EVALUATION OF CONCRETE MIX DESIGNS TO MITIGATE EARLY-AGE SHRINKAGE CRACKING IN BRIDGE DECKS

By

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EVALUATION OF CONCRETE MIX DESIGNS TO MITIGATE EARLY-AGE

SHRINKAGE CRACKING IN BRIDGE DECKS

Abstract

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Early-age shrinkage cracking has been observed in many concrete bridge decks in Washington State and elsewhere around the U.S. The cracking increases the effects of freeze-thaw damage, spalling, and corrosion of steel reinforcement, thus resulting in premature deterioration and structural deficiency of the bridges. In this study, the main causes of the early-age cracking in the decks are identified, and concrete mix designs as a strategy to prevent or minimize the shrinkage cracking are evaluated. Different sources (Eastern and Western Washington) and sizes of aggregates are considered, and the effects of paste content, cementitious materials (cement, fly ash, silica fume, slag), and shrinkage reducing admixture (SRA) are evaluated. A series of concrete shrinkage and mechanical property tests are performed. The outcomes of this study identify optimum concrete mix designs as appropriate mitigation strategies to reduce or eliminate early-age shrinkage cracking and thus help minimize shrinkage-associated cracking in the concrete bridge decks, potentially leading to a great deduction in bridge deck maintenance costs.

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Dedication

This thesis is dedicated to my family

who have always given me emotional support.

CHAPTER ONE

INTRODUCTION

1.1 Problem Statement

Early-age shrinkage cracking of concrete bridge decks is a common problem in the U.S. When the induced tensile stress is larger than the tensile strength of the concrete, cracking occurs (Fig. 1.1). According to a survey conducted by Krauss and Rogalla (1996), more than 100,000 bridges in the U.S. experienced early-age transverse cracking problem (Fig. 1.2).

The presence of early-age cracking in concrete bridge decks increases the effects of freeze-thaw damage, spalling due to sulfate and chloride penetration, and corrosion of steel reinforcement, thus resulting in premature deterioration and structural deficiency of the bridges. A recent investigation by the Washington State Department of Transportation (WSDOT) found transverse, full-depth cracks (Fig. 1.3) in the decks of all inspected bridges developed as a result of early-age concrete shrinkage (occurring within 48 hours after the deck concrete is poured). These cracks in the bridge decks provide an avenue for water, de-icing chemicals, sulfates, and other corrosive agents to penetrate into the concrete and substantially diminish the decks' service life. Concrete deck repair is expensive and can result in significant traffic delays. Accordingly, there is an urgent need to reduce the extent of this cracking and thereby prevent the premature deterioration. Although the concrete materials, concrete mix designs, design specifications and construction technologies have changed a lot over the years, shrinkage cracking still remains a significant problem and is prevalent in construction.

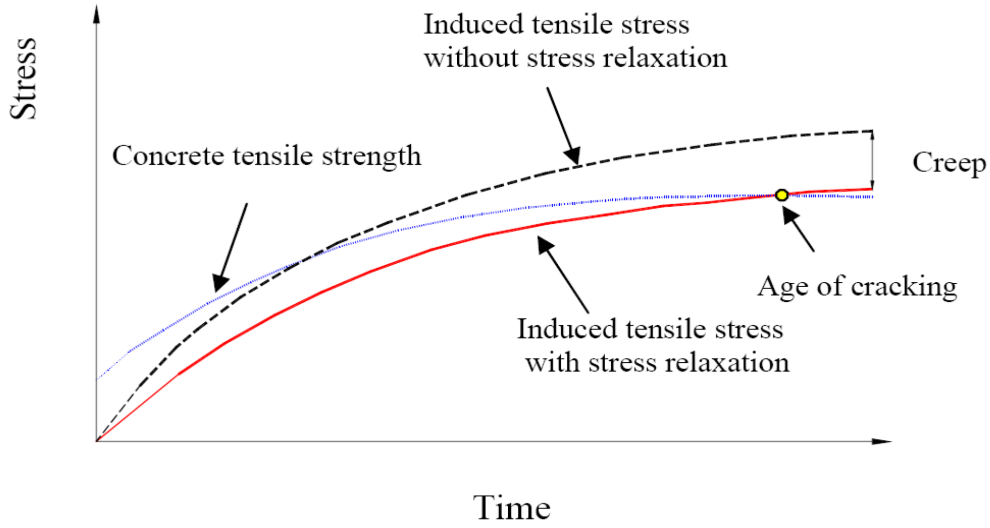


Fig. 1.1 Mechanism of Cracking (From Neville, 1996)

Fig. 1.2 Early-age Shrinkage Cracking in Concrete Bridge Decks (Crowl and Sutak 2002)

Fig. 1.3 Transverse, Full-depth Cracks Developed within 48-hour of Pouring

1.2 Objectives and Scope of Study

The objectives of the proposed study are five-fold: (1) to determine the primary causes of the transverse shrinkage cracking, (2) to identify appropriate mitigation strategies to reduce or eliminate early-age shrinkage cracking in the concrete bridge decks, (3) to evaluate current WSDOT concrete mix designs for their mechanical and shrinkage related properties, (4) to develop and evaluate new concrete mix designs using local materials from Washington for their mechanical and shrinkage related properties, and (5) to recommend the improved mix designs and practices to mitigate early-age shrinkage cracking.

1.3 Organizations

Seven chapters are included in this study. Chapter one introduces the problem statement and objectives of this study. Chapter two provides a literature review on the past research related to the early-age shrinkage cracking. Chapter three presents the method of developing new concrete mix designs and the finalized concrete mix designs used in this study. Chapter four introduces all the test methods that are adopted in this study. Chapter five summarizes the test results of concrete mix designs with Eastern Washington aggregates, while Chapter six is concerned with the test results of concrete mix designs with Western Washington aggregates. Chapter seven offers concluding remarks and recommendations.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

This literature review reviews past studies related to this study, identifies the causes of the early-age cracking in concrete bridge decks and develops recommendations for appropriate strategies to prevent or minimize this cracking.

Shrinkage cracking of bridge deck can be caused by many different reasons, such as material properties, restraint types, construction methods, environmental conditions, etc. Many researchers have performed laboratory studies and literature reviews on shrinkage and cracking potentials of concrete using different kinds of methods. Also, the American Association of State Highway and Transportation Officials (AASHTO) and the American Society for Testing and Materials (ASTM) provide test methods and specifications that can be used to analyze the behavior of concrete. In this chapter, the previous studies and test methods are reviewed, and recommendations for the experimental research of this study are provided.

2.2 Types of Shrinkage

Generally there are three different kinds of shrinkage for concrete: plastic shrinkage, autogenous shrinkage and drying shrinkage. Plastic shrinkage and autogenous shrinkage happen at an early age of the concrete, while drying shrinkage takes place over a long period of time.

2.2.1 Plastic Shrinkage

Plastic shrinkage is caused by a rapid loss of water on the concrete surface before the concrete hardens. This loss of water can be caused by many reasons, such as evaporation or suction by a dry sub-base. In the fresh concrete, concrete materials have not formed into a whole body and are still surrounded by water. When too much water rapidly evaporates, the water that remains in the concrete will not be sufficient, and voids occur within concrete, leading to the occurrence of plastic shrinkage cracking.

According to Schaels and Hoover (1988), environmental conditions, such as wind and temperature, have great influence on plastic shrinkage cracking of concrete. To reduce plastic shrinkage, the rate of water evaporation should be reduced. Therefore, when there is a high-speed wind, concrete casting should be avoided, or wind breaks and fogging should be used to prevent water loss. Because water evaporation only happens at the surface, plastic shrinkage cracking only occurs at the surface, and it is usually small.

2.2.2 Autogenous Shrinkage

Autogenous shrinkage happens when the concrete begins to hydrate. It is caused by the self-desiccation of concrete during the hydration process due to lack of water in concrete that has a low water-cement ratio. Autogenous shrinkage is also usually small. But for concrete using high-range-water-reducing admixture (HRWRA) and fine materials, such as silica fume, it may become an important factor leading to shrinkage cracking (Paillere et al. 1989).

To prevent autogenous shrinkage, low water-cement ratios are not preferred because there is not enough water for the cement to hydrate. When it is necessary to use low water-cement ratio, other methods should be used to compensate for the lack of water due to the low water-cement ratio in the concrete mix design.

2.2.3 Drying Shrinkage

Indicated by the pattern of early-age transverse cracking, drying shrinkage is present at bridge decking shrinkage cracking (Krauss and Rogalla, 1996). It is caused by loss of water in hardened concrete. Drying shrinkage can be explained by three main mechanisms: capillary stress, disjoining pressure and surface tension, each of which plays an important role within a certain range of relative humidity (Mindess et al. 2003). Normally bridge decks will experience relative humidity from 45% to 90%, which is when the capillary stress mechanism plays the important role.

Many factors can directly affect the drying shrinkage of concrete, such as paste volume, water-cement ratio, aggregates type, environment conditions and curing methods. Of all these factors, paste volume is the most important one. Drying shrinkage will be greatly reduced if the paste volume is reduced (Xi et al. 2003; Tritsh et al. 2005; Darwin et al. 2007; Delatte et al. 2007).

2.2.4 Creep

While early-age cracking in bridge deck is mainly due to concrete shrinkage, creep helps to relax shrinkage. The study by Altoubat et al. (2001) found that the tensile creep relaxes the shrinkage stress by 50% and doubles the failure strain capacity. It is generally believed that creep will help reduce shrinkage of concrete, as the mechanism shown in Fig. 1.1.

2.3 Effect of Concrete Properties on Deck Cracking

2.3.1 Paste Content and Water-to-cement Ratio

As aforementioned, paste content is a very important factor that affects the shrinkage behavior of bridge decks, since it leads to volume changes. Reducing paste content results in a decrease in free shrinkage (Bissonnette et al. 1999; Darwin et al. 2007). Water content plays two roles. Increasing water content increases the shrinkage tendency of concrete, and at the same time increases creep. Creep can help reduce shrinkage.

Decreasing the water-to-cement ratio can decrease drying shrinkage but at the same time it increases autogenous shrinkage. Bissonnette et al. (1999) and Darwin et al. (2007) stated that free shrinkage is not significantly influenced by the water-to-cement ratio. However, Weiss et al. (1999) concluded that the concrete with a low water-to-cement ratio may be more likely to develop early-age cracking due to increased autogenous shrinkage. There is no definitive conclusion of the effect of water-cement ratio. It is generally believed that a very high water-to-cement ratio will cause more shrinkage.

As a result, the cement content and the water-to-cement ratio are limited to reduce the risk of shrinkage cracking. Literature indicates that a minimum cement content should be suggested to reduce cracking (Brown et al. 2001). The experimental study by Xi et al. (2003) showed that a concrete mix with a cement content of about 470 lb/yd^3 and water-to-cement ratio of about 0.4 could be a possible optimum mix.

2.3.2 Cement Type

Cement type also plays an important role in shrinkage cracking of bridge decks, as the drying shrinkage of concrete is affected by the cement fineness. Finer cement particles generate greater heat of hydration and require a greater amount of water during the hydration process, which may lead to the increased risk of cracking in the concrete. As a result, Type II Portland cement is preferred to reduce cracking. Replacing Type I/II Portland cement with Type II Portland coarse-ground cement lowers the free shrinkage and shrinkage rate, and adding a shrinkage-reducing admixture significantly reduces these values (Tritsch et al. 2005).

2.3.3 Aggregates Size and Type

The properties of concrete depend on cement paste and aggregates. In contrast to the cement paste, aggregates do not shrink or creep. When cement paste shrinks, aggregates provide restraint. Krauss and Rogalla (1996) found that aggregate type is the most significant factor affecting concrete cracking. It is generally believed that larger aggregates decrease the cracking tendency of bridge decks. Large aggregates can form a rigid frame in the concrete, which prevent cement paste from shrinking freely. However, as bridge decks are becoming thinner, the optimized aggregate size to both resist shrinkage cracking and satisfy workability requirements should be studied. The properties of aggregates determine the amount of restraints that will be applied to cement paste.

Burrows (1998) found that limestone aggregate has higher resistance to cracking than other types of aggregates. It has the best restraint when the aggregate does not shrink at all. Also, the ratio of elastic moduli of aggregate and cement is important on the shrinkage of concrete. If the ratio $E_{aggregate} / E_{cement}$ is higher, then the concrete has lower shrinkage potential (Troxell et al. 1958).

2.3.4 Air Content

Past literature shows no definite conclusion about the effect of air content on the shrinkage cracking of bridge decks (Xi et al. 2003). Schmitt and Darwin (1995) suggested that an air content of 6% by volume or more should be considered.

2.3.5 Slump

Slump is used as an indicator of concrete workability. If there is an excessive slump caused by high water-to-cement ratio, the concrete will have a high shrinkage. Krauss and Rogalla (1996) found that concrete mixes with a low water-to-cement ratio, low cement content, and low slump performed best. Generally, the slump of concrete is controlled within a reasonable range, and there is no definite relation between the change of slump and the change of cracking tendency of concrete.

2.4 Cementitious Materials and Admixtures in Concrete

2.4.1 Silica Fume

Silica fume is a pozzolanic material, and its particle size is about 1.0 μm. The use of silica fume in concrete can achieve a lower permeability, which is good for the durability issues of bridge decks. However, it has a high hydration heat so that it has a higher tendency of plastic shrinkage cracking. Autogenous shrinkage may be aggravated by the use of silica fume as well (Mindess et al. 2003).

NCHRP Report 410, “Silica Fume Concrete for Bridge Decks,” concluded that cracking tendency of concrete was influenced by the addition of silica fume only when the concrete was improperly cured. When concrete is cured for 7 days under continuously moist conditions, there is no statistically significant effect of silica fume on the tendency of the concrete to exhibit early-age cracking. Darwin et al. (2007) stated that when cast with a high-absorption coarse aggregate, the addition of silica fume results in a reduction in shrinkage at all ages. Mazloom et al. (2004) studied the replacement of cement with 0%, 6%, 10%, and 15% of silica fume and concluded that the percentage of silica fume replacement did not have a significant influence on the total shrinkage of concrete, but the autogenous shrinkage increased as the increase of silica fume. Krauss and Rogalla (1996) contended that the effect on early-age shrinkage cracking of silica fume is still not clear. Thus, the moderate content of silica fume in a range of 6-8% by mass of cementitious materials in concrete was recommended. When it is used, fog sprays or keeping moist after the placement of concrete is suggested for 7 days continuously (Schmitt and Darwin 1995).

2.4.2 Fly Ash

Fly ash is also a pozzolanic material. It is used to replace part of the Portland cement in the concrete mixture so that the rate of concrete hydration will slow down. Thus, the rate of early-age strength gain is also reduced, which leads to less cracking tendency. Fly ash also improves the workability of concrete, such as enhancing the ultimate strength of concrete and reducing the permeability of concrete. However, Darwin et al. (2007) stated that when cast with a high-absorption coarse aggregate, the addition of fly ash increases initial shrinkage and only slightly reduced ultimate shrinkage.

The percentage replacement of fly ash for Portland cement should be concerned during the application of fly ash as different amount of fly ash in a concrete mix affects the properties of the concrete. Fly ash is now commonly used as one additive in concrete mixtures as many state DOTs use it in their concrete mix design.

Generally, there are two types of fly ash, Class F and Class C. Class F fly ash possesses pozzolanic properties but does not have self-cementing properties. Class C fly ash has both pozzolanic and self-cementing properties. Based on the specific cement, the percentage replacement of portland cement should be determined accordingly (Xi et al. 2003).

2.4.3 Ground Granulated Blast-Furnace Slag (GGBFS)

Ground granulated blast-furnace slag (GGBFS) is added to Portland cement to increase the concrete strength and durability. The use of GGBFS can improve the strength as well as the durability of concrete. NCHRP Report 566 "Guidelines for Concrete Mixtures Containing Supplementary Cementitious Materials to Enhance Durability of Bridge Decks," recommended that the addition of fly ash or GGBFS to the concrete does not generally affect the cracking tendency of the concrete greatly if the total cementitious volume is not changed. Cracking (drying shrinkage) might be reduced if the improved workability of the mixture containing the fly ash or GGBFS contributes to reduced water demand and reduced paste volume (Lawler et al. 2007).

2.4.4 Shrinkage-Reducing Admixtures

As discussed before, bridge decks will normally experience relative humidity from 45% to 90%, which is when the capillary stress mechanism plays the important role. Shrinkage-reducing admixtures (SRA) can lower the surface tension of pore water, thus reducing drying shrinkage.

Many researchers have found that the use of SRA in concrete reduced the shrinkage and cracking tendency (Shah et al. 1992; Brown et al. 2001; Tritsch et al. 2005; Brown et al. 2007). Weiss et al. (2002; 2003) stated that SRA significantly enhanced the cracking resistance of concrete by reducing the rate of shrinkage and the overall magnitude of shrinkage. SRA reduced the surface energy of the water so there is less tension to make the concrete shrink. However, research (Folliard and Berke 1997; Weiss et al. 2003) also found that SRA may cause a slight decrease in the compressive strength of concrete.

2.4.5 Fiber Admixture

When fiber is added to concrete, the property of concrete changes in relation with the amount of fiber added. Steel fiber can improve the strength of concrete. Shah and Weiss (2006) stated that the inclusion of randomly distributed steel fibers can slightly delay the age of visible cracking. Because fibers act as restraint inside the concrete, they can reduce the amount of cracking (Sun et al. 2000; Banthia 2000). The fibers only play a role when cracking is developed, and they are thus useful for post-cracking control.

2.5 Other Factors Related to Shrinkage Cracking

2.5.1 Restraint Type

After concrete hardens, the concrete deck endures restraint from both inside and outside the concrete. The outside supporting girders apply strong restraint to the concrete bridge deck, which constrains the shrinkage deformation of the deck. At the same time, the internal reinforcement of the concrete deck also constrain the shrinkage of the concrete. Therefore, the concrete deck experiences high stress, which may lead to its cracking. French et al. (1999) found that bridge decks on simple-supported prestressed girders showed significantly less cracking than decks on continuous steel girders in their field study. Krauss and Rogalla (1996) found that decks supported by steel girders usually have higher risks of transverse deck cracking and higher tensile stresses than the ones with concrete girder construction. Rogalla et al. (1995) found that larger girder and closer spacing tend to be more prone to cracking. So using smaller girder and wider spacing will reduce the cracking tendency.

2.5.2 Construction Method

Construction method may have a very large influence on the early-age shrinkage cracking of concrete bridge decks. It is suggested that placing positive moment regions successively on one day and then after three days placing negative moment regions may minimize cracking (Issa 1999).

Finishing is also a factor that affects early-age bridge deck shrinkage cracking. The literature stated that a delayed finishing can cause concrete to crack more easily (Krauss and Rogalla 1996).

Curing is also an important factor that influences early-age bridge deck shrinkage cracking. Immediately after finishing, use of wet curing should be applied (Babaei and Purvis 1996).

2.5.3 Environmental Conditions

Concrete should be placed during cool weather to reduce cracking, because the hydration reaction will slow down in low temperature, thus reducing the heat that is generated from the hydration process. So the thermal stress is controlled to be a small amount, which will help to reduce early-age thermal cracking. Other times that will increase the temperature in concrete during the hydration process should also be avoided, such as the time around noon. The study by French et al. (1999) recommended that the ambient air temperature ranged between highs of approximately 18 to 21^oC (65 to 70^oF) and lows of approximately 7 to 10^oC (45 to 50^oF).

When the wind is strong, windbreaks should be used to keep the concrete moist and prevent high evaporation of concrete surface water. Windbreaks or fogging should be used if the wind speed is more than 0.2 lb/ft^2/hr.

2.6 Test Methods

2.6.1 General Review on Test Methods of Concrete Shrinkage Cracking

Many researchers have developed different methods of evaluating the shrinkage cracking

tendency of concrete using a wide range of test apparatus. Tritsch et al. (2005) divided these restrained shrinkage tests into three categories: plate tests, linear tests, and ring tests.

In the plate tests, flat concrete specimens were tested. Different researchers used different specimen dimensions and different test details. But usually those specimens are thin and the maximum aggregate sizes are small or no coarse aggregates are used. In some tests the results were inconsistent and conflicted with each other. Free shrinkage tests are also considered as an addition to these restrained tests.

The linear test used specimens of rectangular cross section. Specimens of many different dimensions were used in these tests, such as 8.5 x 12 x 150 cm (3.4 x 4.7 x 59 in.)

(Paillère et al. 1989), and 40 x 40 x 1,000 cm (1.6 x 1.6 x 39.4 in.) (Bloom and Bentur 1995). In these linear tests, one end of the concrete specimen is fixed, and the other end is connected to an instrument that applies and records the force that is required to keep the specimen in its original length. A companion specimen with the same dimension is also cast, with one end fixed and the other free to shrink, as a control specimen to the restrained one.

The ring test was used by many researchers to evaluate the shrinkage cracking tendency and behavior of concrete and cement-based materials under restraint. It is the most common test method used. Many different concrete rings were tested under a variation of restrained conditions. The dimensions of the concrete ring as well as the test procedure vary greatly from each other. More details on the ring tests are presented in Section 1.6.3.

2.6.2 Cracking Frame and Fracture Energy

Réunion Internationale des Laboratoires d'Essais et de recherche sur les Matériaux et les Constructions (RILEM) uses the cracking frame method as a standard test TC 119 for cracking evaluation. The cracking frame as shown in Fig. 2.1 was developed by Springenschmid (1994) after an extensive research on the test methods for restrained shrinkage of concrete was conducted.

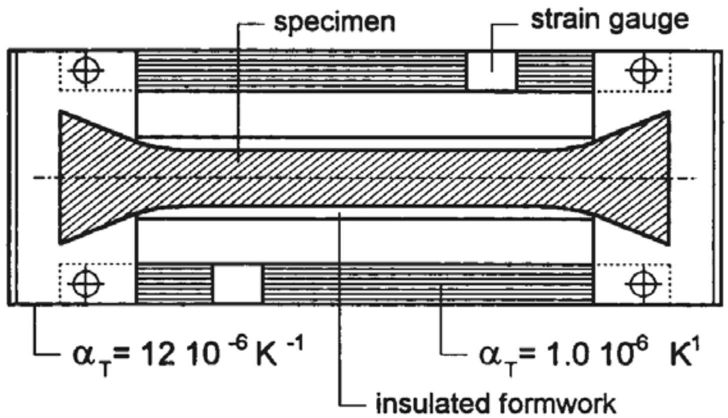


Fig. 2.1 Cracking Frame (Springenschmid et al. 1994)

The cracking frame can be used for the contraction test as well as the expansion test of concrete, and the restraint stresses are recorded continuously. Comparing with the ring test, the cracking frame can represent the actual restraint conditions of the concrete bridge decks caused by the restraint from girders. As shown in Fig. 2.1, the test is made up of a concrete beam and two surrounding steel bars in the longitudinal direction and also two steel cross-heads at each end. In the cracking frame, the concrete can be cooled to the surrounding temperature. It is first inspected for four days. If it does not crack in four days, its temperature is decreased at a fixed rate until cracking occurs. The temperature that cracking occurs is recorded as an indication of the cracking resistance property of the concrete mix in actual service conditions. The lower this temperature can be, the better the cracking resistance.

Fracture energy of concrete can be used to evaluate the drying shrinkage cracking property of concrete. Guo and Gilbert (2000) showed that the fracture energy can represent the actual amount of energy that is needed for a crack to occur upon unit area or fracture surface. In this test, a three-point bending test is performed upon a notched beam, and the displacement of the beam and corresponding applied load are recorded. By using the recorded load-displacement curve and some data reduction equations, the fracture energy of the beam can be calculated, from which the relation between the fracture energy and the cracking resistance behavior of the beam can be established.

2.6.3 Ring Test Method

As aforementioned, the ring test method is used to evaluate the relative drying shrinkage cracking tendency of different concrete mixes under different conditions. The ring test restrains the concrete using a steel ring, thus inducing a stress on the surrounding concrete ring. When this stress becomes larger than the tensile strength of the concrete ring, the concrete ring will crack. The times that it takes for rings made of different concrete mixes to crack are recorded and then compared with each other. The longer it takes a concrete ring specimen to crack, the lower tendency of drying shrinkage cracking it has.

The ring test is simple and easy to conduct. Also, it evaluates most of the important factors that affect the drying shrinkage cracking tendency at one time. Furthermore, the cracking in the concrete ring is easily recognized and recorded. Therefore, the ring test method has become the most popular method for evaluating the restrained drying shrinkage of concrete.

Both the American Association of State Highway and Transportation Officials (AASHTO) and the American Society for Testing and Materials (ASTM) have a developed ring test as one of their standard tests, and they are:

- AASHTO PP34-99. “Practice for Estimating the Crack Tendency of Concrete”.

- ASTM C 1581-04. "Standard Test Method for Determining Age at Cracking and Induced Tensile Stress Characteristics of Mortar and Concrete under Restrained Shrinkage".

2.6.3.1 AASHTO Ring Test

The AASHTO ring test is used to compare the relative restrained shrinkage cracking tendency of different concrete mix designs. It can be used to compare factors such as cement paste content and water-to-cement ratio, cement type, aggregates size and type, air content, slump and admixtures in concrete as related to the time and cracking relation of concrete. However, it does not take the specific restraint type, the construction method and environmental conditions into consideration, so it cannot predict the concrete cracking in actual service. The standard utilizes the following apparatus as shown in Fig. 2.2.

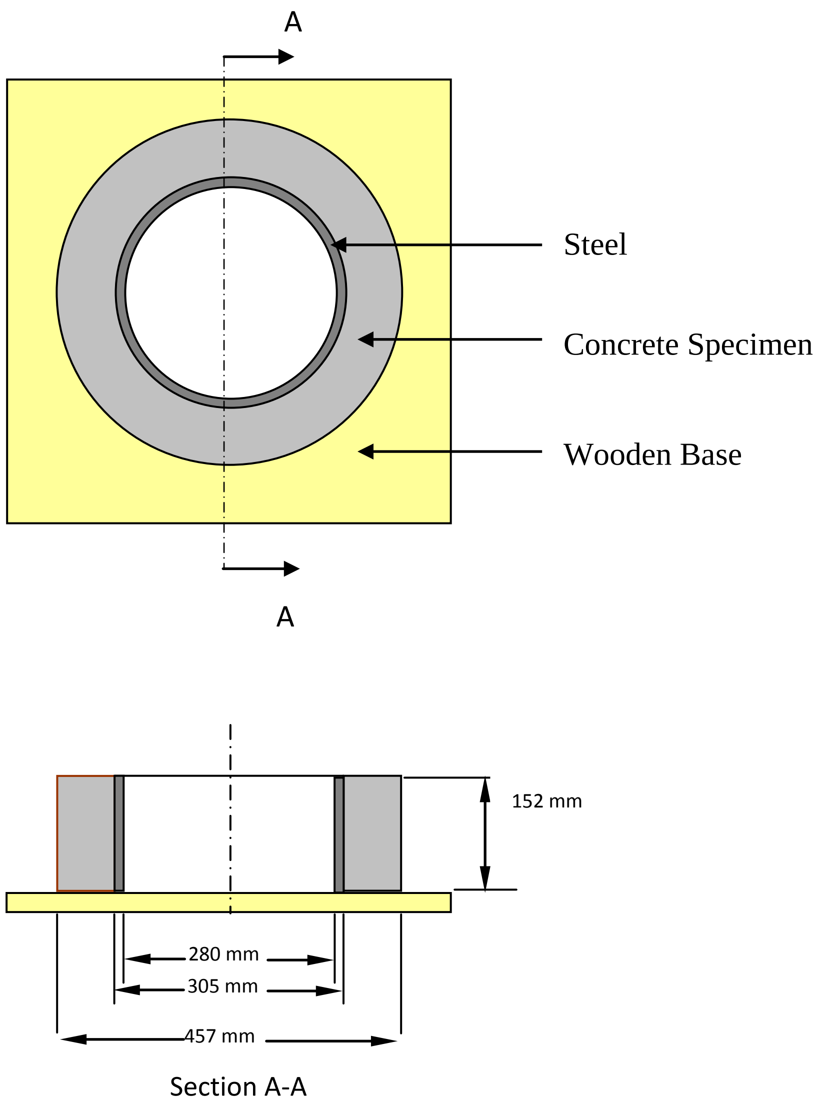


Fig. 2.2 Diagrams of Ring Specimen (Reprinted from AASHTO PP34-99)

The standard inside steel ring has a wall thickness of 12.7 ± 0.4 mm (1/2 ± 1/64 in.), an outside diameter of 305 mm (12 in.), and a height of 152 mm (6 in.). However, structural steel pipe conforming to ASTM A 501 or A 53M/A 53 12-in. extra-strong pipe with an outside diameter of 324 mm (12 ¾ in.) and wall thickness 13 mm (1/2 in.) may be substituted. The outer ring can be made of 6.4 mm thick (1/4 in.) cardboard form tube (sonotube) with an inside diameter of 457 mm (18 in.). Four strain gages are mounted on the inner surface of the steel ring at equidistant points at midheight. Data acquisition equipment shall be compatible with the strain instrumentation and automatically record each strain gage independently. Forms can be made of 24 in. by 24 in., 5/8 in. thick (0.6 x 0.6 x 0.016 m) plywood sheet; or resin-coated or polyethylene-coated plywood. Curing can be applied by using prewetted burlap covered with plastic.

The outer forms are removed at an age of 24±1 hr, and then the specimens are moved to the condition room with a constant air temperature of 73.5 ± 3.5 ^{o}F (23 ± 2 ^{o}C) and 50 ± 5 % relative humidity. The time and strain from the strain gages are recorded every 30 minutes, and review of the strain and visual inspection of cracking is conducted every 2 or 3 days. A sudden strain decrease of more than 30 micro strain in one or more strain gages usually indicates cracking. After the concrete ring cracks, record the time and the cracking length and width on the exterior radial face.

2.6.3.2 ASTM Ring Test

Similarly, the ASTM ring test is also used to evaluate the relative drying shrinkage cracking tendency of concrete under restraint. It can also be used to compare factors such as cement paste content and water-cement ratio, cement type, aggregates size and type, air content, slump and admixtures in concrete as related to the time and cracking relation of concrete. As with the AASHTO ring test, the ASTM ring test does not take the specific restraint type, construction method and environmental conditions into consideration, and therefore, it cannot predict the concrete cracking in actual service.

The standard inside steel ring has a wall thickness of 0.50 ± 0.05 in. (13± 0.12mm), an outer diameter of 13.0 ± 0.12 in. (330 ± 3.3mm) and a height of 6.0 ± 0.25 in. (152 ± 6mm). At least two electrical resistance strain gages are wired in a quarter-bridge configuration.

Data acquisition system should be compatible with strain instrumentation and automatically record each strain gage independently with resolution ± 0.0000005 in./in. at intervals no greater than 30 minutes. The base can be made of epoxy coated plywood or other non-absorptive and non-reactive surface. The outer ring can be made of PVC pipe or Steel outer ring or other, in accordance with F441, with 16.0 ± 0.12 in. (406 ± 3 mm) inside diameter and 6.0 ± 0.25 in. (152 ± 6 mm) height. The testing environment has the condition of 73.5 ± 3.5 ^oF (23.0 ± 2.0 ^oC) and 50 ± 4% relatively humidity. The dates and strain from the strain gages must be recorded at least every 30 minutes. Record ambient temperature and relatively humidity every day. A sudden decrease of more than 30 microstrain in compressive strain in one or both strain gages indicate cracking. After the concrete ring cracks, record the time and the cracking length and width on the exterior radial face. Monitor the specimen for two additional weeks after cracking.

2.6.3.3 Comparison between the AASHTO and ASTM Ring Tests

In general, both the AASHTO Ring Test and the ASTM Ring Test use the same theory and procedures. However, there are some differences between the two methods. The main differences between them are the concrete ring dimensions and the maximum size of aggregates allowed. The AASHTO standard concrete ring is 3 in. thick, with inner diameter of 12 in. and outer diameter of 18 in., whereas the ASTM concrete ring is 1.5 thick, with inner diameter of 13 in. and outer diameter of 16 in. The ASTM requires that the maximum size of aggregate should be less than 1/2 in., while there is no specific requirement in the AASHTO. Because the concrete ring is thicker in AASHTO than in ASTM, AASHTO allows greater aggregate size. Also, the duration of the ASTM test is 28 days; while there is no specified duration in AASHTO. Because the AASHTO concrete ring is thicker, it will need more time to crack. So typically the AASHTO ring test may last for 56 days to 90 days (Delatte et al. 2007). The curing conditions are also slightly different between the two test methods.

2.6.3.4 Effect of Geometry of the Ring Test

As mentioned previously, ring tests of many different dimensions have been conducted in the past, and the results are not the same. The dimensions play an important role in determining the properties of concrete mixes in the ring test. A finite element analysis was performed by Krauss and Rogalla (1996) on the ring test. Their analysis showed that when the inner steel rings have the thicknesses between 13 mm (1/2 in.) and 25 mm (1 in.), the stress and drying shrinkage tendency of concrete are not very different. A thinner inner steel ring induces larger steel stress, and a thicker inner steel ring induces larger concrete stress. Also, the concrete shrinkage stress reduces when the height of the concrete ring increases from 76 mm (3 in.) to 152 mm (6 in.). Thus, a thicker and shallower steel ring induces high stress in concrete as expected.

