field samples should parallel the laboratory testing as closely as possible.

One important variable to incorporate in the field testing will be concrete with and without LWA replacement. Ideally, this would be done on the same project, but this is probably not practical in most cases, except for staged construction. It is therefore more likely that similar projects would be investigated. Mixtures that have been shown in the laboratory to be exceptionally prone to cracking, such as HP with low absorption # 8 aggregate, will not be recommended for field testing.

It will also be necessary to monitor the performance of the bridge decks, particularly with respect to early age cracking. Therefore, periodic crack surveys and nondestructive evaluation are recommended.

Specification Recommendations

The HP mixtures most resistant to cracking used a blend of # 57 and # 8 coarse aggregate, regardless of absorption level. These mixtures are shown in Table 12 through Table 14. If it is necessary to use # 8 coarse aggregate, the medium absorption aggregates were more crack resistant than low absorption aggregates. Adding fine LWA to the mixture improved the cracking resistance of low and medium absorption aggregates. High absorption coarse aggregates were the least crack resistant.

Class C Option 1 is highly crack resistant, and Class S is moderately crack resistant. These mixtures were made with # 57 coarse aggregate only during this research. Class C and

Class S mixtures with smaller coarse aggregate and more paste would be likely to be less crack resistant, but these were not tested.

Training Module

A training module has been prepared using Microsoft PowerPoint to aid in implementation of the research. This will be based on the OTEC and TRB presentations. It is important to provide the rationale behind the recommendations made for reducing cracking of HP and other transportation concrete.

Market and Audience

The main market for this research study is ODOT, particularly the district bridge engineers. Other elements of the transportation community can adopt recommendations from the findings of this project for extending bridge deck life. The results will be disseminated to the transportation community through the organizations discussed below.

Concrete is the most widely used construction material in the world. Due to the reduced maintenance costs associated with improved concrete durability, the benefits of reducing cracking through internal curing will be substantial. In addition to ODOT and its suppliers and contractors, the audience will be broadly based throughout the concrete community.

Impediments to Successful Implementation

The impediments to successful implementation are expected to be:

- Availability of materials, particularly coarse aggregates with the appropriate level of absorption, and
- Cost of LWA
- Availability of storage bins and dispensing equipment for an additional mixture constituent (LWA)

An additional potential problem may be the difficulty of pumping concrete with substantial amounts of lightweight aggregate. This is unlikely to be a problem at the replacement levels considered in this research.

Institutions and Individuals

ACI and TRB are pioneers in the use of internal curing. Important individuals include members of TRB committees and ACI Committee 325. Through participation in ACI and TRB, the research team has developed an extensive web of contacts at FHWA (several are members of his ACI 325 Committee), FAA, and other state and federal agencies.

Activities for Implementation

The main implementation activities, focused on ODOT, included an OTEC presentation and the training module discussed above. Other implementation activities will include the reports, presentations at TRB annual meetings, and presentations at ACI Conventions.

Criteria for Evaluating Implementation

The primary benefit of implementing internal curing will be a long term reduction in maintenance costs and longer life for concrete pavements and structures. This benefit will, of course, be difficult to document in the short term. Therefore, evaluation will focus on measuring reduction in cracking, particularly for high strength and high performance concrete. During this research appropriate estimates of the benefits of internal curing will be developed, documented, and included in reports.

Costs of Implementation

Costs of implementation are unknown but will depend on the availability of local materials that promote internal curing. Where coarse aggregates with moderate absorption are available, e.g. ODOT District 12, costs of implementation will be very small. If locally available coarse aggregates have low absorption and lightweight fine aggregate is used, costs will be somewhat higher. The estimated cost of LWA is 3 or 4 cents per pound (6 ½ to 9 cents per kg). Costs of using innovative materials, e.g. superabsorbent polymers, are unknown. However, in all cases life cycle costs for structures and pavements will be substantially reduced.

CONCLUSIONS

Under some circumstances, implementation of HPC in structures has been accompanied by early cracking. This is particularly important for large, thin elements such as bridge decks. Since this cracking allows harmful chemicals to corrode the reinforcing steel, it defeats the purpose of using HPC in the first place. Silica fume is often used in bridge deck HPC to reduce the concrete permeability, but the use of silica fume has been associated with an aggravated risk of early cracking.

Crack Reduction Recommendations

In the present study, the strongest effect on cracking was due to the replacement of a small maximum size coarse aggregate with an aggregate blend. Only one of eight specimens made with an aggregate blend cracked before 90 days elapsed. Increasing the coarse aggregate absorption level from low to medium had a less dramatic effect, as did the introduction of LWA for internal curing to the low absorption coarse aggregate. Internal curing also enhanced the early age as well as the ultimate strength of the concrete. Unfortunately the lab results did not replicate the field results as cleanly as would have been desired and there remain some unexplained discrepancies.

Results at Cleveland State University showed a great benefit in high performance mixtures with a combination of #57 and #8 aggregate. Ordinarily ODOT's high performance mixture has only #8 aggregate. No matter what the level of absorption, the shrinkage was dramatically reduced when a blended mixture was tested.

Internal Curing Implementation

Numerous investigators have studied internal curing to mitigate early age bridge deck cracking. The most common method used is the replacement of a portion of the fine aggregate with a saturated structural LWA. Another method is the addition of SAP.

Field experience in Northeast Ohio reported by ODOT District 12 indicated that a third method is available. This is the selection of an appropriate absorption level of coarse aggregate, which provides enough water to prevent self-dessication and autogenous shrinkage of low w/cm mixtures distributed throughout the concrete matrix. The use of a well graded, larger maximum size coarse aggregate, with a corresponding reduction in the amount of cement paste, has had probably the greatest effect on the reduction of bridge deck cracking in ODOT District 12.

The primary purpose of this research was to develop methods to economically produce more durable high performance concrete using internal curing. This was accomplished through literature review, research, discussion, test specimens, and interpretations of results of all of the above. This research authenticated the benefits of internal curing for concrete properties and assessed alternative methods to promote internal curing – e.g. coarse aggregate with absorption > 1 %, lightweight fine aggregate. This research also confirmed the findings in *A Survey of High Performance Concrete Bridge Decks,* Volume IV, ODOT District 12, 04-01-02 (Crowl and Sutak, 2002).

During the study, researchers found that by using a coarse aggregate with an absorption capacity of less than 1%, along with a lightweight fine aggregate, the amount of shrinkage was decreased. Furthermore, the strength values were increased. The results indicate that it may be favorable or possible for ODOT to use a coarse aggregate with an absorption capacity less than 1% combined with a lightweight fine aggregate. Thus, there may be a potential to allow for the usage of aggregates that would be excluded by a limitation to coarse aggregate with an absorption of 1.00% or greater as defined per ASTM C 127."

There were numerous benefits of internal curing for high performance concrete. The cracking tendencies were reduced. Concrete mixtures that did not have the lightweight fine aggregate cracked in less time. Specimens that contained the lightweight fine aggregate were far superior when dealing with shrinkage. Concrete strengths were also improved with LWA. The internal curing process allowed all of the cement to hydrate properly. Compressive strengths increased by up to twenty percent when the lightweight fine aggregate was used. Clearly, however, the most important benefit of the internal curing process was the decrease in cracking tendencies.

Limestone versus Gravel Coarse Aggregate

Most of the research was carried out on limestone, with only limited testing of gravel. The main reason was the wider use of limestone. However, gravels showed higher cracking tendency. Lower flexural and tensile strengths are typically expected for gravels, which might account for this result. Introduction of LWA reduced the cracking tendency of concrete made with gravel.

Recommendations for Future Research

Most of the research documented in this report was conducted in a controlled laboratory environment. A field testing plan, as suggested in the Implementation Recommendations chapter, would be useful to extend the research.

Additional laboratory research would be useful to investigate the other potential benefits of internal curing. Experimental research should address:

- Effect of internal curing on permeability of concrete. Lam (2005) documented reductions in permeability for the mixtures he investigated.
- Effect of internal curing on freeze-thaw durability.
- Effect on concrete fatigue strength, for pavement design.

On the latter point, a recently concluded study by Amer (2007) documented improved fatigue life for roller-compacted concrete pavement. The soft LWA might provide a crack tip blunting effect, slowing the propagation of fatigue related microcracking within the pavement.

REFERENCES

Amer, Nader (2007) “Laboratory Evaluation of Fatigue Behavior of Thin Bonded Overlays on Roller-Compacted Concrete Pavements, Doctoral Dissertation, Cleveland State University, May 2007.

American Concrete Institute Committee 116. (2000) Cement and Concrete Terminology. ACI 116R-00.

ACI Committee 209. (1992). “Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures (ACI 209R-92).” American Concrete Institute. Farmington Hills, Michigan.

ACI Committee 209. (2005). “Report on Factors Affecting Shrinkage and Creep of Hardened Concrete (ACI 209.1R-05).” American Concrete Institute. Farmington Hills, Michigan.

ACI Committee 224. (2001). “Control of Cracking in Concrete Structures (ACI 224R-01).” American Concrete Institute. Farmington Hills, Michigan.

ACI Committee 318 (2005). “Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05),” American Concrete Institute. Farmington Hills, Michigan.

Aitcin, P.C. 1999. “Demystifying Autogenous Shrinkage.” *Concrete International*. Volume 21 Number 11. pp. 54-56. 1999.

AASHTO PP34-98 (1998), “Standard Practice for Estimating the Crack Tendency of Concrete,” American Association of State Highway and Transportation Officials, Washington, D.C.

ASTM C39/C39M. (2005). Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.

ASTM C114. (2005). Standard Test Methods for Chemical Analysis of Hydraulic Cement.

ASTM C138/C138M. (2001). Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete.

ASTM C143/C143M. (2005). Standard Test Method for Slump of Hydraulic Cement Concrete.

ASTM C150. (2005). Standard Specification for Portland Cement.

ASTM C157/C157M. (2004). Standard Test Method for Length Change of Hardened Hydraulic-Cement, Mortar, and Concrete.

ASTM C231. (2004). Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method.

ASTM C260. (2001). Standard Specification for Air-Entraining Admixtures for Concrete.

ASTM C311. (2005). Standard Test Methods for Sampling and Testing Fly Ash or Natural Pozzolans for Use in Portland-Cement Concrete.

ASTM C469. (1994e1). Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression.

ASTM C494/C494M. (2005). Standard Specification for Chemical Admixtures for Concrete.

ASTM C496. (1996). Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens.

ASTM C989. (2005). Standard Specification for Ground Granulated Blast-Furnace Slag for Use in Concrete and Mortars.

ASTM C 1090-01 (2001), "Standard Test Method for Measuring Changes in Height of Cylindrical Specimens from Hydraulic-Cement Grout," ASTM International.

ASTM C1240. (2005). Standard Specification for Silica Fume Used in Cementitious Mixtures.

ASTM C1581. (2004). Standard Test Method for Determining Age at Cracking and Induced Tensile Stress Characteristics of Mortar and Concrete under Restrained Shrinkage.

Bentz, D.P., Jensen, O.M., Hansen, K.K., Olesen, J.F., Stang, H., and Haecker, C.J., (2001). <http://ciks.cbt.nist.gov/bentz/finedtu> "Influence of Cement Particle Size Distribution on Early Age Autogenous Strains and Stresses in Cement-Based Materials," Journal of the American Ceramic Society, 84 (1), 129-135, 2001.

Bentz, D.P. and Jensen, O.M. (2004). "Mitigation Strategies for Autogenous Shrinkage Cracking." Cement and Concrete Composites, Volume 26 (6), p. 677-685.

Bentz, D.P., Lura, P. and Roberts, J.W. (2005). "Mixture Proportioning for Internal Curing." Concrete International, Vol. 27, No. 2, p. 35-40, February 2005.

Bisschop, J. and Van Mier, J.G.M. (2002). "How to Study Drying Shrinkage Microcracking in Cement-Based Materials Using Optical and Scanning Electron Microscopy." Cement and Concrete Research, 32, p. 279-287.

Burdette, E.G. and Salem, R. (2005). "Development of High Performance Concrete Mixture for Tennessee Bridge Decks." Center for Transportation Research, University of Tennessee-Knoxville, Tennessee 37996. Project Number TNSPR-RES1236

"Concrete Durability a Multibillion-Dollar Opportunity" (1988) *Concrete International,* January 1988.

Crowl, D., and Sutak, M. (2002), *A Survey of High Performance Concrete Bridge Decks,* Volume IV., ODOT District 12, 04-01-02.

Cusson, D., Hoogeveen, T., and Mitchell, L. "Restrained Shrinkage Testing of High-Performance Concrete Modified with Structural Lightweight Aggregate." Seventh International Symposium on the Utilization of High-Strength/High-Performance Concrete, Russell, H. G., American Concrete Institute (ACI) International, Washington, D.C., 2005, Volume II, pp. 1353 – 1372.

Czarnecki, B. and Kroman, J. (2005). "Evaluation of Cracking Tendency and Unrestrained Shrinkage of High-Performance Concrete Mixes in Cast-in-Place and Precast Bridge Applications." Seventh International Symposium on the Utilization of High-Strength/High-Performance Concrete. ACI International, SP-228, p. 1315-1328.

de Jesus Cano Barrita, F., Bremner, T.W., and Balcom, B.J. (2004). "Use of Magnetic Resonance Imaging to Study Internal Moist Curing in Concrete Containing Saturated Lightweight Aggregate." High Performance Structural Lightweight Concrete, ACI International, SP-218.

Fitch, M.G., Abdulshafi, O. and Barnhart, J. (2002). "Ohio Bridge Maintenance Manual." Prepared in Cooperation with The Ohio Department of Transportation, The U.S. Department of Transportation, Federal Highway Administration, and The Ohio State University, Department of Civil and Environmental Engineering.

Geiker, M.R., Bentz, D.P. & Jensen, O.M. (2004). "Mitigating Autogenous Shrinkage by Internal Curing." High Performance Structural Lightweight Concrete, ACI International, SP-218, p. 143-153.

Hammer, T.A., Bjontegaard, O., and Sellevold, E.J. (2004). "Internal Curing – Role of Absorbed Water in Aggregates." High Performance Structural Lightweight Concrete, ACI International, SP-218, p. 131-141.

Hansen, P. F., and Jensen, O. M. (2000). "Autogenous deformation and RH-change in perspective." Cement and Concrete Research.

Harmon, K.S. (2005). "Recent Research Projects to Investigate Mechanical Properties of High-Performance Lightweight Concrete." Seventh International Symposium on the Utilization of High-Strength/High-Performance Concrete. ACI International, SP-228, p. 991-1008.

Hoff, G.C. (2003). "Internal Curing of Concrete Using Lightweight Aggregates." Theodore Bremner Symposium on High-Performance Lightweight Concrete, p. 185-203.

Hoff, G.C. (2002). "The Use of Lightweight Fines for the Internal Curing of Concrete." Hoff Consulting LLC, 250 Saddlewood Lane, Clinton, Mississippi, 39056.

Holm, T. A., Ooi, O. S. and Brenmer, T. W. "Moisture Dynamics in Lightweight Aggregate and Concrete." Theodore Bremner Symposium on High-Performance Lightweight Concrete, Sixth CANMET/ACI International Conference on Durability of Concrete, Reis, J. P. and Holm, T. A., editors, Thessaloniki, Greece, June 2003, pp. 167 – 184

Hossain, A. B., and Weiss, W. J. (2005). "The Role of Specimen Geometry and Boundary Conditions on Stress Development and Cracking in the Restrained Ring Test," to appear in the Cement and Concrete Research Journal, from: http://bridge.ecn.purdue.edu/~concrete/weiss/publications/journal.shtml.

Ioannides, A.M., and Mills, J.C., (2006a), "Effect Of Larger Sized Coarse Aggregates On Mechanical Properties Of Portland Cement Concrete Pavements And Structures," Volume 1 of 2, Report for Ohio Department of Transportation, State Job Number 148030, The University of Cincinnati, April 2006.

Ioannides, A.M., Walsh, K.M., and Miller, R.A. (2006b), "Effect Of Larger Sized Coarse Aggregates And Of Microsilica On Environmental Properties Of Portland Cement Concrete Pavements And Structures," Volume 2 of 2, Report for Ohio Department of Transportation, State Job Numbers 148000 and 148030, The University of Cincinnati, June 2006.

Jensen, O.M., Bentz, D.P. and Lura, P., (2004), *Autogenous Deformation of Concrete,* American Concrete Institute SP-220.

Lam, H. (2005). "Effects of Internal Curing Methods on Restrained Shrinkage and Permeability." Portland Cement Association, Research and Development Information Serial No. 2620.

Lindquist, W.D., Darwin, D., Browning, J., and Miller, G.G. (2006). "Effect of Cracking on Chloride Content in Concrete Bridges," *ACI Materials Journal,* Vol. 103, No. 6, November-December 2006.

Mack, E. (2006). *Using Internal Curing to Prevent Concrete Bridge Deck Cracking*, Master's Thesis, Cleveland State University, Cleveland, Ohio, June 2006.

Mahmud, H.B., Majuar, M.F.M., Zain, and Hamid, N. (2005). "Strength, Durability and Shrinkage of High-Strength Rice Husk Concrete." Seventh International Symposium on the Utilization of High-Strength/High-Performance Concrete. ACI International, SP-228, p. 189-201.

McGovern, M. (2002), "Wanted: Cause of Early Cracking," pp. 3 – 4, *Concrete Technology Today,* March 2002.

Mindess, S., Young, J.F., and Darwin, D., *Concrete,* 2nd edition, Prentice-Hall, 2003.

Moon, J.H., Rajabipour, F., Pease, B., and Weiss, J. (2005). "Autogenous Shrinkage, Residual Stress, and Cracking in Cementitious Composites: The Influence of Internal and External Restraint". Purdue University.

National Bridge Inventory (U.S. Department of Transportation, Federal Highway Administration). Website address: http://www.fhwa.dot.gov/bridge/nbi.htm

National Institute of Standards and Technology (NIST) (2005). Concrete Monograph, Chapter 8. http://ciks.cbt.nist.gov/~garbocz/monograph/chapter8.html. Accessed July 28, 2005.

Ries, J. and Holm, T. (2004), *High Performance Structural Lightweight Concrete,* American Concrete Institute SP-218.

Roberts, J. (2004), "Internal Curing in Pavements, Bridge Decks, and Parking Structures Using Absorptive Aggregates to Provide Water to Hydrate Cement Not Hydrated by Mixing Water," paper presented to Committee AFN40 (formerly A2E05) Concrete Materials and Placement Techniques (Donald A. Streeter, chair) at Transportation Research Board 83rd Annual Meeting, Washington, D.C., January 2004.

Russell, H.G. (1999), "ACI Defines High-Performance Concrete," *Concrete International: Design and Construction*, February 1999.

See, H., Attiogbe, E.K., and Miltenberger, M.A. (2002). "Potential for Restrained Shrinkage Cracking of Concrete and Mortar." ASTM International, Volume 26, Issue 2.

Shilstone, Sr., J.M., 1990. "Concrete Mixture Optimization." *Concrete International: Design and Construction*. Volume 12 Number 6. pp. 33-39. 1990.

Shilstone, J.M. Sr., and Shilstone, J.M. Jr. (2002) *Performance-Based Concrete Mixtures and Specifications for Today,* Concrete International, pp. 80 – 83, February 2002, Farmington Hills, MI: American Concrete Institute.

Shioanti, T., Bisschop, J. and Van Mier, J.G.M. (2003). "Temporal and Spatial Development of Drying Shrinkage Cracking in Cement-Based Materials." Engineering Fracture Mechanics, 70, p. 1509-1525.

State of Ohio (2005), Department of Transportation Columbus, Ohio. *Construction and Material Specifications.* January 1st 2005.

Suksawang, N., Nassif, H., and Mohammad, A. (2005). "Creep and Shrinkage of High-Performance/High-Strength Concrete." Seventh International Symposium on the Utilization of High-Strength/High-Performance Concrete. ACI International, SP-228, p. 1397-1415.

Tia, Mang, Rajarajan Subramanian, Danny Brown, and Chick Broward. (2005). "Evaluation of Shrinkage Cracking Potential of Concrete Used in Bridge Decks in Florida." Department of Civil and Coastal Engineering. College of Engineering. University of Florida. Project Number: 4910-4504-797-12. 2005.

Villareal, V.H., and Crocker, D.A. (2007), "Better Pavements through Internal Hydration," pp. 32 – 36, *Concrete International*, Vol. 29, No. 2, February 2007.

Voigt, T., Bui, V.K., and Shah, S.P. (2004), "Drying Shrinkage of Concrete Reinforced with Fibers and Welded Wire Fabric," pp. 233 – 241, ACI Materials Journal, Volume 101, Number 3, May – June 2004.

Weber, S. and Reinhardt, H.W. (2003). "Modeling the Internal Curing of High-Strength Concrete Using Lightweight Aggregates." Theodore Bremner Symposium on High-Performance Lightweight Concrete, p. 45-63.

Weiss, W. J, Yang, W., and Shah, S. P., (2000). "Influence of Specimen Size and Geometry on Shrinkage Cracking." Journal of Engineering. Mechanics, 126(1), 93-101.

Weiss, W. J, Yang, W., and Shah, S. P., (2001). "Using Fracture to Predict Restrained Shrinkage Cracking: The Importance of Specimen Geometry." Fracture Mechanics for Concrete Materials, ACI International, SP-201.

Wu, J., Romero, P. (2002). "Internal Curing of High Performance Concrete Using Lightweight Aggregate, Results on 90-day Testing." The University of Utah, Department of Civil and Environmental Engineering.

Zhutovsky, S., Kovler, K., and Bentur, A. (2004). "Assessment of Water Migration Distance in Internal Curing of High-Strength Concrete." Autogenous Deformation of concrete, ACI International, SP-220, p. 181-197.

ACKNOWLEDGEMENTS

This study was sponsored by the Ohio Department of Transportation (ODOT), under a research contract entitled "Evaluation of High Absorptive Materials to Improve Internal Curing of Low Permeability Concrete," State Job No. 134227, Federal Contract Number 20809. The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Ohio Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation. The research was also supported by the Civil Engineering Department and the University Transportation Center of Fenn College of Engineering at Cleveland State University. This support is gratefully acknowledged.

Interest in preventing bridge deck cracking was very high and many individuals contributed to the success of this study. One key individual was John Roberts at Northeast Solite who provided considerable background information in addition to research supplies. The research team would like to acknowledge and thank the following individuals for their specific contributions: Dale Crowl for his support and research with ODOT, Dean Martin at St Marys for their contribution of cement and GGBF slag, Bill Newkirk at ISG for fly ash, Matt Clark at Axim for silica fume, Jeb Brooks at Cuyahoga Concrete for fine and coarse aggregates, Carl Barricelli at Ontario Stone Corporation for coarse aggregates, John Roberts at Northeast Solite for lightweight fine aggregates, and Charles Nmai at Degussa for supplying concrete admixtures. The second author would especially like to thank Albert Wilhelmy for allowing him to use his trucks throughout this project to pick up supplies and materials.

There were many individuals at Cleveland State University that aided in the success of this research. They are Rich Martin, Blake Kreuer, Nader Amer, Dan Miller, Aleksandr Mrkajic, Dave Epperly, and Ali Kaddah. The second author would also like to thank Stacey Farren for her help with his thesis.

