The Effect of Initial Curing Temperature and Duration on the 28-Day Compressive Strength of Concrete

by

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Abstract

The objective of this study was to determine the effect of initial curing temperature and duration on the 28-day compressive strength of concrete. To determine the relationship between initial curing temperature and duration on compressive strength, concrete cylinders were initially cured at six temperatures for three different initial curing durations. The six different initial curing temperatures used were 60 ºF, 68 ºF, 78 ºF, 84 ºF, 90 ºF, and 100 ºF, while the three initial curing durations used were 24 hours, 48 hours, and 72 hours. After the initial curing duration was complete, the cylinders were moved to final curing in a moist cure room that maintained a temperature of at 73.5 ± 3.5 ºF, until testing at 28 days. Eight different concretes were produced to have elevated fresh concrete temperatures to simulate summer placement conditions. The effects of Type I and Type III portland cement and fly ash, slag cement, and silica fume were assessed. Using an initial curing temperature of 68 ºF as a basis of comparison, the relative 28-day compressive strength differences were recorded for each initial curing temperature. An acceptable limit of ±10% was used to evaluate the strength differences.

The results confirmed that as initial curing temperature increases, the 28-day compressive strength of the concrete decreases. When initially cured at a temperature of 100 ºF, a maximum reduction of 23% in the 28-day compressive strength was observed. It is critical to maintain an initial curing temperature from 60 ºF to 80 ºF to remain within the chosen acceptable limit of ±10% for relative strength differences for concrete from the same batch. Additionally, it was concluded that within an initial curing temperature range from 60 ºF to 80 ºF, an increase to 72 hours in initial curing duration will not significantly affect the 28-day compressive strength of the concrete.

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Chapter 1: Introduction

1.1 Background

The 28-day compressive strength of concrete is the most common test to assess the quality of hardened concrete. When constructing concrete structures, it is crucial to ensure that all provided concrete has adequate strength. Therefore, very specific testing criteria are specified in AASHTO T 23 (2018) and ALDOT 501 (2022) to assess the quality of the concrete delivered to the jobsite. Concrete cylinders are most often tested at 28 days to ensure the concrete compressive strength is acceptable and meets or exceeds the strength assumed during the design phase. These strength results represent the quality of the concrete provided by the concrete producer and not the in-place strength of concrete, as the purpose of these 28-day compressive strength tests is to confirm that the concrete provided has acceptable strength. Figure 1-1 shows concrete test cylinders being prepared for initial curing.

Figure 1-1: Concrete cylinders being prepared for initial curing

Different curing environments between cylinders and in-place concrete will result in different concrete maturity and strengths. There are many different factors that can affect concrete compressive strength; therefore, one must take care to follow the specification requirements in AASHTO T 23 (2018) and ALDOT 501 (2022). These specifications require that for initial curing, concrete cylinders must remain in a temperature environment ranging from 60 ^{o}F to 80 ^{o}F for 24 to 48 hours. An initial curing box is used on jobsites in order to meet the temperature requirements of AASHTO T 23 (2018) and ALDOT 501 (2022). Figure 1-2 shows a typical initial curing box used to properly cure concrete test cylinders on the jobsite.

Figure 1-2: Initial curing box used at the jobsite

Cylinders shall be capped to prevent any loss of moisture. After the initial curing duration is achieved, cylinders are to be transported to their final curing location where they are demolded and placed in a final curing environment. The final curing method must maintain a temperature of 73.5 ± 3.5 ^{o}F and provide a relative humidity of 100% for the remainder of the curing duration. At the specified testing age, cylinders are broken to determine the compressive strength which is then compared to the specified design strength (f'_c) to determine if the concrete provided has adequate strength.

Again, because of the curing conditions and other parameters that are standardized, these strengths do not represent the in-place strength. By following the procedures set forth in AASHTO T 23 (2018) and ALDOT 501 (2022), the same concrete cured and tested in two different parts of the country should have similar strengths. Standard cured cylinders should be similar, while the two locations will have very different in-place concrete strengths due to temperature and humidity differences.

Research has shown that concrete cured at high temperatures has an initial strength gain that is much larger than concrete cured at low temperatures (Carino and Lew 2006). However, there is a cross-over effect that results in a lower long-term strength of concrete cured at elevated temperatures (Carino and Lew 2006). As a result, if an initial curing temperature of concrete cylinders is increased, it might have an increased 3- or 7-day compressive strength but lower 28-day strength. If producers, contractors, and testing agencies do not follow the proper specifications when testing concrete cylinders, the determined strength will not be accurate and can be low in some instances. When low 28-day cylinder strengths are recorded, investigations must take place that are costly and can cause major delays.

1.2 Research Objectives

There are two main objectives of this study. The first objective is to determine how much initial curing temperature affects the 28-day compressive strength of concrete. The second objective is to determine the effect of initial curing duration on the compressive strength of concrete. Although not a primary objective, evaluation of different concrete types allows a review of different cementitious materials and the effects of elevated initial curing temperatures on their 28-day compressive strength.

1.3 Research Approach

Laboratory batches of eight different concretes were produced to test the influence of initial curing temperature and duration on the 28-day compressive strength of concrete cylinders. For each batch, six different initial curing temperatures were used. These temperatures were 60 ºF, 68 ºF, 78 ºF, 84 ºF, 90 ºF, and 100 ºF. Using a reference of 68 ºF, the relative strength differences were determined for each initial curing temperature. For each temperature, three concrete cylinders were tested to determine an average 28-day compressive strength, and a fourth cylinder was used to measure temperature development in the concrete specimen. Concretes representative of ALDOT bridge applications were proportioned with varying types of cementitious materials. A total of 24 batches were produced consisting of eight different mixture proportions. Fresh properties of concrete were tested in accordance with AASHTO T 119 (2018), T 152 (2019), and T 309 (2020). Each concrete mixture was batched and tested at least twice. Concrete cylinders were allowed to initially cure for approximately 24 hours one time and 48 hours the other. Upon completion of all 24- and 48-hour batches, four concrete mixtures were chosen based off the results of the previous batches. The mixtures that had the most significant 28-day strength reductions were chosen for the 72-hour initial curing duration. An initial curing duration of 72 hours will permit the testing agency to move cylinders that are made on a Friday to their final curing location on Monday. This will prevent the need to move cylinders to their final curing location on either Saturday or Sunday. After all the 72-hour batches were completed, four verification batches were randomly chosen and tested. The 28-day strength of the concrete obtained due to different initial curing conditions was only compared to itself so the variations between compressive strengths of different batches is negligible.

1.4 Thesis Organization

Chapter 2 includes a literature review on concrete curing, materials and production of concrete, compressive strength testing and various initial curing research projects. Chapter 2 has an emphasis on previous work that reviewed the effect of elevated curing temperature on the compressive strength of concrete. The experimental research plan used in this study is covered in Chapter 3. The raw materials used and mixture proportions are also covered in Chapter 3. Chapter 4 includes the presentation and discussion of all the test results. A summary, conclusions, and recommendations of the study are covered in Chapter 5. Appendix A consists of the 28-day compressive strengths and temperature development plots of each batch. Appendix B consists of the data acquired for all verification batches. Appendix C consists of the batching procedures used in this study.

Chapter 2: Literature Review

This chapter covers the technical background needed to understand concrete production, concrete curing, and concrete strength testing. The main focus of this chapter is on the effect of elevated curing temperature on the compressive strength of concrete.

2.1 Introduction to Concrete Curing

2.1.1 What is Concrete Curing?

The strength of concrete is a complex relationship of factors that is directly impacted by its age. As concrete ages, its microstructure develops and the overall strength of the concrete increases. This increase in strength related to time is considered the concrete maturity (Carino and Lew 2001). As relative humidity decreases during curing, the compressive strength of cementitious materials within concrete will decrease (Sun et al. 2020). The maturity of concrete is an age-related method to determine the strength of concrete strength in which temperature plays a vital role (Carino and Lew 2001). Concretes cured at higher temperatures have an increased maturity and higher initial strengths but will have lower long-term strengths than those cured in lower temperatures (Carino and Lew 2001). Concrete curing is any method that helps to control the temperature development of concrete specimens and reduce the movement of moisture in and out of the concrete (Hameed 2009). Concrete cylinders used for quality assurance testing have specific conditions because the results from their strengths are used for acceptance (NRMCA 2014). When curing concrete cylinders, it is important to remember that the cylinders do not represent the in-place strength of concrete, but the quality of the concrete delivered to the jobsite (Obla et al. 2018). If field cured cylinders are allowed to be the basis of acceptance for concrete, the concrete producer may be unfairly judged on the quality of their product.

2.1.2 Methods of Curing

The most common methods for curing in-place concrete members are fogging, applying chemical membranes, and wet curing with burlap and/or plastic sheets. The possibilities to adequately cure in-place concrete members are numerous, but the ones mentioned are the most common (Mather 1990). When curing concrete cylinders, there are strict specifications, namely AASHTO T 23 (2018) and ALDOT 501 (2022), that must be followed. Standard curing of concrete in accordance with AASHTO T 23 (2018) and ALDOT 501 (2022) includes initial and final curing. Initial curing of concrete occurs at the jobsite, followed by final curing in a controlled laboratory setting. These specifications have been developed to reliably and repeatedly represent the strength of the concrete provided to the jobsite and not the in-place concrete strength. By controlling the curing environment of cylinders, the outside temperature and humidity cannot affect the strength development of the concrete. It is well known that the curing temperature of concrete will impact the strength development and must be managed to prevent any increase or loss of strength (Arslan et al. 2017).

2.1.3 Necessity of Curing

Concrete curing is necessary to ensure adequate strength and durability of in-place concrete and for standardized strength testing to determine the quality of concrete delivered to the jobsite. Although the concrete producer is responsible for producing the concrete and delivering it to the jobsite, it is not the concrete producer's responsibility to properly place and cure concrete. Any number of things could affect the in-place strength, and therefore using standardized testing when determining the strength of concrete is crucial to accurately represent the strength of the concrete delivered to the jobsite (NRMCA 2014). When concretes are specified, they are tested using the same standards that are supposed be used to make concrete cylinders on the jobsite (Goeb 1986). A concrete producer's cylinder strength is determined in an environment that meets the temperature and humidity requirements of AASHTO T 23 (2018). Therefore, when cylinders are not made, cured, and tested in accordance with AASHTO T 23 (2018) and ALDOT 501 (2022), the concrete strength is potentially not a reflection of the actual strength provided.

Many errors in producing and curing cylinders will result in lower compressive strengths. NRMCA (2014) stated that:

> Almost all deficiencies in handling and testing cylinders will result in a lower measured strength. All violations add up to cause significant reductions in measured strength. The frozen cylinders; extra days in the field; damage during transportation; delay in stripping molds and curing at the lab; improper capping; and insufficient care in breaking cylinders. (NRMCA 2014)

The statement from NRMCA (2014) demonstrates how there are many different variables that can reduce the strength of cylinders and it is crucial to prevent any deficiencies because they quickly add up to potential large strength decreases. When low 28-day cylinder results occur, all stakeholders are forced to investigate the potential causes of the low 28-day cylinder strength, project delays occur, and the Contractor and concrete supplier have to get ready to deal with the consequences of the potentially substandard concrete. For economical and, more importantly, safety concerns, it is crucial to follow the proper test procedures to obtain an accurate representation of the concrete strength.

2.2 Concrete Materials and Production

2.2.1 General Production

The four materials that are used to produce concrete are cementitious materials, fine aggregate, coarse aggregate, and water. While many different cementitious materials exist, portland cement is most commonly used (Darwin et al. 2016). Portland cement is produced in many different types. Many commercial projects will consist of Type I/II or Type III cement. Type I/II cement has increased resistance to sulfate attack (Struble 2006). Type III cement is often used because it can result in increased early-age strengths (Struble 2006).

Materials known as supplementary cementitious materials (SCMs) are often used to replace the amount of portland cement used for economic, environmental, and performance purposes (Zamora et al. 2019). Typical SCMs in Alabama include fly ash, slag cement, and silica fume.

2.2.2 Portland Cement

2.2.2.1 Introduction

Cement is a manufactured material that upon reacting with water transforms the materials from a plastic state to a hardened material (Struble 2006). Although, cement is one of the most importance materials in concrete, it is a goal of the industry to limit cement content in concrete mixtures to help mitigate the emission of global greenhouse gases (Miller and Moore 2020). By reducing cement content, the total cost and the environmental impact is reduced (Schlorholtz 2006). Although reducing cement usage is important, it is crucial that the physical properties of concrete are not negatively impacted when substituting with other materials.

2.2.2.2 Hydration Process

Immediately after cement comes in contact with water a chemical reaction known as hydration begins (Bullard et al. 2010). This hydration process produces a cement paste that stiffens and eventually becomes a solid material (Darwin et al. 2016). During the hydration process substances such as C_3S and C_3A react with water to form hydration products (Bullard et al. 2010). Cement hydration is an exothermic process, releasing heat (Samarai et al. 1983). As the heat of hydration is increased, the rate of hydration in increased and vice versa. Figure 2-1 shows the relationship between curing temperature and the heat of hydration for cement.

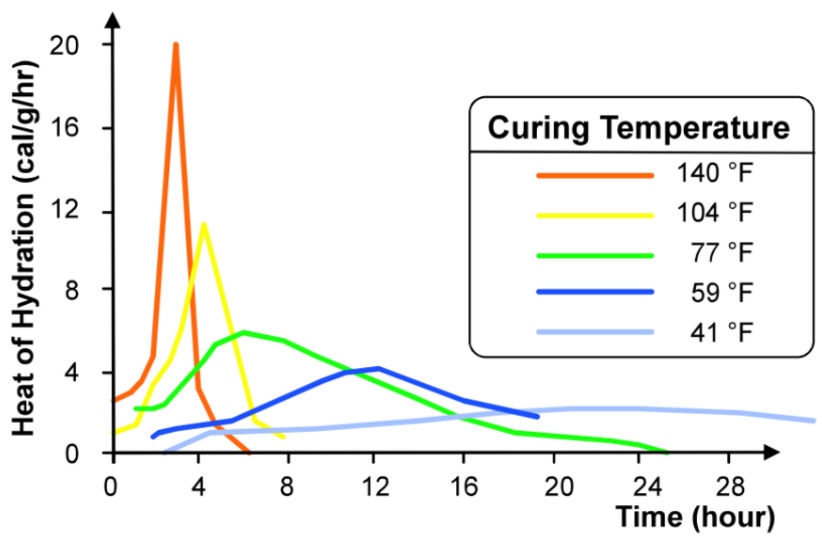

Figure 2-1: Cement heat of hydration (adapted from Samarai et al. 1975)

The data in Figure 2-1 shows how as the heat of hydration increases, the total time for the hydration process to finish decreases. As the temperature is increased during hydration the initial compressive strengths will be higher, but the long-term strengths will decrease (Dejeto and Kurumisawa 2015). While increased short-term strengths can speed up the construction process, loss in long-term strength might lead to potential problems in the future.

2.2.3 Supplementary Cementitious Materials

Supplementary cementitious materials (SCMs) are commonly used when producing concrete. SCMs are a variety of materials that are “finely divided and therefore form pastes to supplement portland cement paste” (Schlorholtz 2006). SCMs are most often used for their economic and environmental impacts as they are by-products of other industries and have little production of greenhouse gases (Darwin et al. 2016). These by-products would be unnecessary waste if not reused in the concrete industry. In addition to helping costs and the environment, SCMs have certain advantages when compared to ordinary portland cement.

2.2.3.1 Fly Ash

Fly ash is the byproduct of coal powered energy production and is often used as a replacement to portland cement (Darwin et al. 2016). Class C and Class F fly ash are the most common fly ashes and are characterized by their carbon content. Class F fly ash will have a CaO content less than 18% and Class C fly ash will have a CaO content greater than 18% (ASTM C 618 2022). Class F fly ash will have a slower early-age strength development but is much more effective at mitigating alkali-silica reactions and has better sulfate resistance (Schlorholtz 2006).

2.2.3.2 Slag Cement

Slag cement is a byproduct of the production of iron (ACI Committee 226 1987). Slag cement can be used in large dosages of approximately 20-80% by mass (Prusinski 2006). It has a very slow rate of hydration and therefore is ideal for mass concrete applications to reduce the temperature development during the hydration process (ACI Committee 226 1987). The durability of slag cement concrete is increased due to a decrease in permeability with the use of slag cement (Prusinski 2006). Alkali-silica reaction is mitigated, and sulfate resistance is increased in slag cement concretes. (Hogan and Meusel 1981).

2.2.3.3 Silica Fume

Silica fume is an extremely fine non-crystalline silica produced as a byproduct of elemental silicon or alloys containing silicon (ACI CT 2018). It is composed of very small droplets of condensed silicon dioxide (SiO_2). Silica fume has a very high SiO_2 content, varying based on the specific silicon content of the alloy used to produce the silica fume (Campos et al. 2020). The increased surface area of silica fume due to its fineness increases the water demand, requiring the use of a high-range water-reducing admixture (Liew 2021). Silica fume is a pozzolan and reduces the CH composition, strengthening the interfacial transition zone (ITZ) (Campos et al. 2020). The fineness also contributes to the increased ITZ properties (Campos et al. 2020). Typically, a maximum of 10% can be added to a mixture (Kunal and Siddique 2016). Silica fume increases the water demand and reduces workability when used in high dosages (Campos et al. 2020). Drying shrinkage can be reduced with the addition of silica fume (Tazawa and Yonekura 1986).

2.2.4 Aggregate

Coarse and fine aggregates are added to a concrete mixture for a variety of purposes. The use of aggregates reduces the amount of cement paste needed (Graves 2006). The type of aggregate used influences the workability of fresh concrete (Graves 2006). For hardened concrete, the aggregates can affect the strength (Graves 2006).

Coarse aggregate is typically a type of crushed limestone or river gravel and comes in many different sizes and gradations. By increasing the amount of aggregate, the amount of paste in the concrete is reduced and the drying shrinkage is reduced (Karaguler and Yatagan 2018). Fine aggregate consists of particles passing the No. 4 sieve while coarse aggregates are retained on the No. 4 sieve (ACI Committee E-701 2016). When proportioning concrete, the batch weights of aggregates should be quantified in saturated-surface-dry state (Darwin et al. 2016).

2.2.5 Water

Water is necessary for the hydration process of concrete. Water is crucial in activating the cement products, but the more water in a concrete mixture, the lower the compressive strength will be when the cementitious material content remains consistent (Darwin et al. 2016). It is crucial when designing concrete mixtures to ensure an adequate w/c ratio. The lower the w/c ratio is, the stronger the concrete will be (Darwin et al. 2016). Although, the concrete may be stronger with a low w/c ratio, the workability will decrease. Regarding proportioning a concrete mixture, "as water is added, the plasticity and fluidity of the mix increase (that is, its workability improves), but the strength decreases because of the larger volume of voids created by the free water" (Darwin et al. 2016). Therefore, a minimum amount of water is desired for every concrete mixture.

2.2.6 Batching

Batching of concrete must follow ASTM C 192 (2019). When performing laboratory batches, the final product must closely resemble the product that will be supplied to the field. Therefore, it is crucial to follow these procedures to ensure that representative mixtures are produced.

2.3 Making and Curing Concrete Specimens

2.3.1 Making of Concrete Cylinders

Proper procedures must be followed when making concrete cylinders for the purpose of strength testing. Before molding of specimens can begin, sampling and testing of the provided concrete must take place. On typical ALDOT projects, the tests include slump, air, and temperature tests that must adhere to AASHTO T 119 (2018), T 152 (2019), and T 309 (2020), respectively. Molding of 6" x 12" cylinders must be done in accordance with AASHTO T 23 (2018) and ALDOT 501 (2022).

2.3.2 General Curing of Concrete Structures

Concrete must be adequately cured to ensure that the desired hardened concrete properties are achieved. Concrete curing as defined by ACI CT (2021) is "action taken to maintain moisture and temperature conditions in a freshly placed cementitious mixture to allow hydraulic cement hydration and (if applicable) pozzolanic reactions to occur so that the potential properties of the mixture may develop." According to Darwin et al. (2016) "curing can be achieved by keeping exposed surfaces continually wet through sprinkling, ponding, or covering with plastic film or by the use of sealing compounds, which, when properly used, form evaporation-retarding membranes." Concrete curing must be done on all structures to ensure it develops the proper strength. Some common forms of proper curing are using wet burlap ponding, and fogging (Prusinski 2006). Certain curing compounds can also be applied to freshly placed concrete to help ensure adequate curing (Vandenbossche 1999).

2.3.3 Curing of Concrete Cylinders

There are two types of curing conditions for concrete test cylinders included in AASHTO T 23 (2018). The two curing conditions are standard curing and field curing.

2.3.3.1 Standard Curing

The curing of concrete cylinders for quality assurance purposes on ALDOT projects is regulated by AASHTO T 23 (2018) and Section 501 of the ALDOT Standard Specifications for Highway Construction (ALDOT 501 2022). Standard curing according to AASHTO T 23 (2018) consists of initial curing and final curing.

Initial curing occurs on the jobsite for a duration of up to 48 hours (AASHTO T 23 2018). In regard to initial curing AASHTO T 23 (2018) states "immediately after molding and finishing, the specimens shall be stored for a period up to 48 h in a temperature range from 16 to 27 °C (60 to 80 °F) in an environment preventing moisture loss from the specimens" (AASHTO T 23 2018). Although no minimum initial curing duration is specified in AASHTO T 23 (2018), the cylinders are not allowed to be transported to final curing until 8 hours after final set. For ALDOT projects, the moist environment must be maintained using a "cylinder curing box with a minimum capacity of 22 test cylinders 6" X 12" (150 mm X 300 mm) in size, equipped with heating/cooling capabilities, automatic temperature control, and a maximum/minimum (high/low) temperature readout" (ALDOT 501 2022). In AASHTO T 23 (2018) high-strength concrete, specified as concrete that has a design strength greater than 6000 psi has the same duration requirements and has stricter initial curing temperature requirements from 68 °F to 78 °F.

For final curing, AASHTO T 23 (2018) states that "on completion of initial curing and within 30 min after removing the molds, cure specimens with free water maintained on their surfaces at all times at a temperature of 23 ± 2°C (73.5 ± 3.5°F) using water storage tanks or moist rooms". The final curing conditions of AASHTO T 23 (2018) are the same for normal- and high-strength concrete. Moist rooms are chambers that are temperature regulated and have a fog machine that keeps the chamber at 100% relative humidity in accordance with AASHTO M 201 (2015). Water storage tanks are temperature-controlled tanks in which cylinders are completely submerged in accordance with AASHTO M 201 (2015). After demolding, cylinders are placed in the moist cure room or are completely submerged within their respective water bath. Final curing takes place until the desired age of the concrete is met. Common standard curing durations are 3, 7, 28, and 56 days, with an age of 28 days the often used for acceptance testing.

In ALDOT 501 (2022), the standard curing of concrete cylinders is similar to AASHTO T 23 (2018); however, it includes a minimum initial curing duration while AASHTO T 23 (2018) does not. It specifies that cylinders must remain in an "initial curing period of not less than 24 hours or more than 48 hours. During the initial curing period, the specimens shall be stored in a moist environment at a temperature range between 60 ºF to 80 ºF (16 ºC to 27 ºC), preventing any loss of moisture for up to 48 hours" (ALDOT 501 2022). ALDOT 501 (2022) has included stricter specifications regarding the initial curing duration for concrete cylinders used for acceptance criteria.

2.3.3.3 Field Curing

While standard curing is used to verify the quality of the concrete delivered to the project, field curing is used to help estimate the strength of in-place concrete. When field curing is done, specimens are molded and allowed to cure as close to the actual structure as possible. (AASHTO T 23 2018). By replicating the curing environment of in-place concrete, cylinders will be subject to similar temperature and humidity values (Obla et al. 2005). Field curing should never be used for the acceptance of concrete as "field-cured specimens are used to determine if a structure may be put into service, evaluate the adequacy of curing and protection of the concrete in the structure, and to help determine form and shoring removal times" (Obla et al. 2018).

2.4 Concrete Compressive Strength

The compressive strength of concrete is the most common test used to assess the acceptability of concrete. Many factors affect the compressive strength of concrete such as water-to-cementitious materials ratio, air content, concrete age, material composition, curing conditions, and testing conditions (Ozyildirim and Carino 2006). When determining the compressive strength of concrete, the use of concrete cylinders is most often used in the United States. A compression machine is used to apply an axial load on the cylinder. The load is increased at a controlled rate until the cylinder fails. Using the failure load and surface area of the cylinder, the strength of concrete can then be determined in psi.

2.4.1 Standard Test Specifications for Compressive Strength

The standard specification for determining the compressive strength of concrete cylinders either molded or taken as drilled cores is AASHTO T 22 (2020). Using this standard specification, concrete strengths can be accurately tested and evaluated to determine for acceptability of the concrete. Like all other specifications, it is crucial to follow AASHTO T 22 (2020) to accurately assess the compressive strength of the concrete provided by the producer.

2.4.2 Concrete Age and Maturity

The age of concrete is one of the most significant factors for the strength of concrete. Figure 2-2 illustrates the strength development of concrete with time using a concrete that consists of 100% Type I cement.

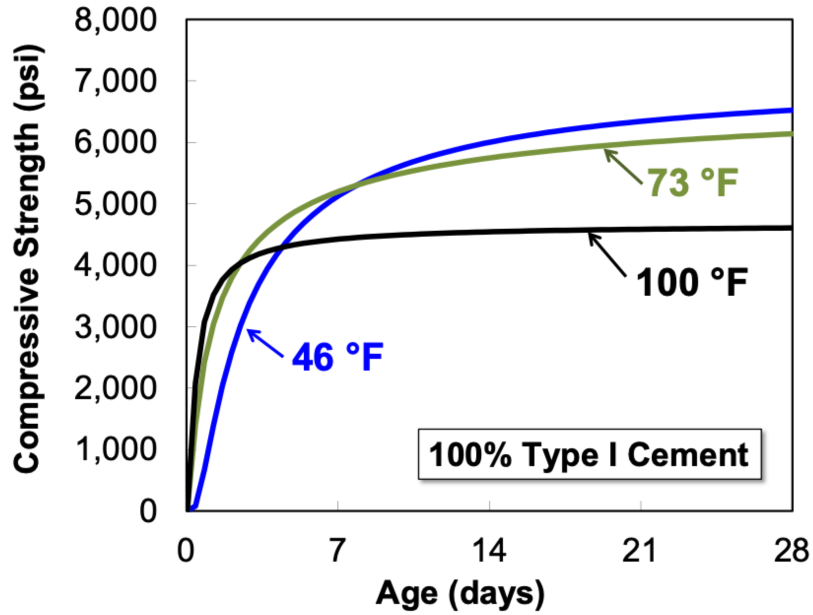

Figure 2-2: Compressive strength development for specimens made with 100% Type I cement cured at different constant temperatures (adapted from Brooks et al. 2007)

Figure 2-2 shows that during the first 3 days, the strength gain is rapid and as the age reaches 28 days the strength has stabilized. Also, the initial strengths are largest when cured at 100 °F, but the long-term compressive strength is greatest when cured at 46 °F.

Although age is an important variable that affects the strength development of concrete, the rate of strength is also affected by the temperature of the concrete and surrounding environment (Carino and Lew 2001). A method known as the maturity method can be used to account for the effects of temperature and time on the compressive strength of concrete (Carino and Lew 2001). According to Saul (1951) "concrete of the same mix at the same maturity (reckoned in temperature-time) has approximately the same strength whatever combination of temperature and time go to make up that maturity." Initially, when exposed to higher temperatures, the maturity of concrete is much more than if exposed to lower temperatures and results in increased early-age compressive strengths (Carino and Lew 2001). At a certain age, there is a cross-over and the concretes exposed to warmer temperatures have lower compressive strengths when compared to concrete with cooler temperature development (Carino and Lew 2001). Carino and Lew (2001), coined this change in strength between warm and cold concrete temperatures as the "cross-over effect" and it shows how the classical maturity method is insufficient in accounting for temperature effects on the long-term compressive strength of concrete. Tests results for the maturity method show that "for equal values of the maturity index, specimens with higher early-age temperatures resulted in higher initial strengths and lower long-term strength" (Carino and Lew 2001).

2.5 Research on the Effect of Initial Curing on Concrete Strength

Special care must be taken when concreting in hot weather. Hot weather concreting occurs mostly due to hot air temperatures but can result from various other issues as well. The ACI CT (2021) defines hot weather concreting as "one or a combination of the following conditions that tends to impair the quality of freshly mixed or hardened concrete by accelerating the rate of moisture loss and rate of cement hydration, or otherwise causing detrimental results: high ambient temperature; high concrete temperature; low relative humidity; and high wind speed." As mentioned in Section 2.2.2.2, when the hydration of cement is accelerated, the long-term compressive strength may decrease.

Some research projects have looked at the effect of initial curing temperature on the compressive strength of cylinders. A comprehensive study on the effect of mixing temperature on the strength of concrete was performed in 1958 by Klieger. The study consisted of the evaluation of curing temperatures ranging from 40 °F to 120 °F. In the study, concrete cubes were exposed to various curing conditions and durations. After, the compressive strengths were determined and compared, Klieger (1958) found that cylinders cured at 105 °F for 7 days and then moved to normal curing of 73 °F had approximately 1000 psi less strength than when compared to cylinders that remained at 73 °F for the entire 28 days. It was concluded that "increasing the initial and curing temperatures results in considerably lower strengths at 3 months and one year" (Klieger 1958). This research shows how important initial curing temperature is to the compressive strength of concrete specimens.

Regarding the initial curing duration, Bloem (1969) studied the effect of high initial curing temperature for various durations. Cylinders were initially cured at 100 °F for 1, 3, and 7 days and then moved to final curing until testing at 28-days. Compressive strengths of the three different curing environments compared to standard cured cylinders are shown in Figure 2-3.