Delatte et al. (2007) compared the ring geometry using two sets of specimens. Both sets used two 406 mm (16 in.) and two 457 mm (18 in.) outer diameter concrete rings cast around inner steel ring of 12 in. diameter at the same time from the same mixture. From their study, they developed an equation for time-to-crack versus ratio of ring radii:

$$\frac{R_{\mathrm{o}}}{R_{\mathrm{i}}} = -0.0025t^2 + 0.13t + 0.3188 \qquad (1.1)$$

where

R_o is the outside radius of concrete ring;

R_i is the inside radius of the concrete ring; and

t is the time to crack.

2.6.4 Summary of Test Methods

As reviewed in this section, several test methods have been developed for measuring the drying shrinkage cracking tendency of specimens consisting of different concrete mixes or other different conditions. Among these methods, the ring test method is simple and easy to conduct, and it can be used to compare most of the factors that affect the cracking tendency of concrete at the same time. Also, it is easier for the concrete to develop visual cracks. Because of these merits, the ring test method was adopted by many researchers. However, it should be noticed that the ring test method only reflects the relative cracking tendency of concrete with different mixes and different conditions, and it cannot represent the concrete in actual service life.

The ring test method will be adopted in this study. The AASHTO PP34-99 will be considered using structural pipe with an outside diameter of 324 mm (12.75 in.). The AASHTO ring test for this study produces concrete ring thickness of 66.5 mm (2.625 in.). The ASTM ring test produces concrete rings of 38 mm (1.5 in.), which limited the maximum size of aggregate to be 13 mm (1/2 in.). In this study, we will include aggregates with maximum nominal size of 1.5 in. or larger.

2.7 Other Related Work

Folliard and Berke (1997) evaluated the effect of shrinkage-reducing admixture (SRA) on high-performance concrete properties. The mechanical properties, free shrinkage and restrained shrinkage cracking were investigated. For the restrained ring test, a concrete ring with 50 mm (2 in.) thickness and 150 mm (6 in.) height was cast around a steel pipe with inner diameter of 250 mm (10 in.) and outer diameter of 300 mm (12 in.). Then, the specimens were put into drying condition of $20^{o}C$ and 50% RH. Free shrinkage concrete prisms were 75 x 75 x 285 mm (3 x 3 x 11.2 in.). Their study concluded that the use of SRA greatly reduced drying shrinkage cracking in laboratory ring specimens, despite concrete containing SRA exhibited lower early strengths than companion mixtures without SRA.

Xi et al. (2001) studied the development of optimal concrete mix design for bridge decks. Four different tests (i.e., compressive strength test, rapid chloride permeability test, restrained ring test, free shrinkage test) were performed to evaluate the properties of concrete. The AASHTO ring test was adopted with modification. Two concrete rings of 6 in. height with 12 in. and 18 in. inner and outer diameters were cast for each concrete mix. After one day of curing, the specimens were put in the lab with temperature of 72 ^{o}F and relative humidity of 35%. Two concrete beams of 3 x 3 x 12 in. were made for the free shrinkage test for drying shrinkage test. Their study included two phases. 18 mix designs were formulated in Phase I to get some good mixes that satisfied requirements. Phase II was to finalize the good mix designs from Phase I to be used in the field. It was found that cracking was related to the cement content. A proper increase of coarse aggregate could reduce cracking potentially; Class F fly ash had better cracking resistance than Class C fly ash.

Tritsch et al. (2005) evaluated the shrinkage and cracking behavior of concrete using the restrained ring and free shrinkage tests. Their study was made of a series of preliminary tests and three test programs. The steel ring had a thickness of 13 mm (1/2 in.) with an outside diameter of 304 mm (12 in.). The concrete ring specimens were 76 mm (3 in.) or 51 mm (2 in.) thick. Both the steel and concrete rings were 76 mm (3 in.) tall. In each program, the concrete was exposed to drying condition of about $21^{o}C$ ($70^{o}F$) and 50% relative humidity. Free shrinkage specimens of 76 x 76 x 286 mm (3 x 3 x 11 in.) dimension were also casted. Their concrete mix design included a typical mix from both the Kansas DOT and Missouri DOT and seven laboratory mixes. The results showed that the ultimate free shrinkage increased as the paste content of concrete increased. Adding a shrinkage reducing admixture (SRA) significantly decreased the free shrinkage and shrinkage rate. Early-age free shrinkage was reduced by increasing the curing time, although curing time did not have influence on the restrained shrinkage rate at the start of drying. Surface to volume ratio influenced shrinkage in the way that the increase of surface to volume ratio caused the increase of free shrinkage and restrained shrinkage. Of the 39 restrained rings in their study, only the Missouri DOT mix cracked, which had the highest paste content and highest shrinkage rate of all. As a result of this study, they recommended that the concrete mix with lower paste content should be used; the shrinkage reducing admixtures (SRA) can be used to reduce shrinkage cracking.

Gong (2006) investigated the cracking behavior of high-performance concrete using restrained ring test, fracture test and numerical analysis method. They used the AASHTO type ring specimen test to study the restrained cracking characteristics of different concrete mixtures. The steel ring had inside and outside diameters of 280 and 305 mm (11 and 12 in.), respectively. The outside diameter of the concrete was 457 mm (18 in.). The heights of both steel ring and concrete ring were 152 mm (6 in.). Four strain gages were used at four equidistant mid-height locations on the interior side of the steel ring to monitor the strain. A strain drop of 30 microstrains would indicate cracking development. The concrete ring was cured for 24 hours and then moved to a condition chamber with constant air temperature of 23 ^{o}C (73 ^{o}F) and relative humidity of 50%. Free shrinkage and the mechanical properties, such as direct tensile strength, compressive strength, as well as modulus of elasticity, were also studied. They concluded that the AASHTO ring test could capture the cracking onset of high-performance concrete with reasonable accuracy. The test results showed that under the same condition, the gravel generally had better cracking resistance than limestone. High cementitious materials and low w/cm leaded to earlier cracking. A cracking index was recommended,

$$C_{ind} = \frac{100 \times f_c^{'\frac{1}{10}}}{\varepsilon_{free} E^{1.2}} \tag{1.2}$$

where: $f_c^{'}$ is the standard 28-day compressive strength,

ε_{free} is the 90-day free shrinkage strain,

E is the modulus of elasticity at 28 days.

Based on the data of real-life field, the cracking index was estimated and used to get a threshold cracking onset day to assess the cracking potential of different mix designs using gravel. Their study also stated that more experimental work is needed to establish a more reliable relationship between the cracking index and the basic properties of concrete.

Delatte et al. (2007) studied the effect of using high-absorptive materials to improve internal curing of low permeability concrete to reduce shrinkage cracking using free shrinkage and restrained ring tests. Besides field observation, they conducted experimental research in four phases: concrete mixtures using traditional Ohio DOT materials and mixture designs, concrete mixtures using high absorption fine lightweight aggregate, concrete mixtures using coarse aggregate with a larger nominal size in blended mixture, and field testing. For restrained ring test, they used a 330 mm (13 in.) outside diameter steel tube acted as restraint, which has a thickness of 13 mm (1/2 in.). The diameter of the outer form for the concrete ring was either 406 mm (16 in.) or 457 mm (18 in.) with a height of 152 mm (6 in.). The outer form was removed 24 hours after casting. Specimens were moved to an environmental chamber at a temperature of 22^oC and a relative humidity of 50%. Two strain gages were mounted at opposite mid-height of the inner surface of the steel ring to monitor the strain development. The unrestrained or free shrinkage specimens were 76.2 x 76.2 x 254 mm (3 x 3 x 10 in.) beams. Two sets of beams were made, one set kept in water bath and the other at the environmental chamber. Their research concluded that the strongest effect on cracking was to replace a small maximum size coarse aggregate (#8) with an aggregate blend of #57 and #8. Increasing the coarse aggregate absorption level from low to medium was less effective in reducing shrinkage cracking. The introduction of light weight aggregate for internal curing also had a less effect on shrinkage cracking. Thus, the use of a larger size aggregate (e.g., #57) or a blend of sizes was recommended for reducing shrinkage cracking of bridge decks.

2.8 Potential Causes of Early-age Shrinkage Cracking

Several state DOTs have conducted studies (Folliard et al. 2003; Delatte et al. 2007) on early-age cracking in concrete bridge decks and identified some potential causes and remedies. Based on a survey of the available studies, the early-age shrinkage cracking in concrete bridge decks can be caused by a number of mechanisms, including one or more of the following:

- Delay in curing, wind, low humidity and hot weather causing plastic shrinkage.
- High strength or high-performance decks with low water-cementitious material ratio resulting in autogenous shrinkage due to self-desiccation.
- Improper mix design with high cement content or high quantity of water, resulting in high drying shrinkage.
- Restraint from deep longitudinal girders and their connections (e.g., shear studs) increasing the restrained shrinkage stresses.
- Low tensile strength resulting in less resistance to cracking.
- High modulus of elasticity of concrete causing high stresses for a given shrinkage strain.
- Low creep properties that do not allow for stress relaxation.
- Temperature differential between the newly placed deck and supporting girders with different shrinkage rates causing induced stress in concrete.
- High curing temperatures causing excessive evaporation of water.

2.9 Remedies for Enhancing Shrinkage Cracking Resistance

To reduce and/or eliminate shrinkage cracks, a variety of strategies have been employed, and they include:

- Improved curing practices to prevent excessive loss of water due to evaporation (e.g., using continuous fogging and wind breaks in construction immediately after finishing).
- Internal curing strategies (Delatte et al. 2007) - (a) Using an optimized combination of coarse aggregate gradation (e.g., replacing a small maximum size coarse aggregate

with a blend of small and large aggregates); (b) Utilizing high absorption aggregate (e.g., absorption level > 1%); (c) Replacing fine aggregate with light weight aggregate (LWA) of high water absorption; and (d) Employing super absorbent polymer particles (SAP) as an alternative to moderately absorptive aggregate or expanded shale structural lightweight aggregate particle replacement.

- Improved mix designs and reduce the paste content- mixture proportion optimization with locally-available materials (e.g., decreasing the volume of water and cement and maintaining an air content above 6%). Use larger size aggregates with optimized gradation to reduce the need of water and cementitous materials in concrete.
- Improved construction methods to reduce the shrinkage restraint.
- Addition of single or hybrid fibers (specific fiber types and mix combinations need to be matched to achieve the desired characteristics) to increase the bonding strength of concrete to resist concrete shrinkage cracking.
- Incorporation of shrinkage-reducing admixtures (SRA). SRA reduces the surface tension of water and were found to reduce concrete free shrinkage greatly by many researches. Currently SRA has not been used in concrete bridge decks in Washington State. So SRA will be evaluated with local Washington State materials.
- If shrinkage reducing admixtures and/or synthetic fibers are used in the mix design, a compatibility study is needed. As a chemical additive, shrinkage reducing admixtures may cause changes in the mechanical properties of concrete, such as flexural strength, compressive strength, etc.
- Inclusion and effects of fly ash (C and F), slag, and silica fume. The replacement of cement using fly ash will slow down the hydration process of concrete, and it reduces

the early-age strength of concrete.

Based on the above remedies, the improvement of concrete mix designs is considered in this study as a viable strategy to mitigate the early-age shrinkage cracking. Several factors in mix designs, such as paste content, cementitious materials (admixtures) (fly ash, silica fume, slag), SRA, size and type of aggregates, etc., will be investigated in order to arrive at the optimum mix designs with reduced or eliminated early-age shrinkage cracking problem.

2.10 Review of Adopted Test Methods

In order to evaluate the factors in the concrete mix designs that affect the shrinkage cracking of concrete, a number of tests must be conducted. According to the state of the concrete when it is being tested, these tests can be put into two classes: (1) fresh concrete tests, and (2) hardened concrete tests.

Fresh concrete property tests evaluate the following properties of concrete: air content, slump, and unit weight. The hardened concrete property tests can be further divided into two sub-classes. The first one is about the early-age properties, such as the compression strength of concrete, the flexural strength of concrete, and the modulus of elasticity of concrete. The second is the drying shrinkage of concrete, which include the free shrinkage and the restrained shrinkage. Depending on the importance of other properties and applications, some additional tests (e.g., permeability, freeze/thaw, scaling) may also be conducted for the finalized candidate mixture(s) with the best shrinkage cracking resistance in order to develop a concrete mix performance matrix. For each concrete mix, the tests considered in this study are summarized in Table 2.1.

Table 2.1 Fresh and hardened property tests

Properties of Concrete	Test Methods
Fresh Properties of Concrete	
Air content	ASTM C 231/AASHTO T 152
Slump	ASTM C 143/AASHTO T 119
Unit Weight	ASTM C 138
Hardened Properties of Concrete	
Compression Strength of Concrete	ASTM C 39/AASHTO T 22
Flexural Strength of Concrete	ASTM C 78/AASHTO T97
Splitting Tensile Strength of Concrete	ASTM C 496/AASHTO T 198
Modulus of Elasticity of Concrete	ASTM C 496
Unsealed Free Shrinkage	ASTM C 157 AASHTO T 160
Sealed Free Shrinkage	ASTM C 1090
Restrained Shrinkage of Concrete	AASHTO PP34-99

CHAPTER THREE

MATERIALS AND SELECTION OF CONCRETE MIX DESIGNS

3.1 Introduction

The primary goal of this study is to develop and evaluate different concrete mix designs using large nominal size aggregates, different cementitious and admixture material proportions, and different sources of aggregates to identify the concrete mix designs that will have the best cracking resistance as well as good mechanical properties.

3.2 Materials

3.2.1 Cementitious Materials

The cementitious materials, including Portland cement, fly ash (FA), silica fume (SF), and slag (SL), are provided by Lafarge NA – PNW District. The properties and chemical contents are listed in Table 3.1.

Table 3.1 Properties and chemical contents of cementitious materials

	Cement	Fly Ash	Silica Fume	Slag
Specific Gravity	3.15	2.04	2.2	2.89
SiO_2, %	20	53.3		
Al_2O_3, %	4.6	23.1		
Fe_2O_3, %	3.3	3.4		
CaO, %	64.6	10		
MgO**, %**	0.8	1.1		
SO_3, %	2.7	0.1		
Loss on Ignition	2.6	0.4		
Limestone	3			

3.2.2 Aggregates

Coarse aggregates from both eastern Washington (EW) and western Washington (WW) are used. Eastern Washington coarse aggregates are provided by Central Pre-Mix Concrete Company in Spokane, WA. Four different sizes of coarse aggregates are used in this study: the nominal sizes 2.5 in., 2.0 in., 1.5 in., and 3/8 in. The gradations of the coarse aggregates are presented in Table 3.2. The specific gravities are listed in Table 3.3.

Table 3.2 Eastern Washington Coarse Aggregate Gradations (sieve analysis)

	Eastern Washington 3/8" Pea Gravel	Eastern Washington 1.5"	Eastern Washington 2"	Eastern Washington 2.5"
Sieves	Cumulative % Passing	Cumulative % Passing	Cumulative % Passing	Cumulative % Passing
2"1/2			-	100
2"			-	91.7
1"1/2		100	-	22.5
1"1/4		94.8	-	6.2
1"		64.4	-	0.8
3/4"		11.4	-	0.2
5/8"		2.8	-	
1/2"	100	1.3		
3/8"	98.5	0.6		
1/4"	67.8			
#4	37.3			
#8	3			
#16	0.4			

Table 3.3 Specific Gravity of Eastern Washington Aggregates

Aggregates	EW 2.5"	EW 2"	EW 1.5"	EW 3/8"	Sand
Specific Gravity	2.7	2.7	2.7	2.67	2.65

While the western Washington coarse aggregates are provided by Glacier NW, Seattle, WA. The gradations of western aggregates are listed in Table 3.4. The specific gravities are given in Table 3.5.

Table 3.4 Western Washington Coarse Aggregate Gradations (sieve analysis)

	Eastern Washington 3/8" Pea Gravel	Eastern Washington 1.5"	Eastern Washington 2"	Eastern Washington 2.5"
Sieves	Cumulative % Passing	Cumulative % Passing	Cumulative % Passing	Cumulative % Passing
2"1/2				
2"				
1"1/2		100		
1"1/4		91.6		
1"		48		
3/4"		2.4		
5/8"		0.6		
1/2"	100	0.5		
3/8"	86.4	0.4		
5/16"	64.6	0.1		
1/4"	38.5			
#4	13.9			
#8	0.7			
#16	0.2			
#200	0.1			

Table 3.5 Specific Gravities of Western Washington Coarse Aggregates

Aggregates	WW 2.5"	WW 2"	WW 1.5"	WW 3/8"
Specific Gravity	2.7	2.7	2.7	2.67

Fine aggregate is provided by Central Pre-Mix Concrete Company in Spokane, WA. The fine aggregate meets Class 1 WSDOT Sand requirements. The specific gravity of five aggregate is also listed in Table 3.3. The detailed gradation is listed in Table 3.6.

Table 3.6 Fine Aggregate Gradation (sieve analysis)

	Fine Aggregate	
Sieves	Individual % Retained	Cumulative % Passing
3/8"	0	100
1/4"	0.5	99.5
#4	1.8	97.7
#8	13.4	84.3
#16	23.3	61
#30	18.8	42.2
#50	24.5	17.7
#100	13.6	4.1
#200	1.9	2.2

3.2.3 Chemical Admixtures

Three types of chemical admixtures are used: air entraining admixture (AEA), shrinkage reducing admixture (SRA), and high range water reducing admixture (HRWRA).

DARAVAIR 1000 air-entraining admixture from Grace Construction Products is used to ensure proper air content in all the concrete mixes. According to the information from the product instructions, it is based on a high-grade saponified rosin formulation and chemically similar to vinsol-based products. The adding amount is decided by the recommended addition rate from the product instructions and adjusted according to practice.

ADVA 190 high-range water-reducing admixtures from Grace Construction Products is adopted to achieve the desired slump value as well as reducing the water content in all concrete mixes. It is a polycarboxlate-based admixture specifically designed for concrete industry. Its adding rate is also determined according to the product instructions and adjusted by practice.

Eclipse Plus shrinkage reducing admixture (SRA) from Grace Construction Products is added to some of the concrete mixes to reduce concrete drying shrinkage. Eclipse Plus decreases drying shrinkage by reducing the surface tension of water, which causes a force pulling in on the walls of the pores in concrete. Its adding rate is also decided by its recommended amount and by practice. When Eclipse Plus shrinkage reducing admixture (SRA) is added, the same amount of water is taken out.

3.3 Mix Design Rationale and Considerations

In order to evaluate the concrete mix designs for mitigating shrinkage cracking, a number of factors in the mix designs are considered. First, the mix designs take different selections and proportions of cementitious materials and chemicals into consideration. Supplementary cementitious materials (SCM), such as fly ash (FA), silica fume (SF), and slag (SL), are being used by many DOTs to partially replace cement in a concrete mix. Single replacements of cement by SCM are evaluated. To further reduce the cement content, the replacements of cement by binary combination of SCM is also performed. Based on the literature review, the single replacement of cement is selected as 20% by fly ash or slag, and 4% by silica fume.

Second, larger sizes of aggregates are considered in the mix designs as well to reduce paste content as suggested by the literature review and to reduce shrinkage cracking tendency of concrete. As suggested by the WSDOT, aggregates with nominal size of 1.5 in. (38.1 mm) are used to replace the current WSDOT normal use, of which the nominal size aggregates of 1 in. (24.5 mm). When the larger sizes of aggregates are used, the paste contents of concrete mix designs are correspondingly reduced compared with small size aggregates. In this study, the large sizes of aggregates of 1.5 in. (38.1 mm), 2.0 in. (50.8 mm), and 2.5 in. (63.5 mm) are evaluated, along with two sources of aggregates, i.e., Eastern and Western Washington aggregates.

Shrinkage reducing admixture (SRA) was also used as suggested by the literature review to reduce the drying shrinkage of concrete.

3.4 Mix Design and Procedures by ACI 211.1-91

ACI Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete (ACI 211.1-91) provides the procedures for developing proportions for hydraulic cement concrete. Basically, concrete is composed of Portland cement, aggregates, sand and water. It may also contain other kinds of cementitious materials, such as fly ash, silica fume, and slag, as well as some chemicals such as air entrainer, water reducer, and shrinkage reducer. In ACI 211.1-91, a series of requirements are followed, and then the intended mix designs are developed according to these requirements. The following are the procedures that the mix designs in this study are developed using the aggregates of nominal size 1.5 in. (38.1 mm).

Step 1: Choice of Slump. For pavements and slabs, the slumps recommended by ACI 211 are 1 to 3 in. (25.4 to 76.2 mm) (see Table 3.7). Also, it states that if chemical admixtures are used in the mix and the water-cement ratio does not increase, slump may be increased as long as the concrete does not have segregation or excessive bleeding.

Table 3.7 Recommended slumps for various types of construction (From ACI 211.1-91)

Types of construction	Slump, in.	
	Maximum+	Minimum
Reinforced foundation walls and footings	3	1
Plain footings, caissons, and substructure walls	3	1
Beams and reinforced walls	4	1
Building columns	4	1
Pavements and slabs	3	1
Mass concrete	2	1

Step 2: Choice of maximum size of aggregate. Compared with WSDOT current applications, larger sizes of aggregates are considered in this study, and the maximum size of aggregate is 1.5 in (38.1 mm) or larger.

Step 3: Estimation of mixing water and air content. According to ACI 211.1-91 (Table 3.8), for concrete with slump of 1 to 3 in. and a maximum size aggregate of 1.5 in. (38.1 mm), water content of 250 to 275 lbs and air content of 5.5% are recommended. Also, it states that when water reducer is used, the water content can be reduced by 5% or more.

And when the rounded aggregate is used, the water content can be reduced by 25 lbs for air entrained concrete. Based on these criteria, the water content for this study will be 212.5 to 236.5 lbs. Thus, 220 lbs of water is chosen in this study.

Table 3.8 Approximate mixing water and air content requirements for different slumps and nominal sizes of aggregates (From ACI 211.1-91)

Water, lb/yd³ of concrete for indicated nominal maximum sizes of aggregate								
Slump, in.	⅜ in.*	½ in.*	¾ in.*	1 in.*	1-½ in.*	2 in.*,†	3 in.†,‡	6 in.†,‡
Non-air-entrained concrete								
1 to 2	350	335	315	300	275	260	220	190
3 to 4	385	365	340	325	300	285	245	210
6 to 7	410	385	360	340	315	300	270	—
More than 7*	—	—	—	—	—	—	—	—
Approximate amount of entrapped air in non-air-entrained concrete, percent	3	2.5	2	1.5	1	0.5	0.3	0.2
Air-entrained concrete								
1 to 2	305	295	280	270	250	240	205	180
3 to 4	340	325	305	295	275	265	225	200
6 to 7	365	345	325	310	290	280	260	—
More than 7*	—	—	—	—	—	—	—	—
Recommended averages§ total air content, percent for level of exposure:								
Mild exposure	4.5	4.0	3.5	3.0	2.5	2.0	1.5**,††	1.0**,††
Moderate exposure	6.0	5.5	5.0	4.5	4.5	4.0	3.5**,††	3.0**,††
Severe exposure‡‡	7.5	7.0	6.0	6.0	5.5	5.0	4.5**,††	4.0**,††

Step 4: Selection of water-cement or water-cementitious materials ratio. Based on most studies in the literature, a water-cementitious materials ratio of 0.4 is used in this study.

Step 5: Calculation of cementitious materials. When a water content of 220 lbs (Step 3) and a water-cementitious material ratio of 0.4 (Step 4) are used, the total of cementitious materials is calculated to be 550 lbs.

Step 6: Estimation of coarse aggregate content. According to ACI 211.1-91, the volume of aggregates can be determined by considering the nominal maximum aggregate size and the fineness modulus of fine aggregate. The fine aggregate modulus is 2.7, and the nominal size of aggregate considered in this study is 1.5 in. (38.1 mm). Therefore, the volume ratio of aggregate is 0.72, resulting in 1,847 lbs of course aggregate per cubic yard of concrete.

Table 3.9 Volume of coarse aggregate per unit of volume of concrete (From ACI 211.1-91)

Nominal maximum size of aggregate, in.	Volume of oven-dry-rodded coarse aggregate* per unit volume of concrete for different fineness moduli of fine aggregate+			
	2.40	2.60	2.80	3.00
⅜	0.50	0.48	0.46	0.44
½	0.59	0.57	0.55	0.53
¾	0.66	0.64	0.62	0.60
1	0.71	0.69	0.67	0.65
1½	0.75	0.73	0.71	0.69
2	0.78	0.76	0.74	0.72
3	0.82	0.80	0.78	0.76
6	0.87	0.85	0.83	0.81

Step 7: Estimation of fine aggregate content. Two methods are provided to calculate the amount of fine aggregate needed: the weight deduction method and the volume method. The weight deduction method uses the total estimated weight of fresh concrete (Table 3.10) to deduct all the other materials to obtain the amount of fine aggregate needed. The volume method is based on the volume of the amount of fine aggregate needed, which is calculated by using the total volume to deduct the volume of all the other materials.

Table 3.10 Estimation of weight of fresh concrete (From ACI 211.1-91)

Nominal maximum size of aggregate, in.	First estimate of concrete weight, lb/yd³*	
	Non-air-entrained concrete	Air-entrained concrete
3/8	3840	3710
1/2	3890	3760
3/4	3960	3840
1	4010	3850
1 1/2	4070	3910
2	4120	3950
3	4200	4040
6	4260	4110

Step 8: Adjustments for aggregate moisture. The aggregates calculated in the above steps are in their oven-dry state. They should be adjusted according to their actual water content when casting.

Step 9: Trial batch adjustments. This step is to make adjustments to the above developed estimation of proportions of concrete mix design so that the total volume will be one cubic yard as assumed in the estimation.

However, ACI 211.1-91 has one disadvantage. It can only be used to develop a mix design using one size coarse aggregate, such as the 1.5 in. (38.1 mm) aggregate. In practice, if only 1.5 in. nominal size coarse aggregate is used, the workability and the concrete will be low, and the coarseness factor will be too high. To avoid these results, #8 aggregate with nominal size of 3/8 in. is introduced into the mix design to replace part of the 1.5 in. aggregate. The KU (University of Kansas) mix program is introduced next to compensate for the disadvantage of ACI 211.1-91, especially when large sizes of aggregates and a combination of different sizes of aggregates are considered in the mix designs.

3.5 Mix Optimization and Analysis by the KU Mix Program

The KU Mix program developed by the researchers at University of Kansas (http://www.silicafume.org/ku-mix.html) is based on Microsoft Excel to design concrete mixes, and it includes function for aggregate optimization. It also includes a series of procedures to develop a concrete mix. The following is how the KU mix program works.

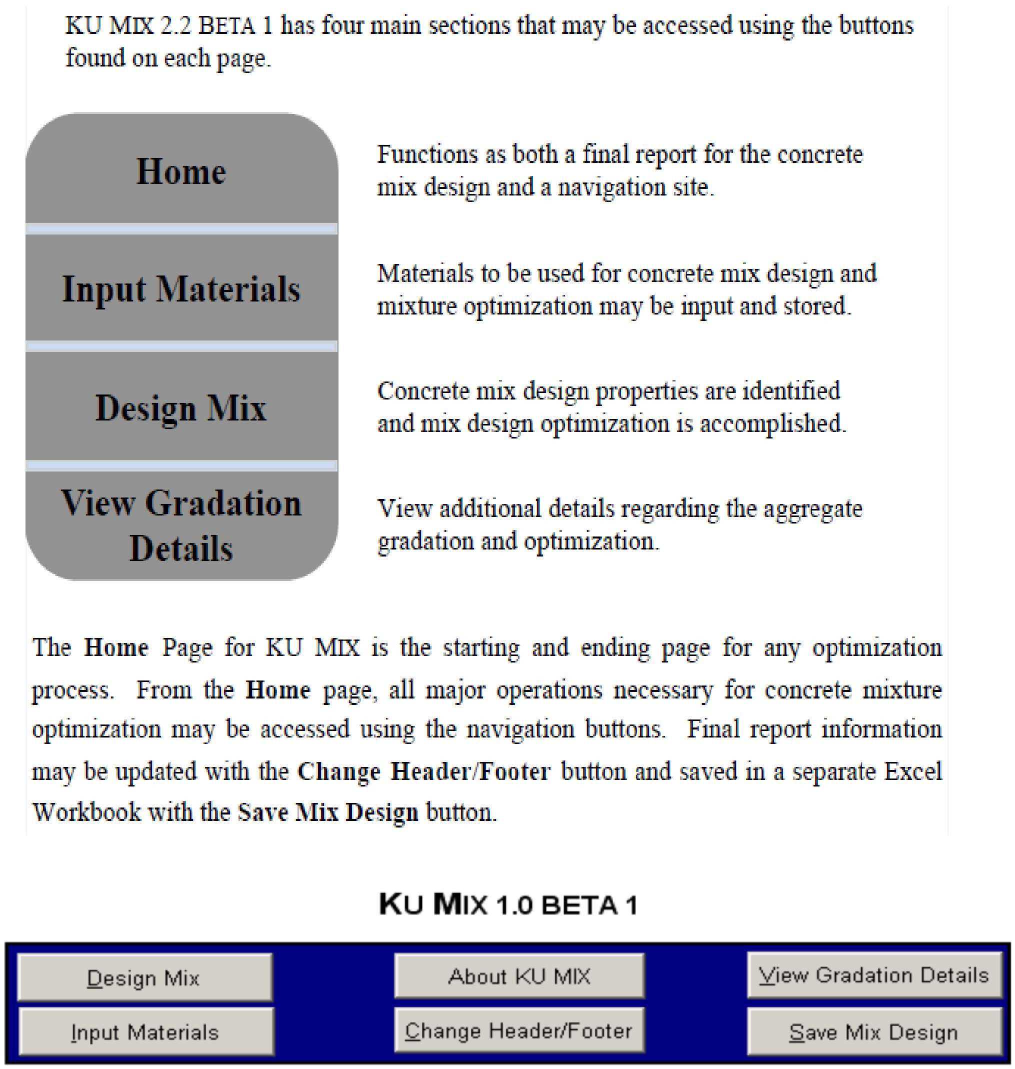


Fig. 3.1 KU Interface (From KU Mix Program)

Step 1: Input Materials. Before using the KU Mix program to design a concrete mix (see Fig. 3.1), the material properties must be first input. Four basic materials are required: cementitious materials, aggregates, air entraining agents, and other admixtures. After all material information is input, the design procedure can be started.

Step 2: Design Concrete Mix. There are four parts in this step. First, the cementitous materials, water-cementitous materials ratio, and air content are selected and their amounts are entered. Second, aggregates are selected from previously entered ones (in Step 1). Third, chemical admixtures are selected and their amounts are entered. After all these three sub-steps, click "Optimize Mix Design". The mix design is then accomplished, and it can be viewed through "View Mix Design", as shown in Fig. 3.2.

Besides the use for developing a mix design, the KU Mix also provides the gradation details of aggregates and sand. And the workability factor and coarseness factor are given.

As aforementioned, the ACI 211.1-91 is only capable of developing a mix design using one coarse aggregate. While in the KU Mix, different sizes of coarse aggregates can be selected to compose a concrete mix. Therefore, by introducing the KU Mix program to supplement the shortcoming of ACI 211.1-91, a better and optimized mix design can be developed. The concept is to first use ACI 211 to determine the water-cementitous materials ratio, water content, cementitious material contents, and air content. The KU Mix is then utilized to determine the amount of aggregates needed.

The following is a two-step guideline on how to develop a viable mix design using a combined ACI 211.1-91 and the KU Mix program.

Step 1: Using ACI 211.1-91 to preliminarily select basic amount of constituent materials. As previously introduced, for aggregate with nominal size of 1.5 in. in this study, the water content of 220 lbs, the cementitious material content of 550 lbs, and air content of 5.5% are preliminarily selected following ACI 211.1-91. However, per the WSDOT recommendations, the air content should be in the range from 6.5% to 9%. Thus, the targeted value for the air content is chosen to be 8%.

Step 2: Using the KU Mix to finalize the concrete mix design. Input all material information into the KU Mix, and also enter the values obtained from Step 1. In the "Select Aggregates" procedure, select both 1.5 in. and 3/8 in. coarse aggregates as well as sand. Thus, more than one kind of coarse aggregates are introduced into the concrete mix design. Then, by following the procedures discussed for the KU Mix program, a concrete mix design is developed.