APPENDICES

Appendix A: Fresh Concrete Properties and Admixture Dosage

Mixture Design	Date	Weight of Mixture		Amount of Air Entrainment Admixture (mL)	Amount of HRWR Admix (ml)	Measured Slump		Measured Air Content (%)	Designed Unit Weight		Measured Unit Weight	
		lbs	kg			in	cm		lbs/ft^3	kg/m^3	lbs/ft^3	kg/m^3
HP #3 High	8/11/05	337	153	20	140	6.00	15.24	8.00	137.3	2199.2	135.7	2173.7
HP #3 High	9/15/05	334	151	20	180	7.00	17.78	9.00	137.3	2199.2	135.7	2174.3
HP #3 High	9/20/06	256	116	12	130	5.00	12.70	6.80	136.8	2191.5	140.4	2248.8
HP #3 Medium	10/4/05	317	144	10	140	7.50	19.05	8.50	137.1	2196.3	136.0	2178.5
HP #3 Medium	5/16/06	376	171	20	220	3.00	7.62	9.00	136.8	2191.5	135.6	2172.1
HP #3 Medium	10/16/06	256	116	12	150	7.00	17.78	9.00	136.8	2191.5	134.0	2146.5
HP #3 Medium w/LWA	10/4/06	398	181	15	170	6.00	15.24	7.60	133.2	2133.2	135.6	2172.1
HP #3 Medium w/LWA	9/25/06	388	176	15	160	3.50	8.89	5.60	133.2	2133.2	140.4	2249.0
HP #3 Low	12/6/05	411	186	10	140	7.00	17.78	6.80	140.4	2248.8	145.0	2322.4
HP #3 Low	8/25/05	337	153	20	140	7.00	17.78	6.80	138.9	2224.6	-	-
HP #3 Low w/LWA	5/18/06	377	171	20	130	-	-	6.40	136.9	2193.1	142.2	2277.8
HP #3 Low w/LWA	4/18/06	497	225	10	150	7.00	17.78	5.00	136.5	2186.4	138.4	2217.0
HP #3 Low w/LWA	11/1/06	241	109	10	150	7.00	17.78	5.00	136.5	2186.4	-	-
HP #3 Gravels	10/31/06	416	189	12	170	6.50	16.51	8.00	139.1	2228.2	140.8	2255.4
HP #3 Gravels	10/30/06	405	184	15	130	5.00	12.70	8.80	139.1	2228.2	138.8	2223.4
HP #3 Gravels	10/31/06	416	189	12	170	6.50	16.51	8.00	139.1	2228.2	140.8	2255.4
HP #3 Gravels w/LWA	11/8/06	424	192	15	170	7.25	18.42	9.00	135.3	2167.0	134.4	2152.9
HP #3 Gravels w/LWA	10/30/06	372	169	12	130	5.00	12.70	8.50	135.3	2167.0	137.2	2197.7

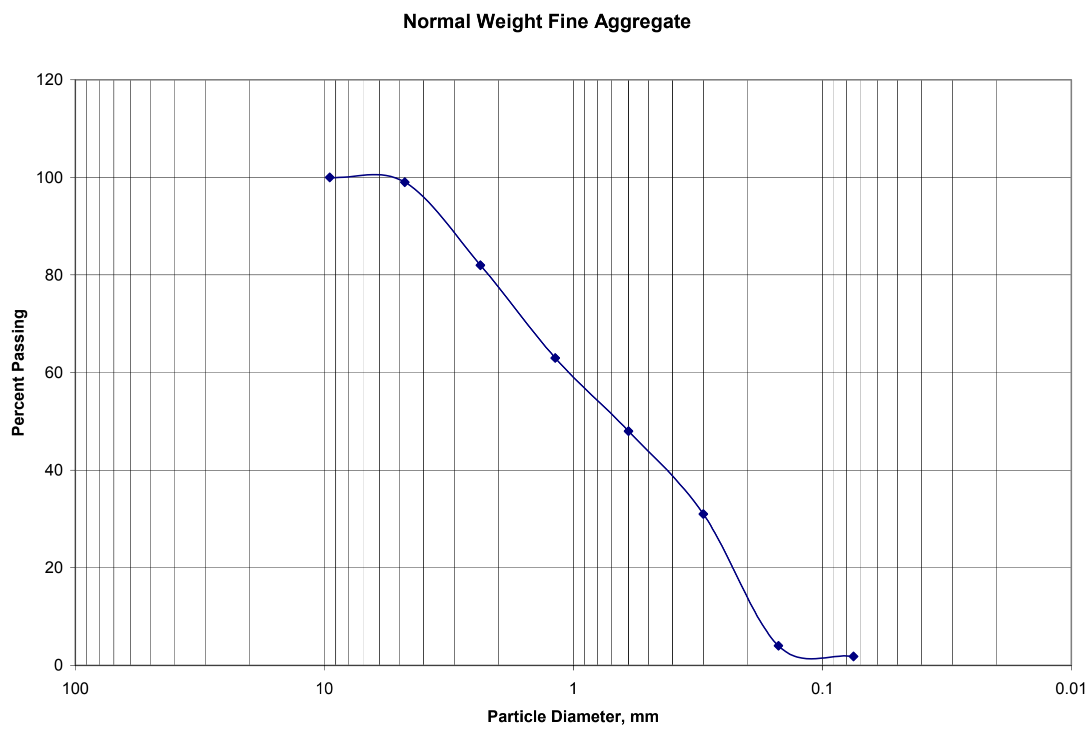
Mixture Design	Date	Weight of Mixture		Amount of Air Entrainment Admixture (mL)	Amount of HRWR Admix (ml)	Measured Slump		Measured Air Content (%)	Designed Unit Weight		Measured Unit Weight	
		lbs	kg			in	cm		lbs/ft^3	kg/m^3	lbs/ft^3	kg/m^3
HP #3 Blended High	5/23/06	438	199	10	200	3.00	7.62	5.20	137.0	2194.8	144.4	2313.1
HP #3 Blended High	5/25/06	330	150	10	150	3.00	7.62	5.00	137.0	2194.8	145.6	2332.3
HP #3 Blended Medium	6/1/06	330	150	30	220	3.50	8.89	5.30	137.0	2194.2	146.8	2351.5
HP #3 Blended Medium	5/30/06	438	199	10	180	4.00	10.16	5.40	137.0	2194.2	143.6	2300.3
HP #3 Blended Low	6/6/06	453	205	15	170	6.00	15.24	7.50	141.7	2270.5	144.4	2313.1
HP #3 Blended Low	6/8/06	342	155	15	120	6.25	15.88	7.20	141.7	2270.5	146.8	2351.5
HP #3 Blended Low w/LWA	7/25/06	363	165	18	130	6.00	15.24	8.00	137.9	2209.1	140.8	2255.4
HP #4 High	9/8/05	426	193	20	220	5.50	13.97	8.50	138.5	2217.9	137.3	2199.3
HP #4 High	11/8/05	524	238	10	250	5.00	12.70	73.50	138.5	2217.9	138.3	2215.4
HP #4 High	5/5/06	380	172	10	180	-	-	6.20	138.2	2213.8	142.0	2274.6
HP #4 Medium	10/11/05	368	167	10	190	7.00	17.78	7.50	138.3	2214.9	137.3	2199.3
HP #4 Medium	4/28/06	379	172	20	200	-	-	7.00	138.0	2210.4	142.0	2274.6
HP #4 Low	12/8/05	422	191	10	140	7.00	17.78	7.50	141.6	2268.5	143.4	2297.0
HP #4 Low	9/1/05	324	147	20	140	3.00	7.62	8.50	140.0	2243.2	-	-
HP #4 Low w/LWA	11/22/05	386	175	10	180	8.00	20.32	7.00	137.8	2207.3	139.0	2226.6
HP #4 Low w/LWA	4/21/06	364	165	10	180	-	-	6.00	138.2	2213.3	142.0	2274.6
HP #4 Gravels	8/24/06	247	112	20	140	3.00	7.62	5.80	140.3	2247.6	146.0	2338.7
HP #4 Gravels	8/22/06	411	186	20	170	3.00	7.62	5.70	140.3	2247.6	145.2	2325.9
HP #4 Blended High	6/13/06	441	200	12	170	5.50	13.97	7.00	138.2	2213.8	142.6	2284.2

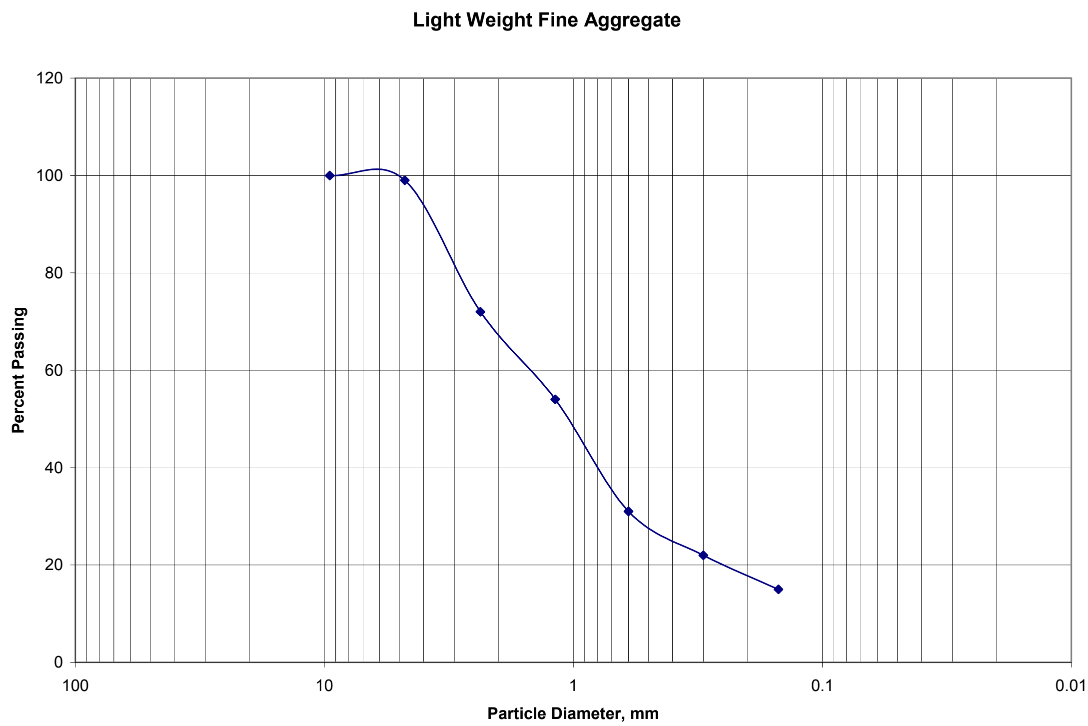
Mixture Design	Date	Weight of Mixture		Amount of Air Entrainment Admixture (mL)	Amount of HRWR Admix (ml)	Measured Slump		Measured Air Content (%)	Designed Unit Weight		Measured Unit Weight	
		lbs	kg			in	cm		lbs/ft^3	kg/m^3	lbs/ft^3	kg/m^3
HP #4 Blended High	6/15/06	333	151	10	150	6.00	15.24	7.60	138.2	2213.8	138.8	2223.4
HP #4 Blended Medium	7/6/06	319	145	15	170	6.25	15.88	6.00	138.2	2213.1	140.8	2255.4
HP #4 Blended Medium	6/20/06	405	184	15	200	6.50	16.51	8.00	138.2	2213.1	138.4	2217.0
HP #4 Blended Low	7/18/06	367	166	10	180	6.50	16.51	7.00	143.0	2289.8	144.4	2313.1
HP #4 Blended Low	7/20/06	408	185	10	150	7.00	17.78	7.00	143.0	2289.8	143.6	2300.3
HP #4 Blended Low w/LWA	7/25/06	367	166	14	137	6.50	16.51	8.00	139.2	2229.3	136.4	2184.9
HP #4 Blended Gravels	8/30/06	385	175	20	170	8.00	20.32	9.00	140.6	2251.6	137.0	2194.5
HP #4 Blended Gravels	9/11/06	338	153	10	150	6.50	16.51	7.20	140.6	2251.6	142.8	2287.4
HP #4 Modified High	9/20/05	333	151	10	190	7.00	17.78	7.00	138.7	2222.1	138.7	2221.8
HP #4 Modified High	1/31/06	488	221	10	180	7	17.78	7	138.7	2222.1	-	-
HP #4 Modified Medium	10/13/05	393	178	10	190	7.00	17.78	8.00	138.7	2221.4	137.7	2205.7
HP #4 Modified Medium	5/17/06	381	173	16	215	2.25	5.72	5.75	138.4	2216.6	144.4	2313.1
HP #4 Modified Medium w/LWA	4/13/06	306	139	10	160	6	15.24	6	135.0	2161.9	140.6	2251.6
HP #4 Modified Medium w/LWA	4/11/06	395	179	10	180	6	15.24	6.5	139.5	2235.2	135.0	2161.9
HP #4 Modified Low	1/24/06	419	190	10	200	4.00	10.16	7.20	143.2	2294.2	142.8	2287.4

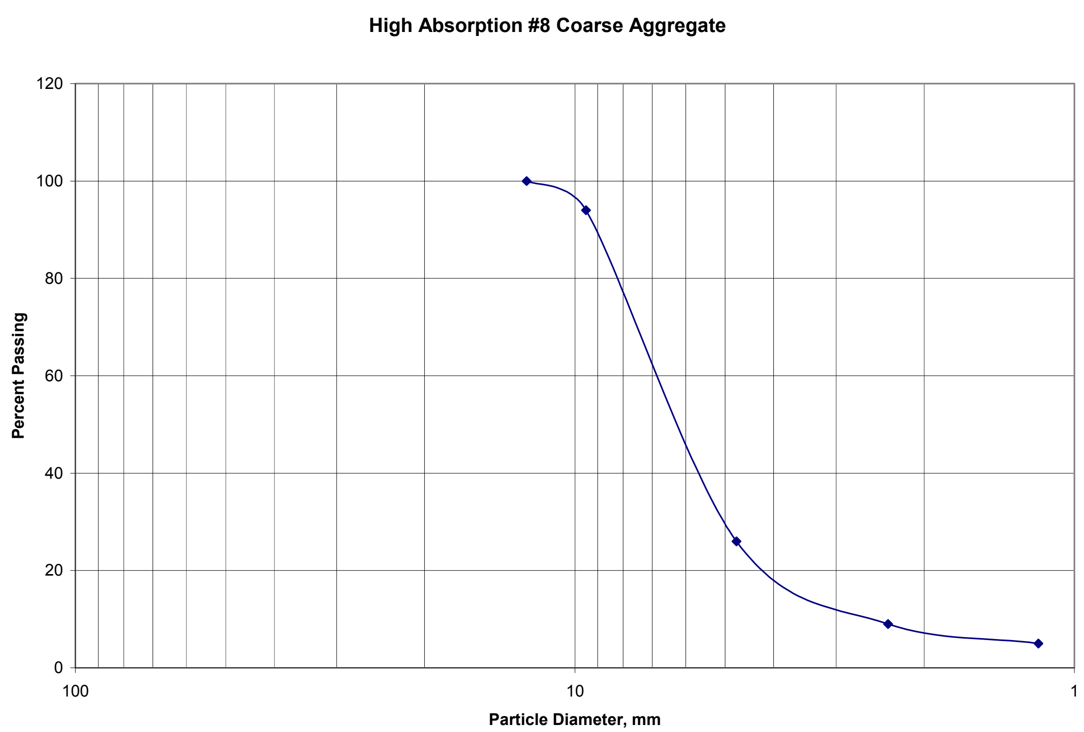
Mixture Design	Date	Weight of Mixture		Amount of Air Entrainment Admixture (mL)	Amount of HRWR Admix (ml)	Measured Slump		Measured Air Content (%)	Designed Unit Weight		Measured Unit Weight	
		lbs	kg			in	cm		lbs/ft^3	kg/m^3	lbs/ft^3	kg/m^3
HP #4 Modified Low	9/6/05	333	151	20	120	4	10.16	7.2	140.0	2243.2	-	-
HP #4 Modified Low w/LWA	11/29/05	401	182	10	150	7.00	17.78	7.50	139.7	2237.9	140.6	2252.2
HP #4 Modified Low w/LWA	5/25/06	384	174	15	180	6.00	15.24	9.00	139.7	2237.9	139.2	2229.8
HP #4 Modified Gravels	8/30/06	340	154	15	100	4	10.16	7.8	141.1	2260.4	142.8	2287.4
HP #4 Modified Gravels	9/11/06	377	171	10	130	5	12.70	5.5	141.1	2260.4	146.8	2351.5
Class S High	8/1/06	364	165	18	-	4.25	10.80	5.40	138.0	2211.2	141.6	2268.2
Class S High	8/3/06	378	171	20	-	5.00	12.70	6.00	138.0	2211.2	142.8	2287.4
Class S Medium	8/24/06	408	185	20	-	2.25	5.72	5.70	138.0	2209.9	144.8	2319.5
Class S Medium	8/21/06	422	191	20	-	3.00	7.62	4.50	138.0	2209.9	142.8	2287.4
Class S Low	8/1/06	438	199	18	-	4.00	10.16	5.00	143.1	2292.2	148.0	2370.7
Class S Low	8/3/06	331	150	15	-	6.00	15.24	5.10	143.1	2292.2	146.0	2338.7
Class C Option 1 High	4/4/06	432	196	22	-	4.00	10.16	6.00	137.4	2200.8	141.8	2270.9
Class C Option 1 High	3/6/06	330	150	15	-	5.00	12.70	6.00	139.1	2227.5	140.6	2251.6
Class C Option 1 Medium	3/28/06	421	191	22	-	6.00	15.24	7.00	137.4	2200.8	138.9	2225.0
Class C Option 1 Medium	3/30/06	317	144	15	-	5.00	12.70	6.50	137.4	2200.8	142.2	2277.3
Class C Option 1 Low	3/7/06	560	254	-		6.00	15.24	7.50	139.0	2225.9	-	-
Class C Option 1 Low	3/9/06	293	133	30	-	4.50	11.43	8.00	139.0	2225.9	140.6	2252.2

Mixture Design	Date	Weight of Mixture		Amount of Air Entrainment Admixture (mL)	Amount of HRWR Admix (ml)	Measured Slump		Measured Air Content (%)	Designed Unit Weight		Measured Unit Weight	
		lbs	kg			in	cm		lbs/ft^3	kg/m^3	lbs/ft^3	kg/m^3
Class C Option 1 Low w/LWA	3/21/06	423	192	25	-	6.00	15.24	6.50	138.3	2215.2	141.4	2264.5
Class C Option 1 Low w/LWA	3/23/06	378	171	15	-	6.00	15.24	6.00	138.3	2215.2	141.0	2258.0
Micro Overlay High w/Fibers	2/14/06	580	263	10	210	6.00	15.24	7.50	137.4	2200.6	139.8	2238.6
Micro Overlay High w/Fibers	2/16/06	426	193	10	150	5.00	12.70	8.50	137.4	2200.6	135.7	2174.3
Micro Overlay High	2/21/06	421	191	10	190	5.00	12.70	8.00	137.4	2200.6	137.4	2200.1
Micro Overlay High	2/23/06	308	140	10	-	7.00	17.78	9.00	137.4	2200.6	134.1	2148.7
Micro Overlay High w/LWA	3/2/06	278	126	10	170	6.00	15.24	8.50	135.7	2173.4	135.7	2174.3
Micro Overlay High w/LWA	2/28/06	603	274	10	600	7.00	17.78	8.00	135.7	2173.4	136.1	2180.8
Field Test #1: HP #4 Modified High	6/28/06	10 cy	7.6 m^3	55 oz	114 oz	6.50	16.51	6.80	138.7	2222.1	139.6	2236.2
Field Test #2: HP #4 Modified Medium	6/29/06	9 cy	6.9 m^3	36 oz	1152 oz	6.00	15.24	6.60	138.7	2221.4	142.8	2287.4
Field Test #3: Class C Option 1 Medium	7/19/06	10 cy	7.6 m^3	140 oz	900 oz	1.25	3.18	7.00	137.4	2200.8	-	-

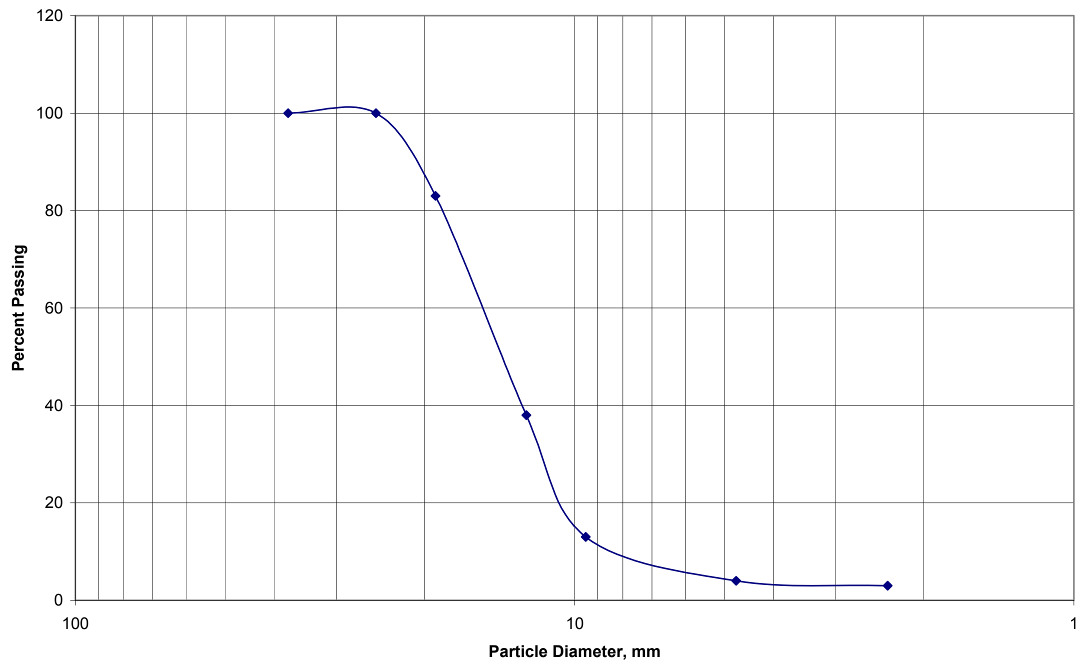
Appendix B: Sieve Analysis of Aggregates



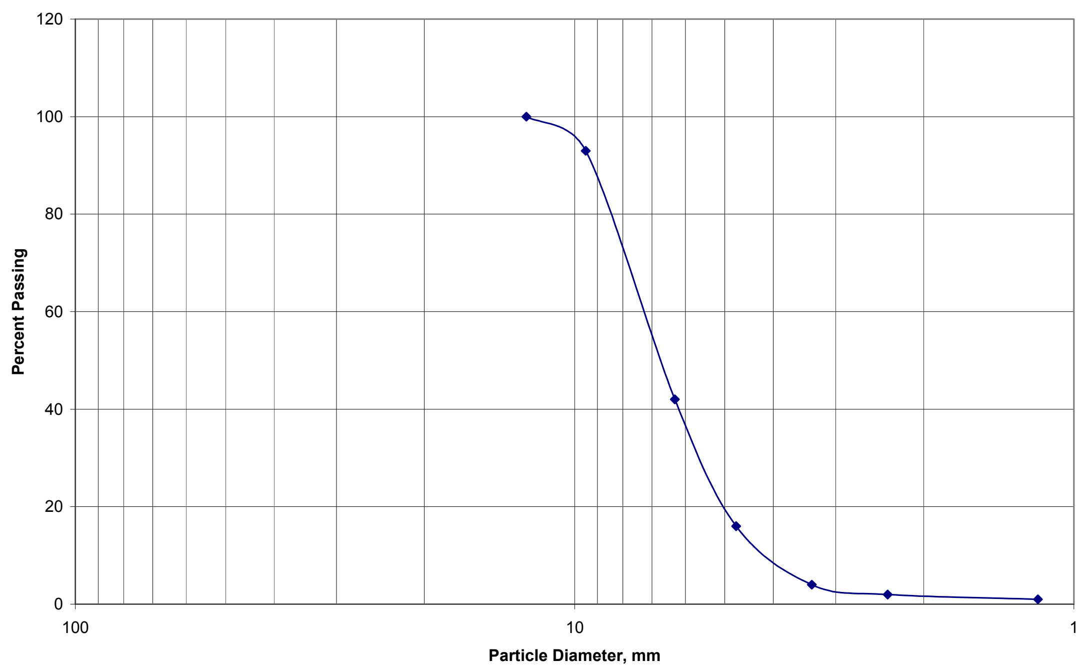

Light Weight Fine Aggregate
120
100
80
60
40
20
0
Percent Passing
100
10
1
0.1
0.01
Particle Diameter, mm


High Absorption #8 Coarse Aggregate
120
100
80
60
40
20
0
Percent Passing
100
10
1
Particle Diameter, mm