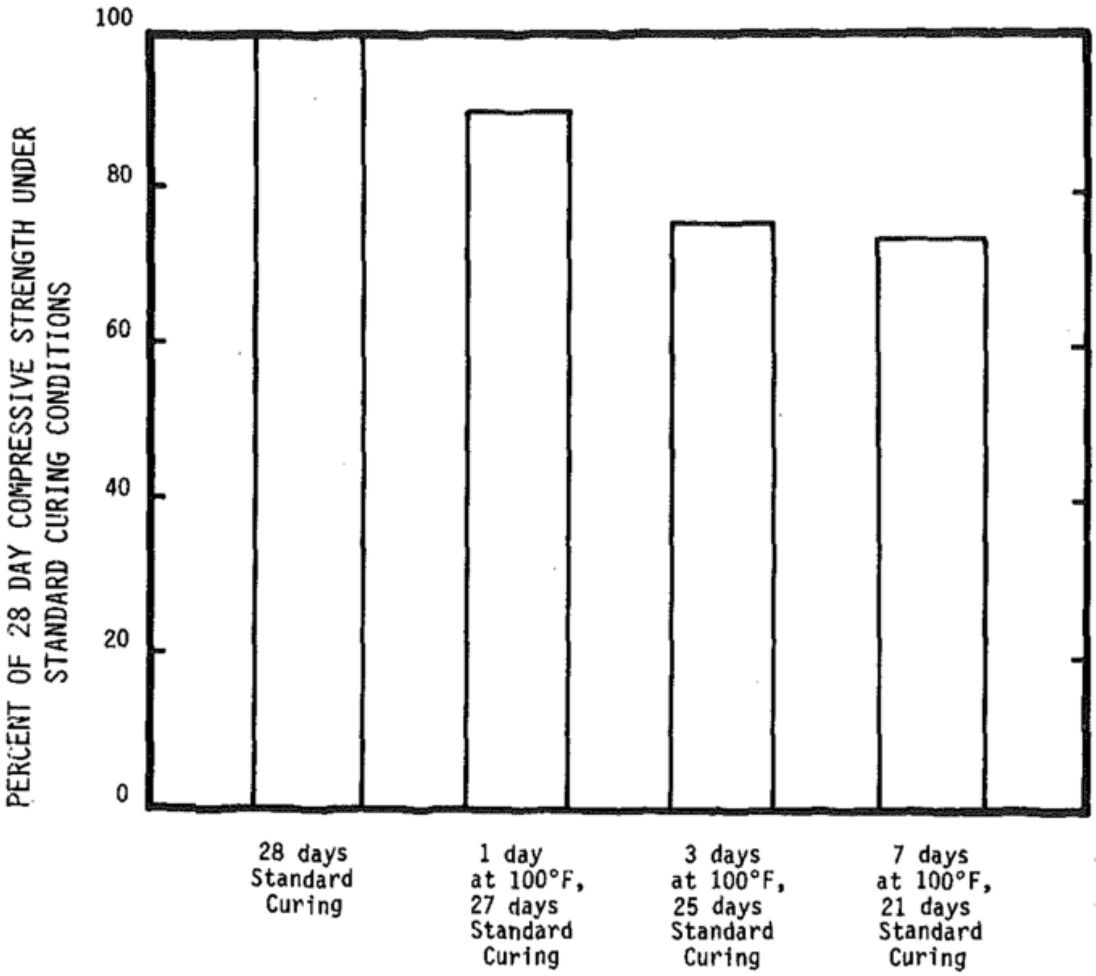

Figure 2-3: Effect of high initial curing temperature (Bloem 1969)

Figure 2-3 reveals that as initial curing duration is increased the compressive strength of concrete cylinders cured at 100 °F will reduce when compared to concrete cured under standard conditions. The longer the concrete was subjected to the elevated initial curing temperature, the more 28-day compressive strength is lost when compared to the cylinders that remained in standard curing.

Meininger (1983) reviewed the effects of initial curing temperature and duration on the compressive strength of concrete cylinders. This work was performed in response to changes in ASTM C 31 and the study consisted of four different initial curing conditions and two initial curing durations. The four different initial curing conditions evaluated were 60 °F water, 60 °F air, 80 °F water, and 80 °F air. The temperature range consisted of the current AASHTO T 23 (2018) range from 60 °F to 80 °F. Cylinders that were cured in the air were done within a controlled laboratory and had plastic coverings to prevent moisture loss. The cylinders cured in water were fully submersed in a water tank. After the desired initial curing duration, cylinders were demolded and moved to standard curing in a moist room for the remainder of the 28 days. The compressive strength results are summarized in a Table 2-1.

Table 2-1: Summary of average strength for each condition and time of curing (adapted from Meininger 1983)

Condition	Average Strength at 28-Days, psi (%)	
	Cement A	Cement B
	INITIAL ONE-DAY FIELD CURE	
60 °F water	6078 (100)	6094 (100)
60 °F air	5611 (92)	5926 (97)
80 °F water	5424 (89)	5692 (93)
80 °F air	4896 (81)	5373 (88)
	INITIAL TWO-DAY FIELD CURE	
60 °F water	6069 (100)	6078 (100)
60 °F air	5399 (89)	5810 (95)
80 °F water	5353 (88)	5629 (92)
80 °F air	4842 (80)	5290 (87)

These results show that for both mixtures, the water-cured cylinders have a higher compressive strength than the air-cured cylinders for both initial curing temperatures and durations. Additionally, the average strengths are greater for the cylinders initially cured at 60 °F when compared to the cylinders cured at 80 °F. Meininger (1983) concluded that “increasing the initial curing period from one to two days only reduced compressive strength by about 1%." The results further suggest that a lower initial curing temperature will result in stronger concrete at 28 days and that initial curing duration is not as significant as the temperature in affecting 28-day concrete compressive strength.

Research was performed by Obla et al. (2005) at the NRMCA Laboratory to determine the effect of non-standard curing on compressive strength of concrete. In this study, four different curing environments were tested. The curing environments consisted of 1) standard curing of 73 °F and 100% relative humidity for the control, 2) laboratory air-drying at 73 °F for the entire curing duration, 3) curing outside for 48 hours and then moist curing for the remainder, and 4) outside curing for the entire curing duration. Initially, the tests were performed to simulate cold weather concreting with an average air temperature of 32 °F. Compressive strengths were tested at 1, 3, 7, 28 and 90 days and the results showed the effect of different curing environments on compressive strength. Table 2-2 shows the strength reductions relative to the standard curing control cylinders. Figure 2-4 includes a plot of the compressive strength versus concrete age, as well as the average daily air temperature.

Table 2-2: Strength results for cold weather (adapted from Obla et al. 2005)

Age, days	Control Strength, psi (1)	Percent of control strength at same age		
		Lab Air-dry (2)	Out 48 h, moist (3)	Outside (4)
1	1508	-	-	-
3	2828	-	46%	14%
7	3852	95%	68%	40%
28	4745	88%	78%	66%
90	5374	74%	90%	82%

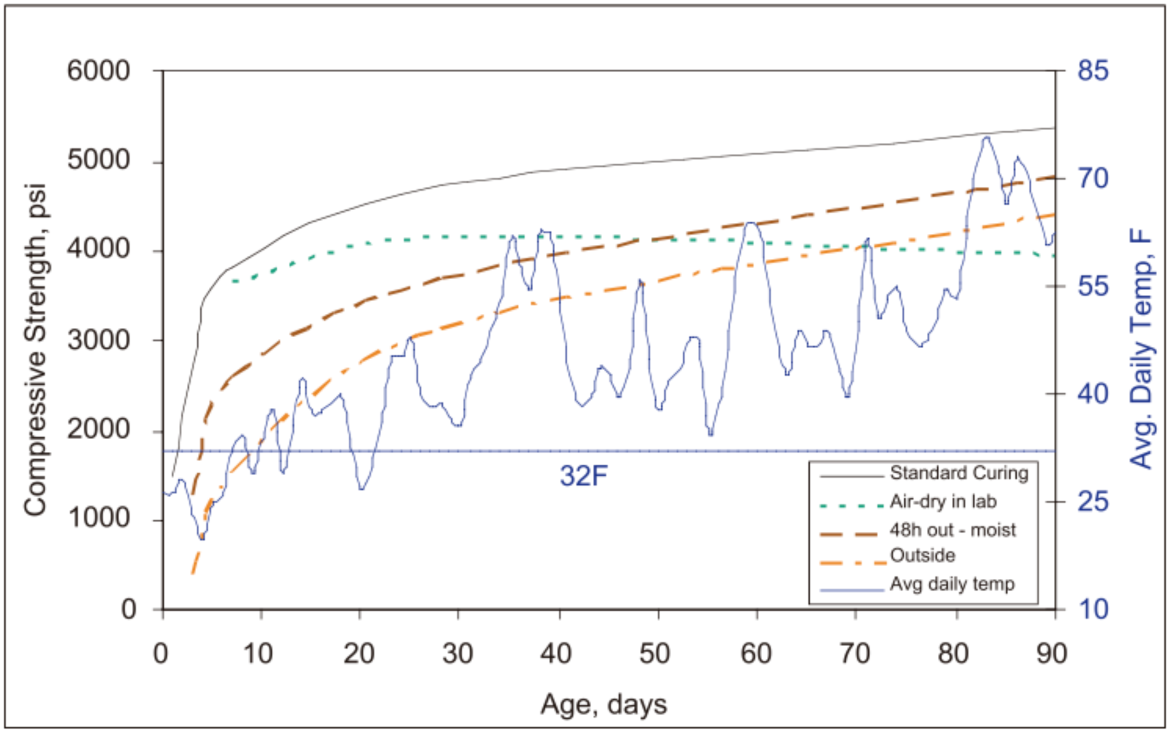

Figure 2-4: Plot of concrete strength – cold weather exposure (adapted from Obla et al. 2005)

At each age tested, all three non-standard curing environments had compressive strengths less than the standard cured cylinders. The concrete cured in the outside (4) environment, had the lowest percent of the control compressive strength at ages of 3, 7, and 28 days. At a concrete age of 7 and 28 days, the concrete cured in the lab, air-dry (2) environment had the largest percent of the control compressive strength. However, at 90 days the compressive strength for the concrete cured in the lab, air-dry (2) environment had the lowest percent of the control compressive strength. The concrete cured outside for 48 hours and then moist curing for the remainder had the largest percentage of the control compressive strength at an age of 90 days. Table 2-2 shows that at 7 and 28 days, both of the outdoor curing environments, had larger compressive strength reductions than the lab, air-dried concrete. As the curing duration increases the low curing temperatures begin to be advantageous when comparing the compressive strengths of the concrete cured outside to the strengths of the lab, air-dried specimens. Figure 2-4 shows a cross-over effect between the 48 out-moist and the air-dry lab cylinders at approximately 52 days. In addition to simulating cold weather concreting, this study was repeated during warm weather conditions.

For the warm weather study, the lab, air-dry cylinders were not tested. The results for the warm weather study are summarized in Table 2-3. Figure 2-5 includes a plot of the compressive strength versus concrete age, as well as the average daily air temperature for the warm weather conditions.

Table 2-3: Strength results for warm weather (adapted from Obla et al. 2005)

Age, days	Control Strength, psi (1)	Percent of control strength at same age	
		Out 48 h, moist (2)	Outside (3)
1	784	180%	180%
3	2370	89%	86%
7	3176	81%	90%
28	4384	78%	84%
90	5659	84%	80%

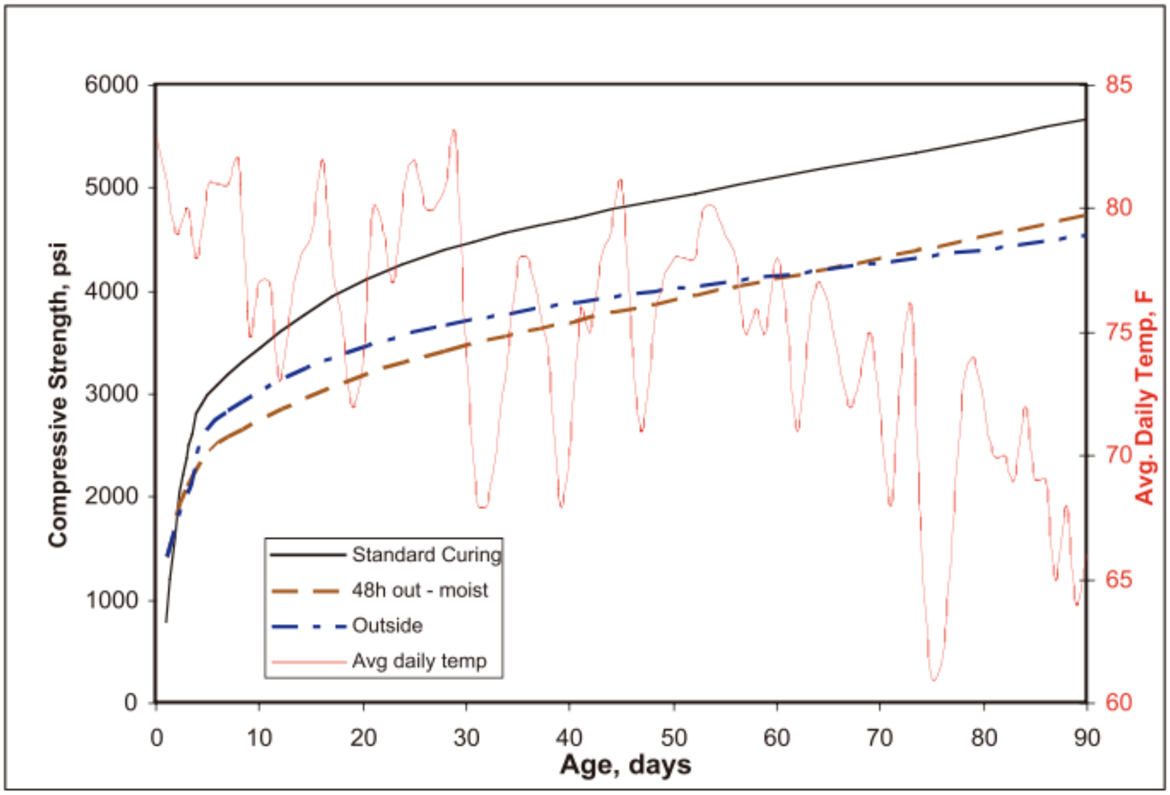

Figure 2-5: Plot of concrete strength – warm weather exposure (adapted from Obla et al. 2005)

As expected, the one-day compressive strengths are greater than the control. This is due to the heat increasing the rate of hydration and accelerating the strength development. For the outside (3) environment, an increase in the percent of control compressive strength occurred when the curing duration was increased from three to seven days and decreased as the curing duration was extended to 28 and 90 days. The concrete cured outside for 48 hours and then moist cured for the remainder had a decreasing percent of the control strength as the curing duration increased. For both curing environments, a cross-over effect occurred within the first ten days. Figure 2-5 shows the cross-over effect and illustrates how much more effective standard curing was for the compressive strength of concrete during the warm weather exposure conditions.

Although the warm and cold weather studies have different results, they lead to the same conclusions. When concrete cylinders are not cured properly, the compressive strengths can be reduced. The results show that initial curing temperatures play a major role in the strength development of concrete.

Another study performed by Un and Baradan in 2011 reviewed strength differences in respect to temperature and humidity in portland cement mortar. They found that "critical climatic conditions created in study caused decreases up to 40% in compressive strength and 30% in flexural strength compared to standard curing. This means that poor curing conditions and unproper climatic conditions may create significant undesirable results" (Un and Baradan 2011).

The effect of high curing temperature on ground-granulated blast furnace slag was evaluated by Shumuye et al. (2021). Four different concrete mixtures were tested with varying amounts of slag cement. Each mixture was cured at two different temperatures: a normal curing temperature of 20 ± 2 °C (68 ± 3.6 °F) and an elevated curing temperature of 45 ± 2 °C (113 ± 3.6 °F). The relative humidity remained consistent at 95%. Concrete specimens were demolded after 24 hours and the compressive strengths were determined at 7, 28, and 56 days. Table 2-4 shows the strength results of each batch. The G-30 mixture consists of a concrete with 30% of the cementitious material consisting of slag cement. The G-50 and G-70 consists of concrete that includes 50% and 70% of the cementitious material as slag cement, respectively.

Table 2-4: Variation in compressive strength versus curing time and temperature under different slag replacement ratios (adapted from Shumuye et al. 2021)

Curing time (day)	Mixture Code								
	G-30			G-50			G-70		
	MPa		%	MPa		%	MPa		%
	NT.C	ET.C		NT.C	ET.C		NT.C	ET.C	
7	22.7	27.4	20.7	22.4	28.8	28.6	17.7	20.3	14.7
28	32.4	30.0	-7.4	30.0	30.1	0.3	29.2	22.7	-22.2
56	37.4	31.4	-16.0	30.9	30.1	-2.6	28.7	23.3	-23.1

NT.C normal temperature curing
ET.C elevated temperature curing

From Table 2-4, it is determined that the initial compressive strength of each concrete at 7 days is larger when the curing temperature is elevated. However, as the concrete ages, the long-term strengths begin to decrease as the curing temperature is elevated. When reviewing the G-50 batch, the 7-day compressive strength was 28.8 MPa (4180 psi) for the elevated curing temperature environment, while the normal curing environment has a compressive strength of only 22.4 MPa (3250 psi), a strength difference of 28.6%. At 7 days, the elevated curing temperature helped the concrete gain strength. The 28-day strengths are almost identical at 30.0 (4350 psi) and 30.1 (4360 psi) MPa. This is a strength difference of only 0.3%, so these results are similar. However, once the concrete age is 56 days, the elevated curing temperature causes the compressive strength to decrease. A compressive strength of 30.9 MPa (4480 psi) is recorded for the normal-cured concrete, while only 30.1 MPa (4360 psi) was recorded for the elevated-cured concrete. The elevated-cured concrete had a strength reduction of only 2.6% when compared to the normal-cured specimens. The same trends are shown when reviewing the results of the G-30 and G-70 mixtures. At 28-days the strength differences for the elevated-cured specimens are -7.4% for the G-30 and -22.2% for the G-70 mixture when compared to the normal-cured specimens. At 56-days the strength differences of the elevated cured specimens are -16% for the G-30 and -23.1% for the G-70 mixture when compared to the normal-cured specimens. The overall findings suggest that when curing temperature is increased the long-term compressive strength of concrete will be reduced.

From the various research studies reviewed it can be concluded that the curing temperature of concrete plays a vital role in developing the compressive strength of concrete specimens. From the extensive amounts of tests performed, it can be concluded that if the curing temperature of concrete specimens is not adequately controlled, the 28-day compressive strength will be reduced in a significant manner. Although some studies have evaluated the effect of curing temperature on compressive strength, more research on the effects of temperature and duration of initial curing on the compressive strength of concrete needs to be performed.

Chapter 3: Laboratory Testing Procedure

This chapter provides the experimental plan used to determine the effect of initial curing temperature and duration on the 28-day compressive strength of concrete. Details on the raw materials used, mixture proportions, batching and molding of concrete cylinders, initial and final curing methods, and compressive strength testing are included. Also, the method of analysis used to compare compressive strength results is covered in this chapter.

3.1 Introduction

The main objectives of the study were to determine the effect of initial curing temperature and duration on the 28-day compressive strength of concrete cylinders. Using the Auburn University Advanced Structural Engineering Laboratory, tests were performed to determine how varying initial curing temperatures and initial curing durations will affect the 28-day compressive strengths of concrete. When performing laboratory tests, there was an emphasis on using elevated initial curing temperatures that are typically experienced during summer months in Alabama. Various concrete mixtures containing different SCMs were tested to best understand the effects of initial curing temperature and initial curing duration on the 28-day compressive strength.

3.2 Objectives

It is well known that the curing temperature of freshly placed concrete can greatly impact the long-term compressive strength of concrete. In most cases, an increase in temperature will result in an increased rate of early-age strength gain. However, as discussed in Section 2.1.1, a decrease in the long-term strength is generally expected when cured at elevated temperatures. The objective of the laboratory procedure was to determine how the 28-day compressive strength of concrete cylinders was affected when initially cured at a variety of temperatures and durations. Using initial curing temperatures of 60 °F, 68 °F, 78 °F, 84 °F, 90 °F, and 100 °F, the relative strength gain or loss was determined with respect to the cylinders initially cured at 68 °F. Other than the use of these initial curing temperatures, some of which are non-standard, all other requirements of AASHTO T 23 (2018) and ALDOT 501 (2022) were followed. Additionally, initial curing durations of approximately 24 hours, 48 hours, and 72 hours were used to investigate the relationship between 28-day compressive strength and initial curing duration. Regarding the initial curing temperature, it was expected that as the initial curing temperature increases the 28-day compressive strength of cylinders should decrease. The initial curing duration was expected to affect the 28-day compressive strengths in a more complex manner. For the initial curing temperatures warmer than the final curing temperature of 73.5 ± 3.5°F, the increase in duration was expected to reduce the 28-day strength. In contrast, when initial curing temperatures were colder than the final curing temperature, the 28-day strengths were expected to increase. The concepts of concrete maturity were used to predict how initial curing durations will affect the concrete strength. The use of multiple concrete mixtures made with different SCMs allowed the research team to evaluate the response of different concretes to differences in initial curing temperatures and durations.

3.3 Procedures

3.3.1 General Procedures

Eight different concrete mixtures were used throughout the laboratory procedure. Each of the eight mixtures were mixed at least twice. Initial curing durations of just greater than 24 hours and just below 48 hours were used for each mixture. The decision to use 24- and 48-hour curing durations was to account for the specifications outlined in AASHTO T 23 (2018) and ALDOT 501 (2022). These specifications require an initial curing duration of 24 to 48 hours. After reviewing the data from the 24- and 48-hour batches, it was decided to also evaluate how much an initial curing duration of 72 hours would affect the 28-day concrete strength. If an initial curing duration of 72 hours is allowed, contractors would be able to transport cylinders from a Friday placement on the following Monday, removing the need to send an employee to demold and transport cylinders to final curing on the weekend. Also, contractors could save time and money by having more flexibility to transport the cylinders to the final curing location. Cylinders made two days apart could potentially be transported together instead of having to make separate trips to the final curing location. The four batches that had the greatest strength difference with 24- and 48-hours of initial curing were chosen, and the procedure was repeated with a 72-hour initial curing duration. All batches were mixed at elevated temperatures targeting fresh concrete temperatures between 90 °F and 100 °F to mimic hot weather concreting. For each batch, cylinders were initially cured in water baths set at constant temperatures of 60 °F, 68 °F, 78 °F, 84 °F, 90 °F, and 100 °F. After the specified initial curing duration, specimens were then demolded and transferred to the final curing fog room which maintained a temperature of 73.5 ± 3.5 °F and provided a relative humidity of 100%. Specimens remained in the final curing fog room until their 28-day strengths were determined using a compression testing machine. Using the average strength of the concrete cylinders cured at 68°F, the relative strength differences for the cylinders cured at the other curing temperatures were calculated. These relative strength differences were then compared to determine the effect of initial curing practices on the 28-day compressive strength of concrete.

3.3.2 Mixture Proportions

A total of eight concrete mixtures, each with proportions commonly used in ALDOT bridge applications, were used to evaluate the effect of initial curing temperature on 28-day compressive strength. The abbreviations used for different cementitious materials in this study can be found in Table 3-1. A summary of the concrete mixture proportions can be found below in Table 3-2. All mixtures were proportioned to have a fixed water-to-cementitious material ratio of 0.44. The total amount of cementitious material per batch was equal to 620 pcy. When supplementary cementitious materials were used, they were substituted by percentage of mass. After calculating the volume of material produced, the increase or decrease in total volume, due to the addition of SCMs, was adjusted to yield one cubic yard (27 cubic feet) by adjusting the amount of fine aggregate used.

This method was used in all batches except for the batch with 30% Class C fly ash. When changing the quantity of fine aggregate, both the fine aggregate and coarse aggregate amount was adjusted. This inconsistency was not noticed until after the batches were completed. Although this difference occurred, the total weight of excess coarse aggregate when compared to other batches was only 85 pcy. Excess coarse aggregate of 85 pcy will have a negligible effect on strength, therefore it was concluded that the batches and results were comparable. Additionally, all batches using only Class C fly ash had the same batching error, and the concretes were only compared to themselves. The results for these batches would still be applicable and useful to this study.

Another error in the batch proportions was a higher than actual specific gravity of Class F fly ash. It was incorrectly assumed that the specific gravity was equivalent to cement specific gravity of 3.15 instead of 2.6. As a result, the batch sheet accounted for a smaller volume of fly ash then what was provided. The percentage of fly ash used remained accurate because it was calculated by mass and not volume. More sand was supplied than was required and as a result there was a larger yield than necessary. Because of the small amount of fly ash used, this error was negligible and only increased the total yield, originally 27 cubic feet, by 0.13 cubic feet when using 20% Class F fly ash and 0.2 cubic feet when using 30% Class F fly ash. The nature of the experiment and the fact that the strength results of the concrete were only compared to concrete from the same mixture, was reassurance that this small error would not affect the overall findings of this study. Because the 28-day strength of each concrete initially cured at 68°F was used to determine the relative strength differences for the respective batch any unknown errors or variability would cancel out therefore allowing the results to remain valid.

Table 3-1: Cementitious Material Abbreviations

Type I	Type I Portland Cement
FFA	Class F Fly Ash
CFA	Class C Fly Ash
SC	Slag Cement
SF	Silica Fume
Type III	Type III Portland Cement

Table 3-2: Concrete Mixture Proportions Evaluated

Concrete Type	Material Composition (lb/yd^3)							
	Water	Cement	FA*	CA*	Class F Fly Ash	Class C Fly Ash	Slag Cement	Silica Fume
100% Type I	273	620	1216	1800	-	-	-	-
30% FFA	273	434	1214	1800	186	-	-	-
30% CFA	273	434	1096	1885	-	186	-	-
50% SC	273	310	1190	1800	-	-	310	-
10% SF	273	558	1190	1800	-	-	-	62
20% CFA & 30% SC	273	310	1203	1800	124	-	186	-
20% CFA & 10% SF	273	434	1189	1800	124	-	-	62
100% Type III	273	620	1216	1800	-	-	-	-

*Aggregate in saturated-surface-dry state (SSD)

3.3.3 Raw Materials Used

3.3.3.1 Portland Cement

To ensure the most accurate results, the same portland cement source was used throughout the entirety of the laboratory batches. Type I/II and Type III cement was supplied by Argos Cement. The only times Type I/II cement was not used was when concrete was batched with 100% Type III cement. All batches consisting of supplementary cementitious materials used the Type I/II cement produced by Argos. Both the Type I/II and Type III cements had a specific gravity of 3.15.

3.3.3.2 SCMs

3.3.3.2.1 Fly Ash

Fly Ash is the most common supplementary cementitious materials found in ALDOT concrete mixture designs. It is a byproduct of energy production from combusting coal. By incorporating fly ash in concrete mixtures, the industry can improve the environmental effects of concrete production (Darwin et al. 2016). The use of fly ash reduces the amount of cement used and helps reduce the amount of fly ash sent to landfills. Fly Ash helps reduce the heat development of concrete without reducing the strength (Schlorholtz 2006). The Class F fly ash, sourced by the SEFA Group had a specific gravity of approximately 2.60. The Class C fly ash, sourced by Plant Miller of Boral Resources had a specific gravity of 2.61.

3.3.3.2.2 Slag Cement

Slag cement is a byproduct of iron production. Like fly ash, the replacement of cement with slag cement has significant environmental benefits. The use of slag cement reduces the amount of cement used and helps reduce the amount of waste produced by the iron industry (Darwin et al. 2016). Significant reductions in heat can be accounted to the use of slag cement in concrete, and it is common to use large quantities of slag cement when performing mass concrete applications (Prusinski 2006). The slag cement used was sourced by the Cape Canaveral Plant of Lehigh Cement and had a specific gravity of 2.86.

3.3.3.2.3 Silica Fume

Silica fume has significant environmental impacts and helps to increase the strength of concrete. However, increased plastic shrinkage and drying shrinkage cracking occurs with large quantities of silica fume (Baghabra Al-Amoudi et al. 2004). Therefore, it is not common to see batch proportions with more than 10% silica fume (Kunal and Siddique 2016). The silica fume used was sourced by Elkam Materials and had a specific gravity of 2.20.

3.3.3.3 Coarse and Fine Aggregate

Coarse aggregate consisted of #57 Crushed Granite. The aggregate was provided by Vulcan Materials from their Loachapoka quarry. The coarse aggregate had a bulk specific gravity of 2.628 and an absorption capacity of 0.51 percent. Fine aggregate consisted of 1773 Sand from Wiregrass's Ariton pit. The fine aggregate had a bulk specific gravity of 2.629 and an absorption capacity of 0.4 percent.

3.3.3.4 Chemical Admixtures

3.3.3.4.1 Water Reducer

Two water-reducing admixtures were used for the laboratory concrete batches. For all batches not including silica fume MasterPozzolith 322 was used. Batches with silica fume used a high-range water-reducing admixture called MasterGlenium 7920.

3.3.3.3.2 Air Entraining Admixture

A target air content of 4% ± 2% was desirable, therefore, an air entraining admixture was added to each batch. MasterAir AE 90 was used as air entraining admixture in all concretes.

3.3.4 Batching

Batching took place in the Structural Concrete Materials laboratory of AU's Advanced Structural Engineering Laboratory. A 9 ft^3 revolving steel drum mixer was used for all batches. Figure 3-1 shows the drum mixer located in the concrete lab.

Figure 3-1: Concrete mixer used

The first step of batching was to weigh out all the material the day before using 5-gallon buckets. It was necessary to provide an excess amount of coarse and fine aggregate to accommodate for moisture corrections that were performed just prior to mixing the concrete

Upon completion of weighing out the materials, they were placed in an environmental chamber set at an elevated temperature. In Figure 3-2, a member of the research team is seen weighing all materials for the next day's batch.

Figure 3-2: Weighing aggregate

The aggregate was then allowed to heat over night to simulate the excessive temperatures of hot weather concreting. Before leaving the materials in the environmental chamber, all buckets were adequately sealed with lids. It was crucial to ensure a proper seal to prevent loss of moisture while in the environmental chamber. Figure 3-3, shown below, includes all materials prepared for a batch. The image was taken just before placing the buckets in the environmental chamber that is shown to the right of the image.

Figure 3-3: Concrete materials before entering environmental chamber

Initially, a temperature setpoint of 95 °F was used for the environmental chamber. Later a temperature of 105 °F was used because the initial batches did not have the desired fresh concrete temperature of between 90 °F and 100 °F. After the first two batches it was determined that the time it took to remove the materials from the environmental chamber and then begin batching, allowed for the materials to cool to below 90 °F. For this reason, the temperature of the environmental chamber was increased for all remaining batches. Not all the fresh concrete temperatures were within the desired temperature range, some were just below 90°F and one exceeded 100 °F. Before batching could begin, moisture corrections were performed in accordance with ASTM C 566 (2019), and the aggregate batch weights were corrected.

In Figure 3-4, the moisture content of fine aggregate is being determined. This was done by first determining the weight of the wet aggregate. After determining the wet weight, the aggregate was heated to remove all moisture and then reweighed. By comparing the wet and dry weight of aggregate, the moisture content was calculated and the correct amount of water to be used during the batching process could be determined.

Figure 3-4: Moisture corrections for fine aggregate

3.3.4.1 Standard Batching Procedure

Batching consisted of two different procedures. For all mixes that did not include silica fume, the standardized batching procedure found in Appendix C was used. After all material was added in accordance with the standardized batching procedure, fresh concrete property tests were performed. These consisted of slump, air content, concrete temperature, and unit weight tests. Figure 3-5 shows the unit weight of concrete test being prepared. A target value from 140 pcf to 150 pcf was used for this study. A target air content of 2-6% was used and using an air meter shown in Figure 3-6 the air content was evaluated.

Figure 3-5: Preparation for Unit Weight Test

Figure 3-6: Air Content Test

The slump of concretes batched was used to determine the workability of concrete. A target range of 2.5 inches to 5.0 inches was used. Figure 3-7 shows a slump test being performed by the research team.