Washington State Department of Transportation
, Washington

CONCRETE MIX DESIGN
Compressive Strength: 4000D

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 100%	550 lb	3.15	2.80
SL--GGBFS / 123 / 0%	0 lb	2.89	
FA--Fly Ash Type-F / 123 / 0%	0 lb	2.04	
SF--Silica Fume / 123 / 0%	0 lb	2.18	
Water	218 lb	1.00	3.49
new-#4 / #4 / 37.5%	1160 lb	2.70	6.89
new-#8 / #8 / 38%	1177 lb	2.68	7.04
New-Class 2 sand / sand / 24.5%	759 lb	2.65	4.59
Total Air, percent	8%		2.16
Daravair® 1000 Air Entrai / Grace Construction	14.5 fl oz (US)	1.02	0.02
Eclipse Plus SRA / Grace Construction	0 fl oz (US)	0.96	0.00
Adva 190 HRWRA / Grace Construction	17.4 fl oz (US)	1.10	0.02
[1]The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.			27.01

Total Water Content (including water in admixtures), lb			220	
Water / Cementitious Material Ratio:			0.4	
Concrete Unit Weight, pcf			143.2	
Target Slump, in.			BLANK	
Paste Content, percent			23.43%	
Workability Factor (WF)	Target:	34.9	Actual:	34.9
Coarseness Factor (CF)	Target:	60.9	Actual:	60.9

Fig. 3.2 Concrete Mix Design by KU Mix Program

The concrete mix designs developed using KU Mix program are given in the appendix.

3.6 Mix Designs

By combining the ACI 211.1-91 and the KU Mix program, the mix designs for this study are developed and summarized in Table 3.11 along with the benchmark mix design from the WSDOT.

3.7 Concluding Remarks

In this chapter, the rationale and procedures for coming up all the mix designs using ACI 211.1-91 and the KU Mix program are provided and discussed. In these mix designs, the effects of adding different SCMs, such as fly ash, silica fume, and slag, to concrete mix design to partially replace cement are discussed. In comparison with the current WSDOT practice of using #67 aggregate, the newly developed mix designs with a large size of aggregates (e.g., #4 aggregate) using ACI 211.1-91 and the KU Mix program are elaborated.

Table 3.11 Mix Designs

A. Phase One

Mixtures	Cement	FA	SF	Slag	Paste (%)	#4	#8	Sand	w/cm	Air	Water
EW-SRA	550	0	0	0	23.43	1161	1177	759	0.4	8	220
EW-SL-SRA	440	0	0	110	23.61	1161	1180	749	0.4	8	220
EW	550	0	0	0	23.43	1161	1177	759	0.4	8	220
EW-SF-SRA	462	0	22	0	20.79	1154	969	1087	0.4	8	193.6
EW-FA-SRA	440	110	0	0	24.54	1160	1192	697	0.4	8	220
EW-FA-SL-SRA	330	110	0	110	24.72	1159	1194	686	0.4	8	220
EW-FA	440	110	0	0	24.54	1160	1192	697	0.4	8	220
EW-FA-SF-SRA	352	110	22	0	21.9	1187	1205	772	0.4	8	193.6
WW-SRA	550	0	0	0	23.43	1017	811	1259	0.4	8	220
WW-SL-SRA	440	0	0	110	23.61	1016	813	1250	0.4	8	220
WW	550	0	0	0	23.43	1017	812	1260	0.4	8	220
WW-SF-SRA	462	0	22	0	20.79	1044	822	1342	0.4	8	193.6
WW-FA-SRA	440	110	0	0	24.54	1014	821	1203	0.4	8	220
WW-FA-SL-SRA	330	110	0	110	24.72	1013	822	1194	0.4	8	220
WW-FA	440	110	0	0	24.54	1014	821	1204	0.4	8	220
WW-FA-SF-SRA	352	110	22	0	21.9	1041	832	1286	0.4	8	193.6

Note: EW=Eastern Washington Coarse Aggregates, SRA=Shrinkage-Reducing Admixtures, SL=Slag, SF=Silica Fume, FA=Fly Ash, and WW=Western Washington Coarse Aggregates.

B. Control Mixes

Mixtures	Cement (lb/yd3)	Fly Ash (lb/yd3)	Slica fume (lb/yd3)	Slag (lb/yd3)	3/4" Aggregate (lb/yd3)	Sand (lb/yd3)	w/cm	Air Content (%)	Water (lb)
WSDOT	660	75			1730	1250	0.34	6.5	250
LD-WSDOT	564				1830	1270	0.48	4.8	270

C. Phase Two

Mixtures	Cement (lb/yd^3)	2.5" Aggregate (lb/ yd^3)	2" Aggregate (lb/ yd^3)	3/8" Aggregate (lb/ yd^3)	Sand (lb/ yd^3)	w/cm	air	water
EW 2	525	-	1072.6	850	1240	0.4	8	210
EW 2.5	500	1125	-	850	1240	0.4	8	200
WW 2	525	-	1072.6	850	1240	0.4	8	210
WW 2.5	500	1125	-	850	1240	0.4	8	200

CHAPTER FOUR

TEST METHODS

4.1 Introduction

This chapter describes the test methods considered in this study for the fresh, hardened, and shrinkage properties in concrete mix designs.

4.2 Concrete Mixing Procedures

Concrete mixing procedures are developed based on concrete mixing guidelines and the literature, following the relevant AASHTO and ASTM standards. The following steps are the concrete mixing procedures used in this study:

1. All the materials are batched by weight.
2. Two pounds of water and two pounds of cement are mixed together and then used to wet the inside drum of the concrete mixer. Then, the paste is dumped.
3. All the pre-weighted aggregates and sand are added into the mixer and mixed for 1/2 minute.
4. All the pre-weighted cementitious materials (cement, fly ash, silica fume or slag) are added into the mixer. The air-entraining admixture (AEA) is added into half of the water, and the water solution is then added into the mixer. They are mixed for 3 minutes;
5. The rest water is added, and they are mixed for 2 minutes;
6. Water reducing admixture (WRA) and SRA are added separately, and they are then mixed for 3 minutes;

7. The mix is rested for 2 minutes;
8. It is mixed for the final 2 minutes;
9. The slump test is first conducted;
10. The air content test is then conducted; and
11. Necessary adjustments of WRA and AEA are made until the targeted slump and air content are achieved.

4.3 Fresh Property Tests

Slump and air content are tested for every trail mix design following the standards of: (1) AASHTO T 119/ASTM C 143-08 Slump of Hydraulic Cement Concrete, (2) AASHTO T 152/ASTM C 231 -08 Air Content of Freshly Mixed Concrete by the Pressure Method, and (3) AASHTO T 196/ASTM C 173 Air Content of Freshly-mixed Concrete by the Volumetric Method.

4.3.1. Slump Test

The slump test (Fig. 4.1) is performed immediately after the mixing procedures to avoid loss of slump. The procedures for conducting slump test are as follows:

1. The mold is dampened and placed on a required surface.
2. Fill the mold with three layers of concrete.
3. Rod each layer with 25 strokes of the tamping rod.
4. In filling and rodding the top layer, heap the concrete above the mold.
5. Slump is immediately measured by determining the vertical difference between the top of the mold and the displaced original center of the top surface of the specimen.

Fig. 4.1 Slump Test

Fig. 4.2 Air Content Test by Pressure Method

Fig. 4.3 Device for Air Content Test by Volumetric Method

4.3.2 Air Content Test

Air content test (Figs. 4.2 and 4.3) is performed to measure the air content in the fresh concrete. The procedures for air content test using the pressure method (Fig. 4.2) are as follows:

1. Dampen the interior of the measuring bowl and place it on required surface.
2. Fill the mold with three layers of concrete.
3. Rod each layer with 25 strokes of the tamping rod.
4. Strike off the top surface till the bowl is just level full.
5. Assemble the apparatus and measure the air content.

The procedures for air content test using the volumetric method (Fig. 4.3) are as follows:

1. Dampen the interior of the measuring bowl and place it on required surface.
2. Fill the mold with two layers of concrete.
3. Rod each layer with 25 strokes of the tamping rod.
4. Strike off the top surface till the bowl is just level full.
5. Add water according to the standards.
6. Displace the volume of air and measure the air content according to the standards.

4.4 Mechanical Property Tests

4.4.1 Compressive Test and Young's Modulus Test

ASTM C 39/AASHTO T 22 is used to measure the compressive strength of the concrete mixes. ASTM C 469 is used to measure the static modulus of elasticity. For every mix, four cylinder specimens of 6 in. diameter x 12 in. height are cast. Two are tested at the age of 7 days, and two at 28 days. The test is conducted using a hydraulic machine (Fig. 4.4) at a constant load rate of 35±7 psi/s. The cylinder specimens are first loaded till about 40% of its ultimate strength and then unloaded. After that, the displacements and responding loads are recorded to calculate the Young's modulus. Then, the specimens are unloaded again and then reloaded till failure, with all displacements and loads data being recorded.

The test procedures for the Young's modulus and compressive strength include:

1. Remove specimen from curing environment.
2. Place specimen into testing apparatus ensuring clean surfaces and center the specimen under the center of the thrust.

3. The load indicator is zeroed and the loading plate bears on the specimen
4. Apply the load continuously and without shock till 40% of the estimated ultimate load to make sure that the setup is connected correctly then unload.
5. Apply till 40% of the estimated ultimate load and record the dial meter readings at every 5,000 lbs of force increase.
6. Unload and then reload till the cylinder fails. Record the maximum load.
7. Calculate the modulus of elasticity and the compressive strength of the specimen.

Fig. 4.4 Compressive and Modulus of Elasticity Test

4.4.2 Flexural Strength Test

ASTM C 78/AASHTO T97 procedures are followed for measuring the flexural strength of the concrete beam specimens. The concrete beam has a dimension of 4 in x 4 in x 15 in. A 12 in. span is used, which made the height of the beam of 4 in., i.e., 1/3 of the span, following the standards. This test is also conducted using the same hydraulic machine (see Fig. 4.5), using the loading rate of 125-175 psi/min.

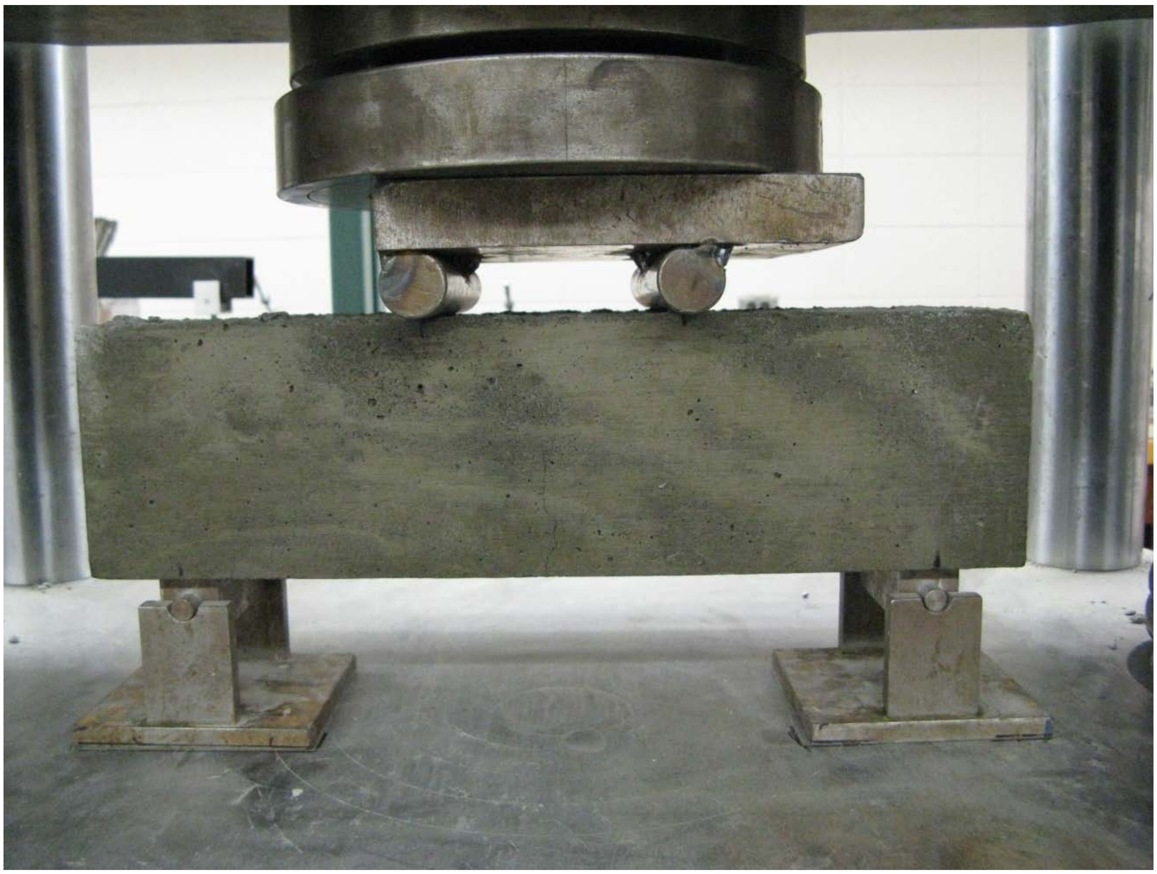


Fig. 4.5 Flexural Strength Test

The procedures for measuring the flexural strength of concrete are as follows:

1. Remove specimen from curing environment, and kept it moist until specimen is tested.
2. Place specimen in loading apparatus.

3. Load to 3-6% of estimated load, and check to make sure that load applying or support blocks are touching the specimen fully.
4. Load specimen continuously and without shock at 125-175 psi/min. Calculate loading rate using:

   $$r = \frac{Sbd^2}{L}$$

   where:

   r = loading rate, lb/min

   S = rate of stress, psi

   b = avg width, in.

   d = avg depth, in.

   L = span length, in.
5. After failure, measure specimen dimensions: b and d
6. Calculate the modulus of rupture for flexure.

4.5 Shrinkage Property Tests

4.5.1 Free Shrinkage Test

The free shrinkage test is carried out following ASTM C157/C157M and AASHTO T160 “Length Change of Hardened Hydraulic-Cement Mortar and Concrete”. Three 4 in x 4 in x 11.25 in prisms are cast using the concrete batch for both the free shrinkage and restrained shrinkage specimens in every mix design. The prisms are put into the condition room at the same time as the ring specimens, and they are demolded 24 hours after casting.

The condition room is maintained at a temperature of 76 ± 3^{o}F and a relative humidity of 50 ± 4%.

A shrinkage frame with three dial meters is made in order to monitor the free shrinkage, as shown in Fig. 4.6. The shrinkage frame uses rollers to support the specimens so that they are able to shrink free of abrasion, and the dial meters are installed to record the shrinkage value continuously. The specimens are put onto the shrinkage frame immediately after demolding. And the initial readings of the dial meters are recorded. Readings are then taken every 24 hours for the first 7 days, and then on the 14^{th}, 21^{st}, and 28^{th} days.

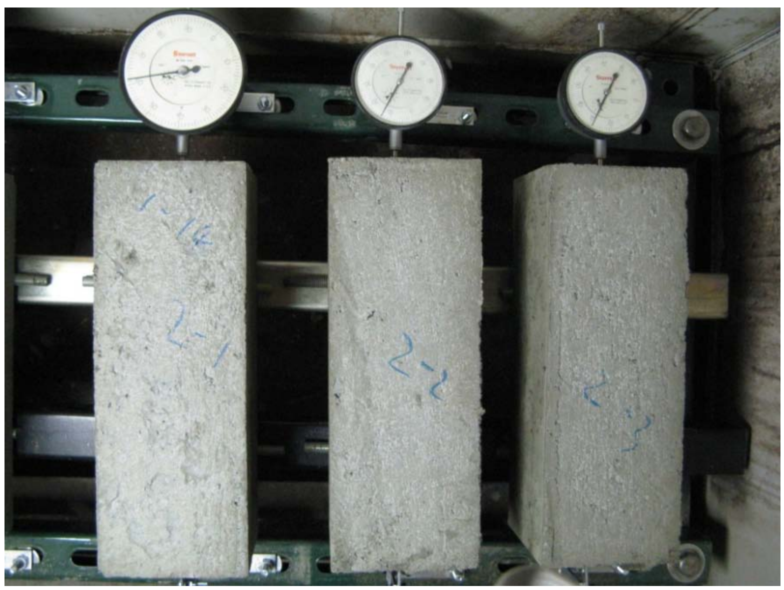


Fig. 4.6 Free Shrinkage Test

4.5.2 Restrained Shrinkage Test

As aforementioned in Chapter 2, the restrained shrinkage test adopts the AASHTO ring and follows the standards AASHTO PP34-99 “Estimating the Cracking Tendency of Concrete”. Test apparatus is fabricated as discussed in the literature review (see Section 2.6.3 in Chapter 2).

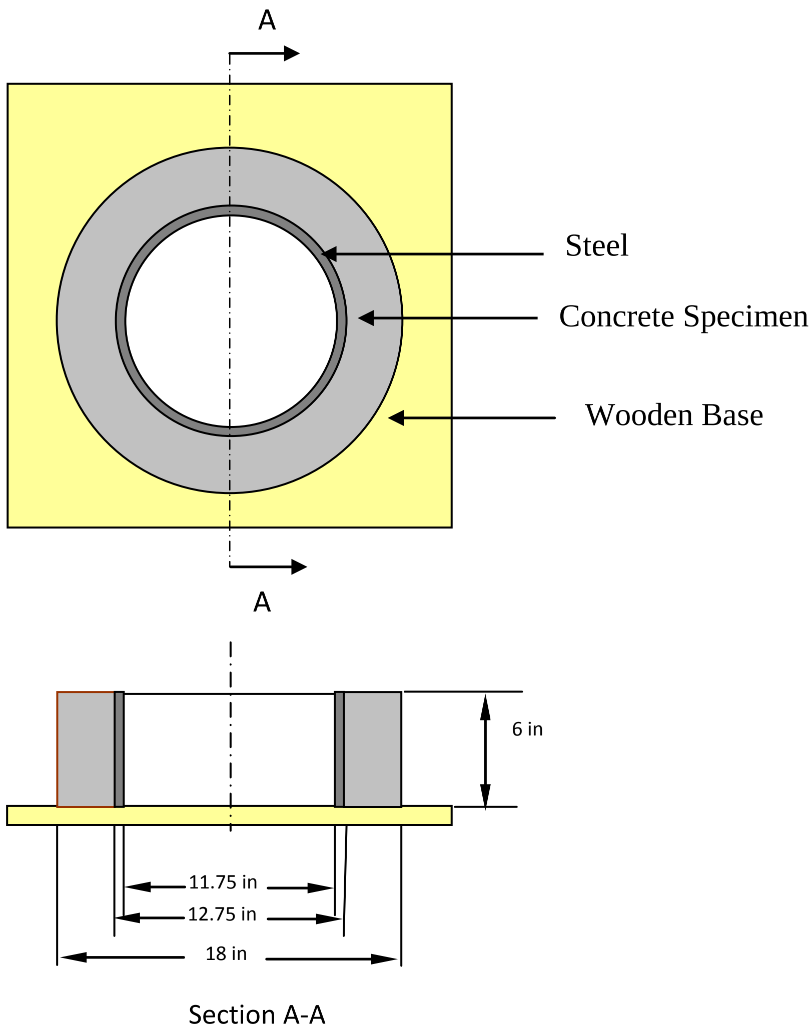


Fig. 4.7 Diagrams of Ring Specimen Used (Reprinted from AASHTO PP34-99)

Structural steel pipe conforming to ASTM A 501 or A 53M/A 53 12-in. extra-strong pipe with an outside diameter of 324 mm (12 ¾ in.) and wall thickness 13 mm (1/2 in.) is used for fabricating the inner steel ring (see Figs. 3.4 and 3.5). The outer ring is made of polyethylene board with an inside diameter of 457 mm (18 in.). Four strain gages are mounted on the inner surface of the steel ring at equidistant points at midheight (see Figs. 4.7 and 4.8). Data acquisition equipment from Vishay Company is used for the strain instrumentation, and it automatically records each strain gage every second independently.

Wooden forms are made of 24 in. by 24 in., 5/8 in. thick (0.6 x 0.6 x 0.016 m) plywood sheet, and the top surface is coated with epoxy to ensure that the concrete rings are able to shrinkage freely.

Three ring specimens are cast for most of mix designs, two 6 in. tall rings and one 3 in. tall ring. The outer forms are removed at an age of 8 h., and then the specimens are moved to the conditional room (Fig. 4.9) with a constant air temperature of 75 ± 3.5 ^{o}F and 50 ± 4 % relative humidity. The data from the strain gages are recorded every second, and review of the strain and visual inspection of cracking are conducted every 2 or 3 days.

Fig. 4.8 Restrained Shrinkage Ring Apparatus

Figure 4.9 Data Acquisition System in the Condition Room

CHAPTER FIVE

PERFORMANCE OF MIX DESIGNS WITH EASTERN WASHINGTON AGGREGATES

5.1 Introduction

This chapter reports on concrete mix designs using aggregates from eastern Washington. Two phases are carried out. Phase one is composed of eight concrete mixes designed in this study and one current WSDOT concrete mix design, which serves as a benchmark mix. Phase two includes two concrete mix designs using two larger sizes of aggregates (i.e., 2 in. and 2.5 in.). In this chapter, the materials and test results for concrete mixes using eastern Washington aggregates are presented. The eight concrete in Phase one mixes use the eastern Washington aggregates with a nominal size of 1.5 in. In contrast, the current WSDOT concrete mix design uses aggregates with a nominal size of ¾ in. The two concrete mix designs using the larger size aggregates consider aggregates with nominal sizes of 2 in. and 2.5 in. Fresh, hardened, and shrinkage properties are evaluated.

5.2 Fresh Property Tests

Following the test methods given in Chapter 4, the slump test and air content test are performed for each concrete mix to evaluate the workability.

5.2.1 Slump Test

The slump test follows ASTM C 143/AASHTO T 119 “Slump of hydraulic cement concrete”. Based on ACI 211.1-91 “Standard Practice for Selecting Proportions for Normal

Heavyweight, and Mass Concrete" and also the recommendations of WSDOT, the slump value of at least 3 in. is selected. However, as stated in ACI 211.1-91, when chemical admixtures are used and this chemical admixture-treated concrete has the same or lower water-cementitous materials ratio and does not exhibit segregation potential or excessive bleeding, the slump value may be accordingly increased. In this study, the High Range Water Reducing Admixture (HRWRA or superplasticizer) is used to increase the slump value since the cement paste content chosen in all the concrete mixes are low.

5.2.2 Air Content Test

Two methods of air content test are conducted, the pressure method and the volumetric method. The pressure method follows AASHTO T 152/ASTM C 231 "Air Content of Freshly-mixed Concrete by the Pressure Method", while the volumetric method follows AASHTO T 196/ASTM C 173 "Air Content of Freshly-mixed Concrete by the Volumetric Method". As stated in the AASHTO standards, the pressure method applies to concretes and mortars made with relatively dense aggregates, and it does not apply to concrete with light-weight aggregates, air-cooled blast-furnace slag, or aggregates of high porosity. In this study, dense aggregates are used. So the pressure method is performed for most concrete mixes except for those including slag in them, in which case the volumetric method is utilized, due to the porous nature of the added slag in the mix. The ACI recommended value for air content is 5.5 percent for severe exposure when the nominal maximum aggregate size is 1.5 in. However, in a recent WSDOT bridge deck project, the WSDOT requires the air content to be a minimum of 6.5 percent and a maximum of 9.5 percent. Therefore, the desired air content in this study is chosen as 8 percent whenever possible.

5.2.3 Test Results of Fresh Concrete Properties

The slump test and air content test data are listed in Table 5.1. The slump values are in the range of 3 to 6 in., which indicates good workability of all the concrete mixes. The air contents are also within the desired range for most of the concrete mixes. For concrete mix with EW-FA-SRA, it is lower than the desired value. However, the use of several chemicals made the desired concrete properties difficult to achieve, especially when the three chemicals are all used in one concrete mix.

Table 5.1 Slump and Air Content Data

Mixtures	EW-SRA	EW-SL-SRA	EW	EW-SF-SRA	EW-FA-SRA	EW-FA-SL-SRA	EW-FA	EW-FA-SF-SRA	WSDOT	EW 2"	EW 2.5"
Slump (in.)	4.8	6.5	3.7	3.3	4.6	6	5.8	3.5	4	5	5.5
Air Content (%)	7.2	n/a*	7.8	7.8	3	n/a*	10	7.5	6.5	7	10

Note: In the air content test of mix designs with slag (SL), the invalid pressure method is used.

5.3 Mechanical Property Tests

Three basic mechanical properties are evaluated for all the concrete mixes: the compressive strength, the modulus of elasticity, and the flexural strength. These tests are conducted to ensure that the designed concrete meets the requirements for the intended applications.

5.3.1 Compressive Strength Test

The compressive strength test follows ASTM C 39/AASHTO T 22 “Compressive Strength of Cylindrical Concrete Specimens”. For bridge deck applications, the WSDOT requires the minimum compressive strength to be 4,000 psi at 28 days. Besides the measurement at 28 days, the compressive strength of all concrete mixes at 7 days is also tested to demonstrate the strength development. The test data for the compressive strength is listed in Table 5.2 and also graphically presented in Figs. 5.1 and 5.2.

Table 5.2 Compressive Strength Test Data (psi)

Mixtures	EW-SRA	EW-SL-SRA	EW	EW-SF-SRA	EW-FA-SRA	EW-FA-SL-SRA	EW-FA	EW-FA-SF-SRA	WSDOT	EW 2"	EW 2.5"
7-day Strength	4228	5691	4337	4792	3892	3369	2921	3739	6199	3887	3566
28-day Strength	4989	6947	4556	5582	4515	5461	3466	4234	7226	4400	4248

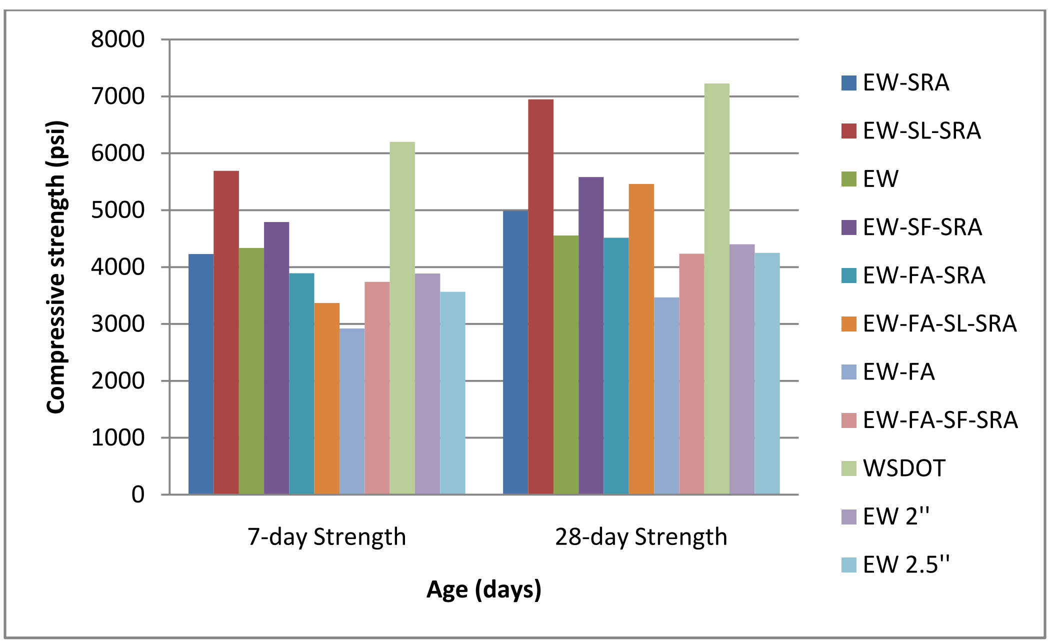
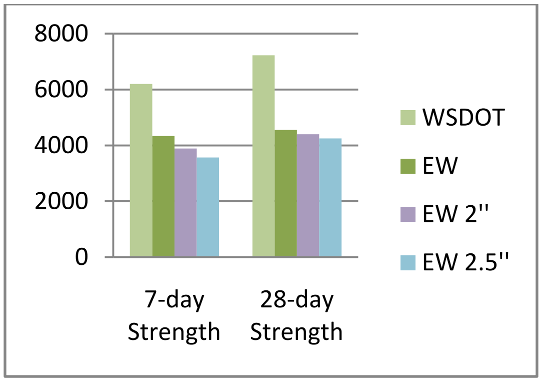
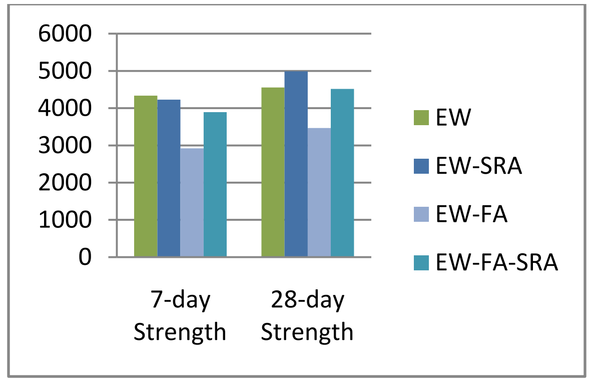


Fig. 5.1 Compressive Strength of Concrete Mixes with Eastern Washington Aggregate





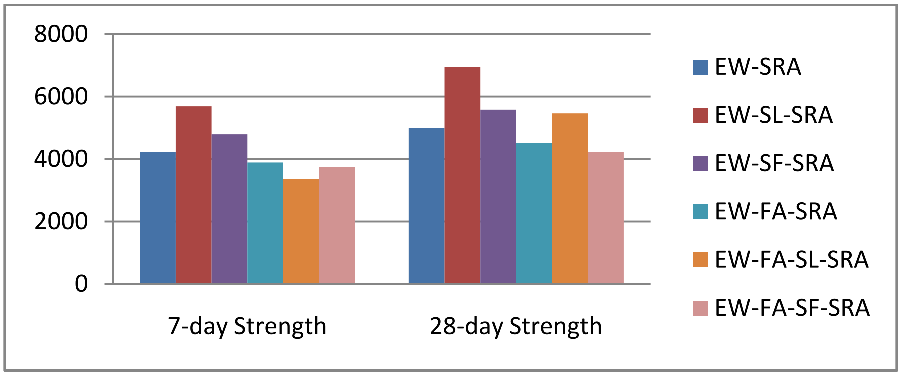


Fig. 5.2 Compressive Strength Comparison of Concrete Mixes with Eastern Washington Aggregate

The current WSDOT concrete mix design has the highest compressive strength at all times. With the increase of nominal aggregate size and the reduced paste contents in concrete mix designs, the compressive strengths of concrete decrease. The minimum WSDOT compressive strength requirement is 4,000 psi at 28 days. All these four concrete mix designs satisfy the minimum strength requirement. The replacement of cement by slag or silica fume increases the compressive strength of concrete. 20% replacement of cement by slag increases the 28-day strength of concrete from 4,989 psi to 6,947 psi for concrete mixes with SRA. 16% replacement of cement by using 4% silica fume increases the 28-day compressive strength from 4,989 psi to 5,582 psi for concrete mix with SRA. For concrete using cement only and without any other cementitious materials, the addition of SRA does not seem to change the compressive strength much. However, when 20% of cement is replaced by FA, the concrete strength for concrete mix without SRA decreases significantly, with the 28-day compressive strength decreasing from 4,556 psi to 3,466 psi, which is more than 20 percent of decrease. However, when SRA is added, the 28-day compressive strength of concrete mix using 20% replacement of cement by FA reduces only slightly. SRA increases the 28-day compressive strength of FA concrete from 3,466 psi to 4,515 psi, which is greater than the WSDOT minimum requirements of 4,000 psi. The combinations of SL + FA and SF + FA exhibit the combined effects for the compressive strength than when SL, SF, FA are applied separately.

5.3.2 Modulus of Elasticity Test

The modulus of elasticity test follows ASTM C469 “Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression”. The modulus of elasticity at 7 days and 28 days are tested for all the concrete mixes. The test data is listed in Table 5.3 and graphically shown in Fig. 5.3.

