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Effect of Lightweight Aggregate Moisture Content on Internally Cured Concrete

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EFFECT OF LIGHTWEIGHT AGGREGATE MOISTURE CONTENT ON INTERNALLY CURED CONCRETE

Effect of Lightweight Aggregate Moisture Content on Internally Cured Concrete

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by

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Abstract

Putting an end to the rapid deterioration of concrete structures in the United States will only occur through the introduction of better materials and construction methods. The American Society of Civil Engineers (ASCE) reports the overall condition of concrete bridges in the United States to be a “C+”. Though better than other areas of the infrastructure, there is still room for improvement in concrete bridges. One major area that may be improved is that of the concrete bridge deck. The bridge deck is comprised of the actual driving surface. It is subject to many deterioration mechanisms including freeze/thaw cycles, de-icing salts, and cyclic loading. The bridge deck must be highly durable to combat the negative effects it is subjected to. A recent advancement to increase durability is the placement of a pre-saturated lightweight aggregate (LWA) inside concrete. This LWA will offset cracking associated with drying shrinkage and self-desiccation. The pre-saturated LWA is an expanded clay or expanded shale material which is able to absorb water into the pores of its structure and release that water to hydrating cement. The current research is focused on determining the effects of this LWA soaking durations of 1, 3, and 7 days. A constant replacement rate of 300 lbs/yd^3 was used for the current study. A control mixture was cast for comparison purposes which contained no LWA. The research program concrete mix designs were developed in accordance with the Arkansas State Highway and Transportation Department bridge deck specification. Reported results included shrinkage/strain, compressive strength, modulus of rupture, and modulus of elasticity. Findings indicated that a 1 day soaked LWA preformed equivalent or better than extended soaking durations at mitigating concrete shrinkage.

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Chapter 1 Introduction

1.1 Introduction

As with any construction project, the service life of the constructed facility is as short as its weakest element. Some elements of a particular structure do not undergo the same degree of physical assault as others. A bridge deck will be subject to variable loading and unloading, freeze thaw conditions, deicing salts, and other deleterious substances. As such, the bridge deck has the potential to be the weakest link in any bridge simply because of the conditions it will be subject to throughout its life. A bridge deck must be designed and constructed carefully to mitigate potential damage and deterioration mechanisms to ensure the serviceability of the bridge deck.

In 2013, a “D+” was awarded to the overall infrastructure of the United States by the American Society of Civil Engineers (ASCE) infrastructure report card (ASCE, 2013). Based on this scoring, there is room for improvement in every aspect of the overall infrastructure. The quickest approach to solve the current failing infrastructure would be to spend the estimated 3.6 trillion dollars required to fix all of the problems (ASCE, 2013). This spending would only fix the current deficiencies and would not implement new or better products and systems. A better solution would be to work towards developing new products produced through research to aid designers and contractors in developing a more sustainable infrastructure. New technologies developed through research further not only the advancement of knowledge, but also lead to real world applications of that technology. These advancements will provide the strategic foundation for improving America’s infrastructure.

In the 2013 ASCE report, bridges fared slightly better than the entire collection of U.S. infrastructure systems, and received an overall grade of “C+” (ASCE, 2013). The report card notes that of the 607,380 bridges in the United States, just over 11 percent are structurally deficient (ASCE, 2013). Many of those bridges are nearing the end or at least the latter half of their expected design life. This poses a major threat to transportation routes for all forms of commerce. Without bridges to connect the various interstates and highways, there would be no interstate commerce. Many man hours may be lost yearly if the deterioration of the nation's bridges continues to remain unchecked. These lost hours cannot be made up and would be destructive to the financial future of the nation. In essence, our bridges must be protected to ensure the societal and economic wellbeing of the nation.

1.2 Research

In order to preserve the investment made into new bridges and the nation's infrastructure as a whole, there is a need to increase the service life of these projects. One solution to increasing their service life is by increasing the durability of the materials used to build these projects. For concrete bridges, increased durability is directly related to decreasing the number and size of cracks associated with new construction. These cracks may form as a result of several different mechanisms; however, the end effect is usually the same. Concrete cracking introduces deleterious substances to the internal mechanics of the concrete. Deleterious substances include water and salts which deteriorate the concrete and steel reinforcement. The deterioration caused by invasive substances leads to shorter than expected lifespans of the concrete. One of the best ways to combat the effects of cracking with any concrete structure is to properly cure it. Curing may take several forms, but if implemented properly it will reduce cracking and increase concrete lifespans. One such form of curing is internal curing.

Internal curing provides much needed water for cement hydration to the internal recesses of the concrete. It provides a basic function (that of an internal humid environment) which all concrete needs, but in a different form than traditionally provided. In this project, internal curing will be studied to determine its applicability in mitigating shrinkage cracking in concrete bridge decks. Bridge deck cracking is one of the most noticeable forms of cracking in any bridge because it is in direct connection with all who traverse the bridge. As mentioned previously, bridge decks must be able to withstand many adverse conditions. Proper techniques must be utilized during all stages of construction to ensure the durability and highest quality of the concrete. This will ensure a long lasting deck that does not need to be resurfaced every few years.

The proposed research will study the attributes of internally cured concrete using a pre-saturated lightweight aggregate (LWA). The aggregate will absorb water inside its porous structure and release that water while the cement is hydrating. It will be cast directly into the concrete during mixing and will be based on a direct replacement rate with the limestone coarse aggregate. The effects from the introduction of LWA into the concrete will be monitored on both fresh and hardened concrete properties.

Internally cured concrete using a pre-saturated LWA is an area that has been previously examined. Some effects from introducing LWA into concrete are known. However, the effects of employing different soaking times on the LWA (1, 3, and 7 days) will be evaluated in this study. The proposed research will determine the advantages, if any, of increasing the soaking time of the LWA prior to incorporating it into the mix. By reducing the soaking time of the LWA in the concrete, the cost associated with the LWA will be mitigated as well. This cost mitigation will increase the viability of internally cured concrete as an option for major construction projects.

Another goal of the research is to determine the applicability of internally cured concrete for the mitigation of shrinkage in concrete bridge decks. The research will aid the overall knowledge of the field of internal curing and will provide paths for the inclusion of LWA into more concrete mix designs. By providing another alternate curing method to construction management professionals, the probability of longer service lives of concrete structures can increase. Through proper management and development of more innovative technologies, longer lifespans and increased durability are possible for new infrastructure. These techniques may even be used to overhaul the current infrastructure to extend its service life. By improving the existing structures and developing new sustainable construction methods, the future for the nation's infrastructure is bright.

Chapter 2 Literature Review

2.1 Historical Perspective of Internal Curing

The use of light weight aggregates (LWA) began during the Roman Empire (Bentz & Weiss, 2011). LWA was utilized in the Pantheon to aid in reducing the dead weight of the arched roof (Bentz & Weiss, 2011). An added benefit of the inclusion of LWA in the Pantheon was the supply of water to the internal recesses of the hydrating cement matrix stimulating internal curing. Although some benefits of LWA were realized long ago, Philleo (1991) was the first to verbalize the idea of internally cured concrete (Bentz & Weiss, 2011).

During the latter half of the Twentieth century, vast developments in the mechanical properties of concrete were realized. Although many of these effects were positive, some were negative. New high strength concrete mixtures were being developed and re-developed (PCA, 1994). As the compressive strength of the concrete increased, other properties such as the modulus of elasticity, permeability, and tensile strength were also affected. The concrete suffered from self-desiccation brought about by the lack of available water for hydration of the cement particles (Bentz & Weiss, 2011). Self-desiccation is the loss of water within the cement matrix due to the lack of necessary mix water to hydrate the cement (Holt, 2001). Suction stresses are formed due to the hydrating cement particles attempting to pull water from within the cement matrix (Holt, 2001). These suction stresses may lead to cracking if water is not allowed to alleviate the hydrating cement particles. To combat the negative effects associated with the new higher strength concrete, research investigating the water available for hydration was performed. One solution to the self-desiccation problem experienced with high strength concrete was internal curing (Bentz & Weiss, 2011).

2.2 Internal Curing Overview

Internal curing is the process of placing water in an encapsulated state within the concrete for use during hydration of the cement matrix (Henkensiefken et al., 2011). There are many ways water can be encapsulated and batched directly into the concrete mixture itself. ACI (2010) describes internal curing as "supplying water throughout a freshly placed cementitious mixture using reservoirs, via pre-wetted lightweight aggregates, that readily release water as needed for hydration or to replace moisture lost through evaporation or self-desiccation" (Bentz & Weiss, 2011). These reservoirs include, but are not limited to: lightweight expanded clay and shale aggregates (LWA), superabsorbent polymers, Bentonite clays, and naturally occurring porous materials such as pumice (Jensen & Lura, 2006). The internal curing material is pre-soaked in water. By encapsulating the additional water, the water cement (w/c) ratio of the mixture is not affected. The water is considered additional water and only utilized during hydration (Bentz et al., 2005). The water added from internal curing is similar to the water added from external curing. Water added from external curing practices i.e. wet burlap, soaked plastic sheeting, or a curing compound is not added to the w/c of traditional mix design. Similarly water absorbed by the LWA is not accounted for in the w/c of internally cured concrete (Roberts, 2006). Unlike external curing which applies water only to the first few millimeters of the outer surface, internal curing water is distributed more evenly during curing which allows for the hydration of interfacial transition zones (ITZ) located around the aggregate particles (Henkensiefken et al., 2009). Provided in Figure 2.2.1 is a graphical representation of the difference between external and internal curing techniques.

The water located inside the LWA is desorbed due to internal suction stresses generated as the hydrating cement particles become devoid of available water (Bentz & Snyder, 1999).

The suction pressure allows the water to be desorbed from the LWA on an as needed basis for hydration as described by ACI (ACI, 2012). The water provided from the LWA decreases the internal pressure buildup that was caused by the hydrating cement particles becoming devoid of necessary water (Holt, 2001). By providing the necessary water for hydration, many of the negative effects associated with hydrating low w/c concretes are avoided (Bentz & Weiss, 2011). The negative side effects associated with various types of drying shrinkage are provided in Section 2.3.

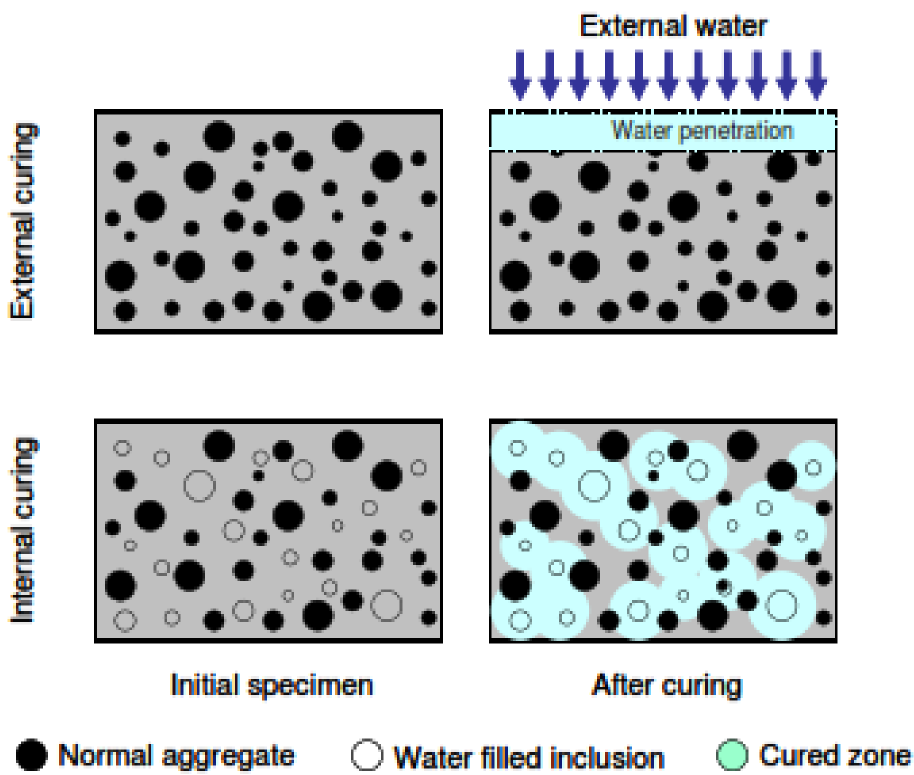


Figure 2.2.1. LWA moisture transfer to hydrating cement paste (Castro et al., 2010).

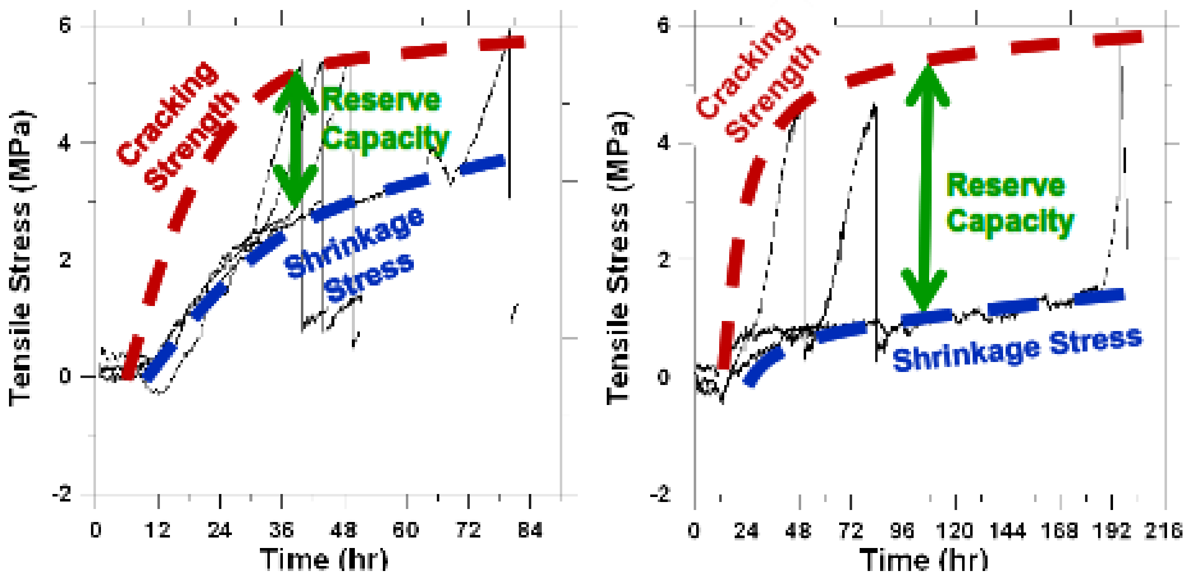
2.3 Mechanical Properties of Internally Cured Concrete from Previous Research

Studies into a curing mechanism to meet the hydration needs of high strength concrete began by utilizing various internal water sources that distributed water to the hydrating cement particles. Early studies focused on the use of pre-wetted LWA (Bentz & Weiss, 2011). This aggregate type has been the topic of many journal articles and conference papers. Coarse and fine aggregates under pre-soaked and dry conditions have been investigated (Famili et al., 2012). Because LWA's are porous and have been in commercial use in the United States since the 1940's, this product is ideal for internal curing research (Bentz & Weiss, 2011).

Numerous research agencies have investigated the effects of internal curing on the mechanical properties of concrete. However, not all of the researchers have come to the same conclusions. For instance, by using internal curing the compressive strength of concrete has increased (Roberts, 2006) and decreased (Famili et al., 2012). The increase in compressive strength was attributed to better hydration of the cement matrix (Roberts, 2006). The increase in hydration water leads to an increase in the volume of strength producing calcium silica hydrate (C-S-H) in the cement matrix of the concrete thus improving the early and later age strengths (Roberts, 2006). Other research indicated that the hardness of the LWA was an equally important, if not more important factor, than the increase in the hydration products in determining compressive strength (Bentz & Weiss, 2011). If the compressive strength of the LWA aggregate itself is considerably lower than that of the normal weight aggregate, then the compressive strength of the concrete may decrease (Bentz & Weiss, 2011). Another factor affecting the decrease in compressive strength is the amount of LWA replaced in the mixture (Bentz & Weiss, 2011). If a relatively small replacement rate was utilized, the influence on compressive strength would be equally insignificant (Bentz & Weiss, 2011).

The modulus of elasticity and tensile strength were other properties associated with internal curing of which researchers differed (Byard & Schindler, 2010). It was no surprise, however, that these properties exhibit the same results as that of compressive strength. Most strength characteristics associated with concrete function in direct relation to the compressive strength; as the compressive strength is modified the other strength properties are also modified. The modulus of elasticity for concrete is directly related to the elastic modulus of the aggregate utilized in the mix design (Byard & Schindler, 2010). The elastic modulus of kiln cooked clay and shale LWA is considerably lower than normal weight aggregates (Byard and Schindler, 2010). As such, the modulus of elasticity decreases with large increases of LWA in the mixture, despite the increase in the hydration products (Byard & Schindler, 2010).

Relative humidity, permeability, and durability are positively affected by internal curing (Golias et al., 2012). It has been well established that internal curing increases the internal relative humidity of concrete (Geiker et al., 2004). The increase in relative humidity was directly associated with the additional water located inside the concrete (Geiker et al., 2004). Similar to creating a 100 percent relative humidity zone around the concrete through external curing, internal curing creates a similar environment within the concrete (Kovler & Jensen, 2005). The increase in relative internal humidity decreases the internal suction stresses associated with the hydrating cement particles attempting to pull water from an external source (Geiker et al., 2004). Twelve days after mixing, the LWA concrete maintained a 95 percent relative humidity level compared to the reference (non-LWA) concrete that loses relative humidity much more quickly following mixing (Geiker et al., 2004). Due to the increase in available water and subsequent decrease in hydration stresses, the permeability of internally cured concrete decreases (Golias et al., 2012). This decrease occurs as the hydration products fill in the void space located in the cementitious matrix (Kovler & Jensen, 2005). With an increase in hydration products and decrease in permeability, the longevity or durability of the concrete will be greatly increased (Kovler & Jensen, 2005). Anytime the mortar matrix is densified, a subsequent increase in concrete durability is produced. The increase in durability is directly related to the increased ability of the concrete to prevent water from penetrating the exterior surface (Mehta & Kumar, 2006). Through a reduction in penetration water, the ingress of deleterious substances is also reduced (Mehta & Kumar, 2006). Provided in Figure 2.3.1 is a schematic showing the increase in reserve capacity, leading to long term durability as a result of internal curing. Reserve capacity is the buffer zone between the maximum shrinkage stress of the specimens and the cracking strength of the concrete (Bentz & Weiss, 2011). Internal curing provides water to alleviate stress buildup which decreases the shrinkage stresses providing a larger buffer between the overall shrinkage stress and the strength at which the concrete cracks (Bentz & Weiss, 2011).