Figure 3-7: Slump Test

Table 4-1 includes the fresh concrete properties of each batch. When the desired slump was not achieved, additional water-reducing admixture was added in accordance with the standardized batching procedure. Once the workability test results were satisfactory, the batching process was complete, and the molding of cylinders could begin.

3.3.4.2 Batching with Silica Fume

For batches with silica fume, a different mixing procedure was required. Silica fume is extremely fine and in densified form needs to be violently broken apart to react with water. To accommodate this, all the silica fume and coarse aggregate were added first and allowed to mix. This violent procedure facilitates the breakup of silica fume particles and prepares the material for the addition of water. The specific procedure used to batch all concretes with silica fume can be found in Appendix C. A high-range water-reducing admixture, MasterGlenium 7920, was used to increase workability while maintaining the water-to-cementitious ratio of the concrete for all batches that used silica fume. Normal water-reducing admixture, MasterPozzolith 322, which was used for all other batches could not be used because it did not provide sufficient water reduction needed when using fine silica fume. If the desired workability was not achieved, more water-reducing admixture was added in accordance with the standardized batching procedure. Like the normal batching process, once the desired fresh concrete properties were achieved, molding of cylinders proceeded.

3.3.5 Molding of Cylinders

Concrete cylinders were made using 6” x 12” plastic cylinder molds. For each batch a total of 24 cylinders were made. Four cylinders were made for each initial curing temperature. Three cylinders were used to determine the 28-day compressive strength, while the fourth was used for measuring the concrete temperature development. All cylinders were made in accordance with AASHTO T 23 (2018). After capping, cylinders were left in their initial curing temperature-controlled water baths. To measure the temperature development of one concrete cylinder in each initial curing bath, a straw was inserted through a hole in the cap allowing a temperature probe to be inserted. The straws allowed for removal and reuse of the probes once the concrete hardened and temperature testing on a cylinder was complete.

3.3.5.1 Initial Curing and Final Curing

3.3.5.1.1 Initial Curing

Initial curing of concrete cylinders consisted of water curing at select temperatures chosen to represent the effect of an increased initial curing temperature. Six curing tanks were constructed to create initial curing temperatures of 60 °F, 68 °F, 78 °F, 84 °F, 90 °F, and 100 °F. Initial curing temperatures were chosen to assess the AASHTO T 23 (2018) and ALDOT 501 (2022) limits and determine if the temperature and duration ranges can be altered. AASHTO T

23 (2018) requires all cylinders of normal-strength concrete to be cured within a temperature range from 60 °F to 80 °F. Concrete with a design strength greater than 6000 psi has even stricter temperature ranges of 68 °F and 78 °F (AASHTO T 23 2018). Therefore, 68 °F was chosen as the control because it was close to the middle of the initial range of 60 °F to 80 °F and also represented the minimum for the high-strength concrete temperature range. The 60 °F curing tank was used to ensure there was representation of the effects from colder curing environments. However, the focus was on hot weather concreting; therefore, the other four temperatures were chosen to be above the control temperature of 68 °F. The first temperature, 78 °F, is close to the upper limit of 80°F of AASHTO T 23 (2018) and was the maximum temperature of the high strength concrete limit. Finally, the last three temperatures of 84 °F, 90 °F, and 100 °F were chosen by using three similar temperature differences until reaching 100 °F. The curing tanks were constructed by using igloo coolers that were retrofitted with an internal cooling/heating system. The laboratory setup of all six curing tanks is shown in Figure 3-8.

Figure 3-8: Laboratory initial curing setup

The cooling systems for the curing tanks were constructed using copper piping that was connected to a water circulator. Each circulator pumped hot or cold water through the pipes and ensured the water temperature in the cooler maintained the desired initial curing temperature. Temperature probes were inserted from the circulator into the water to determine whether to heat or cool the circulated water. The temperature that was maintained was the *water in the curing tank*, and not the water in the circulator and piping. This is important to note because AASHTO T 23 (2018) and ALDOT 501 (2022) state that the curing environment and not the concrete must maintain the desired temperature. The water temperature in the circulator and copper piping may have been hotter or colder than the actual curing tank water temperature. The circulated water had to heat/cool the entire curing tank; therefore, it would have to output water at different temperatures. Figures 3-9 and 3-10 show pictures of a typical curing tank and circulator setup.

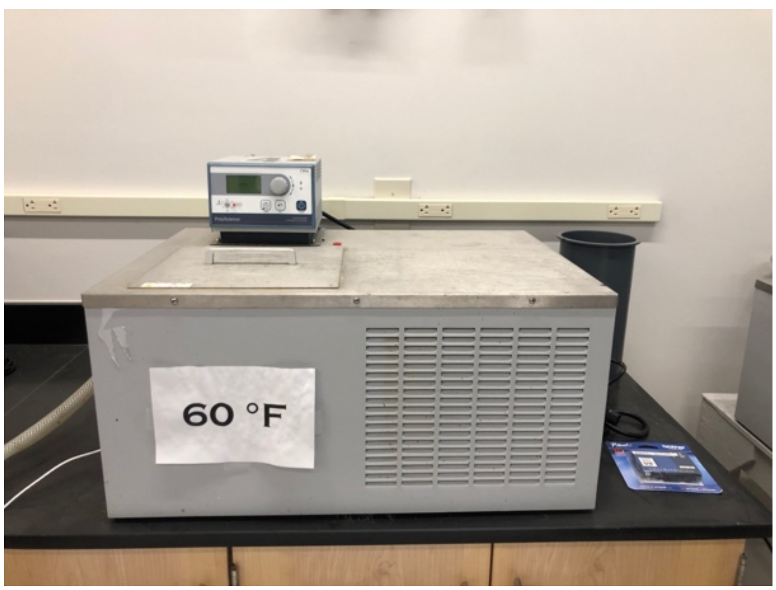

Figure 3-9: Water Circulator

Figure 3-10: Initial Curing Tank

All initial curing tanks included a small submersible circulation pump that contributed to a consistent distribution of water and allowed for a consistent water temperature throughout the entire initial curing tank. Without a submersible circulation pump, the top and bottom of the cooler water temperatures would vary (Fleming 2023). A submersible circulation pump can be seen in Figure 3-11.

Figure 3-11: Submersible Circulation Pump

To verify and record the water temperature while initial curing took place, Hobo Pendant MX temperature probes were placed in each curing tank. These probes allowed for a temperature reading to be logged throughout the entire curing period at an interval of 15 minutes. For each batch, there were four cylinders per cooler with one being used for a temperature reading. The cylinders with temperature readings were not used for strength tests. Every fifteen minutes, a reading of the concrete temperature was recorded using a HOBO Thermocouple. A typical curing tank with cylinders in their initial curing location can be seen in Figure 3-12.

Figure 3-12: Cylinders in Initial Curing Tank

In Figure 3-12, it is clear how temperature probes were inserted into the center of the cylinder and capped to retain moisture like all other cylinders. Using the readings from both the concrete and water temperature, plots of temperature versus time were developed to ensure the desired curing environment was maintained. In addition, temperature development of the concrete cylinders in the varying initial curing temperatures were captured. As an example, Figure 3-13 shows the temperature versus time plot for one of the concrete batches. The 48-hour initial curing duration of 10% Silica Fume Concrete is shown in Figure 3-13.

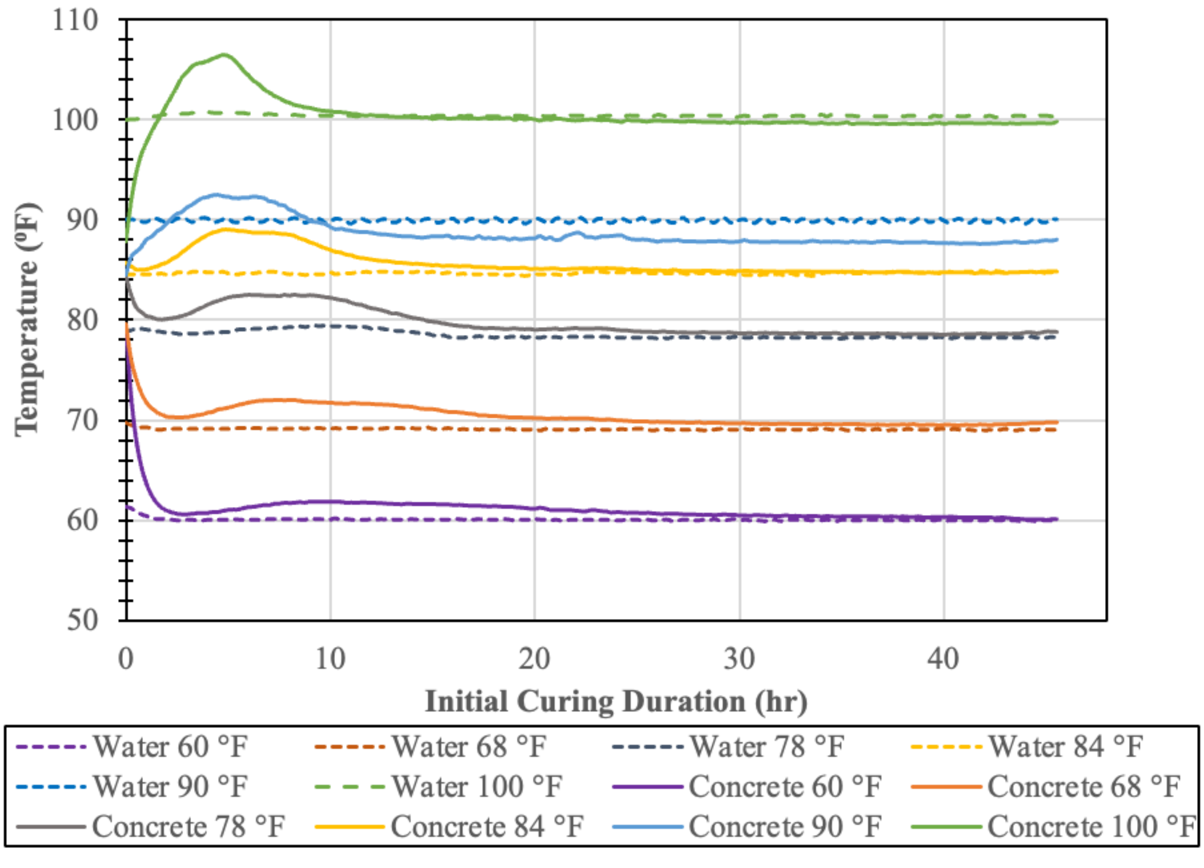

Figure 3-13: Example plot of initial curing temperatures

All temperature plots can be found in Appendix A. Depending on the desired initial curing duration, cylinders were left in their initial curing environment for 24, 48, or 72 hours. After the desired curing duration was completed, the cylinders were removed from their molds and moved to their final curing location.

3.3.5.1.2 Final Curing

All final curing practices followed AASHTO T 23 (2018) and ALDOT 501 (2022). First, the cylinders were removed from their initial curing environment and then demolded and labelled. Once demolded, the cylinders were moved to the final curing room as it was critical to transfer the cylinders in under 30 minutes, per specification. This room was constructed by Darwin Chambers and maintained a temperature of 73.5 ± 3.5 °F and provided a 100% relative humidity. Figure 3-14 shows the final curing room with cylinders from three concrete batches in their final curing location.

Figure 3-14: Cylinders in Final Curing Room

As shown in Figure 3-14, cylinders undergoing final curing were placed on wire shelves. Using wire shelves helped to prevent any water ponding. Cylinders remained in the final curing room until reaching an age of 28 days and were then tested to determine their 28-day compressive strength.

3.3.6 Strength Testing

Concrete cylinder compressive tests followed the standard set forth by ASTM C 1231 (2015) and AASHTO T 22 (2022). All cylinders were tested using the same machine, a Forney Variable Drive Technology Automatic machine with a capacity of 600 kips. Additionally, within each concrete batch, one operator was used to test the cylinders. This helped to remove variability that could arise between operators. ASTM C 1231 (2015) was used for the unbonded capping of cylinders. Using ASTM C 1231 (2015) it was determined to use neoprene pads along with baby powder to ensure an equal stress distribution. A durometer value of 70 was used for the neoprene pads, and they were replaced after 50 tests. The cylinders were loaded at a rate of 35 psi/s until failure, and the maximum stress obtained was recorded. Three cylinders were tested at each initial curing temperature to determine the average 28-day strength for this set of cylinders. While testing, only three cylinders were removed from the curing room at a time. When cylinders are allowed to dry, they can have a slightly higher strength than cylinders in moist state. By limiting the number of cylinders removed at a time, the exposed cylinders did not have enough time to lose moisture. This practice ensured that moisture loss which, could affect the measure 28-day compressive strength, did not occur.

3.3.6.1 Strength Differences

Strength differences were determined relative to the average compressive strengths measured of the cylinders in the 68 °F initial curing tank. Using a reference temperature as 68 °F, the relative strength difference was determined as a percent for each of the other curing environments. When comparing the relative strength differences, each batch of concrete was only compared to itself because the 28-day compressive strength of that batch of concrete when initially cured at 68 °F was used as the reference strength for the entire batch. The first step in determining the relative strength differences was to determine the 28-day compressive strength of concrete for each initial curing temperature. As an example, Figure 3-15 shows the 28-day compressive strength of a single batch of 100% Type I cement with respect to each initial curing temperature.

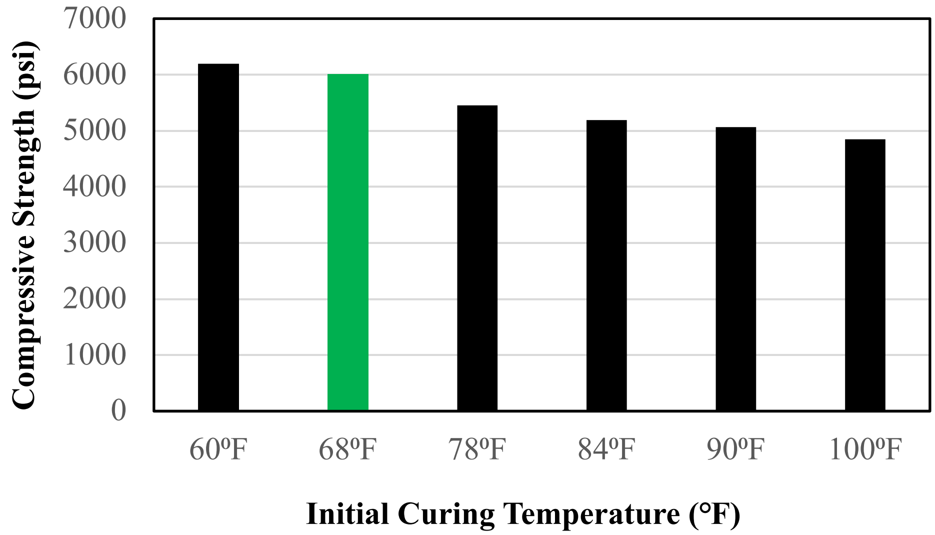

Figure 3-15: Average 28-Day compressive strength of concrete

The 68 °F curing location in Figure 3-15 is plotted as a green bar because it is the reference that all strength differences would be taken from for the respective batch. After the 28-day compressive strengths were determined, the relative strength differences were determined for each initial curing temperature with respect to the concrete initially cured at 68 °F. Figure 3-16 shows the relative strength differences from the 28-day strengths in Figure 3-15. No bar is shown for the 68 °F initial curing temperature because it is the reference temperature.

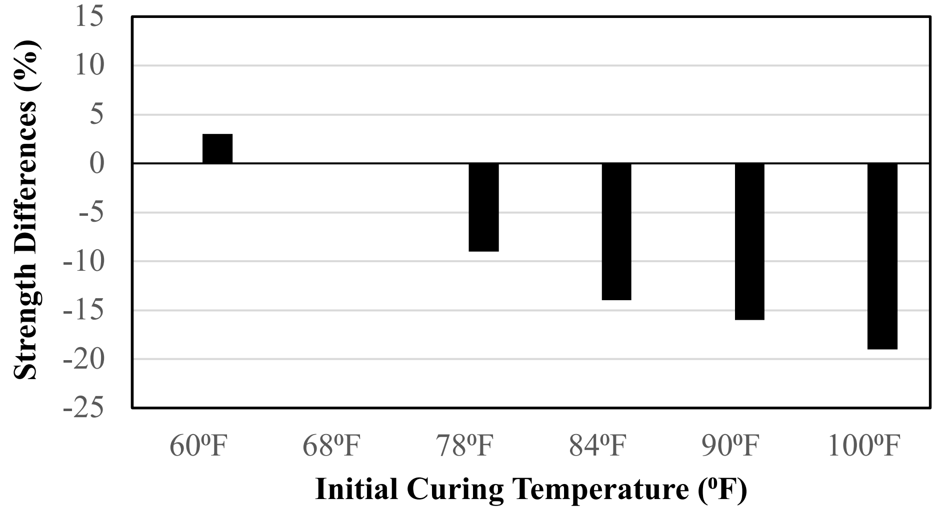

Figure 3-16: Relative strength differences example

The equation used to determine the relative strength difference was:

$$\text{Strength Difference} = \frac{f_c\ at\ T - f_c\ at\ 68°F}{f_c\ at\ 68°F} \times 100 \qquad \textbf{(Equation 3-1)}$$

Where:

Strength Difference = relative strength differences (%);

f_c at T = 28-day compressive strength at specific initial curing temperature (psi); and

f_c at 68 °F = 28-day compressive strength at initial curing temperature of 68 °F (psi).

Average strengths of each initial curing duration were used for all strength difference calculations. Although some variability is expected, using an average strength ensured the results were the best representation of the actual concrete strength. Within each initial curing temperature, the cylinder strengths were evaluated for any outliers. An outlier was determined to be any cylinder that had a relative difference greater than ±7.8% when compared to the other two cylinders it was cured with. A value of ±7.8% was chosen to determine outliers in accordance with the Acceptable Range of Individual Cylinder Strengths of AASHTO T 22 (2020). Over the course of this study, only six outliers were detected. These few outliers were not used in calculations when determining the relative strength differences between initial curing environments.

In order to adequately evaluate the effect of initial curing temperature and duration on compressive strength it was necessary to determine a limit on what is an acceptable strength difference between samples cured at various initial curing temperatures. The limit had to be determined before evaluating any of the effects of initial curing temperature and duration on the strength results. AASHTO T 22 (2017) has a single operator coefficient of variation for three cylinders tested in laboratory conditions as ±7.8%. This coefficient of variation is for "companion cylinders prepared from the same sample of concrete and tested by one laboratory at the same age" (AASHTO T 22 2017). Although tested in a laboratory, the concrete samples are compared to cylinders that were cured at different temperatures. More variation is to be expected when initially cured at varying temperatures, and therefore an acceptable limit of ±10% was chosen to evaluate the results of cylinders cured at different initial curing temperatures.

Chapter 4: Presentation and Discussion of Results

This chapter includes the results and discussion of the experimental procedure covered in Chapter 3. The fresh concrete properties and the relative strength differences of each batch are presented and discussed.

4.1 Fresh Concrete Property Results

The fresh concrete properties of each concrete batch are summarized in Table 4-1. The adherence to the target ranges mentioned in Section 3.3.4.1 were evaluated. All fresh concrete property tests were within the target ranges before molding of cylinders took place.

4.1.1 Slump

The target range for the slump of fresh concrete, as mentioned in Section 3.3.4.1, ranged from 2.5 inches to 5.0 inches. All batches, including the four verification batches, had measured slump values within the target range.

4.1.2 Air Content

The target value for the air content of fresh concrete, as mentioned in Section 3.3.4.1, ranged from 2% to 6%. All batches, including the four verification batches, had measured air content values within the target range.

4.1.3 Unit Weight

The target value for the unit weight of fresh concrete, as mentioned in Section 3.3.4.1, was from 140 pcf to 150 pcf. All batches, including the four verification batches, had measured unit weights within the target range.

4.1.4 Temperature

The target temperature range for fresh concrete, as mentioned in Section 3.3.4, ranged from 90 °F to 100 °F. Three concrete batches had fresh concrete temperatures that fell outside of the chosen target range. The three batches that had fresh concrete temperatures outside of the chosen target range were the 100% Type I PCC mixtures when initially cured for a duration of both 24 and 48 hours, and the batch of 100% Type III PCC when initially cured for 72 hours. Although, the three batches had fresh concrete temperatures outside of the chosen target range, the nature of this study allowed the use of the concrete to determine relative strength differences resulting from changes in initial curing temperature and duration.

Table 4-1: Fresh Concrete Properties

Fresh Concrete Properties				
Mixture Type	Slump (in.)	Unit Weight (lb/ft^3)	Air Content (%)	Fresh Concrete Temperature (°F)
24-Hour Initial Curing				
100% Type I	2.5	149.0	3.5	77
30% Class F Fly Ash	4.0	145.2	2.9	90
30% Class C Fly Ash	4.0	147.0	3.0	90
50% Slag Cement	2.5	143.2	3.9	95
10% Silica Fume	2.5	142.2	5.0	98
20% Class F Fly Ash 30% Slag Cement	3.0	144.0	4.0	90
20% Class F Fly Ash 10% Silica Fume	3.5	142.5	5.0	97
100% Type III PCC	2.5	144.0	4.5	95
48-Hour Initial Curing				
100% Type I	3.5	145.3	4.5	85
30% Class F Fly Ash	4.5	146.0	2.9	90
30% Class C Fly Ash	2.5	146.0	2.5	91
50% Slag Cement	3.5	143.0	5.0	92
10% Silica Fume	3.5	142.5	5.5	94
20% Class F Fly Ash 30% Slag Cement	4.0	143.4	4.0	95
20% Class F Fly Ash 10% Silica Fume	4.5	141.0	6.0	90
100% Type III	2.5	143.0	5.0	90
72-Hour Initial Curing				
100% Type I	3.5	142.2	5.5	96
50% Slag Cement	3.5	142.8	5.0	95
10% Silica Fume	3.0	143.0	5.0	96
20% Class F Fly Ash 10% Silica Fume	4.0	140.0	6.0	95
Verification Batches				
30% Class C Fly Ash	4.0	146.7	3.0	97
100% Type III	3.0	143.4	5.0	102
50% Slag Cement	4.0	142.0	4.0	95
100% Type I	3.5	143.0	4.0	94

4.2 Individual Concrete Relative Strength Differences

4.2.1 100% Type I Portland Cement Concrete

The 100% Type I PCC mixture was one of the four concrete mixtures that included a batch with an initial curing duration of 72 hours. Although most concrete batches in the industry include at least one supplementary cementitious material, some batches still consist of only portland cement. It was expected that without any supplementary cementitious material the temperature effects would be severe. The results from all three initial curing durations are compared in Table 4-2. Figure 4-1 plots the values found in Table 4-2 against the ±10% relative strength difference limit.

Table 4-2: 100% Type I PCC relative strength differences

Initial Curing Temperature	Initial Curing Duration		
	24 hrs	48 hrs	72 hrs
60 °F	3	3	4
78 °F	-9	-6	-4
84 °F	-14	-12	-7
90 °F	-16	-11	-7
100 °F	-19	-16	-11

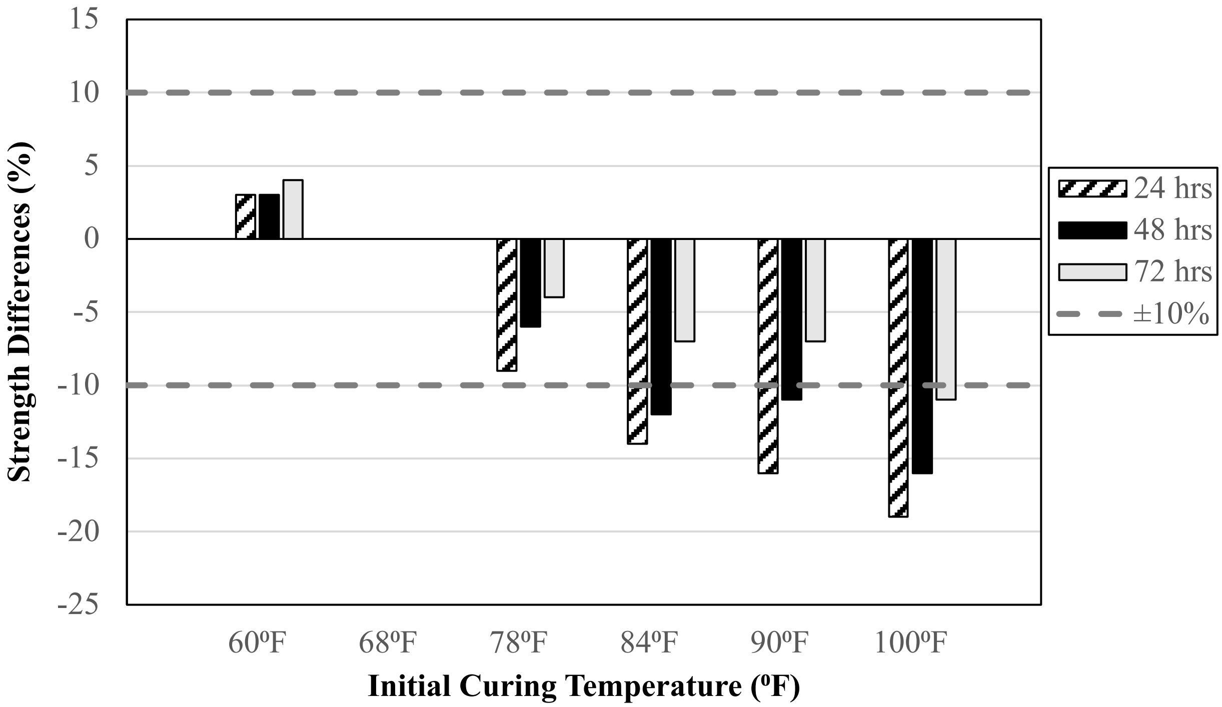

Figure 4-1: Relative 28-day strength differences for 100% Type I PCC

All three initial curing durations show relative strength differences that become more extreme with increasing initial curing temperature. Within an initial curing temperature range from 60 °F to 80 °F, the relative strength differences were within the chosen acceptable limits for all three initial curing durations. A clear trend of more relative strength differences with increasing initial curing temperature can be identified in Figure 4-1. At initial curing temperatures of 60 °F and 78 °F the relative strength differences are within the chosen acceptable relative strength difference limits for all three initial curing durations. As the initial curing temperature is increased to 84 ºF and beyond, the relative strength differences begin to fall outside of the acceptable relative strength difference limits. At an initial curing temperature of 100 ºF, all three initial curing durations result in relative strength differences greater than the acceptable limits.

4.2.2 30% Class F Fly Ash Concrete

Table 4-3 includes the values of strength differences for the 30% Class F fly ash batches and Figure 4-2 plots the values shown in Table 4-3 against the ±10% relative strength difference limit.

Table 4-3: 30% Class F Fly Ash Concrete relative strength differences

Initial Curing Temperature	Initial Curing Duration	
	24 hrs	48 hrs
60 °F	1	4
78 °F	-6	-4
84 °F	-7	-8
90 °F	-7	-7
100 °F	-12	-8

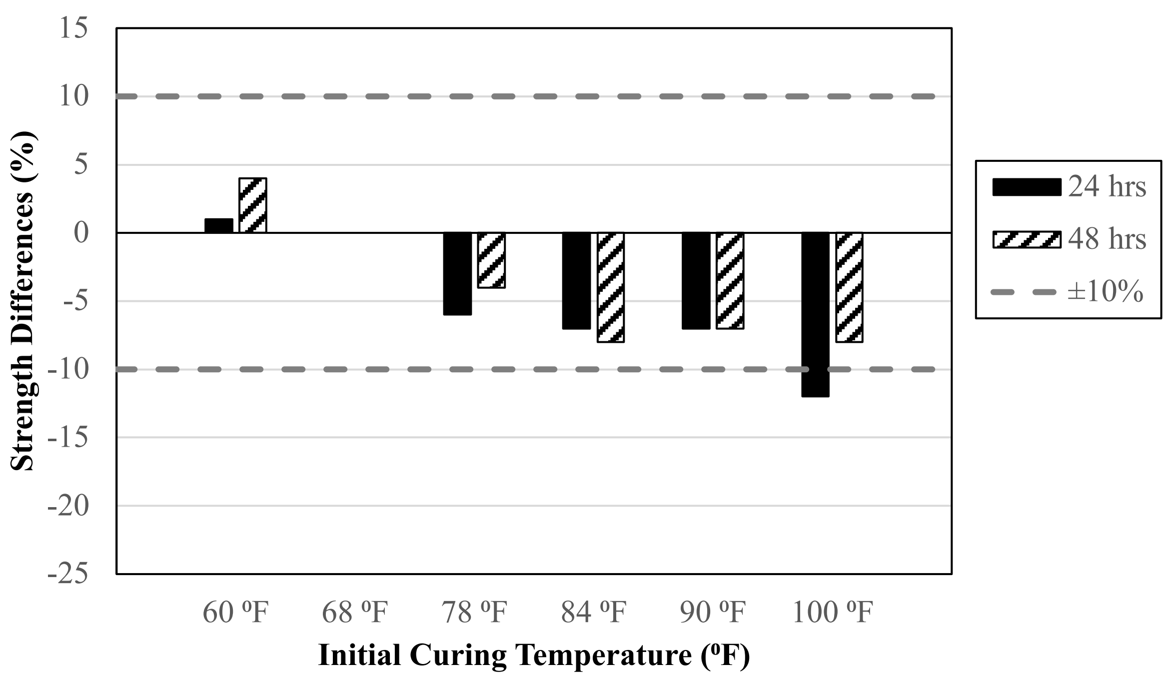

Figure 4-2: Relative 28-day strength differences for 30% Class F Fly Ash Concrete

For the 30% Class F fly ash mixture, strength differences from the 24- and 48-hour batches were not as extreme as some other mixtures and therefore a 72-hour batch was not performed. For both initial curing durations, the most extreme relative strength differences occurred when cylinders were cured at 100 °F. At initial curing temperatures of 78 °F, 84 °F, and 90 °F in the 24-hour initial curing duration batch, the strength differences were all similar. These relative strength differences were -6%, -7%, and -7%, respectively. Within an initial curing temperature range from 60 °F to 90 °F all the relative strength differences were within the acceptable limits of ±10%. Upon inspection of Figure 4-2, the relative strength differences are similar for both initial curing durations. Only the cylinders initially cured at 100 °F for 24 hours visually stands out at -12% with the others all being around -7%. When cured at a lower temperature of 60 °F there was a relative strength gain of 4% for an initial curing duration of 48 hours compared to only 1% for 24 hours. Only the relative strength difference for the cylinders cured at 100 °F for a duration of 24 hours was outside the acceptable limit of ±10%.

4.2.3 30% Class C Fly Ash Concrete

Table 4-4 includes a summary of the relative strength differences measured for the 30% Class C fly ash mixture. Figure 4-3 consists of a plot of the values from Table 4-4 against the ±10% relative strength difference limit.