Table 5.3 Modulus of Elasticity Test Data (x10^6 psi)

Mixtures	EW-SRA	EW-SL-SRA	EW	EW-SF-SRA	EW-FA-SRA	EW-FA-SL-SRA	EW-FA	EW-FA-SF-SRA	WSDOT	EW 2"	EW 2.5"
7-day Modulus	4.15	5.00	3.75	3.75	4.20	3.70	3.20	4.15	4.55	3.70	4.00
28-day Modulus	4.75	5.15	3.85	3.85	4.50	4.40	3.60	4.90	4.75	4.10	4.10

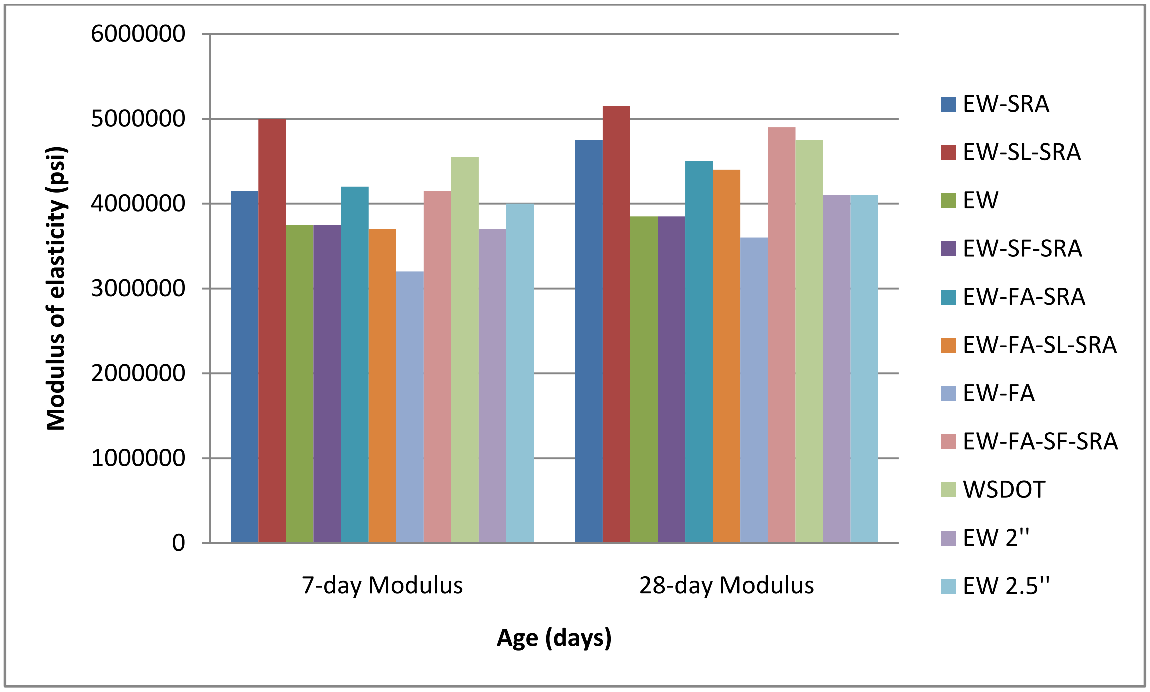


Fig. 5.3 Modulus of Elasticity of Concrete Mixes with Eastern Washington Aggregate

As for compressive strength, EW-SL-SRA has very high modulus of elasticity. The replacement of cement by slag increases both the compressive strength and the modulus of elasticity of concrete. The inclusion of SRA increases the modulus of elasticity of both EW and EW-FA. EW-FA has the lowest modulus, just as it has the lowest compressive strength. EW has a lower modulus than that of the control WSDOT mix. However, EW 2.0 and EW 2.5 have slightly higher modulus than EW.

5.3.3 Flexural Strength Test

The flexural strength test follows AASHTO T 97/ASTM C 78 “Standard Method of Test for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)”. In order to get the flexural strength development, the flexural strength tests are conducted at 3, 7, 14, and 28 days for all the concrete mixes. The test data is listed in Table 5.4 and also graphically in Figs. 5.4 and 5.5.

Table 5.4 Flexural Strength Test Data (psi)

Mixtures	EW-SRA	EW-SL-SRA	EW	EW-SF-SRA	EW-FA-SRA	EW-FA-SL-SRA	EW-FA	EW-FA-SF-SRA	WSDOT	EW 2"	EW 2.5"
3-day	699	748	575	638	574	436	549	544	867	636	523
7-day	793	863	709	673	713	517	602	678	939	633	612
14-day	955	961	740	858	789	681	563	763	1032	687	696
28-day	823	944	709	820	760	762	634	844	1070	700	546

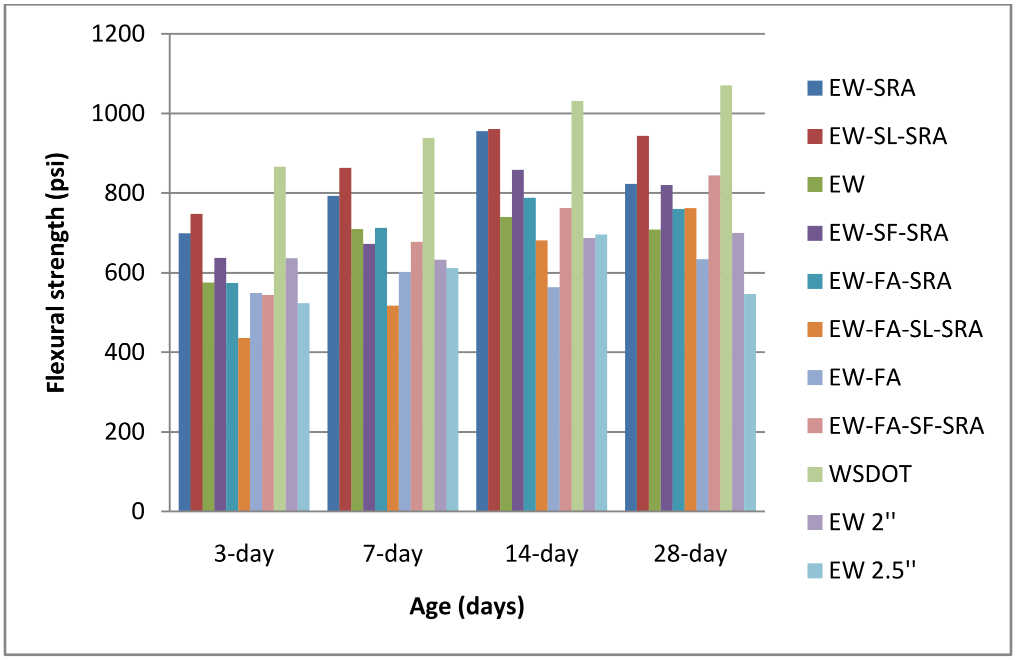


Fig. 5.4 Flexural Strength of Concrete Mixes with Eastern Washington Aggregate

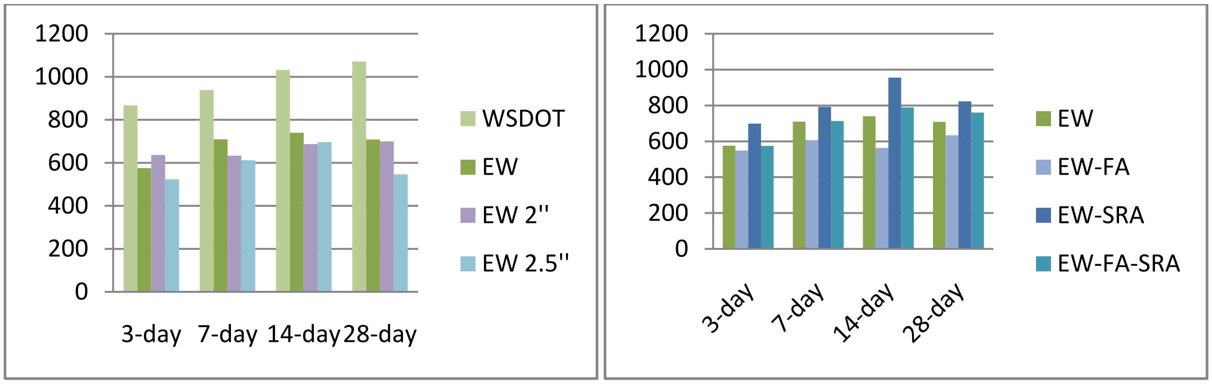


Fig. 5.5 Flexural Strength Comparison of Concrete Mixes with Eastern Washington Aggregate

For the early-age shrinkage cracking, the early-age flexural strength plays an important role. For the mix designs of WSDOT, EW, EW 2'' and EW 2.5'', the trends of flexural strength are similar to the compressive strength except that the flexural strength of EW is higher than that of WSDOT at 3 days. By comparing two pairs of the mixes with and without SRA, i.e., EW-SRA with EW and EW-FA-SRA with EW-FA, the data show that when SRA is used, the early-age flexural strength increases. SRA not only increases the early-age flexural strength, but also later strength. For the 28-day strength, the addition of SRA increases the flexural strength of mix EW by 16% and increases the flexural strength of EW-FA by 20%. The replacement of cement by FA decreases the flexural strength of concrete at all ages, both for the EW-FA mix and for the EW-FA-SRA mix. The replacement of cement by slag increases the flexural strength of concrete at all ages for EW-SL-SRA. However, when combined with fly ash, the flexural strength decreases at all ages for EW-FA-SL-SRA when compared with EW-SRA. The replacement of cement by silica fume also decreases the flexural strength of EW-SF-SRA when compared with EW-SRA. This is probably caused by the low paste content considered in all the mix designs, since there is not a strong bond between the paste and aggregates. On the other hand, the low paste content can reduce the shrinkage tendency as demonstrated in the shrinkage property tests.

5.4 Shrinkage Property Tests

Two tests on shrinkage properties of all the concrete mixes are conducted: free shrinkage and restrained shrinkage. The free shrinkage test shows the basic shrinkage property of concrete without any restraint; while the restrained shrinkage test illustrates the combination of concrete tensile strength and shrinkage properties and relatively mimics the condition of concrete deck being restrained by girders.

5.4.1 Free Shrinkage Test

Free shrinkage test follows AASHTO T 160 (ASTM C 157) “Length Change of Hardened Hydraulic Cement Mortar and Concrete”. Free shrinkage data are collected at 1, 2, 3, 4, 5, 6, 7, 14, 21, and 28 days, respectively, from which the free shrinkage tendency diagrams are drawn for all concrete mixes. According to a recent WSDOT bridge deck construction regulation, the free shrinkage at 28 days should be less than 320 microstrains. The free shrinkage data is listed in Table 5.5, and their tendency diagrams are shown in Figs. 5.6 to 5.8.

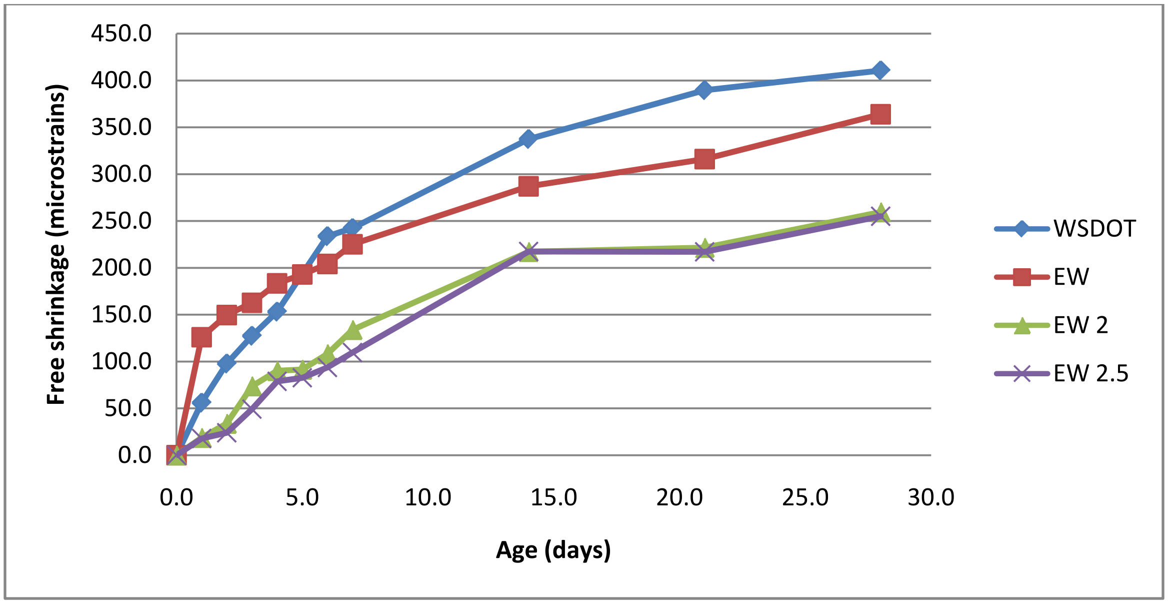


Fig. 5.6 Free Shrinkage of WSDOT, EW, EW2, EW2.5

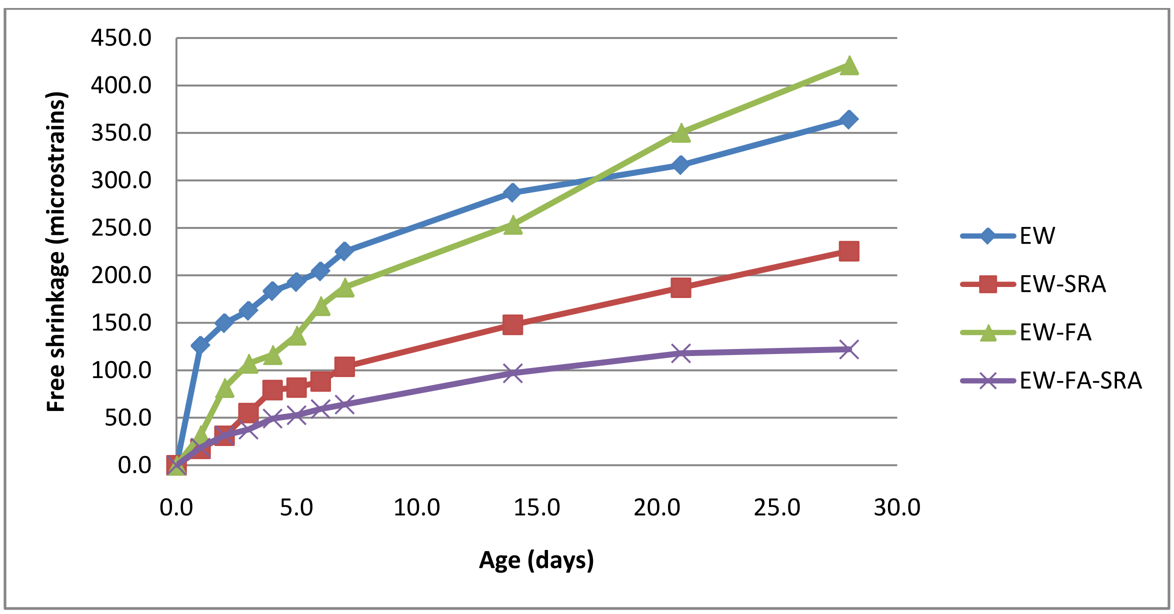


Fig. 5.7 Free Shrinkage of EW, EW-SRA, EW-FA and EW-FA-SRA

Table 5.5 Free Shrinkage Test Data (microstrain)

Mixtures Days	EW-SRA	EW-SL-SRA	EW	EW-SF-SRA	EW-FA-SRA	EW-FA-SL-SRA	EW-FA	EW-FA-SF-SRA	WSDOT	EW 2"	EW 2.5"
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.0	17.4	40.0	125.9	-20.7	18.7	17.5	32.0	2.6	56.0	18.7	17.8
2.0	31.1	57.2	149.6	-16.0	31.4	31.1	81.8	16.2	97.8	33.8	24.0
3.0	55.1	68.7	162.7	-5.9	37.6	47.4	107.3	31.2	127.4	73.8	49.2
4.0	79.3	88.3	183.4	-4.1	49.2	57.2	116.4	46.7	153.2	90.1	78.8
5.0	81.9	n/a	192.9	3.9	52.7	67.3	136.9	54.5	n/a	91.6	82.7
6.0	88.3	129.2	204.1	13.9	59.3	84.4	168.0	55.7	233.8	108.1	93.6
7.0	103.8	134.8	225.2	21.0	64.0	93.3	187.6	65.5	242.4	133.9	109.6
14.0	147.9	175.4	287.0	69.0	96.9	132.1	253.6	102.4	337.5	217.2	217.5
21.0	187.0	189.6	316.1	97.2	117.9	157.0	350.5	137.6	389.6	221.6	217.2
28.0	225.5	210.2	363.9	119.4	122.1	185.8	421.6	156.8	410.7	259.6	255.1

The influence of SRA on the free shrinkage is shown in Fig. 5.6. For EW and EW-FA, no SRA is added, and their free shrinkages at 28 days are all more than 320 microstrains. For EW and EW-FA, the replacement of cement by fly ash reduces the early age free shrinkage, especially in the first 14 days. However, it increases later, making the 28-day free shrinkage larger than the one without fly ash. When SRA is used, the free shrinkages of both of EW-SRA and EW-FA-SRA are reduced considerably, especially for EW-FA-SRA. For EW and EW-FA, the addition of SRA reduces their 28-day free shrinkage by 38% and 71%, respectively. The combination of fly ash and SRA has a greatest effect, reducing the free shrinkage value of that mix to 122.1 microstrains, which is far below the WSDOT limit of 320 microstrains.

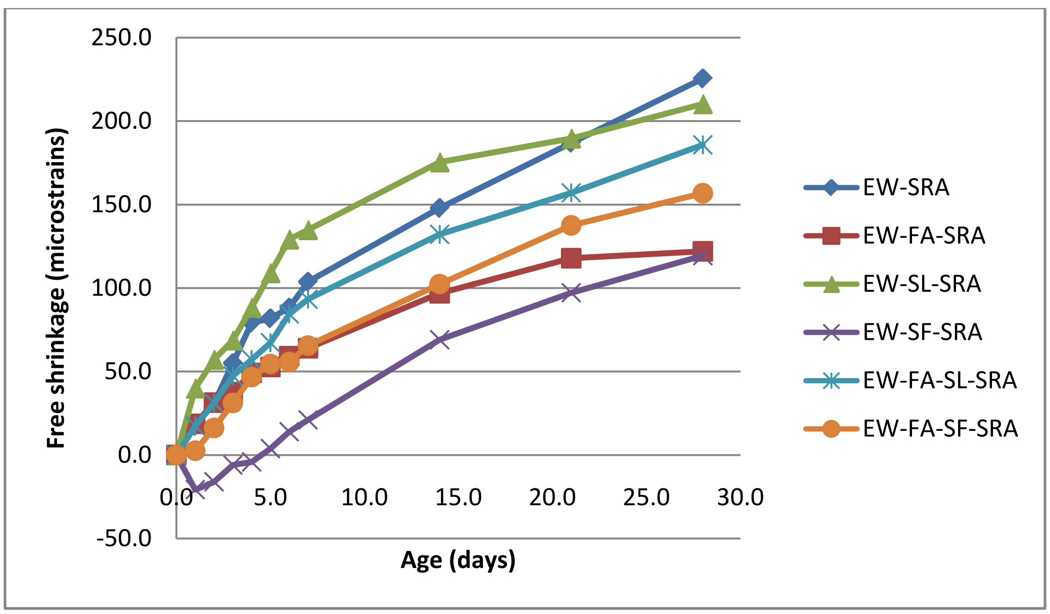


Fig. 5.8 Free Shrinkage of Eastern Washington Concrete Mixes with SRA

When SRA is used, the mix EW-SRA without using any cementitious materials except cement has the largest 28-day free shrinkage. For EW-SF-SRA, the free shrinkage values are even minus values for the first 4 days, which means that the concrete beam sample expanded during the first 4 days. The samples of EW-SRA shrink later, and they have the smallest 28-day free shrinkage of all the eight concrete mixes. The replacement of cement by slag increases early-age free shrinkage but eventually reduces the 28-day free shrinkage. Both the combination of fly ash with slag and fly ash with silica fume decrease the free shrinkage when compared with the one only with cement.

5.4.2 Restrained Shrinkage Test

The restrained shrinkage test follows AASHTO PP 34-99 “Cracking Tendency Using a Ring Specimen”. The restrained ring test data is listed in Table 5.6.

Table 5.6 Restrained Ring Test Data (Days of cracking)

Mixtures	EW-SRA	EW-SL-SRA	EW	EW-SF-SRA	EW-FA-SRA	EW-FA-SL-SRA	EW-FA	EW-FA-SF-SRA	WSDOT	EW 2"	EW 2.5"
6" Ring No.1	no crack	no crack	13.1	no crack	no crack	no crack	4.8	no crack	8.0	12.1	14.5
6" Ring No.2	no crack	no crack	17.6	no crack	no crack	no crack	7.8	no crack	11.6	8.9	28.0
3" Ring	no crack	25.9	10.9	no crack	n/a	no crack	3.3	no crack	n/a	n/a	n/a

Of the four concrete mixes WSDOT, EW, EW 2’’, EW 2.5’’, the ring specimens of WSDOT concrete mix crack the earliest, though it has the highest flexural (tensile) strength of all the four. The WSDOT concrete mix design has the largest paste content and very large free shrinkage. From this mix, it can be seen that the tensile strength is not the most critical factor in preventing early-age shrinkage cracking. None of the 6 in. ring specimens containing SRA cracked within 28 days, even for the concrete mixes that have low flexural and compressive strengths. No shrinkage cracking of the concrete ring is closely associated with the low free shrinkage of all the concrete mixes using SRA. The cracking of a ring specimen is the mutual effects of concrete free shrinkage and concrete flexural (tensile) strength. When the free shrinkage values are low, the induced tensile stresses on specimens in the ring are low. For EW and EW-FA, the free shrinkages are large, and the flexural strengths are low, leading to the cracking of the rings early or within 28 days. Although the early-age free shrinkage of EW-FA is smaller than EW, its flexural strength is smaller than EW, and consequently, EW-FA cracks earlier than EW does.

5.5 Concluding Remarks

From a material point of view, the cracking potential of concrete is the combined effects of its mechanical property (i.e., primarily its tensile strength) and its shrinkage property (i.e., shrinkage strain). The concrete mix design with a high flexural (tensile) strength and low free shrinkage has the best cracking resistance. The current WSDOT concrete mix design with eastern Washington aggregate has very high flexural strength. However, its free shrinkage is also very large. As a result, its ring specimens cracked early and before 28 days. For the concrete mix design EW in this research using aggregates of nominal size 1.5 in., the paste volumes is reduced when compared with the current WSDOT mix. Although its strength is correspondingly reduced a lot when compared with the WSDOT design, it cracks later since it has a small early-age free shrinkage. The replacement of cement by slag and by silica fume increases the strength of concrete, while the replacement of cement by fly ash decreases the early-age strength of concrete significantly. The ring specimens of EW-FA crack earlier than those of EW. When SRA is added to the concrete mixes, the free shrinkage for all the mixes is dramatically reduced. None of 6 in. tall ring of the mix designs containing SRA cracks within 28 days. SRA is highly recommended to be used in concrete applications to mitigate the early-age cracking in concrete bridge decks. Trial batches are recommended to be cast and tested for quality control purpose before any field application.

CHAPTER SIX

PERFORMANCE OF MIX DESIGNS WITH WESTERN WASHINGTON AGGREGATES

6.1 Introduction

This chapter reports on test results for concrete cast using coarse aggregates from western Washington. All the materials and the experimental procedures are the same as those introduced in Chapter 5 for the eastern Washington aggregate, except that all the coarse aggregates used are from western Washington.

6.2 Fresh Property Tests

As for eastern Washington aggregates, the slump test and air content test are conducted as quality control of concrete fresh properties. The slump test follows ASTM C 143/AASHTO T 119 “Slump of hydraulic cement concrete and for air content test”. Two methods are considered. The pressure method follows AASHTO T 152/ASTM C 231 “Air Content of Freshly-mixed Concrete by the Pressure Method”, and the volumetric method follows AASHTO T 196/ASTM C 173 “Air Content of Freshly-Mixed Concrete by the Volumetric Method”.

The slump test results are from 3.25 in. to 6.5 in., and the air content test results are from 4.5% to 10% (see Table 6.1). Because several chemicals are added to the concrete mix, it is difficult to achieve the preferable air content range of 6.5% to 9.5% for some concrete mixes with Western Washington aggregate. The test results are listed in Table 6.1.

Table 6.1 Slump and Air Content Data

Mixtures	WW-SRA	WW-SL-SRA	WW	WW-SF-SRA	WW-FA-SRA	WW-FA-SL-SRA	WW-FA	WW-FA-SF-SRA	LD-WSDOT	WW 2"	WW 2.5"
Slump (in.)	4.2	3.8	3.6	5	3.25	6.5	3.8	5.3	4	3.3	5.8
Air Content (%)	4.8	5.5	8.8	6.2	5	4.5	8	7.1	4.8	9.8	10

6.3 Mechanical Property Tests

The compressive strength, modulus of elasticity, and flexural strength are tested for each concrete mix design.

6.3.1 Compressive Strength Test

ASTM C 39/AASHTO T 22 "Compressive Strength of Cylindrical Concrete Specimens" is followed for the compressive test of concrete cylinder specimens. The compressive strength test ensures that the designed/tested concrete mix designs meet the minimum WSDOT 28-day compressive strength requirements of 4,000 psi. The test results are presented in Table 6.2 and graphically in Figs. 6.1 and 6.2.

Table 6.2 Compressive Strength Test Data (psi)

Mixtures	WW-SRA	WW-SL-SRA	WW	WW-SF-SRA	WW-FA-SRA	WW-FA-SL-SRA	WW-FA	WW-FA-SF-SRA	LD-WSDOT	WW 2	WW 2.5
7-day Strength	4971	4356	4766	6591	4175	3779	3221	3809	3461	6002	6003
28-day Strength	6322	5651	5652	7725	5310	5060	3966	5263	4432	6578	6485

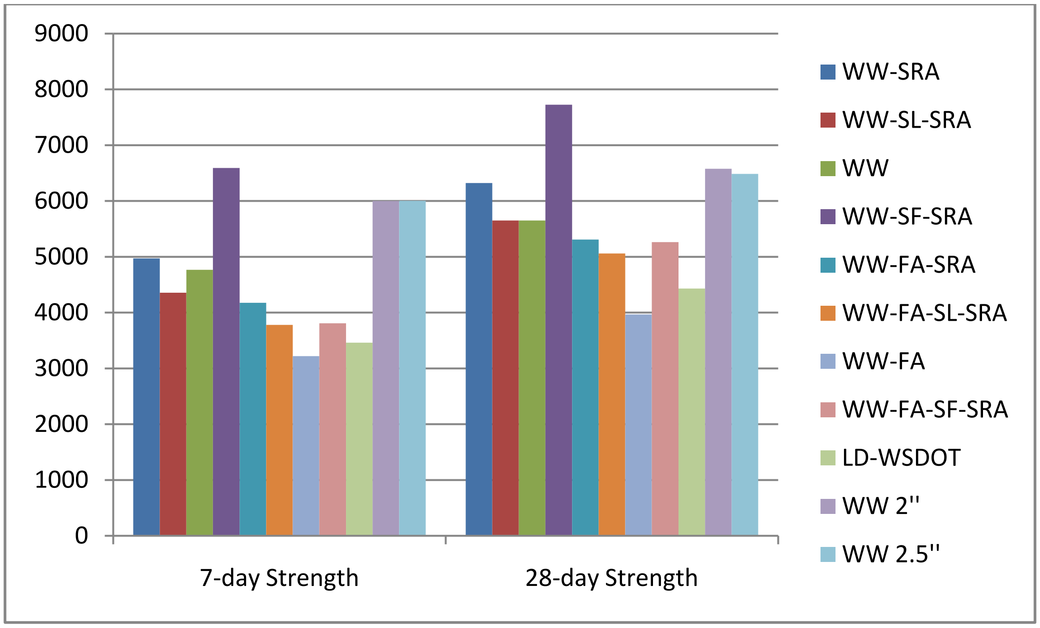


Fig. 6.1 Compressive Strength of Western Washington Concrete Mixes (psi)

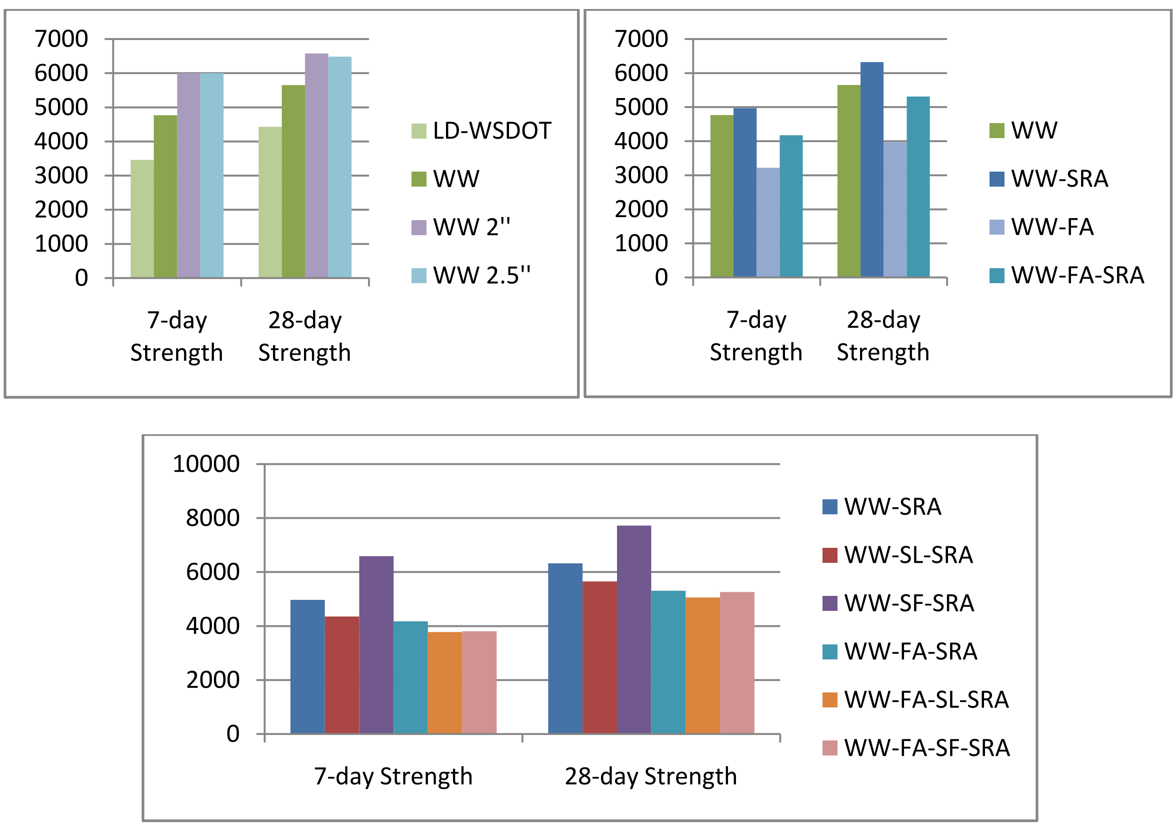


Fig. 6.2 Compressive Strength Comparisons

The comparisons among LD-WSDOT, WW, WW2'', and WW 2.5'' show that LD-WSDOT had the lowest compressive strength at all ages. LD-WSDOT used the w/c ratio of 0.48, and its paste volume is 25.8%. Its high w/c ratio leads to its low strength. The nominal size 2'' aggregates for WW 2'' include both the larger size aggregates and are well graded. The aggregates for WW 2'' and WW 2.5'' are similar except for that WW 2.5''has around 8% more larger size aggregates. Well graded aggregates require less paste to achieve good workability and thus have better bonding between aggregates and cement paste. Therefore, the compressive strength of WW 2'' and WW 2.5'' are similar, and both are larger than that of WW.

The addition of SRA increases the 28-day compressive strength of both WW and WW-FA, which is consistent with the observation for the eastern Washington aggregate data. Also, the replacement of cement using FA reduces the strength of concrete both with and without the addition of SRA. WW-FA has the lowest compressive strength of all the mixes, and its compressive strength is below the WSDOT minimum requirement of 4,000 psi at 28 days.

The mix design WW-SF, i.e., the replacement of cement by silica fume, has the highest compressive strength at all ages. The replacements of cement by other cementitious materials, such as fly ash, slag, and the combination of two cementitous materials, are all lower than that of WW.

6.3.2 Modulus of Elasticity Test

ASTM C469 “Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression” is followed for modulus of elasticity test of cylinder specimens. The test results are presented in Table 6.3 and shown graphically in Fig. 6.3.