A. Normal concrete shrinkage stress. B. Internally cured concrete shrinkage stress.

Figure 2.3.1 Increased reserve capacity through internal curing (Bentz & Weiss, 2011).

2.4 Shrinkage Types Associated with Early Age Concrete

2.4.1 Plastic Shrinkage

ACI defines plastic shrinkage as “shrinkage that takes place before cement paste, mortar, grout, or concrete sets” (ACI, 2012). Plastic shrinkage occurs by water being lost due to environmental factors (Shaeles & Hover, 1988). Factors such as internal concrete temperature, ambient air temperature, ambient humidity, and wind velocity contribute directly to plastic shrinkage (Mehta & Monteiro, 2006). As water evaporates from the surface of the concrete while it is setting, internal water must be drawn upon to keep the external concrete moist (Shaeles & Hover, 1988). As the internal water (bleed water) is drawn from within the concrete, tensile strains occur which may lead to cracking associated with the plastic shrinkage (Holt, 2001). Bleed water may also be drawn from the concrete by the subgrade, the formwork, or the internal aggregate (Mehta & Monteiro, 2006; Holt, 2001). According to Shaeles and Hover (1988) to avoid the deleterious effects from plastic shrinkage cracking, protection of the concrete is required. Activities such as pre-soaking the subgrade, adding wind breaks, using chilled water and chilled aggregates, or re-vibrating the concrete before it sets, aid in protecting the concrete from plastic shrinkage (Mehta & Monteiro, 2006).

The negative effects of plastic shrinkage may also be reduced through the addition of a pre-soaked medium providing water during heavy bleed water evaporation periods. Suction stresses, similar to those produced from the aforementioned hydrating cement particles following initial set, are created during evaporation of the bleed water (Mehta & Monteiro, 2006). Internal curing provides a source of internal water to alleviate the stresses developed during evaporation of the bleed water. However, the water that is desorbed from the LWA to prevent suction cannot provide water for hydration in the event that drying shrinkage occurs. Concrete experiencing large amounts of autogenous (volume change) shrinkage typically does not experience significant bleed water loss associated with plastic shrinkage. The reasoning for this is that concrete experiencing large autogenous shrinkage stresses do not contain the amount of mixing water needed to experience large plastic shrinkage losses.

2.4.2 Drying Shrinkage

As the concrete dries or hydrates, there is a decrease in the total volume of concrete compared to the product of raw materials utilized in batching (Ahmad et al., 2010). ACI defines drying shrinkage as "shrinkage resulting from loss of moisture" (ACI, 2012). Moisture may occur in concrete in multiple forms i.e. that of mix water, chemical admixtures, and water stored in the aggregate. Drying shrinkage begins as soon as the water and cement come into contact with one another (Mehta & Monteiro, 2006). Part of drying shrinkage is the loss of volume associated with the volume change in products due to the formation of the C-S-H, which begin immediately when water is added (Mehta & Monteiro, 2006). The negative effects associated with drying shrinkage are the tensile stresses generated with the change in volume (Mehta & Monteiro, 2006). As the spacing between the cementitious materials decreases due to the formation of C-S-H, there is a decrease in available water to traverse the cement matrix. As such the stresses generated by the drawing or suction of water from surrounding pore spaces increases (Holt, 2001). The stress range generated from capillary suction is on the order of 10 to 100 MPa (Holt, 2001). Chemical shrinkage and autogenous shrinkage are two types of drying shrinkage that produce stresses due to the hydrating cement matrix. Together they produce an effect known as self-desiccation which causes cracking due to the suction stresses generated. Both shrinkage types are discussed in detail below.

2.4.2.1 Chemical Shrinkage

The volume of the final product of cement mortar is less than the sum of the added parts of water and cement (Mehta & Monteiro, 2006). Chemical shrinkage is defined as the total amount of volume reduction due to the product difference of the chemical reaction in producing C-S-H (Mehta & Monteiro, 2006). Chemical shrinkage includes the shrinkage that occurs inside the pore space of the cement matrix after the concrete becomes rigid. Chemical shrinkage is sometimes referred to as "hardening shrinkage" (Tazawa et al., 1995). Before the initial set occurs, chemical shrinkage occurs, but no stress or strain is generated (Byard & Schindler, 2010). Chemical shrinkage is a continual process ongoing after the concrete sets. Chemical shrinkage following initial set is termed autogenous shrinkage (Byard & Schindler, 2010).

2.4.2.2 Autogenous Shrinkage

Autogenous shrinkage is defined by ACI as the "change in volume produced by continued hydration of cement, exclusive of effects of applied load and change in either thermal condition or moisture content (ACI, 2012). This shrinkage occurs within the mortar matrix when the concrete is in a sealed environment or closed system with no loading (Holt, 2001; Mehta & Monteiro, 2006). Chemical shrinkage and autogenous shrinkage produce the same effect while the concrete is in an unrestrained form (Byard & Schindler, 2010). The meaning of unrestrained does not imply reinforcing steel, but rather the state of the hydrating mortar matrix. An unrestrained state is one in which the C-S-H have not sufficiently filled the mortar matrix to impede the volume reduction of the concrete. As the C-S-H fill in the mortar matrix, the concrete becomes rigid eliminating continued length change associated with volume change (Tazawa et al.,1995); although hydration of the concrete continues. Chemical shrinkage is the total volume reduction of the chemical reaction for C-S-H, including both the total volume reduction and the pore space reduction of the cement matrix (Tazawa et al., 1995). Autogenous shrinkage is solely the volume reduction associated with the change of length of the concrete specimen (Tazawa et al., 1995). Another term commonly associated with the volume change associated with this length change is the "macroscopic" reduction in volume of the concrete structure (Holt, 2001). Figures 2.4.1 and 2.4.2 illustrate the difference chemical and autogenous shrinkage as related to the hydration of the cement. Specifically the type of shrinkage associated with the volume difference of the concrete is provided in Figure 2.4.1, while the shrinkage associated with each phase of hydration is provided in Figure 2.4.2.

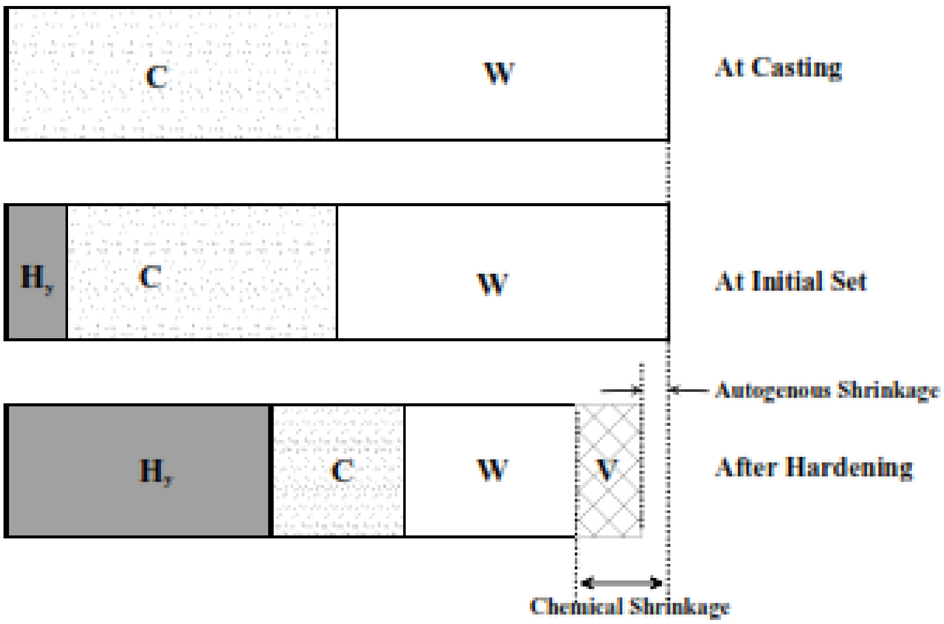


Figure 2.4.1 Autogenous & chemical shrinkage volume difference (Holt, 2001).

Where:

C = Unhydrated Cement

W = Unhydrated Water

H_y = Hydration Products

V = Voids Generated by Hydration

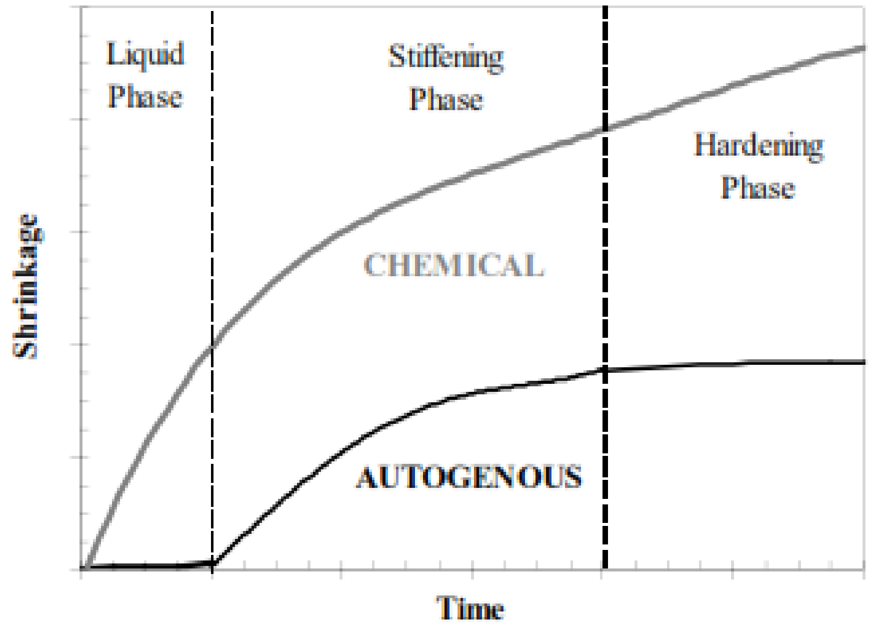


Figure 2.4.2. Autogenous & chemical shrinkage difference by phase (Holt, 2001).

2.4.3 Thermal Shrinkage

The chemical reaction resulting in the formation of the C-S-H generates heat (Mehta & Monteiro, 2006). In smaller structures with a high surface to volume ratio, the heat of hydration is dissipated without any adverse effects (Mehta & Monteiro, 2006). Concrete structures that are relatively large (approximately 3 feet or greater in all directions) or utilize large amounts of cement may encounter heat of hydration issues in the form of thermal shrinkage (Gajda & Vangeem, 2002). Concrete of this scale is referred to as mass concrete (Gajda & Vangeem, 2002). As the depth of a concrete member increases, the heat generated from hydration is unable to dissipate to the environment (Mehta & Monteiro, 2006). As the internal heat increases, stresses form along the thermal transition zones of the cooling concrete (Mehta & Monteiro, 2006). The cooling of the concrete induces tensile stresses especially along the weak ITZ of the cement mortar and the aggregate (Mehta & Monteiro, 2006). The tensile stresses cause microcracking which affect durability, and in extreme cases may decrease the compressive strength of the concrete (Mehta & Monteiro, 2006). However mass concrete is not the only concrete members that must be mindful of thermal issues.

Thin section members are also at risk for thermal effects associated with concrete placement. Though thin members do not generate as much heat as mass concrete, however they are susceptible to thermal gradient cracking (ACI 224.1r, 2007). When placing thin sections such as concrete bridge decks, changing exterior temperatures may cool one side of the member more rapidly than the opposite side of the member (ACI 224.1r, 2007). This creates a differential volume change which produces tensile stress in the concrete along the thermal gradient (ACI 224.1r, 2007). These tensile stresses can produce cracking in the concrete leading to shorter lifespans of the concrete members (ACI 224.1r, 2007). Serious distortion of concrete members may occur if the stresses generated by thermal gradients are not accounted for (ACI 224.1r, 2007). Protecting concrete from all forms of thermal shrinkage and gradients is vital to the success of long term durability of concrete members.

2.5 Lightweight Aggregates

Light-weight aggregates (LWA) are those that have a lower specific gravity than traditional aggregate sources. ACI defines LWA as an "aggregate of low density, such as: (a) expanded or sintered clay, shale, slate, diatomaceous shale, perlite, vermiculite, or slag; (b) natural pumice, scoria, volcanic cinders, tuff, and diatomite; or (c) sintered fly ash or industrial cinders used in lightweight concrete" (ACI, 2012). LWA's have a network of internal pores which decrease their specific gravity from normal weight aggregates. It is the coarse porous network that provides the ability of LWA to internally cure concrete (Bentz & Weiss, 2011). The LWA is able to absorb a large amount of water with respect to its initial volume. Many of the LWA's in use today are able to absorb between five and 25 percent of their total volume in water (Hoff, 2002). The water located inside this internal pore structure is then desorbed as needed from the suction produced by the hydrating cement grains (Hoff, 2002). The larger pores of the saturated LWA contain the lowest molecular forces, allowing their water to be released before smaller pore openings (Bentz & Snyder, 1999). The minimum pore size necessary for desorption of water is 100 nm (Byard & Schindler, 2010). The desorption capacity of LWA varies according to pore size distribution with each different LWA (Byard & Schindler, 2010). It is preferable to use LWA's that release 90 percent or more of absorbed water (Bentz & Weiss, 2011). If the chosen LWA does not release most of their absorbed water, then a larger volume of LWA must be placed in the mixture (Bentz & Weiss, 2011). This higher LWA content will hydrate the necessary cement particles (Bentz & Weiss, 2011). However, there may be other deleterious effects such as reduced compressive strength and lower modulus of elasticity associated with the increased LWA content (Bentz & Weiss, 2011).

Manufactured LWA's are those produced through an industrial process other than what occurs in nature. Many LWA's utilized today are produced in this capacity as the process can provide assurance of the mechanical properties for the given LWA (Byard & Schindler, 2010). The raw material, whether clay or slate, is mined and brought to a processing plant (Byard & Schindler, 2010). At the plant, the material is heated until it is in a plastic state (Byard & Schindler, 2010). Gases form within the mineral during the heating process, which provide a pore structure during cooling (Byard & Schindler, 2010). It is important to note that this pore structure within the LWA is not completely connected (Byard & Schindler, 2010). The disconnected pore structure allows the concrete to maintain its low permeability despite containing a highly porous aggregate. The trapped gases create voids or pores allowing the LWA to absorb and desorb water for use in internal curing (Byard & Schindler, 2010).

Other sources of internal curing mechanisms include super absorbent polymers, wood pulp, sintered fly ash, and expanded slag (Byard & Schindler, 2010). Internal curing mechanisms such as super absorbent polymers and wood pulp provide no structural performance for the concrete (Byard & Schindler, 2010). Due to the availability and structural integrity of LWA over that of other internal curing sources, LWA's are often the material of choice for real world applications of internal curing (Byard & Schindler, 2010).

2.5.1 Expanded Clay LWA

LWA's produced through a manufacturing process have predictable properties guaranteed by the quality assurance program set forth by the manufacturer. Expanded clay is one such material that is produced in a rotary kiln manufacturing process (Jensen & Lura, 2006). The gases produced during the formation of expanded clay leave behind a highly porous structure (Jensen & Lura, 2006). Kiln cooked expanded clay has measured porosities of up to 90 percent (Jensen & Lura, 2006). However, the water absorption potential is considerably lower than 90 percent. The disconnection of pores leads to the considerably lower absorption values compared to the 90 percent pore structure (Jensen & Lura, 2006).

2.5.2 Expanded Shale LWA

Similar to expanded clay, expanded shale is produced in the same process. Rotary kilns heat crushed slate to nearly 1200^{o} C until they reach a plastic state (Jensen & Lura, 2006). The process for manufacturing LWA is shown in Figure 2.5.1. Gases entrapped during this process provide the necessary pore structure for internal curing (Jensen & Lura, 2006). The pores produced in manufactured slate, such as Stalite, are smaller than pores produced in similar LWA materials (Jensen & Lura, 2006). Due to the smaller pores, expanded slates, such as Stalite do not release imbibed water as easily as clays (Jensen & Lura, 2006). Instead, they may hold a certain percentage of water until the relative humidity drops below 70 percent (Jensen & Lura, 2006).

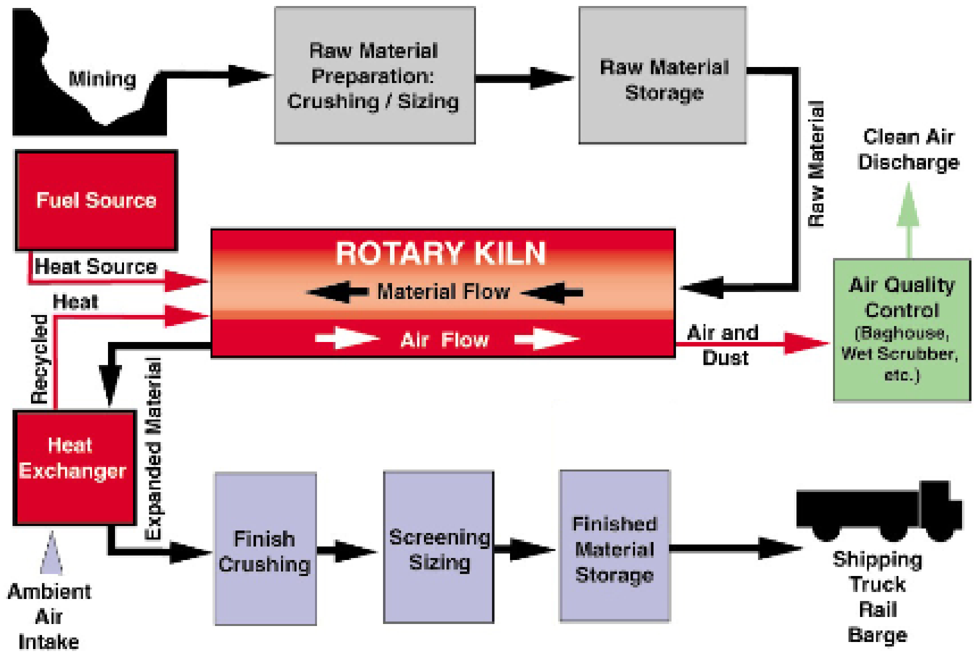


Figure 2.5.1. Maufactoring process of expanded LWA (Byard & Schindler, 2010).

2.5.3 Superabsorbent Polymers

Superabsorbent polymers (SAP) are substances that may absorb up to 5000 times their initial weight of liquid in certain applications (Jensen, 2013). Through a connection of similar linked molecules, SAP's retain the ability to not only absorb large amounts of water, but to release that absorbed water to hydrating cement particles (Kolver & Jensen, 2005). It is the combination of absorption and release that makes SAP's valuable to concrete mix design (Jensen, 2013). During hydration, the cement matrix utilizes available mix water, but requires more water for complete hydration of low w/c concrete (Bentz & Weiss, 2011). Similar to LWA's, SAP's are able to desorb their absorbed water as needed for complete hydration of the cement matrix (Jensen, 2013).