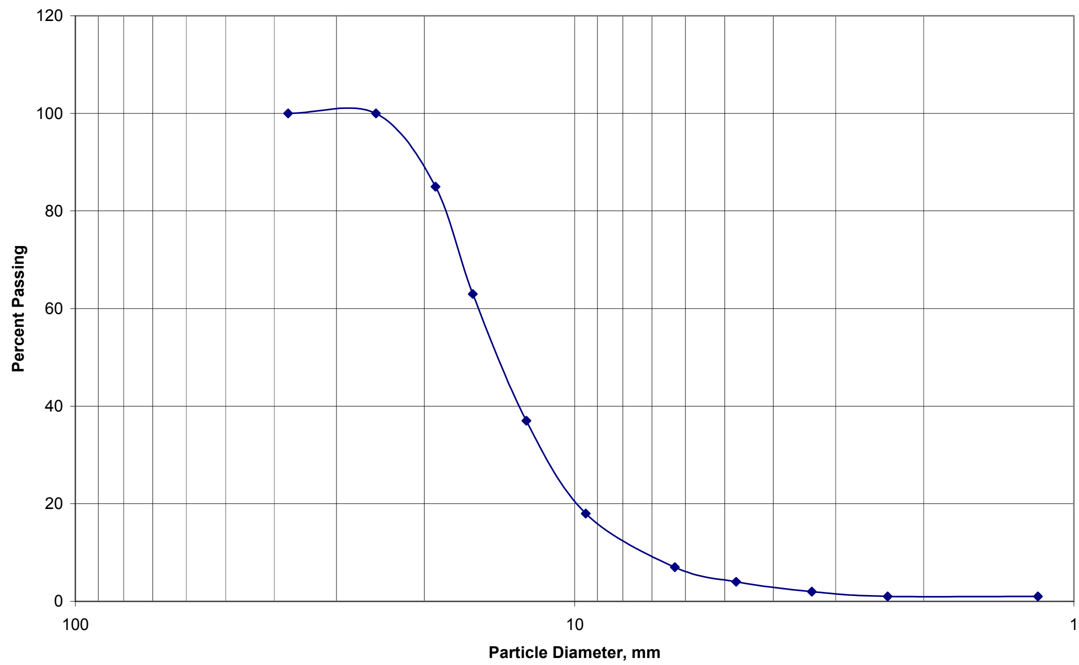
High Absorption #57 Coarse Aggregate



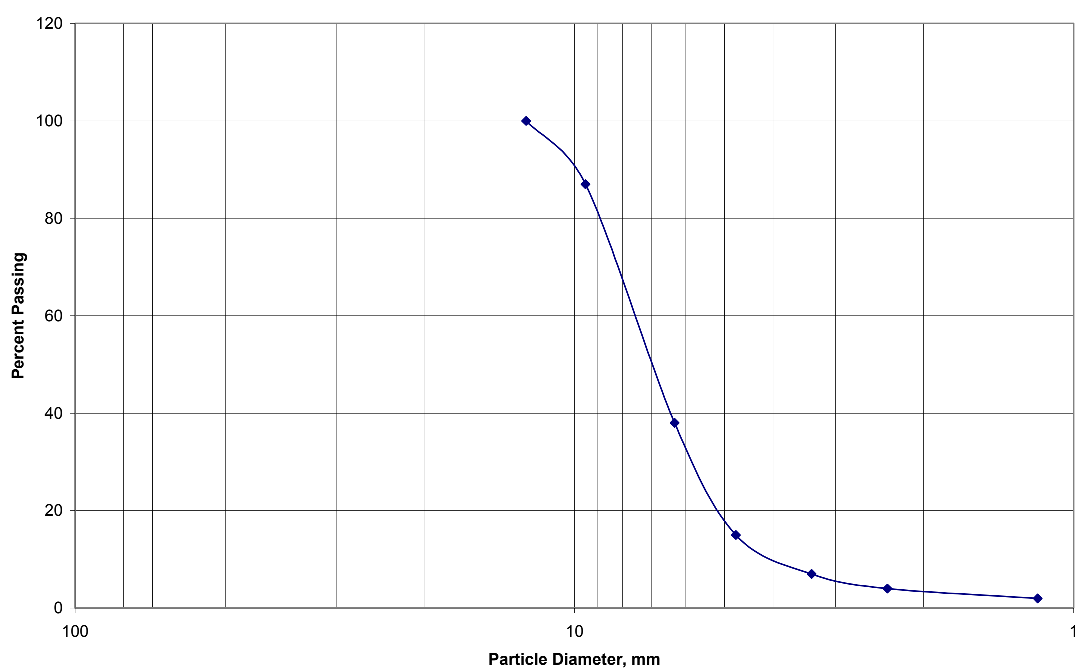
Medium Absorption #8 Coarse Aggregate

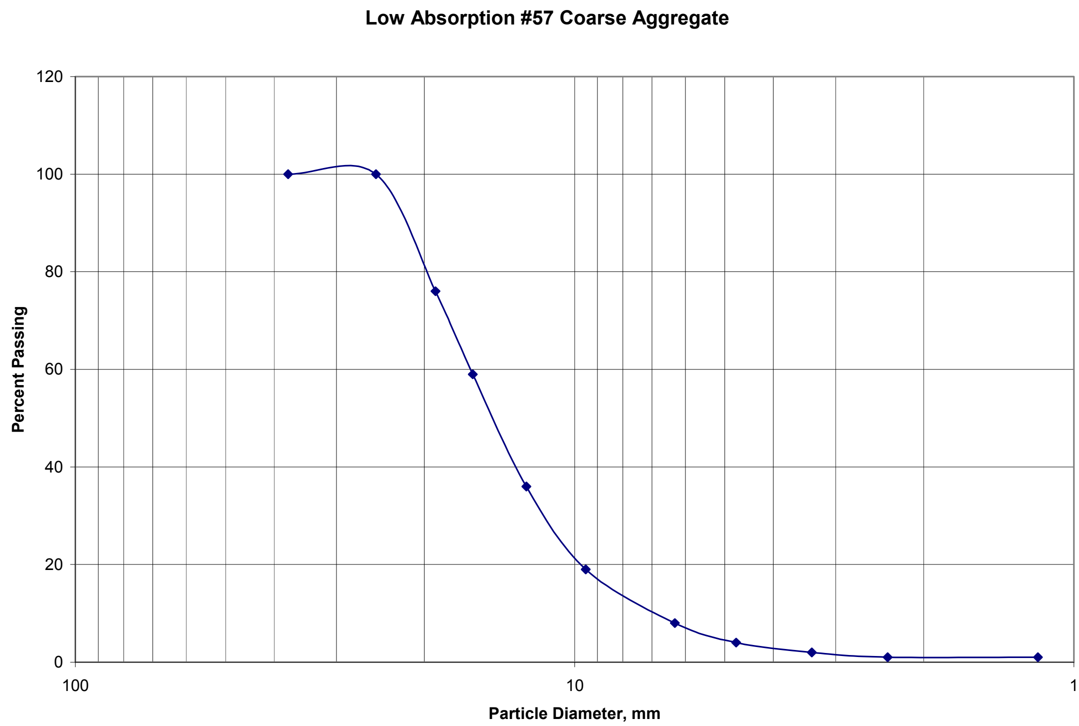


Medium Absorption #57 Coarse Aggregate

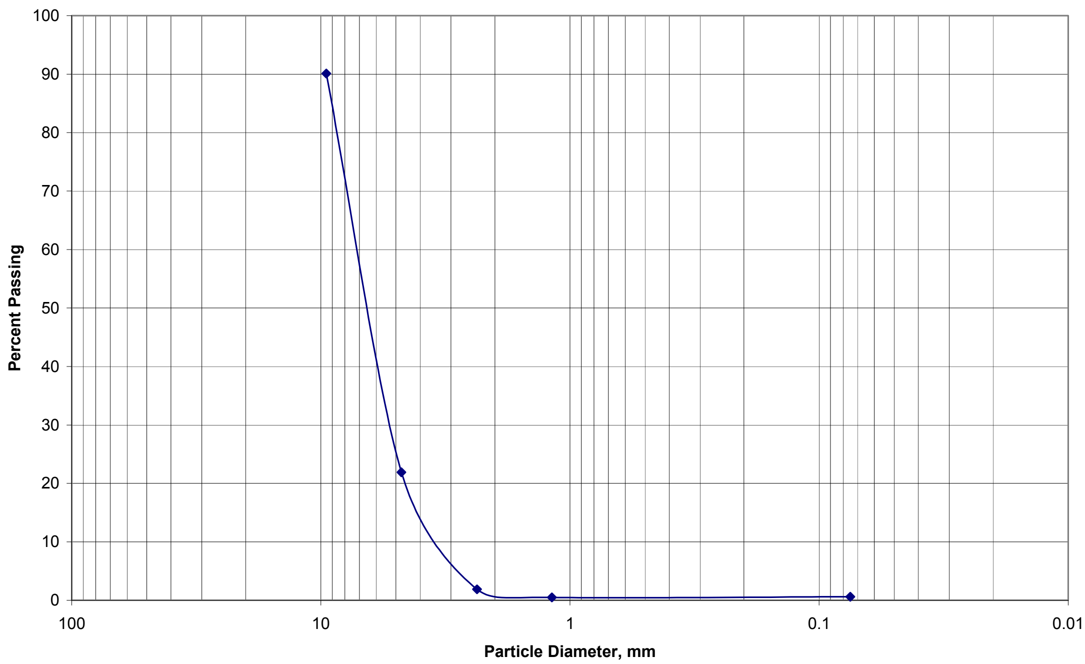


Low Absorption #8 Coarse Aggregate

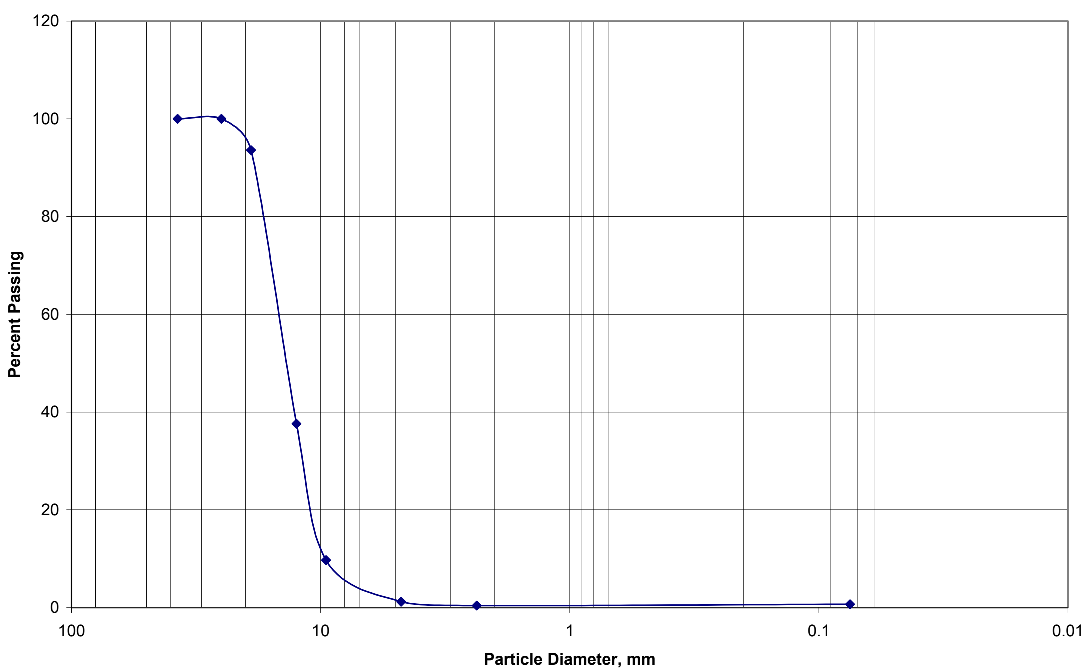



Low Absorption #57 Coarse Aggregate
Percent Passing
120
100
80
60
40
20
0
100
10
1
Particle Diameter, mm

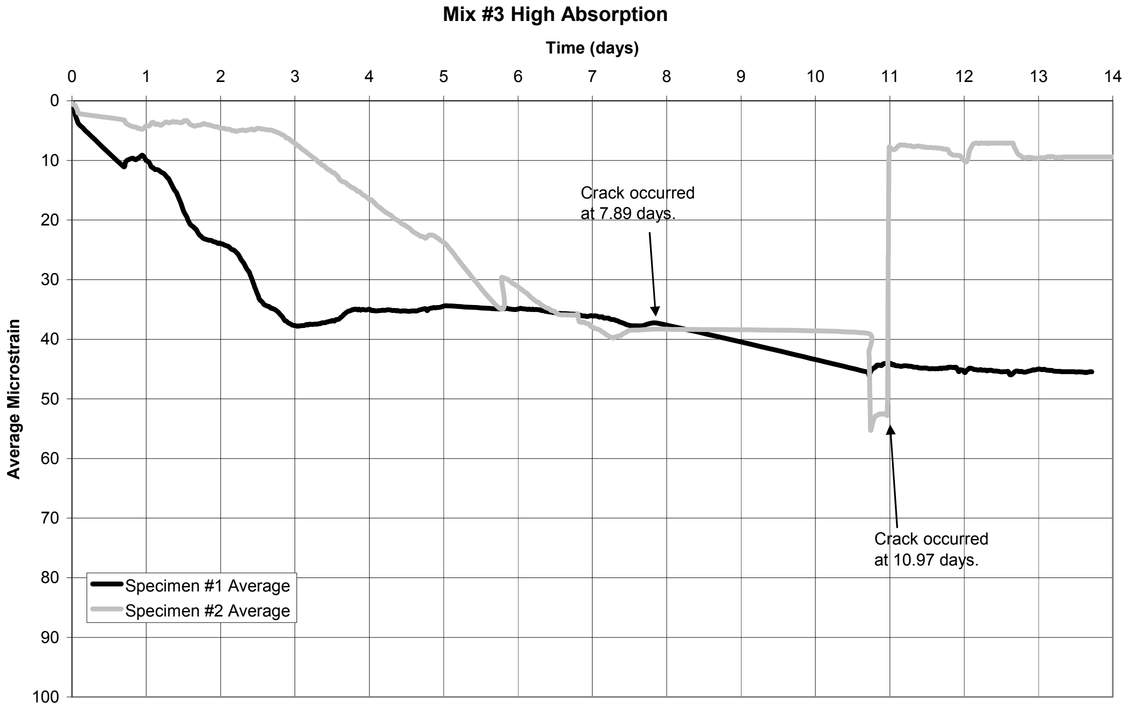
Fairborn Gravel #8

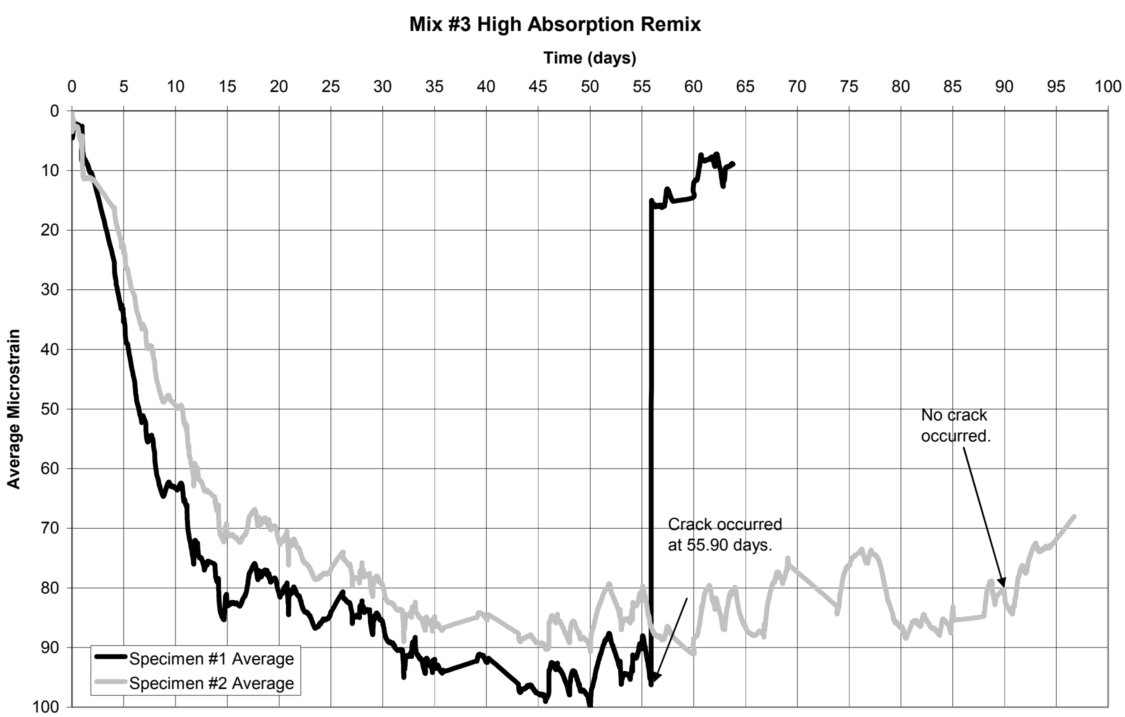


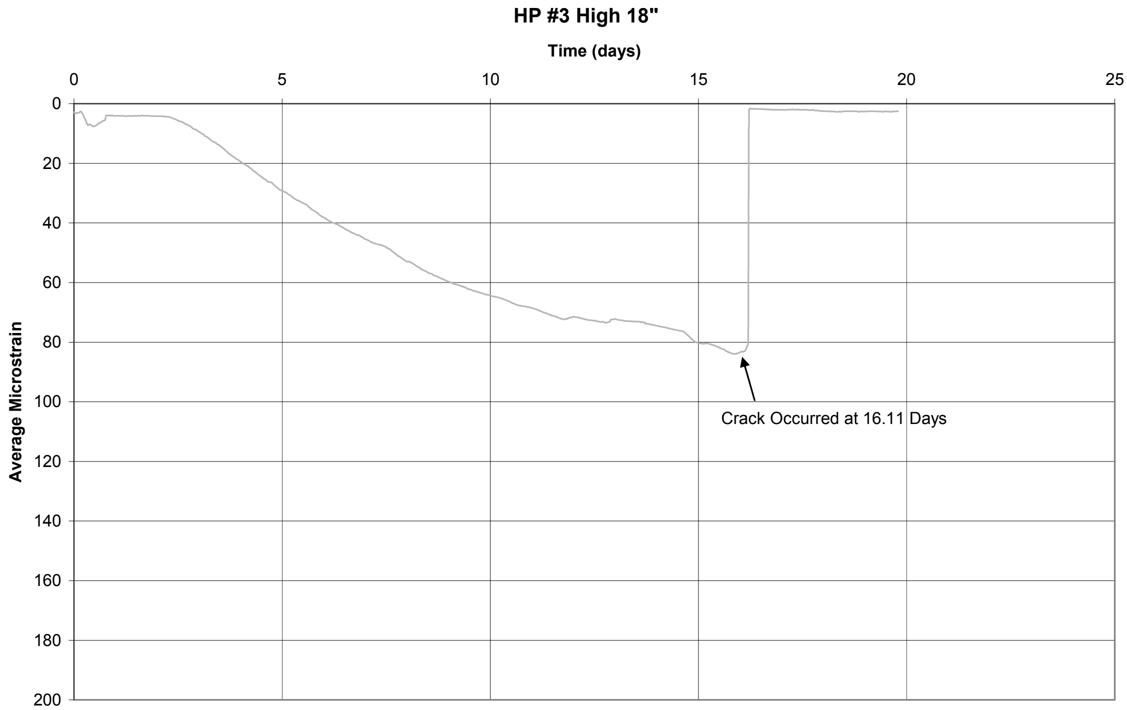
Fairborn Gravel #57

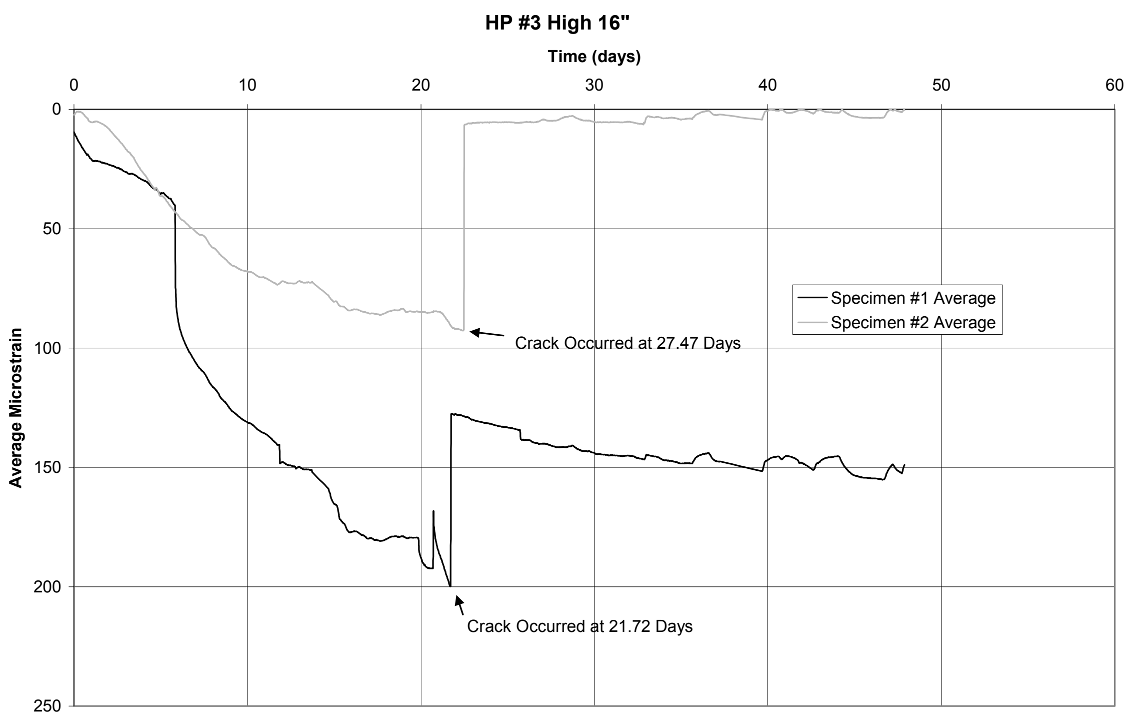


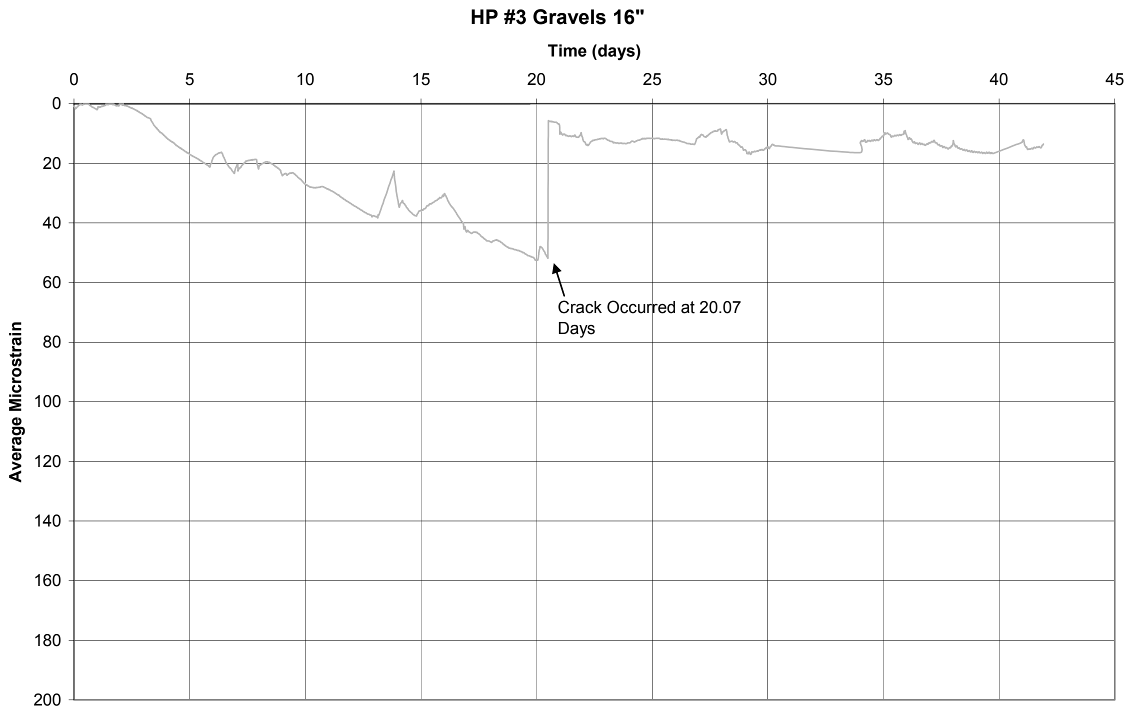
Appendix C: Restrained Shrinkage Strain Data