Table 6.3 Modulus of Elasticity Test Data (x10^6 psi)

Mixtures	WW-SRA	WW-SL-SRA	WW	WW-SF-SRA	WW-FA-SRA	WW-FA-SL-SRA	WW-FA	WW-FA-SF-SRA	LD-WSDOT	WW 2"	WW 2.5"
7-day Modulus	4.15	3.7	4.55	5.1	4	4	3.6	4	2.95	4.85	4.45
28-day Modulus	4.65	3.95	4.7	5.35	4.8	4.3	3.95	4.45	3.4	5.15	5.25

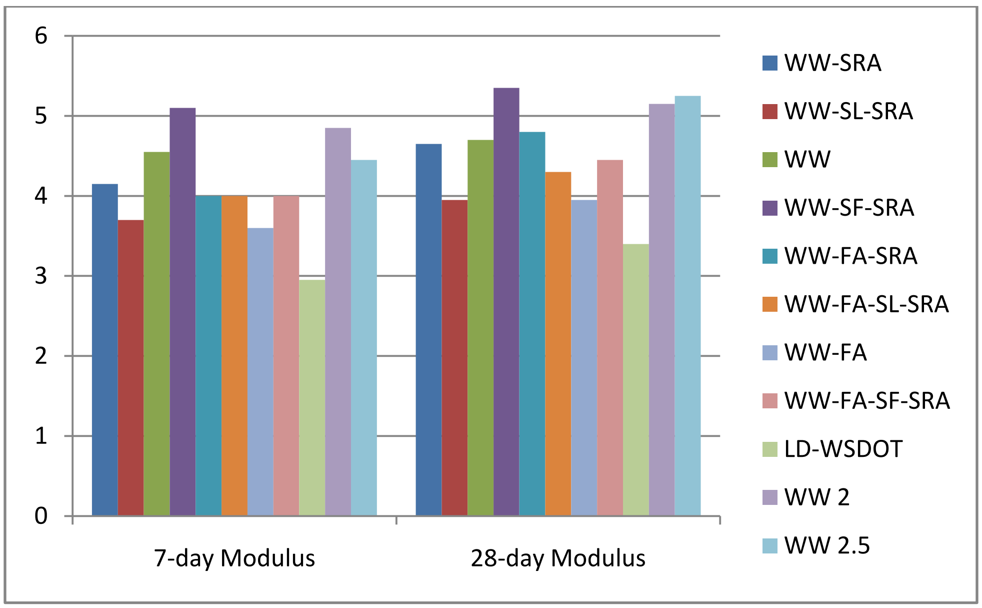


Fig. 6.3 Modulus of Elasticity of Western Concrete Mixes

The replacement of cement by silica fume has the highest modulus among all these western mixes. The LD-WSDOT has the lowest. WW 2 and WW 2.5 have a higher modulus than that of WW, which is consistent with the compressive strength comparison. Unlike in eastern concrete mixes, the replacement of cement by slag decreases the modulus of original concrete mix.

6.3.3 Flexural Strength Test

AASHTO T 97/ASTM C 78 “Standard Method of Test for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)” is followed for the flexural strength test of beam specimens. The flexural strength of all the concrete mix designs at 3, 7, 14, and 28 days are tested. The test results are shown in Table 6.4 and also graphically in Figs. 6.4 and 6.5.

Table 6.4 Flexural Strength Test Data (psi)

Mixtures	WW-SRA	WW-SL-SRA	WW	WW-SF-SRA	WW-FA-SRA	WW-FA-SL-SRA	WW-FA	WW-FA-SF-SRA	LD-WSDOT	WW 2"	WW 2.5"
3-day	646	579	630	784	508	445	449	451	412	833	900
7-day	851	754	725	887	625	576	562	589	499	749	869
14-day	903	788	814	919	713	688	607	688	594	825	954
28-day	915	753	762	1022	744	759	577	831	748	932	921

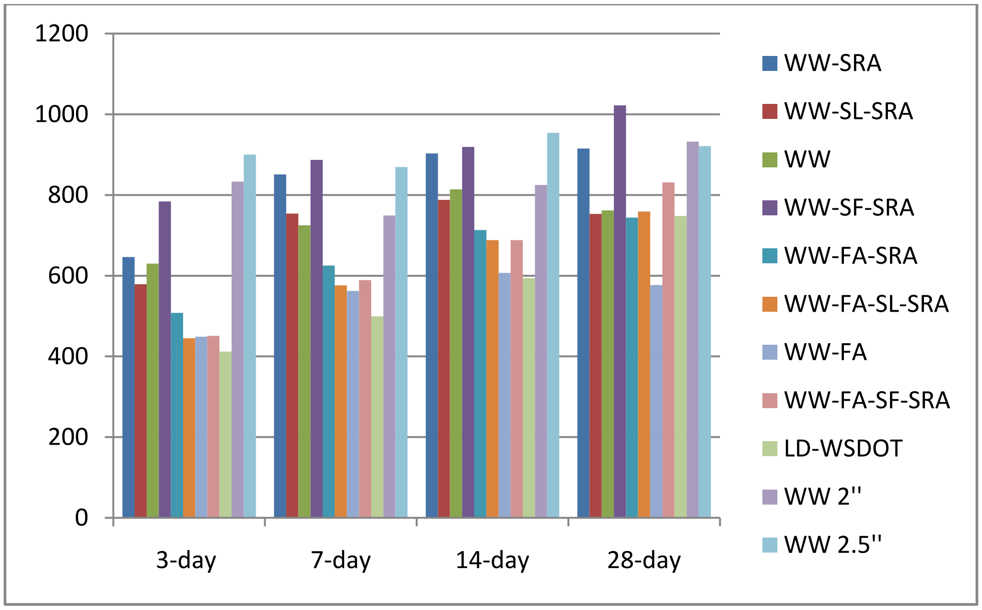
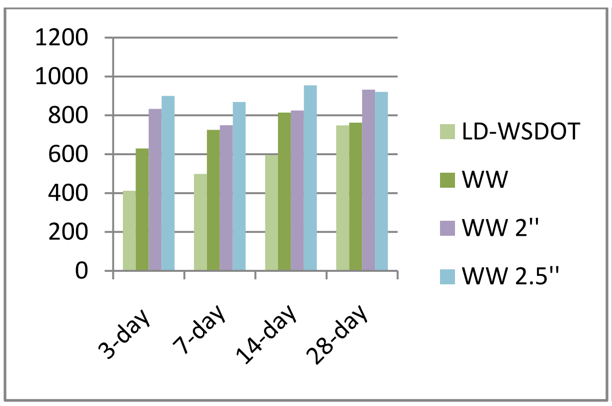
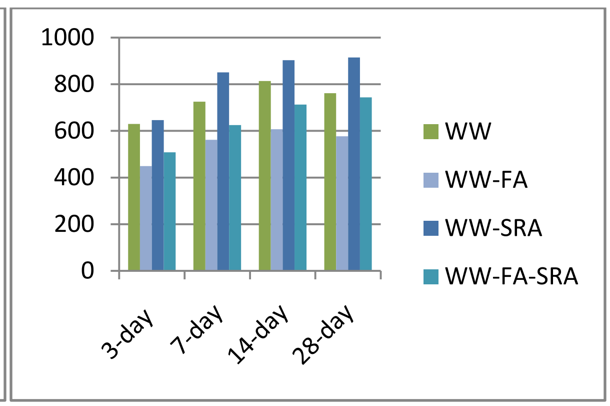


Fig. 6.4 Flexural Strength of Western Concrete Mixes





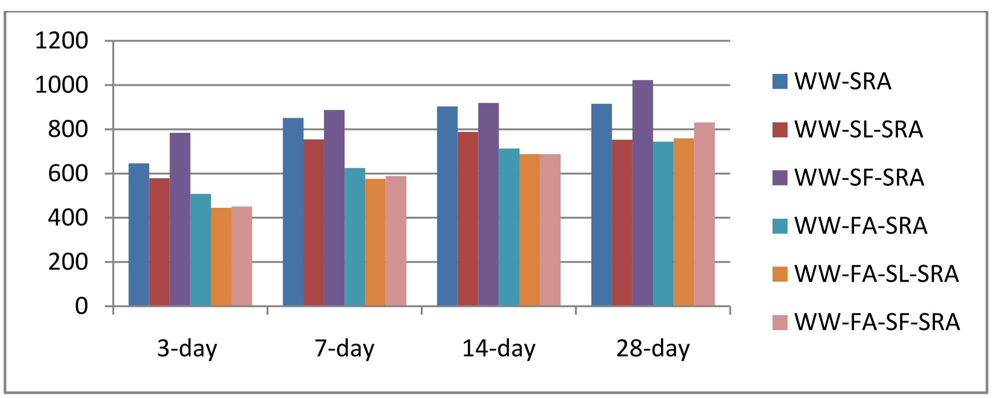


Fig. 6.5 Flexural Strength Comparison

Among the four concrete mix designs of LD-WSDOT, WW, WW 2'', and WW 2.5'', the flexural strength trend is similar to that of the compressive strength. LD-WSDOT has the lowest flexural strength, and WW 2'' and WW 2.5'' have the highest. Also, the addition of SRA increases the flexural strength of both WW and WW-FA. And the replacement of cement by fly ash reduces the flexural strength at all ages. Among those mix designs containing SRA, the replacement of cement by silica fume has the highest flexural strength at all ages. The replacements of cement by other cementitious materials, such as fly ash, slag, or the combination of different cementitious materials, all have smaller flexural strengths than that of WW-SRA.

6.4 Shrinkage Property Tests

Similarly to the tests performed for the mix designs with the eastern Washington aggregate, the free shrinkage and restrained shrinkage tests are conducted for all concrete mixes using the western coarse aggregates.

6.4.1 Free Shrinkage Test

AASHTO T 160 (ASTM C 157) "Length Change of Hardened Hydraulic Cement Mortar and Concrete" is followed for the free shrinkage test using beam specimens. The same test methods are adopted as in Chapter 5 for the eastern Washington aggregates. The free shrinkage test results are presented in Table 6.5 and graphically shown in Figs. 6.6 and 6.7.

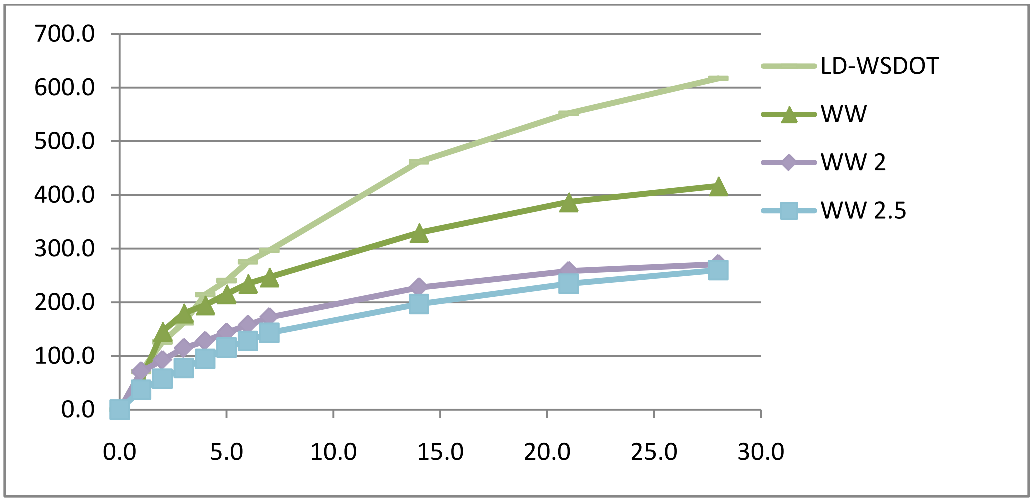


Fig. 6.6 Free Shrinkage of LD-WSDOT, WW, WW2, WW2.5

In Fig. 6.6, the free shrinkages of concrete mix designs without SRA and without any replacement of cement with other cementitious materials are compared. LD-WSDOT has a w/c ratio of 0.48, and it has the largest paste content of the four concrete mixes, leading to the highest free shrinkage. WW has the second largest value of free shrinkage. Both of these two have free shrinkage larger than 320 microstrains at 28-day. The WSDOT required the maximum free shrinkage value to be less than 320 microstrains at 28-day. Therefore, LD-WSDOT and WW do not satisfy the WSDOT requirement of 320 microstrains. The free shrinkage values for WW 2 and WW 2.5 both meet the WSDOT requirements.

Table 6.5 Free Shrinkage Test Data (microstrain)

Mixtures Days	WW-SRA	WW-SL-SRA	WW	WW-SF-SRA	WW-FA-SRA	WW-FA-SL-SRA	WW-FA	WW-FA-SF-SRA	LD-WSDOT	WW 2"	WW 2.5"
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.0	17.2	5.0	53.9	-0.9	2.7	27.3	24.3	1.8	70.8	71.1	37.0
2.0	46.2	31.4	145.5	9.2	5.9	38.2	44.0	2.4	126.2	92.1	57.5
3.0	56.6	52.4	179.6	13.6	9.2	53.0	75.7	5.0	161.8	114.7	77.6
4.0	65.5	61.9	194.7	87.4	87.7	62.5	87.9	7.4	214.2	127.7	94.5
5.0	74.4	63.7	215.4	94.8	96.0	65.8	98.8	9.2	240.3	143.4	115.6
6.0	98.1	69.9	234.7	101.9	100.1	74.1	120.1	14.5	275.3	157.9	128.0
7.0	105.8	88.9	246.8	102.2	104.6	83.3	129.0	19.6	296.6	172.1	143.1
14.0	154.4	107.9	329.5	118.8	117.3	122.7	196.0	30.2	461.6	227.6	196.7
21.0	182.8	163.0	386.7	160.0	156.4	146.7	240.1	111.7	551.8	258.2	234.5
28.0	196.8	188.1	416.6	155.6	160.3	176.9	264.4	129.8	616.9	271.1	259.9

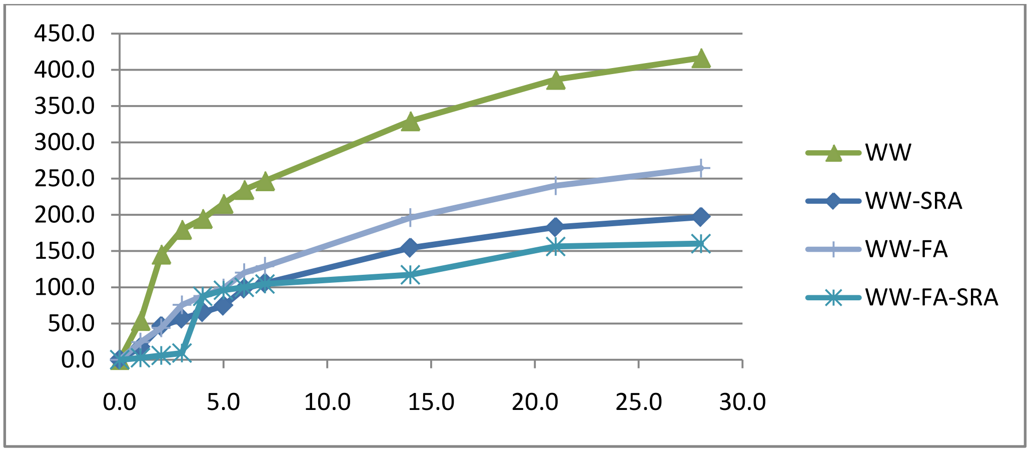


Fig. 6.7 Free Shrinkage of WW, WW-SRA, WW-FA and WW-FA-SRA

Fig. 6.7 shows that when fly ash is used to replace cement, the free shrinkage decreases, both for WW-FA and WW-FA-SRA. Especially, when no SRA is added, fly ash replacement of cement reduces the 28-day free shrinkage of WW by 37%. When SRA is added, the free shrinkage of WW decreases by 53%, from 416.6 microstrains to 196.8 microstrains at 28 days. The free shrinkage of WW-FA decreases by 39%, from 264.4 microstrains to 160.3 microstrains at 28 days.

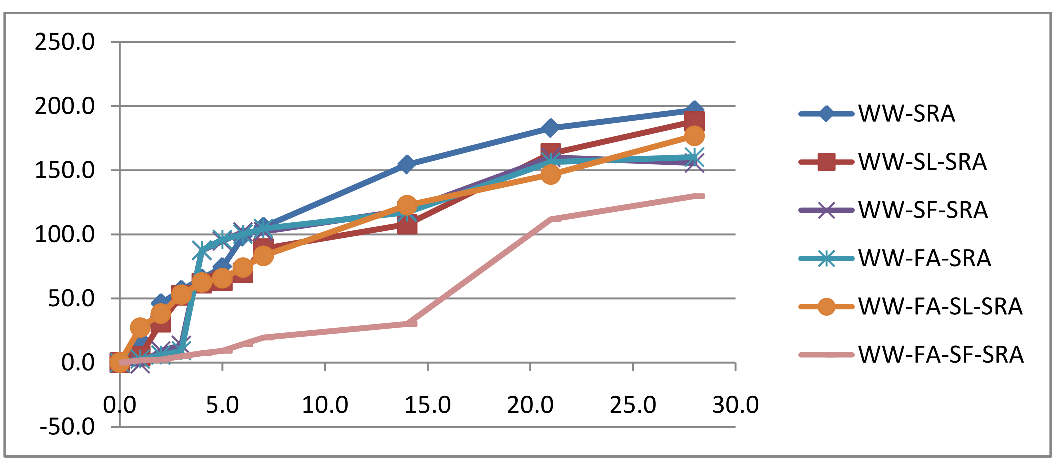


Fig.6.8 Free Shrinkage of Western Concrete Mixes with SRA

Fig. 6.8 shows the free shrinkage of concrete mixes when SRA is included in the mix. All the free shrinkage values are less than 200 microstrains at 28 days. The replacement of cement by other cementitous materials further reduces the free shrinkage of concrete mixes with SRA.

6.4.2 Restrained Shrinkage Test

AASHTO PP 34-99 Cracking Tendency Using a Ring Specimen is followed for the restrained shrinkage test. The test results are presented in Table 6.6.

Table 6.6 Restrained Ring Test Data (cracking days)

Mixtures Days	WW-SRA	WW-SL-SRA	WW	WW-SF-SRA	WW-FA-SRA	WW-FA-SL-SRA	WW-FA	WW-FA-SF-SRA	LD-WSDOT	WW 2"	WW 2.5"
6" Ring No.1	no crack	no crack	9.4	no crack	no crack	no crack	7.8	no crack	6.7	9.7	10.9
6" Ring No.2	no crack	no crack	13.0	no crack	no crack	no crack	6.3	no crack	8.5	15.4	no crack
3" Ring	20.6	no crack	3.7	no crack	no crack	no crack	5.3	no crack	5.9	n/a	n/a

Of the four mixes (LD-WSDOT, WW, WW 2, and WW 2.5), LD-WSDOT cracks the earliest. LD-WSDOT has the smallest flexural strength at all ages. And its free shrinkage values are always the highest. WW 2 seems to crack a little later than WW, while WW 2.5 has the best cracking resistance of four mix designs. Be aware that the nominal size of aggregates used in WW 2 and WW 2.5 are already closed to the concrete ring thickness in the ring test. Although WW-FA has smaller free shrinkage than that of WW at all ages, both of the 6 in. tall ring specimens of WW-FA crack earlier than those of the WW concrete mix.

This is caused by the low flexural strength of WW-FA. None of the 6 in. rings for all the concrete mixes with SRA addition cracked within 28 days.

6.5 Concluding Remarks

In this chapter, both the fresh concrete, mechanical, and shrinkage properties of mix designs with western Washington aggregate are characterized. The use of SRA significantly reduces the free shrinkage of all concrete mixes. At the same time, the flexural and compressive strength values are larger than those without SRA. The combined effects of the improved strength properties and free shrinkage allow the concrete mixes with SRA to have a high shrinkage cracking resistance. Fly ash replacement of cement decreases the strength of concrete a lot, making the concrete with fly ash more shrinkage cracking vulnerable. Although WW 2 has low free shrinkage and high flexural strength, the ring still cracks within 28 days. However, due to the limitation of ring test apparatus in this research, more research on larger size aggregates is recommended. SRA is again recommended to be used to mitigate early-age cracking problems in bridge deck applications.

CHATPER SEVEN

CONCLUSIONS AND RECOMMENDATIONS

The goal of this study is to find the mitigations strategies for early-age shrinkage cracking in concrete bridge decks. A comprehensive literature is first conducted. Based on the literature and also the recommendations from the WSDOT, 20 concrete mixes are designed. Two current WSDOT concrete mixes are also included as benchmarks for comparisons with other newly developed concrete mix designs. Fresh properties, hardened properties, and shrinkage properties are evaluated for all the 22 groups of concrete mixes. In this chapter, the conclusions and recommendations are presented.

7.1 Conclusions

Based on the experimental evaluation of different mix designs conducted in this study, the following conclusions are obtained.

1. The use of SRA significantly reduces the free shrinkage of all concrete mixes using aggregates from Washington State. It also decreases the restrained shrinkage cracking tendency of all concrete mixes. The laboratory test data show that none of the 6 in. tall concrete ring specimens in the restrained ring test with inclusion of SRA crack within 28 days.
2. The replacement of cement by fly ash decreases the strength of concrete. In the concrete mixes with both the eastern Washington aggregates and western aggregates, concrete containing fly ash cracks earlier than the corresponding concrete without fly ash.

3. Paste volume plays an important role in the free shrinkage of concrete. Concrete mixes with a small paste volume have small tendency of shrinkage cracking. The use of larger size aggregates reduces the paste volume in concrete mix. From the control concrete mixes to concrete using the nominal size of 2.5 in. aggregates, less paste volume are used. Free shrinkage became smaller, and the ring specimens delay the cracking.
4. When SRA is added, the replacements of cement by fly ash, silica fume, and slag further reduce the free shrinkage of concrete. However, they play a less important role than SRA.
5. Concrete cracking resistance is the combined effects of both its flexural (tensile) strength and its free shrinkage property. The concrete mix that has an acceptable tensile strength and low free shrinkage strain is anticipated to have relatively good cracking resistance.
6. ADVA 190 high-range water-reducing admixtures have a great effect on adjusting the workability of concrete. It is able to change the slump test value from almost zero to a high value to achieve the desired workability.
7. When several chemicals are used in one concrete mix, it is difficult to achieve the desired fresh concrete properties, such as air content.
8. Both the size of coarse aggregates and the source of coarse aggregates played a very important role in the property of concrete. As the size of coarse aggregates increases, both the free shrinkage and restrained shrinkage properties are improved.

7.2 Recommendations

Based on the comprehensive experimental program conducted in this study, the following recommendations are suggested for improved concrete design to mitigate the shrinkage cracking in concrete.

1. SRA is recommended to be used in concrete mix to mitigate early-age shrinkage cracking in concrete bridge decks. However, trial batches are recommended to be conducted first before any field applications.
2. Fly ash is not recommended to be used due to its low early-age strength.
3. Concrete designs with less paste volume are recommended to be used to increase the cracking resistance.
4. Large sizes of coarse aggregates are recommended in construction.
5. When several cementitous materials and chemical admixtures are used in the same concrete mix, trial batches are recommended to be evaluated before field applications.

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APPENDIX

Appendix A. Mechanical Test Data

The primary goal of this study is to evaluate the current WSDOT concrete mix designs in bridge deck applications for their shrinkage cracking resistance and to develop and evaluate new concrete mix designs to mitigate early-age shrinkage cracking in concrete bridge decks. With the main focus on shrinkage properties, mechanical properties are also evaluated in parallel to shrinkage properties.

Table A.1 shows the test data for compressive strength of all the concrete mixes evaluated in this study. In most of the concrete mixes, the compressive strengths of the two cylinders sampled from the same concrete mix and tested at the same age are very close to each other. Some of the cylinders sampled from the same concrete mix and tested at the same age show slightly differences due to the sample variations and concrete mix properties.

Table A.2 shows the test data for two beams sampled from the same concrete mix and tested at the same age for flexural strength of the evaluated concrete mix. As in compressive cylinders, some of the data show a relatively large variation of flexural strengths. However, they can still indicate the effects of different concrete mixes on the flexural strength of concrete.

Table A.1 Compressive Test Data for Two Cylinders

Mixes	7 days		28 days	
	Cylinder 1	Cylinder 2	Cylinder 1	Cylinder 2
EW-SRA	3963	4493	5039	4940
EW-SL-SRA	5634	5747	7297	6598
EW	4512	4163	4556	n/a
EW-SF-SRA	4789	4795	5481	5684
EW-FA-SRA	3870	3915	4391	4640
EW-FA-SL-SRA	3319	3419	5713	5210
EW-FA	3074	2767	3450	3481
EW-FA-SF-SRA	3816	3662	4234	5264
WW-SRA	5111	4830	6328	6316
WW-SL-SRA	4414	4297	5379	5924
WW	4787	4744	5681	5622
WW-SF-SRA	6652	6530	7721	7729
WW-FA-SRA	4150	4199	5298	5322
WW-FA-SL-SRA	3637	3921	5248	4872
WW-FA	3232	3211	4081	3850
WW-FA-SF-SRA	3812	3806	5347	5180
LD-WSDOT	3366	3556	4632	4838
WSDOT	6170	6227	7072	7381
EE 2	4230	3544	4584	4216
EE 2.5	3687	3446	4316	4180
WW2	6033	5972	6247	6910
WW2.5	6030	5975	6327	6644

Table A.2 Flexural Test Data for Two Beams

Mixes	3 days		7 days		14 days		28 days	
	Beam 1	Beam 2	Beam 1	Beam 2	Beam 1	Beam 2	Beam 1	Beam 2
EW-SRA	766	631	829	757	809	1102	782	864
EW-SL-SRA	785	711	920	806	1011	910	1034	854
EW	519	632	673	746	784	696	709	708
EW-SF-SRA	678	598	736	609	823	893	800	839
EW-FA-SRA	629	519	741	685	758	819	802	718
EW-FA-SL-SRA	438	435	523	511	748	614	723	801
EW-FA	549	549	620	584	583	543	664	603
EW-FA-SF-SRA	474	614	715	640	725	800	843	846
WW-SRA	672	620	828	874	914	891	868	962
WW-SL-SRA	583	574	729	779	842	734	687	818
WW	657	604	758	693	859	769	790	735
WW-SF-SRA	765	804	874	900	974	865	1040	1003
WW-FA-SRA	530	486	640	610	714	711	731	757
WW-FA-SL-SRA	448	442	562	590	689	687	753	765
WW-FA	518	381	611	513	557	657	567	586
WW-FA-SF-SRA	459	442	589	589	714	661	907	755
LD-WSDOT	437	387	478	520	524	663	752	743
WSDOT	878	856	871	1006	1033	1030	1039	1101
EE2	664	609	695	571	679	695	787	613
EE2.5	575	471	607	616	694	697	568	524
WW2	831	834	768	731	778	871	898	967
WW2.5	908	893	878	860	968	940	986	857

Appendix B. Concrete Mix Designs by KU Mix Program

CONCRETE MIX DESIGN
Compressive Strength: 4000D

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 100%	550 lb	3.15	2.80
SL--GGBFS / 123 / 0%	0 lb	2.89	
FA--Fly Ash Type-F / 123 / 0%	0 lb	2.04	
SF--Silica Fume / 123 / 0%	0 lb	2.18	
Water	207 lb	1.00	3.32
new-#4 / #4 / 37.5%	1161 lb	2.70	6.89
new-#8 / #8 / 38%	1177 lb	2.68	7.04
New-Class 2 sand / sand / 24.5%	759 lb	2.65	4.59
Total Air, percent	8%		2.16
Daravair® 1000 Air Entrai / Grace Construction	38.6 fl oz (US)	1.02	0.04
Eclipse Plus SRA / Grace Construction	129.8 fl oz (US)	0.96	0.14
Adva 190 HWRA / Grace Construction	28.6 fl oz (US)	1.10	0.03
[1]The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.			27.01

Total Water Content (including water in admixtures), lb			220	
Water / Cementitious Material Ratio:			0.4	
Concrete Unit Weight, pcf			143.2	
Target Slump, in.			BLANK	
Paste Content, percent			23.43%	
Workability Factor (WF)	Target:	34.9	Actual:	34.9
Coarseness Factor (CF)	Target:	60.9	Actual:	60.9

Fig. B.1 Concrete Mix Design for EW-SRA

CONCRETE MIX DESIGN
Compressive Strength: 4000D

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 80%	440 lb	3.15	2.24
SL--GGBFS / 123 / 20%	110 lb	2.89	0.61
FA--Fly Ash Type-F / 123 / 0%	0 lb	2.04	
SF--Silica Fume / 123 / 0%	0 lb	2.18	
Water	207 lb	1.00	3.32
new-#4 / #4 / 37.6%	1160 lb	2.70	6.89
new-#8 / #8 / 38.2%	1179 lb	2.68	7.05
New-Class 2 sand / sand / 24.2%	748 lb	2.65	4.52
Total Air, percent	8%		2.16
Daravair® 1000 Air Entrai / Grace Construction	33.4 fl oz (US)	1.02	0.04
Eclipse Plus SRA / Grace Construction	129.8 fl oz (US)	0.96	0.14
Adva 190 HWRA / Grace Construction	35.2 fl oz (US)	1.10	0.04
[1]The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.			27.00

Total Water Content (including water in admixtures), lb			220	
Water / Cementitious Material Ratio:			0.4	
Concrete Unit Weight, pcf			142.9	
Target Slump, in.			BLANK	
Paste Content, percent			23.61%	
Workability Factor (WF)	Target:	34.9	Actual:	34.9
Coarseness Factor (CF)	Target:	60.9	Actual:	60.9

Fig. B.2 Concrete Mix Design for EW-SL-SRA

CONCRETE MIX DESIGN
Compressive Strength: 4000D

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 100%	550 lb	3.15	2.80
SL--GGBFS / 123 / 0%	0 lb	2.89	
FA--Fly Ash Type-F / 123 / 0%	0 lb	2.04	
SF--Silica Fume / 123 / 0%	0 lb	2.18	
Water	218 lb	1.00	3.49
new-#4 / #4 / 37.5%	1160 lb	2.70	6.89
new-#8 / #8 / 38%	1177 lb	2.68	7.04
New-Class 2 sand / sand / 24.5%	759 lb	2.65	4.59
Total Air, percent	8%		2.16
Daravair® 1000 Air Entrai / Grace Construction	14.5 fl oz (US)	1.02	0.02
Eclipse Plus SRA / Grace Construction	0 fl oz (US)	0.96	0.00
Adva 190 HRWRA / Grace Construction	17.4 fl oz (US)	1.10	0.02
			27.01

[1]The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.

Total Water Content (including water in admixtures), lb			220	
Water / Cementitious Material Ratio:			0.4	
Concrete Unit Weight, pcf			143.2	
Target Slump, in.			BLANK	
Paste Content, percent			23.43%	
Workability Factor (WF)	Target:	34.9	Actual:	34.9
Coarseness Factor (CF)	Target:	60.9	Actual:	60.9

Fig. B.3 Concrete Mix Design for EW

CONCRETE MIX DESIGN
Compressive Strength: 4000D

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 95.5%	462 lb	3.15	2.35
SL--GGBFS / 123 / 0%	0 lb	2.89	
FA--Fly Ash Type-F / 123 / 0%	0 lb	2.04	
SF--Silica Fume / 123 / 4.5%	22 lb	2.18	0.16
Water	182 lb	1.00	2.92
new-#4 / #4 / 36%	1154 lb	2.70	6.85
new-#8 / #8 / 30.2%	969 lb	2.68	5.79
New-Class 2 sand / sand / 33.9%	1087 lb	2.65	6.57
Total Air, percent	8%		2.16
Daravair® 1000 Air Entrai / Grace Construction	26 fl oz (US)	1.02	0.03
Eclipse Plus SRA / Grace Construction	129.8 fl oz (US)	0.96	0.14
Adva 190 HWRA / Grace Construction	26.7 fl oz (US)	1.10	0.03
[1]The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.			26.99

Total Water Content (including water in admixtures), lb			194	
Water / Cementitious Material Ratio:			0.4	
Concrete Unit Weight, pcf			144.0	
Target Slump, in.			BLANK	
Paste Content, percent			20.79%	
Workability Factor (WF)	Target:	34.8	Actual:	39.4
Coarseness Factor (CF)	Target:	61.1	Actual:	64.8

Fig. B.4 Concrete Mix Design for EW-SF-SRA

CONCRETE MIX DESIGN
Compressive Strength: 4000D

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 80%	440 lb	3.15	2.24
SL--GGBFS / 123 / 0%	0 lb	2.89	
FA--Fly Ash Type-F / 123 / 20%	110 lb	2.04	0.86
SF--Silica Fume / 123 / 0%	0 lb	2.18	
Water	204 lb	1.00	3.27
new-#4 / #4 / 38%	1160 lb	2.70	6.89
new-#8 / #8 / 39.1%	1192 lb	2.68	7.13
New-Class 2 sand / sand / 22.9%	697 lb	2.65	4.22
Total Air, percent	8%		2.16
Daravair® 1000 Air Entrai / Grace Construction	29.4 fl oz (US)	1.02	0.03
Eclipse Plus SRA / Grace Construction	204 fl oz (US)	0.96	0.21
Adva 190 HWRA / Grace Construction	17.1 fl oz (US)	1.10	0.02
[1]The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.			27.03

Total Water Content (including water in admixtures), lb			220	
Water / Cementitious Material Ratio:			0.4	
Concrete Unit Weight, pcf			141.4	
Target Slump, in.			BLANK	
Paste Content, percent			24.54%	
Workability Factor (WF)	Target:	34.9	Actual:	34.9
Coarseness Factor (CF)	Target:	60.7	Actual:	60.7

Fig. B.5 Concrete Mix Design for EW-FA-SRA

CONCRETE MIX DESIGN
Compressive Strength: 4000D

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 60%	330 lb	3.15	1.68
SL--GGBFS / 123 / 20%	110 lb	2.89	0.61
FA--Fly Ash Type-F / 123 / 20%	110 lb	2.04	0.86
SF--Silica Fume / 123 / 0%	0 lb	2.18	
Water	207 lb	1.00	3.32
new-#4 / #4 / 38.2%	1159 lb	2.70	6.88
new-#8 / #8 / 39.3%	1193 lb	2.68	7.13
New-Class 2 sand / sand / 22.6%	686 lb	2.65	4.15
Total Air, percent	8%		2.16
Daravair® 1000 Air Entrai / Grace Construction	33.4 fl oz (US)	1.02	0.04
Eclipse Plus SRA / Grace Construction	129.8 fl oz (US)	0.96	0.14
Adva 190 HWRA / Grace Construction	35.2 fl oz (US)	1.10	0.04
[1]The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.			27.00

Total Water Content (including water in admixtures), lb			220	
Water / Cementitious Material Ratio:			0.4	
Concrete Unit Weight, pcf			141.0	
Target Slump, in.			BLANK	
Paste Content, percent			24.72%	
Workability Factor (WF)	Target:	34.9	Actual:	34.9
Coarseness Factor (CF)	Target:	60.6	Actual:	60.6

Fig. B.6 Concrete Mix Design for EW-FA-SL-SRA

CONCRETE MIX DESIGN
Compressive Strength: 4000D

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 80%	440 lb	3.15	2.24
SL--GGBFS / 123 / 0%	0 lb	2.89	
FA--Fly Ash Type-F / 123 / 20%	110 lb	2.04	0.86
SF--Silica Fume / 123 / 0%	0 lb	2.18	
Water	218 lb	1.00	3.49
new-#4 / #4 / 38%	1159 lb	2.70	6.88
new-#8 / #8 / 39.1%	1191 lb	2.68	7.12
New-Class 2 sand / sand / 22.8%	696 lb	2.65	4.21
Total Air, percent	8%		2.16
Daravair® 1000 Air Entrai / Grace Construction	14.5 fl oz (US)	1.02	0.02
Eclipse Plus SRA / Grace Construction	0 fl oz (US)	0.96	0.00
Adva 190 HWRA / Grace Construction	17.5 fl oz (US)	1.10	0.02
			27.00

[1]The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.