There are three negative side effects associated with utilizing SAP's in concrete mix designs (Jensen, 2013). The first negative effect is related to the structural capacity of SAP's (Jensen, 2013). They are not able to carry load, and as such reduce concrete strength if utilized in large amounts (Jensen, 2013). The second negative effect is the extra porosity produced once the internal curing water has been desorbed (Jensen, 2013). If utilized in sufficiently low quantities, the increased hydration products offset the strength loss due to the SAP's inability to carry load and extra porosity (Jensen, 2013). Concrete strength can increase depending on the amount of SAP's, and reduced cracking associated with self-desiccation (Jensen, 2013). The extra porosity may even be beneficial i.e. aiding in freeze thaw resistance of the concrete (Jensen, 2013). The third downfall associated with SAP's is the cost (Jensen & Lura, 2006). Compared to other internal curing mediums, SAP's are more expensive, and are predominantly utilized in other industries requiring large absorption capacities (Jensen & Lura, 2006).

2.6 Infiltration Depth of Internally Cured Water

An important aspect of internally cured concrete is the depth the water penetrates into the cement matrix. If the water is unable to reach all hydrating cement paste, then it does not accomplish its full potential of internally curing the concrete during hydration. Henkensiefken et al. (2009) examined the penetration depth of water leaving a saturated lightweight aggregate. Henkensiefken et al. (2009) determined the penetration depth of water from a saturated lightweight aggregate to be 2 millimeters using X-Ray absorption techniques.

Further research was conducted on the depth that internal curing water was able to reach during hydration of the cement particles. The new research utilized neutron tomographies to determine the distance traversed by the water through the cement matrix (Bentz & Weiss, 2011). This research provided a depth of 3 millimeters traveled by the water after leaving the LWA

(Bentz & Weiss, 2011). The significance of this research and others similar to it is that it provides a basis for the size of the LWA used for internal curing. If the aggregate size is too large to provide adequate dispersion within the concrete matrix, then all the cement particles may not hydrate (Henkensiefken et al., 2009). Therefore, aggregate size and distance traveled by the water from the LWA directly affect the amount of shrinkage mitigated through using internal curing (Henkensiefken et al., 2009).

Another aspect concerning the distance traveled by the water in the cement matrix is the age of the concrete. Early in the hydration stage of cement, the water can travel further distances than at later ages (Bentz & Weiss, 2011). As the hydration products fill in the void space between the mortar and aggregate, the travel lanes become closed. As such, the distance traveled by water during the each stage of hydration changes based on reaction product volume (Bentz & Weiss, 2011). Provided in Table 2.6.1 is a table of estimated travel distances based on reaction product growth as expressed in concrete age.

Table 2.6.1. Estiamted water travel distance during hydration (Bentz & Weiss, 2011).

Hydration Age	Estimated Travel Distance of Water
Early (i.e., < 1 day)	20 mm
Middle (i.e., 1 day to 3 days)	5 mm
Late (i.e., 3 days to 7 days)	1 mm
Worst Case (i.e., > 28 days)	0.25 mm

The w/c and curing conditions have the potential to affect the travel distance of water from the LWA (Bentz & Weiss, 2011). As the w/c decreases there is an increased need for curing water to reach full hydration of the cement (Bentz & Weiss, 2011). If the w/c is high enough to provide adequate water for complete hydration, suction stresses are not generated by the hydrating cement. This leads to shorter travel distances from water leaving the LWA. Little internal curing research has not been performed for concrete mixtures at the proposed w/c of

0.44, and different w/c have the potential to change the distance traveled by the water upon leaving the LWA (Bentz & Weiss, 2011). Similarly, if the concrete is cured in an environment where water is not lost to the atmosphere, the distance traveled by water leaving LWA is different than that from exposed drying conditions (Bentz & Weiss, 2011). The exposed curing conditions lead to shorter travel distances as the void space is more quickly filled during hydration (Bentz & Weiss, 2011).

2.7 LWA Replacement Rate

To establish a complete understanding of the internal curing process, a proper coarse LWA replacement rate has been determined to enable complete hydration of the cement to mitigate shrinkage. The proper coarse LWA replacement rate ensures that excessive expenses associated with using LWA are mitigated. The time required for adequate soaking must also be determined to ensure complete saturation of the coarse LWA. Complete saturation of the LWA is needed to ensure that the minimal amount of LWA is added for hydration of the cement. By utilizing the minimal amount of LWA, the overall cost associated with LWA is mitigated to the fullest extent. It also mitigates the effect of the LWA on the compressive strength of the LWA concrete.

Replacement rate equations such as Equation (3-1), as presented by Bentz et al. (2005), provided a needed baseline for complete hydration. However, the equation is developed to calculate the amount of LWA fine material required for hydration as opposed to the amount of coarse LWA (Bentz et al., 2005). The other issue concerned with the Equation (3-1) is the w/c utilized to produce complete hydration. Previous research does not agree that a w/c of 0.36 produced complete cement hydration (Delatte & Cleary, 2008). The typical range of w/c required to produce cement hydration is between 0.36 (Bentz et al. 2005) and 0.42 (Delatte & Cleary, 2008).

2.8 Internal Curing in Practice

When a new idea is presented to the engineering community, there are ample opportunities for research, but in many instances these ideas do not become reality. It is exciting to know that internal curing is not only a research topic, but an area that is in practice as well. A ready mix company located in Dallas, TX, TXI, successfully utilizes internally cured concrete for construction of municipal and residential designs (Villarreal & Crocker, 2007). Due to the environmental conditions in northern Texas, there are many applications for concrete pavements (Villarreal & Crocker, 2007). These concrete pavements were improved through the use of internal curing as stronger concrete with improved workability and less cracking was reported (Villarreal & Crocker, 2007). The improvement came through an increase in the density of hydration products around the steel reinforcement (Daigle et al., 2008). The increased hydration products mitigate the ingress of deleterious substances that corrode reinforcement (Daigle et al., 2008). The pavements in north Texas are not high strength concrete, which is the focus of most internal curing research, rather internal curing is used to increase durability and longevity of normal strength concrete (Villarreal & Crocker, 2007). The internally cured concrete of north Texas used a replacement rate of 5 ft^3/yd^3 of LWA for the normal weight aggregate (Villarreal & Crocker, 2007). More than 2,600,000 yd^3 (2,000,000 m^3) of internally cured concrete has been placed for commercial use in the northern Texas area (Bentz & Weiss, 2011). The internally cured concrete has reduced the number and size of the cracks associated with placement of the concrete when compared to the standard TxDOT concrete mix design (Bentz & Weiss, 2011).

Texas is not the only state, however, to use internally cured concrete for commercial applications. States such as Indiana, Ohio, and New York have successfully applied internal curing technology (Bentz & Weiss, 2011). Indiana utilized internal curing in a comparison study of two concrete bridge decks (Bentz & Weiss, 2011). Two box girder bridges were built simultaneously with two different decks constructed for each bridge (Bentz & Weiss, 2011). One bridge was constructed with the standard concrete mixture used by the Indiana highway department while the other was constructed of an internally cured concrete mixture (Bentz & Weiss, 2011). The results are still extremely preliminary as no noticeable differences have been observed between the two decks (Bentz & Weiss, 2011). Ohio has employed internal curing in conjunction with its silica fume, high performance mixture (Bentz & Weiss, 2011). It is placed as a topping for bridge decks and reported equivalent or increased compressive strengths and maintained the entrained air (Bentz & Weiss, 2011). New York has successfully applied internal curing in nine concrete bridge deck toppings (Bentz & Weiss, 2011). Strengths associated with these decks have produced mixed results, as some decks have an improved strength over traditional mixes while others lowered strengths (Bentz & Weiss, 2011). Though there are reported instances of lower strengths, the NYDOT reports no complaints against internally cured concrete (Bentz & Weiss, 2011).

2.9 Extension of Knowledge

A review of the literature reveals many researchers have examined the effects of pre-soaked LWA on the mechanical properties of concrete. However, research to determine the optimal soaking time for coarse LWA has not been identified. There are many articles related to the optimal mix proportions of LWA to reduce the deleterious effects, but few addresses the optimal LWA soaking time to reduce concrete shrinkage. Any change in moisture content and the resulting difference in concrete shrinkage should be evaluated. Prior research looks directly at mitigating the self-desiccation brought about from autogenous shrinkage at w/c's lower than 0.42. Much less research has been conducted on concrete with a w/c larger than 0.42, and most bridge decks are cast with w/c's greater than 0.42. Therefore, the use of coarse lightweight aggregate is reasonable due to the unknown nature of the travel distance of the water associated with a w/c of 0.44. An optimal soak time ensures the coarse LWA is fully saturated allowing the minimal amount of LWA to be utilized in the concrete mixture. By minimizing the amount of LWA added to the mixture, a reduction in the negative side effects associated with LWA mixtures will be obtained. The proposed research into soaking durations will aid in eliminating this gap in the literature. Proposed mixtures include a variation of soak times (1, 3, and 7 days). The intent of the research is to produce results that will aid concrete producers in their internal curing implementation programs furthering the use of internally cured concrete.

Chapter 3 Research Methodology

3.1 Methodology Overview

The intent of the current research is to quantify shrinkage reduction through the use of internally cured concrete using different soaking time frames (1, 3, and 7 days) for LWA. To quantify shrinkage reduction, a methodology for analyzing concrete specimens must include a quantifiable measurement of shrinkage. Current practice techniques include the use of a length change comparator and scanning electron microscopy to determine shrinkage reduction (Bentz & Weiss, 2011). Another method to quantify the shrinkage reduction is to calculate the amount of water needed to hydrate the cement (Bentz et al., 2005). With defined shrinkage quantities, shrinkage results from internally cured concrete may be compared to a traditional limestone control mixture to determine the amount of shrinkage reduction. All methods compare a control mixture (i.e. non-internally cured concrete) to an internally cured concrete. The methodology used for this research program builds upon methods already in current practice.

Current practice techniques such as using a length change comparator and others have been established for measuring the amount of shrinkage within internally cured concrete. However, the majority of concrete used as the control mixture is for concrete susceptible to self-desiccation. Self-desiccation occurs when available mix water is not sufficient to completely hydrate the cement (Bentz and Weiss, 2011). A review of the literature reveals that any mixtures with a w/c greater than 0.36 (Bentz et al. 2005) to 0.42 (Delatte and Cleary, 2008) contains sufficient water to hydrate all cement particles. This research program looks at establishing the amount of shrinkage reduced with internal curing of concrete specimens developed with a w/c of 0.44. Based on the review of the literature a w/c of 0.44 contains an adequate amount of mix water to hydrate the cement. It is known that concrete shrinks due to drying shrinkage even when enough water is in the mixture for complete hydration (Mehta & Monteiro, 2006). This shrinkage may cause drying shrinkage cracking due to the product volume change. It is more noticeable in heavily reinforced sections due to the extra restraint confining the concrete from shrinking. Given that the concrete shrinks, this research provides an option to reduce internal stresses due to drying shrinkage through the use of an internal curing aggregate.

3.2 Methodology of Mix Design Development

3.2.1 LWA Moisture Content

To begin the research, an understanding of the expanded clay and shale properties was needed to aid in the development of the concrete mix designs. The LWA was added to the concrete in a saturated non-surface dry condition. For applicability of the research to real world solutions, the amount of water located on the outside of the aggregate was necessary to calculate the w/c. To aid in establishing the amount of water located on the outside of the LWA, previous research at the University of Arkansas (Floyd, 2012) was referenced. Previous research developed lightweight self-consolidating concrete mixtures using lightweight clay and shale aggregates. This research established average total moisture contents for the LWA in a non-surface dry condition. The lightweight self-consolidating concrete research revealed average saturated non-surface dry expanded clay and shale aggregate moisture contents of 26 and 22 percent. The absorption capacity of expanded clay provided by Old Castle was 15 percent and expanded shale provided by Buildex was 12.9 percent. With known moisture contents and absorption capacities for the LWA, excess water was accounted for in the mix design w/c.

To verify the previous work performed at the University of Arkansas, tests were conducted to check the moisture contents of the LWA. This began by taking the LWA moisture content to zero. The aggregate was placed in an oven at 350^{o}F ± 9^{o}F for a minimum of 24 hours.

Some highly saturated aggregate showed visible signs of moisture after 24 hours of oven drying, and was placed in the oven for another 24 hour drying period. With the aggregate at zero percent moisture, the soak duration began. Three soak durations were chosen to determine the LWA's absorption capacity within a specified time frame. The three soak durations were 1, 3, and 7 days. The LWA was placed in water tight containers and soaked for the specified time duration. Once the aggregate completed its specified soak time, the aggregates were drained and a sample was collected. The sample was weighed in its saturated non-surface dry condition and placed in the oven for 24 hours to dry. Once the aggregate dried, a dry weight was taken and the total moisture content was calculated. With known absorption capacities for both the clay and shale, excess moisture was determined and accounted for in the w/c. Results from the LWA moisture content testing program are provided in Table 3.2.1.1. Provided in Table 3.2.1.2 are the LWA moisture contents measured during batching of the concrete for comparative purposes. From Tables 3.2.1.1 and 3.2.1.2, it was determined that the moisture content of the LWA varied during individual testing.

Table 3.2.1.1. Measured LWA moisture contents.

Measured LWA Moisture Contents		
	Clay (%)	**Shale (%)**
1 Day	27	19
3 Day	25	18
7 Day	28	23

Table 3.2.1.2. Mixture LWA moisture contents.

Mixture LWA Moisture Contents		
	Clay (%)	**Shale (%)**
1 Day	24	17
3 Day	28	18
7 Day	21	24

The results provided in Tables 3.2.1.1 and 3.2.1.2 yield insight into the behavior of pre-soaked non-surface dry LWA. The moisture contents were typically within 7 percent of one another when comparing similar soak times. This provided justification for the standardization of the assumed LWA moisture content throughout the research program. The determination was based on the data provided in Tables 3.2.1.1 and the data provided from the previous lightweight self-consolidating concrete mix designs developed at the University of Arkansas (Floyd, 2012). The lightweight self-consolidating concrete mix designs used the same aggregate source for both the lightweight clay and lightweight shale as the internally cured concrete mix designs. Due to the larger amount of data for the lightweight self-consolidating concrete, it was determined to use the same moisture contents established for the expanded clay and expanded shale. The average saturated non-surface dry moisture content for expanded lightweight clay and shale was 26 and 22 percent.

3.2.2 Mix Design Development

With average saturated non-surface dry moisture contents established for each of the LWA's, mix designs were developed for the different LWA sources. A control mix design containing no coarse LWA was designed for comparing the shrinkage mitigation of the LWA concrete. The concrete mix designs were developed in conjunction with the Arkansas State Highway and Transportation Department (AHTD) concrete bridge deck specification. The AHTD concrete bridge deck specification contained four requirements applicable to the research program. The four requirements were as follows: (1) Minimum of 611 lbs/yd^3 of cement, (2) Slump of 1 - 4 inches, (3) Water-cement ratio of 0.44, and (4) Minimum compressive strength of 4000 psi at 28 days.

The quantity of cement used in each mix design was 611 lbs/yd^3. Ash Grove portland cement Type I/II was used for all mixtures. As cement is the most expensive ingredient in concrete, it is assumed for real world applications that the minimum amount of cement is used for mix designs. With the amount of cement established the amount of water for each mix design was calculated to be 270 lbs/yd^3 using a w/c of 0.44. With the two mix design requirements met for the AHTD specification, the coarse aggregate was set to 1700 lbs/yd^3 and sand was used to fill the remaining cubic yard. The air content was designed at 2 percent per cubic yard. No entrained air was used in the development of the mix designs. ADVA Cast 575 was used as a high range water reducer to ensure workability of the mix design.

The aforementioned mix design was the basis for the LWA mix designs. A set replacement rate of LWA was used to ensure comparable results. The replacement rate was based on the equation provided by Bentz et al. (2005). Utilizing Equation (3-1), a replacement rate of 265 pounds of clay LWA and 308 pounds of shale LWA was calculated for each LWA type. Therefore, a replacement rate of 300 lb/yd^3 of LWA was utilized for each LWA in the investigation. Based on this replacement rate, the mix design using LWA replaced 300 pounds of coarse limestone aggregate with 300 pounds of LWA. The cement content, w/c, and total water content remained the same as the control mixture while the coarse limestone was reduced to 1400 lbs/yd^3. ADVA Cast 575 was used as a high range water reducer to ensure workability of the concrete. Mix designs remained constant throughout testing for each aggregate type; however, the soaking time frames of the LWA concrete mixtures were varied. Saturated surface dry mix designs for both the control and LWA mixtures are provided in Table 3.2.2.1. With mix designs established for both the control and LWA mixtures, batching of internally cured concrete began.

$$M_{LWA} = \frac{c_f \text{ x } CS \text{ x } \alpha_{max}}{S \text{ x } \phi_{LWA}}$$

Equation (3-1) LWA fines replacement rate (Bentz et al., 2005).

M_{LWA} = Max amount of LWA (fines) for cement hydration (lb/yd^3)

C_f = Cement factor (content) for concrete mixture (lb/yd^3)

CS = Chemical shrinkage of cement (lb/lb)

α_{max} = Maximum expected degree of hydration of cement

S = Degree of saturation of aggregate (0 to 1)

ϕ_{LWA} = Absorption of LWA (lb/lb)

Table 3.2.2.1. Mix designs for control and LWA mixtures.

Mixture Proportions: Saturated Surface Dry Condition			
Mix Design	**Control**	**Clay**	**Shale**
Cement (lb/yd^3)	611	611	611
Coarse Aggregate (lb/yd^3)	1700	1400	1400
Fine Aggregate (lb/yd^3)	1440	1107	1178
LWA Aggregate (lb/yd^3)	0	300	300
Water (lb/yd^3)	270	270	270
w/cm	0.44	0.44	0.44
HRWR ADVA 575 (oz/cwt)	4	4	4

3.3 Batching Process

Batching of the internally cured concrete and control mixture began with amassing the necessary aggregate to perform the intended research. Moisture contents of the coarse and fine aggregate were taken to determine the amount of moisture contained on the outside of the aggregate. The moisture content was determined by similar methods used to determine the moisture content of the LWA. A sample was obtained from the bulk aggregate. The sample was weighed in its moist condition and placed in an oven to dry for a minimum of 24 hours. To ensure the moisture content was not changed in the bulk aggregate during the 24 hour drying period of the sample aggregate, all necessary batch aggregate was obtained at the same time of the moisture content sample. The bulk aggregate necessary for batching was placed in water tight containers with lids to ensure moisture loss was prevented. By measuring the moisture content of the aggregate, the w/c was accurately accounted for in the mix designs. The moisture content of the LWA was assumed to be 26 percent for the expanded clay and 22 percent for the expanded shale as previously covered in Section 3.2.1. To saturate the LWA, it was placed in buckets with water and sealed with lids for the intended soak duration. The excess water was then removed from the LWA by placing a perforated lid on the bucket and allowing the water to escape as shown in Figure 3.3.1. Moisture content testing and batch weights of the LWA were obtained, once the excess water had been removed from the LWA. Once moisture contents were obtained the aggregates were weighed to match mix design and batch size. Cement was provided in 92.4 lbs bags and weighed as needed depending on individual batch size. Mixing water was obtained based on mixture proportion requirements and the aggregate moisture content.