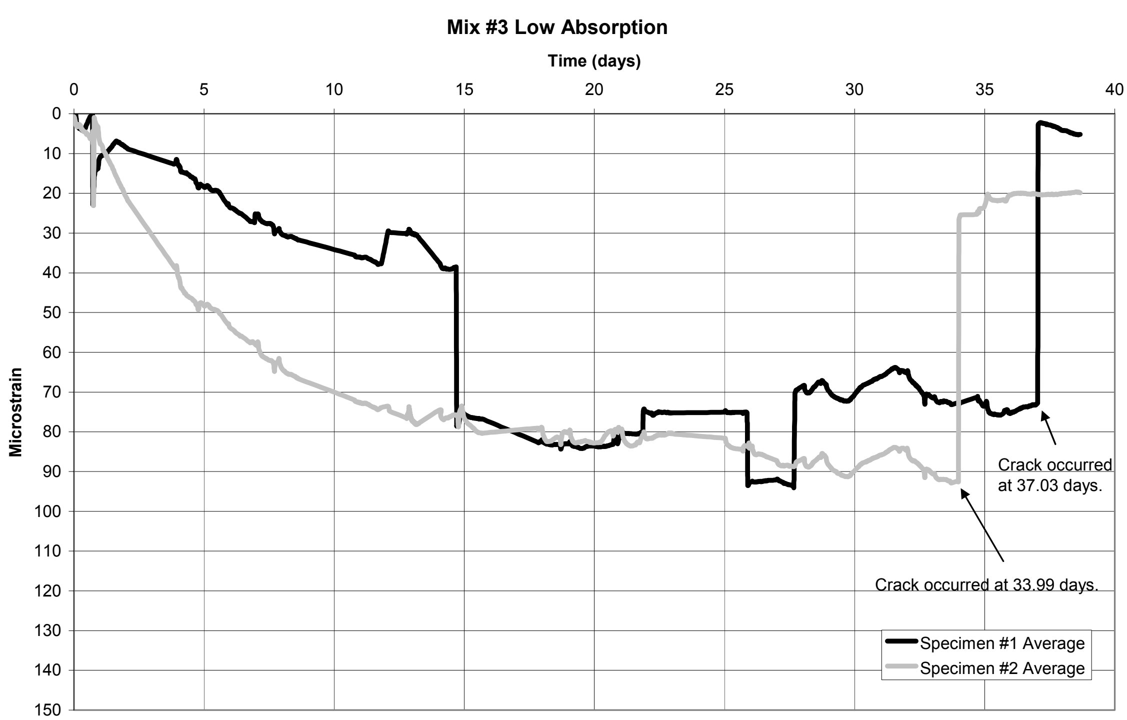




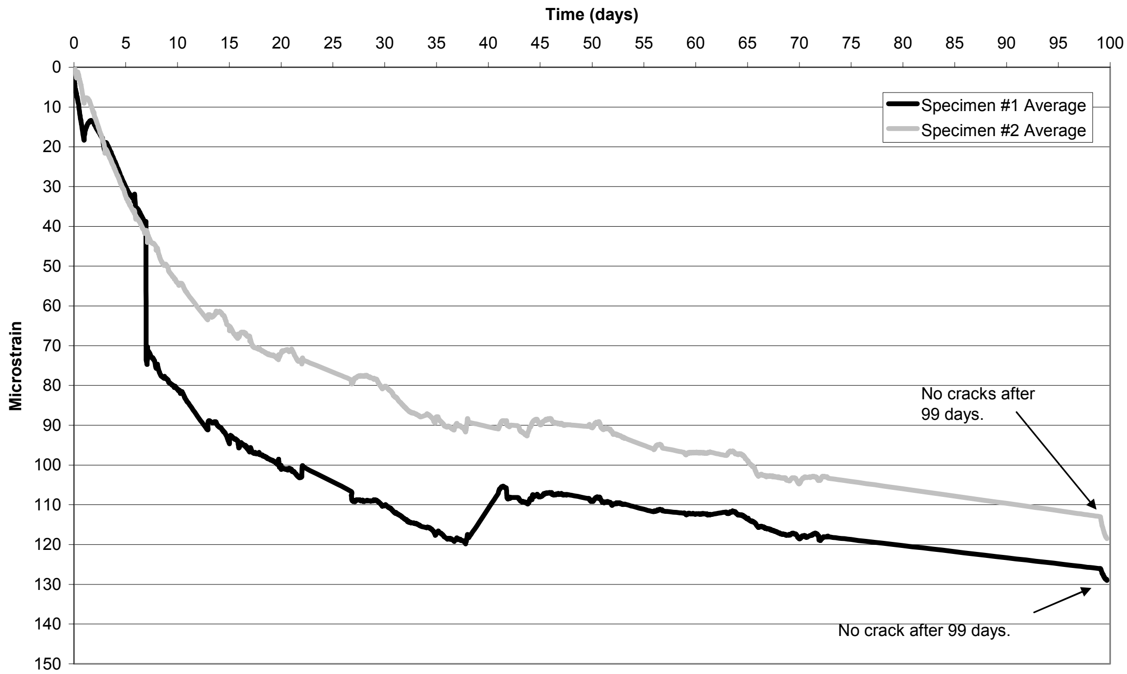

HP #3 High 18"
Time (days)
0
5
10
15
20
25
0
20
40
60
80
100
120
140
160
180
200
Average Microstrain
Crack Occurred at 16.11 Days


HP #3 High 16"
Time (days)
0
10
20
30
40
50
60
0
50
100
150
200
250
Average Microstrain
Specimen #1 Average
Specimen #2 Average
Crack Occurred at 27.47 Days
Crack Occurred at 21.72 Days

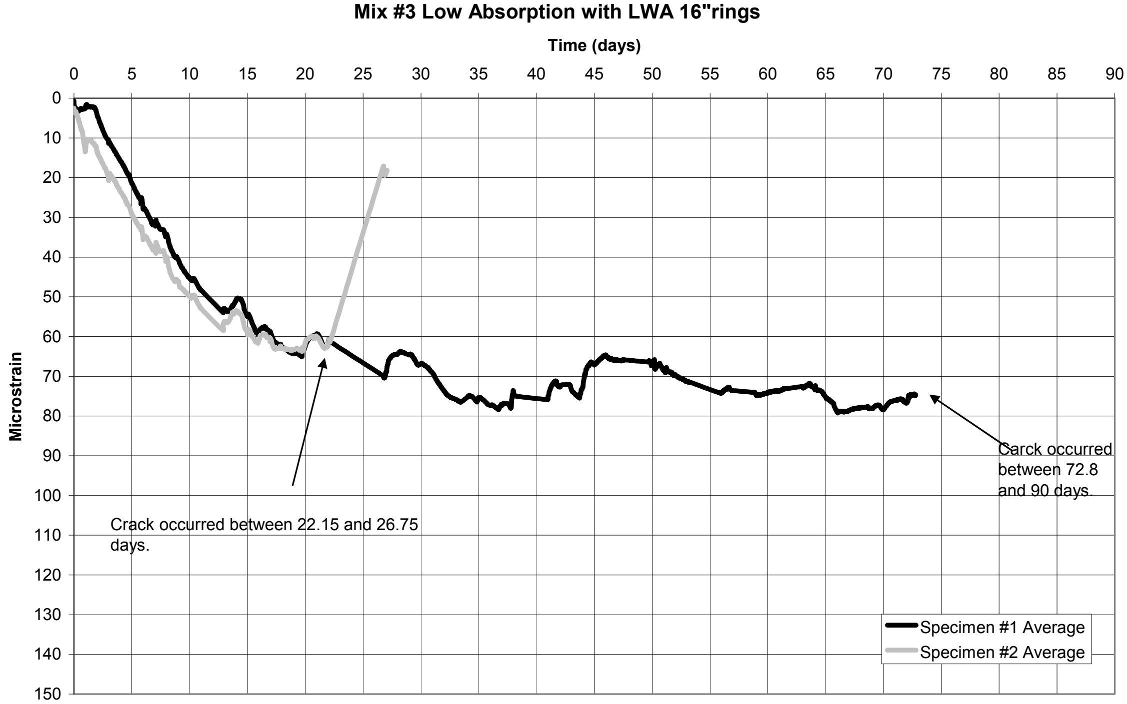

HP #3 Gravels 16"
Time (days)
0
5
10
15
20
25
30
35
40
45
Average Microstrain
0
20
40
60
80
100
120
140
160
180
200
Crack Occurred at 20.07
Days

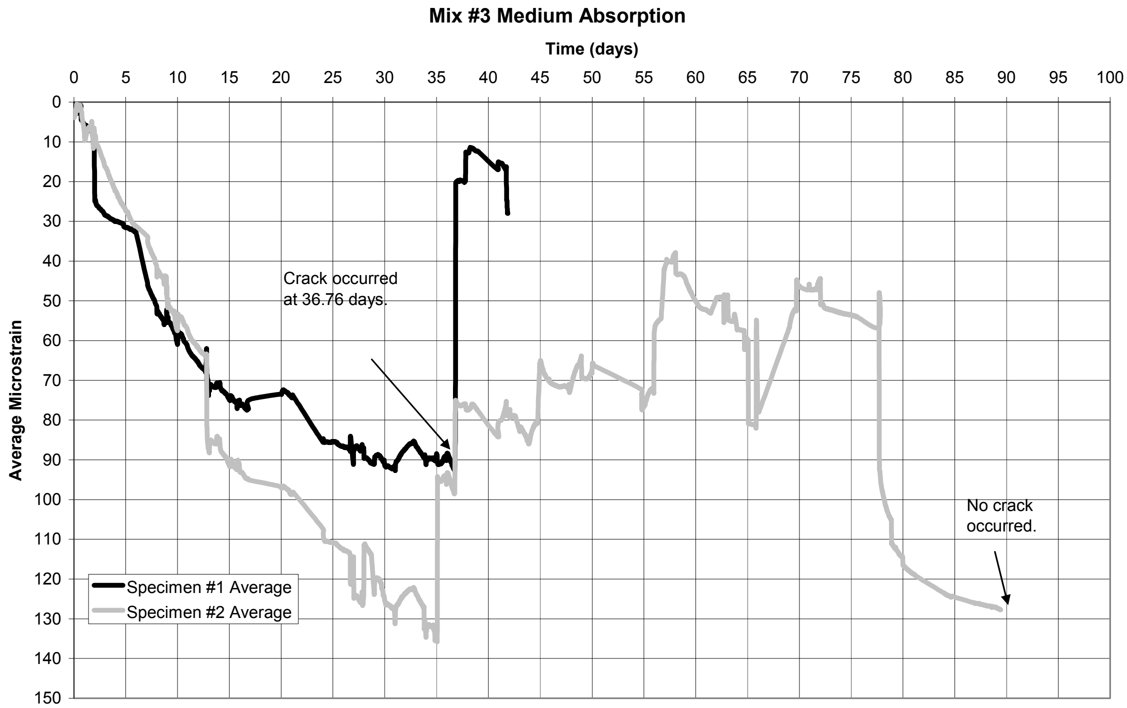

Mix #3 Low Absorption
Time (days)
0
5
10
15
20
25
30
35
40
Microstrain
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
Crack occurred
at 37.03 days.
Crack occurred at 33.99 days.
Specimen #1 Average
Specimen #2 Average

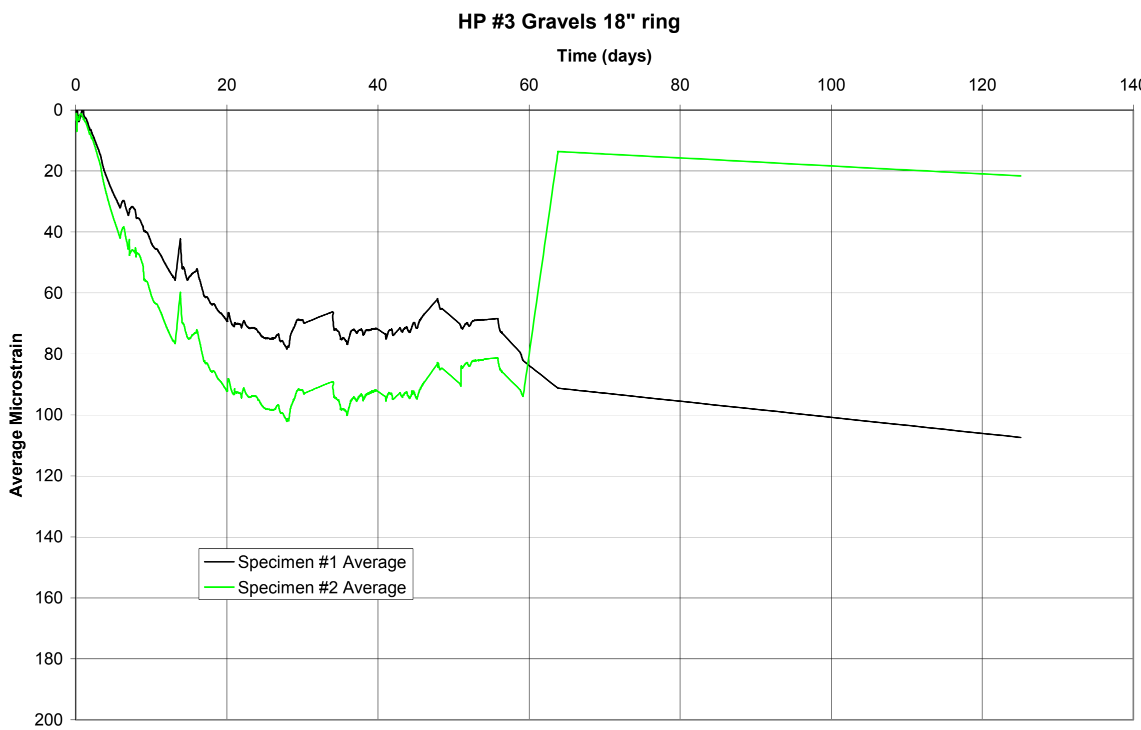
Cedarville #3 with LWA 18"rings

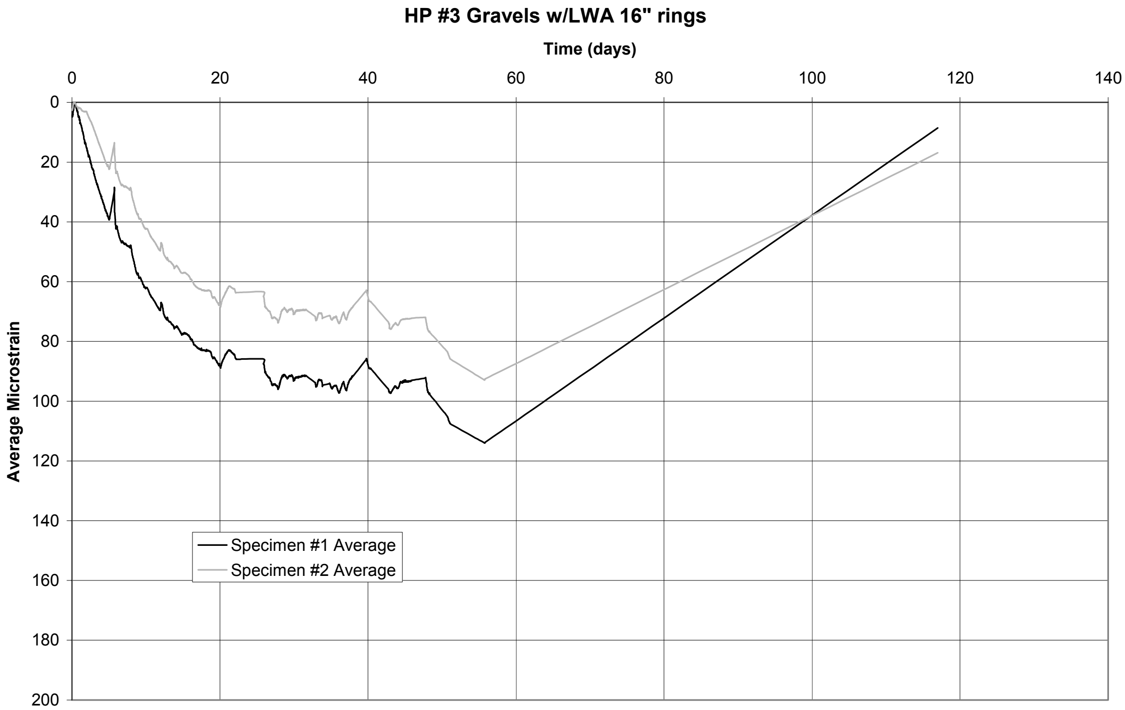


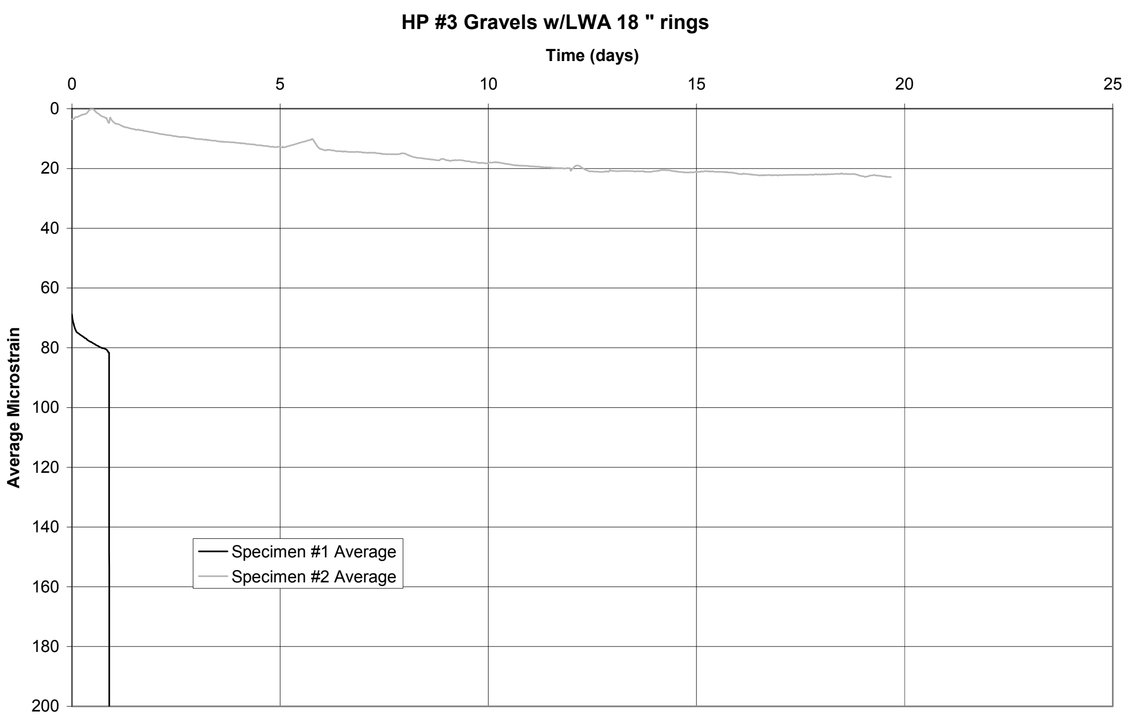
Mix #3 Low Absorption with LWA 16"rings

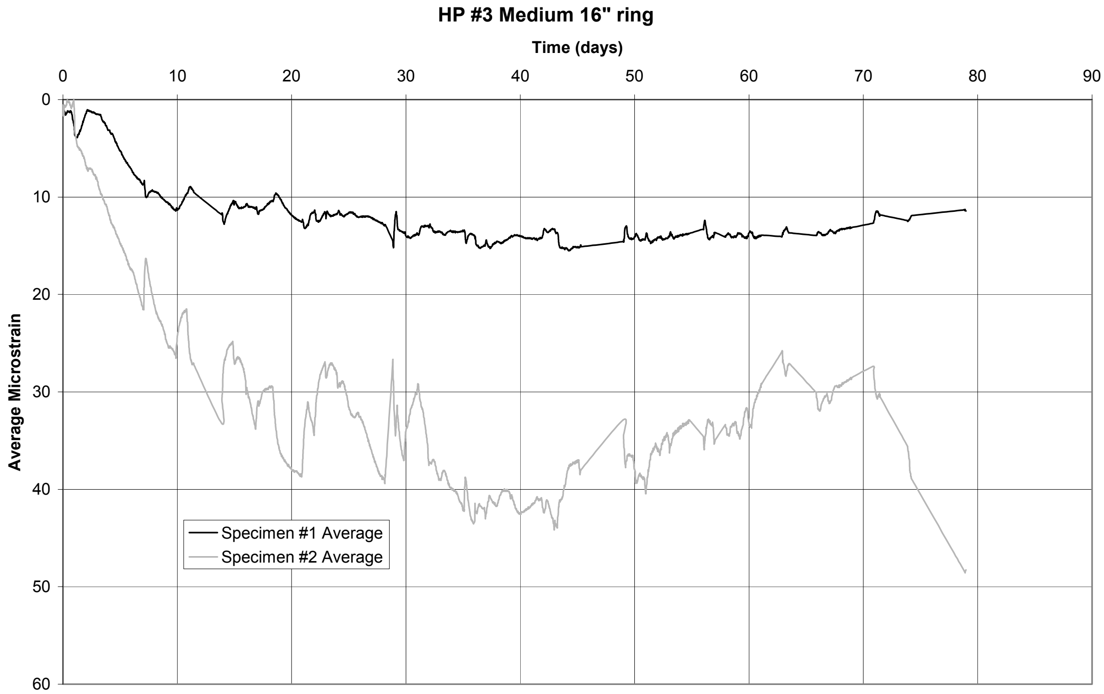


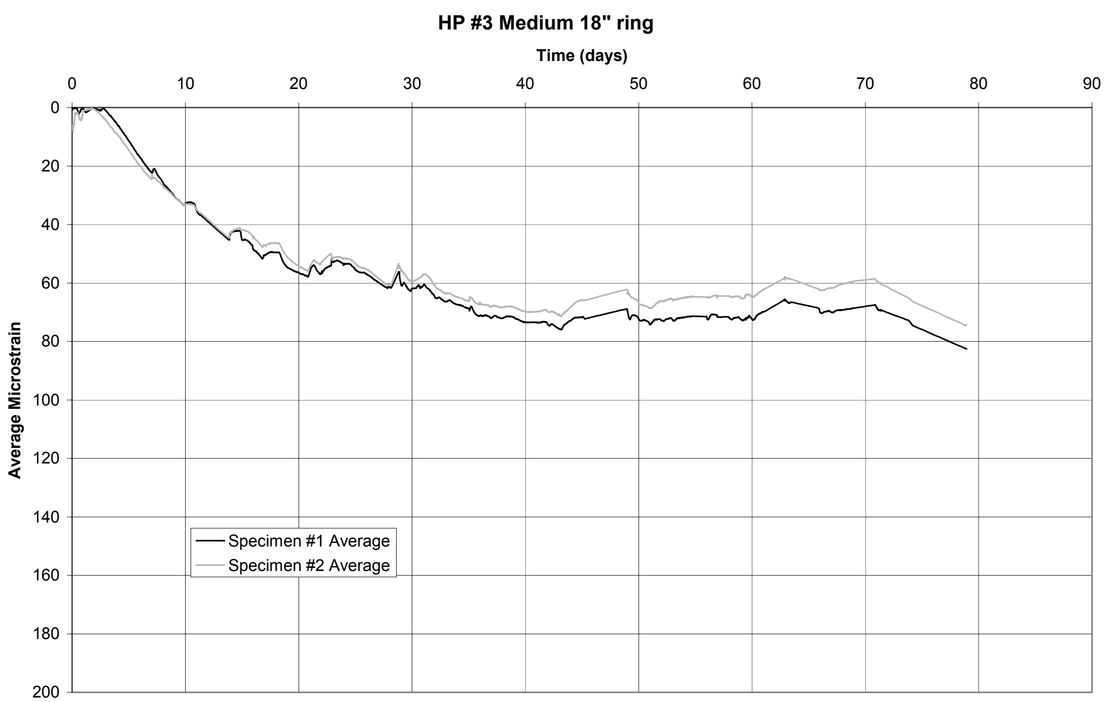

Mix #3 Medium Absorption
Time (days)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
95
100
Average Microstrain
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
Crack occurred
at 36.76 days.
No crack
occurred.
Specimen #1 Average
Specimen #2 Average