Total Water Content (including water in admixtures), lb			220	
Water / Cementitious Material Ratio:			0.4	
Concrete Unit Weight, pcf			141.3	
Target Slump, in.			BLANK	
Paste Content, percent			24.54%	
Workability Factor (WF)	Target:	34.9	Actual:	34.9
Coarseness Factor (CF)	Target:	60.7	Actual:	60.7

Fig. B.7 Concrete Mix Design for EW-FA

CONCRETE MIX DESIGN
Compressive Strength: 4000D

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 72.7%	352 lb	3.15	1.79
SL--GGBFS / 123 / 0%	0 lb	2.89	
FA--Fly Ash Type-F / 123 / 22.7%	110 lb	2.04	0.86
SF--Silica Fume / 123 / 4.5%	22 lb	2.18	0.16
Water	181 lb	1.00	2.90
new-#4 / #4 / 37.5%	1188 lb	2.70	7.05
new-#8 / #8 / 38.1%	1206 lb	2.68	7.21
New-Class 2 sand / sand / 24.4%	773 lb	2.65	4.67
Total Air, percent	8%		2.16
Daravair® 1000 Air Entrai / Grace Construction	33.4 fl oz (US)	1.02	0.04
Eclipse Plus SRA / Grace Construction	129.8 fl oz (US)	0.96	0.14
Adva 190 HWRA / Grace Construction	26 fl oz (US)	1.10	0.03
[1]The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.			27.00

Total Water Content (including water in admixtures), lb			194	
Water / Cementitious Material Ratio:			0.4	
Concrete Unit Weight, pcf			142.4	
Target Slump, in.			BLANK	
Paste Content, percent			21.90%	
Workability Factor (WF)	Target:	34.9	Actual:	34.9
Coarseness Factor (CF)	Target:	60.9	Actual:	60.9

Fig. B.8 Concrete Mix Design for EW-FA-SF-SRA

CONCRETE MIX DESIGN
Compressive Strength: 4000D

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 95.5%	462 lb	3.15	2.35
SL--GGBFS / 123 / 0%	0 lb	2.89	
FA--Fly Ash Type-F / 123 / 0%	0 lb	2.04	
SF--Silica Fume / 123 / 4.5%	22 lb	2.18	0.16
Water	183 lb	1.00	2.93
WW-new-#4 / #4 / 32.5%	1044 lb	2.70	6.20
WW-new-#8 / #8 / 25.6%	822 lb	2.68	4.92
New-Class 2 sand / sand / 41.8%	1342 lb	2.65	8.12
Total Air, percent	8%		2.16
Daravair® 1000 Air Entrai / Grace Construction	14.5 fl oz (US)	1.02	0.02
Eclipse Plus SRA / Grace Construction	129.8 fl oz (US)	0.96	0.14
Adva 190 HWRA / Grace Construction	17.4 fl oz (US)	1.10	0.02
[1]The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.			27.01

Total Water Content (including water in admixtures), lb		194		
Water / Cementitious Material Ratio:		0.4		
Concrete Unit Weight, pcf		143.9		
Target Slump, in.		BLANK		
Paste Content, percent		20.79%		
Workability Factor (WF)	Target:	34.9	Actual:	34.9
Coarseness Factor (CF)	Target:	61.0	Actual:	61.0

Fig. B.9 Concrete Mix Design for WW-SRA

CONCRETE MIX DESIGN
Compressive Strength: 4000D

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 80%	440 lb	3.15	2.24
SL--GGBFS / 123 / 20%	110 lb	2.89	0.61
FA--Fly Ash Type-F / 123 / 0%	0 lb	2.04	
SF--Silica Fume / 123 / 0%	0 lb	2.18	
Water	210 lb	1.00	3.37
WW-new-#4 / #4 / 33%	1016 lb	2.70	6.03
WW-new-#8 / #8 / 26.4%	813 lb	2.68	4.86
New-Class 2 sand / sand / 40.6%	1250 lb	2.65	7.56
Total Air, percent	8%		2.16
Daravair® 1000 Air Entrai / Grace Construction	14.5 fl oz (US)	1.02	0.02
Eclipse Plus SRA / Grace Construction	129.8 fl oz (US)	0.96	0.14
Adva 190 HWRA / Grace Construction	17.4 fl oz (US)	1.10	0.02
			27.00

[1]The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.

Total Water Content (including water in admixtures), lb			220	
Water / Cementitious Material Ratio:			0.4	
Concrete Unit Weight, pcf			142.6	
Target Slump, in.			BLANK	
Paste Content, percent			23.61%	
Workability Factor (WF)	Target:	34.9	Actual:	34.9
Coarseness Factor (CF)	Target:	60.7	Actual:	60.7

Fig. B.10 Concrete Mix Design for WW-SL-SRA

CONCRETE MIX DESIGN
Compressive Strength: 4000D

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 100%	550 lb	3.15	2.80
SL--GGBFS / 123 / 0%	0 lb	2.89	
FA--Fly Ash Type-F / 123 / 0%	0 lb	2.04	
SF--Silica Fume / 123 / 0%	0 lb	2.18	
Water	218 lb	1.00	3.49
WW-new-#4 / #4 / 32.9%	1017 lb	2.70	6.04
WW-new-#8 / #8 / 26.3%	812 lb	2.68	4.86
New-Class 2 sand / sand / 40.8%	1260 lb	2.65	7.62
Total Air, percent	8%		2.16
Daravair® 1000 Air Entrai / Grace Construction	14.5 fl oz (US)	1.02	0.02
Eclipse Plus SRA / Grace Construction	0 fl oz (US)	0.96	0.00
Adva 190 HWRA / Grace Construction	17.4 fl oz (US)	1.10	0.02
			27.01

[1]The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.

Total Water Content (including water in admixtures), lb			220	
Water / Cementitious Material Ratio:			0.4	
Concrete Unit Weight, pcf			142.9	
Target Slump, in.			BLANK	
Paste Content, percent			23.43%	
Workability Factor (WF)	Target:	34.9	Actual:	34.9
Coarseness Factor (CF)	Target:	60.7	Actual:	60.7

Fig. B.11 Concrete Mix Design for WW

CONCRETE MIX DESIGN
Compressive Strength: 4000D

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 95.5%	462 lb	3.15	2.35
SL--GGBFS / 123 / 0%	0 lb	2.89	
FA--Fly Ash Type-F / 123 / 0%	0 lb	2.04	
SF--Silica Fume / 123 / 4.5%	22 lb	2.18	0.16
Water	183 lb	1.00	2.93
WW-new-#4 / #4 / 32.5%	1044 lb	2.70	6.20
WW-new-#8 / #8 / 25.6%	822 lb	2.68	4.92
New-Class 2 sand / sand / 41.8%	1342 lb	2.65	8.12
Total Air, percent	8%		2.16
Daravair® 1000 Air Entrai / Grace Construction	14.5 fl oz (US)	1.02	0.02
Eclipse Plus SRA / Grace Construction	129.8 fl oz (US)	0.96	0.14
Adva 190 HWRA / Grace Construction	17.4 fl oz (US)	1.10	0.02
			27.01

[1]The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.

Total Water Content (including water in admixtures), lb			194	
Water / Cementitious Material Ratio:			0.4	
Concrete Unit Weight, pcf			143.9	
Target Slump, in.			BLANK	
Paste Content, percent			20.79%	
Workability Factor (WF)	Target:	34.9	Actual:	34.9
Coarseness Factor (CF)	Target:	61.0	Actual:	61.0

Fig. B.12 Concrete Mix Design for WW-SF-SRA

CONCRETE MIX DESIGN
Compressive Strength: 4000D

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 80%	440 lb	3.15	2.24
SL--GGBFS / 123 / 0%	0 lb	2.89	
FA--Fly Ash Type-F / 123 / 20%	110 lb	2.04	0.86
SF--Silica Fume / 123 / 0%	0 lb	2.18	
Water	210 lb	1.00	3.37
WW-new-#4 / #4 / 33.4%	1014 lb	2.70	6.02
WW-new-#8 / #8 / 27%	821 lb	2.68	4.91
New-Class 2 sand / sand / 39.6%	1203 lb	2.65	7.28
Total Air, percent	8%		2.16
Daravair® 1000 Air Entrai / Grace Construction	14.5 fl oz (US)	1.02	0.02
Eclipse Plus SRA / Grace Construction	129.8 fl oz (US)	0.96	0.14
Adva 190 HWRA / Grace Construction	17.4 fl oz (US)	1.10	0.02
[1]The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.			27.01

Total Water Content (including water in admixtures), lb			220	
Water / Cementitious Material Ratio:			0.4	
Concrete Unit Weight, pcf			141.0	
Target Slump, in.			BLANK	
Paste Content, percent			24.54%	
Workability Factor (WF)	Target:	34.9	Actual:	34.9
Coarseness Factor (CF)	Target:	60.5	Actual:	60.5

Fig. B.13 Concrete Mix Design for WW-FA-SRA

CONCRETE MIX DESIGN
Compressive Strength: 4000D

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 60%	330 lb	3.15	1.68
SL--GGBFS / 123 / 20%	110 lb	2.89	0.61
FA--Fly Ash Type-F / 123 / 20%	110 lb	2.04	0.86
SF--Silica Fume / 123 / 0%	0 lb	2.18	
Water	210 lb	1.00	3.37
WW-new-#4 / #4 / 33.4%	1013 lb	2.70	6.01
WW-new-#8 / #8 / 27.1%	822 lb	2.68	4.92
New-Class 2 sand / sand / 39.4%	1194 lb	2.65	7.22
Total Air, percent	8%		2.16
Daravair® 1000 Air Entrai / Grace Construction	14.5 fl oz (US)	1.02	0.02
Eclipse Plus SRA / Grace Construction	129.8 fl oz (US)	0.96	0.14
Adva 190 HWRA / Grace Construction	17.4 fl oz (US)	1.10	0.02
[1]The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.			27.00

Total Water Content (including water in admixtures), lb		220		
Water / Cementitious Material Ratio:		0.4		
Concrete Unit Weight, pcf		140.7		
Target Slump, in.		BLANK		
Paste Content, percent		24.72%		
Workability Factor (WF)	Target:	34.9	Actual:	34.9
Coarseness Factor (CF)	Target:	60.4	Actual:	60.4

Fig. B.14 Concrete Mix Design for WW-FA-SL-SRA

CONCRETE MIX DESIGN
Compressive Strength: 4000D

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 80%	440 lb	3.15	2.24
SL--GGBFS / 123 / 0%	0 lb	2.89	
FA--Fly Ash Type-F / 123 / 20%	110 lb	2.04	0.86
SF--Silica Fume / 123 / 0%	0 lb	2.18	
Water	218 lb	1.00	3.49
WW-new-#4 / #4 / 33.4%	1014 lb	2.70	6.02
WW-new-#8 / #8 / 27%	821 lb	2.68	4.91
New-Class 2 sand / sand / 39.6%	1204 lb	2.65	7.28
Total Air, percent	8%		2.16
Daravair® 1000 Air Entrai / Grace Construction	14.5 fl oz (US)	1.02	0.02
Eclipse Plus SRA / Grace Construction	0 fl oz (US)	0.96	0.00
Adva 190 HWRA / Grace Construction	17.4 fl oz (US)	1.10	0.02
[1]The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.			27.00

Total Water Content (including water in admixtures), lb			220	
Water / Cementitious Material Ratio:			0.4	
Concrete Unit Weight, pcf			141.1	
Target Slump, in.			BLANK	
Paste Content, percent			24.54%	
Workability Factor (WF)	Target:	34.9	Actual:	34.9
Coarseness Factor (CF)	Target:	60.5	Actual:	60.5

Fig. B.15 Concrete Mix Design for WW-FA

CONCRETE MIX DESIGN
Compressive Strength: 4000D

Material / Source or Designation / Blend[1]	Quantity (SSD)	S.G.	Yield, ft^3
Type I/II Cement / Cement Producer / 72.7%	352 lb	3.15	1.79
SL--GGBFS / 123 / 0%	0 lb	2.89	
FA--Fly Ash Type-F / 123 / 22.7%	110 lb	2.04	0.86
SF--Silica Fume / 123 / 4.5%	22 lb	2.18	0.16
Water	183 lb	1.00	2.93
WW-new-#4 / #4 / 33%	1041 lb	2.70	6.18
WW-new-#8 / #8 / 26.3%	832 lb	2.68	4.98
New-Class 2 sand / sand / 40.7%	1286 lb	2.65	7.78
Total Air, percent	8%		2.16
Daravair® 1000 Air Entrai / Grace Construction	14.5 fl oz (US)	1.02	0.02
Eclipse Plus SRA / Grace Construction	129.8 fl oz (US)	0.96	0.14
Adva 190 HWRA / Grace Construction	17.4 fl oz (US)	1.10	0.02
			27.01

[1]The blend percentage indicated (by weight) is listed separately for cementitious materials and aggregates.

Total Water Content (including water in admixtures), lb		194		
Water / Cementitious Material Ratio:		0.4		
Concrete Unit Weight, pcf		142.1		
Target Slump, in.		BLANK		
Paste Content, percent		21.90%		
Workability Factor (WF)	Target:	34.9	Actual:	34.9
Coarseness Factor (CF)	Target:	60.7	Actual:	60.7