Figure 3.3.1. Removal of excess water after soaking duration for LWA.

Batching began by placing all coarse aggregate with half the mixing water in the mixer, and then turning the mixer on. With the coarse aggregate and water mixing, the sand, cement, and remaining water with the high range water reducer were added to produce concrete. The concrete mixture was continuously mixed until all constituents were uniformly combined. Mixing of the concrete was conducted in accordance with ASTM 192/C192M-07. A deviation occurred from ASTM 192/C192M-07 in that the concrete was not allowed to mix 3 minutes followed by a 3 minute rest followed by a final 2 minute mixing period. The concrete was continuously mixed until it achieved a uniform consistency. See Figure 3.3.2 for photographs of the concrete mixer and the mixing concrete. The exact time frame for mixing varied depending on the batch size for a given specimen. Once the concrete was thoroughly mixed, it was transported to a position where testing of the fresh concrete properties was performed.

Figure 3.3.2. Concrete mixer and the mixing concrete.

3.4 Measurement of Fresh Concrete Properties

With the concrete in a plastic state, the fresh concrete properties were measured. For comparison of the control mixture and the internally cured concrete, the workability and unit weight were measured for each concrete mixture. Workability of the concrete was measured by the slump test. It provided a comparison to determine if the LWA affected the ability to place the concrete or the flow of the internally cured concrete when compared to the control mixture. Slump tests on the fresh concrete were conducted by following ASTM C143/C143M-05a. The slump tests results are provided in in Table 4.3.1.

Knowing the LWA had a lower unit weight than the coarse limestone, it was expected that the unit weight of the concrete would be lower in the internally cured specimens compared to that of the control mixture. Concrete is classified as normal weight and lightweight by ACI 318-08 based on the unit weight of the specimen. ACI 318-08 defines normal weight concrete as having a density in the range of 135 – 160 lb/ft^3, while lightweight is defined as having a unit weight in the range of 90 - 115 lb/ft^3 (ACI, 2008). The unit weight of concrete was determined to establish which code guidelines would govern its development when used in field applications. The unit weight also provided insight into the strength and durability of the concrete. Unit weight testing was conducted in accordance with ASTM C138/C138M-07. Unit weight results are provided in Table 4.3.1.

3.5 Measurement of Hardened Concrete Properties

3.5.1 Compression Testing

With the workability and unit weight measurements recorded, specimens were cast for testing the hardened properties of the concrete. For each mixture proportion, twelve cylinder specimens which measured 4 inches in diameter by 8 inches in depth were cast for compression testing. Prior to casting, the cylinder molds were sprayed with a lubricant which ensured easy form removal. Concrete cylinder casting and curing was conducted in accordance with ASTM C192/C192M-07. A deviation from ASTM C192/C192M-07 occurred during the making of the concrete cylinders. The cylinders were tapped more than 15 times on the outside of the form to ensure consolidation in the low slump mixtures. A Forney compression machine model F-400F-LC1 was used to conduct compression testing. Three cylinder specimens were tested in compression at 1, 7, 28, and 56 days of age. Compression strength testing was conducted in accordance with ASTM C39/C39M-05e1. A deviation from ASTM C39/C39M-05e1 was that diameter measurements of specimens were not recorded during testing. The specimen molds were not deformed during use, and no diameter measurements were taken. Compression strength test results are found in Section 4.3 in Table 4.3.1. Provided in Figure 3.5.1.1 are images of unbroken and broken concrete specimens.

Figure 3.5.1.1. Unbroken and broken concrete cylinders.

3.5.2 Shrinkage Testing

Shrinkage testing was conducted using two methods. The first method used a length change comparator. The second method used embedded concrete strain gages. Both types of testing utilized steel form work which produced specimens that measured 4 inches by 4 inches by 10 inches. Similar consolidation techniques were utilized for both testing techniques.

3.5.2.1 Drying Shrinkage Testing Using a Length Change Comparator

Shrinkage testing using the Forney length change comparator began with casting the specimens. Four specimens or prisms were cast using the formwork for each mix design. All formwork was coated in a form release spray to aid in de-molding. Gage studs were placed into the end of the formwork and became part of the specimen following set. The gage studs aided in measuring the amount of length change throughout the specimen. Specimens were placed and consolidated in accordance with ASTM C192/C192M-07.

Upon consolidation and finishing, the concrete prisms were stored inside the formwork in a greater than or equal to 50 percent humidity environment for 24 hours ± 30 minutes. The top of the specimens was open to the ambient air inside the environmental chamber. Following the 24 hour setting period, the specimens were removed from the formwork. Each specimen was identified with the batch date, LWA replacement rate, LWA soak time, and specimen number. Once the specimens were removed from the formwork, an initial reading was taken and this reading became the baseline for subsequent measurements. Testing was performed in accordance with ASTM C490-04 except that median readings were recorded from the dial instead of the lowest dial reading. The four specimens from each mix design were measured twice to ensure an accurate reading, and then were returned to the environmental chamber for continued curing. The specimens were stored on rollers inside the environmental chamber to ensure unrestrained shrinkage. The environmental chamber was maintained at a 50 ± 4 percent humidity environment with a temperature of 73°F ± 3°F in accordance with ASTM C157/C157M-06.

With an initial length recorded for 1 day of curing, subsequent shrinkage measurements were taken to determine the effectiveness of the LWA at mitigating drying shrinkage. Testing continued for 112 days, with readings taken at 1, 7, 14, 21, 28, 56, 90, and 112 days. With known lengths for the concrete prisms throughout the first 112 days, shrinkage measurements were calculated for each prism. The two data points for each time measurement were averaged and subtracted from the initial length. The initial length was expected to be the longest recorded length of each prism. Strain was calculated by dividing the length change by the gage length (which was 10 inches). Shrinkage results using the length change comparator are provided in Section 4.2.1 in Figures 4.2.1.1 and 4.2.1.2. Provided in Figure 3.5.2.1.1 are pictures of the Forney length change comparator used during the internal curing research.

Figure 3.5.2.1.1. Forney length change comparator.

Drying shrinkage began as the C-S-H began to form, which is typically during the first 24 hours. Therefore, some concrete shrinkage was not measured when the length change comparator was used because the specimens remained in the forms for 24 hours. However, the length change comparator did provide an opportunity to examine the shrinkage mitigation potential of the coarse clay and shale LWA. By comparing the length change of the mixtures containing LWA to the control mixture, the length change comparator showed the reduction in shrinkage attained through the use of the LWA.

3.5.2.2 Drying Shrinkage Testing Using Vibrating Wire Strain Gages

Similar to drying shrinkage testing using a length change comparator, drying shrinkage testing with vibrating wire strain gages began with specimen production. Each specimen contained a strain gage at the center of the prism in all directions. The gage wires extruded from the concrete and attached to the data collection system.

Curing of the concrete using vibrating wire strain gages began immediately following placement. Each concrete form produced two concrete specimens. As mentioned previously, drying shrinkage during the first 24 hours was not recorded using the length change comparator. In the strain gage specimens, gage studs were not placed in the specimens as the strain gages recorded measurements instead of taking an external reading. This allowed the first 24 hours of shrinkage data to be recorded. To ensure plastic shrinkage was mitigated, the specimens were sealed. By mitigating plastic shrinkage, all shrinkage during the first 24 hours was drying or chemical shrinkage. This removed excess variables due to possible air changes from entering and exiting the environmental chamber. To seal the concrete, polyurethane wrapping was placed around the specimen and formwork. The wrapping covers all sides of the formwork, and was not removed until 24 hours ± 30 minutes following placement.

Upon completion of the 24 hour setting time, the polyurethane sheeting was removed. The concrete formwork was removed with careful consideration not to disturb the sample. However, sample disturbance was noticed in the data as excessively large strain due to impact from a rubber mallet which aided in form removal. Errant data due to form removal was removed from the data. Once removed from the formwork, each specimen was given a numerical value to correlate it with the matching strain gage. Each specimen also had the batch date, LWA soak time, and the LWA replacement rate recorded on its end. These inscriptions were made to aid during data analysis for the determination of strain associated with each strain gage. Similar to the previous testing technique, the specimens were stored on rollers to aid in unrestrained shrinkage. The specimens were cured in the same environment as the previous testing method.

The use of embedded vibrating wire strain gages provided an opportunity to collect more data than the length change comparator testing. The data collection system used Geokon model number 4200 concrete embedment strain gages. The gages measured direct strain and the internal temperature of the concrete. The strain gages had the capacity to measure up to 3000 με. The strain gages had the resolution of 1.0 με and had a gage length of 6 inches. The gages had the ability to read an active temperature range of -20^{o}C to +80^{o}C.

The gages were connected to a data collection system capable of taking readings for a 16 channel vibrating wire strain gage configuration. A Campbell Scientific CR10X data collection system was utilized for collecting the temperature and strain gage measurements. The data was collected from the CR10X data collection system through the use of Campbell Scientific's software PC400. The data was collected as a dat file, which was then converted to a text file and subsequently imported and analyzed in Excel. Graphs representing strain vs. time were generated for the data.

Unlike the length change comparator tests, the vibrating wire strain gages recorded strain directly. There was no conversion associated with the data analysis, only that initial strain must be zeroed with respect to the zero time interval. With the data presented in strain, a plot of the strain versus time was produced. This provided an opportunity to determine quantitatively the amount of shrinkage reduction through the introduction of LWA as an internal curing mechanism. Similar to length change comparator testing, a control specimen was cast to compare the shrinkage mitigation of the LWA specimens. Results similar to the length change comparator data were collected. The results are provided in Section 4.2.2 in Figures 4.2.2.1 and 4.2.2.2. Depicted in Figure 3.5.2.2.1 is a picture of the vibrating wire strain gage and data collection system.

Figure 3.5.2.2.1. Vibrating wire strain gage and specimen storage.

3.5.3 Modulus of Rupture Testing

Modulus of rupture (MOR) testing provided the flexural strength of concrete. A control, shale LWA, and clay LWA specimen were prepared for testing. The same mix design was used for MOR testing as was used throughout the research program. The soak time for each LWA mix design was 1 day given that the 1 day soaking specimens performed equivalent to other soaking durations. Specimens were cast in pre-fabricated concrete molds. The molds were 4 inches by 4 inches by 16 inches in length. The concrete was placed in two lifts with 32 rods for each lift in accordance with ASTM C192/C192M-07. The specimens were stored inside the environmental chamber for 24 hours. During the first 24 hours inside the environmental chamber, the specimens were left inside the formwork with the top section exposed. Following 24 hours, the specimens were removed from the formwork. Each specimen had the batch date, LWA replacement rate, and a number indicating each specimen. Following the de-molding process, the specimens were immersed in a calcium hydrated water bath to ensure a 100 percent humidity curing environment.

At 28 days of age, the specimens were tested for their flexural strength in accordance with ASTM C78-08. The prisms were placed in the testing apparatus with third point loading applied to the prisms. Following failure three measurements were taken to the nearest 0.05 inch to determine the failure plane. With the necessary information provided the modulus of rupture was calculated using:

$$R = {}^{(P * L)}\!\big/_{(b * d^2)}$$

Equation (3-2) Modulus of rupture.

Where:

R = Modulus of Rupture (psi)

P = Maximum Applied Load as Recorded by Testing Machine (pounds)

L = Span Length (inches)

b = Average Width of Specimen at Fracture (inches)

d = Average depth of Specimen at Fracture (inches)

The modulus of rupture provided insight into the flexural strength of the concrete and was an indirect measure of tensile strength. Modulus of rupture results are provided in Section 4.5 in Table 4.5.1. Provided in Figure 3.5.3.1 is the testing apparatus with a test specimen.

Figure 3.5.3.1. Modulus of rupture testing device with specimen.

3.5.4 Modulus of Elasticity Testing

Moduli of elasticity (MOE) testing specimens were cast at the same time as the MOR specimens. A control, shale LWA, and clay LWA were produced for comparison purposes. Similar to MOR testing, the only soaking duration used for MOE testing was 1 day soaked LWA in conjunction with a 300 pound coarse aggregate replacement rate. Specimens were 4 inches in diameter by 8 inches in depth. Cylinders were produced in accordance with ASTM C192/C192M-07. With the specimens consolidated in the formwork, they were moved to the environmental chamber for 24 hours. The specimens were left uncapped. Following the 24 hours needed to harden, the molds were removed from the concrete specimens. Each specimen had the batch date, LWA replacement rate, and LWA soak time inscribed on it for quality control. The specimens were then immersed in a calcium hydrated water bath to ensure a 100 percent humidity curing environment.

Following the 28 day curing time, the specimens were prepared for MOE testing. Preparation included the grinding of the ends so that the plane of each face of the cylindrical specimen was within 0.002 inches. This is in accordance with ASTM C469/C469M-02e1. With samples prepared for testing, the measuring apparatus was attached to each concrete specimen. The samples were then placed in the Forney compression machine for testing. MOE for each specimen was determined by measuring the compressive strength and strain at two individual points. The first point of measurement was 0.00005 strain and the corresponding compressive strength. The second point of measurement was the strain at 40 percent of the average compressive strength of the concrete. These two points of measurement were in accordance with ASTM C469/C469M-02e1. Once the two points were attained, the MOE was calculated in accordance with the following equation:

$$E = {}^{(S_2 - S_1)}\!\big/_{(\varepsilon_2 - 0.00005)}$$

Equation (3-3) Modulus of elasticity.

Where:

E = Chord Modulus of Elasticity (psi)

S_2 = Stress Corresponding to 40 percent of ultimate load (psi)

S_1 = Stress Corresponding to a longitudinal strain of 50 millionths (psi)

ε_2 = Longitudinal Stain Produced by S_2

Results of modulus of elasticity testing were reported to the 50,000 psi. Calculated values for the present research are presented in Section 4.6 in Table 4.6.1. Provided in Figure 3.5.4.1 is a picture of the MOE testing apparatus with specimen.

Figure 3.5.4.1. Modulus of elasticity testing device with specimen.

Chapter 4 Results and Discussion

4.1 Results Overview

The first data assembly began with a collection of absorption capacities from the LWA specimens. The study needed a baseline for the excess outer water located on the LWA to determine the amount of water added to the mix design w/c from the surface of the LWA. The results of the LWA absorption capacities were shown in section 3.2.1 in Tables 3.2.1.1 and 3.2.1.2. With absorption capacities and excess water determined, mix designs were developed and concrete specimens were cast. Mix designs are provided in Table 4.1.1. Strain was measured with a length change comparator and vibrating wire strain gages to determine total shrinkage. Slump, unit weight, and temperature data were measured for all mixtures. Compressive strength testing was conducted for all specimens as a control to ensure similar mix designs performed as expected. Finally, modulus of elasticity and modulus of rupture tests were conducted to determine differences between the control specimen and the LWA concrete.

Table 4.1.1. Mix designs for control and LWA mixtures.

Mixture Proportions: Saturated Surface Dry Condition			
Mix Design	**Control**	**Clay**	**Shale**
Cement (lb/yd^3)	611	611	611
Coarse Aggregate (lb/yd^3)	1700	1400	1400
Fine Aggregate (lb/yd^3)	1440	1107	1178
LWA Aggregate (lb/yd^3)	0	300	300
Water (lb/yd^3)	270	270	270
w/cm	0.44	0.44	0.44
HRWR ADVA 575 (oz/cwt)	4	4	4

4.2 Shrinkage Results

4.2.1 Length Change Comparator Shrinkage Results

A length change comparator (ASTM C490-04) was used to measure linear shrinkage in the concrete specimens. Length change measurements were taken at intervals until the specimens were 112 days old. The length change measurements provided an insight into the amount of shrinkage reduction through the use LWA. Multiple soaking durations were tested in order to determine the optimal soaking time of the LWA. From Figure 4.2.1.1 it was observed that the 7 day soaked clay produced the least shrinkage in the clay specimens at 112 days. However, it should also be noted that the 1 day clay produced very similar shrinkage results as the 7 day soaked aggregate at 112 days. From Figure 4.2.1.2 it was observed that the 1 day soaked shale produced the least shrinkage for the average of the specimens in this testing program. All LWA concrete specimens did however reduce shrinkage when compared to the control specimens as seen in Figures 4.2.1.1 and 4.2.1.2 in this testing program.

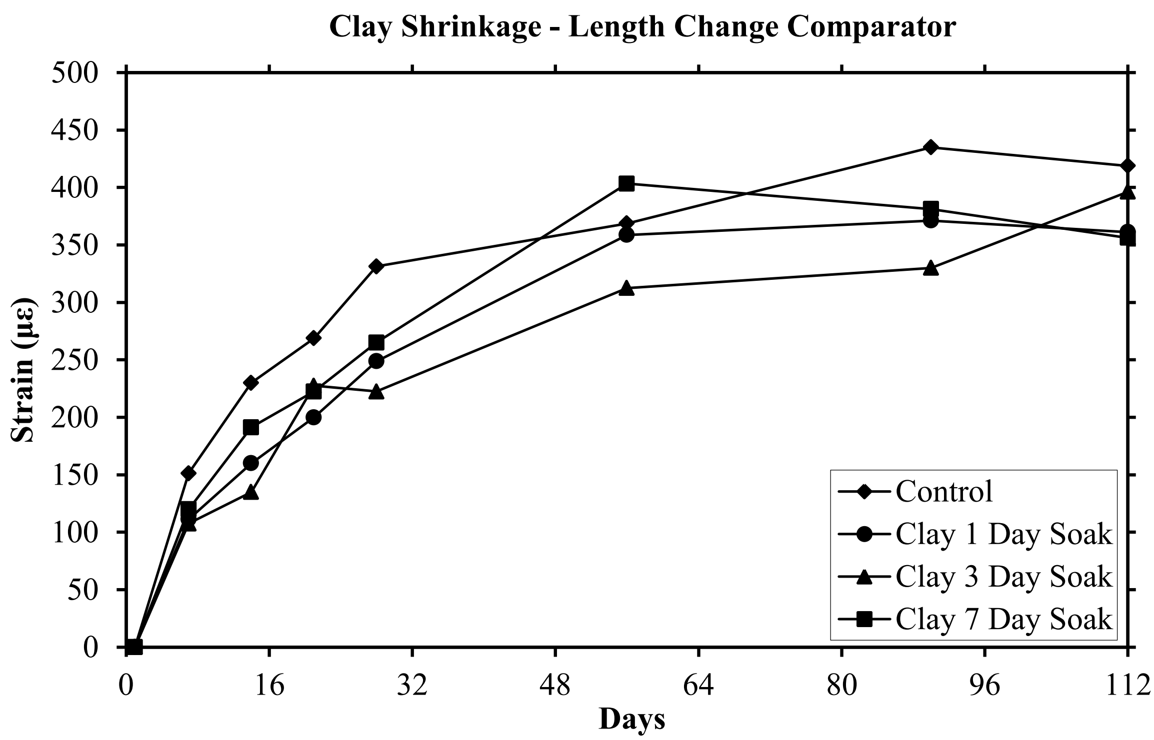


Figure 4.2.1.1. Clay shrinkage results using a length change comparator.