Table 4-4: 30% Class C Fly Ash Concrete relative strength differences

Initial Curing Temperature	Initial Curing Duration	
	24 hrs	48 hrs
60 °F	-1	4
78 °F	-3	-5
84 °F	-6	-4
90 °F	-10	-8
100 °F	-12	-14

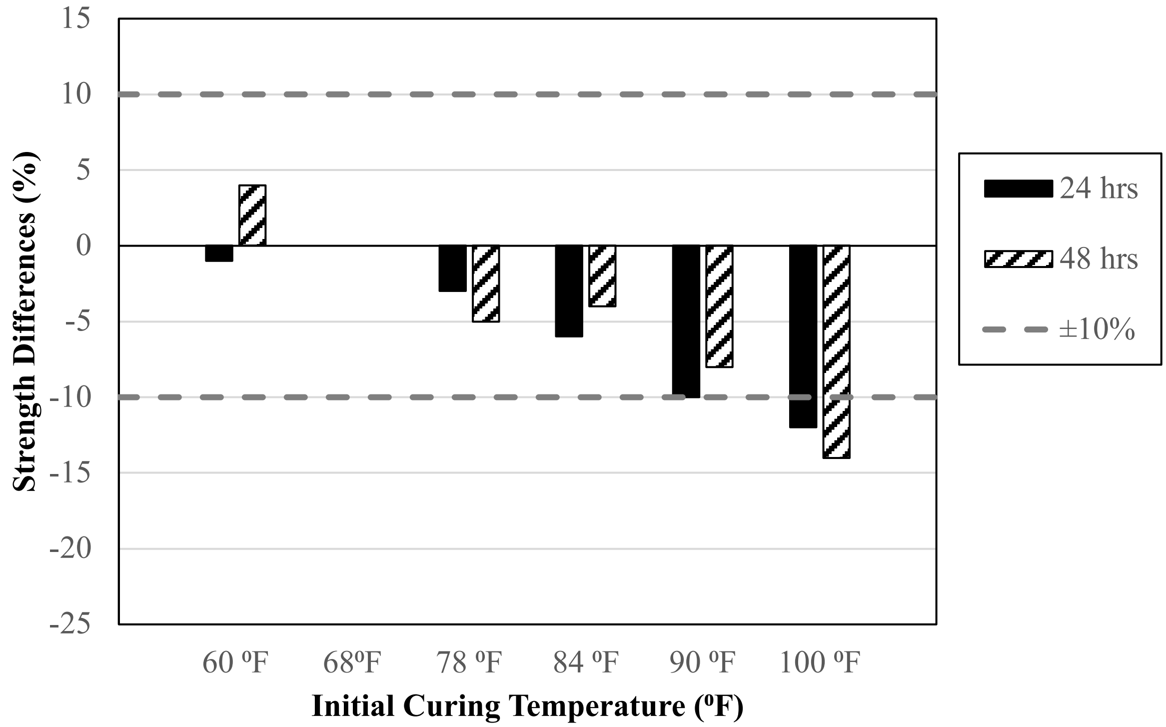

Figure 4-3: Relative 28-day strength differences for 30% Class C Fly Ash Concrete

The relative strength differences from the 24- and 48-hour batches were not as extreme as some other mixtures, and therefore a 72-hour batch was not performed for the mixture with 30% Class C fly ash. A maximum difference of -14% occurred when cured at an initial curing temperature of 100 °F with an initial curing duration of 48 hours. For both initial curing durations, when cured at temperatures of 90 °F and below, the relative strength differences were within the acceptable test limit of ±10%. Although larger than the relative strength differences from the Class F fly ash, the Class C fly ash had mostly low relative strength differences when compared to some of the other mixtures. The relative strength differences did not exceed the acceptable limit until initially cured at a temperature of 100 °F for both initial curing durations. The differences shown in Figure 4-3 help to demonstrate the effect of initial curing temperature on the relative strength differences in the Class C fly ash mixtures. Also, the results do not show a distinct relationship between initial curing duration and 28-day compressive strength.

4.2.4 50% Slag Cement Concrete

Table 4-5 includes the values of strength differences for the three 50% Slag cement batches. Figure 4-4 consists of a plot of the values from Table 4-5 against the ±10% relative strength difference limit.

Table 4-5: 50% Slag Cement Concrete relative strength differences

Initial Curing Temperature	Initial Curing Duration		
	24 hrs	48 hrs	72 hrs
60 °F	7	3	3
78 °F	-5	-1	-4
84 °F	-5	-1	-5
90 °F	-9	-5	-8
100 °F	-15	-8	-9

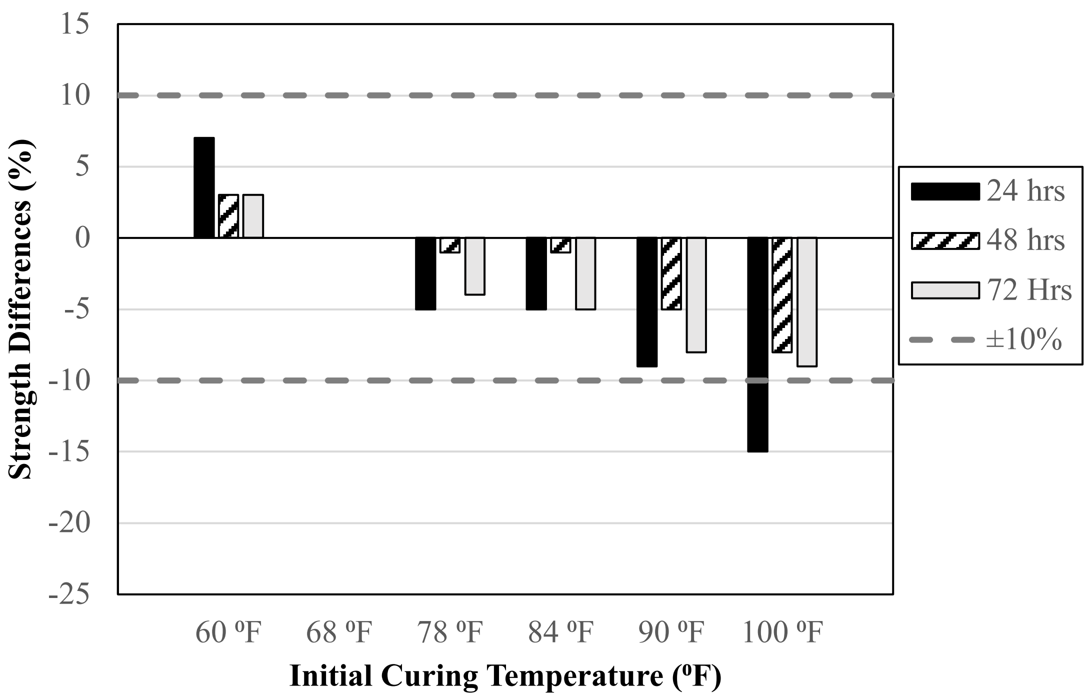

Figure 4-4: Relative 28-day strength differences for 50% Slag Cement Concrete

The maximum relative strength difference of -15% occurred when initially cured for 24 hours at a temperature of 100 °F. The relative strength difference of -15% was the only value to fall outside the acceptable limit of ±10%. The relative strength difference for cylinders initially cured at 100 °F for 24 hours is the largest relative strength difference measured for this mixture.

4.2.5 10% Silica Fume Concrete

Table 4-6 includes the values of strength differences for each 10% Silica Fume batch. Figure 4-5 shows the relative strength differences values from Table 4-6 against the ±10% relative strength difference limit.

Table 4-6: 10% Silica Fume Concrete relative strength differences

Initial Curing Temperature	Initial Curing Duration		
	24 hrs	48 hrs	72 hrs
60 °F	8	10	3
78 °F	-9	-6	-5
84 °F	-15	-11	-11
90 °F	-17	-14	-13
100 °F	-20	-20	-18

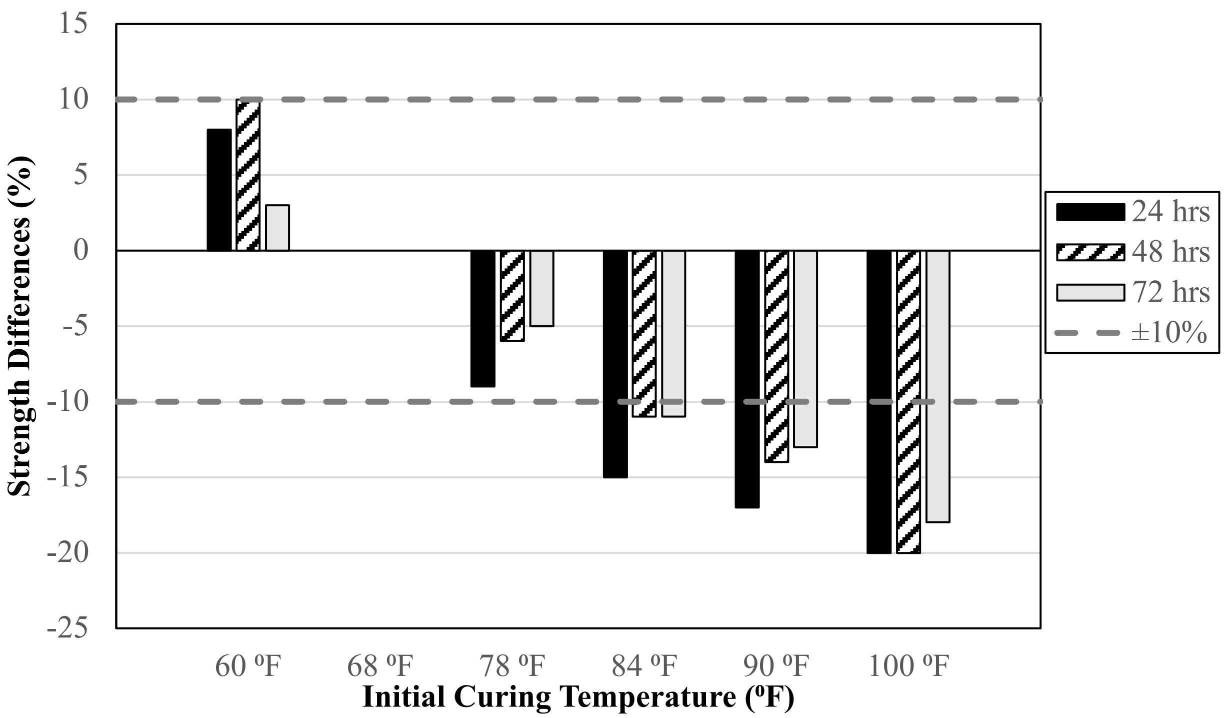

Figure 4-5: Relative 28-day strength differences for 10% Silica Fume Concrete

For the 24- and 48-hour initial curing durations, the 100 °F cured cylinders had a relative strength difference of -20%. Like other batches, the colder curing environment of 60 °F helped improve the strength development, but the differences were larger than most batches. Because of the large strength differences in both the positive and negative directions for the two initial curing durations, the use of a 72-hour initial curing duration was also evaluated for the 10% silica fume mixture. The acceptable strength difference of ±10 was exceeded for all three initial curing durations when initially cured at 84 °F, 90 °F and 100 °F, while the cylinders initially cured at 60 °F and 78 °F were all within the acceptable relative strength difference limit. The 10% Silica Fume batches had the largest relative strength differences when initially cured at 100 ºF. The large difference shows how silica fume can be easily affected by initial curing temperature differences. Both cold and hot curing conditions will affect the compressive strength of concrete batched with silica fume, and special care must be taken to control initial curing temperatures of this concrete type.

4.2.6 20% Class F Fly Ash with 30% Slag Cement Concrete

Table 4-7 includes the values of relative strength differences for the 20% Class F Fly Ash with 30% Slag Cement concrete batches. Figure 4-6 illustrates the relative strength difference values from Table 4-7 against the ±10% relative strength difference limit.

Table 4-7: 20% Class F Fly Ash with 30% Slag Cement Concrete relative strength differences

Initial Curing Temperature	Initial Curing Duration	
	24 hrs	48 hrs
60 °F	8	9
78 °F	-2	1
84 °F	-4	-5
90 °F	-3	-3
100 °F	-9	-9

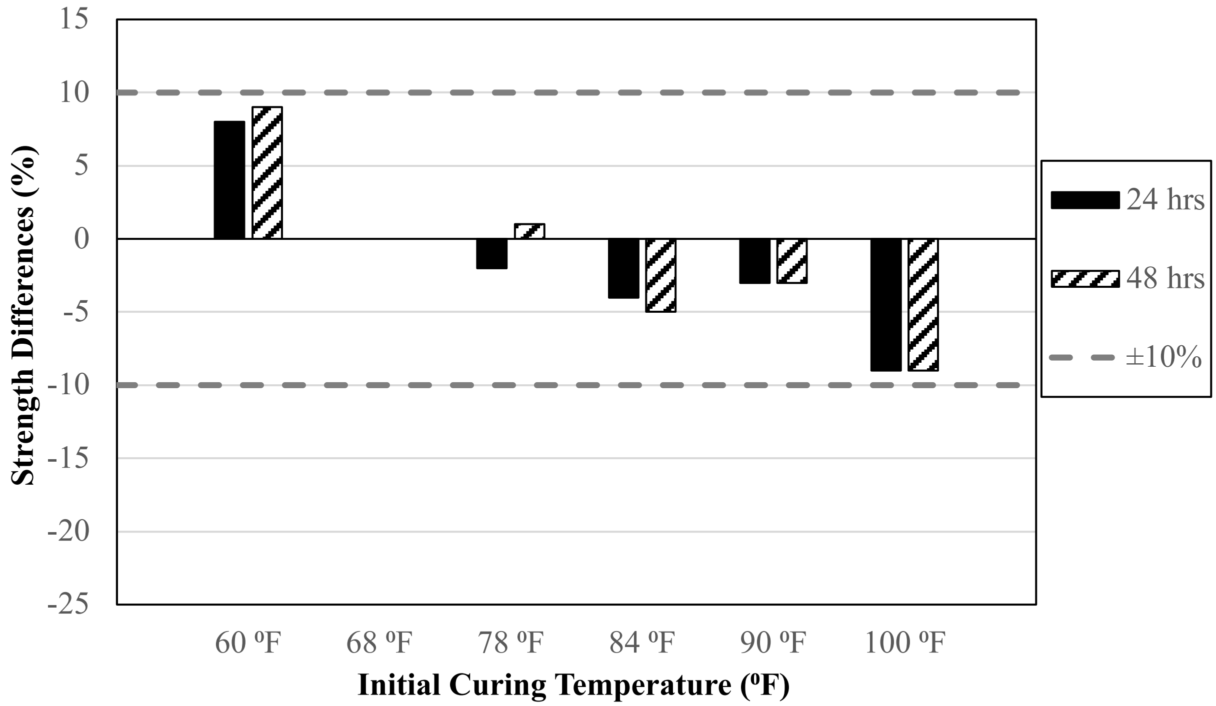

Figure 4-6: Relative 28-day strength differences for 20% Class F Fly Ash with 30% Slag Cement Concrete

The ternary blend of Type I cement, Class F fly ash, and Slag cement had some of the least extreme relative strength differences. It is interesting that when Class F fly ash and slag cement is combined, the relative strength differences are less significant than their individual batches. All the relative strength differences for the 20% Class F fly ash with 30% Slag cement batches were within the acceptable limit of ±10%. Larger differences, shown in Figure 4-6, were recorded for the extremes of 60 °F and 100 °F, but for initial curing temperatures of 78 °F, 84 °F, and 90 °F the differences were within the acceptable limits. Within initial curing temperatures of 78 °F, 84 °F, and 90 °F there is a maximum relative strength difference of only -5%. The small relative strength differences show how effective this blend is at mitigating initial curing temperature effects, and as a result, no 72-hour initial curing duration batch was performed.

4.2.7 20% Class F Fly Ash with 10% Silica Fume Concrete

Table 4-8 includes the values of relative strength differences for the 20% Class F Fly Ash with 10% Silica Fume concrete batches and Figure 4-7 plots the values from Table 4-8 against the ±10% relative strength difference limit.

Table 4-8: 20% Class F Fly Ash with 10% Silica Fume Concrete relative strength differences

Initial Curing Temperature	Initial Curing Duration		
	24 hrs	48 hrs	72 hrs
60 °F	9	9	5
78 °F	-3	-4	-9
84 °F	-6	-11	-13
90 °F	-11	-16	-16
100 °F	-16	-23	-22

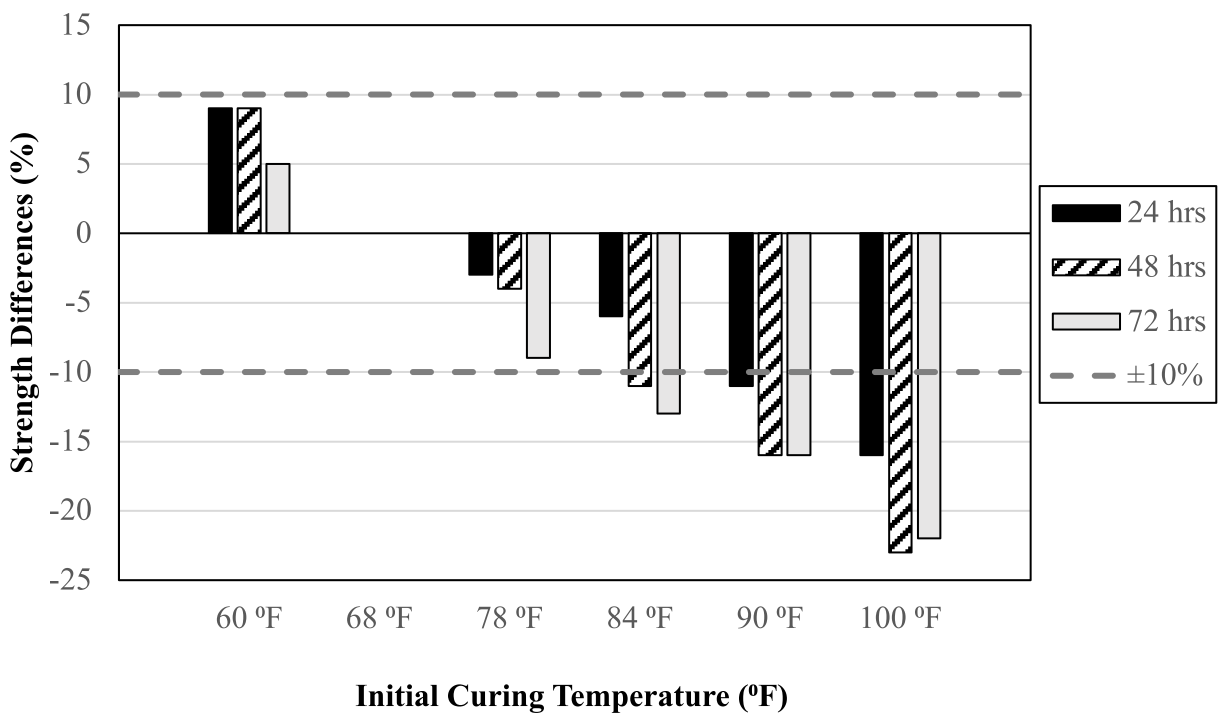

Figure 4-7: Relative 28-day strength differences for 20% Class F Fly Ash with 10% Silica Fume Concrete

Unlike the fly ash and slag cement blend, this ternary blend had significant relative strength differences. Large differences were expected due to the addition of silica fume in the mixture. The relative strength differences of the 24 and 48-hour batches were large enough to warrant the addition of a 72-hour initial curing duration batch. For initial curing temperatures of 60 °F and 78 °F, the relative strength differences are within the acceptable limits for all three initial curing durations. At an initial curing temperature of 84 °F, two of the initial curing durations had relative strength differences that exceeded the acceptable limits. The concrete initially cured at 84 °F for an initial curing duration of 24 hours did not exceed the acceptable limit. As initial curing temperature increased, the relative strength differences grew until a maximum difference occurred at 100 °F for each initial curing duration.

4.2.8 100% Type III Portland Cement Concrete

Table 4-9 includes the values of relative strength differences for the 100% Type III portland cement concrete batches. Figure 4-8 plots the values of Table 4-9 against the ±10% relative strength difference limit.

Table 4-9: 100% Type III PCC relative strength differences

Initial Curing Temperature	Initial Curing Duration	
	24 hrs	48 hrs
60 °F	5	7
78 °F	-2	1
84 °F	-4	-2
90 °F	-3	-5
100 °F	-11	-7

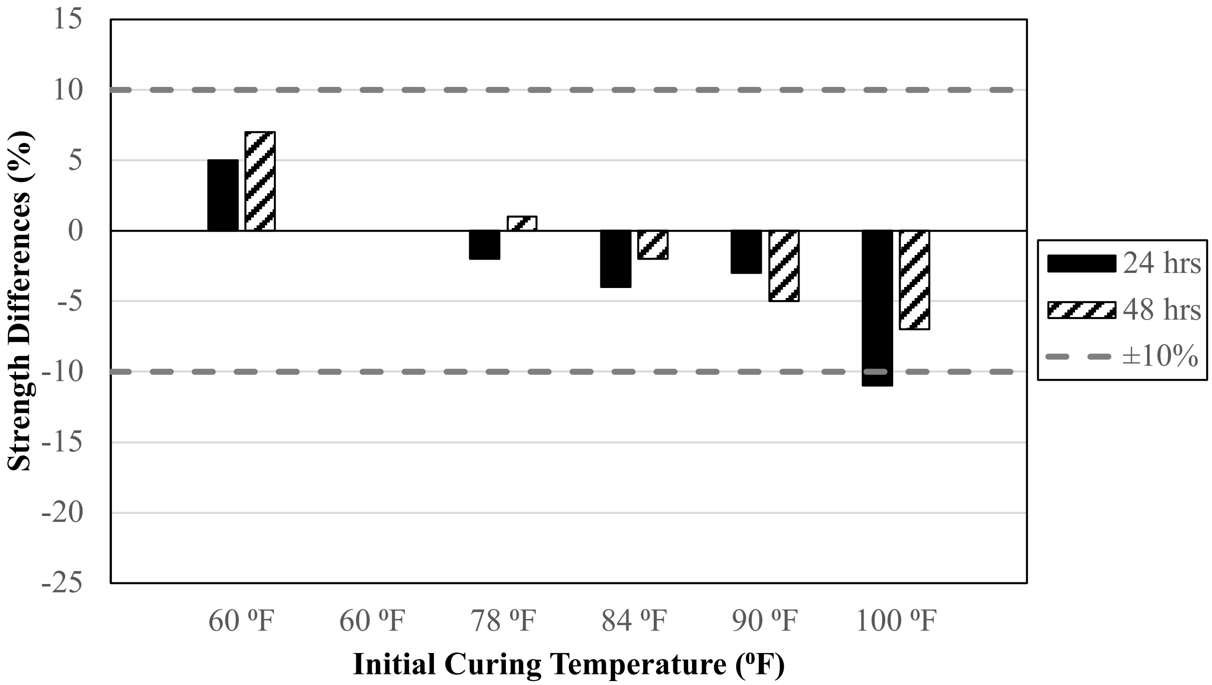

Figure 4-8: Relative 28-day strength differences for 100% Type III PCC

The Type III cement mixture had small relative strength differences for the first two initial curing durations, and therefore no 72-hour batch was evaluated. Only the relative strength differences for the 24-hour batch cured at 100 °F fell outside of the acceptable limit. Minimal relative strength differences are shown in Table 4-9. A maximum relative strength difference of -11% when initially cured for 24-hours occurred while all other differences were within the acceptable limits of ±10%. From the results of the Type III concrete batches, it was concluded that the mixture was less susceptible than some of the other concretes tested to strength differences resulting from temperature effects during initial curing.

4.2.9 Acceptability of 28-Day Concrete Compressive Strength Results

The concrete cylinders in each of the six different initial curing environments were identical concrete sampled from each respective batch. Therefore, each batch developed a unique set of relative strength differences. Only the relative strength differences within each individual batch were compared to other batches. By comparing each concrete specimen's compressive strength to identical concrete, it was not necessary to compare the 28-day compressive strengths between different batches of concrete. However, to ensure that all produced concrete was representative of typical concrete used in the industry, the average 28-day compressive strengths of the cylinders initially cured at 68 °F were analyzed. Figure 4-9 illustrates the 28-day compressive strengths for cylinders initially cured at 68 °F. Four mixtures show three data sets as they were the ones chosen for the 72-hour initial curing durations.

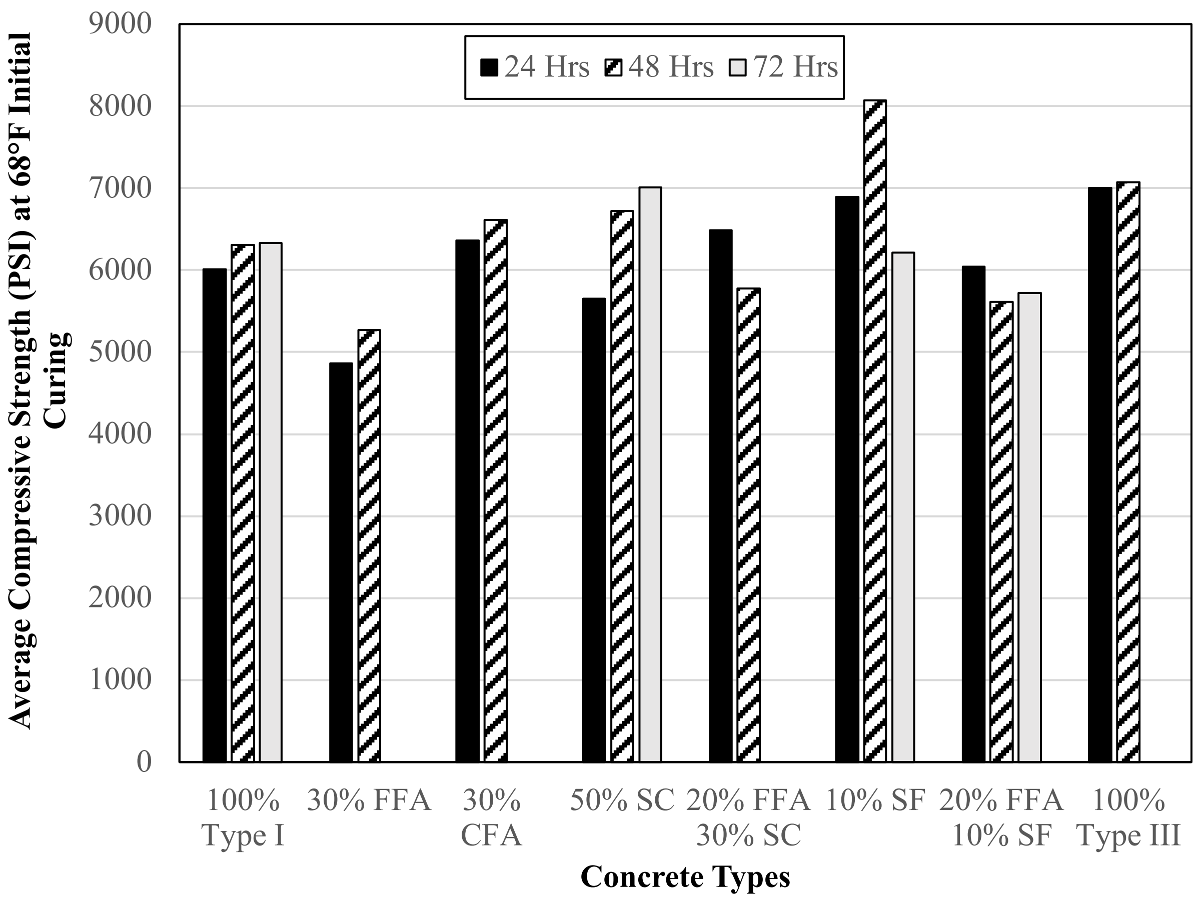

Figure 4-9: Compressive strengths of cylinders initially cured at 68 °F

Differences in strength between the initial curing durations were expected, and it was concluded that all 20 concrete batches had adequate 28-day compressive strength.

4.3 Verification Batches

A major aspect for the validity of the study was whether the results were repeatable. To determine the repeatability of the results, four concrete mixtures were chosen and repeated. The goal of repeating the batches was to show that the relative strength differences were similar to those obtained from the initial batches. First the average strengths of the initial and verification batches were compared at an initial curing temperature of 68 °F to ensure the concrete produced was an acceptable representation of typical concrete produced in the industry. These results are shown in Figure 4-10.

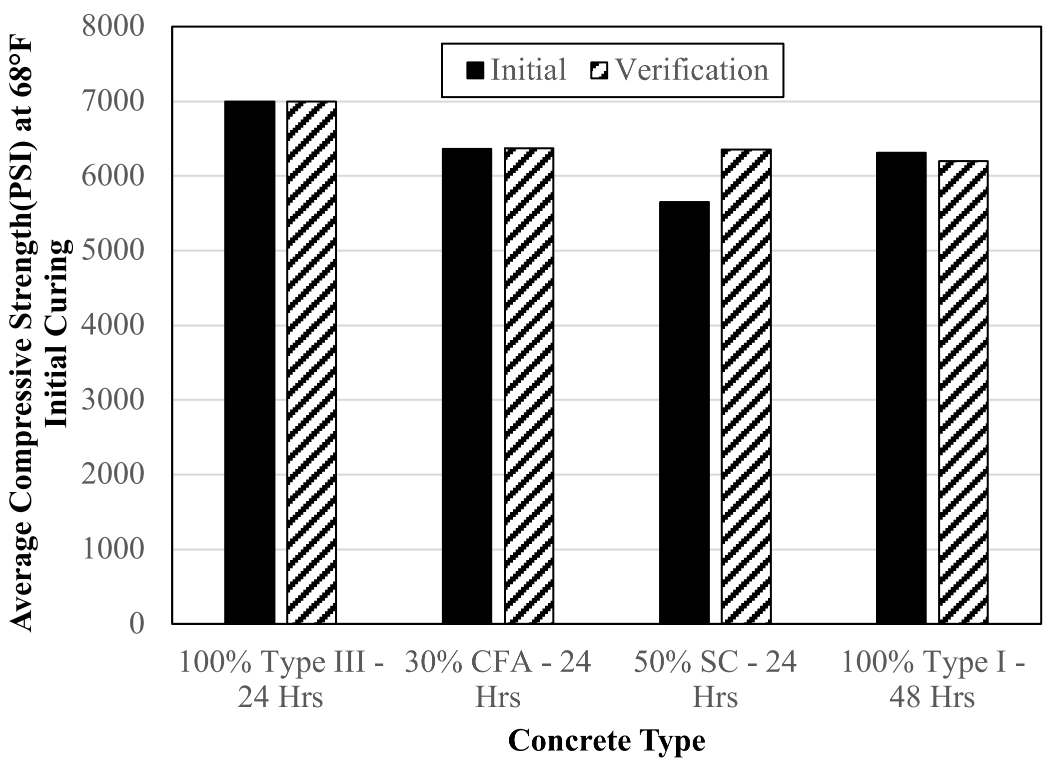

Figure 4-10: Compressive strengths of cylinders initially cured at 68 °F for verification batches

The compressive strengths shown in Figure 4-10 are quite similar. Remarkably, the verification batches for the 100% Type III concrete had an identical compressive strength to its initial batch while the 30% Class C fly ash had only a 10-psi difference. With the variability associated with making and testing concrete test cylinders, it was unexpected that the 28-day compressive strengths would be that close to each other. Overall, after reviewing the results at 68 °F, it was confirmed that the 28-day compressive strengths from the concretes were acceptable for analysis of the relative strength differences.