Fig. B.16 Concrete Mix Design for WW-FA-SF-SRA

Appendix C. Restrained Ring Strain Data

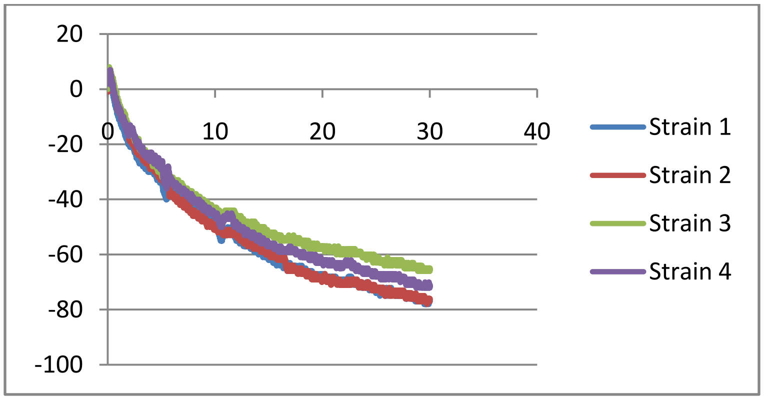

Fig. C.1a Ring Test for EW-SRA, 6 in. Ring A, from day 1

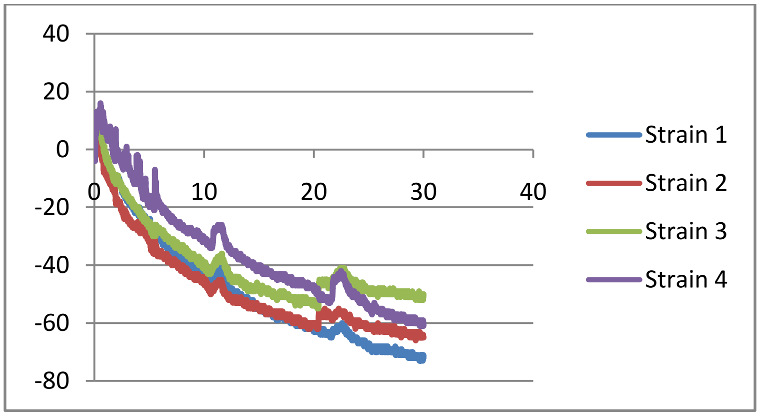

Fig. C.1b Ring Test for EW-SRA, 6 in. Ring B, from day 1

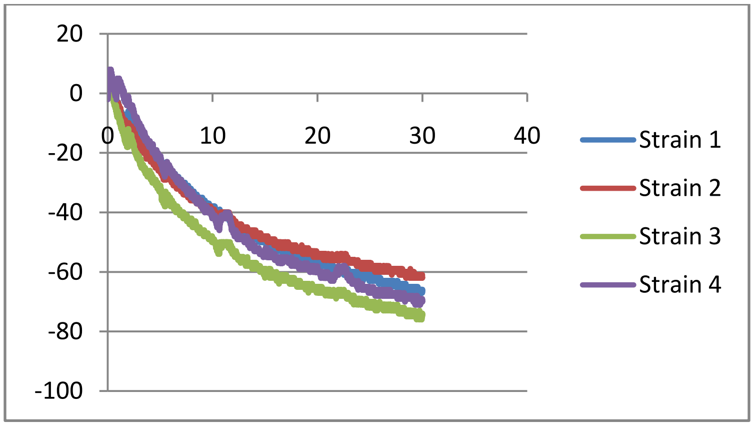

Fig. C.1c Ring Test for EW-SRA, 3 in. Ring, from day 1

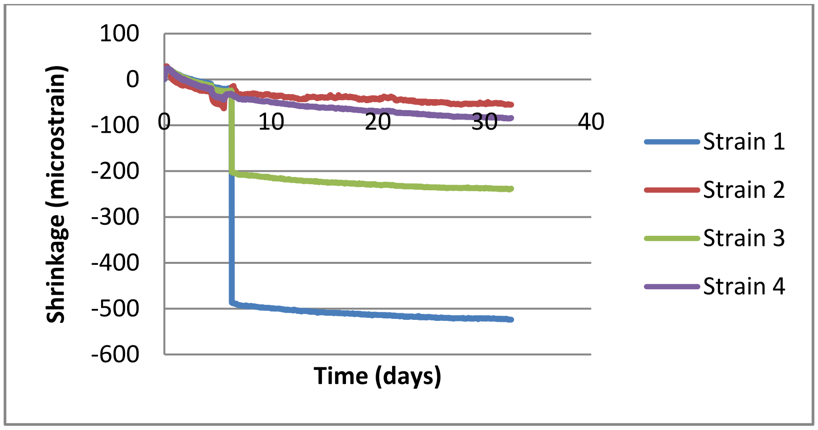

Fig. C.2a Ring Test for EW-SL-SRA, 6 in. Ring A, from day 1

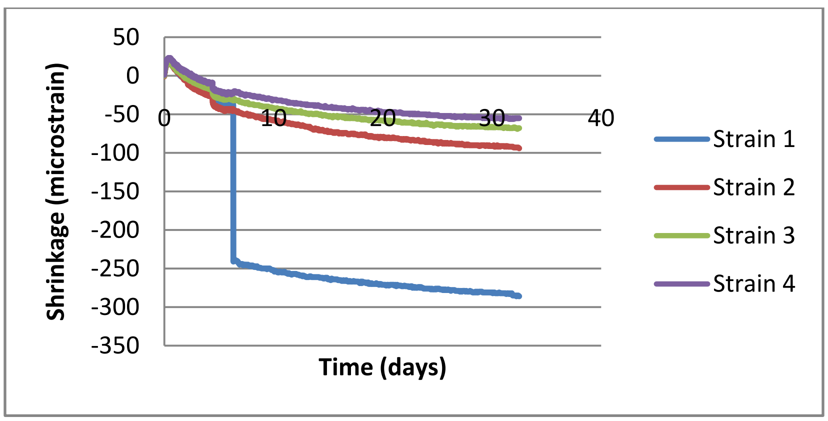

Fig. C.2b Ring Test for EW-SL-SRA, 6 in. Ring B, from day 1

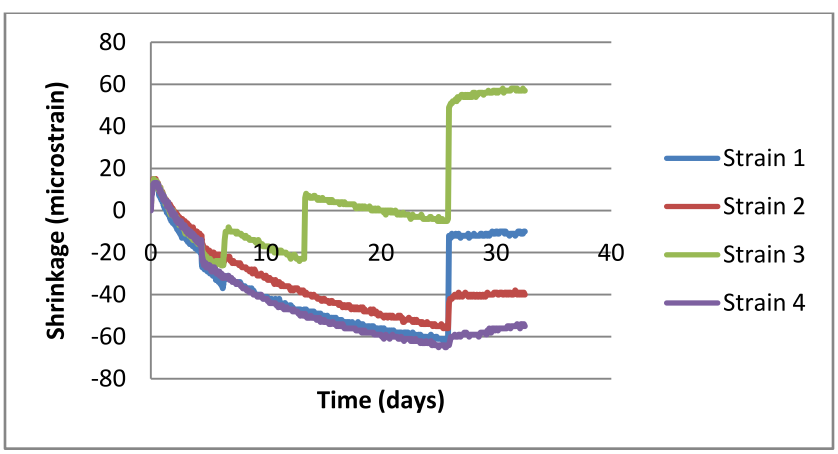

Fig. C.2c Ring Test for EW-SL-SRA, 3 in. Ring, from day 1

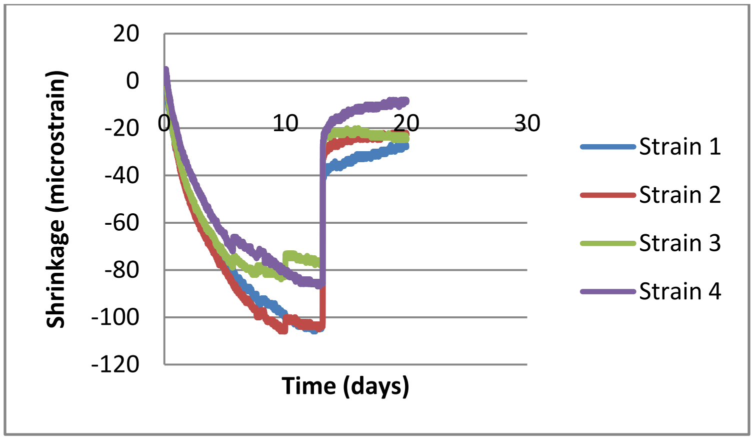

Fig. C.3a Ring Test for EW, 6 in. Ring A, from day 1

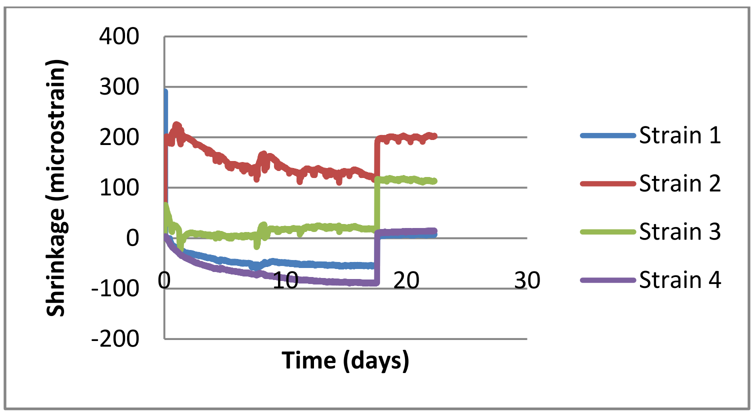

Fig. C.3b Ring Test for EW, 6 in. Ring B, from day 1

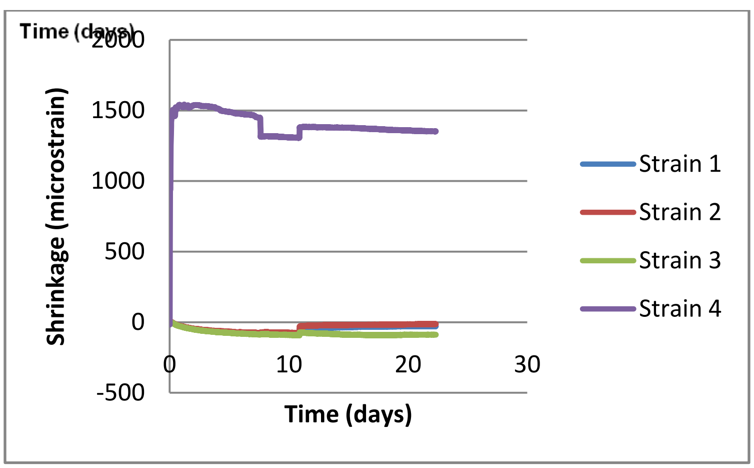

Fig. C.3c Ring Test for EW, 3 in. Ring, from day 1

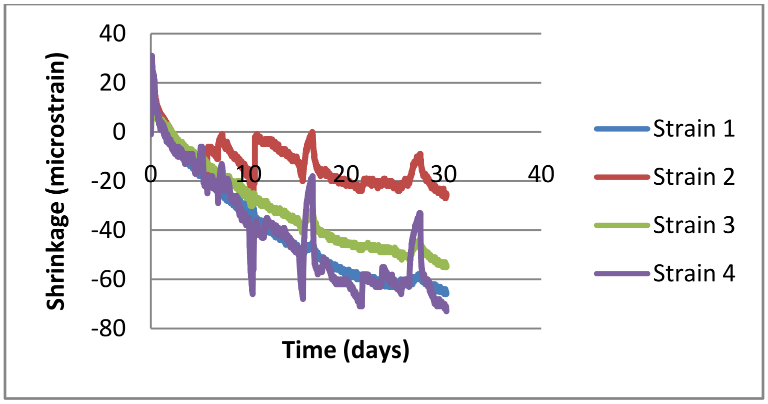

Fig. C.4a Ring Test for EW-SF-SRA, 6 in. Ring A, from day 1

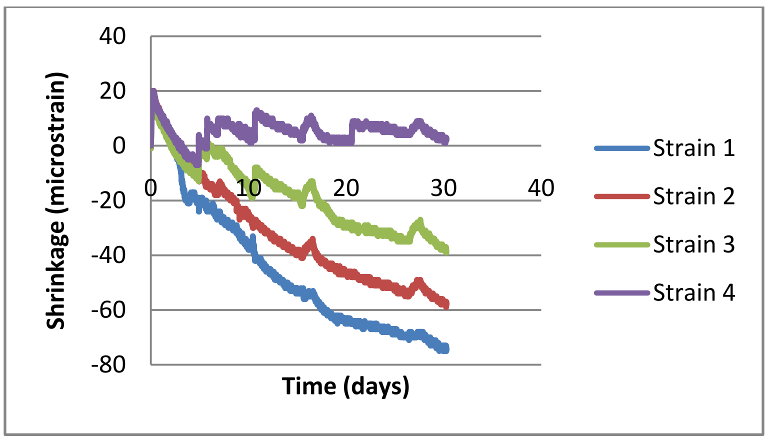

Fig. C.4b Ring Test for EW-SF-SRA, 6 in. Ring B, from day 1

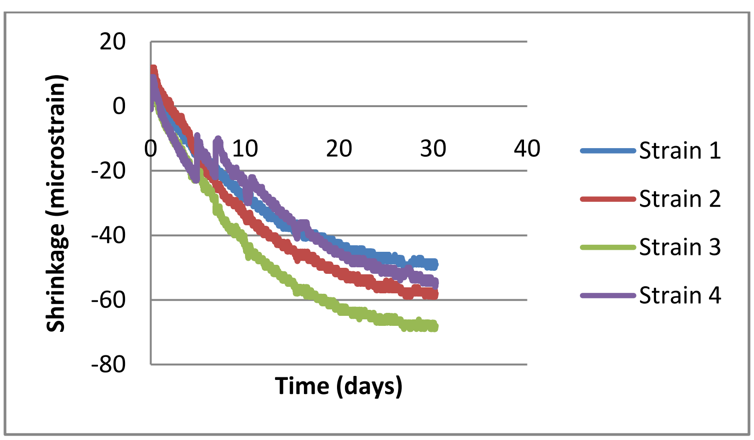

Fig. C.4c Ring Test for EW-SF-SRA, 3 in. Ring, from day 1

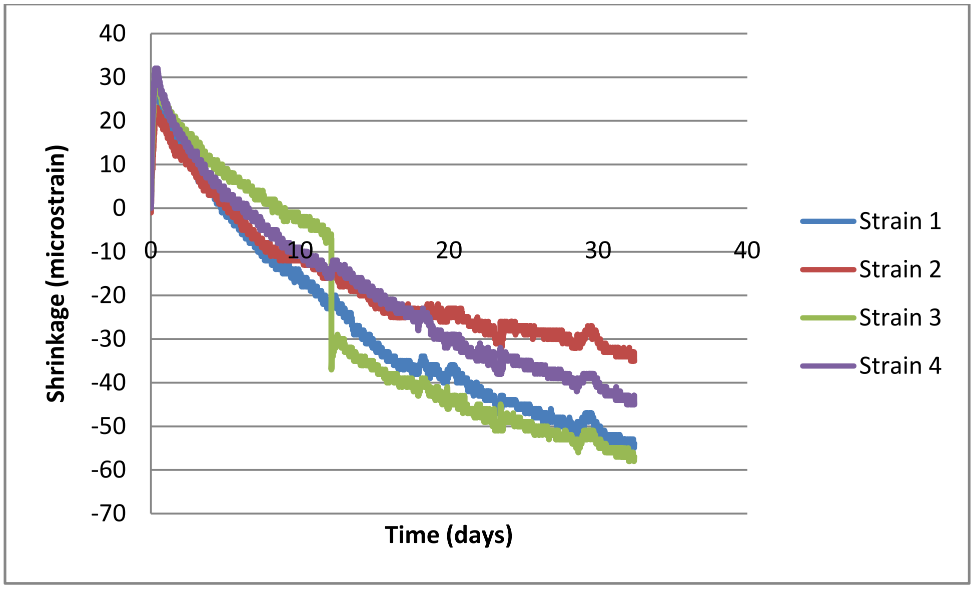

Fig. C.5a Ring Test for EW-FA-SRA, 6 in. Ring A, from day 1

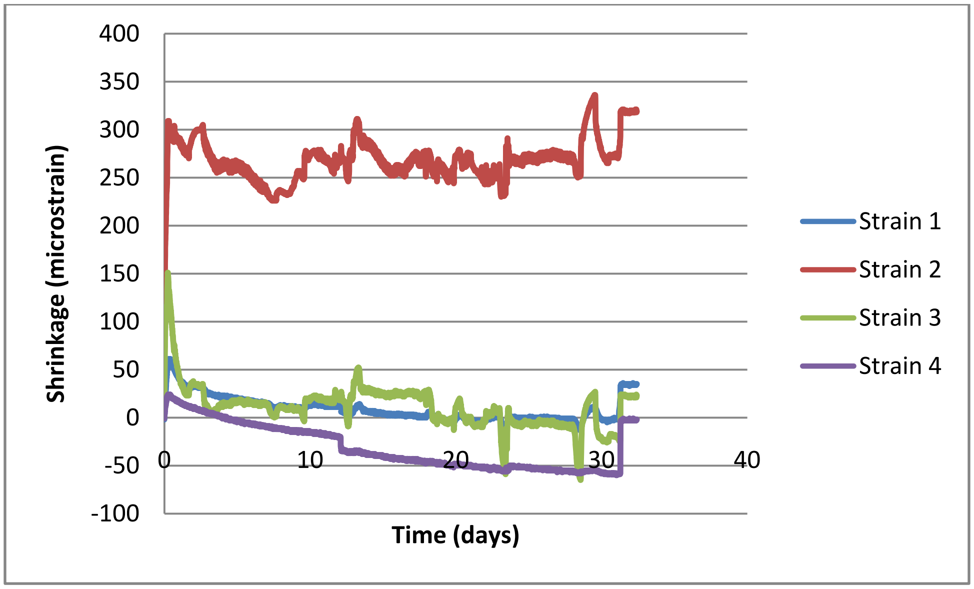

Fig. C.5b Ring Test for EW-FA-SRA, 6 in. Ring B, from day 1

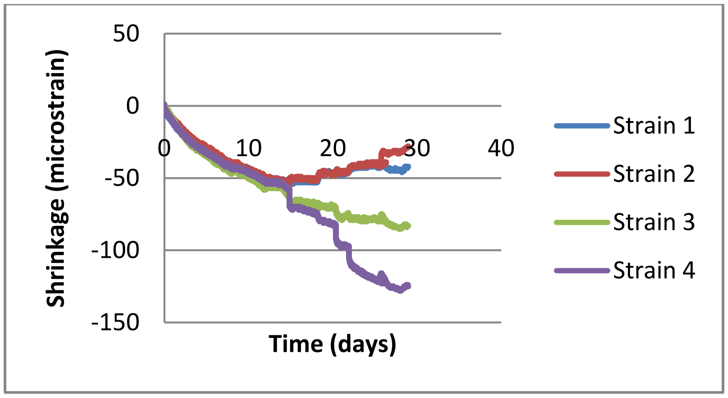

Fig. C.6a Ring Test for EW-FA-SL-SRA, 6 in. Ring A, from day 1

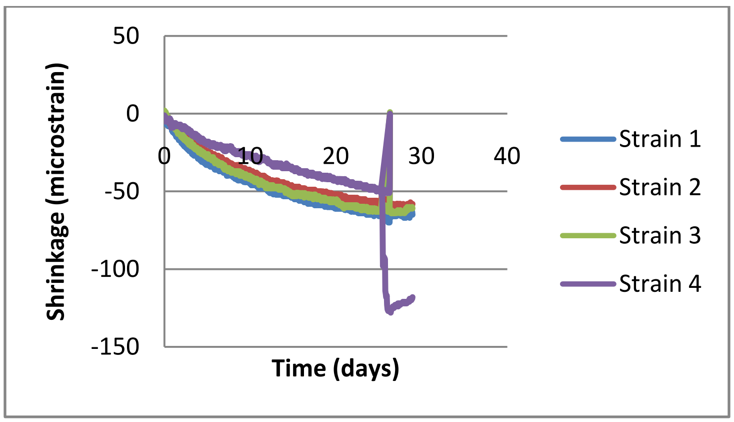

Fig. C.6b Ring Test for EW-FA-SL-SRA, 6 in. Ring B, from day 1

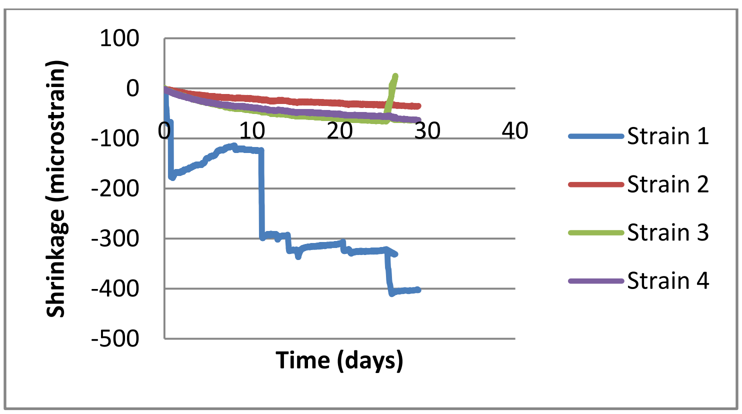

Fig. C.6c Ring Test for EW-FA-SL-SRA, 3 in. Ring, from day 1

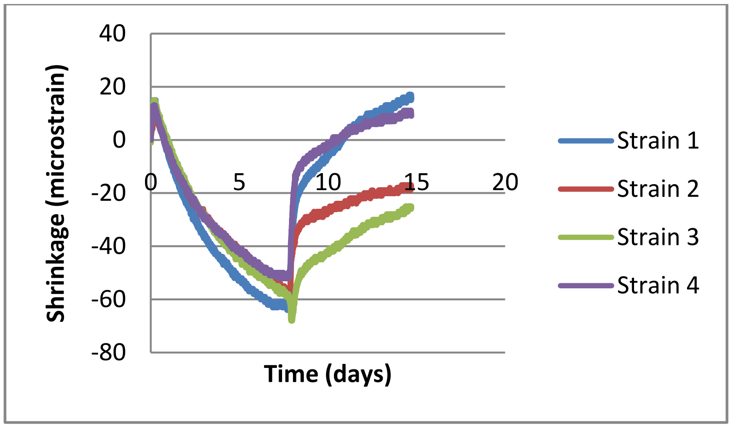

Fig. C.7a Ring Test for EW-FA, 6 in. Ring A, from day 1

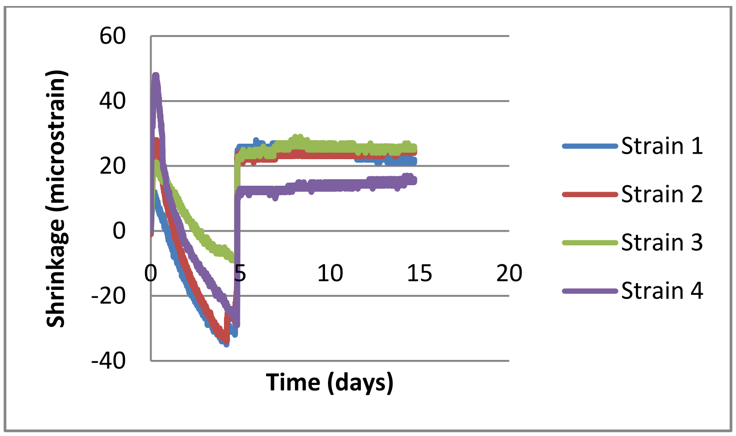

Fig. C.7b Ring Test for EW-FA, 6 in. Ring B, from day 1

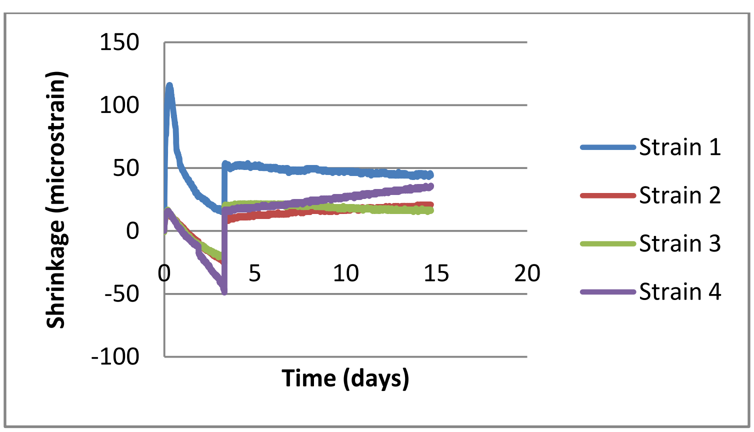

Fig. C.7c Ring Test for EW-FA, 3 in. Ring, from day 1

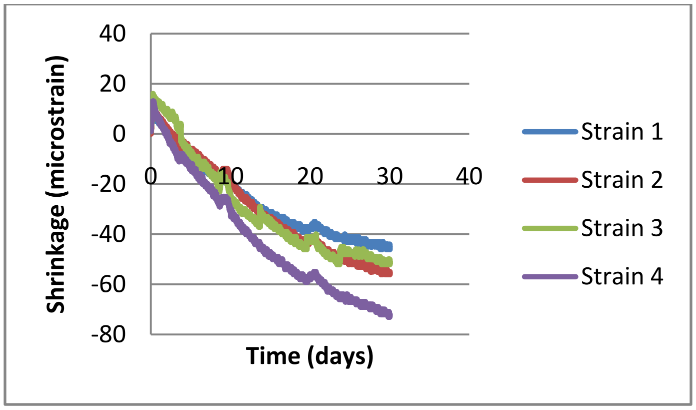

Fig. C.8a Ring Test for EW-FA-SF-SRA, 6 in. Ring A, from day 1

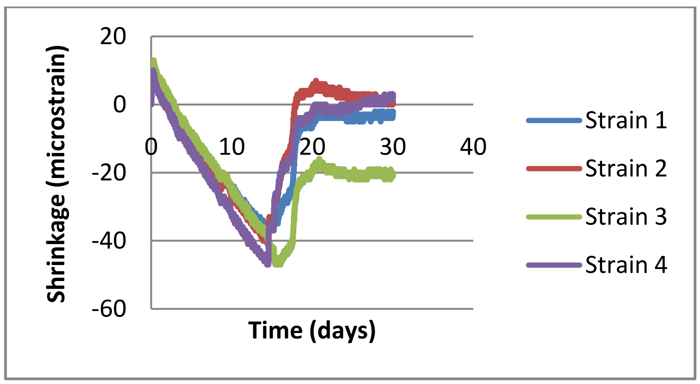

Fig. C.8b Ring Test for EW-FA-SF-SRA, 6 in. Ring B, from day 1

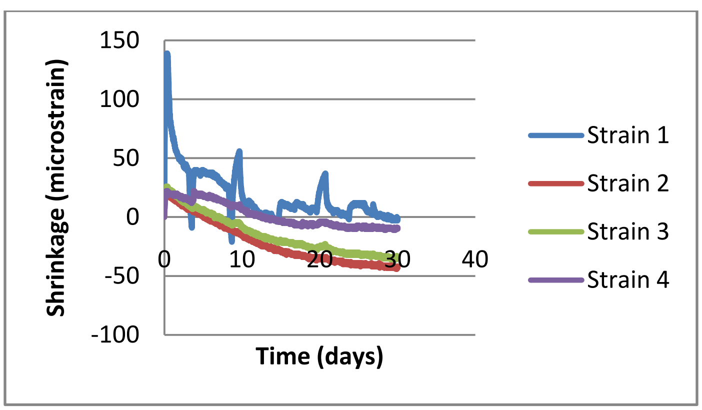

Fig. C.8c Ring Test for EW-FA-SF-SRA, 3 in. Ring, from day 1

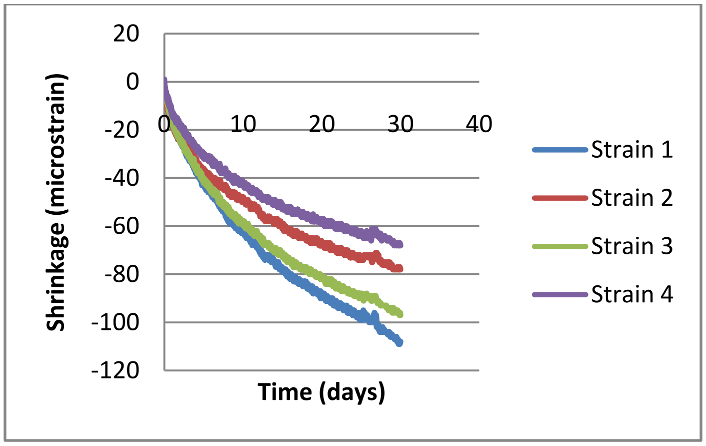

Fig. C.9a Ring Test for WW-SRA, 6 in. Ring A, from day 1

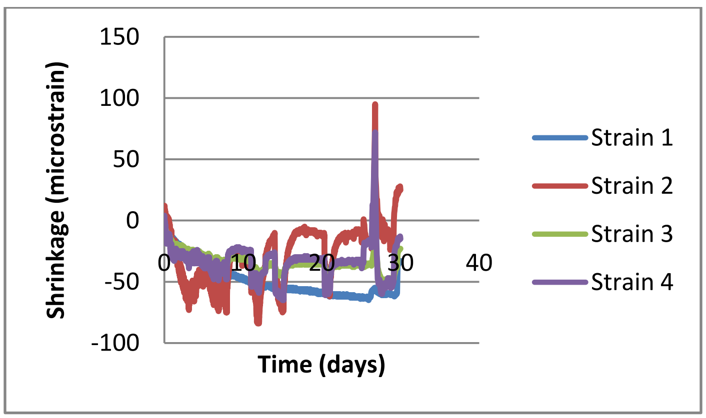

Fig. C.9b Ring Test for WW-SRA, 6 in. Ring B, from day 1

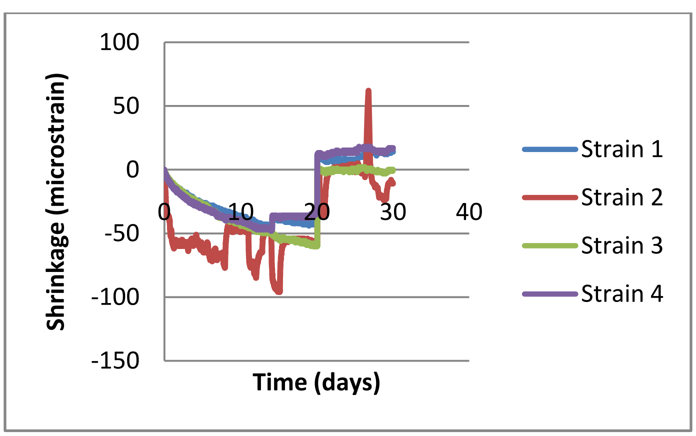

Fig. C.9c Ring Test for WW-SRA, 3 in. Ring, from day 1

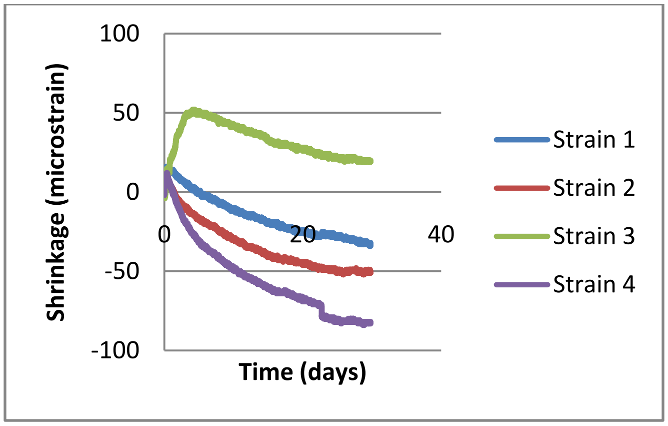

Fig. C.10a Ring Test for WW-SF-SRA, 6 in. Ring A, from day 1

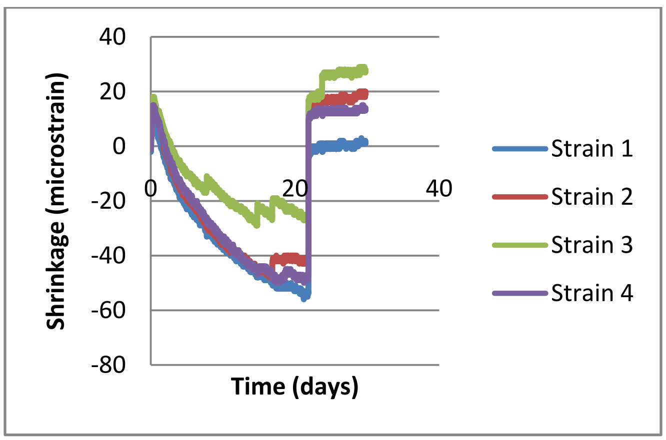

Fig. C.10b Ring Test for WW-SF-SRA, 6 in. Ring B, from day 1

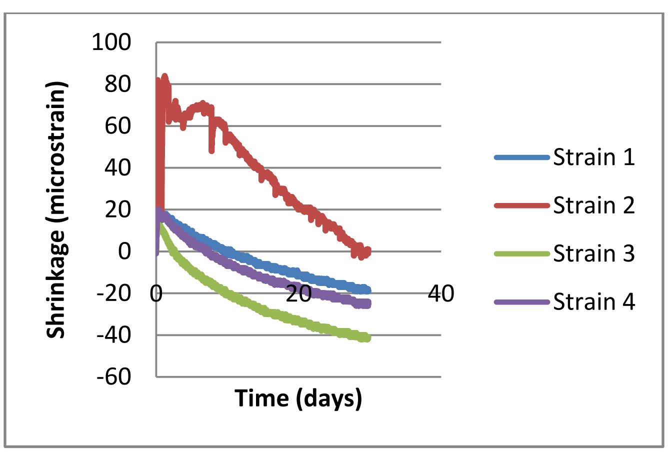

Fig. C.10c Ring Test for WW-SF-SRA, 3 in. Ring, from day 1

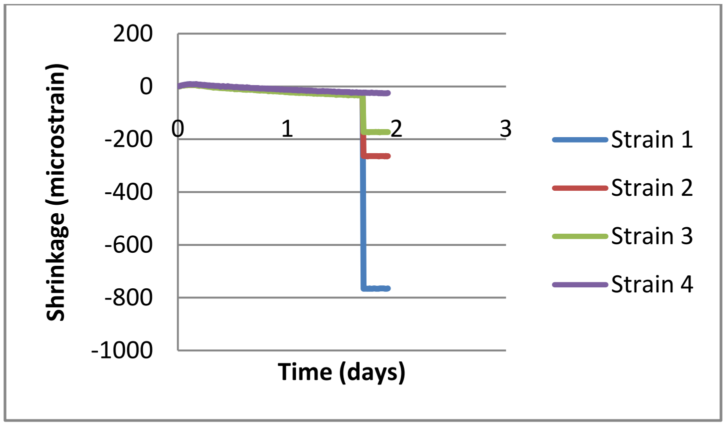

Fig. C.11a Ring Test for WW, 6 in. Ring A, from day 1

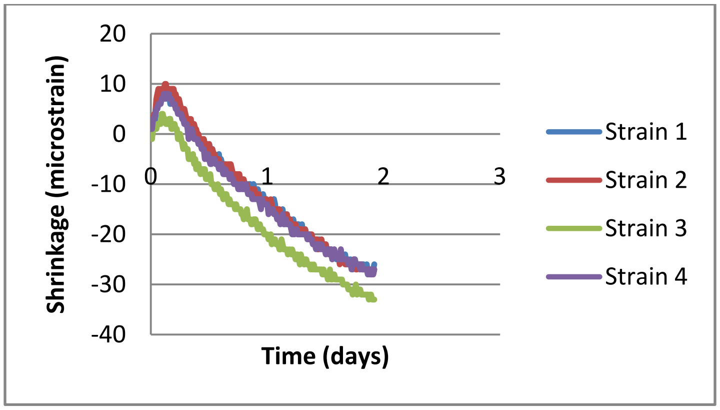

Fig. C.11b Ring Test for WW, 6 in. Ring B, from day 1

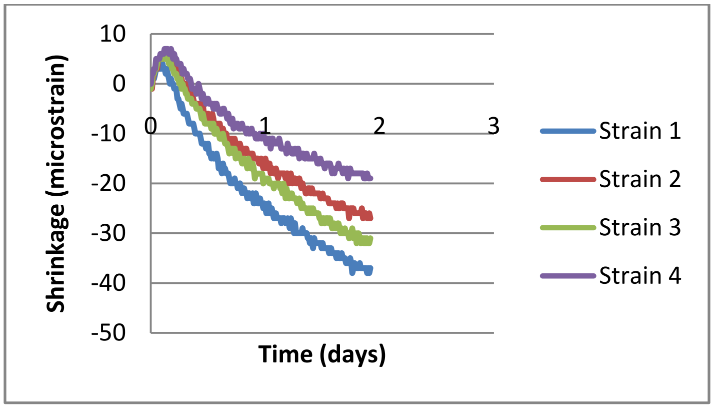

Fig. C.11c Ring Test for WW, 3 in. Ring, from day 1