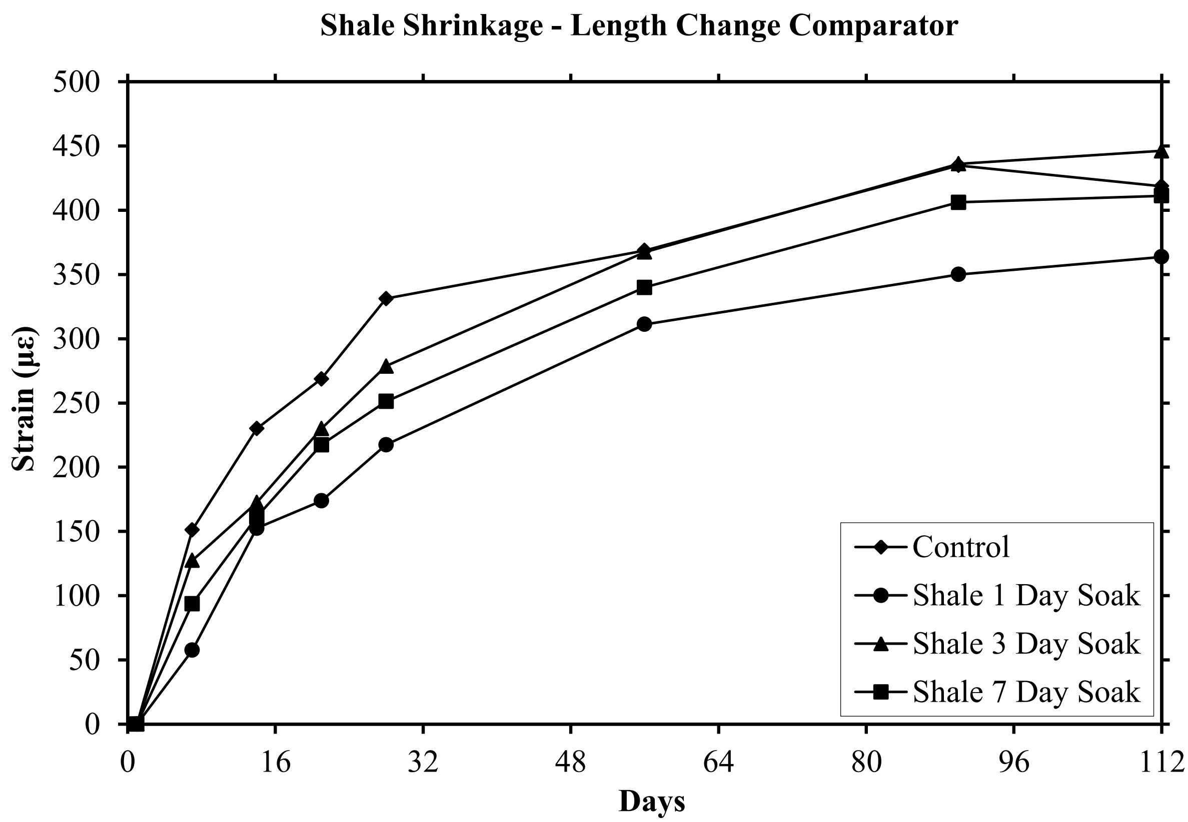


Figure 4.2.1.2. Shale shrinkage results using a length change comparator

As mentioned previously, the results indicated that the 7 day soaked clay and the 1 day soaked shale were optimal performers at mitigating shrinkage out to 112 days. The amount of shrinkage reduction at 28 and 56 day intervals has been provided in Table 4.2.1.1. For instance, at 28 days of age the clay LWA soaked for 1 day had 24.9 percent less shrinkage than the control mixture. Similarly, the same 1 day soaked clay specimen only exhibited 2.7 percent less shrinkage at the 56 day interval than the control specimen. The data indicated that as the concrete aged, there was a reduction in the shrinkage of the concrete. This may be explained that as the concrete hydrates, the number of un-hydrated cement particles decreases. Therefore as the amount of C-S-H increased the amount of shrinkage reduced in the concrete specimens.

In review of the shrinkage reduction at the 28 day interval, the 3 day soaked clay and the 1 day soaked shale produced the least shrinkage. At the 56 day interval, the 3 day soaked clay and the 1 day soaked shale again produced the least shrinkage. A review of Figure 4.2.1.1 revealed that the 3 day soaked clay produced increased expansion at later ages near the 90 and 112 day intervals. However, the shale remained more uniform throughout testing and the 1 day soak duration produced the least shrinkage at all intervals. The shale produced more uniform absorption capacities and shrinkage results than that of the clay specimens. However, the minimum strain produced in the clay and shale LWA was very similar. The 1 day soaked shale produced a strain of 364 με while the 7 day clay produced a strain of 356 με. As mentioned previously, the 1 day soaked clay produced similar results as the 7 day soaked clay. The 1 day soaked clay produced a strain of 361 με. This was very similar to the strain produced within the 1 day soaked shale. The results indicate that both clay and shale were viable options for mitigating shrinkage and produced similar results. Another conclusion is that there is minimal difference in shrinkage mitigation due to aggregate soak duration as long as it is soaked a minimum of 24 hours.

Table 4.2.1.1. LWA shrinkage mitigation (length change comparator).

Mixture	**Percent Difference**	
Days	28	56
Control	0.0%	0.0%
Clay 1 Day	24.9%	2.7%
Clay 3 Day	32.8%	15.3%
Clay 7 Day	20.0%	-9.4%
Shale 1 Day	34.3%	15.6%
Shale 3 Day	15.8%	0.3%
Shale 7 Day	24.2%	7.8%

Due to the nature of testing multiple specimens, there is always the potential for variation in data recordings. As such, confidence intervals were calculated to determine overlaps in shrinkage data produced by the length change comparator. Confidence intervals were calculated for all concrete mix designs at 112 days, with calculations based on a 90 percent confidence level. Provided in Table 4.2.1.2 are the upper and lower confidence interval bounds for the shrinkage data.

Table 4.2.1.2. Confidence interval data for length change comparator specimens.

112 Day Data	Average Strain	Standard Deviation	Confidence Interval	Lower C.I.	Upper C.I.
Control	418.8	76.0	62.5	356.3	481.2
Clay 1 Day Soak	361.3	15.5	12.7	348.5	374.0
Clay 3 Day Soak	396.3	11.8	9.7	386.5	406.0
Clay 7 Day Soak	356.3	178.1	146.5	209.8	502.7
Shale 1 Day Soak	363.8	22.9	18.8	344.9	382.6
Shale 3 Day Soak	446.3	32.2	26.5	419.7	472.8
Shale 7 Day Soak	411.3	18.9	15.5	395.7	426.8

The findings produced from the confidence interval calculations indicate that there was no statistical difference in shrinkage results produced by the length change comparator. The control specimen overlaps the confidence interval of all LWA concrete specimens. The reason is the large scatter in the shrinkage data for the control specimen. The 112 day strain for the 4 control specimens ranges from 370 $\mu\varepsilon$ – 530 $\mu\varepsilon$. Ranges for the LWA were typically less than 75 $\mu\varepsilon$ difference. The 160 $\mu\varepsilon$ difference in the control data causes there to be no statistical significance in the shrinkage results for the length change comparator data. (It should be noted that the 7 day soaked clay concrete has a large confidence interval due to a potential errant reading taken from one of the specimens. However, if the data point was removed, there would still be an overlap between the 7 day soaked clay specimen and the control specimen, and no results would change.) In conclusion, the length change comparator shrinkage data indicated that there is no statistical significance in measured shrinkage between the LWA specimens and the control specimens.

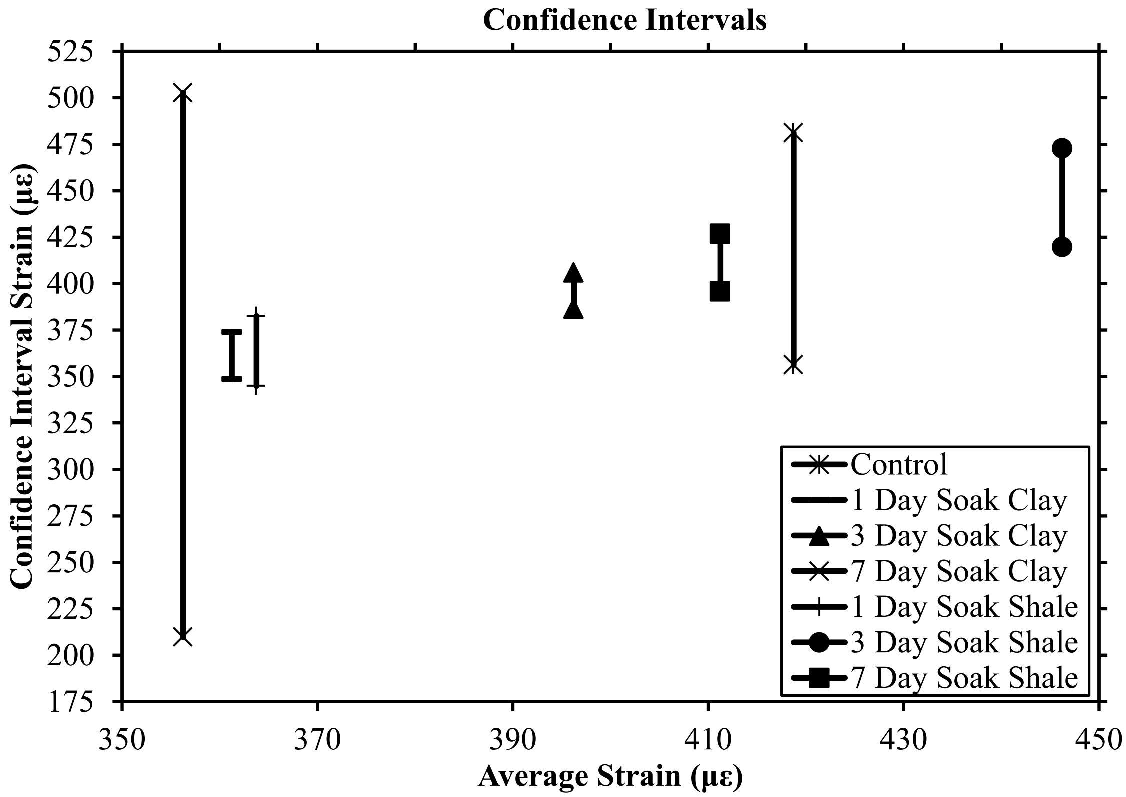


Figure 4.2.1.3. Confidence intervals for length change comparator results.

4.2.2 Vibrating Wire Strain Gage Shrinkage Results

Vibrating wire strain gages produced by Geokon Industries were used to measure the shrinkage of concrete prisms. The strain gages were placed inside the concrete prisms and measured the shrinkage during the hydration of the cement paste. The strain gages were sensitive to all shrinkage and expansion in the concrete specimens. Shrinkage results are provided in Figures 4.2.2.1 and 4.2.2.2. An observation of the figures indicates expansion during the initial hydration phase. The formation of C-S-H expanded the concrete before the majority of the bulk water was chemically combined, thus causing expansion. Following expansion, the bulk water chemically combined in the C-S-H and shrinkage occurred throughout further testing. The results indicate that the 1 day soaked clay and 1 day soaked shale produced the least shrinkage at the 112 day interval. The 1 day soaked clay and 1 day soaked shale produced micro strains of 258 and 286 respectively. Soak time of the LWA did have a minor effect on the shrinkage produced in the concrete specimens. However this effect was small as seen in Figures 4.2.2.1 and 4.2.2.2, as long as the LWA was soaked for 24 hours.

Provided in Table 4.2.2.1 are the LWA shrinkage mitigation at 28 and 56 day intervals as measured with vibrating wire strain gages. The results agreed with the length change comparator results of Table 4.2.1.1 in that, as the concrete aged, there was less shrinkage. The 3 day soaked clay LWA and the 7 day soaked shale LWA produced the least shrinkage when compared to the control specimen at 28 and 56 day intervals. The 3 day soaked clay mixture had a 50.9 and 45.8 percent reduction in shrinkage at the 28 and 56 day interval when compared to the control mixture. The 7 day soaked shale mixture had a 46.7 and 29.8 percent reduction in shrinkage at the 28 and 56 day interval when compared to the control mixture. The 3 day clay and 7 day shale did not produce the least shrinkage at the 112 day interval, as seen in Figures 4.2.2.1 and 4.2.2.2. The 1 day soaked clay and 1 day soaked shale produced the least shrinkage at the 112 day interval as seen in Figures 4.2.2.1 and 4.2.2.2.

Table 4.2.2.1. LWA shrinkage mitigation (vibrating wire strain gage).

Mixture	**Percent Difference**	
Days	28	56
Control	0.0%	0.0%
Clay 1 Day	33.1%	18.4%
Clay 3 Day	50.9%	45.8%
Clay 7 Day	50.2%	32.2%
Shale 1 Day	8.9%	1.8%
Shale 3 Day	36.2%	29.1%
Shale 7 Day	46.7%	29.8%

During testing of the control and 1 day soak duration for clay and shale, complications arose due to a loss of power to the data collection system. This resulted in gaps in the data where strain was not recorded. The gaps in the data may be seen in Figures 4.2.2.1 and 4.2.2.2; however, the general trend of increasing strain is still observed with the missing data. The problem was corrected before the 3 day and 7 day soaking durations were cast and tested.

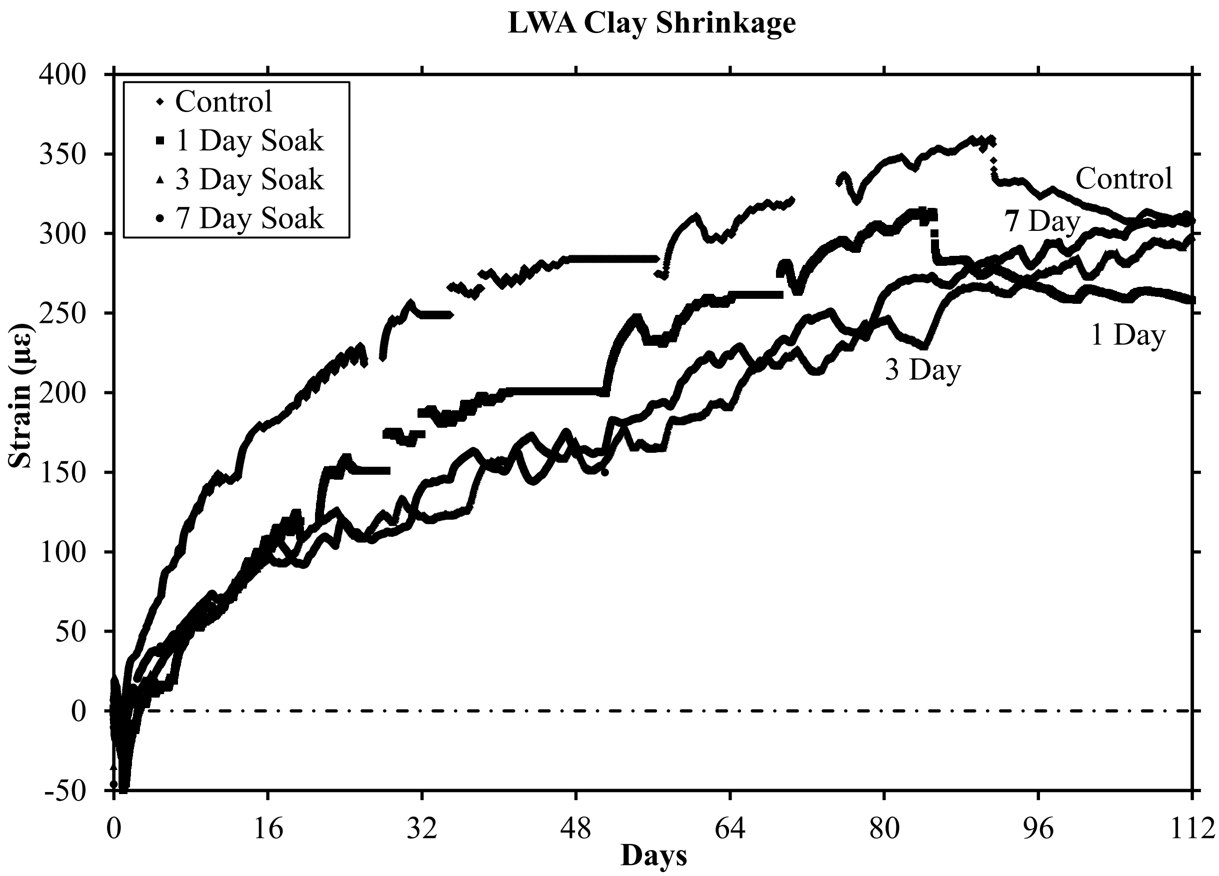


Figure 4.2.2.1. Clay shrinkage results using vibrating wire strain gages.