After completing these verification batches, the results were analyzed, and the repeatability of the experiment was confirmed. Table 4-10 includes the relative strength differences for the verification batches.

Table 4-10: Verification Batches relative strength differences

Initial Curing Strength Differences (%)					
Verification Batches	Initial Curing Temperature				
	60 °F	78 °F	84 °F	90 °F	100 °F
100% Type III 24 Hour – Initial	5	-2	-4	-3	-11
100% Type III 24 Hour -Verification	2	-5	-6	-8	-15
30% CFA 24 Hour – Initial	-1	-3	-6	-10	-12
30% CFA 24 Hour - Verification	7	-2	-4	-5	-11
50% SC 24 Hour - Initial	7	-5	-7	-9	-15
50% SC 24 Hour - Verification	3	-5	-9	-11	-15
100% Type I 48 Hour - Initial	3	-6	-12	-11	-16
100% Type I 48 Hour - Verification	5	-7	-6	-7	-14

As shown in Table 4-10, the four mixtures and curing durations chosen were as follows: 100% Type III 24 hours, 100% PCC 48 hours, 30% Class C Fly Ash 24 hours, and 50% Slag cement 24 hours. Each verification batch showed similar trends to their respective initial batches. Table 4-10 shows a maximum difference of 8% between the relative strength differences for the initial and verification batches. This maximum difference occurs for the 30% Class C Fly Ash concrete when initially cured at a temperature of 60 °F. Most of the differences in relative strength differences for the verification batches are within ±5%, while many are only ±2%

different. Some are identical when compared to the initial batches. Another important finding is the verification batch relative strength differences are consistent with the overall results of the study. All the relative strength differences were within the acceptable limit of ±10% when initially cured in a range from 60 °F to 80 °F. From the values in Table 4-10, it is clear to see that the test methods and procedures used in this study are repeatable, and the results are valid. For a visual representation, the relative strength differences of one verification batch is plotted against its initial batch in Figure 4-11.

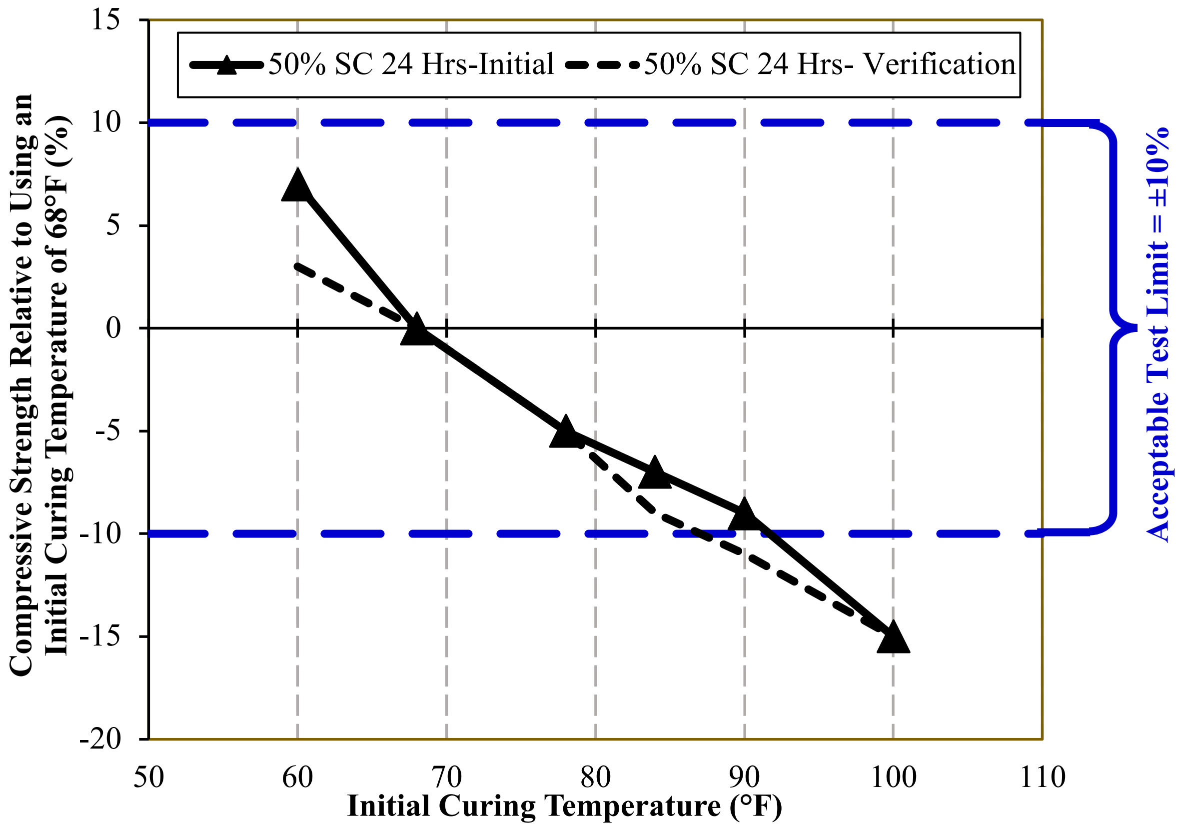

Figure 4-11: 50% Slag Cement Concrete Verification Batch

In Figure 4-11, the values are similar, with the relative strength differences for the 68 °F, 78 °F, and 100 °F initial curing temperatures being identical. All other verification batch plots can be found in Appendix B.

4.4 24-Hour Initial Curing Duration

According to AASHTO T 23 (2018) Section 10.1.2, the initial curing duration has a maximum duration of 48 hours. Although not in AASHTO T 23 (2018), the current edition of ALDOT 501 (2022) specifies a minimum of 24 hours of initial curing. It is common practice within the industry to ensure a minimum of 24 hours of initial curing before moving to final curing. To simulate the minimum initial curing period, cylinders were removed after they reached an age just greater than 24 hours and then placed in the final curing environment until testing at 28 days. After completing 28-day compressive strength tests for each of the concretes, the relative strength differences of the cylinders cured at 24 hours were calculated and are shown in Table 4-11.

Table 4-11: 24-Hour Initial Curing relative strength differences

Concrete	Initial Curing Temperature				
	60 °F	78 °F	84 °F	90 °F	100 °F
100% Type I	3	-9	-14	-16	-19
30% FFA	1	-6	-7	-7	-12
30% CFA	-1	-3	-6	-10	-12
50% SC	7	-5	-5	-9	-15
10% SF	8	-9	-15	-17	-20
20% CFA & 30% SC	8	-2	-4	-3	-9
20% CFA & 10% SF	9	-3	-6	-11	-16
100% Type III	5	-2	-4	-3	-11

Using the data provided in Table 4-11, a few conclusions can be made. As expected, as initial curing temperatures increase beyond the reference of 68 °F, the relative strength differences became more significant. In contrast, when the initial curing temperature decreases to 60 °F, there was typically a positive increase in the relative strength differences. For a numerical representation of the tests that had strength differences greater than the acceptable limit of ±10%,

Table 4-12 was created from Table 4-11 to illustrate which values were acceptable, and which were not.

Table 4-12: 24-Hour Initial Curing relative strength differences versus acceptable limits

Concrete	Initial Curing Temperature				
	60 °F	78 °F	84 °F	90 °F	100 °F
100% Type I	3	-9	-14	-16	-19
30% FFA	1	-6	-7	-7	-12
30% CFA	-1	-3	-6	-10	-12
50% SC	7	-5	-5	-9	-15
10% SF	8	-9	-15	-17	-20
20% CFA & 30% SC	8	-2	-4	-3	-9
20% CFA & 10% SF	9	-3	-6	-11	-16
100% Type III	5	-2	-4	-3	-11

*Values in shaded cells represent relative strength differences outside the acceptable limits

In Table 4-12, the shaded values represent values outside the chosen acceptable limit of ±10%. The first two columns, 60 °F and 78 °F, are the two initial curing temperatures within the specified temperature range of 60 °F to 80 °F in ALDOT 501 (2022) and AASHTO T 23 (2018). No relative strength differences fell outside of the acceptable limit for cylinders cured at these two temperatures. However, as the initial curing temperature is increased past 80 °F, the relative strength differences begin to fall outside of the acceptable limit. Of the cylinders cured at 84 °F, 90 °F, and 100 °F, 12 out of 24 (50%) cylinder relative strength differences fell outside the acceptable limit. The data suggests that it is crucial to maintain an initial curing environment range from 60 °F to 80 °F. If 80 °F is exceeded, one should expect to have relative strength differences outside the acceptable limit of ±10%. Using the values from Table 4-12, Figure 4-12 was created to illustrate the relative strength differences versus the initial curing temperatures for the batches cured at 24 hours.

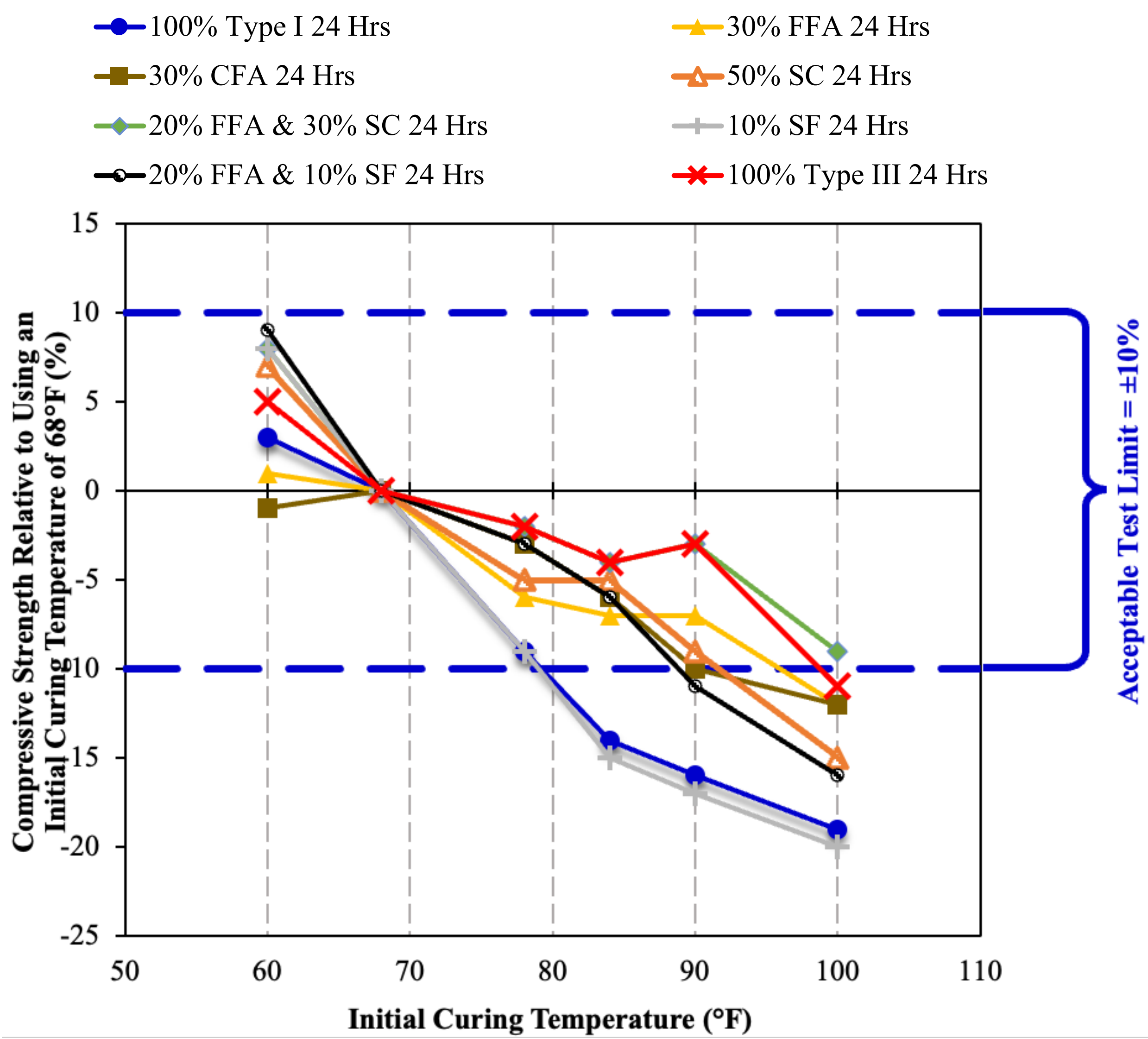

Figure 4-12: 24-Hour Initial Curing relative strength differences

Observation of Figure 4-12 shows that above an initial curing temperature of 80 °F, the concrete relative strength differences fall outside of the acceptable limit. As initial curing temperatures increased beyond the reference temperature of 68 °F, the relative strength differences continued to become closer to the acceptable limit and once above 80 °F fall outside the limit. The 100% Type I and 10% Silica Fume mixtures had the most extreme relative strength differences for every initial curing temperature beyond the reference temperature. As expected, the fly ashes and slag cement help to reduce the impact of initial curing temperature on the relative strength differences.

4.5 48-Hour Initial Curing Duration

In addition to testing the minimum initial curing duration of 24 hours, the maximum allowable curing duration permitted by AASHTO T 23 (2018) was also tested. Using AASHTO T 23 (2018), it was determined to cure cylinders as close to 48 hours as possible. It was critical to ensure that each cylinder was demolded and in the final curing room before the 48-hour mark. After completing 28-day compressive strength tests for each of the mixtures, the strength differences shown in Table 4-13 were determined.

Table 4-13: 48-Hour Initial Curing relative strength differences

Concrete	Initial Curing Temperature				
	60 °F	78 °F	84 °F	90 °F	100 °F
100% Type I	3	-6	-12	-11	-16
30% FFA	4	-4	-8	-7	-8
30% CFA	4	-5	-4	-8	-14
50% SC	3	-1	-1	-5	-8
10% SF	10	-6	-11	-14	-20
20% CFA & 30% SC	9	1	-5	-3	-9
20% CFA & 10% SF	9	-4	-11	-16	-23
100% Type III	7	1	-2	-5	-7

The same methods used for the 24-hour batches were used to determine the relative strength differences for the 48-hour batches. Similarly, as initial curing temperature increased, the relative strength differences became more significant. When the curing temperature decreased to 60 °F, the relative strength difference was in the positive direction for every batch. The results reaffirm the findings from the 24-hour initial curing batches that as concrete initial curing temperature increase, the relative strength differences will become more extreme. Table

4-14 was created from Table 4-13 to illustrate which values were acceptable, and which were not.

Table 4-14: 48-Hour Initial Curing relative strength differences versus acceptable limits

Concrete	Initial Curing Temperature				
	60 °F	78 °F	84 °F	90 °F	100 °F
100% Type I	3	-6	-12	-11	-16
30% FFA	4	-4	-8	-7	-8
30% CFA	4	-5	-4	-8	-14
50% SC	3	-1	-1	-5	-8
10% SF	10	-6	-11	-14	-20
20% CFA & 30% SC	9	1	-5	-3	-9
20% CFA & 10% SF	9	-4	-11	-16	-23
100% Type III	7	1	-2	-5	-7

*Values in shaded cells represent relative strength differences outside the acceptable limits

From the data shown in Table 4-14, for initial curing temperatures inside of 60 °F and 80 °F, the relative strength differences all remained within the acceptable limit of ±10%. Of the values from cylinders initially cured outside of the 60 °F to 80 °F temperature specification, 10 out of 24 (41%) fell outside the ±10% acceptable limit for relative strength differences. Using the data points provided in Table 4-14, a plot of only the 48-hour initial curing batches was created. Figure 4-13 illustrates the relative strength differences obtained for the 48-hour initial curing durations.

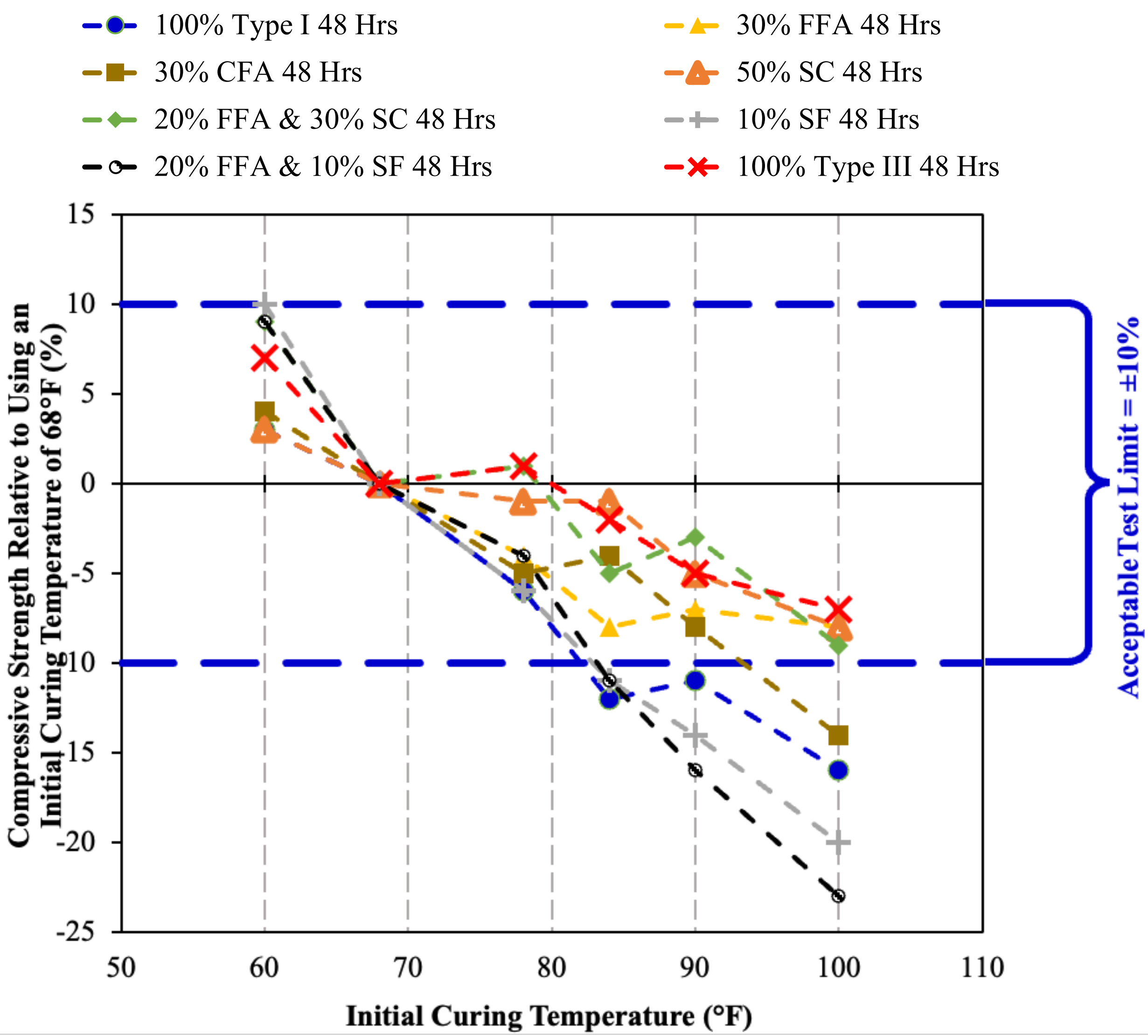

Figure 4-13: 48-Hour Initial Curing relative strength differences

Initial curing durations of 48 hours had significant effects on the relative strength differences of the concrete batches. Between 60 °F and 80 °F, every data point was within the chosen acceptable limit. However, after exceeding the 80 °F curing temperature, the relative strength differences begin to spread out and eventually fell outside of the acceptable limit for many concretes. Like the 24-hour initial curing batch, the 100% Type I and 10% Silica Fume batches had significant relative strength differences, but the 20% Class F fly ash with 10% Silica Fume batch had the largest when the initial curing temperature was increased to 100 °F. The results from the 48-hour batches reaffirm the importance of curing concrete cylinders in an initial curing environment that maintains a temperature range from 60 °F to 80 °F.

4.6 72-Hour Initial Curing Duration

After analyzing the results of the 48-hour initial curing duration batches. It was necessary to determine how much impact the initial curing duration has on the 28-day compressive strength. A third trial of initial curing duration of 72 hours was undertaken. Instead of repeating every batch, four mixture proportions were chosen using the acquired data from the 24- and 48-hour batches and repeated with an initial curing duration of 72 hours. These four concrete mixtures were selected based on their relative strength differences for the 24- and 48-hour initial curing durations. The concrete mixtures that had the greatest relative strength differences at initial curing durations of 24 and 48 hours were chosen as it was hypothesized they would also have the most significant differences when exposed to an initial curing duration of 72 hours. The four concretes chosen were 100% Type I PCC, 50% Slag cement, 10% Silica Fume, and 20% Class F fly ash with 10% Silica Fume. Using the same procedures as the 24 and 48-hour initial curing duration batches, these mixtures were batched and left in their respective initial curing environments for just under 72 hours. Care was taken to ensure the removal and placement of the demolded cylinders in the final curing room occurred before the 72-hour mark. After completing 28-day compressive strength tests for each of the four mixtures, the following strength differences in Table 4-15 were obtained from the test results.

Table 4-15: 72-Hour Initial Curing relative strength differences

Concrete	Initial Curing Temperature				
	60 °F	78 °F	84 °F	90 °F	100 °F
100% Type I	4	-4	-7	-7	-11
50% SC	3	-2	-5	-8	-12
10% SF	3	-5	-11	-13	-18
20% CFA & 10% SF	5	-9	-13	-16	-22

The results of the 72-hour initial curing durations were unexpected because although the initial curing duration was increased by an entire day, the relative strength differences did not change much when compared to the previous two initial curing durations. The maximum relative strength difference occurred in the 20% Class F fly ash with 10% Silica Fume concrete and was -22% when initially cured at a temperature of 100 °F. When initially cured at the same temperature for 48 hours, the relative strength difference was -23%, which is quite large. Similarly, the 100% Type I PCC differences were much less than for the 24- and 48-hour initial curing duration batches. Like the 24-hour and 48-hour initial curing duration batches, a visual representation is provided with the values that fell outside of the acceptable limits shaded in Table 4-16.

Table 4-16: 72-Hour Initial Curing relative strength differences versus acceptable limits

Concrete	Initial Curing Temperature				
	60 °F	78 °F	84 °F	90 °F	100 °F
100% Type I	4	-4	-7	-7	-11
50% SC	3	-2	-5	-8	-12
10% SF	3	-5	-11	-13	-18
20% CFA & 10% SF	5	-9	-13	-16	-22

*Values in shaded cells represent relative strength differences outside the acceptable limits

Table 4-16 shows a similar relationship as the 24-hour and 48-hour initial curing duration tests of the relative strength differences with respect to the initial curing temperature. For initial curing temperatures from 60 °F to 80 °F, the relative strength differences remain within the acceptable limits. Outside of the allowable temperature specification, 8 out of 12 (66%) of the values are outside of the acceptable limit. It is important to note that a higher percentage of values over the acceptable limit does not mean the 72-hour batches had worse results than the 48- and 72-hour initial curing duration batches. The four concrete mixtures chosen were the ones that had the most significant relative strength differences in the previous tests. Therefore, a higher percentage of failures should be expected. If only the mixtures tested in the 72-hour batches are evaluated, the 72-hour initial curing duration has the lowest percentage of failure at 66%. Both the 24- and 48-hour initial curing duration batches have a failure rate of 75% for the four batches performed in the 72-hour trial when the initial curing temperature exceeds 80 °F which is similar to the 72-hour initial curing duration results. A visual representation of the four 72-hour initial curing duration batches is shown in Figure 4-14.

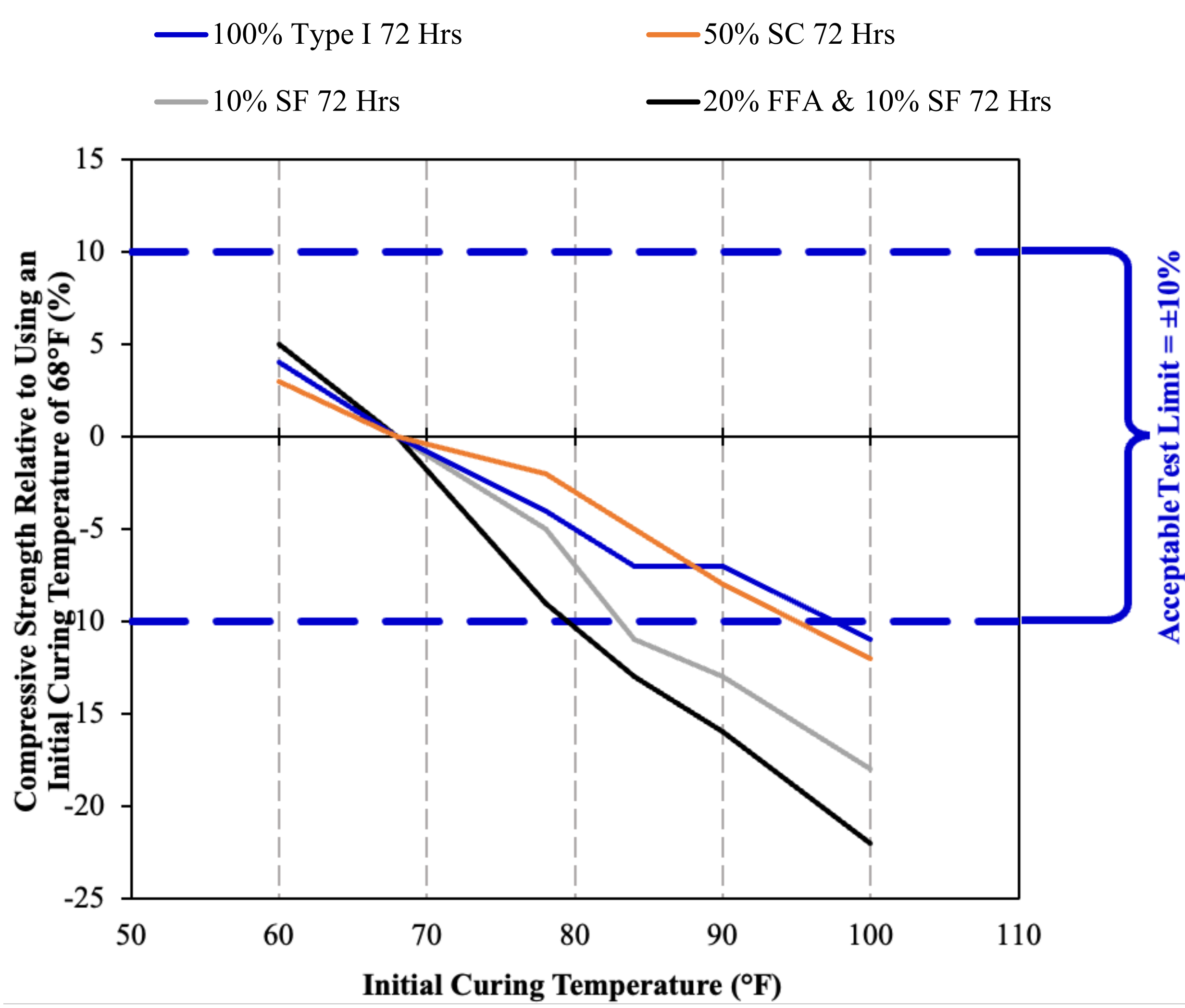

Figure 4-14: 72-Hour Initial Curing relative strength differences

Like the 24- and 48-hour initial curing duration batches, the cylinders that remained in the AASHTO T 23 (2018) curing environment temperature specification had minimal relative strength differences. In Figure 4-14, the 20% Fly Ash with 10% Silica Fume is the closest to exceeding the acceptable limit while remaining in the initial curing temperature requirements. At 78 °F there is a difference of -9% and although, close to the limit, it is still within ±10% and acceptable for this study. As initial curing temperatures increased past the 80 °F mark, the results were similar to those obtained when testing the previous initial curing durations of 24 and 48 hours. The results begin to fall outside of the acceptable limits as the initial curing temperature is increased past 80 °F. At the initial curing temperature of 100 °F, all relative strength differences were outside the acceptable limits of ±10%.

4.7 General Results

The relative strength differences of all 24 concrete batches tested are presented in Figure 4-15.

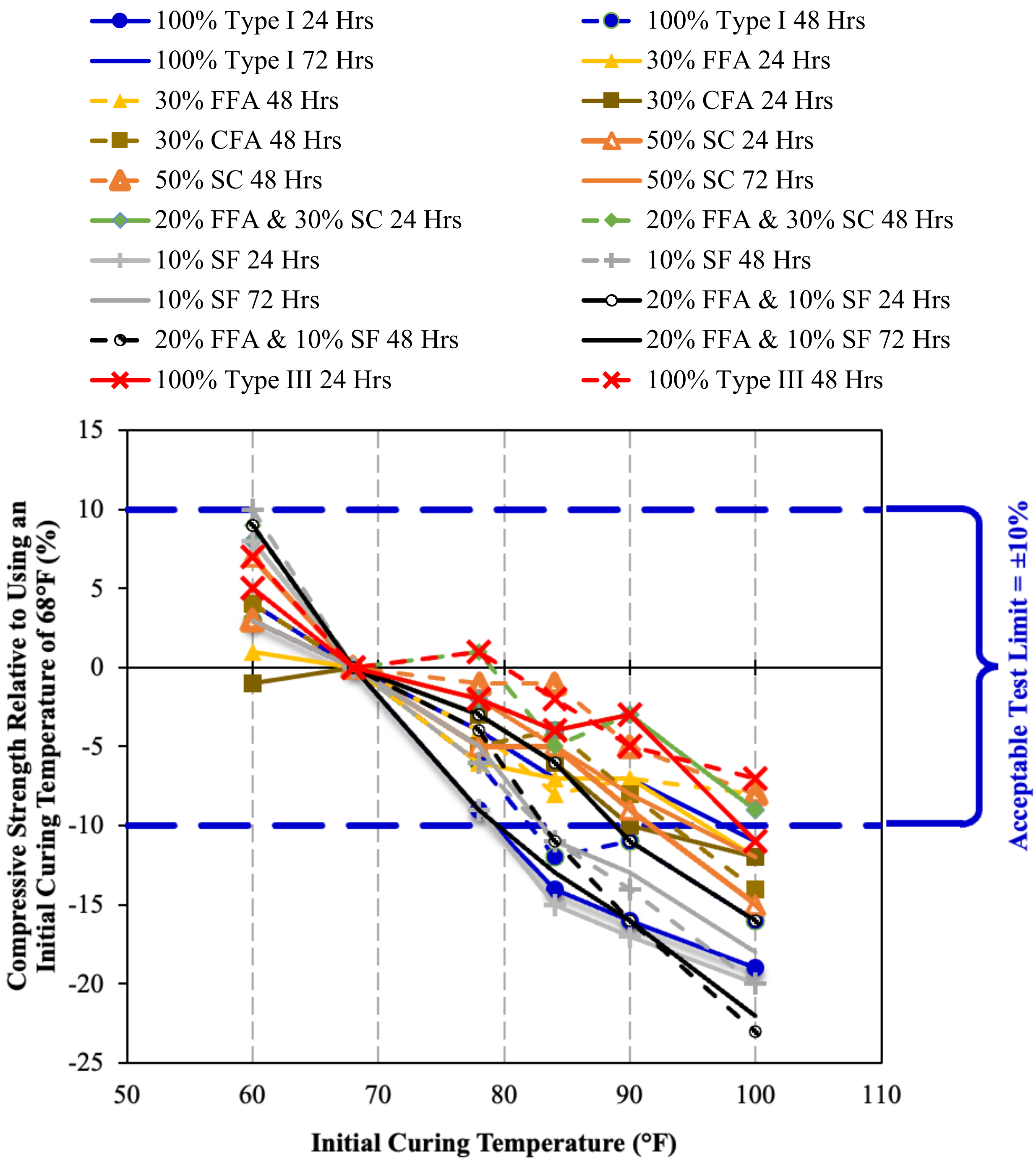

Figure 4-15: Concrete relative strength differences for all batches

From the data presented and Figure 4-15, there is a clear trend of increasing relative strength differences versus increasing initial curing temperature. No clear trend of relative strength differences versus initial curing duration is shown. Within each concrete batch, as the initial curing temperature increased, the relative strength differences became more significant. While the extent of the relative strength differences changed as the initial curing temperatures increased within each batch, an overall trend of more relative strength differences was shown for each batch. The data in Figure 4-15 show that in general, as initial curing temperature increases, the concrete relative strength differences become more significant. The data in Figure 4-15 shows that for initial curing temperatures at values of 60 °F, 68 °F, and 78 °F, the strength differences remained within the acceptable limit of ±10%.