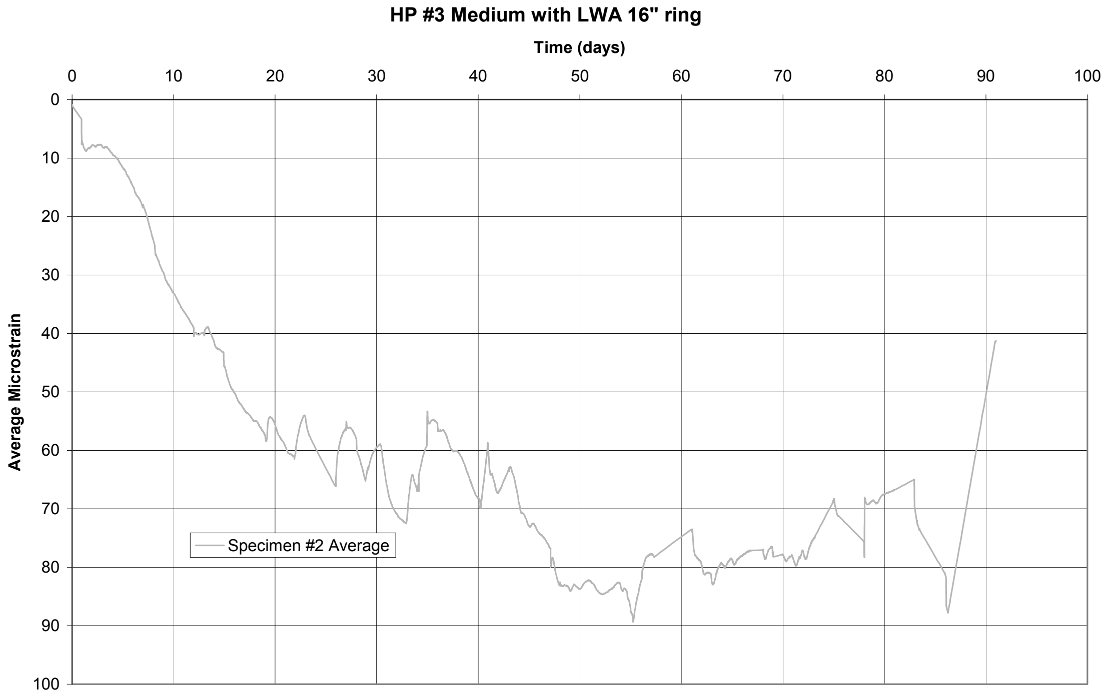

HP #3 Gravels 18" ring
Time (days)
0
20
40
60
80
100
120
140
Average Microstrain
0
20
40
60
80
100
120
140
160
180
200
Specimen #1 Average
Specimen #2 Average

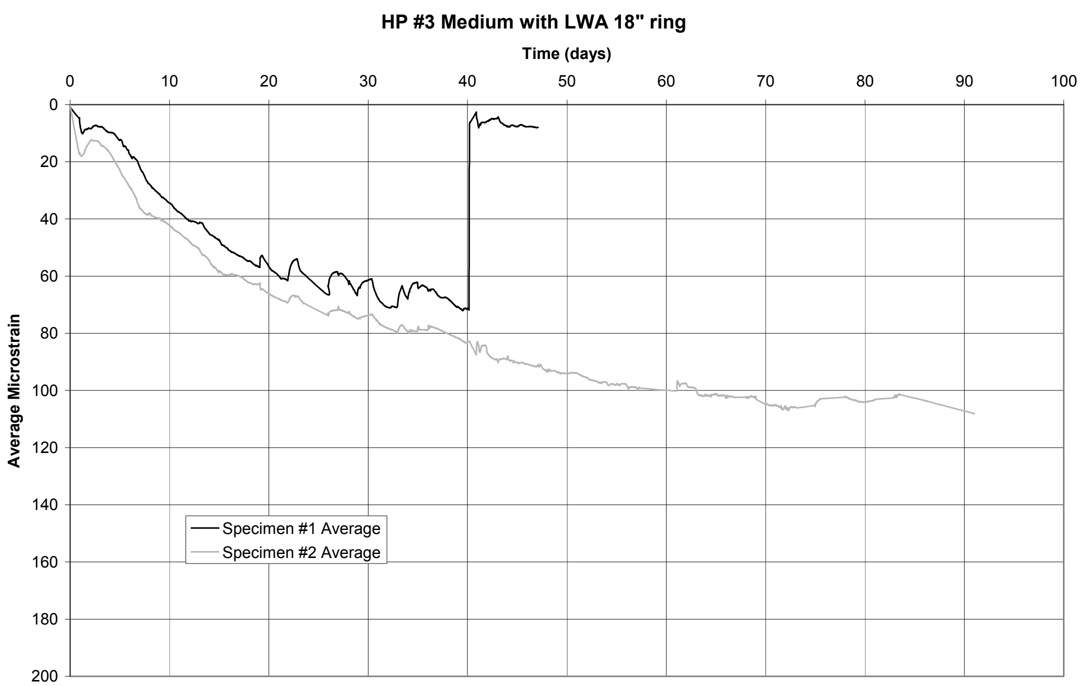

HP #3 Gravels w/LWA 16" rings
Time (days)
0
20
40
60
80
100
120
140
Average Microstrain
0
20
40
60
80
100
120
140
160
180
200
Specimen #1 Average
Specimen #2 Average

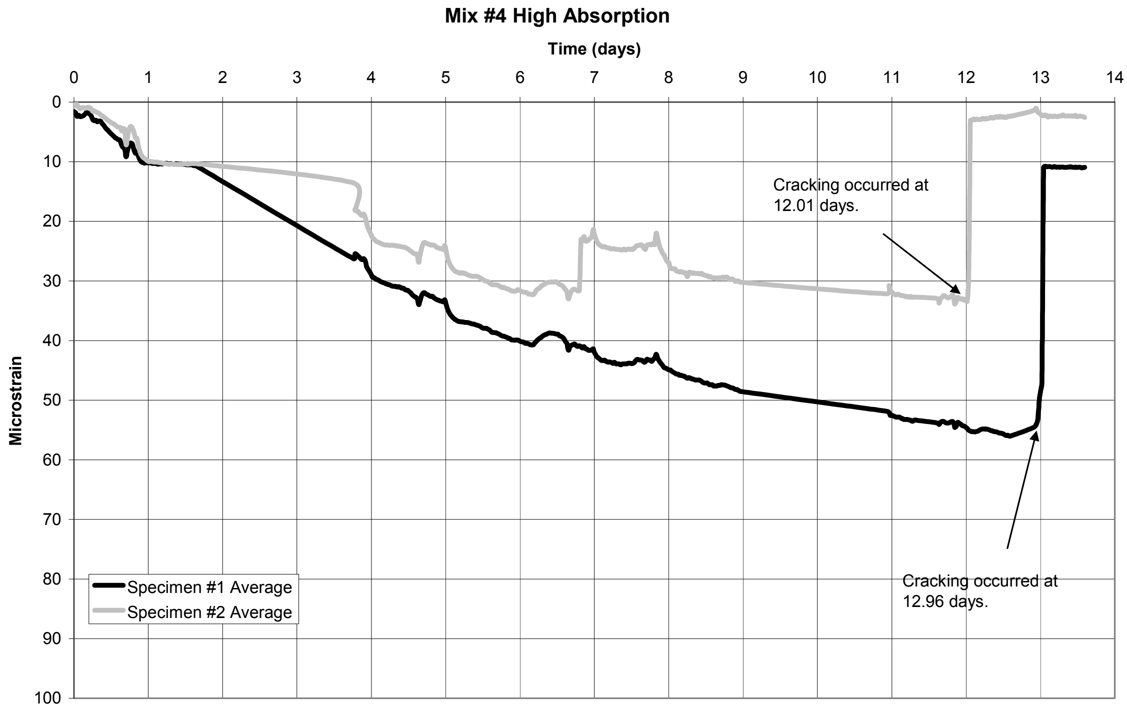

HP #3 Gravels w/LWA 18 " rings
Time (days)
0
5
10
15
20
25
Average Microstrain
0
20
40
60
80
100
120
140
160
180
200
Specimen #1 Average
Specimen #2 Average

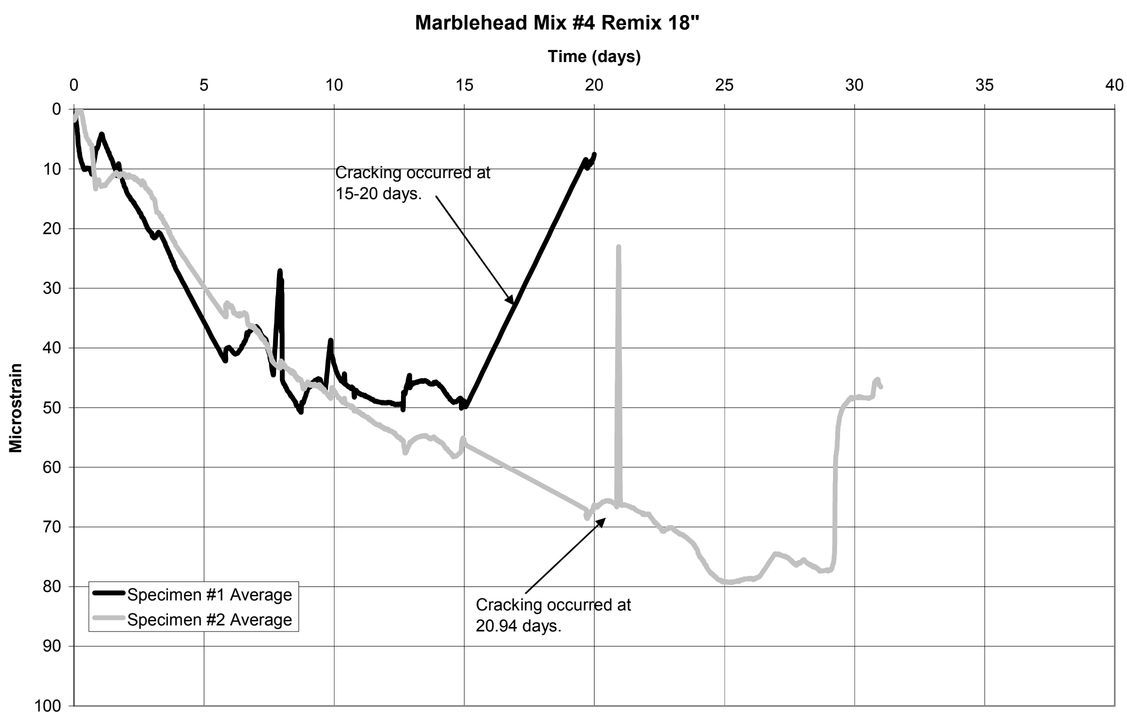

HP #3 Medium 16" ring
Time (days)
0
10
20
30
40
50
60
70
80
90
Average Microstrain
0
10
20
30
40
50
60
Specimen #1 Average
Specimen #2 Average

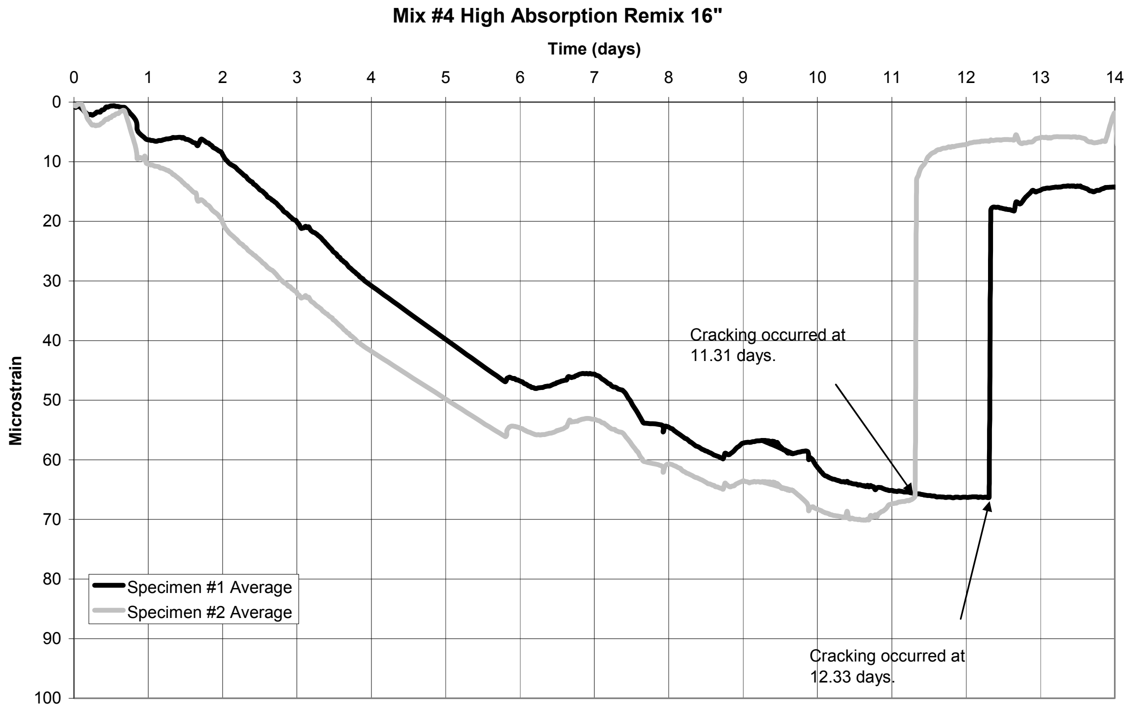

HP #3 Medium 18" ring
Time (days)
0
10
20
30
40
50
60
70
80
90
Average Microstrain
0
20
40
60
80
100
120
140
160
180
200
Specimen #1 Average
Specimen #2 Average

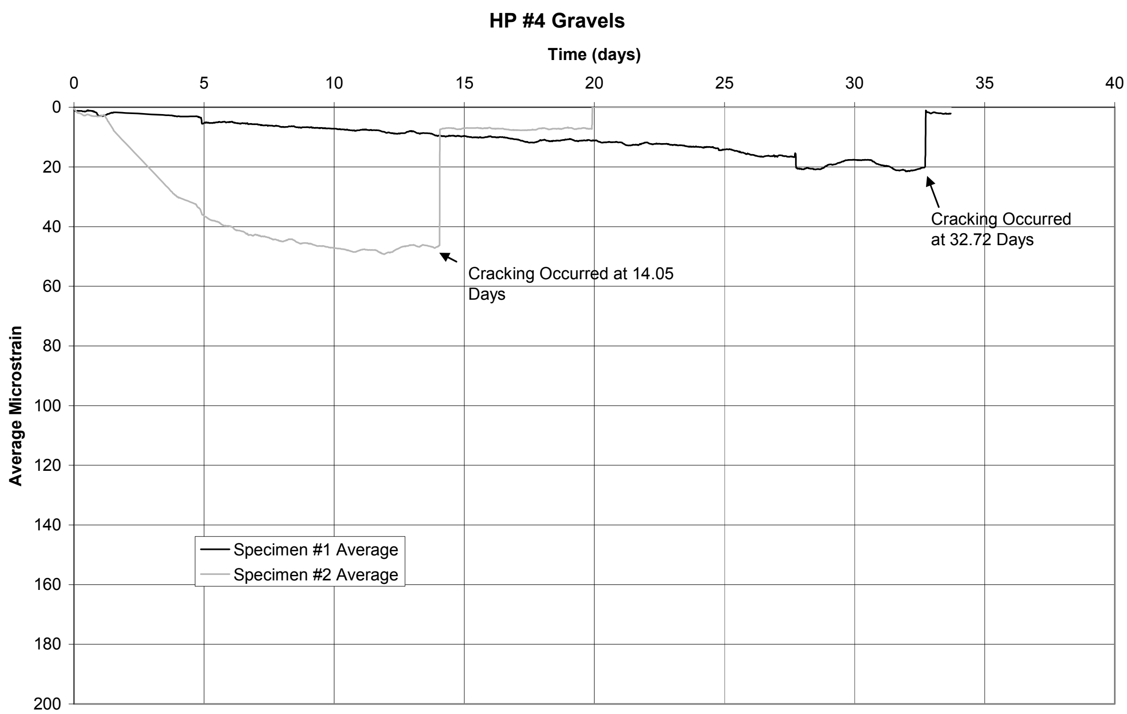

HP #3 Medium with LWA 16" ring
Time (days)
0
10
20
30
40
50
60
70
80
90
100
Average Microstrain
Specimen #2 Average

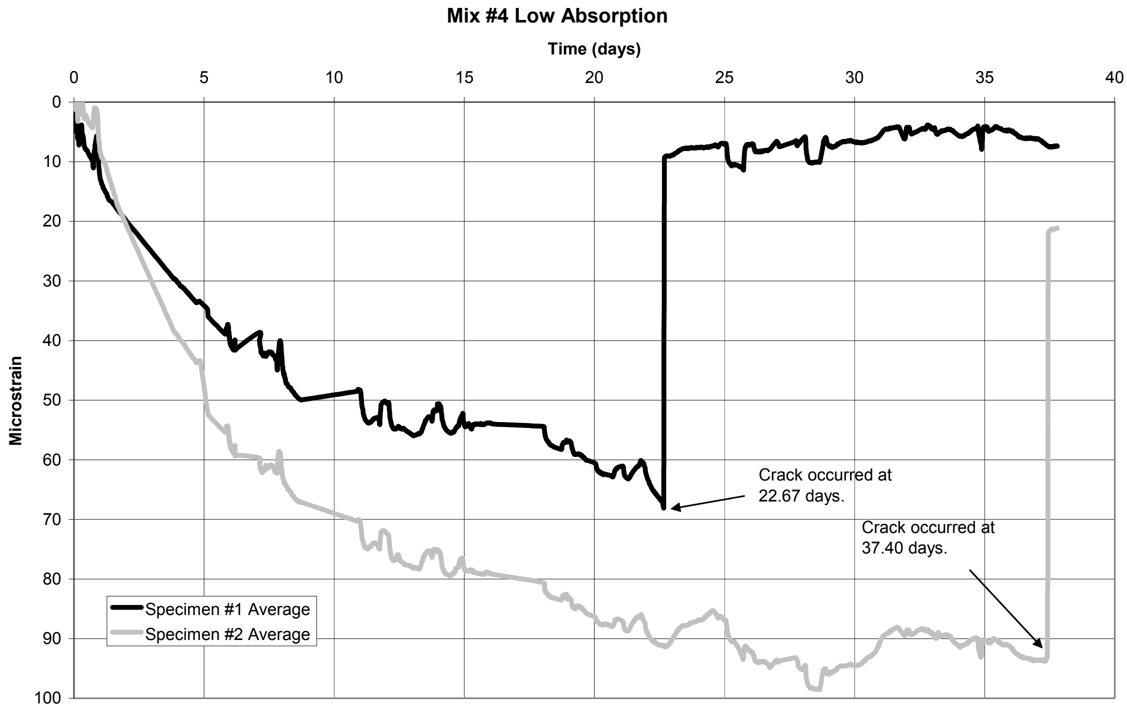

HP #3 Medium with LWA 18" ring
Time (days)
0
10
20
30
40
50
60
70
80
90
100
Average Microstrain
0
20
40
60
80
100
120
140
160
180
200
Specimen #1 Average
Specimen #2 Average

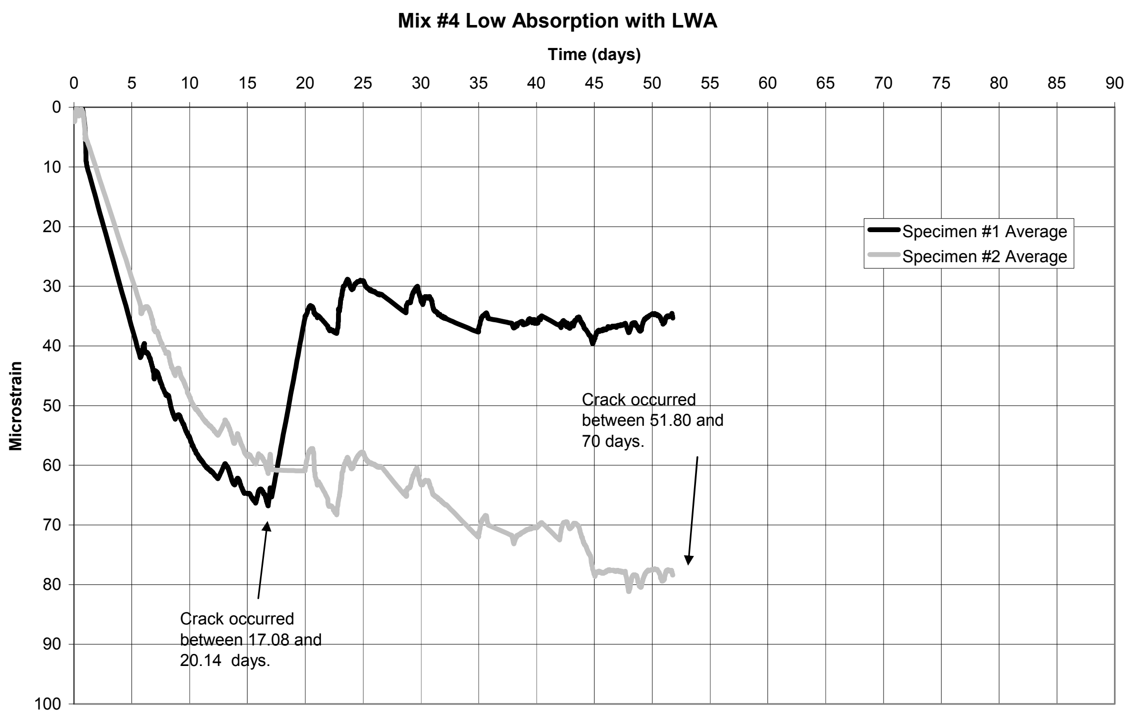

Mix #4 High Absorption
Time (days)
Microstrain
Cracking occurred at
12.01 days.
Cracking occurred at
12.96 days.
Specimen #1 Average
Specimen #2 Average

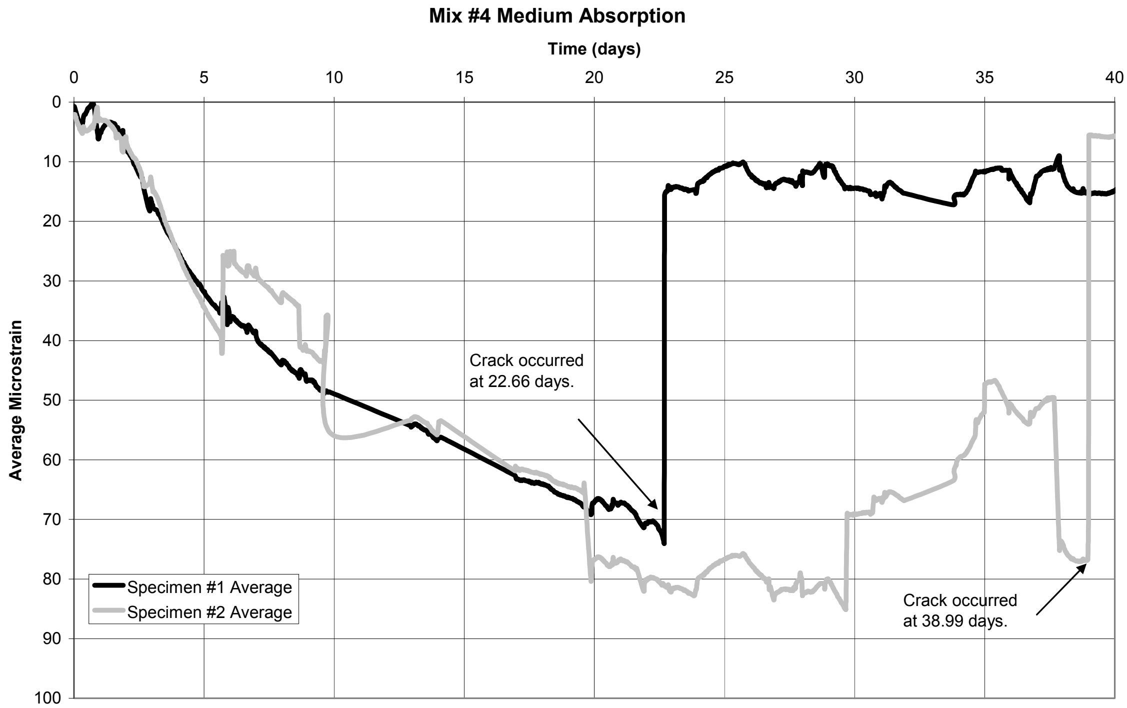

Marblehead Mix #4 Remix 18"
Time (days)
Microstrain
Cracking occurred at
15-20 days.
Cracking occurred at
20.94 days.
Specimen #1 Average
Specimen #2 Average