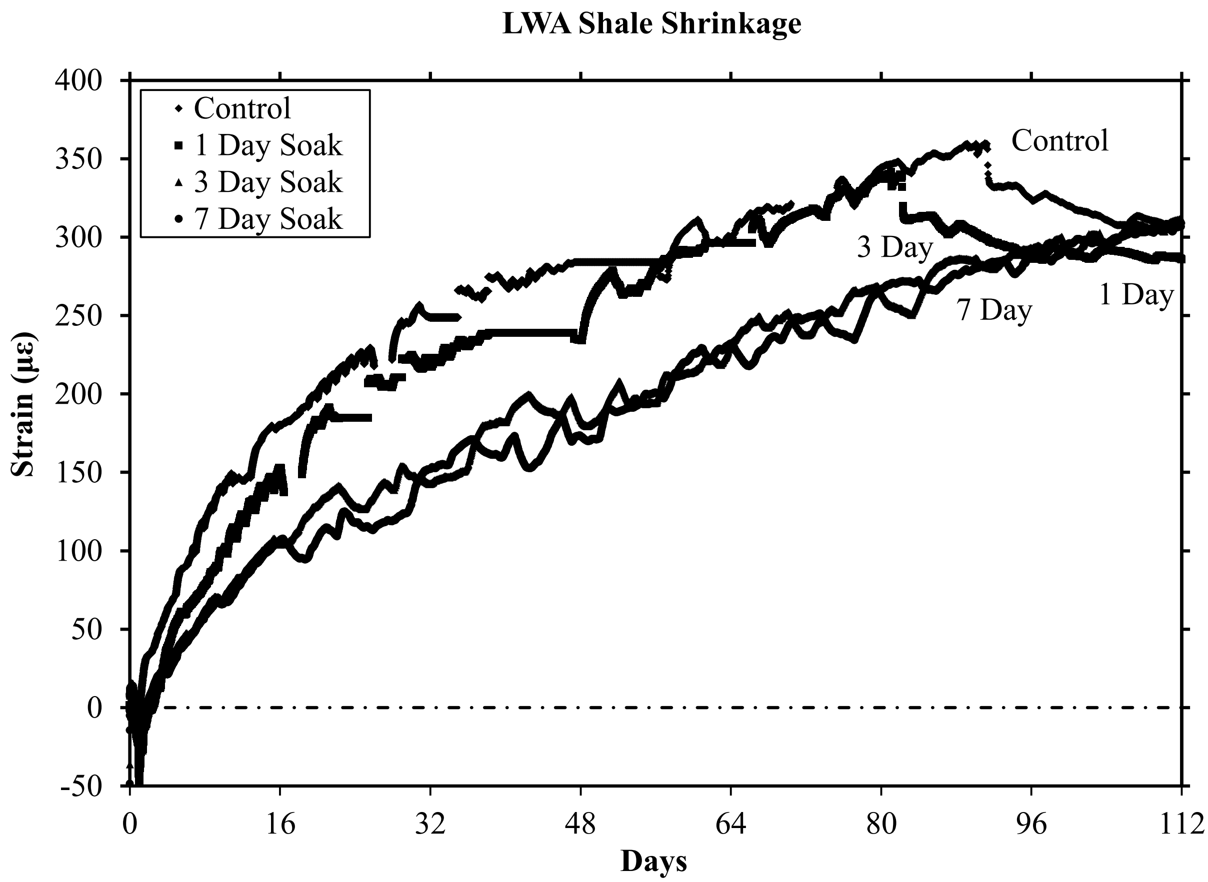


Figure 4.2.2.2. Shale shrinkage results using vibrating wire strain gages.

Confidence intervals were created to determine the applicability of the shrinkage results. The confidence intervals again used 112 day data with a 90 percent confidence level. The strain gage confidence intervals found that there was a statistical significance between the shrinkage data of the control and 1 day soaked LWA specimens. The LWA 1 day soaked clay and shale specimens do not overlap the control data confidence interval. This indicates that the 1 day soaked clay and shale specimens do exhibit less shrinkage than that of the control specimens. The control specimens confidence interval is 296.3 με – 316.3 με while the1 day soaked clay and shale specimens confidence intervals are 251.1 με – 264.4 με and 275.0 με – 296.0 με. The 3 and 7 day soak LWA specimens do overlap the control data and are not able to claim a statistical difference with the control data. Confidence intervals for the strain gage specimens are provided in Table 4.2.2.2.

Table 4.2.2.2. Confidence interval data for strain gage specimens.

112 Day Data	Average Strain	Standard Deviation	Confidence Interval	Lower C.I.	Upper C.I.
Control	306.3	10.5	10.0	296.3	316.3
Clay 1 Day Soak	257.8	6.9	6.6	251.1	264.4
Clay 3 Day Soak	298.3	16.3	15.5	282.9	313.8
Clay 7 Day Soak	307.4	5.0	4.7	302.7	312.1
Shale 1 Day Soak	285.5	11.0	10.5	275.0	296.0
Shale 3 Day Soak	311.6	9.8	9.3	302.3	320.9
Shale 7 Day Soak	306.4	8.9	8.4	297.9	314.8

The 1 day soaked clay specimens produced the confidence interval with the smallest strain values. The small shrinkage values indicate these specimens produced the least shrinkage of all the specimens. It also indicates that the 1 day soaked clay specimens produced the best results for shrinkage mitigation. Following the 1 day soaked clay specimens were the 1 day soaked shale specimens. This group of specimens produced the second lowest shrinkage results, and as seen in the data from Table 4.2.2.2, the results do not overlap the control data. Both the 1 day soaked clay and shale produce confidence intervals which fall outside the strain range for the control specimens. This indicates that there is no increased advantage in shrinkage mitigation to soaking the LWA longer than 24 hours.

As seen from Table 4.2.2.2 and Figure 4.2.2.3, the 1 day soaked clay produced less shrinkage and lower confidence intervals for the strain gage data specimens. A potential explanation for this is the porosity of the expanded LWA. The expanded clay lightweight aggregate has an absorption capacity of 15 percent while the expanded shale has an absorption capacity of 12.9 percent. This difference in absorption capacity leads to less water in the shale specimens. This reduction in curing water in the shale specimens may have led to increased shrinkage when compared to the clay specimens. However, though there was less water in the shale specimens than the clay specimens, there is still a decrease in shrinkage when compared to the control specimens.

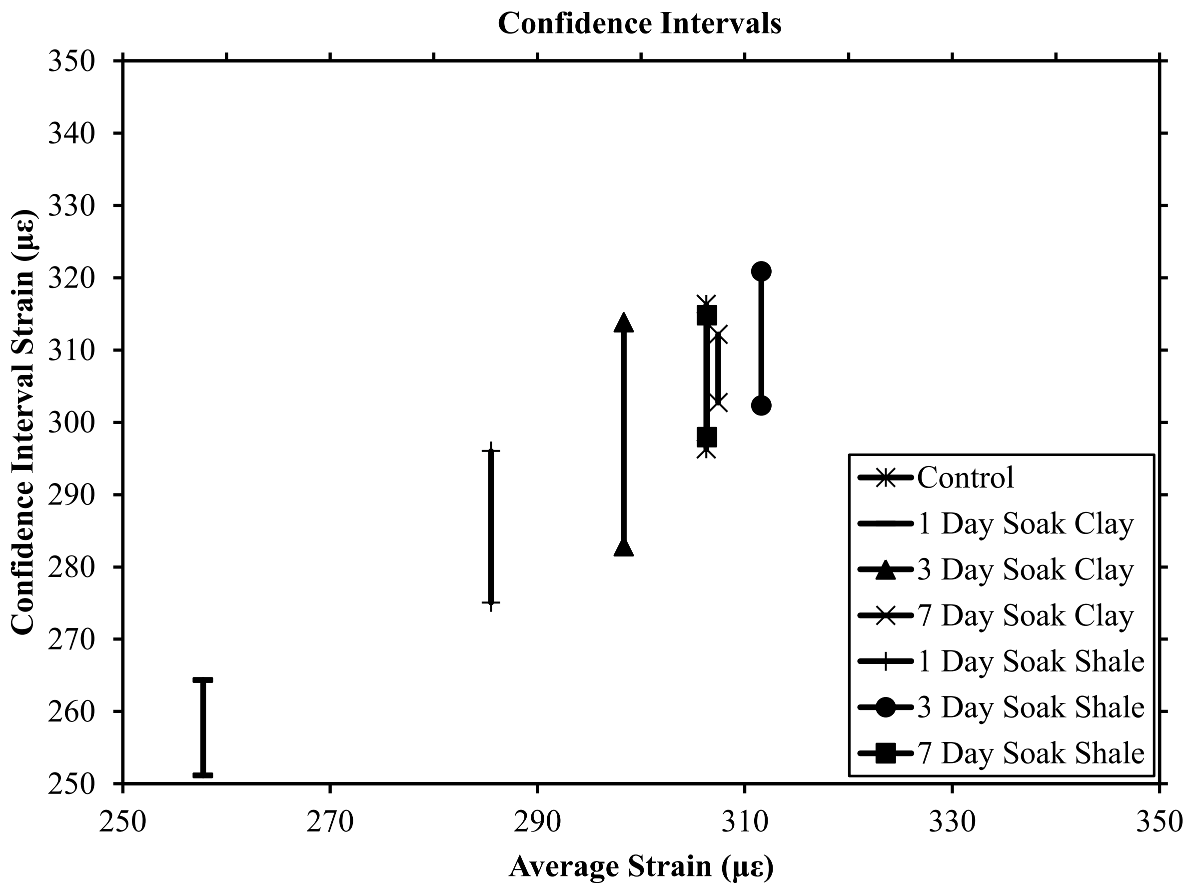


Figure 4.2.2.3. Confidence intervals for vibrating wire strain gage results.

A review of the data at 28 days for both clay and shale LWA mixes indicates that the shrinkage results would be different if testing were not carried out to the 112 day interval. The 28 day data as seen in Figures 4.2.2.3 and 4.2.2.4 indicates that the 3 day soaked clay and the 7 day soaked shale produced the least shrinkage using strain gage data. The least shrinkage produced in the length change comparator data was the 3 day soaked clay and the 1 day soaked shale. The results indicate that the 3 day soaked clay for both measuring devices produced the least shrinkage, while the shale had different soaking durations producing the least shrinkage. However, when comparing the shrinkage reduction at 28 days of both 3 and 7 day soaked specimens to that of the 1 day soaked specimens there is minimal difference. The 1 day soak duration produced nearly equivalent shrinkage reduction to that of the maximum shrinkage reduction soaking durations.

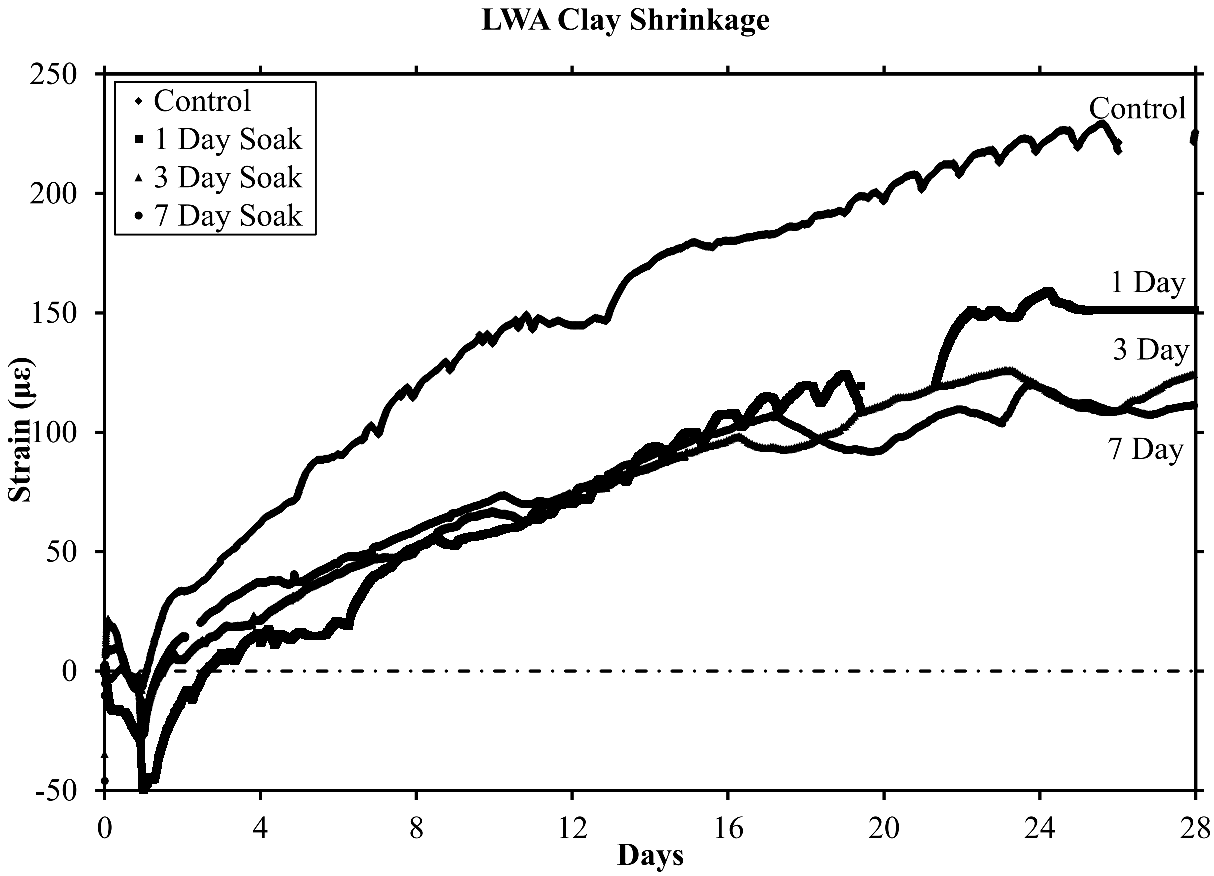


Figure 4.2.2.4. Clay shrinkage results using strain gage data at 28 days.

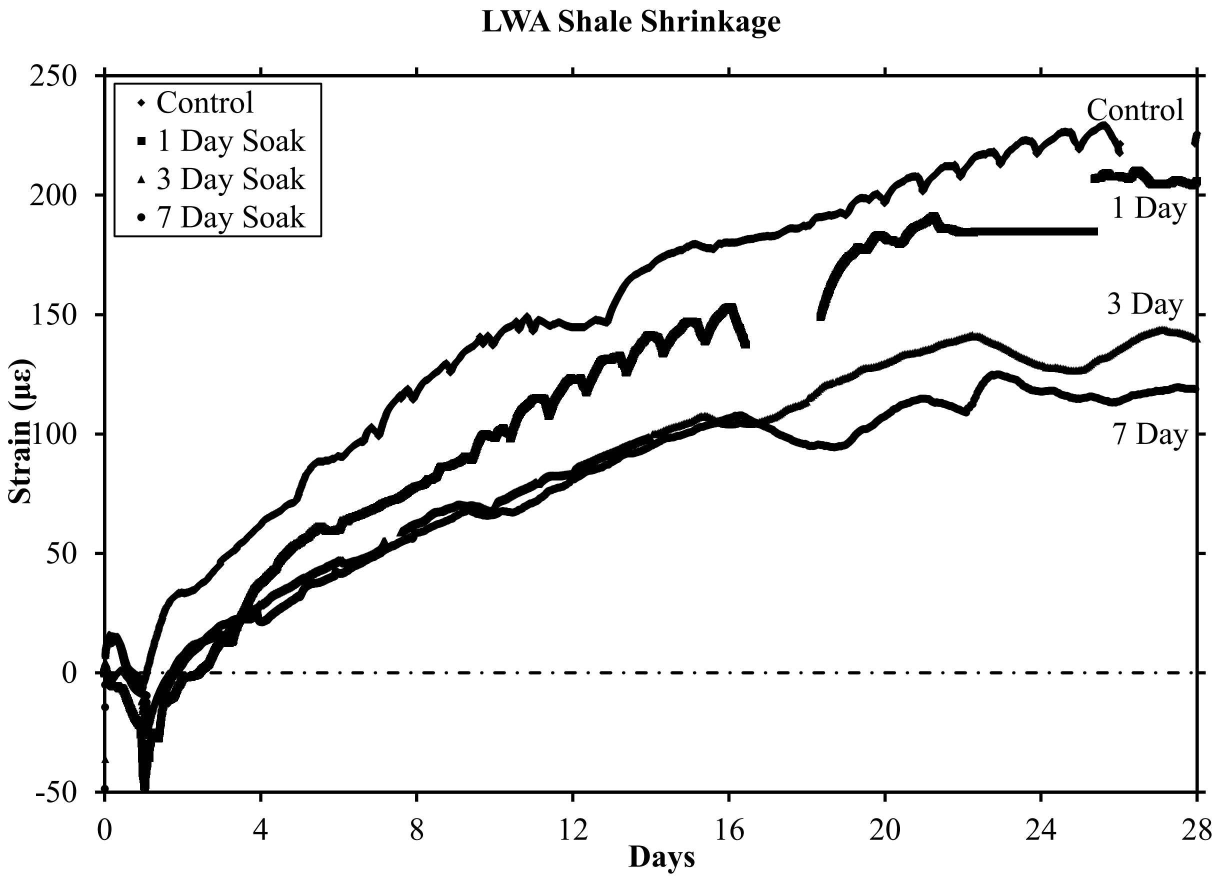


Figure 4.2.2.5. Shale shrinkage results using strain gage data at 28 days.

4.2.3 Differences between Shrinkage Methods

Two testing techniques were used for collecting shrinkage data throughout the testing program. The results of the two measurement types, the length change comparator and vibrating wire strain gages, agreed that the 1 day soaked LWA specimens provided the least shrinkage. However, the length change comparator data measured strain on the order of 100 με greater than the vibrating wire strain gage data. It was expected that the data from both measurement devices would be closer to each other in total shrinkage at the 112 day interval.

Provided in Table 4.2.2.1 are the LWA shrinkage mitigation at 28 and 56 day intervals as measured with vibrating wire strain gages. The results agreed with the length change comparator results of Table 4.2.1.1 in that as the concrete aged there was less shrinkage. However, the percentage difference between the control specimen and the LWA specimens was larger in all cases except in the 1 day soaked shale when comparing strain gage data to length change comparator data. The observation held true for both the 28 day and 56 day data. An explanation for the observed data was due to the wrapping of the strain gage specimens in polyurethane sheeting while the length change comparator specimens were not sealed. The LWA was able to mitigate more chemical shrinkage because water was not lost to the atmosphere during initial set of the specimens. By preventing water loss to the atmosphere, all water located in the LWA was available for hydration of the cement particles. With all water being used for hydration products instead of bleed water mitigation, the overall amount of shrinkage could be reduced more than in the unwrapped specimens.

Though the length change comparator and vibrating wire strain gages measured the change in length of the concrete specimens, there were potential areas for differences in the measured results. One such source of difference in the two testing types was the accuracy and frequency of the collection systems. Both systems agree that shrinkage reduction occurred as a result of using LWA as an internal curing agent. However, the resolution of the instruments involved could affect the amount of total shrinkage measured for each specimen. The length change comparator measured length change differences to ten-thousandths of an inch yielding a resolution of 10 $\mu\varepsilon$. The vibrating wire strain gages measured strain with a resolution of 1 $\mu\varepsilon$. Though both provide a high degree of length change difference capability, the length change comparator data produced one shrinkage measurement at 8 measurement intervals. The vibrating wire strain gage data were collected continuously throughout the entire 112 day curing cycle. This continuous data collection system of the strain gage specimens compared to 8 singular points of the length change comparator specimens provides an opportunity for the data to vary between the two collection systems.