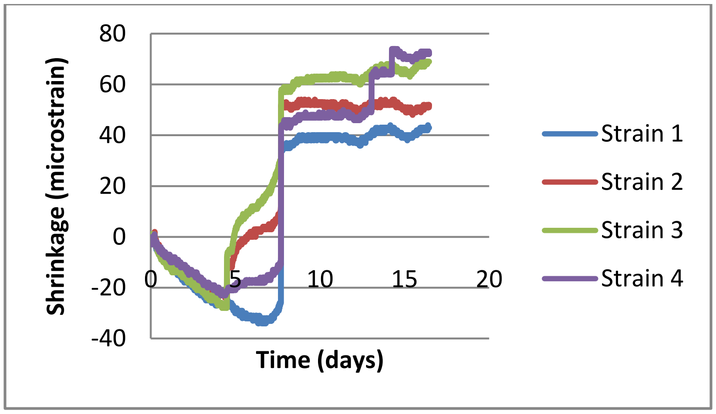

Fig. C.11d Ring Test for WW, 6 in. Ring A, from day 4

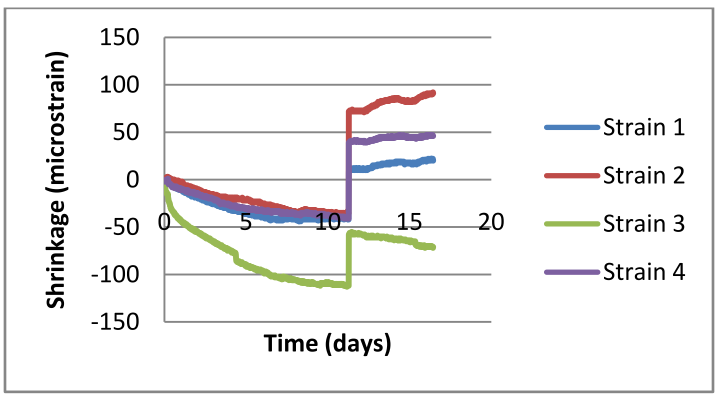

Fig. C.11e Ring Test for WW, 6 in. Ring B, from day 4

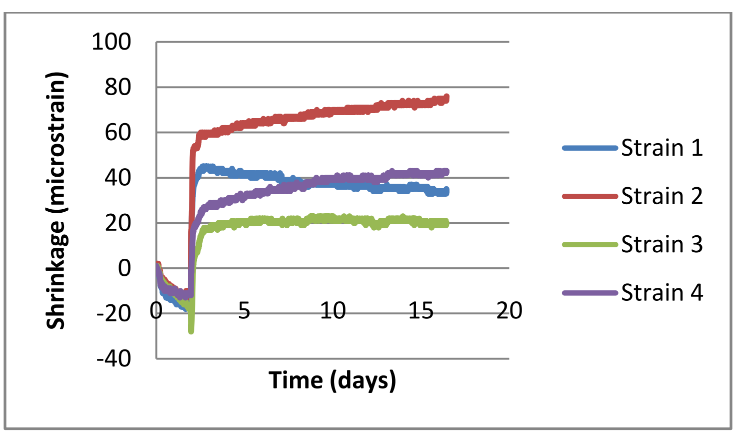

Fig. C.11f Ring Test for WW, 3 in. Ring, from day 4

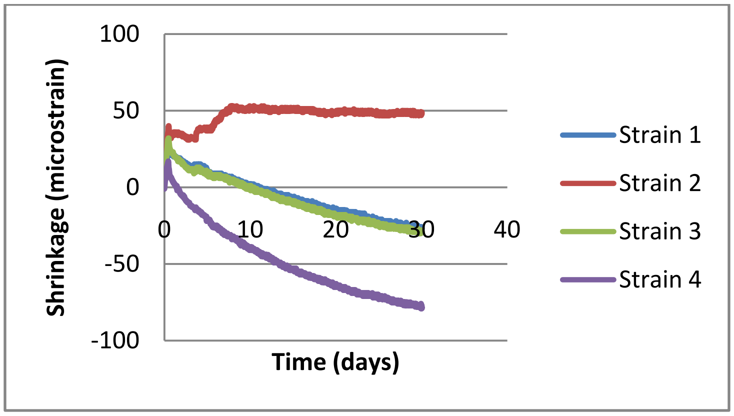

Fig. C.12a Ring Test for WW-SF-SRA, 6 in. Ring A, from day 1

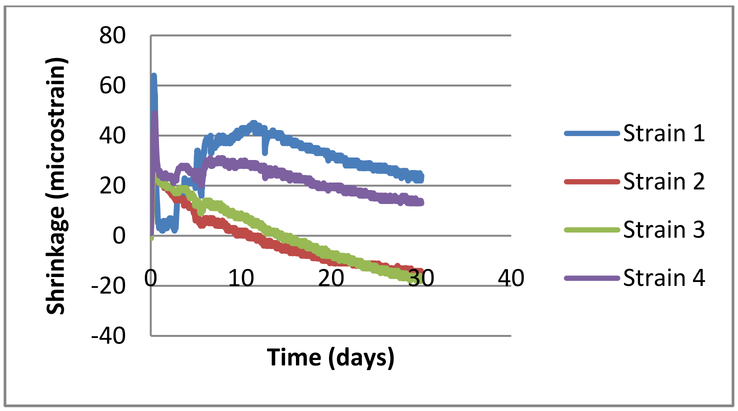

Fig. C.12b Ring Test for WW-SF-SRA, 6 in. Ring B, from day 1

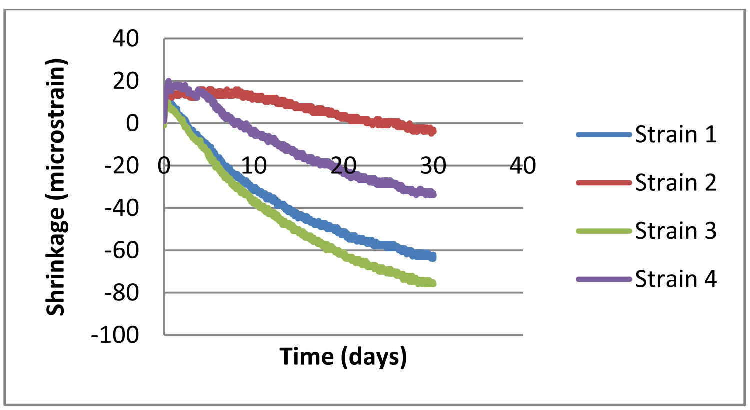

Fig. C.12c Ring Test for WW-SF-SRA, 3 in. Ring, from day 1

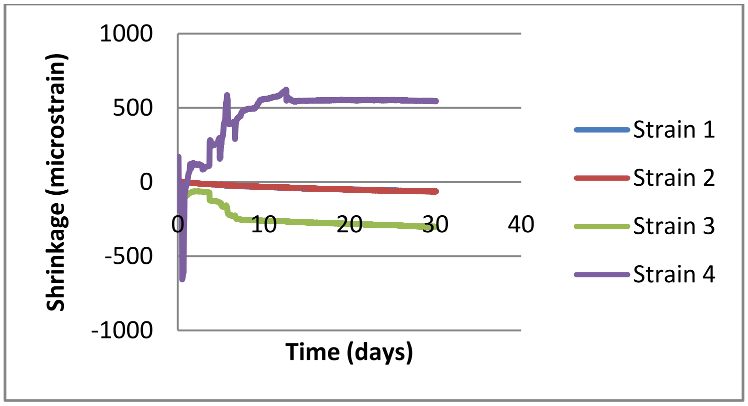

Fig. C.13a Ring Test for WW-FA-SRA, 6 in. Ring A, from day 1

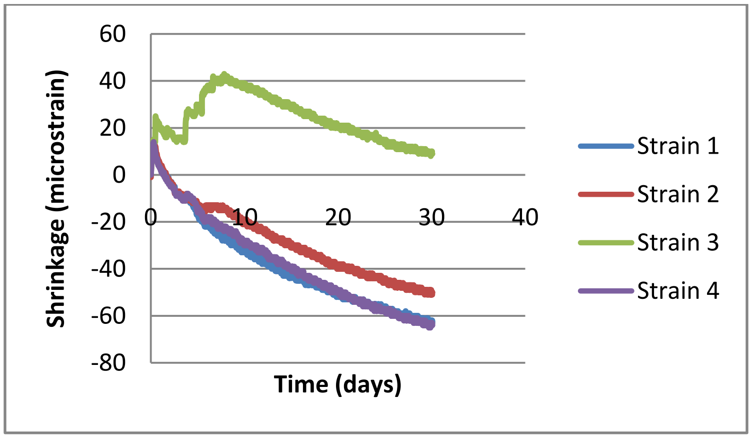

Fig. C.13b Ring Test for WW-FA-SRA, 6 in. Ring B, from day 1

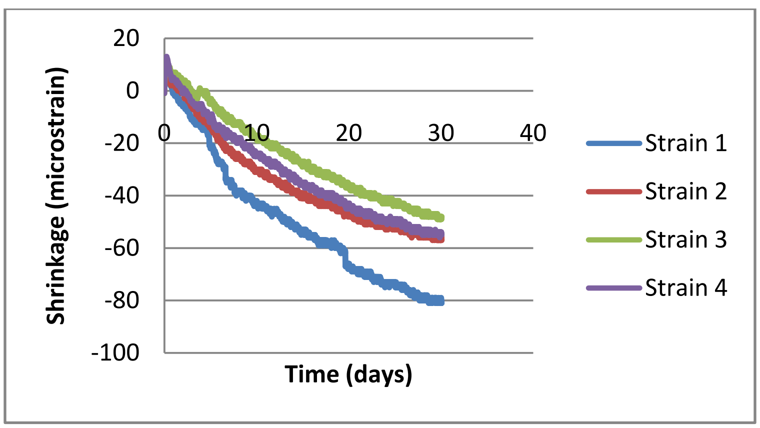

Fig. C.13c Ring Test for WW-FA-SRA, 3 in. Ring, from day 1

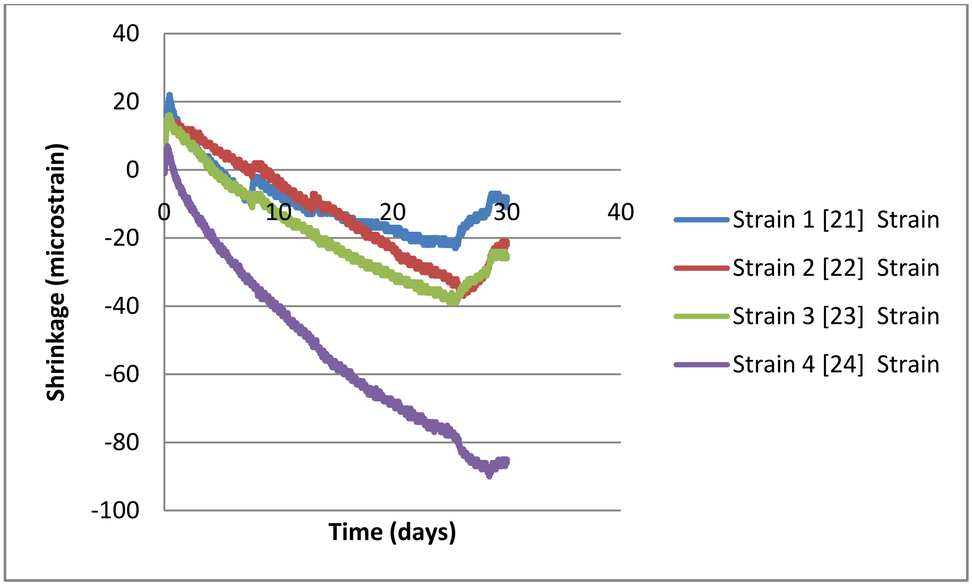

Fig. C.14a Ring Test for WW-FA-SL-SRA, 6 in. Ring A, from day 1

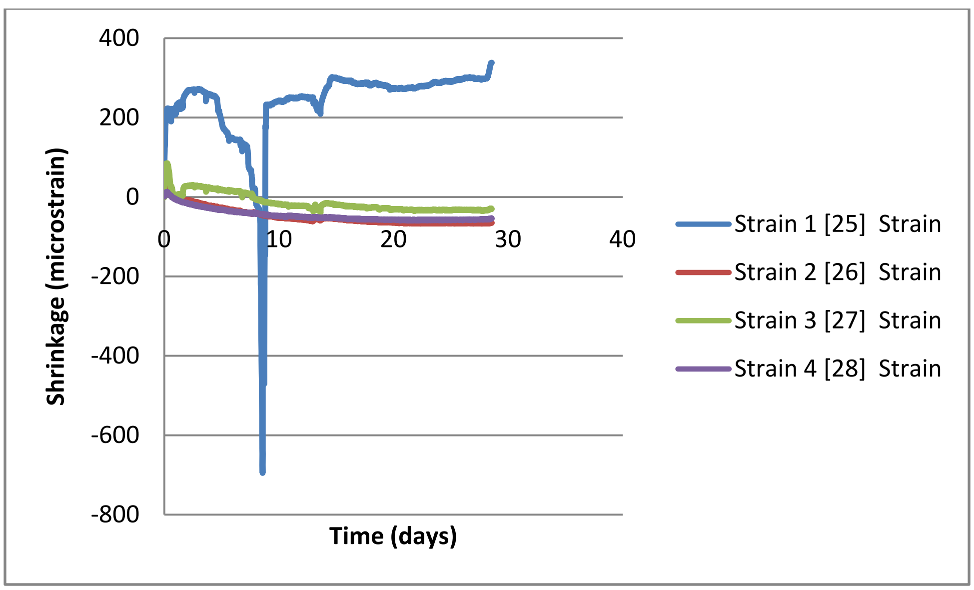

Fig. C.14b Ring Test for WW-FA-SL-SRA, 6 in. Ring B, from day 1

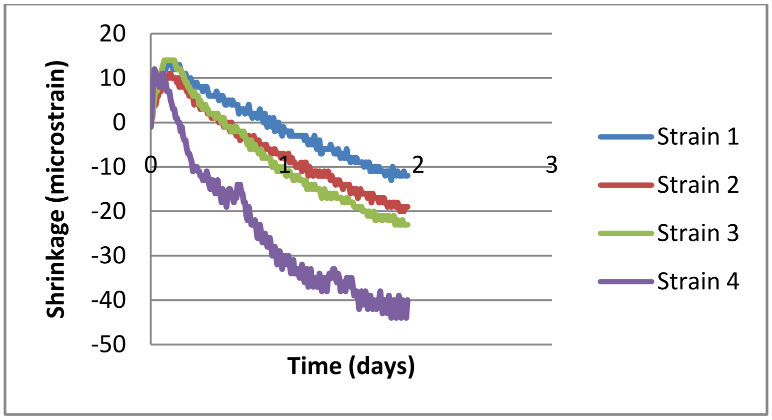

Fig. C.15a Ring Test for WW-FA, 6 in. Ring A, from day 1

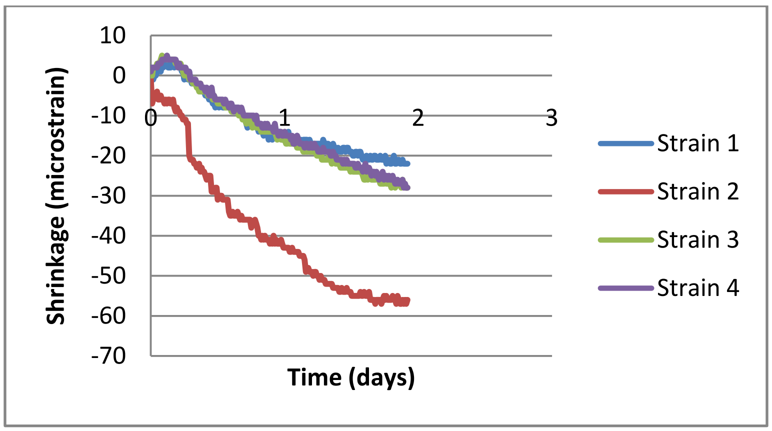

Fig. C.15b Ring Test for WW-FA, 6 in. Ring B, from day 1

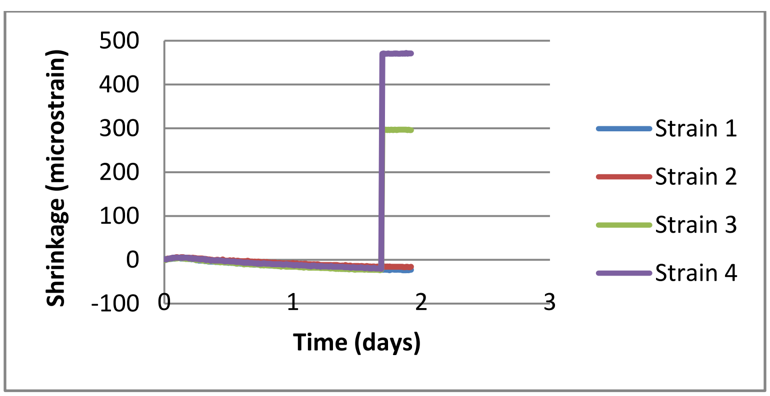

Fig. C.15c Ring Test for WW-FA, 3 in. Ring, from day 1

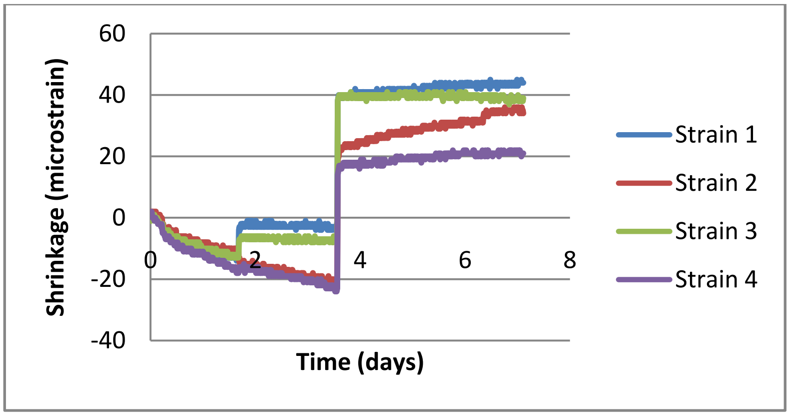

Fig. C.15a Ring Test for WW-FA, 6 in. Ring A, from day 4

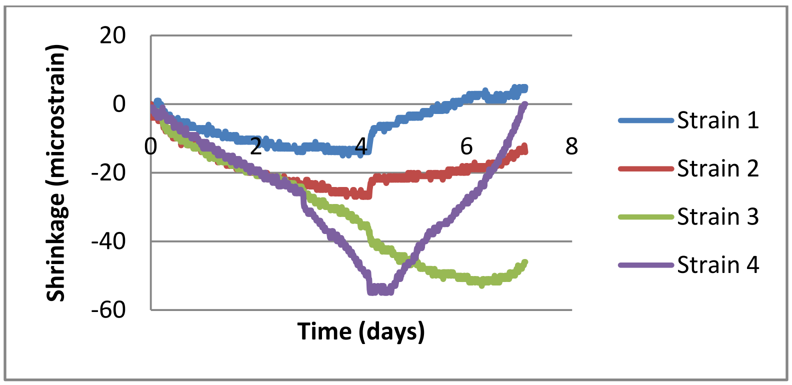

Fig. C.15b Ring Test for WW-FA, 6 in. Ring B, from day 4

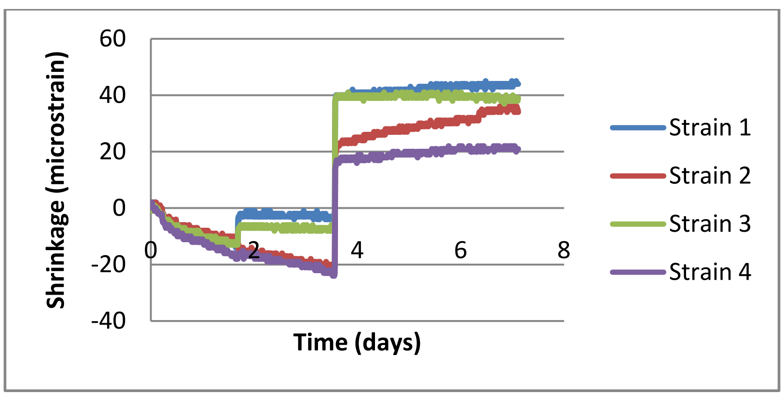

Fig. C.15c Ring Test for WW-FA, 3 in. Ring, from day 4

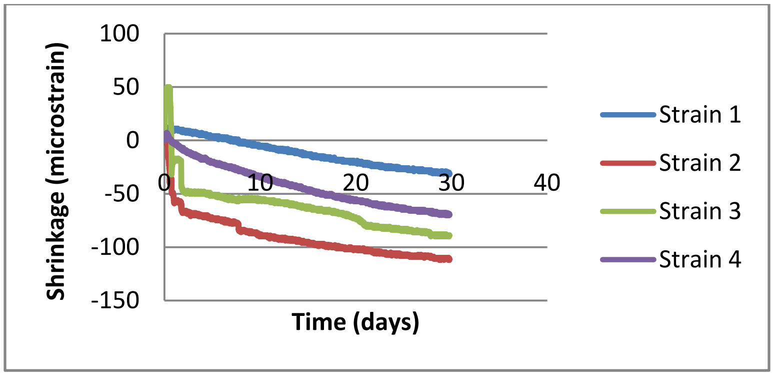

Fig. C.16a Ring Test for WW-FA-SF-SRA, 6 in. Ring A, from day 1

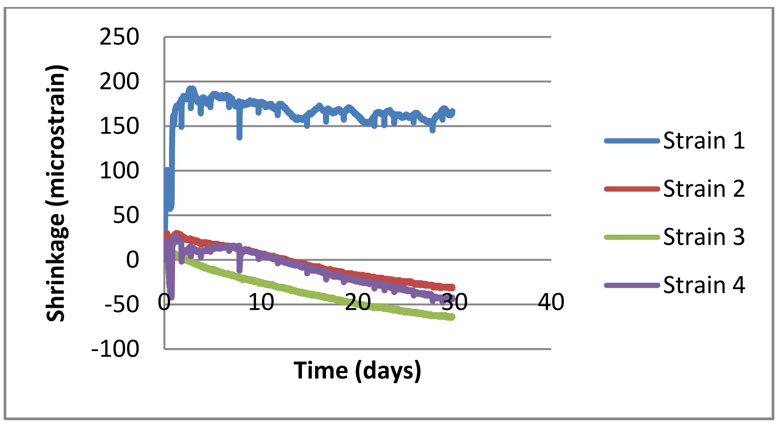

Fig. C.16b Ring Test for WW-FA-SF-SRA, 6 in. Ring B, from day 1

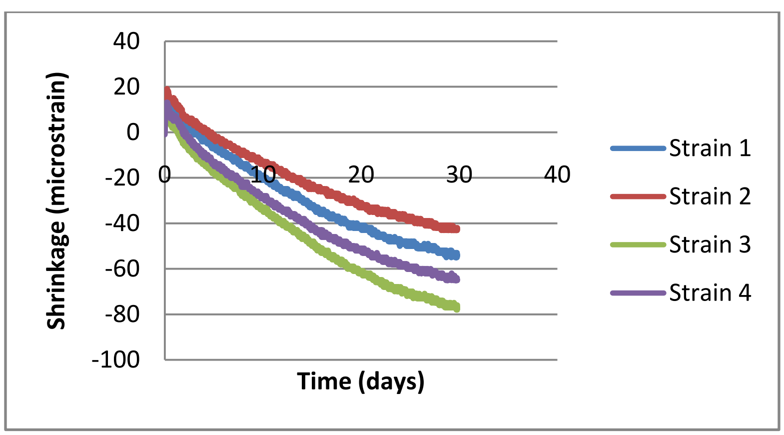

Fig. C.16c Ring Test for WW-FA-SF-SRA, 3 in. Ring, from day 1

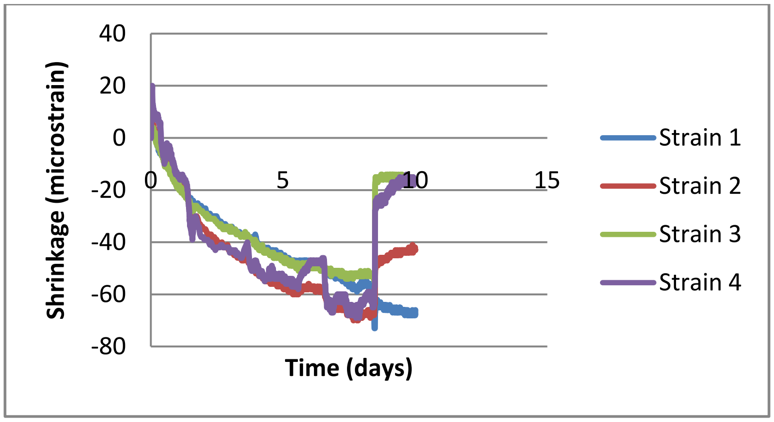

Fig. C.17a Ring Test for LD-WSDOT, 6 in. Ring A, from day 1

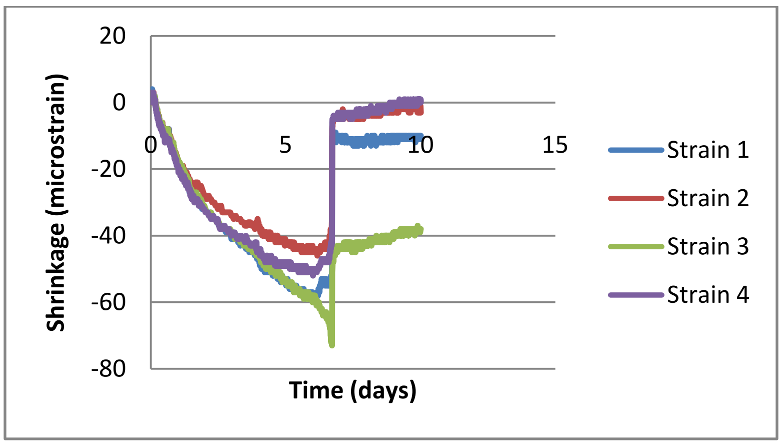

Fig. C.17b Ring Test for LD-WSDOT, 6 in. Ring B, from day 1

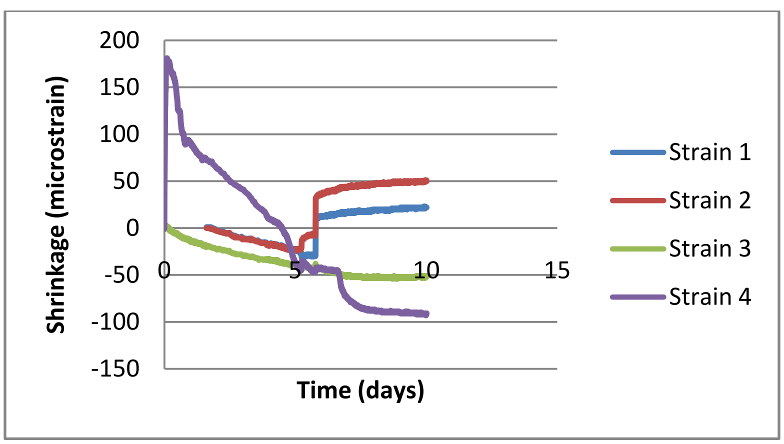

Fig. C.17c Ring Test for LD-WSDOT, 3 in. Ring, from day 1

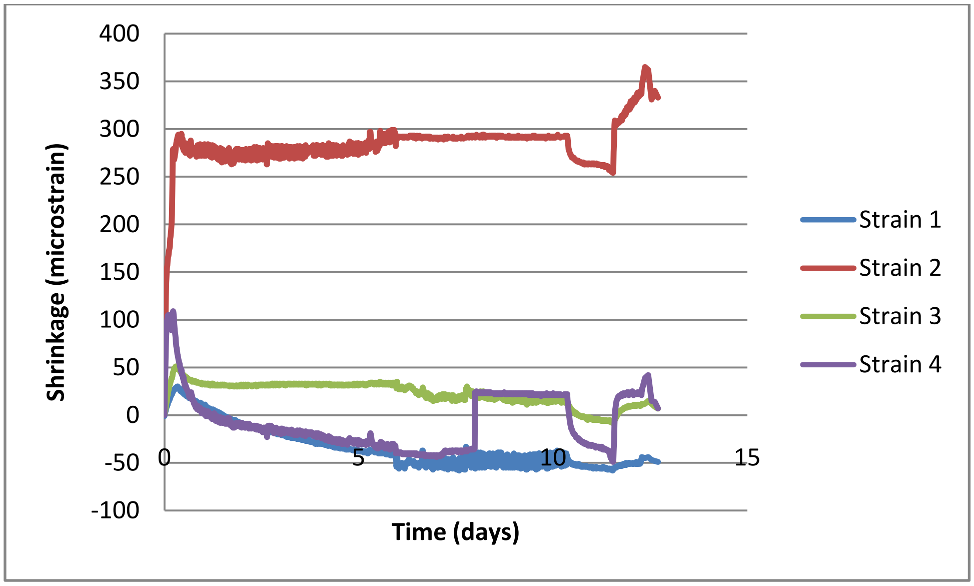

Fig. C.18a Ring Test for WSDOT, 6 in. Ring A, from day 1

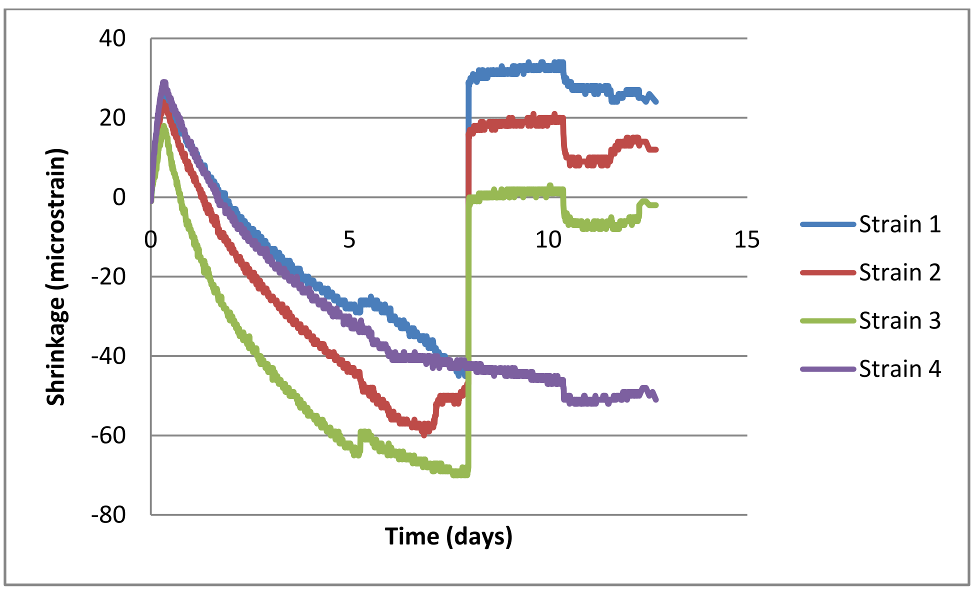

Fig. C.18b Ring Test for WSDOT, 6 in. Ring B, from day 1

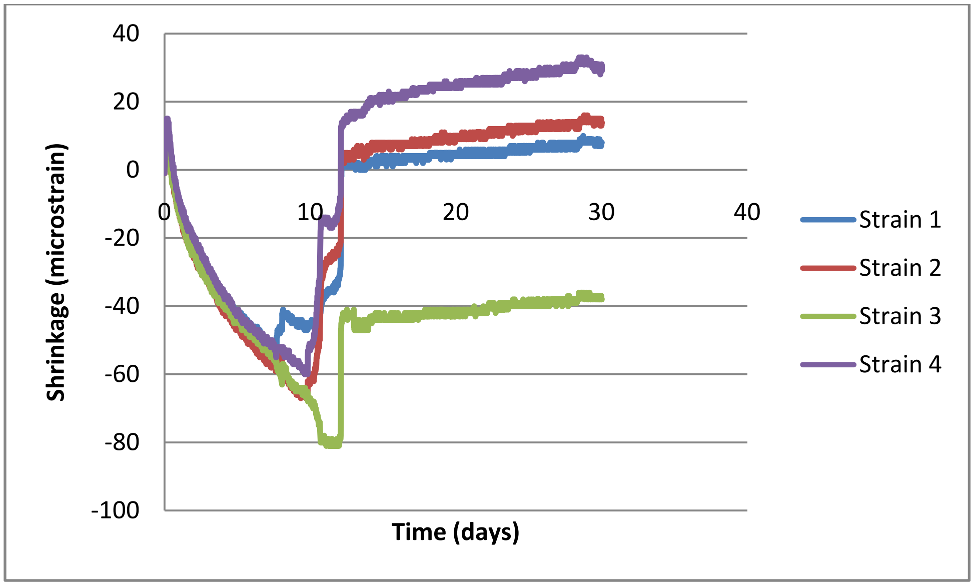

Fig. C.19a Ring Test for EW 2, 6 in. Ring A, from day 1

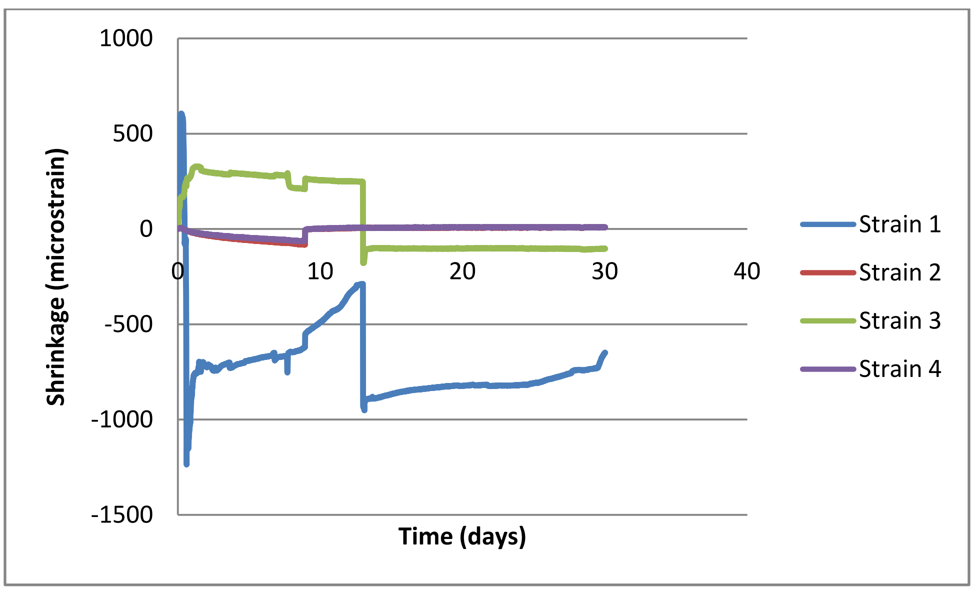

Fig. C.19b Ring Test for EW 2, 6 in. Ring B, from day 1

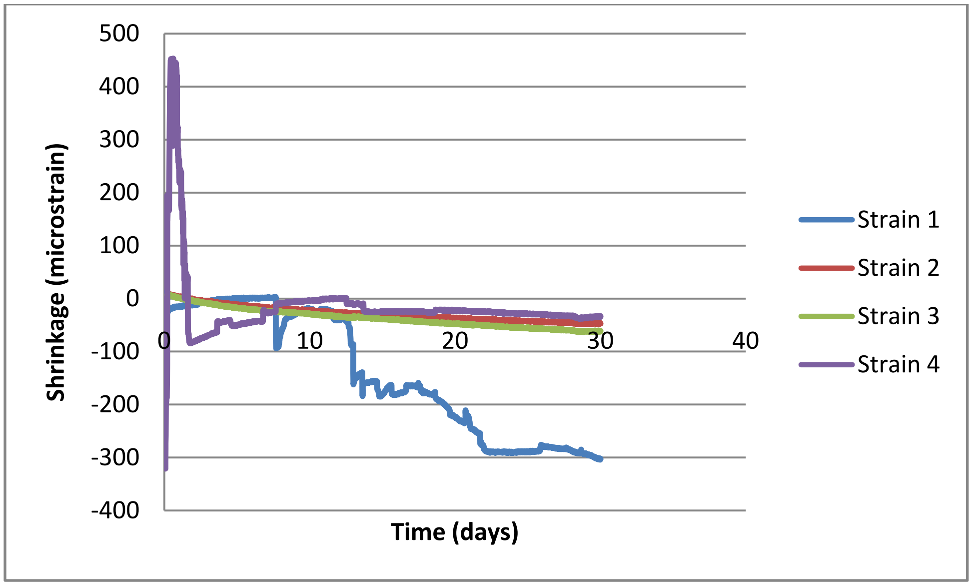

Fig. C.20a Ring Test for EW 2.5, 6 in. Ring A, from day 1

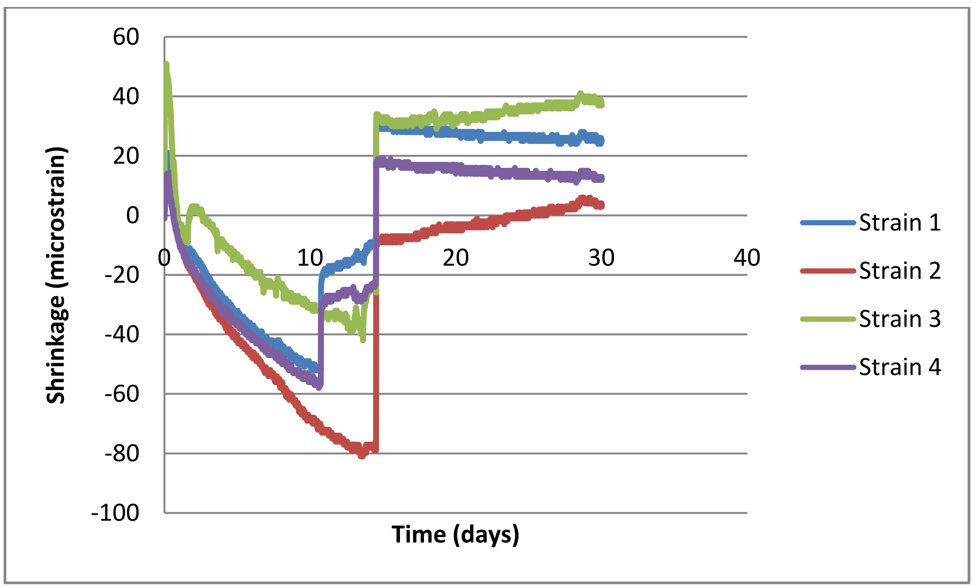

Fig. C.20b Ring Test for EW 2.5, 6 in. Ring B, from day 1

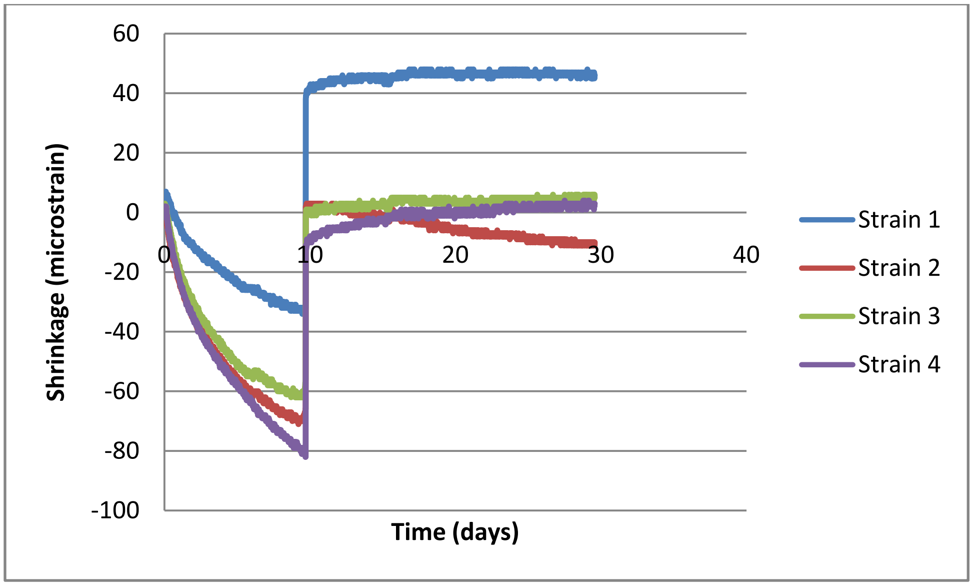

Fig. C.21a Ring Test for WW 2, 6 in. Ring A, from day 1

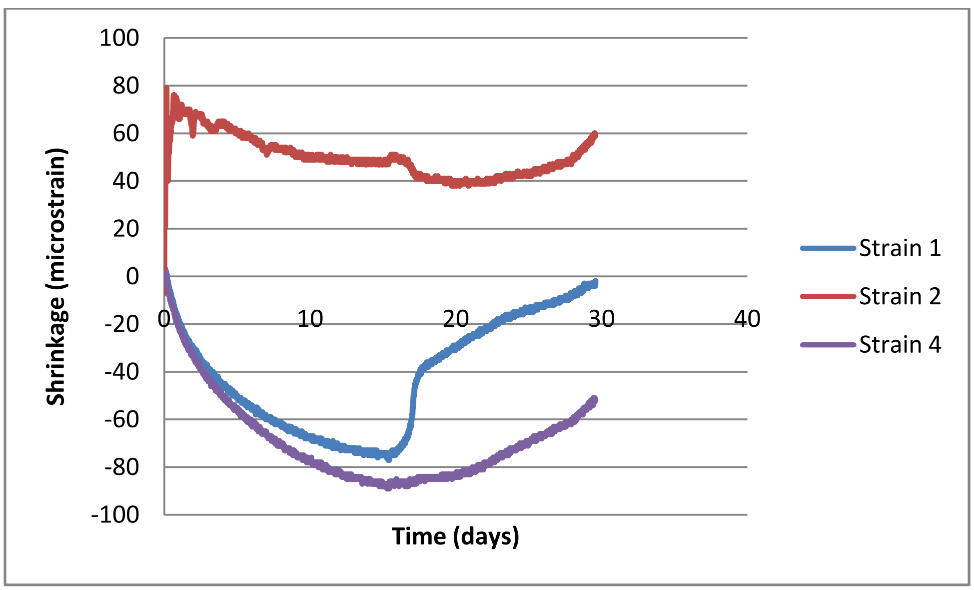

Fig. C.21b Ring Test for WW 2, 6 in. Ring B, from day 1

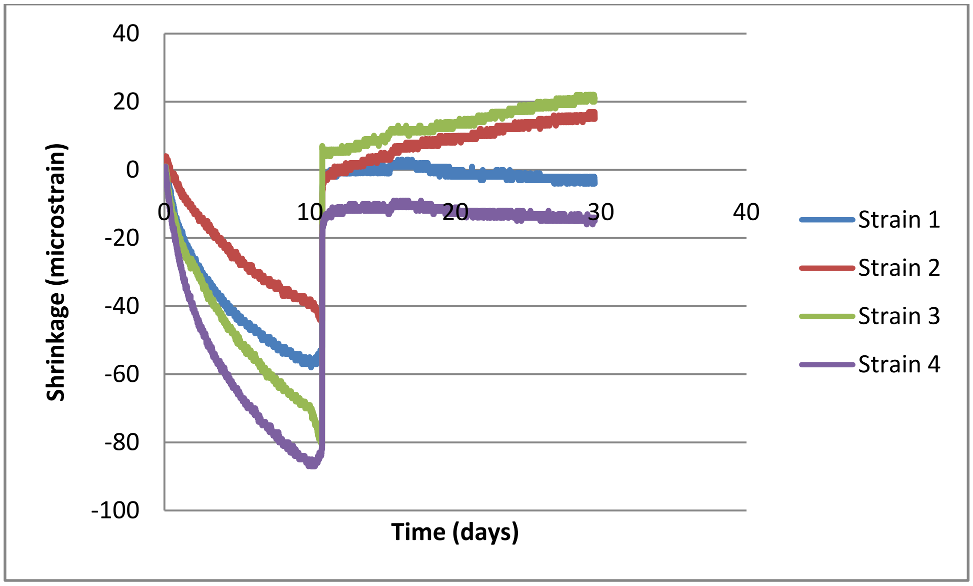

Fig. C.22a Ring Test for WW 2.5, 6 in. Ring A, from day 1

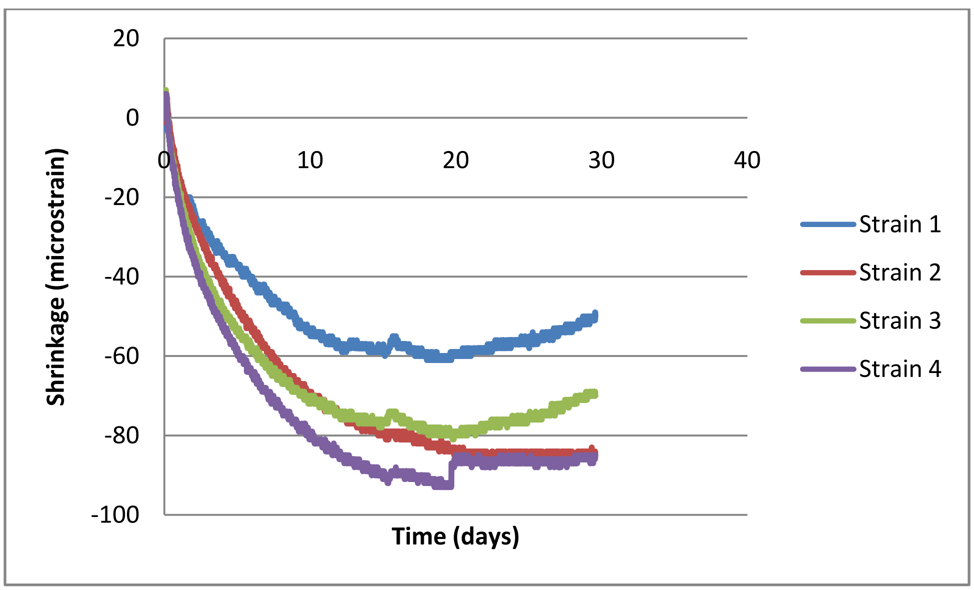

Fig. C.22b Ring Test for WW 2.5, 6 in. Ring B, from day 1