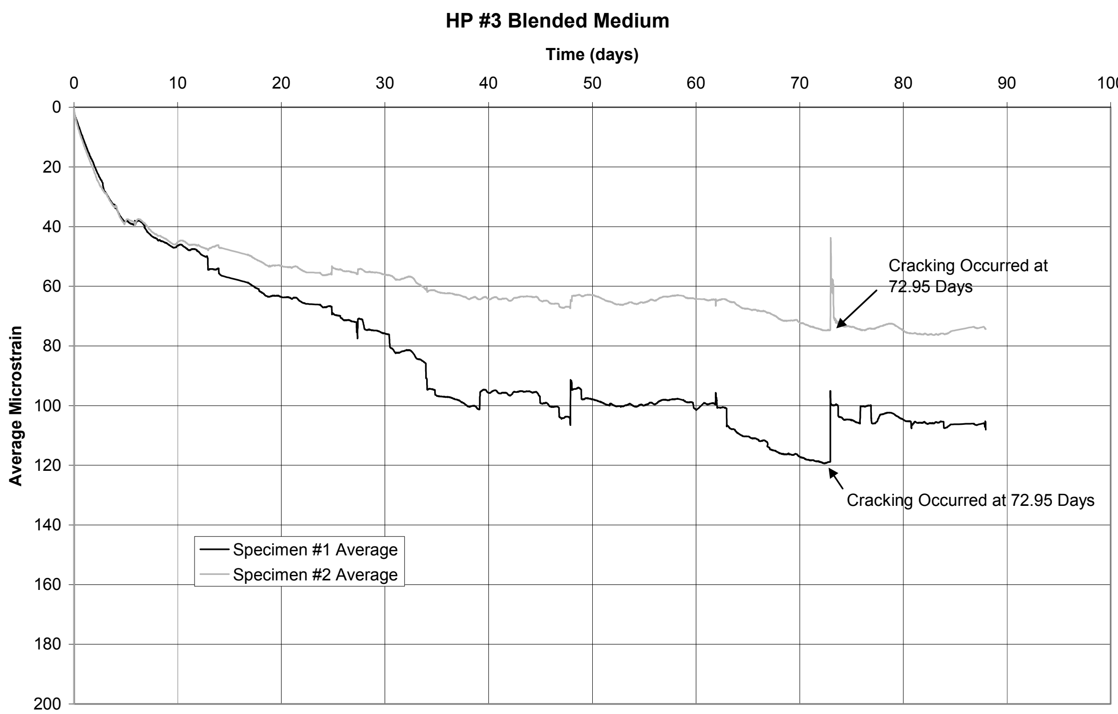

Mix #4 High Absorption Remix 16"
Time (days)
Microstrain
Cracking occurred at 11.31 days.
Cracking occurred at 12.33 days.
Specimen #1 Average
Specimen #2 Average

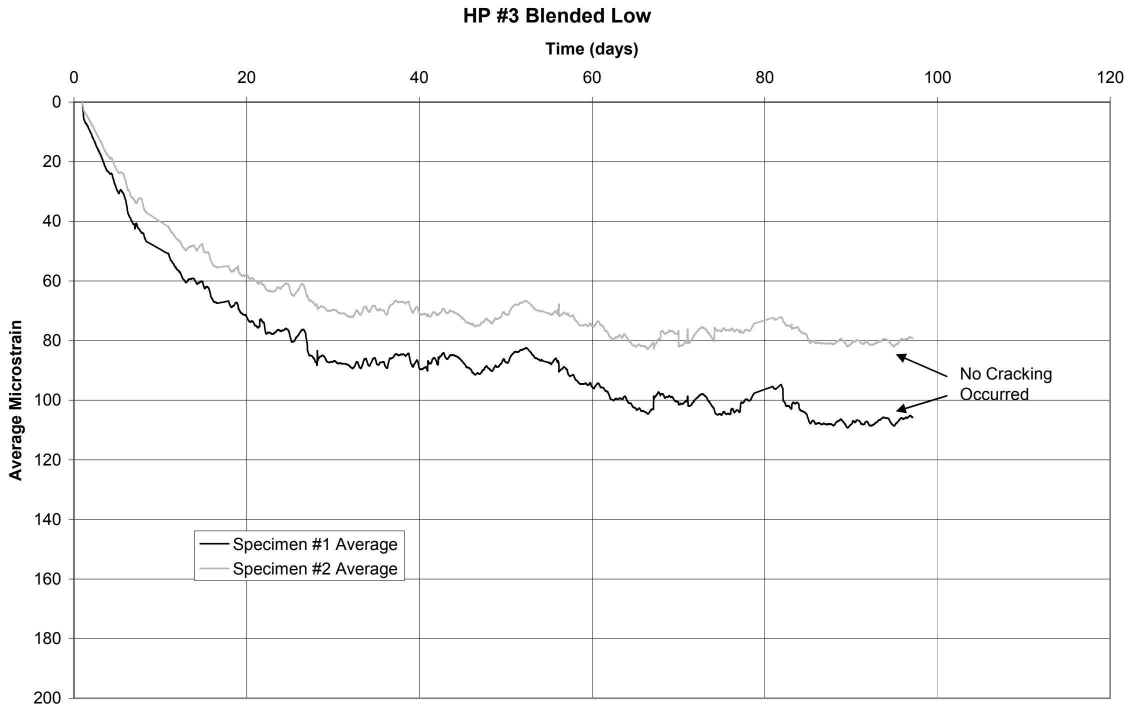

HP #4 Gravels
Time (days)
Average Microstrain
Cracking Occurred at 14.05 Days
Cracking Occurred at 32.72 Days
Specimen #1 Average
Specimen #2 Average

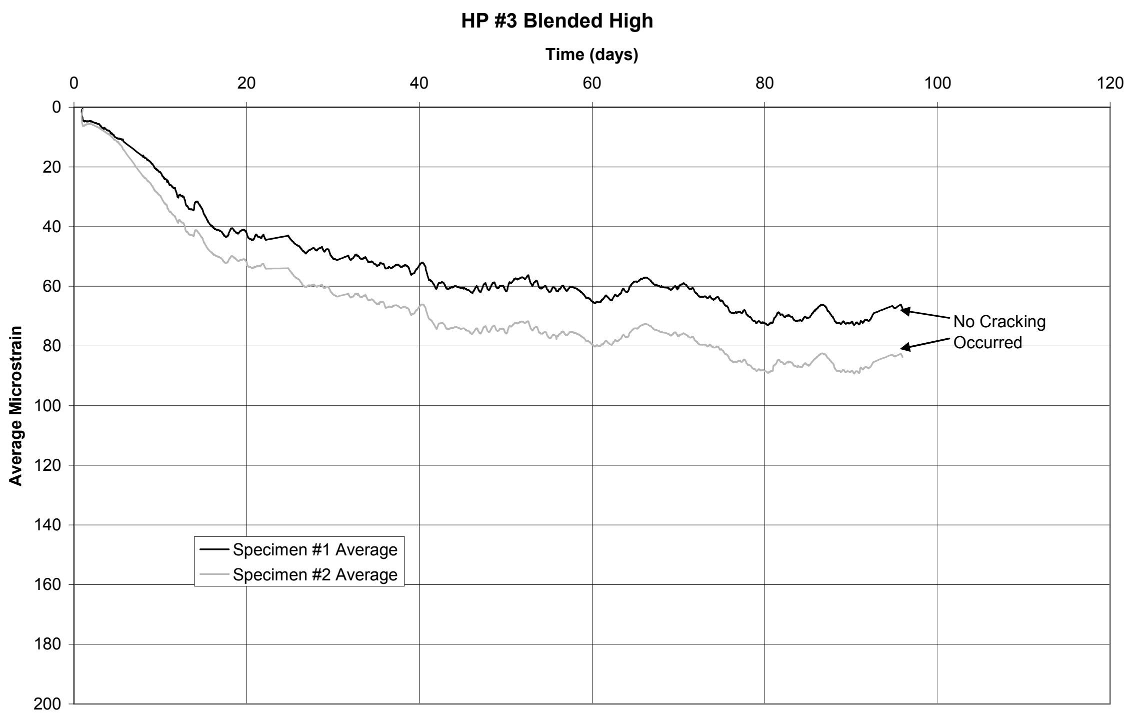

Mix #4 Low Absorption
Time (days)
0
5
10
15
20
25
30
35
40
0
10
20
30
40
50
60
70
80
90
100
Microstrain
Crack occurred at 22.67 days.
Crack occurred at 37.40 days.
Specimen #1 Average
Specimen #2 Average

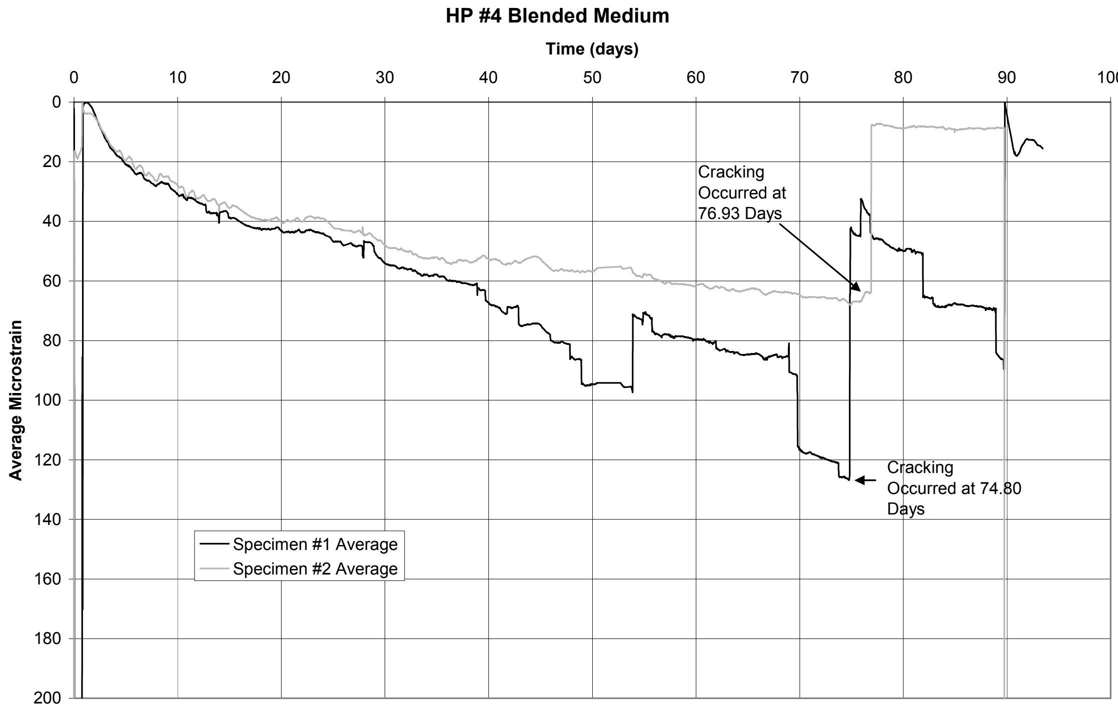

Mix #4 Low Absorption with LWA
Time (days)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
0
10
20
30
40
50
60
70
80
90
100
Microstrain
Specimen #1 Average
Specimen #2 Average
Crack occurred between 51.80 and 70 days.
Crack occurred between 17.08 and 20.14 days.


Mix #4 Medium Absorption
Time (days)
0
5
10
15
20
25
30
35
40
0
10
20
30
40
50
60
70
80
90
100
Average Microstrain
Crack occurred
at 22.66 days.
Crack occurred
at 38.99 days.
Specimen #1 Average
Specimen #2 Average

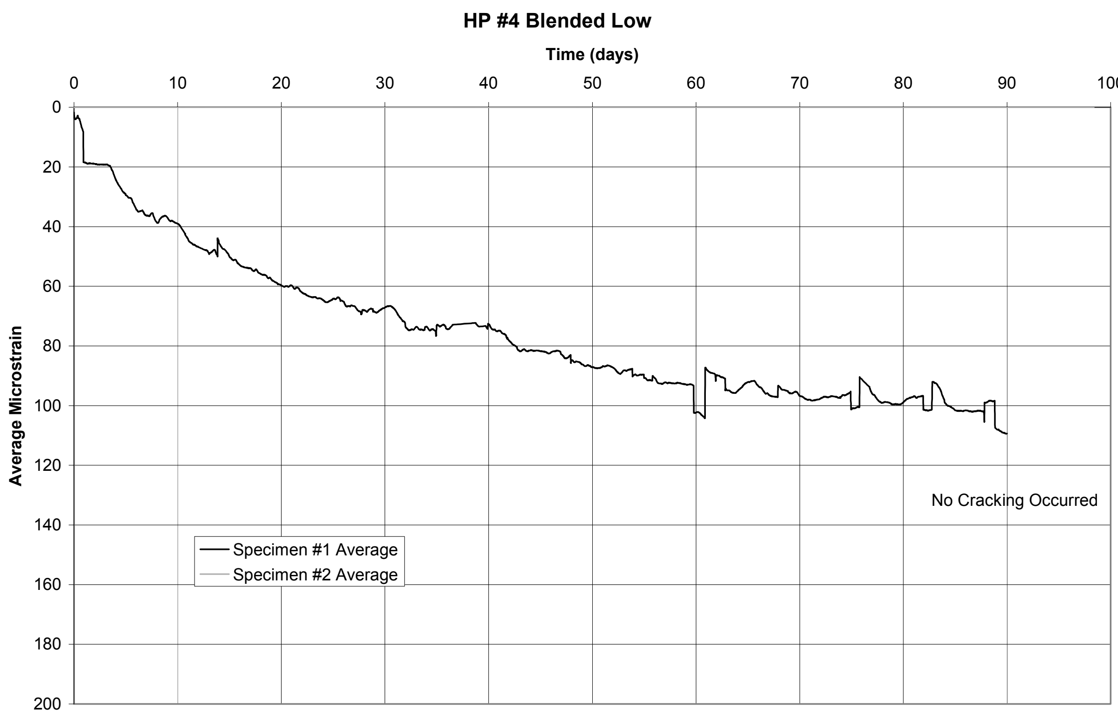

HP #3 Blended Medium
Time (days)
0
10
20
30
40
50
60
70
80
90
100
0
20
40
60
80
100
120
140
160
180
200
Average Microstrain
Cracking Occurred at
72.95 Days
Cracking Occurred at 72.95 Days
Specimen #1 Average
Specimen #2 Average


HP #3 Blended Low
Time (days)
0
20
40
60
80
100
120
0
20
40
60
80
100
120
140
160
180
200
Average Microstrain
No Cracking
Occurred
Specimen #1 Average
Specimen #2 Average

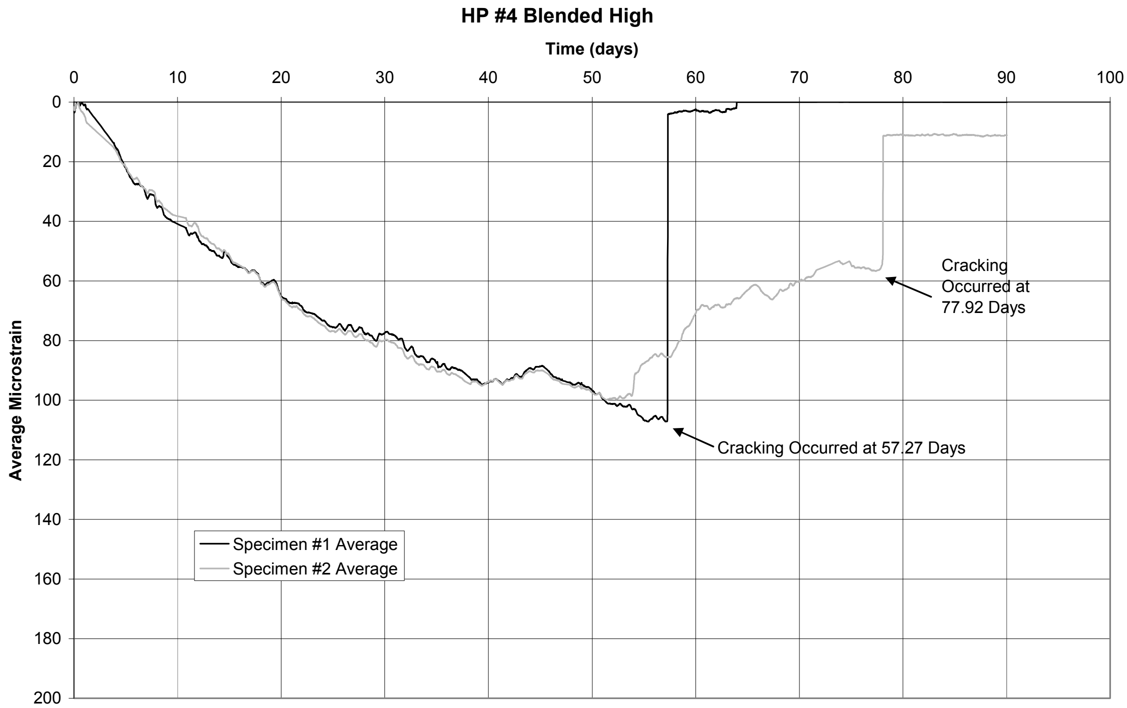

HP #3 Blended High
Time (days)
0
20
40
60
80
100
120
0
20
40
60
80
100
120
140
160
180
200
Average Microstrain
No Cracking
Occurred
Specimen #1 Average
Specimen #2 Average



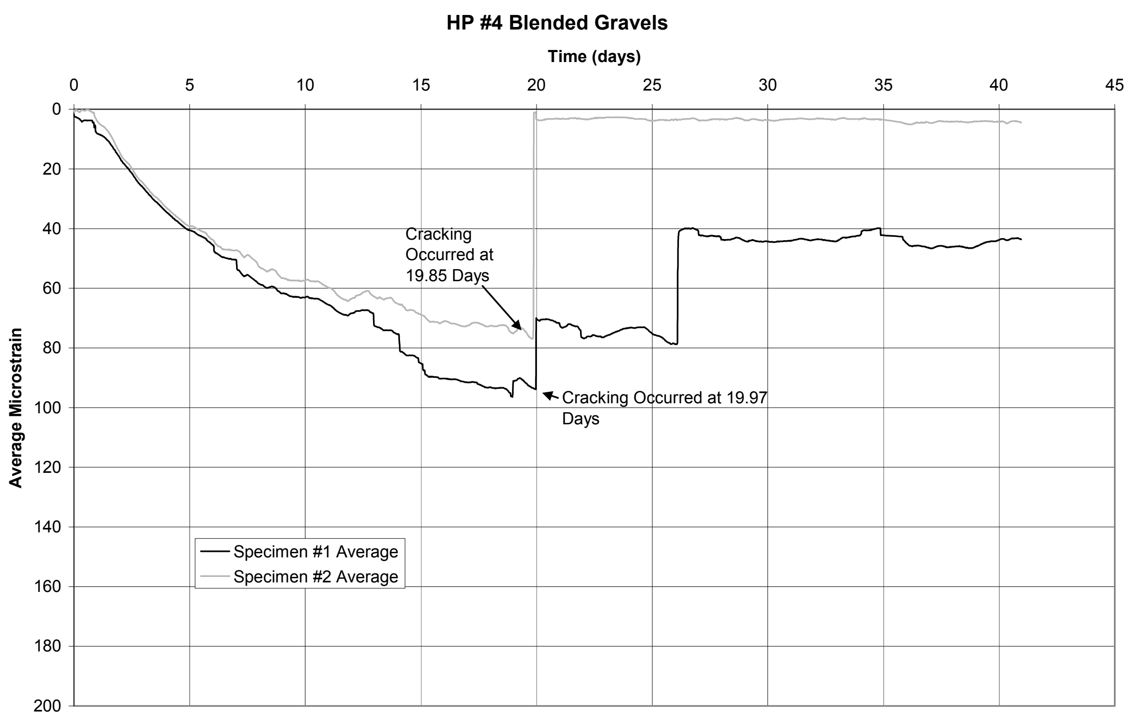
HP #4 Blended Medium
Time (days)
0
10
20
30
40
50
60
70
80
90
100
Average Microstrain
0
20
40
60
80
100
120
140
160
180
200
Cracking Occurred at 76.93 Days
Cracking Occurred at 74.80 Days
Specimen #1 Average
Specimen #2 Average



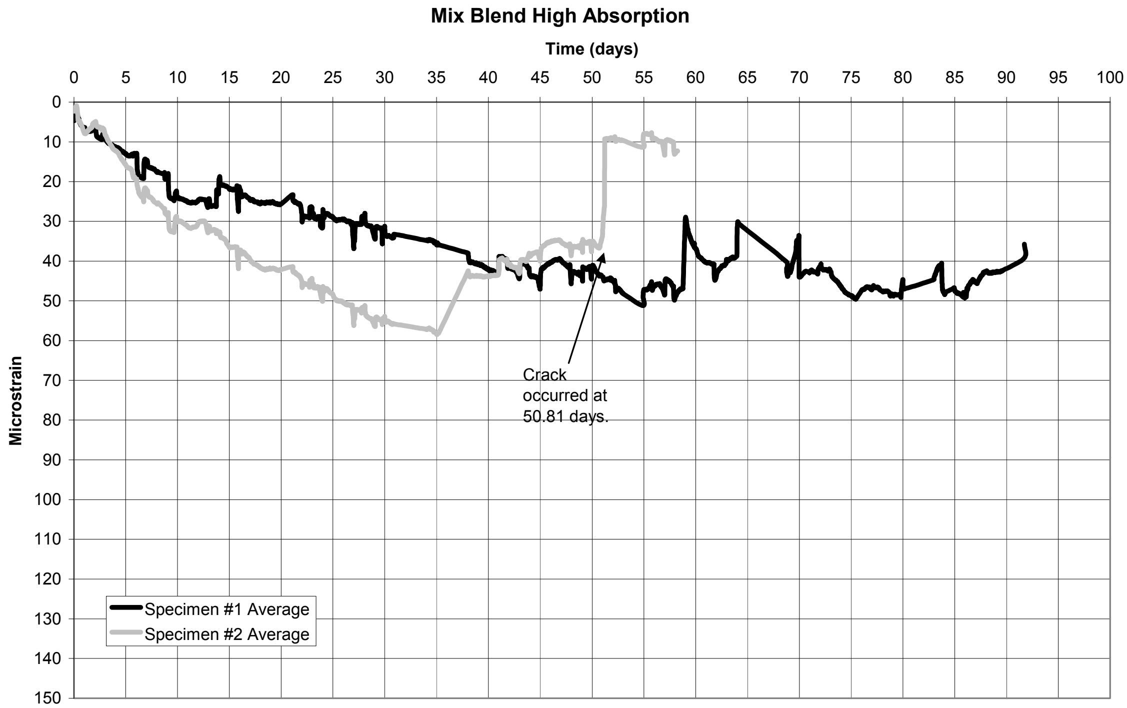
HP #4 Blended Low
Time (days)
0
10
20
30
40
50
60
70
80
90
100
Average Microstrain
0
20
40
60
80
100
120
140
160
180
200
No Cracking Occurred
Specimen #1 Average
Specimen #2 Average