Another possible difference between the two measurement systems could be due to the location of testing. The specimens for the length change comparator data were removed from the environmental chamber during testing. As such, these specimens were subjected to potential temperature and humidity drops during testing. The specimens with internal vibrating wire strain gages were placed in the environmental chamber without being moved or removed. This potential drop in temperature and humidity gradient may have caused slight changes in the data, when comparing the two data collection systems.

4.3 Compressive Strength, Slump, and Unit Weight

Compressive strength testing was carried out at 1, 7, 28, and 56 day intervals. Three specimens were tested at each age. The compressive strength, slump, and unit weight test results are provided in Table 4.3.1. Compressive strength curves are shown in Figures 4.3.1 and 4.3.2, and it is evident that coarse LWA does reduce concrete compressive strength. The control mixture produced the highest compressive strength. The control specimens were expected to be stronger than the LWA specimens due to the lower strength of the LWA. There were no apparent trends in the compressive strength data of different soaking durations (1, 3, and 7 days) between the clay and shale LWA. The data shown in Figures 4.2.1 and 4.2.2 indicates that there is no distinguishable difference in the compressive strength of LWA soaked at 1, 3, and 7 days. The 7 day soaked LWA specimens did not produce the largest compressive strength during testing at either the 28 day or 56 day test interval. The 1 day soaked lightweight aggregate did perform equal to or better than the 3 and 7 day soak duration specimens and produced the highest compressive strength in the LWA clay specimens at 28 days. Sufficient compressive strength is gained for a 1 day soak LWA as for extended soaking durations. The compressive strength of all the concrete mixtures exceeded the minimum compressive strength design criteria of 4000 psi as defined by AHTD.

Slump tests were conducted in accordance with ASTM C143/C143M-05a. The addition of LWA as an ingredient in concrete did not affect the slump of the concrete, nor did the soaking duration of the LWA. With the high coarse aggregate content and relatively low w/c (0.44), the main factor affecting slump was the use of a high range water reducer. The maximum slump criterion, as defined by AHTD, for the research was 4 inches. The slump of three concrete mixtures exceeded the 4 inch maximum requirement. Given that the slump was high for three mixtures, other data provided conclusively there were no deleterious effects caused by the excess high range water reducer used to adjust slump. A review of the unit weight and compressive strength data of the three high slump mixtures revealed there was no decrease in either property. A review of the compressive strength and unit weight data was necessary given that high range water reducer carried the potential to delay strength gain and segregate the concrete when used excessively in concrete production. The data show typical compressive strength values for LWA of approximately 7000 psi and typical unit weight values of approximately 140 lb/ft^3 for LWA concrete at the 300 pound LWA replacement rate. Therefore, the LWA concrete with the higher slump values was of equal quality to the LWA concrete that was within the 4 inch slump limit.

According to ACI 318-08, normal weight concrete has a density between 135 – 160 lb/ft^3 (ACI, 2008). The addition of LWA did lower the unit weight of the concrete; however, all concrete mix designs developed were normal weight according to ACI 318-08. The control mixture had a unit weight of 150 lb/ft^3 due to a large amount of coarse limestone aggregate, while the LWA concretes had unit weights that ranged from 137 lb/ft^3 to 140 lb/ft^3 respectively.

Unit weight provided an insight that the LWA would not affect the concrete in a negative manner by lowering its unit weight. Lightweight concrete is prone to lower tensile strengths and increased shrinkage (Mehta & Monteiro, 2006). A replacement rate greater than 300 lb/ft^3 could result in the concrete being defined as lightweight according to ACI 318-08. A reduction in unit weight does lower concrete member dead loads.

Table 4.3.1. Compressive strength, slump, and unit weight.

Mixture	Compressive Strength (psi)				Slump (in)	Unit Weight (lb/ft^3)
	1 Day	7 Day	28 Day	56 Day		
Control	3520	8450	9070	9570	2.75	150
Clay 1 Day	3300	6190	7040	7290	6.25	138
Clay 3 Day	3060	6010	7020	7510	5.00	139
Clay 7 Day	2730	5760	6870	7330	8.00	139
Shale 1 Day	4210	6450	6840	7570	3.00	137
Shale 3 Day	4370	6590	7200	7640	2.00	140
Shale 7 Day	3880	5950	7090	7410	2.00	138

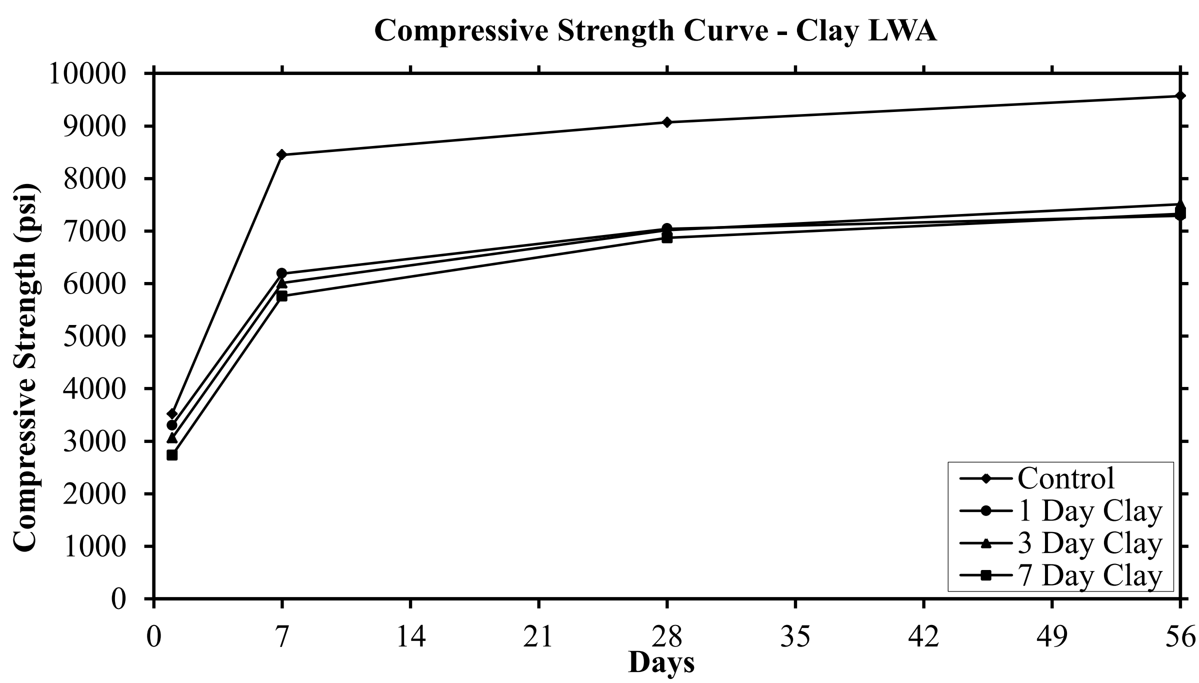


Figure 4.3.1. Clay compressive strength curve.

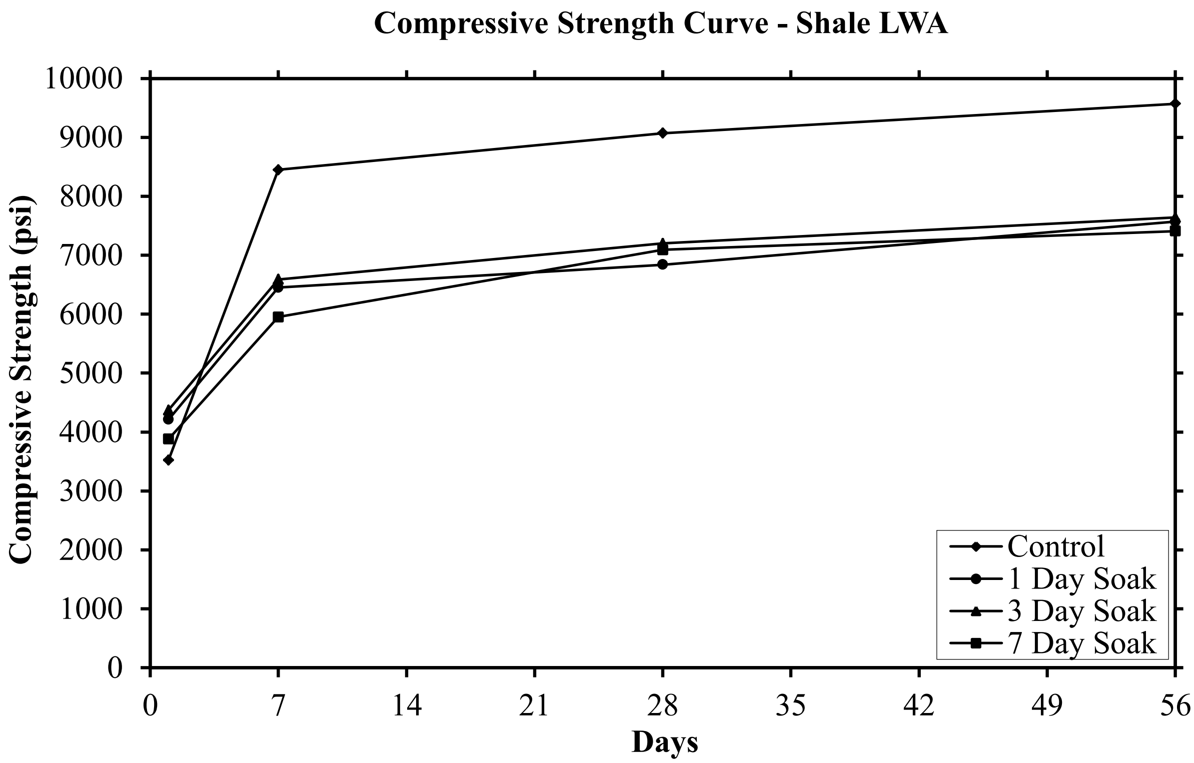


Figure 4.3.2. Shale compressive strength curve.

4.4 Temperature Results

Concrete temperature was measured through the use of the vibrating wire strain gages. The vibrating wire strain gages contain an internal thermistor, which measured the temperature of the concrete throughout the duration of testing. This allowed not only an initial concrete temperature, but also a temperature profile throughout the testing duration. All concrete specimens produced similar temperature reading profiles. Provided in Figure 4.4.1 is a representative sample of the concrete temperature profile during testing.

The concrete saw an initial spike in the temperature as hydration products began. The formation of the hydration products was an exothermic reaction which produced the spike in the temperature gradient profile. Following the initial spike in temperature, there was a sharp decrease in the temperature. This rapid decrease in the temperature was related to the removal of the formwork. The concrete specimens were allowed to cure for 24 hours in the formwork and wrapped in polyurethane sheeting. The removal of the sheeting and forms produced a rapid drop in temperature. The concrete would then find an equilibrium temperature between 72°F and 75°F as seen in Figure 4.4.1, which is in accordance with ASTM C157/C157M-06. As expected, internal curing and soaking duration had no effect on concrete temperature.

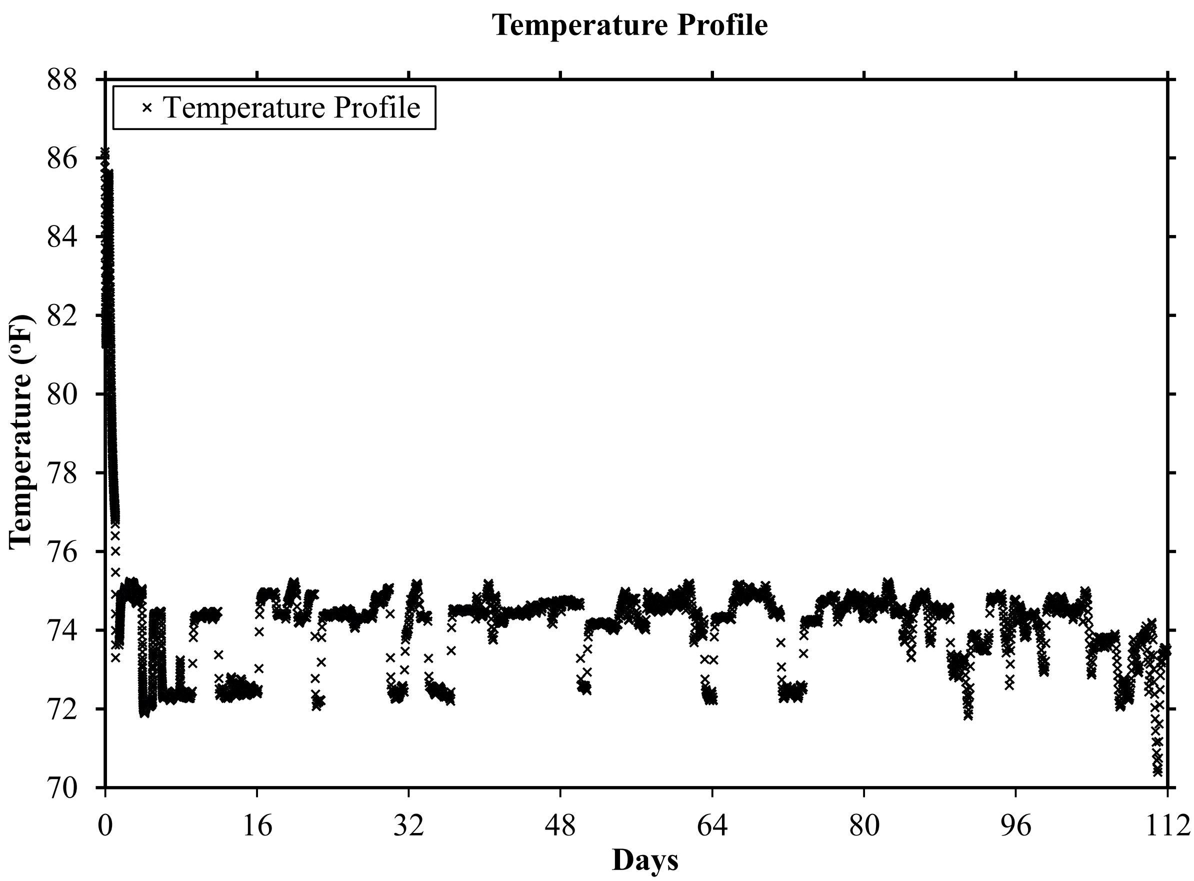


Figure 4.4.1. Temperature profile of concrete specimens throughout testing.

4.5 Modulus of Rupture Results

The flexural strength, Modulus of Rupture (MOR), of the concrete was tested using beams that were 4 inches by 4 inches by 16 inches in length. The beams were loaded (third point loading) to failure and conducted in accordance with ASTM C78-02. MOR testing was performed to determine the influence of the LWA on the flexural strength of the concrete. The

MOR test also is an indirect measure of concrete tensile strength. Concrete specimens containing LWA were weaker in compression; however, there should be an increase in the bond at the interfacial transition zone due to better hydration, which should increase tensile strength. The LWA released the water to the surrounding hydrating cement particles and should provide a better bond product. However, the w/c was not low enough to undergo self-desiccation in the control specimen. This led to similar MOR values for all the specimens in question. Presented in Table 4.5.1 is the MOR values for the control, expanded clay, and expanded shale LWA.

Table 4.5.1. Modulus of rupture data.

Modulus of Rupture Data			
(psi)	**Clay**	**Shale**	**Control**
Spec 1	920	795	900
Spec 2	855	790	835
Spec 3	875	735	805
Measured Avg	885	775	845
ACI Prediction	665	670	700

Section 10 of ASTM C78-02 states that no beam failures shall deviate more than 16 percent from each other if constructed from the same sample of concrete. The largest deviation was found in the control group between specimen 1 and specimen 3. The deviation between the two specimens is approximately 11 percent. This deviation is within the specified criteria of ASTM C78-02.

The MOR test results indicate that the 1 day soaked clay at a 300 pound replacement rate produced the strongest beams in flexure at 885 psi as seen in Table 4.5.1. The 1 day soaked shale at a 300 pound replacement rate produced the weakest beams in flexure at 775 psi. The control specimen's had an average strength of 845 psi. There is an approximate 12 percent difference in the average beam flexural strength of the weakest and strongest specimens.

Though the average clay and average shale specimens did not produce close results, they did stay within the 16 percent deviation as put forth in Section 10 of ASTM C78-02. This requirement does not apply to these specimens as they are not of the same batch. However it does provide insight into typical deviations within the MOR test of similar batch samples. As such, it is concluded that the 1 day soaked 300 pound replacement rate of lightweight clay and lightweight shale did not impact the MOR of the concrete.

Provided in ACI 318-08 is an equation for estimating the modulus of rupture of concrete:

$$f_r = 7.5\lambda\sqrt{f'c}$$

Equation (4-1) ACI prediction equation for modulus of rupture.

Where:

f_r = Modulus of Rupture of concrete (psi)

λ = Lightweight Concrete Modification Factor (λ = 1 for Normal Weight Concrete)

f'_c = Specified Compressive Strength of concrete (psi)

As seen in Table 4.5.1, the estimated MOR value based on ACI 318-08 for the concrete was conservative for the mix designs in this research. The measured concrete MOR values were at a minimum of 1.16 times larger than the ACI 318-08 prediction equation. This underestimation in the MOR strength of the concrete leads to a conservative design. The underestimation agrees with the earlier assumption that the 1 day soaked 300 pound replacement rate of LWA does not negatively affect the MOR. Both LWA mix designs produced similar MOR values as that of the control mix design further indicating that the LWA did not affect the MOR. However, there is a potential for deviations in MOR values at different LWA replacement rates and soaking durations.

4.6 Modulus of Elasticity Results

Modulus of elasticity (MOE) specimens were cast in accordance with ASTM C192/C192M-07 except where specified in Chapter 3. There was an expectation that the MOE for the LWA would be different than that of the control specimen. The lower compressive strength of the LWA compared to the coarse limestone aggregate was reason for the expectation of the lowered MOE. The lower strength LWA was expected to deform more than the limestone under a given loading producing a lower MOE value. The lower MOE value indicates that the LWA concrete was less stiff than the control mix design. Presented in Table 4.6.1 are the measured MOE values. Presented in Figures 4.6.1, 4.6.2, and 4.6.3 are the stress-strain curves developed from ASTM C469/C469M-02e1.

Table 4.6.1. Modulus of elasticity data.