In addition to curing temperature, the effects of initial curing duration on 28-day compressive strength of concrete were analyzed. Figure 4-16 shows the relative strength differences versus initial curing temperature.

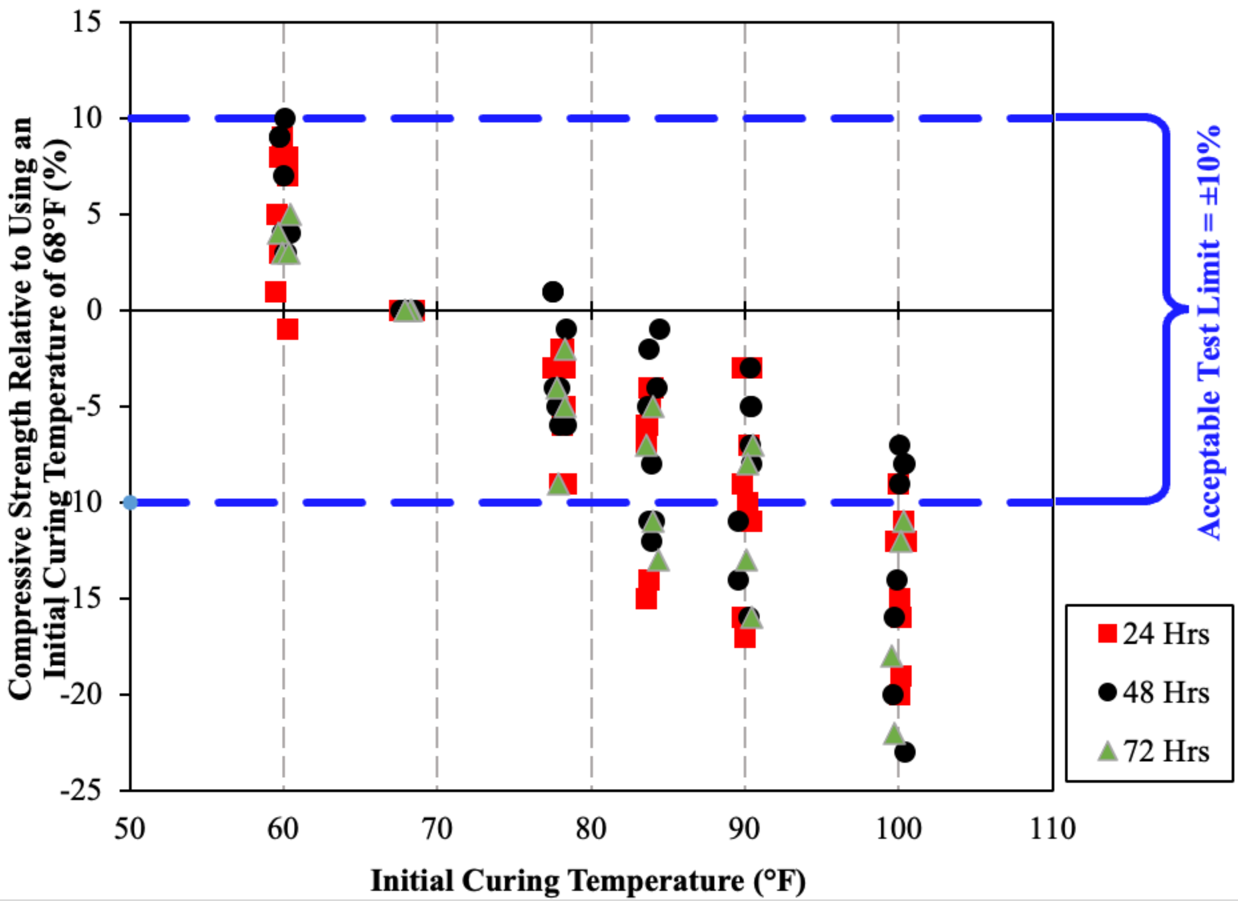

Figure 4-16: Concrete strength differences relative to initial curing duration

It was expected that as initial curing duration was extended that more of the relative strength differences would fall outside of the acceptable limit. However, the increase in initial curing duration did not comprehensively affect the relative strength differences of the concretes tested. In fact, the data suggests that concrete subjected to an initial curing duration of 72 hours has a similar strength development of concrete initially cured for only 24 or 48 hours. Figure 4-16 clearly shows that for the broad range of concretes tested, if the initial curing temperature remains from 60 °F to 80 °F then the relative strength differences should remain within the acceptable limit of ±10% regardless of the initial curing duration.

4.8 Summary

A total of 20 concrete batches and 4 verification batches were produced to determine the impact of concrete initial curing temperature and duration on the 28-day compressive strength of concrete. Using six different initial curing temperatures and three different initial curing durations, the relative strength differences were determined. All strength results were calculated using the average 28-day compressive strength of three cylinders for the respective initial curing environment. The relative strength differences are based off the reference curing temperature of 68 °F. Review of all the acquired data confirms that as initial curing temperature increases the relative strength differences will become larger. The difference in initial curing duration from 24 to 72-hours does not have a significant effect on the compressive strength of concrete. These results are similar to the findings of Meininger (1983) which showed little reduction in compressive strength when the initial curing duration was extended 24 hours. Within initial curing durations of 24 and 72 hours, if the concrete cylinders remain within the specified temperature range from 60 °F to 80 °F, the relative strength differences are expected to remain in the acceptable limit. It is recommended that to ensure the relative strength differences within a certain batch of concrete remains smaller than ±10%, the initial curing environment should remain from 60 °F to 80 °F, as specified by AASHTO T 23 (2018). The acceptable strength difference limit of ±10% is larger than the current AASHTO T 22 (2020) limit of ±7.8%, because the concretes evaluated were initially cured at six different temperatures. The same concrete exposed to the same initial curing environment, would have much smaller relative strength differences. In conclusion, for the most accurate compressive strength results, concrete test cylinders should remain at an initial curing temperature range from 60 °F to 80 °F and have a maximum initial curing duration of 72 hours.

Chapter 5: Summary, Conclusions, And Recommendations

In this chapter, the work performed is summarized. Also, a list of conclusions and recommendations based off the data presented in Chapter 4 is included.

5.1 Summary of Work Performed

The primary objectives of this study were to determine the effect of initial curing temperature and duration on the 28-day compressive strength of concrete. Concrete cylinders were subjected to initial curing temperatures of 60 °F, 68 °F, 78 °F, 84 °F, 90 °F, and 100 °F for one of three different initial curing durations before being placed in a final curing room that remained at a temperature of 73.5 ± 3.5 °F and provided a relative humidity of 100%. Using a reference curing temperature of 68 °F, the relative strength differences in compressive strength was calculated for the concrete initially cured at the other temperatures. Using an acceptable strength difference limit of ±10%, the relative strength differences of the concrete batches were evaluated. Each of the eight concrete mixtures were subjected to initial curing durations of 24 and 48 hours, while four concrete mixtures were initially cured for 72 hours as well. Four verification batches were produced to determine the repeatability of the study. In total, 576 6”x12” cylindrical concrete specimens were tested to determine the effect of initial curing on the 28-day compressive strength of concrete.

5.2 Conclusions

For the broad types of concretes tested in this study, the following conclusions can be made regarding the initial curing of concrete:

- When initial curing temperature remains within a range from 60 °F to 80 °F the determined strength differences do not exceed the acceptable limit of ±10%.

- For initial curing temperatures exceeding 80 °F, the percentage of cylinders exceeding the acceptable limit of ±10% is equal to 50%.
- It is crucial to adhere to the initial curing specification because a maximum strength difference of 23%, almost a quarter of the control strength, was recorded when the specified temperature range was exceeded.
- Initial curing durations varying 24 to 72 hours do not significantly affect the 28-day compressive strength of concrete cylinders if the initial curing temperature remains within the specified 60 °F to 80 °F range.

5.3 Recommendations

Based on the results and conclusions of this studt, the following actions are recommended:

- Continue the requirement of an initial curing temperature environment from 60 °F to 80 °F in ALDOT 501 (2022).
- Increase the maximum allowable initial curing duration in ALDOT 501 (2022) from 48 hours to 72 hours.

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Appendix A: Data

A.1 Concrete Compressive Strength Results

Table A-1: Compressive strength for 100% Type I PCC - 24 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 °F)
60 °F	6140	6190	3
	6240		
	6200		
68 °F	6040	6010	0
	6010		
	5990		
78 °F	5530	5450	-9
	5430		
	5380		
84 °F	5220	5190	-14
	5040		
	5320		
90 °F	5010	5070	-16
	5050		
	5150		
100 °F	4720	4850	-19
	4900		
	4940		

*Outlier

Table A-2: Compressive strength for 30% Class F Fly Ash - 24 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 ° F)
60 °F	4890	4920	1
	5030		
	4840		
68 °F	4940	4860	0
	4890		
	4750		
78 °F	4590	4590	-6
	4610		
	4570		
84 °F	4440	4530	-7
	4490		
	4660		
90 °F	4570	4540	-7
	4590		
	4450		
100 °F	4190	4280	-12
	4280		
	4360		

*Outlier

Table A-3: Compressive strength for 30% Class C Fly Ash - 24 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 º F)
60 °F	6290	6320	-1
	6140		
	6520		
68 °F	6320	6360	0
	6400		
	6360		
78 °F	6180	6200	-3
	6300		
	6120		
84 °F	6050	6000	-6
	5970		
	5980		
90 °F	5690	5740	-10
	5840		
	5680		
100 °F	5460	5570	-12
	5730		
	5530		

*Outlier

Table A-4: Compressive strength for 50% Slag Cement - 24 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 ° F)
60 °F	6190	6040	7
	6030		
	5910		
68 °F	5840	5650	0
	5560		
	5560		
78 °F	5570	5390	-5
	5190		
	5420		
84 °F	5180	5070	-10
	5560*		
	4950		
90 °F	5030	5140	-9
	5100		
	5290		
100 °F	4790	4790	-15
	4790		
	4800		

*Outlier

Table A-5: Compressive strength for 10% Silica Fume - 24 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 ° F)
60 °F	7410	7430	8
	7300		
	7580		
68 °F	6670	6890	0
	7010		
	7000		
78 °F	5990	6260	-9
	6410		
	6370		
84 °F	5790	5870	-15
	5710		
	6100		
90 °F	5640	5700	-17
	5660		
	5800		
100 °F	5640	5520	-20
	5360		
	5560		

*Outlier

Table A-6: Compressive strength for 20% Class F Fly Ash with 30% Slag Cement - 24 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 º F)
60 °F	6790	7010	8
	7140		
	7110		
68 °F	6540	6490	0
	6420		
	6520		
78 °F	6540	6380	-2
	6290		
	6310		
84 °F	6020	6230	-4
	6400		
	6270		
90 °F	6380	6320	-3
	6350		
	6230		
100 °F	5760	5920	-9
	5930		
	6070		

*Outlier

Table A-7: Compressive strength for 20% Class F Fly Ash with 10% Silica Fume - 24 hours initial curing

Curing Location	**28-Day Compressive Strength (psi)**	**Average Compressive Strength (psi)**	**Strength Difference (% From 68 º F)**
60 °F	6730	6570	9
	6530		
	6460		
68 °F	6030	6040	0
	6150		
	5930		
78 °F	5840	5850	-3
	5920		
	5790		
84 °F	5810	5650	-6
	5550		
	5590		
90 °F	5410	5390	-11
	5370		
	5390		
100 °F	5060	5050	-16
	5090		
	5000		

*Outlier

Table A-8: Compressive strength for 100% Type III PCC - 24 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 º F)
60 °F	7560	7320	5
	7250		
	7140		
68 °F	7060	7000	0
	6890		
	7040		
78 °F	6800	6860	-2
	7070		
	6710		
84 °F	6880	6710	-4
	6820		
	6430		
90 °F	6780	6780	-3
	6760		
	6800		
100 °F	6150	6200	-11
	6270		
	6180		

*Outlier

Table A-9: Compressive strength for 100% Type I PCC - 48 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 º F)
60 °F	6410	6510	3
	6500		
	6620		
68 °F	6330	6310	0
	6190		
	6410		
78 °F	5440*	5960	-6
	5930		
	5990		
84 °F	5610	5550	-12
	5560		
	5470		
90 °F	5610	5590	-11
	5520		
	5640		
100 °F	5200	5280	-16
	5400		
	5250		

*Outlier

Table A-10: Compressive strength 30% Class F Fly Ash - 48 hours initial curing

Curing Location	**28-Day Compressive Strength (psi)**	**Average Compressive Strength (psi)**	**Strength Difference (% From 68 ⁰ F)**
60 °F	5500	5480	4
	5330		
	5620		
68 °F	5130	5270	0
	5440		
	5250		
78 °F	4940	5070	-4
	5150		
	5110		
84 °F	4830	4830	-8
	4730		
	4940		
90 °F	4810	4910	-7
	4960		
	4950		
100 °F	4940	4850	-8
	4800		
	4800		

*Outlier

Table A-11: Compressive strength for 30% Class C Fly Ash - 48 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 ^{o} F)
60 °F	6710	6880	4
	6890		
	7030		
68 °F	6530	6610	0
	6540		
	6770		
78 °F	6170	6300	-5
	6230		
	6510		
84 °F	6210	6330	-4
	6300		
	6490		
90 °F	6020	6060	-8
	5970		
	6200		
100 °F	5690	5660	-14
	5550		
	5750		

*Outlier

Table A-12: Compressive strength for 50% Slag Cement - 48 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 ^{0} F)
60 °F	6870	6960	3
	6860		
	7140		
68 °F	6690	6760	0
	6820		
	6760		
78 °F	6890	6700	-1
	6600		
	6600		
84 °F	6710	6660	-1
	6530		
	6730		
90 °F	6550	6440	-5
	6370		
	6390		
100 °F	6130	6190	-8
	6220		
	6210		

*Outlier

Table A-13: Compressive strength for 10% Silica Fume - 48 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 º F)
60 °F	8860	8890	10
	8860		
	8960		
68 °F	7900	8070	0
	7910		
	8400		
78 °F	7760	7570	-6
	7460		
	7480		
84 °F	7040	7220	-11
	7350		
	7270		
90 °F	6870	6970	-14
	7170		
	6880		
100 °F	6370	6430	-20
	6430		
	6480		

*Outlier

Table A-14: Compressive strength for 20% Class F Fly Ash with 30% Slag Cement - 48 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 º F)
60 °F	6060	6310	9
	6350		
	6520		
68 °F	5550	5780	0
	5820		
	5980		
78 °F	5720	5810	1
	5980		
	5720		
84 °F	5410	5510	-5
	5640		
	5480		
90 °F	5510	5580	-3
	5580		
	5640		
100 °F	4740*	5260	-9
	5250		
	5270		

*Outlier

Table A-15: Compressive strength for 20% Class F Fly Ash with 10% Silica Fume - 48 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 º F)
60 °F	5980	6090	9
	6090		
	6210		
68 °F	5530	5610	0
	5650		
	5660		
78 °F	5470	5360	-4
	5350		
	5270		
84 °F	5110	5000	-11
	5050		
	4830		
90 °F	4580	4700	-16
	4670		
	4850		
100 °F	4290	4320	-23
	4360		
	4300		

*Outlier

Table A-16: Compressive strength for 100% Type III PCC - 48 hours initial curing

Curing Location	**28-Day Compressive Strength (psi)**	**Average Compressive Strength (psi)**	**Strength Difference (% From 68 ⁰ F)**
60 °F	7500	7600	7
	7430		
	7860		
68 °F	7190	7070	0
	6870		
	7140		
78 °F	7210	7170	1
	7020		
	7270		
84 °F	6960	6910	-2
	6690		
	7090		
90 °F	6570	6710	-5
	6570		
	7000		
100 °F	6550	6560	-7
	6480		
	6650		

*Outlier

Table A-17: Compressive strength for 100% Type I PCC - 72 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 º F)
60 °F	6750	6570	4
	6450		
	6500		
68 °F	6210	6330	0
	6520		
	6270		
78 °F	6220	6050	-4
	5960		
	5980		
84 °F	5810	5870	-7
	6010		
	5800		
90 °F	5910	5880	-7
	5850		
	5870		
100 °F	5490	5620	-11
	5690		
	5680		

*Outlier

Table A-18: Compressive strength for 50% Slag - 72 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 ⁰ F)
60 °F	7220	7240	3
	7190		
	7300		
68 °F	7280	7010	0
	6860		
	6890		
78 °F	6960	6880	-2
	6790		
	6410*		
84 °F	6570	6670	-5
	6790		
	6650		
90 °F	6720	6460	-8
	6420		
	6240		
100 °F	6280	6150	-12
	6220		
	5960		

*Outlier

Table A-19: Compressive strength for 10% Silica Fume - 72 hours initial curing

Curing Location	**28-Day Compressive Strength (psi)**	**Average Compressive Strength (psi)**	**Strength Difference (% From 68 ^{o} F)**
60 °F	6320	6380	3
	6380		
	6450		
68 °F	6100	6210	0
	6220		
	6300		
78 °F	5730	5930	-5
	5880		
	6170		
84 °F	5400	5530	-11
	5710		
	5480		
90 °F	5370	5430	-13
	5470		
	5450		
100 °F	4890	5080	-18
	5080		
	5260		

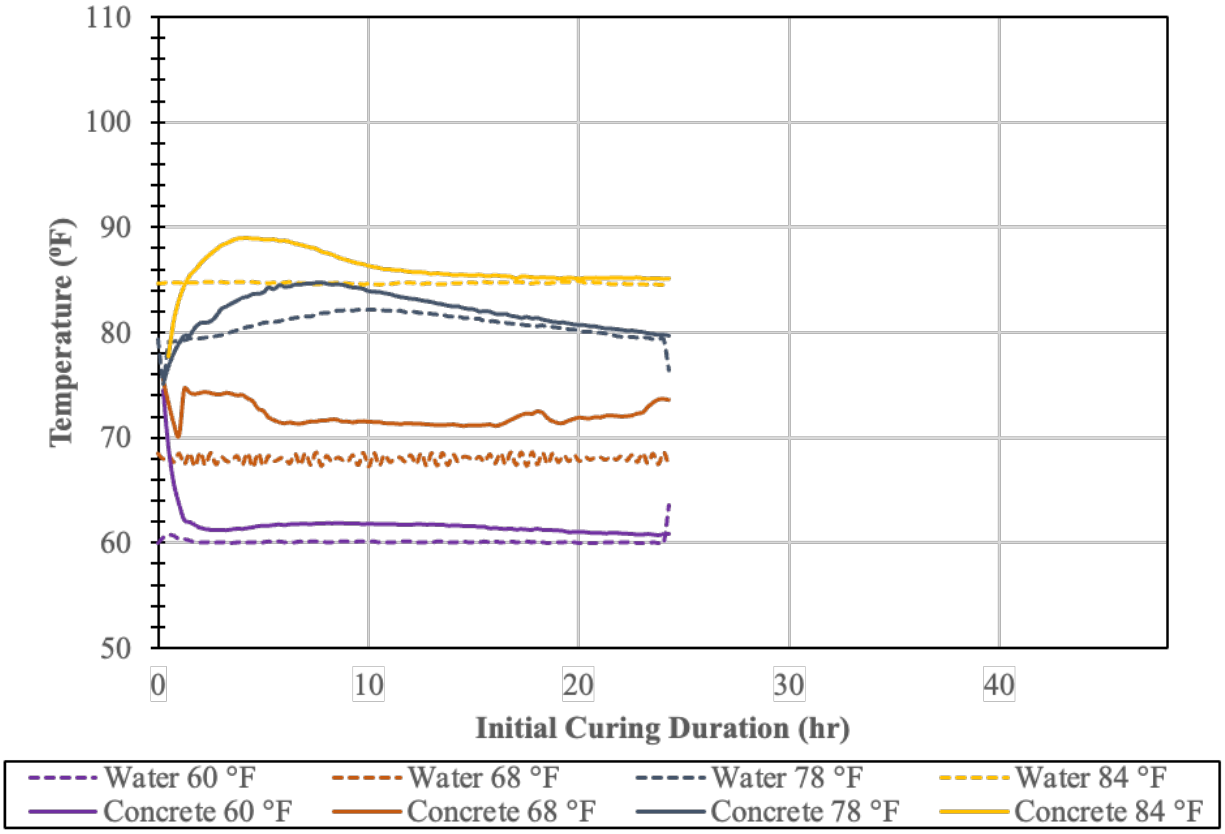
*Outlier

Table A-20: Compressive strength for 20% Class F Fly Ash with 10% Silica Fume - 72 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 º F)
60 °F	5910	6010	5
	6080		
	6030		
68 °F	5660	5720	0
	5790		
	5700		
78 °F	5300	5210	-9
	5150		
	5190		
84 °F	4850	4950	-13
	5040		
	4960		
90 °F	4860	4820	-16
	4760		
	4850		
100 °F	4430	4460	-22
	4450		
	4510		

*Outlier

A.2: Initial Curing Temperature versus Time Plots



Note: 90 °F and 100 °F temperature probes malfunctioned.