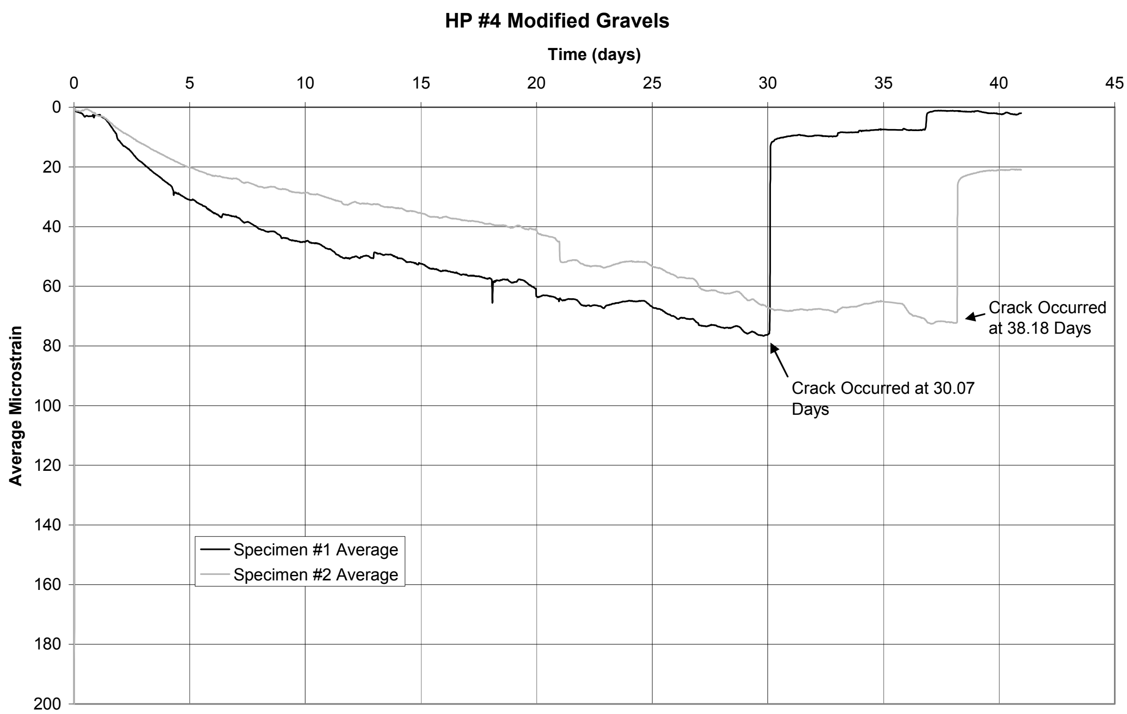


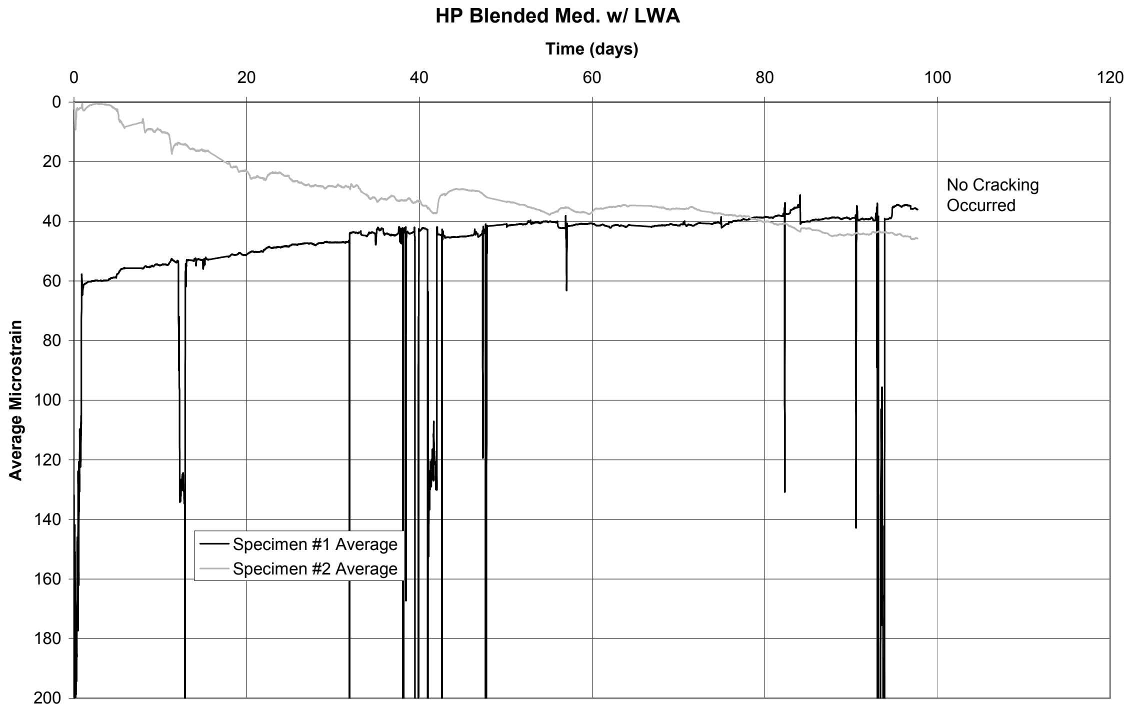
HP #4 Blended High
Time (days)
0
10
20
30
40
50
60
70
80
90
100
0
20
40
60
80
100
120
140
160
180
200
Average Microstrain
Cracking Occurred at 77.92 Days
Cracking Occurred at 57.27 Days
Specimen #1 Average
Specimen #2 Average

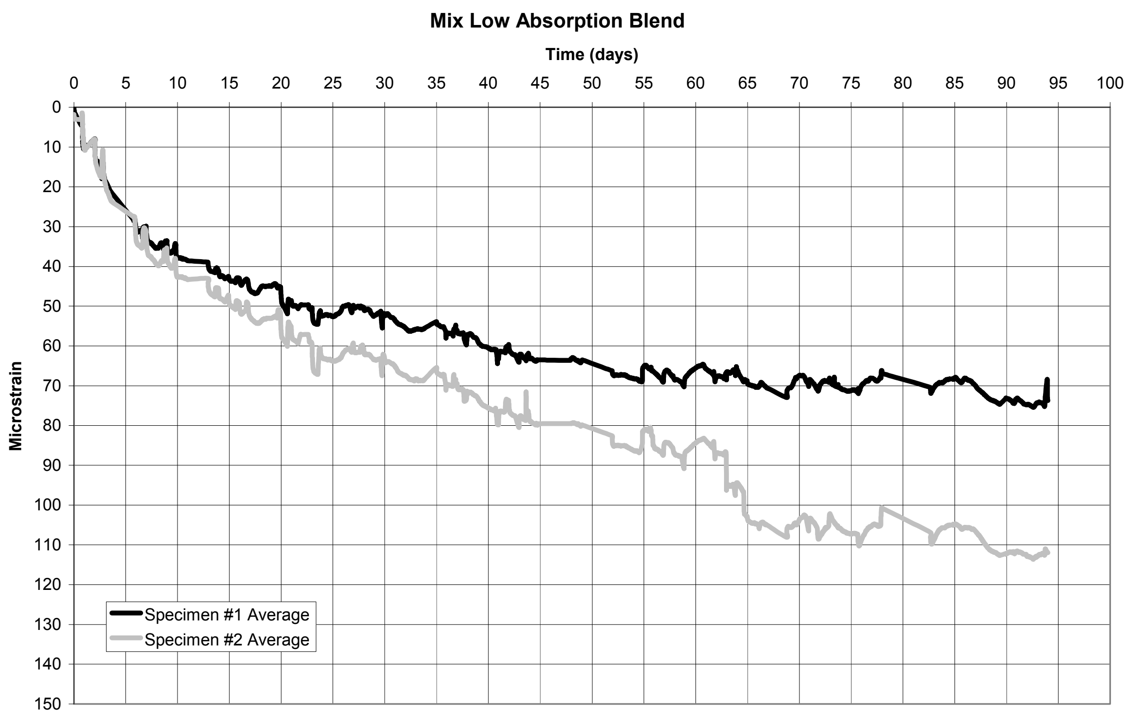


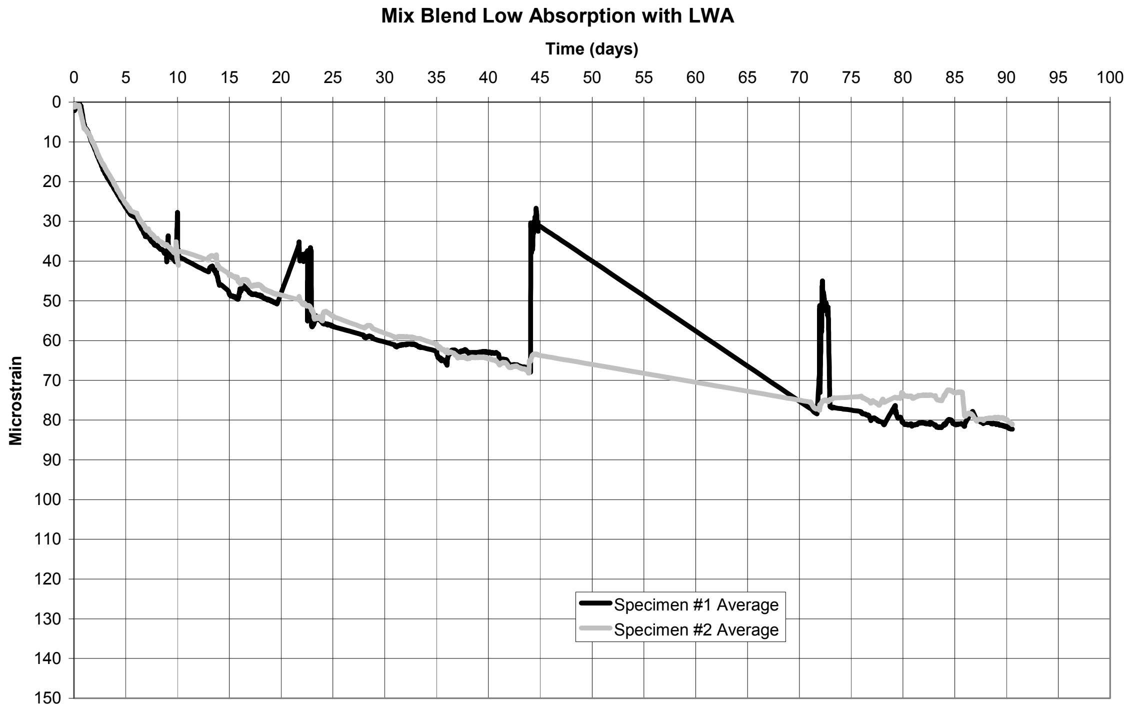
HP #4 Blended Gravels
Time (days)
0
5
10
15
20
25
30
35
40
45
0
20
40
60
80
100
120
140
160
180
200
Average Microstrain
Cracking Occurred at 19.85 Days
Cracking Occurred at 19.97 Days
Specimen #1 Average
Specimen #2 Average

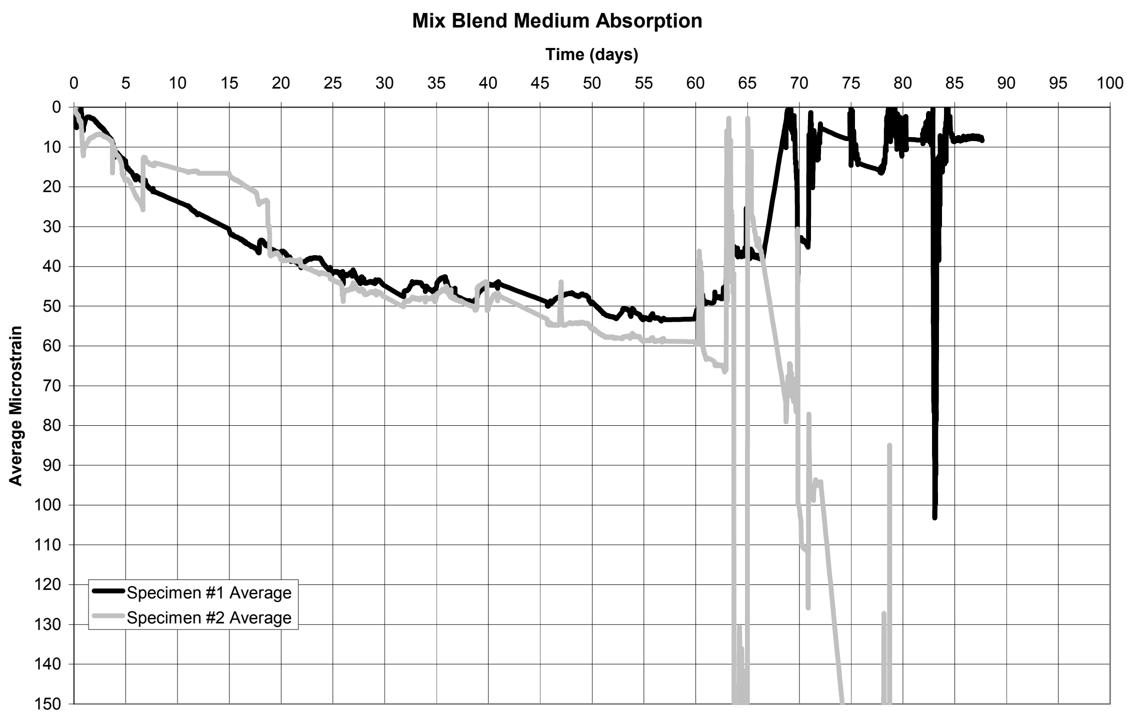

Mix Blend High Absorption
Time (days)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
95
100
Microstrain
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
Crack occurred at 50.81 days.
Specimen #1 Average
Specimen #2 Average


HP #4 Modified Gravels
Time (days)
0
5
10
15
20
25
30
35
40
45
Average Microstrain
0
20
40
60
80
100
120
140
160
180
200
Crack Occurred at 38.18 Days
Crack Occurred at 30.07 Days
Specimen #1 Average
Specimen #2 Average

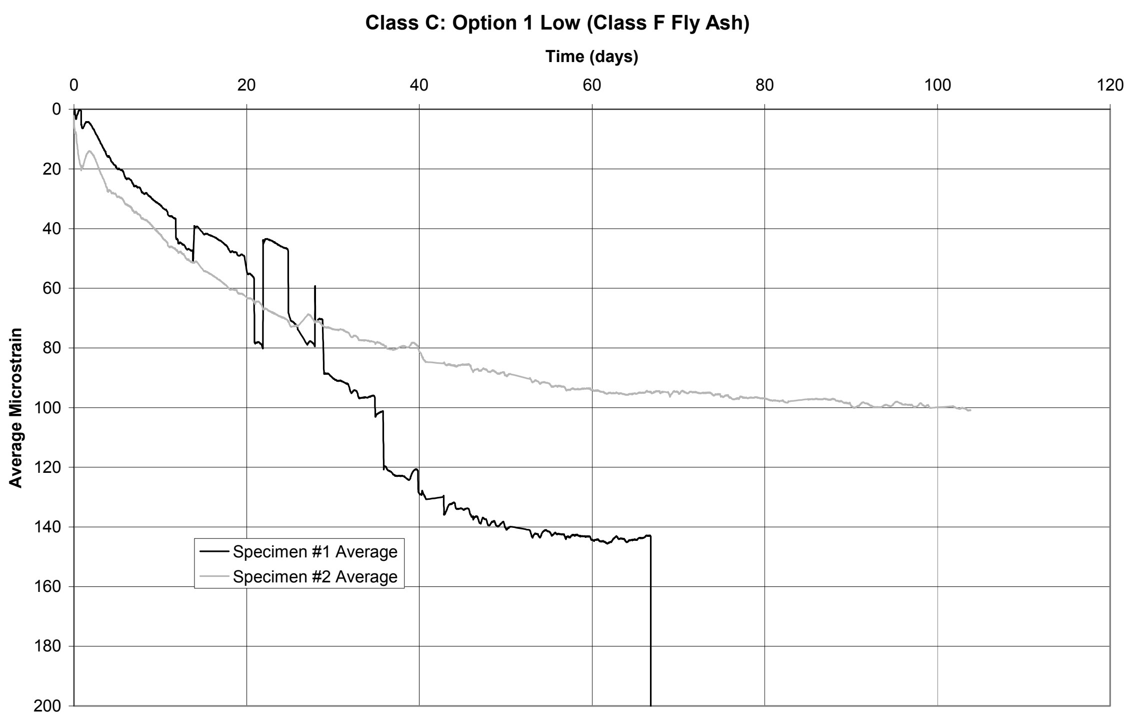

HP Blended Med. w/ LWA
Time (days)
0
20
40
60
80
100
120
0
20
40
60
80
100
120
140
160
180
200
Average Microstrain
No Cracking
Occurred
Specimen #1 Average
Specimen #2 Average


Mix Low Absorption Blend
Time (days)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
95
100
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
Microstrain
Specimen #1 Average
Specimen #2 Average

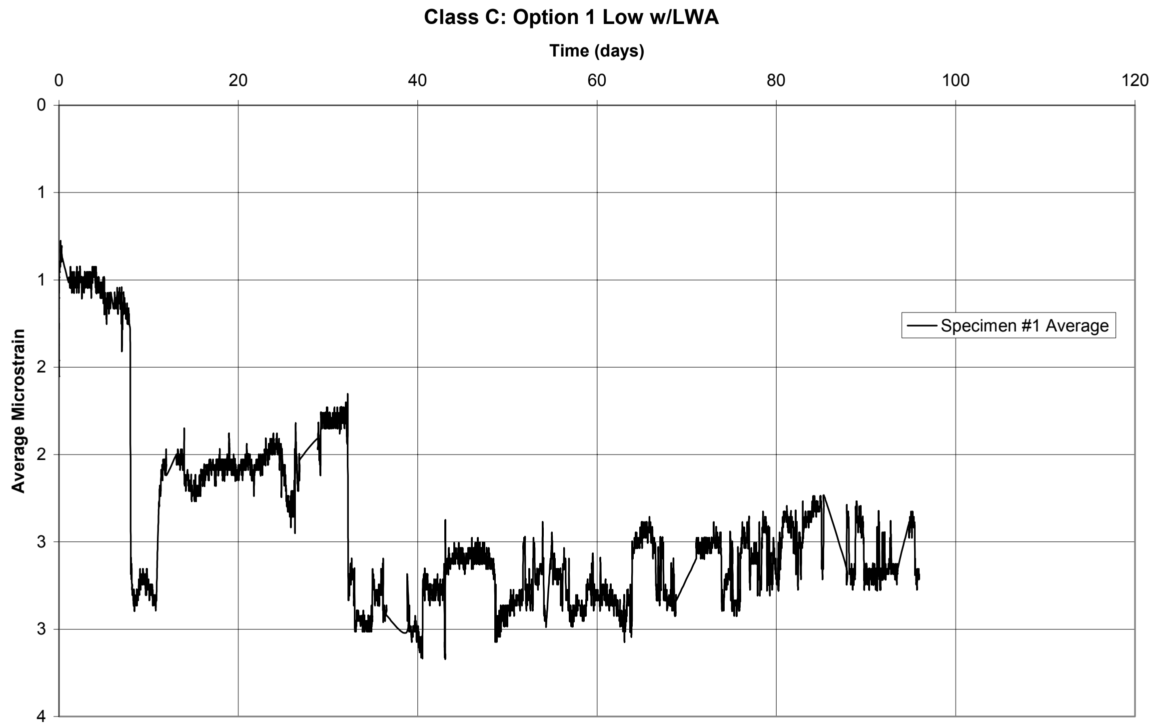

Mix Blend Low Absorption with LWA
Time (days)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
95
100
Microstrain
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
Specimen #1 Average
Specimen #2 Average

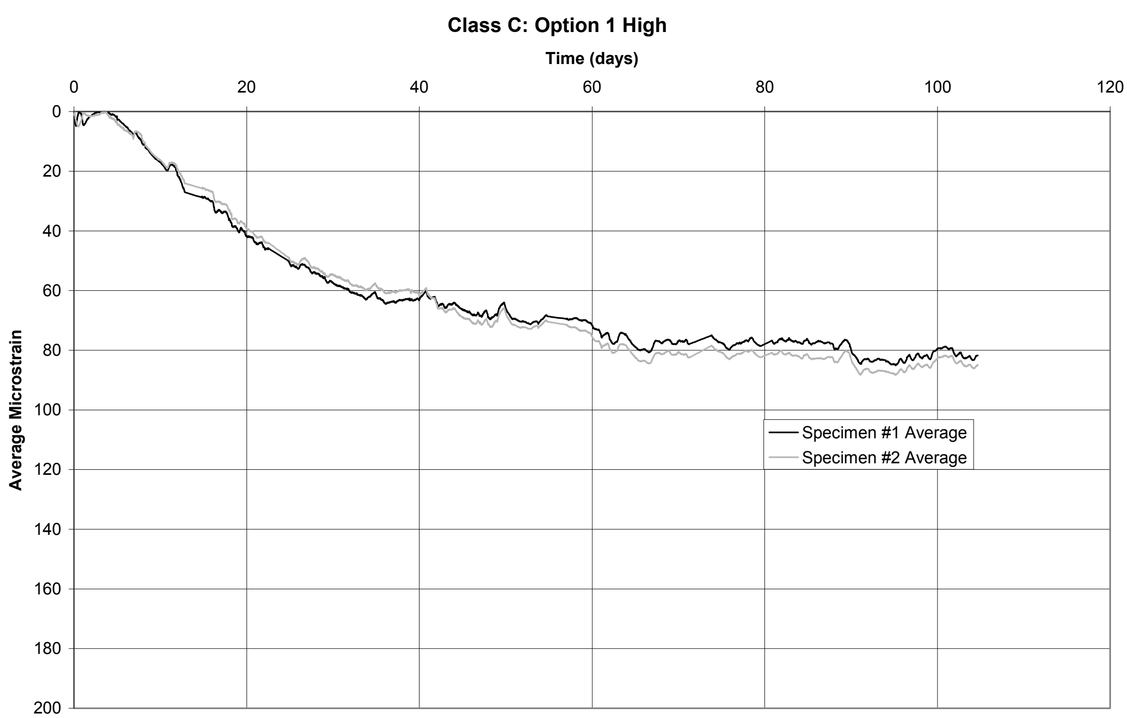

Mix Blend Medium Absorption
Time (days)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
95
100
Average Microstrain
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
Specimen #1 Average
Specimen #2 Average

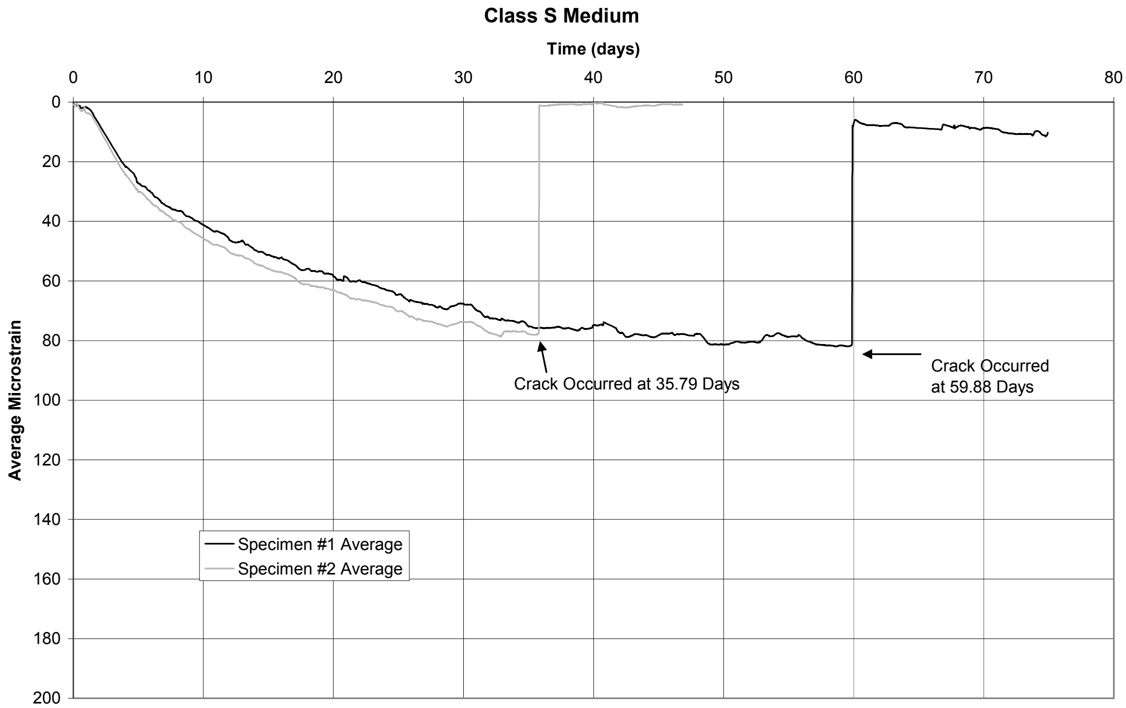
Class C: Option 1 Medium
Time (days)
0
10
20
30
40
50
60
70
80
90
100
0
2
4
6
8
10
12
Average Microstrain
Specimen #2 Average