Modulus of Elasticity Data			
(ksi)	**Clay**	**Shale**	**Control**
Spec 1	4320	4790	6030
Spec 2	4790	4970	6010
Spec 3	5100	5060	6200
Measured Avg	4740	4940	6080
ACI Prediction	4750	4980	5620

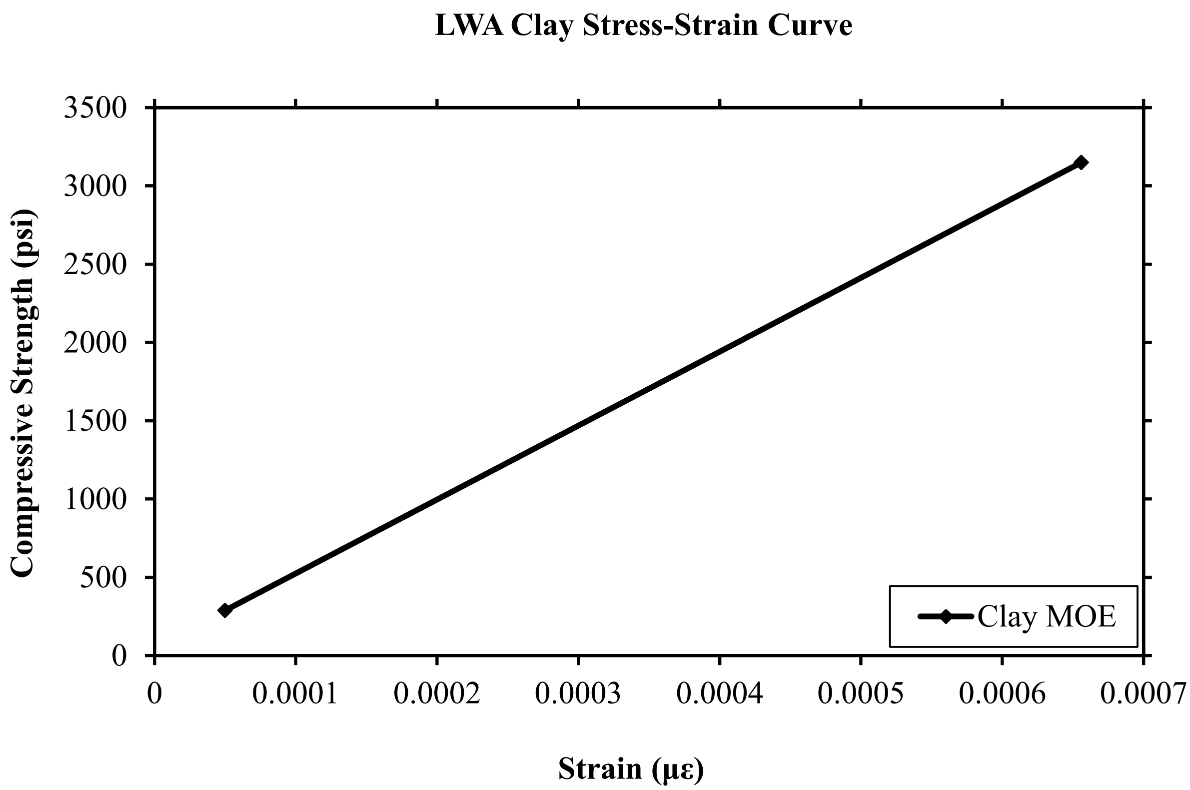


Figure 4.6.1. Stress-strain curve modulus of elasticity LWA clay concrete.

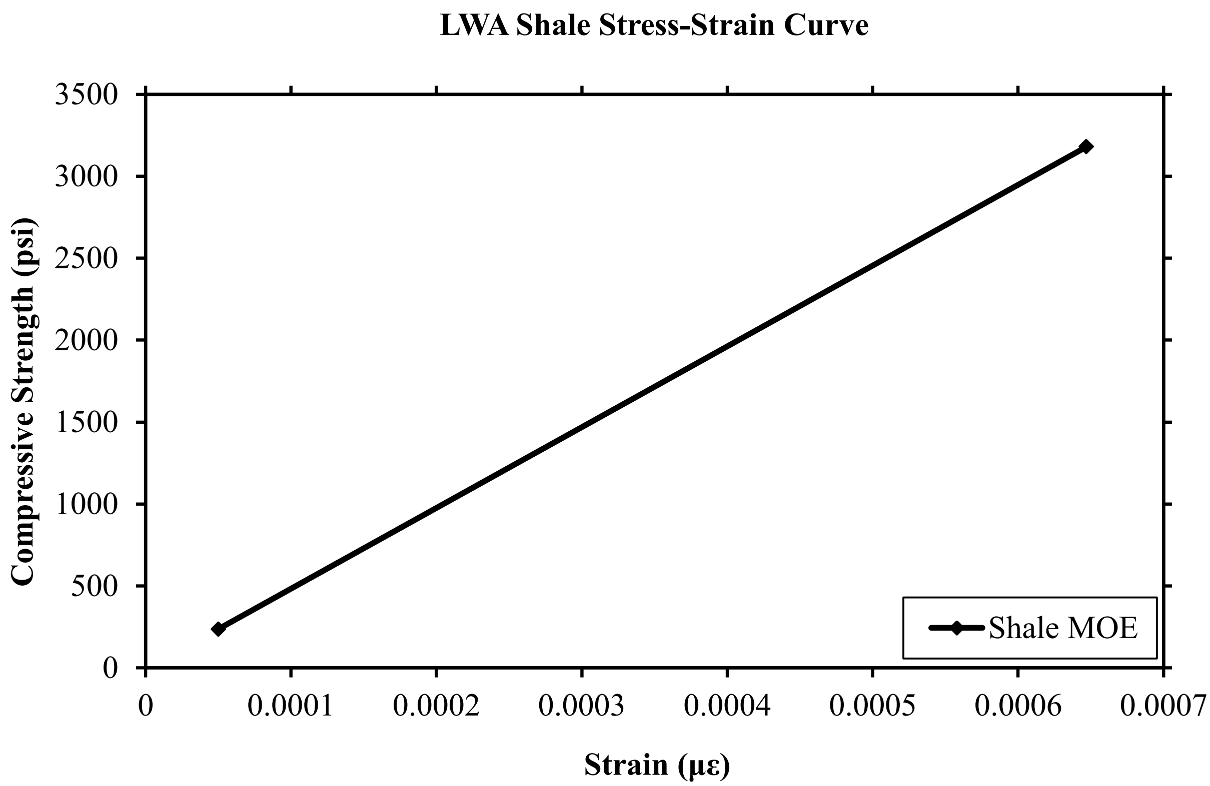


Figure 4.6.2. Stress-strain curve modulus of elasticity LWA shale concrete.

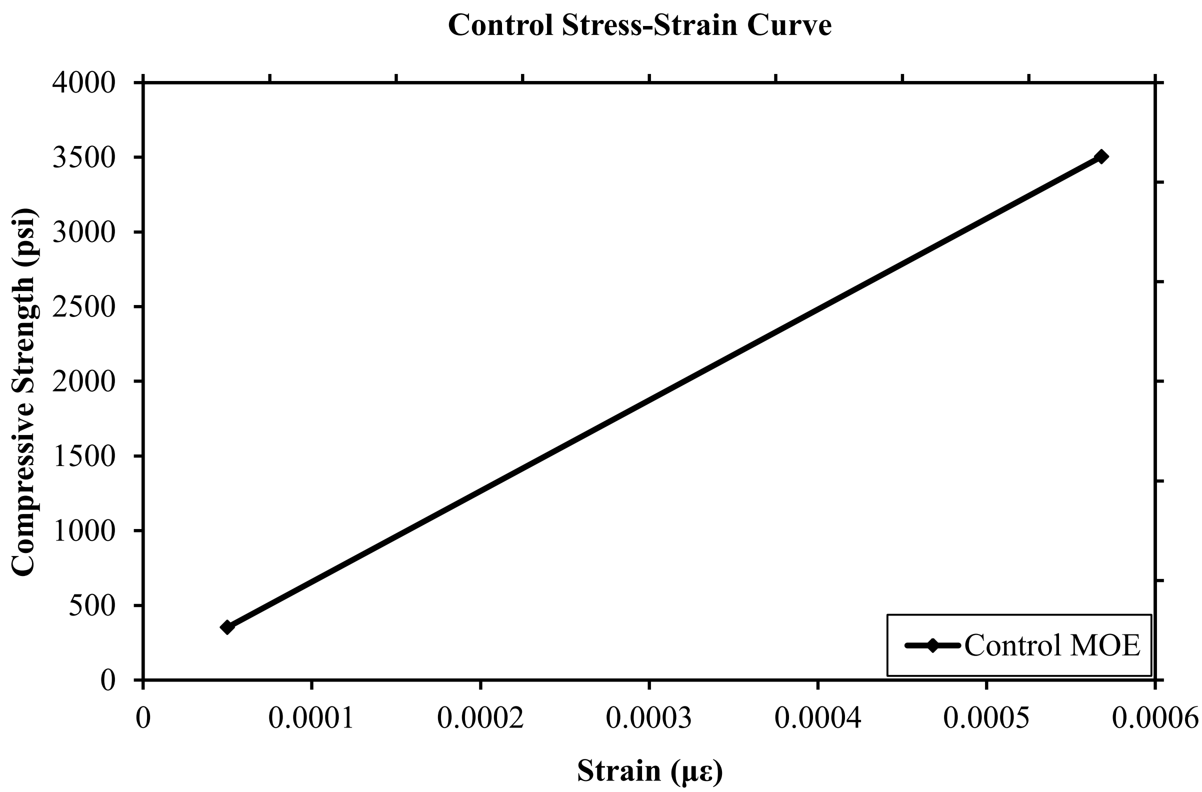


Figure 4.6.3. Stress-strain curve modulus of elasticity control concrete.

As expected the data in Table 4.5.1 is in agreement with the expectation that the MOE would decrease for the LWA mix designs when compared to the control mixture. The measured MOE value for the control mixture is 6080 ksi while the shale and clay are 4940 and 4740 ksi respectively. The results agree with the specific gravities of the aggregates as well. The limestone specific gravity is 2.68, the LWA shale is 1.41, and the LWA clay is 1.25. It is assumed that as the specific gravity of a particular aggregate decrease, there is a decrease in the compressive strength and stiffness as well. The decrease in the aggregate strength is noticeable in the increase in linear deformation during MOE testing and in the concrete compressive strength results. The lightweight clay has the lowest specific gravity and also the lowest MOE.

The clay LWA has a more porous structure than the shale, and the pore structure reduces density and increases deformation.

Provided in ACI 318-08 is a prediction equation to estimate the modulus of elasticity of concrete:

$$E_c = w_c^{1.5} 33\sqrt{f'_c}$$

Equation (4-2) ACI prediction equation for modulus of elasticity.

Where:

E_c = Modulus of Elasticity (psi)

w_c = Density of Concrete (lb/ft^3)

f'_c = Specified Compressive Strength of concrete (psi)

As seen in Table 4.6.1 the MOE equation predicts closely the expected MOE or errs slightly conservative. The LWA clay and shale MOE predicted values were close to the same as the measured values of 4750 psi and 4980 ksi respectively. The control prediction was lower than the measured MOE by approximately 8 percent. The prediction equation found in ACI 318-08 can be used to estimate the MOE of internally cured concrete using LWA. However, the ACI 318-08 prediction equation will produce conservative MOE values.

Provided in ASTM C469/C469M-02e1 is the precision of the MOE testing procedure. Two specimens averaged together should not depart more than 5 percent from an individual batch according to Section 9 of ASTM C469/C469M-02e1. However, specimen 1 and specimen 3 produced with clay departed from the average approximately 8 percent. Though this is out of the specification range, the averaged value of all three specimens produced acceptable results.

Chapter 5 Conclusions

5.1 Conclusions Overview

The object of the previous research was to determine the applicability of 1 day soaked LWA at reducing concrete shrinkage when used as an internal curing agent. Other soaking durations and a control concrete mixture were utilized in the research program to aid in this determination of using 1 day soaked LWA. Concrete shrinkage was evaluated through the use of a length change comparator and by embedding vibrating wire strain gages into concrete prisms. The research looked at the effect of the LWA on the fresh and hardened concrete properties as well. The unit weight, slump, temperature, compressive strength, MOR, and MOE were all measured to determine the effect caused through the introduction of LWA. The conclusions of the research are presented below.

5.2 Conclusions – Fresh Concrete Properties

The addition of LWA as an internal curing mechanism has little effect on concrete slump. Differences in slump were due to high range water reducer. The LWAs, both shale and clay, are very coarse with angular edges similar to the coarse aggregate limestone used in the control specimen. As such, the addition of 300 lbs LWA as a direct substitute for the limestone coarse aggregate had a minimal effect on the slump and workability aspects of the fresh concrete. Mixture workability was affected more by high range water reducer dosage than aggregate choice.

The addition of LWA as an internal curing mechanism did influence concrete unit weight. The control mixture had a unit weight of 150 lb/ft^3 while the shale and clay mixtures ranged from 137 lb/ft^3 – 140 lb/ft^3. The addition of LWA decreased the unit weight of the concrete because of the reduced specific gravity of the LWA. Further additions of LWA with a subsequent decrease in coarse limestone aggregate will continue to decrease concrete unit weight. Even though the unit weight for the LWA mixtures was lower than the control mixture, the LWA mixtures were still considered normal weight by ACI 318-08. In addition to the benefits of internal curing, a lower unit weight will reduce dead loads associated with concrete structures.

5.3 Conclusions – Hardened Concrete Properties

The compressive strength of the concrete was affected by the addition of LWA into the concrete mix. From the literature, Roberts (2006) determined that due to increased hydration of the cement matrix the compressive strength of the concrete would increase. However, the current research found the compressive strength to decrease with the addition of LWA. The LWA has lower specific gravities and lower compressive strength capacities than that of the coarse limestone aggregate. As such, the lower compressive strength of the LWA had more influence on the compressive strength of the concrete than the increased hydration products. This may be due to the w/c. The w/c used for the current research was 0.44 as specified by AHTD bridge deck specification. A w/c of 0.44 provides enough water to hydrate all necessary cement particles in the concrete which prevents the hardened concrete from self-desiccating. Therefore, the increased hydration products are not enough to overcome the lower compressive strength of the LWA.

Concrete shrinkage was directly affected by the addition of LWA into the mix design. The addition of LWA reduced the amount of shrinkage in the concrete. Provided in Tables 4.2.1.1 and 4.2.2.1 are the percent difference in shrinkage reduction of the LWA concrete in comparison to the control mixture. Utilizing a length change comparator the data indicated that the 7 day soaked clay and 1 day soaked shale specimens produced the least shrinkage. From the vibrating wire strain gage data it was determined that the 1 day soaked clay and 1 day soaked shale produced the least shrinkage. The data from both measuring systems concluded that the addition of LWA did decrease shrinkage in the concrete. However, the results were not conclusive regarding which LWA was most effective at reducing concrete shrinkage. Due to the variation from testing multiple concrete specimens for each mixture, confidence intervals were developed to determine the applicability of the LWA at reducing concrete shrinkage. The confidence intervals produced for the length change comparator data do not provide a statistical significance between any of the LWA specimens and the control mixture. The strain gage specimen's confidence intervals do provide a statistical significance to the shrinkage mitigation of the 1 day soaked LWA specimens in comparison to the control mixture. From the strain gage data, it was determined that the 1day soaked clay specimens produced the least shrinkage followed by the 1 day soaked shale specimens. The findings were in accordance with LWA moisture content as the lightweight clay was able to provide more water for cement hydration than the shale aggregate. In conclusion, the 90 percent confidence intervals do provide evidence that there is significant shrinkage reduction through the use of LWA as an internal curing agent. These findings support the use of internal curing as a concrete shrinkage mitigation technique when curing any type of concrete placement.

One of the goals of the research project was to determine if LWA soaked for 1 day was sufficient to observe the benefits of internal curing. The shrinkage results show that a 1 day soaking time does reduce shrinkage equivalently or better than extended soaking durations. This may be best seen in the confidence intervals derived for the strain gage data in Table 4.2.2.2. The 1 day soaked clay and shale specimens were the only two specimens that do not overlap the confidence interval of the control mixture.

The modulus of rupture or the flexural strength of the concrete was not affected by the addition of LWA. The control mixture had an average MOR of 845 psi while the clay and shale had an average MOR of 885 and 775 psi respectively. Unlike the modulus of elasticity, which is affected by the specific gravity of the materials inside the concrete, the MOR is affected more by bond between the materials. The addition of LWA did not result in a noticeable increase in the flexural strength of the concrete when compared to the control mixture. This is in agreement with the compressive strength data. The water provided by the LWA would need to produce more C-S-H than the control mixture for there to be an increase in the flexural strength of the concrete. However, the similar MOR data between all specimens, suggests that the 300 pound replacement rate of coarse LWA did not affect the bond strength of the concrete enough to produce a noticeable difference. ACI 318-08 provides a MOR prediction equation. Data put forth in Table 4.5.1 suggests that the prediction equation is applicable to LWA concrete. However the prediction equation is conservative for the control and LWA concrete.

The modulus of elasticity or stiffness of the concrete was affected by the addition of LWA into the concrete mix design. The modulus of elasticity decreased as LWA content increased. The specific gravity of the LWA was lower than the specific gravity of the limestone aggregate. As specific gravity decreased, strength also decreased, which decreased the modulus of elasticity. ACI 318-08 provides an equation to predict the MOE of the concretes in the research program. The ACI 318-08 prediction equation is applicable to LWA concrete mixtures. The MOE prediction equation provided very accurate measurements of the LWA concrete mixtures, but was conservative in its prediction of the control mixture.

Although the compressive strengths of the internally cured mixtures decreased when compared to the control mixtures, the compressive strength of the internally cured mixtures exceeded AHTD requirements. Furthermore, concrete shrinkage at 112 days was less when the concrete was internally cured. There was little difference in the flexural strength of the internally cured mixtures versus that of the control mixture. Similar to the compressive strength, the MOE of the internally cured mixtures was less than that of the control mixtures.

In conclusion, the current research focused only on internal curing with coarse LWA clay and shale materials. The research found that a 1 day soaking duration of the LWA produced equivalent shrinkage mitigation results when compared to longer soaking durations. Future research is needed to advance the knowledge of soaking duration of internally cured concrete. Further studies are needed to determine the applicability of extended soaking durations past the 7 day interval in internal curing. If the aggregate is left in submerged stockpiles then extended studies need to be conducted on LWA aggregate behavior past the 7 day study. For these extended studies, shrinkage reduction may just be one factor with the research, while the aggregate itself may be another. Does extended submersion breakdown the aggregate itself rendering it ineffective for internal curing and structural ability? Further studies are also needed into the use of pre-saturated fine LWA. Extended soaking durations did not affect the outcome of coarse LWA as long as a minimum soaking duration of 24 hours was provided. However, fine LWA has a different pore size distribution which may affect the ingress rate of water into the LWA. Also bulking is a factor in traditional sands and may also play a role in determining aggregate soak time of fine LWA's. These areas of research need to be investigated to determine their impact on internally cured concrete.

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