Figure A-1: 100% Type I PCC 24 hours initial curing temperatures plot

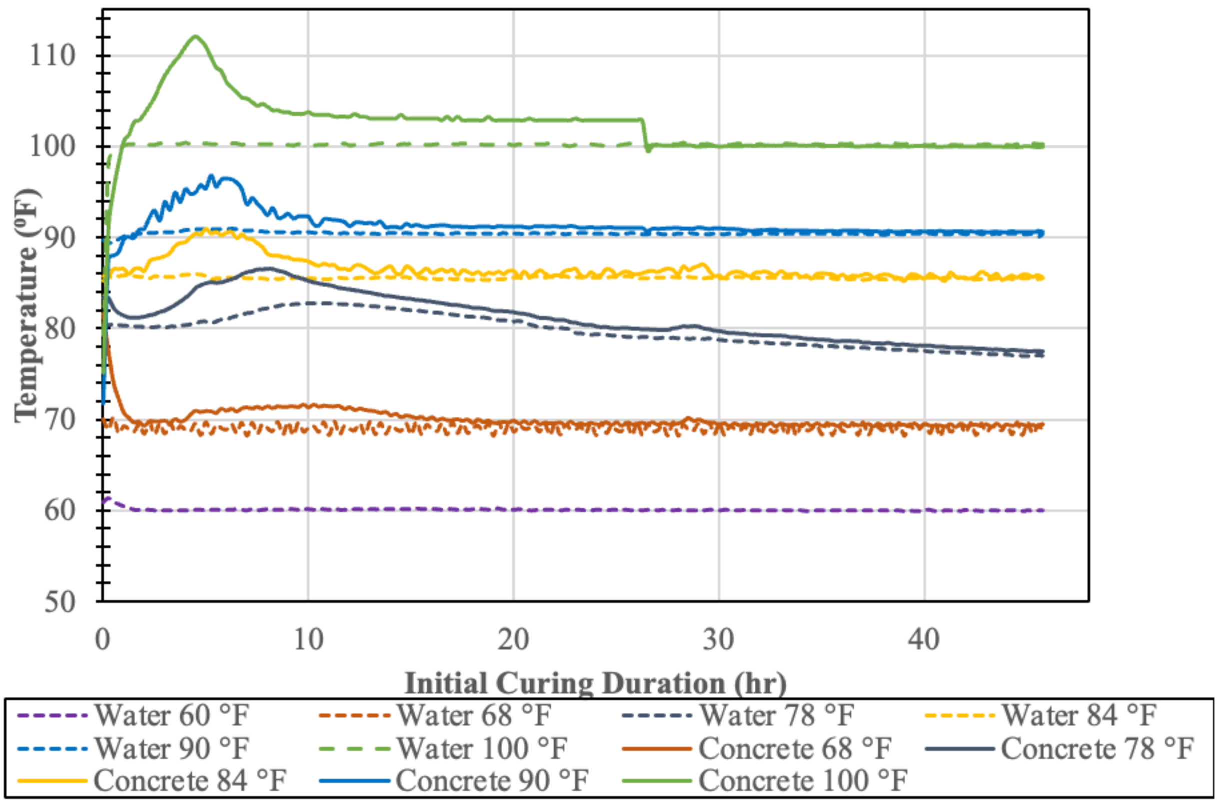


Figure A-2: 30% Class F Fly Ash 24 hours initial curing temperatures plot

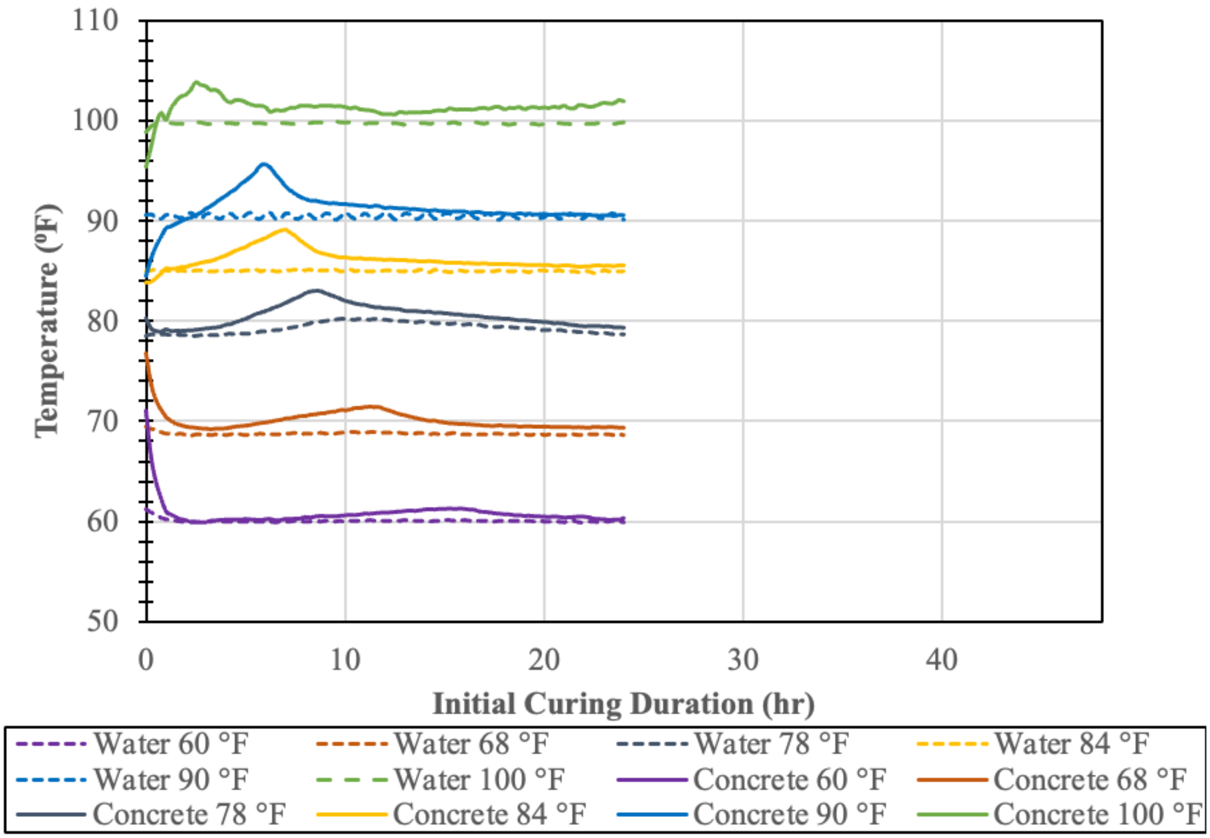


Figure A-3: 30% Class C Fly Ash 24 hours initial curing temperatures plot

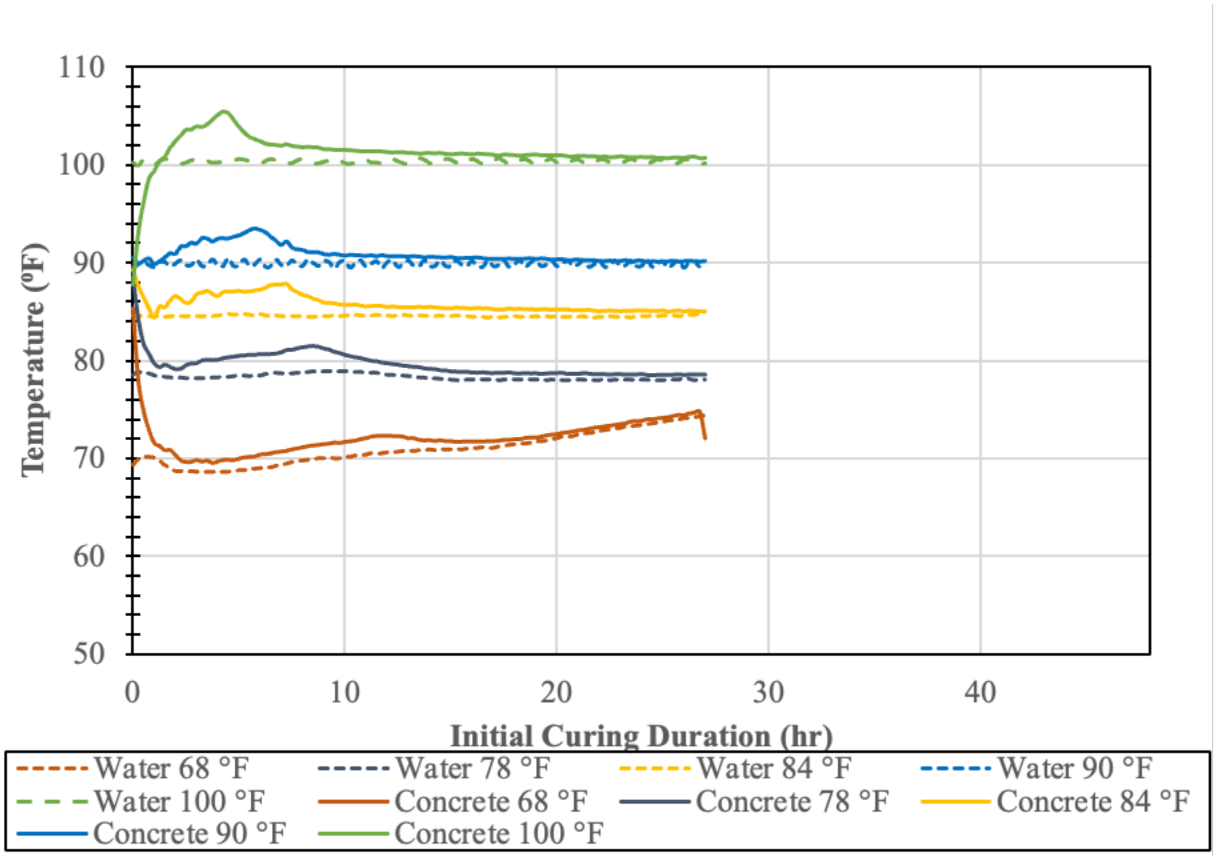


Figure A-4: 50% Slag Cement 24 hours initial curing temperatures plot

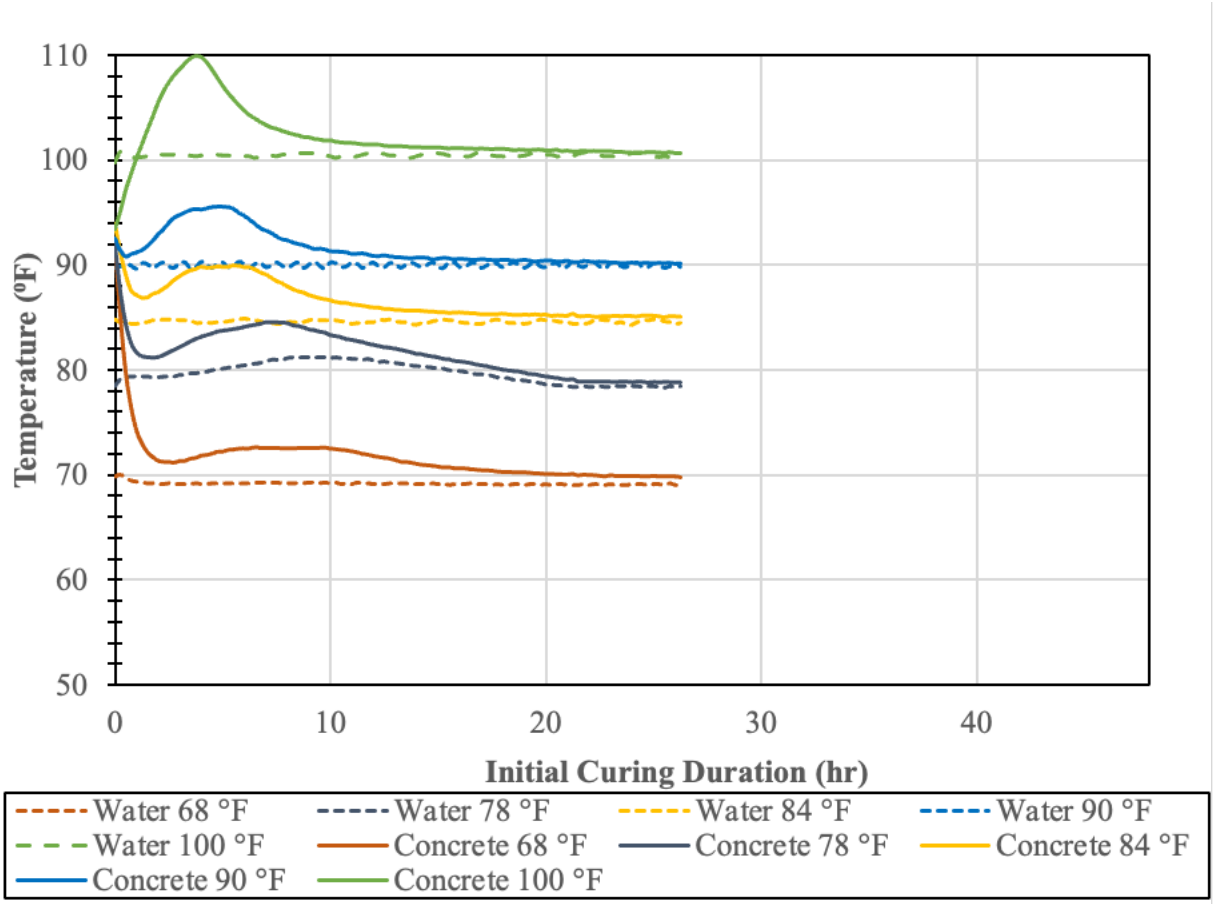


Figure A-5: 10% Silica Fume 24 hours initial curing temperatures plot

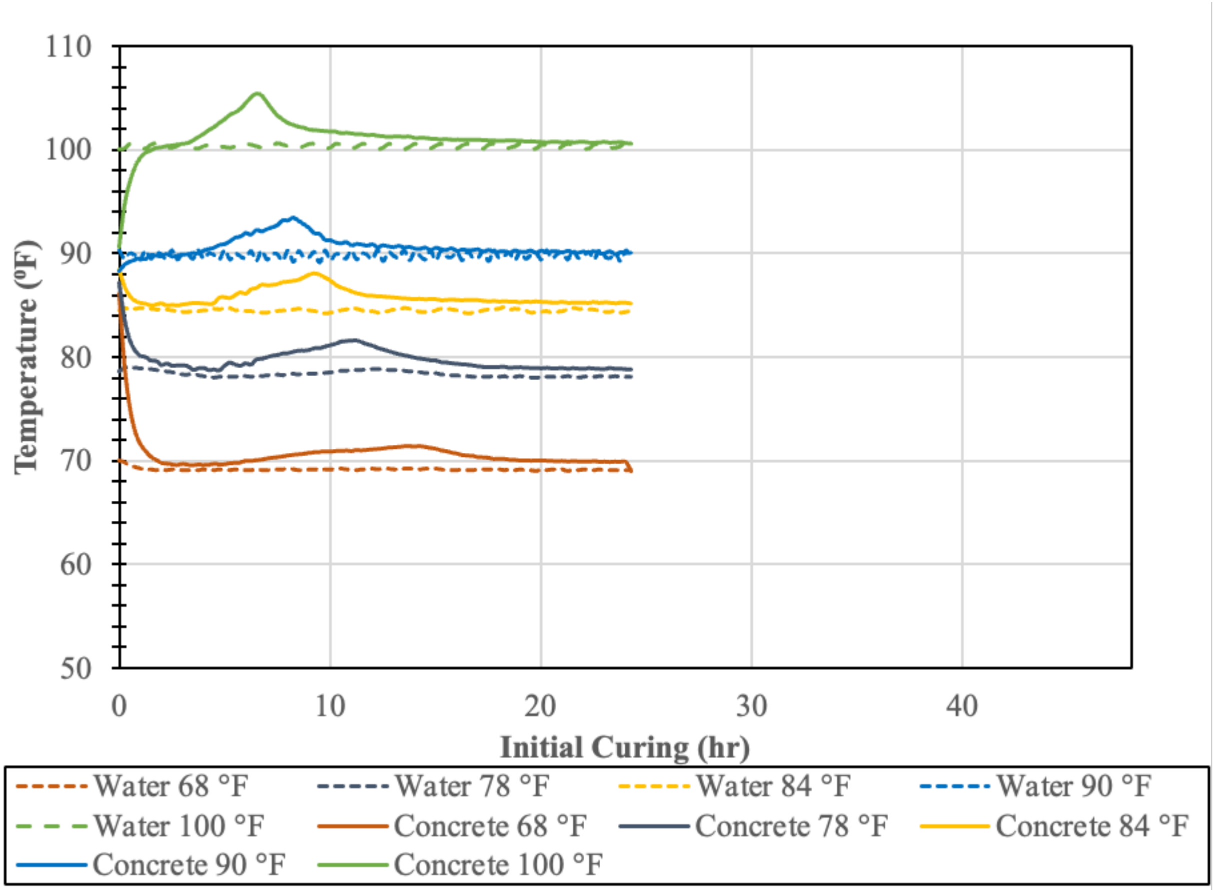


Figure A-6: 20% Class F Fly Ash with 30% Slag Cement 24 hours initial curing temperatures plot

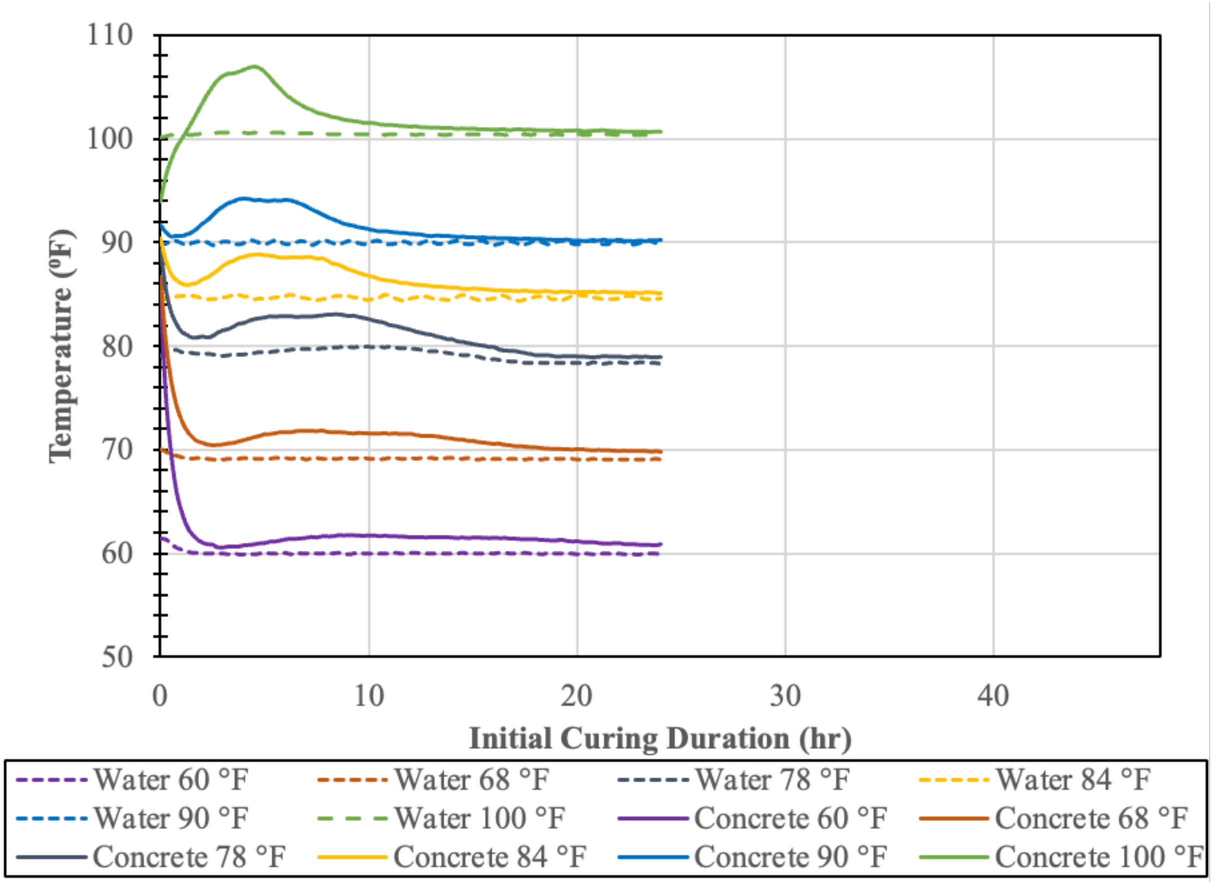


Figure A-7: 20% Class F Fly Ash with 10% Silica Fume 24 hours initial curing temperatures plot

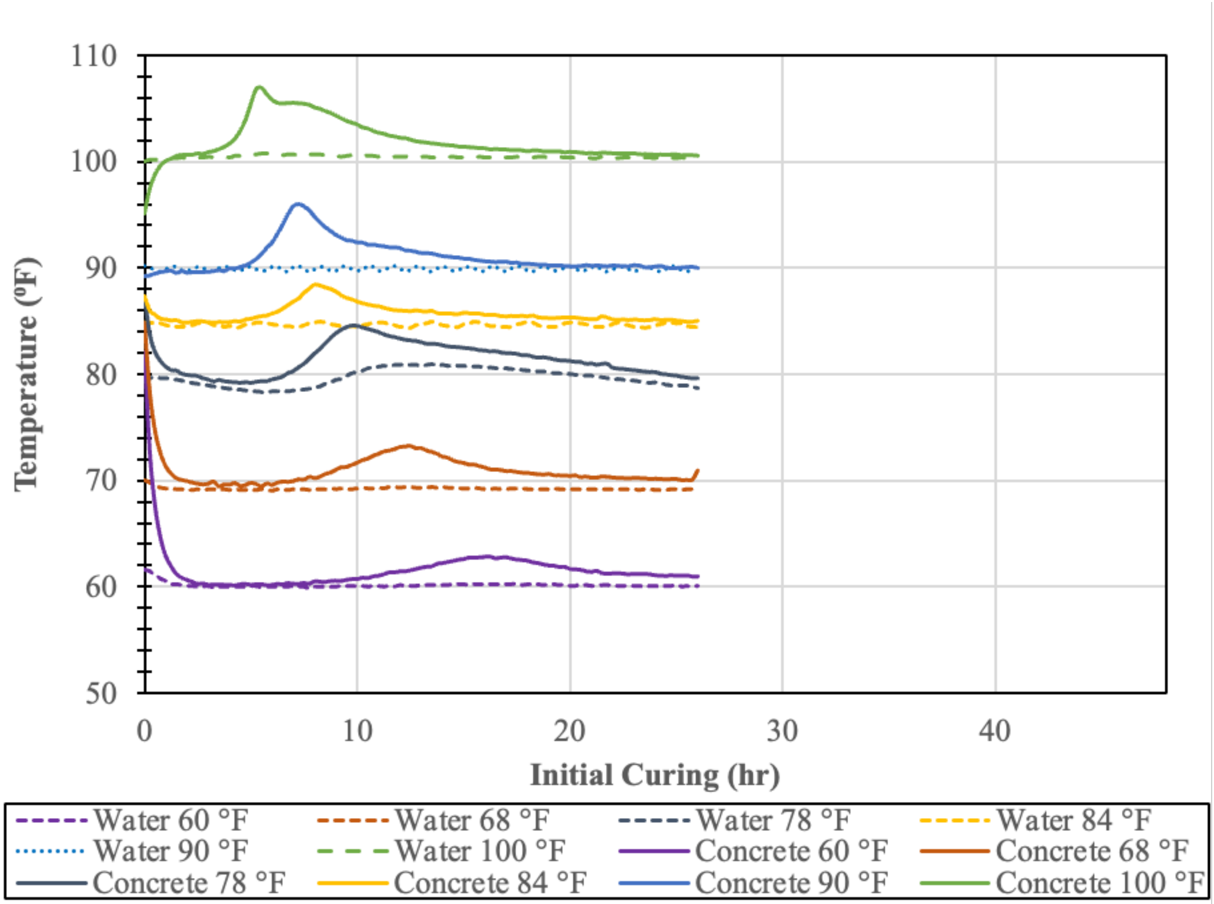
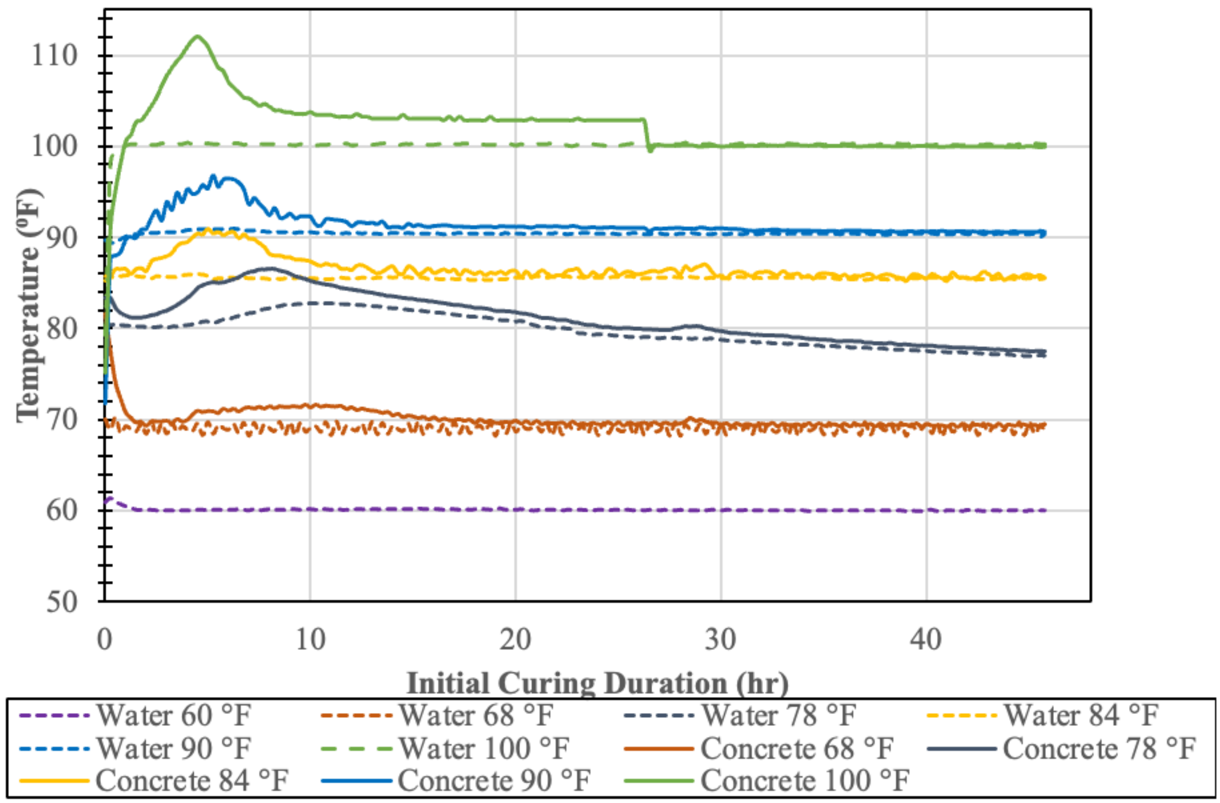