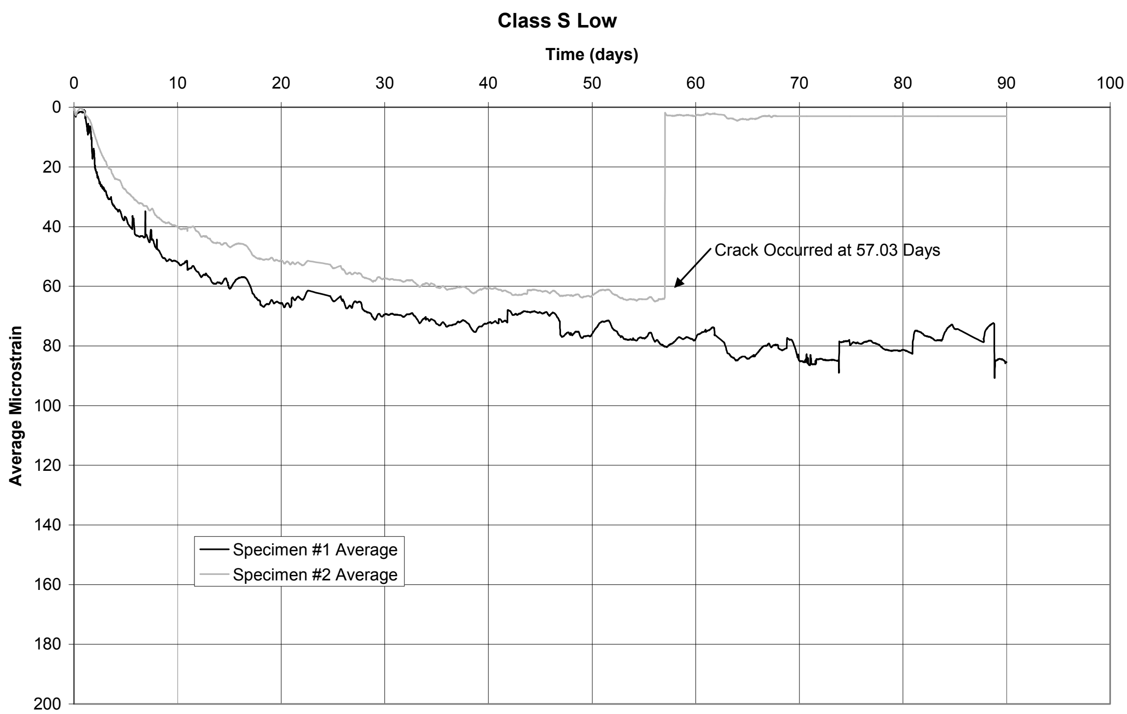


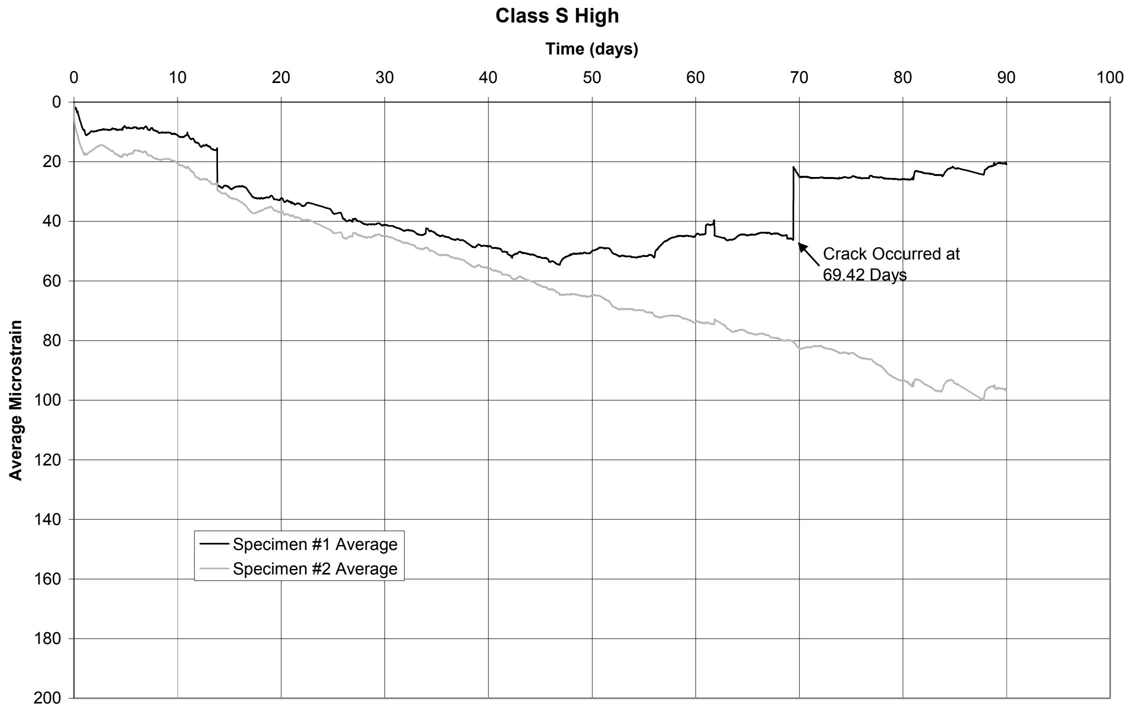
Class C: Option 1 Low (Class F Fly Ash)
Time (days)
0
20
40
60
80
100
120
0
20
40
60
80
100
120
140
160
180
200
Average Microstrain
Specimen #1 Average
Specimen #2 Average

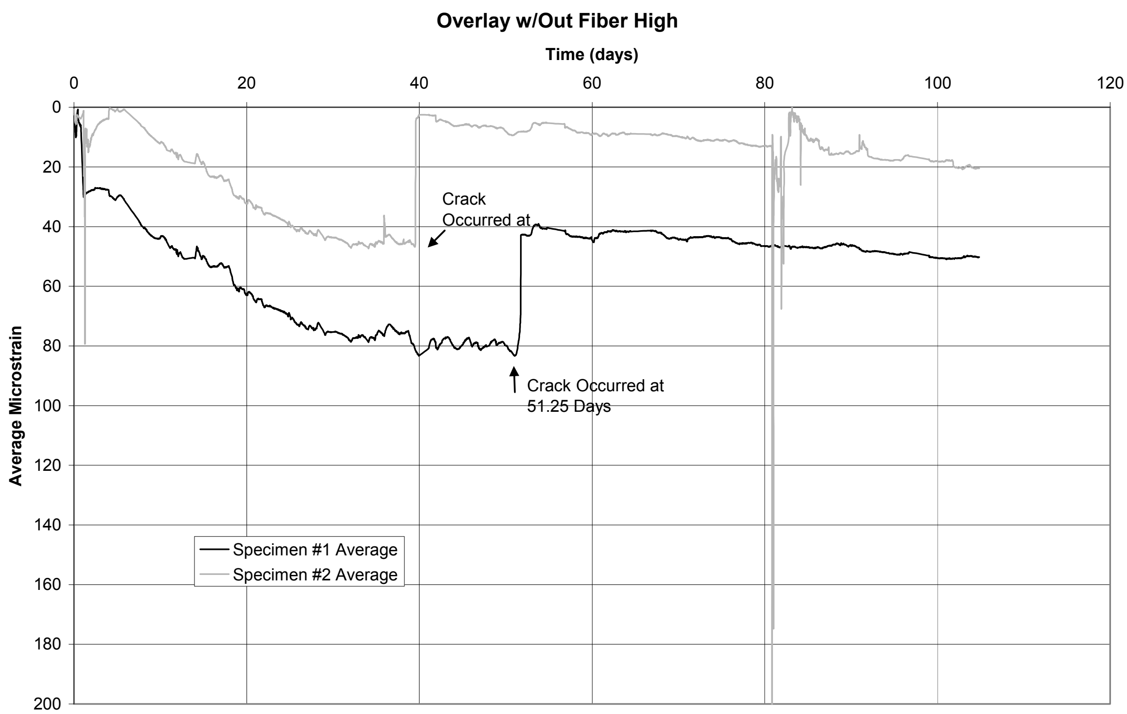

Class C: Option 1 Low w/LWA
Time (days)
0
20
40
60
80
100
120
0
1
1
2
2
3
3
4
Average Microstrain
Specimen #1 Average

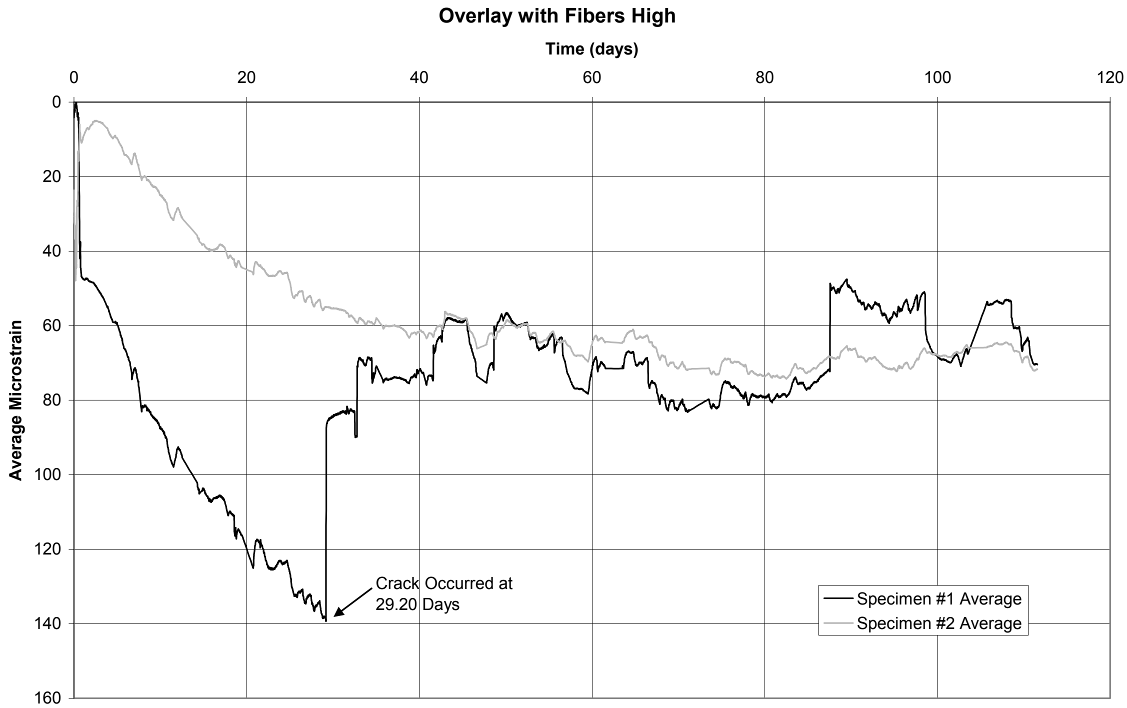

Class C: Option 1 High
Time (days)
0
20
40
60
80
100
120
0
20
40
60
80
100
120
140
160
180
200
Average Microstrain
Specimen #1 Average
Specimen #2 Average


Class S Medium
Time (days)
0
10
20
30
40
50
60
70
80
0
20
40
60
80
100
120
140
160
180
200
Average Microstrain
Crack Occurred at 35.79 Days
Crack Occurred
at 59.88 Days
Specimen #1 Average
Specimen #2 Average

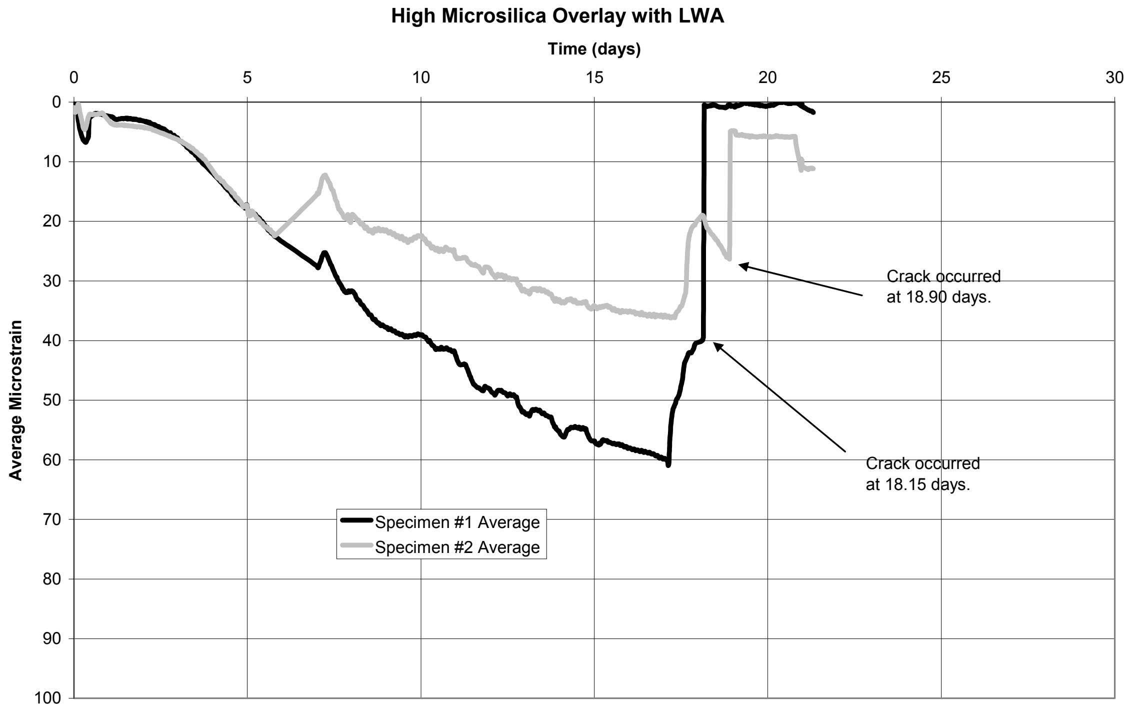

Class S Low
Time (days)
0
10
20
30
40
50
60
70
80
90
100
0
20
40
60
80
100
120
140
160
180
200
Average Microstrain
Crack Occurred at 57.03 Days
Specimen #1 Average
Specimen #2 Average

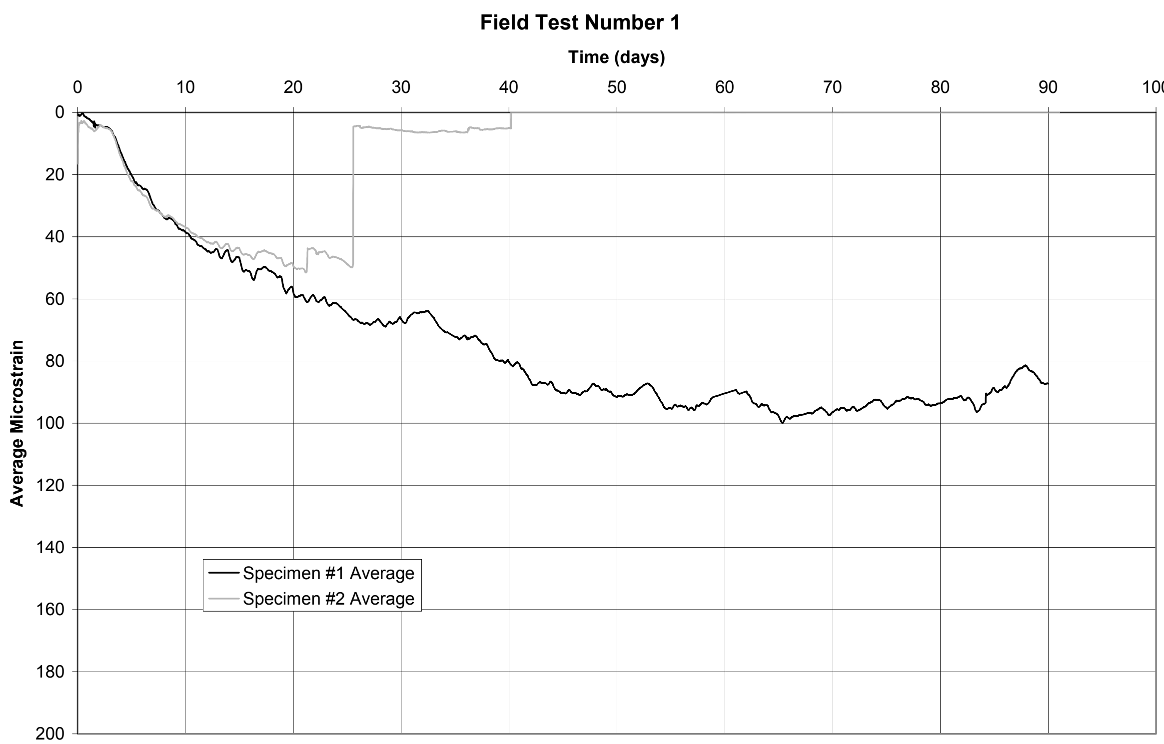

Class S High
Time (days)
0
10
20
30
40
50
60
70
80
90
100
0
20
40
60
80
100
120
140
160
180
200
Average Microstrain
Crack Occurred at
69.42 Days
Specimen #1 Average
Specimen #2 Average

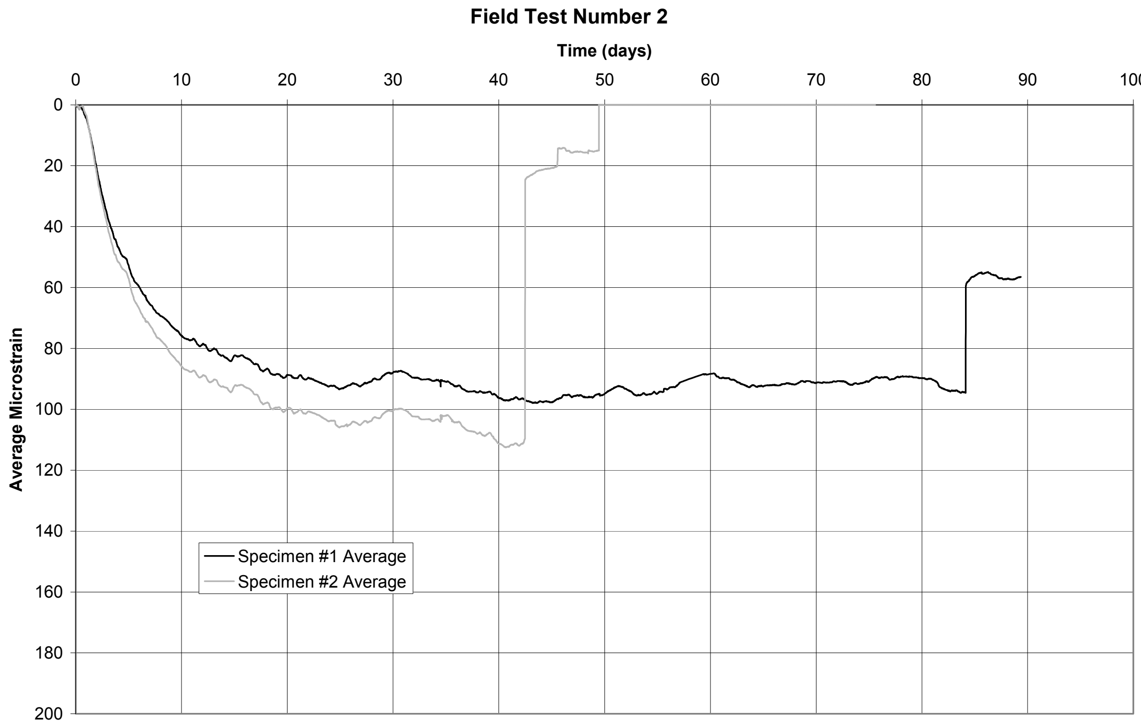

Overlay w/Out Fiber High
Time (days)
0
20
40
60
80
100
120
0
20
40
60
80
100
120
140
160
180
200
Average Microstrain
Crack
Occurred at
Crack Occurred at
51.25 Days
Specimen #1 Average
Specimen #2 Average

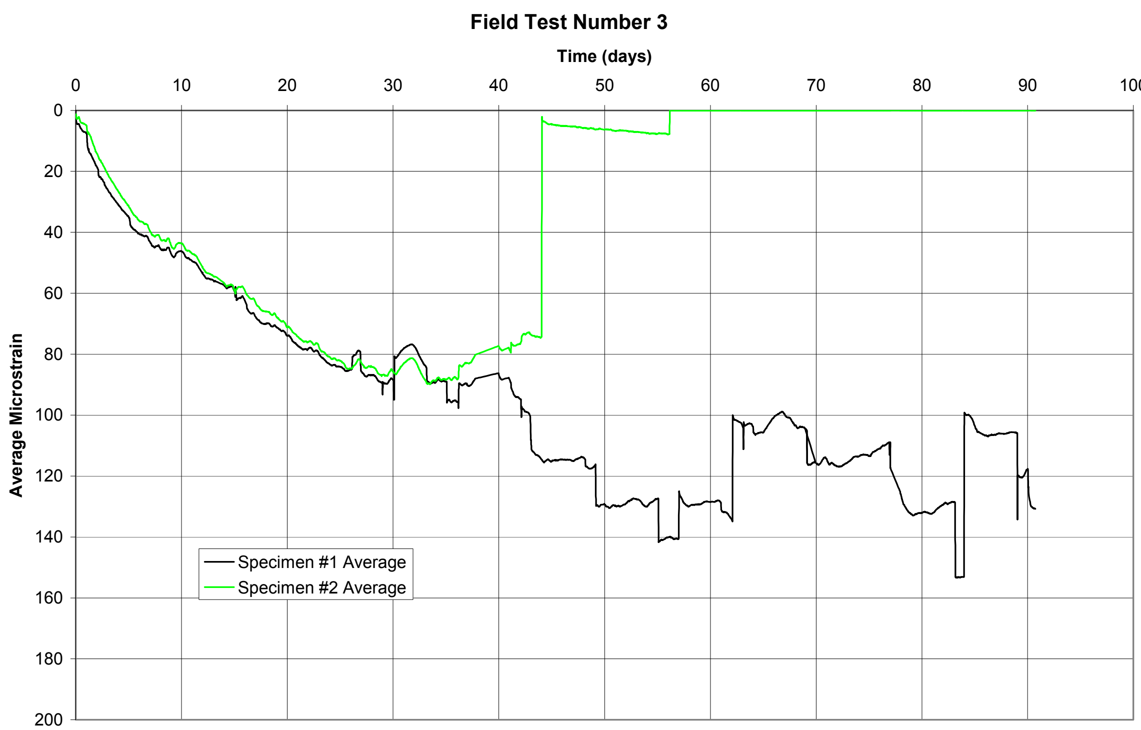


Overlay with Fibers High
Time (days)
0
20
40
60
80
100
120
Average Microstrain
0
20
40
60
80
100
120
140
160
Crack Occurred at
29.20 Days
Specimen #1 Average
Specimen #2 Average


High Microsilica Overlay with LWA
Time (days)
0
5
10
15
20
25
30
0
10
20
30
40
50
60
70
80
90
100
Average Microstrain
Crack occurred
at 18.90 days.
Crack occurred
at 18.15 days.
Specimen #1 Average
Specimen #2 Average


Field Test Number 1
Time (days)
0
10
20
30
40
50
60
70
80
90
100
0
20
40
60
80
100
120
140
160
180
200
Average Microstrain
Specimen #1 Average
Specimen #2 Average


Field Test Number 2
Time (days)
0
10
20
30
40
50
60
70
80
90
100
Average Microstrain
0
20
40
60
80
100
120
140
160
180
200
Specimen #1 Average
Specimen #2 Average


Field Test Number 3
Time (days)
0
10
20
30
40
50
60
70
80
90
100
Average Microstrain
0
20
40
60
80
100
120
140
160
180
200
Specimen #1 Average
Specimen #2 Average