Figure A-8: 100% Type III PCC 24 hours initial curing temperatures plot



Note: 60 °F Concrete temperature probe malfunction

Figure A-9: 100% Type I PCC 48 hours initial curing temperatures plot

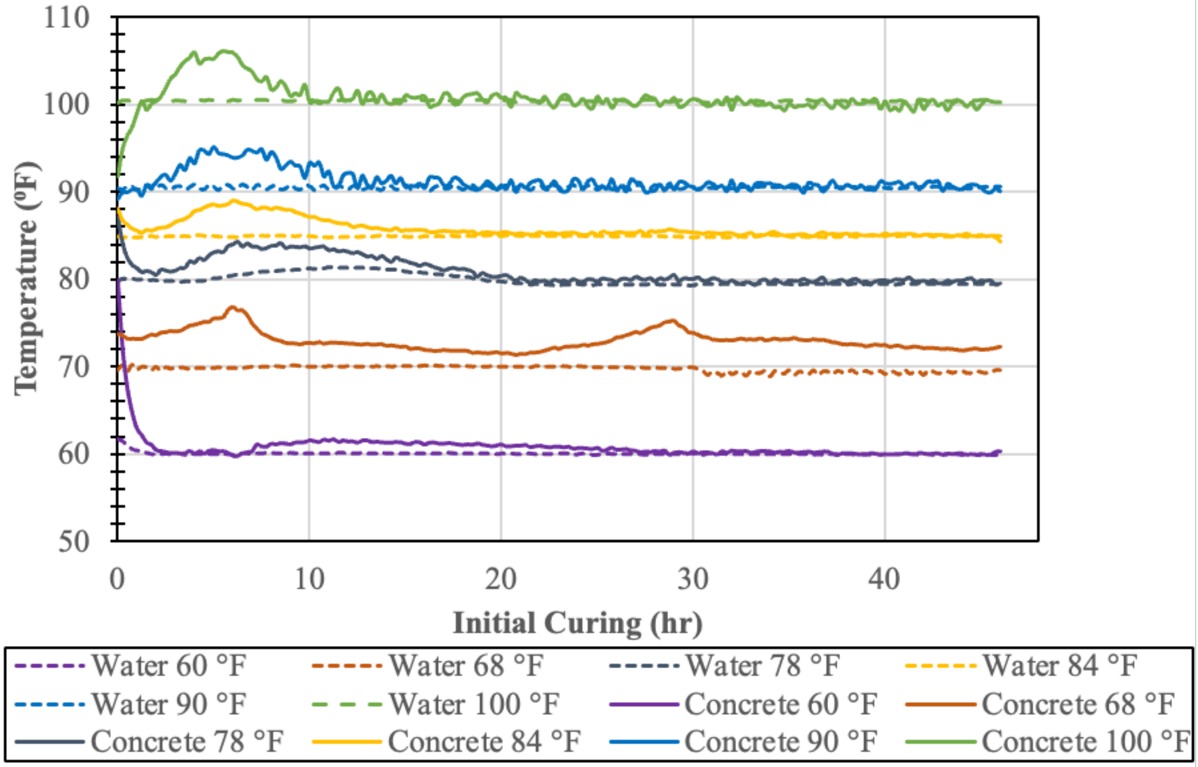


Figure A-10: 30% Class F Fly Ash 48 hours initial curing temperatures plot

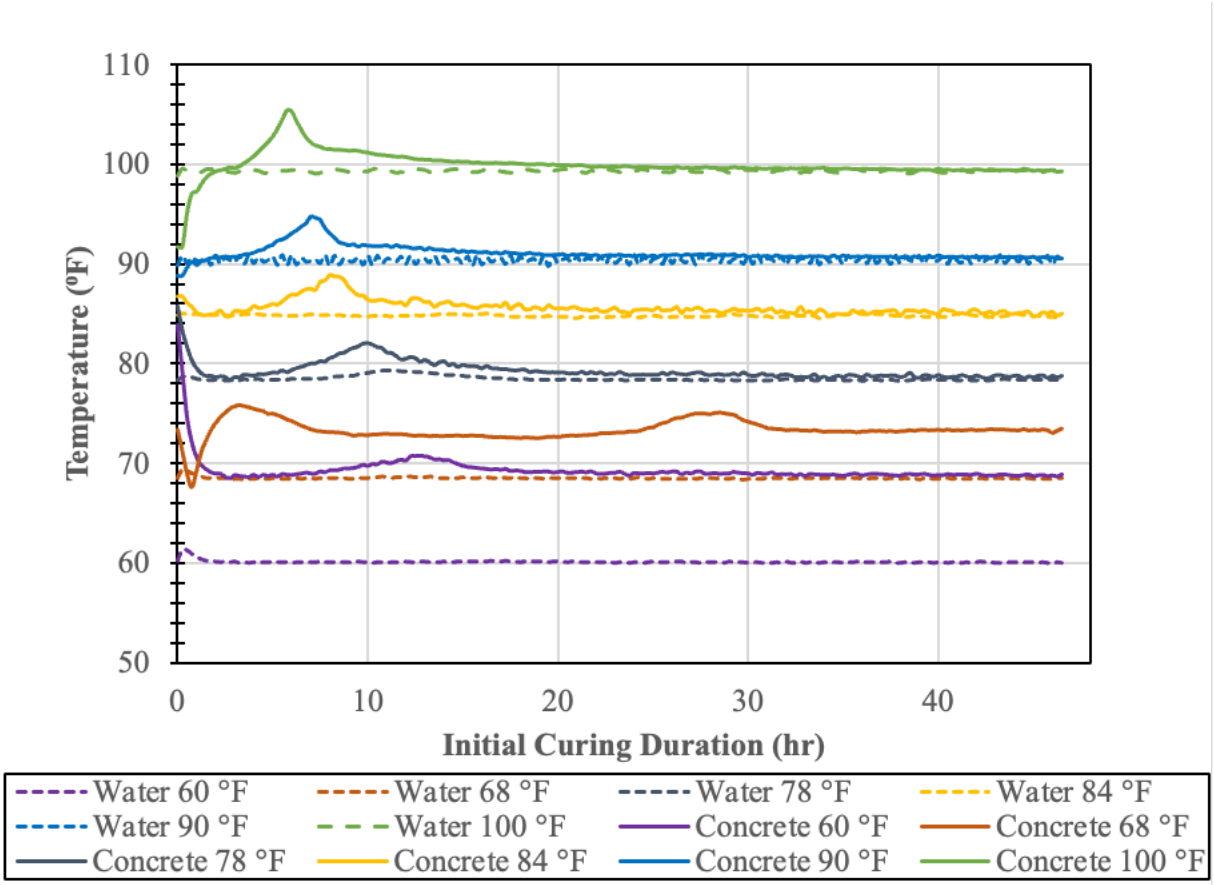


Figure A-11: 30% Class C Fly Ash 48 hours initial curing temperatures plot

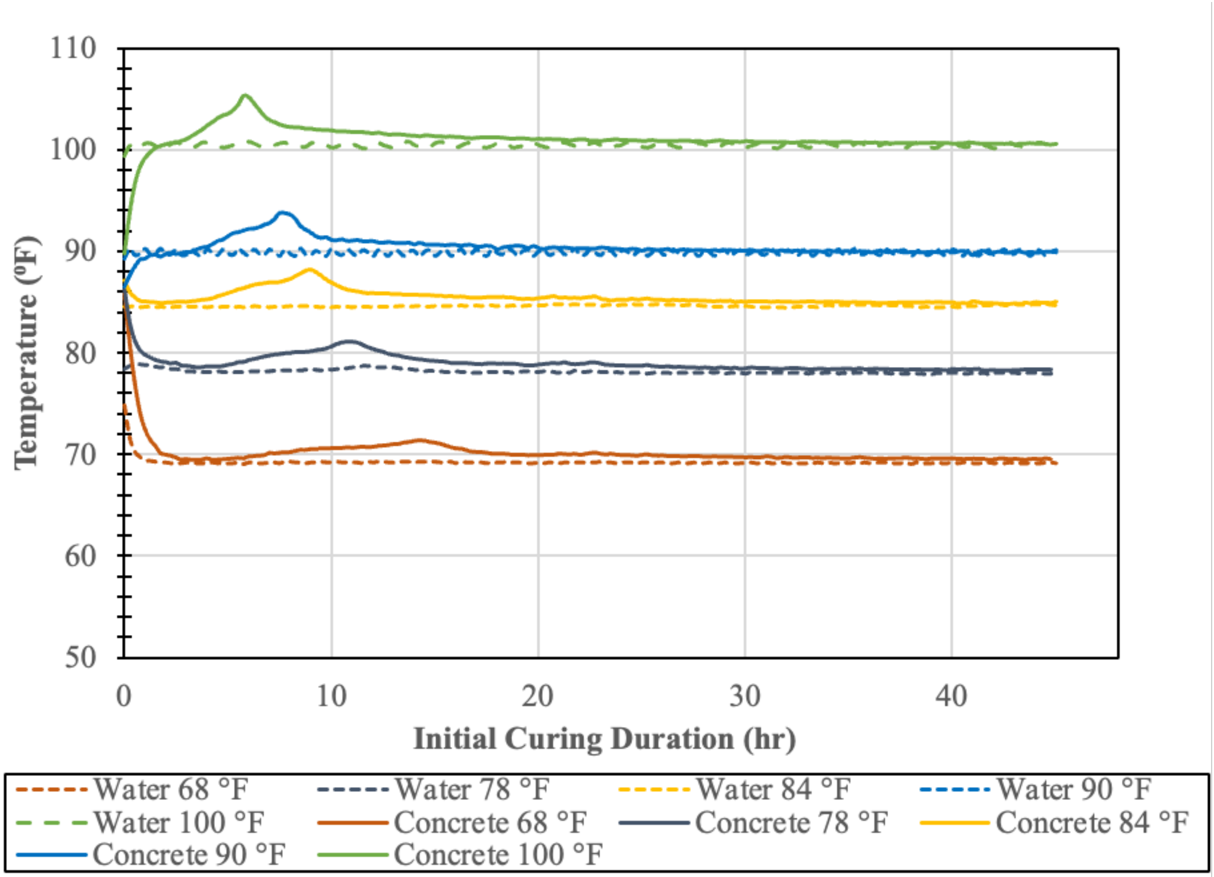


Figure A-12: 50% Slag Cement 48 hours initial curing temperatures plot

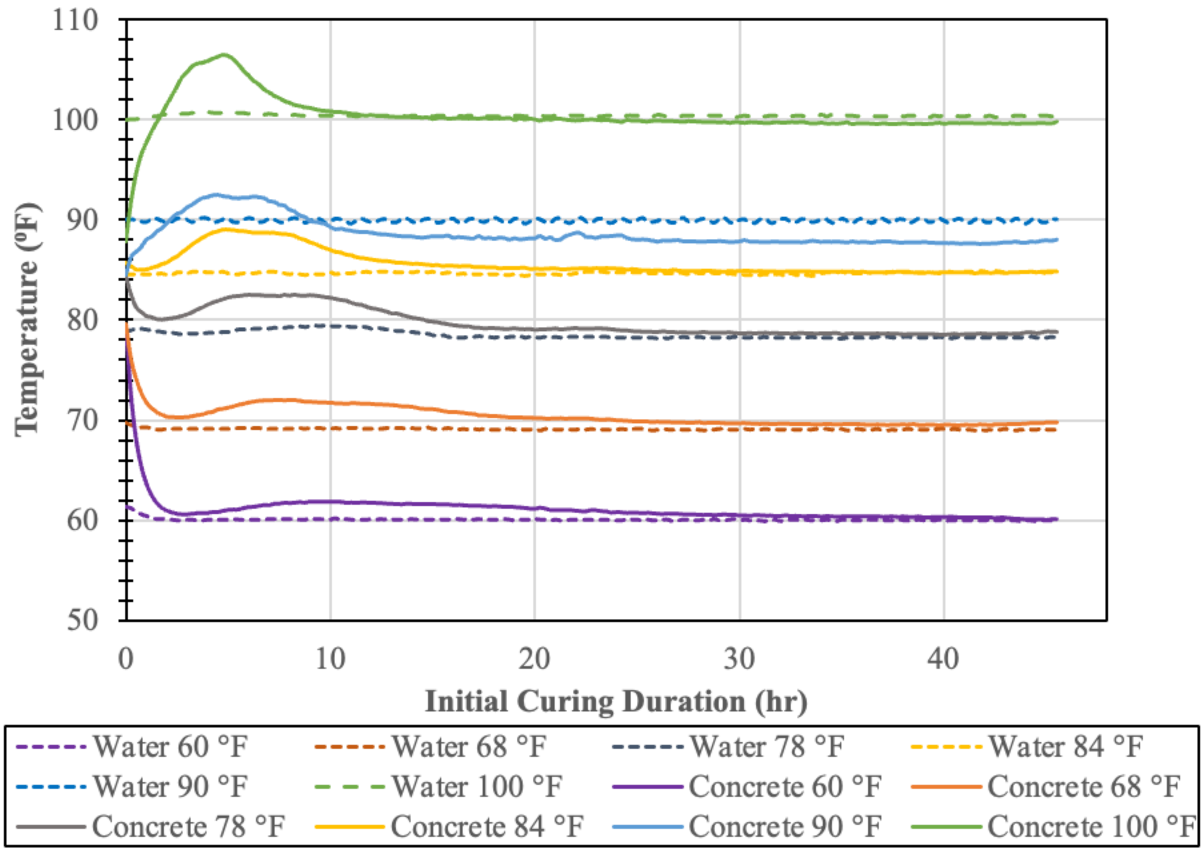


Figure A-13: 10% Silica Fume 48 hours initial curing temperatures plot

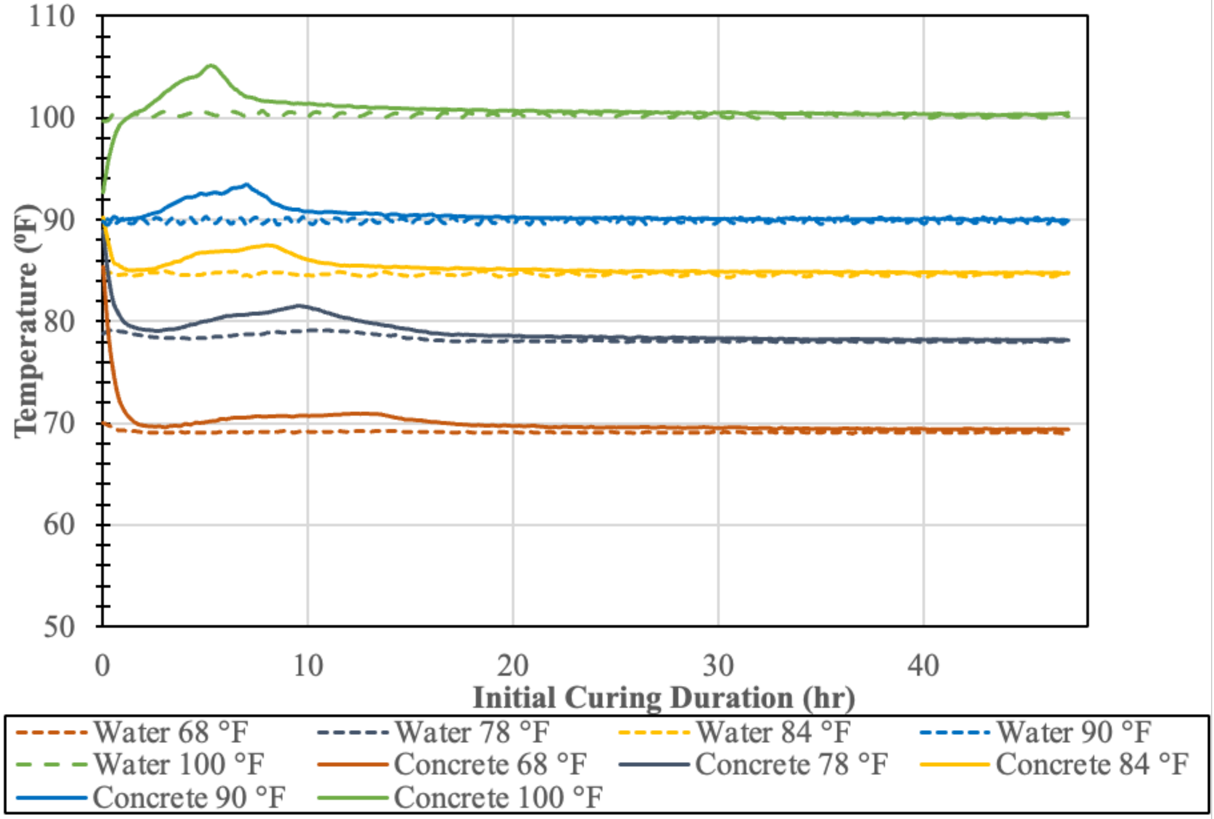


Figure A-14: 20% Class F Fly Ash with 30% Slag Cement 48 hours initial curing temperatures plot

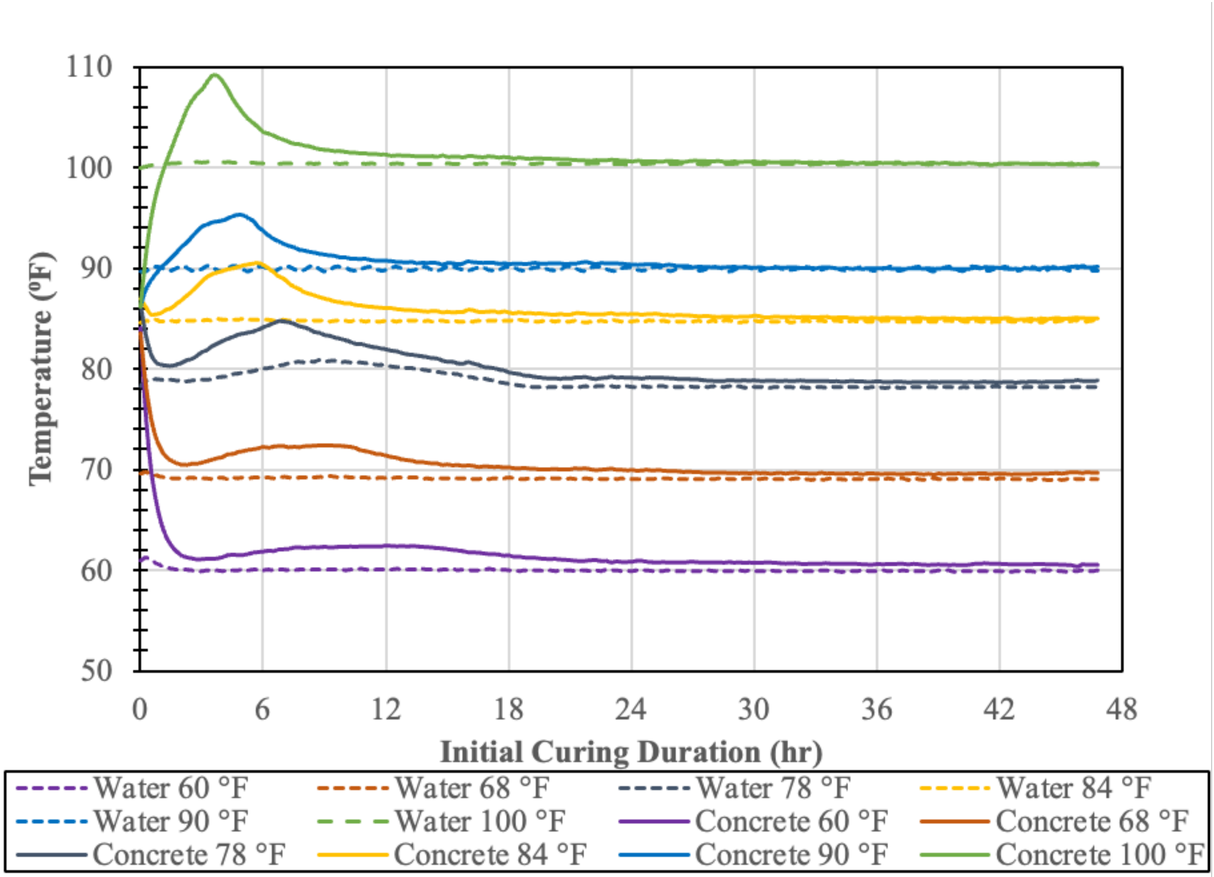


Figure A-15: 20% Class F Fly Ash with 10% Silica Fume 48 hours initial curing temperatures plot

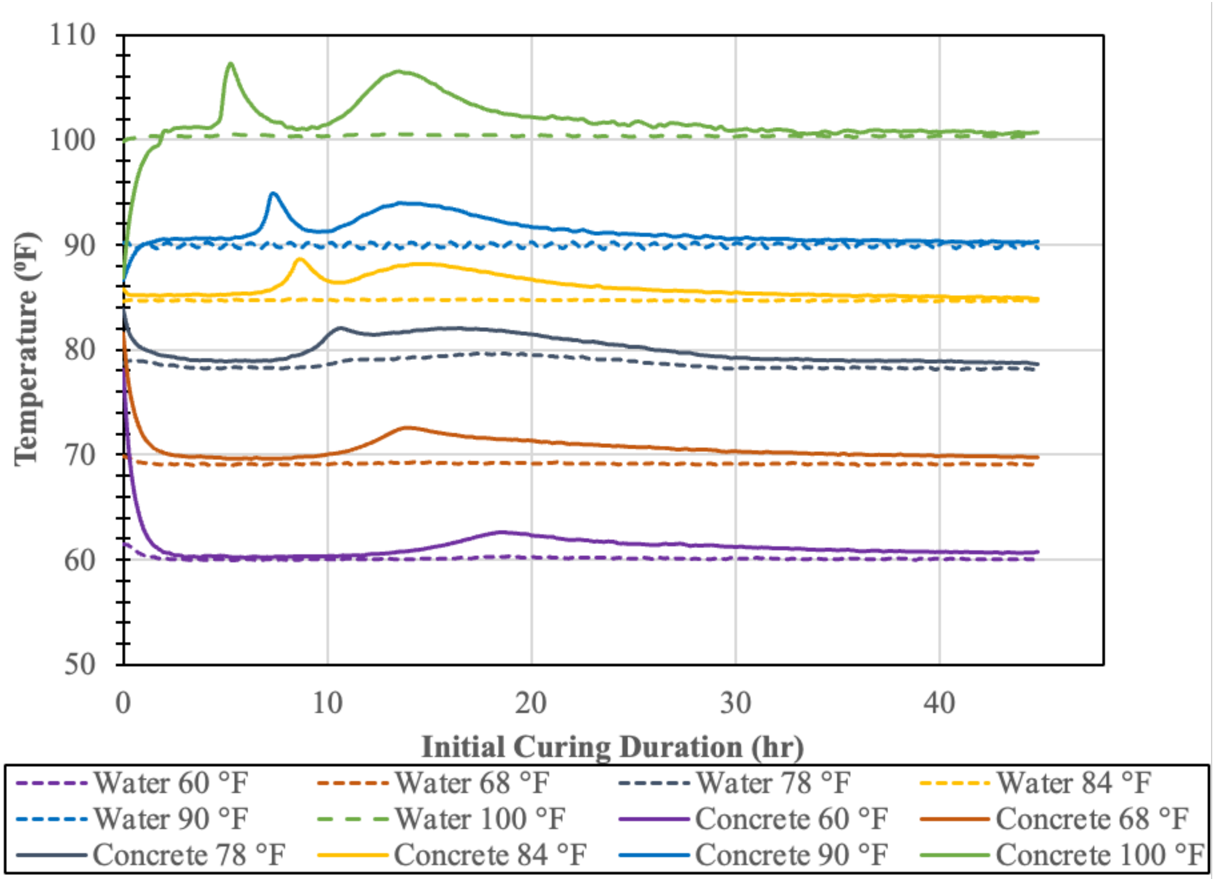


Figure A-16: 100% Type III 48 hours initial curing temperatures plot

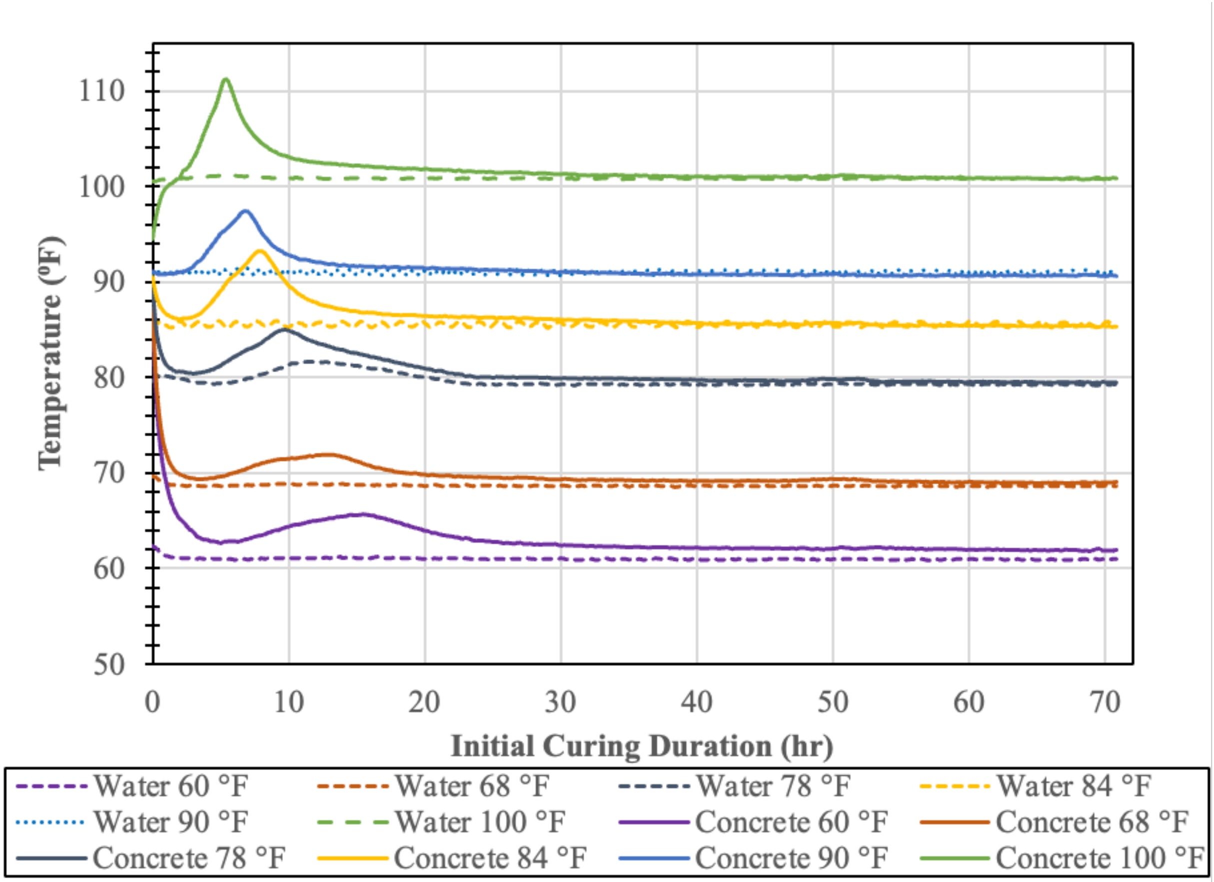


Figure A-17: 100% Type I PCC 72 hours initial curing temperatures plot

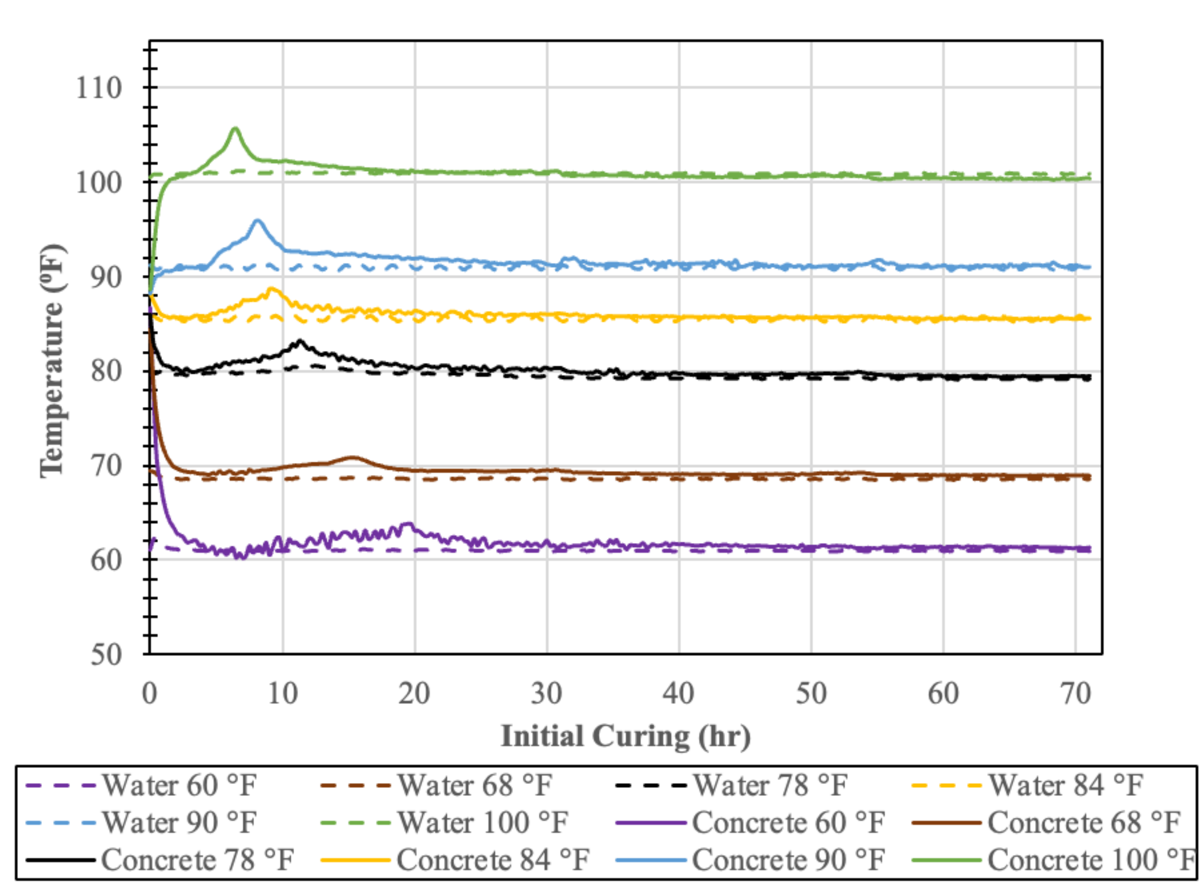


Figure A-18: 50% Slag Cement 72 hours initial curing temperatures plot

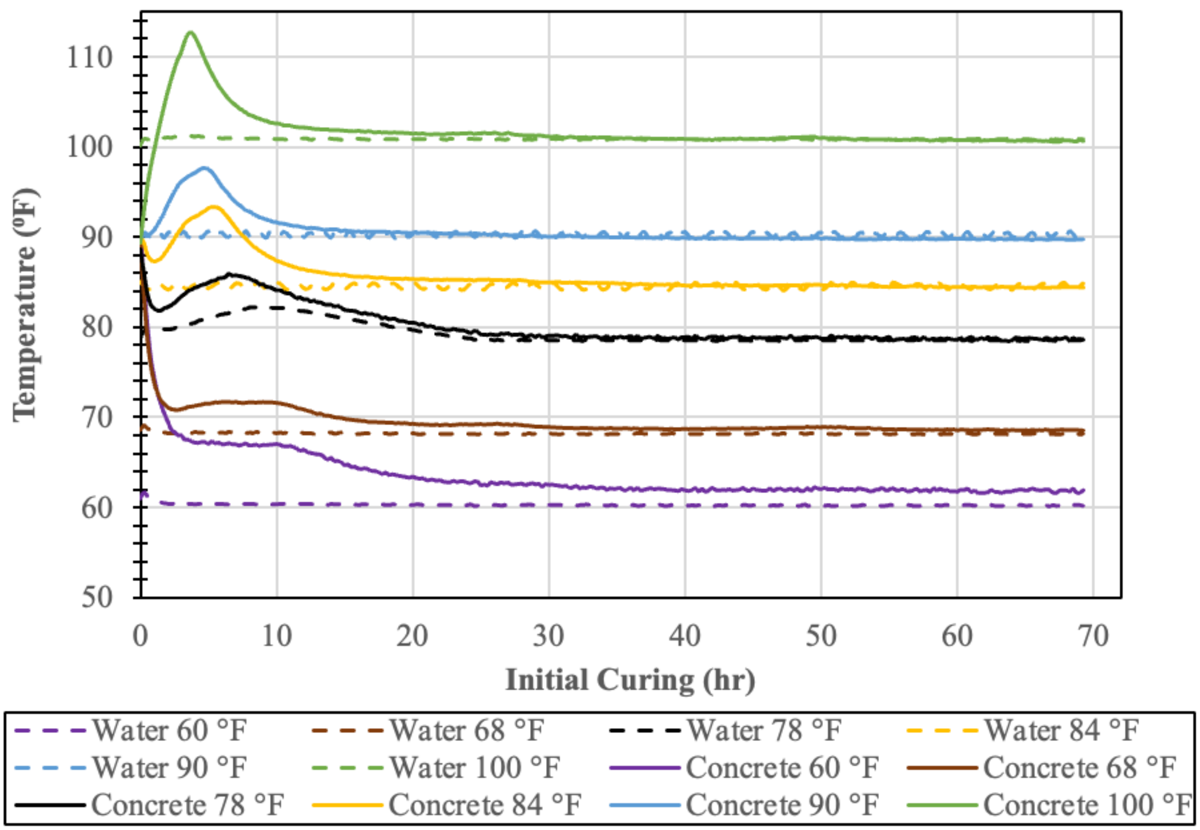


Figure A-19: 10% Silica Fume 72 hours initial curing temperatures plot

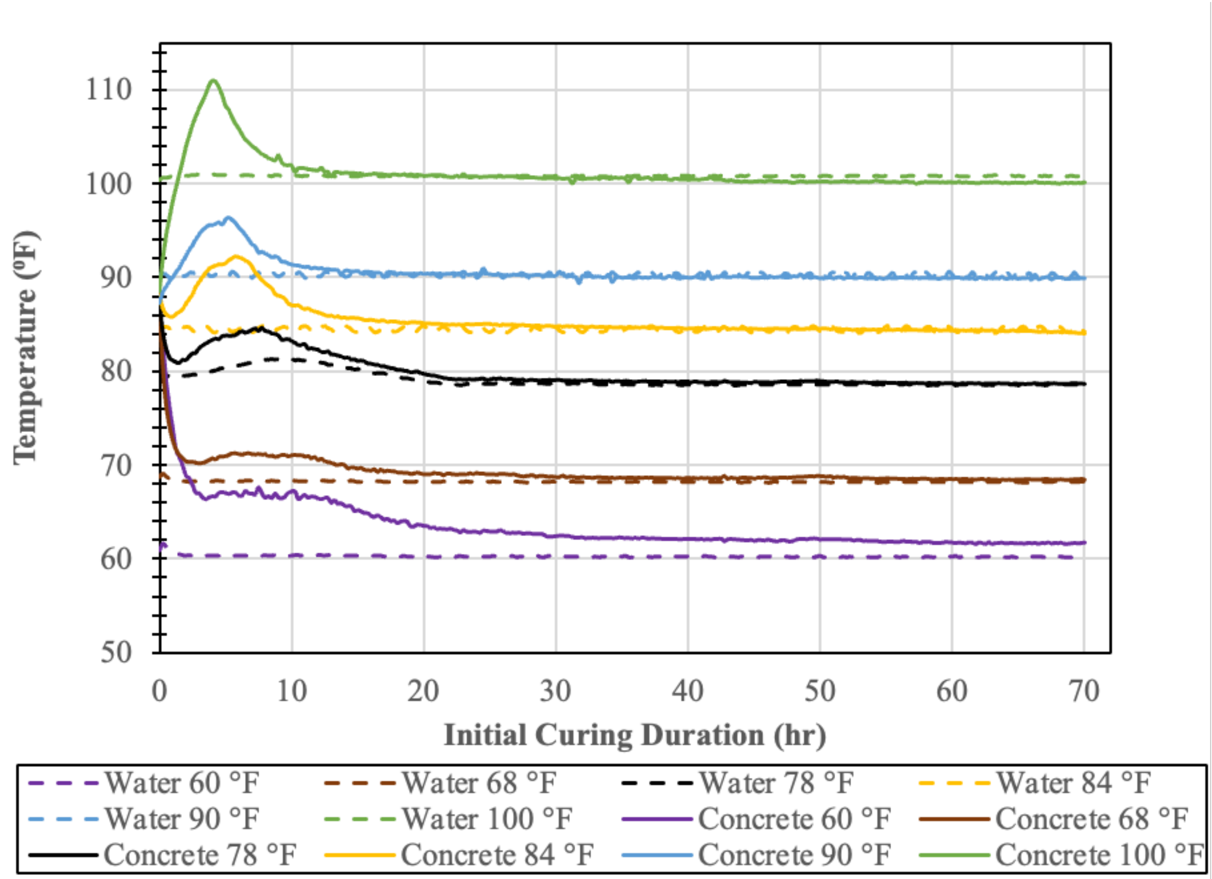


Figure A-20: 20% Class F Fly Ash with 10% Silica Fume 72 hours initial curing temperatures plot

Appendix B: Verification Batches

B.1 Compressive Strength Results of Verification Batches

Table B-1: Compressive strength for verification of 100% Type III PCC - 24 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 º F)
60 °F	7170	7110	2
	7070		
	7090		
68 °F	7140	7000	0
	6750		
	7100		
78 °F	6610	6650	-5
	6670		
	6670		
84 °F	6680	6580	-6
	6490		
	6570		
90 °F	6450	6470	-8
	6480		
	5810*		
100 °F	6150	5970	-15
	5820		
	5950		

*Outlier

Table B-2: Compressive strength for verification of 30% Class C Fly Ash Concrete - 24 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 º F)
60 °F	6910	6820	7
	6840		
	6720		
68 °F	6470	6370	0
	6250		
	6380		
78 °F	6370	6260	-2
	6110		
	6300		
84 °F	6350	6120	-4
	5860		
	6140		
90 °F	6030	6060	-5
	5970		
	6180		
100 °F	5300*	5640	-11
	5760		
	5510		

*Outlier

Table B-3: Compressive strength for verification of 50% Slag Cement Concrete - 24 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 ° F)
60 °F	6660	6520	3
	6330		
	6570		
68 °F	6370	6350	0
	6210		
	6480		
78 °F	5580*	6050	-5
	6130		
	5970		
84 °F	5660	5780	-9
	5910		
	5770		
90 °F	5610	5680	-11
	5610		
	5810		
100 °F	5330	5410	-15
	5590		
	5320		

*Outlier

Table B-4: Compressive strength for verification of 100% Type I PCC - 48 hours initial curing

Curing Location	28-Day Compressive Strength (psi)	Average Compressive Strength (psi)	Strength Difference (% From 68 º F)
60 °F	6630	6480	5
	6430		
	6370		
68 °F	6110	6200	0
	6320		
	6180		
78 °F	5790	5790	-7
	5630		
	5960		
84 °F	5720	5800	-6
	5910		
	5760		
90 °F	5850	5750	-7
	5520		
	5880		
100 °F	5570	5350	-14
	5290		
	5200		

*Outlier

B.2 Relative Strength Difference Plots of Verification Batches

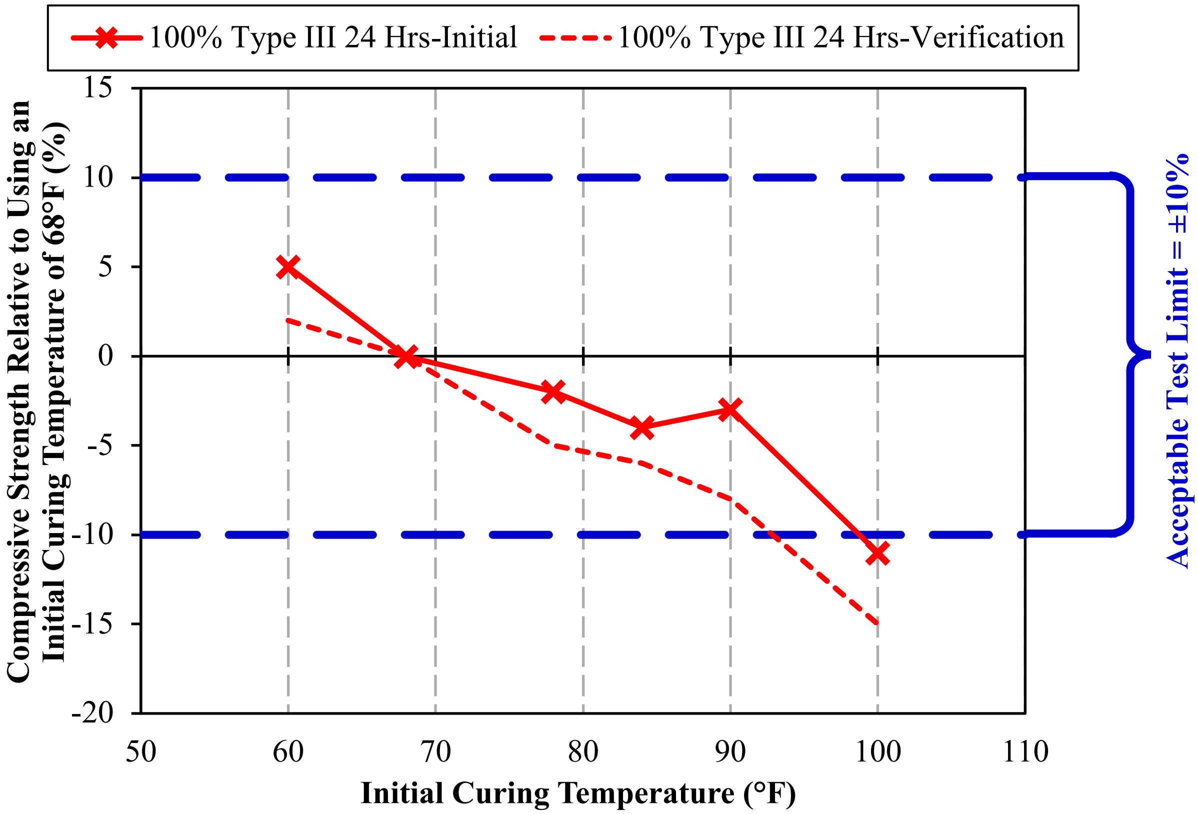


Figure B-1: Relative strength difference plot for verification of 100% Type III PCC - 24 hours initial curing

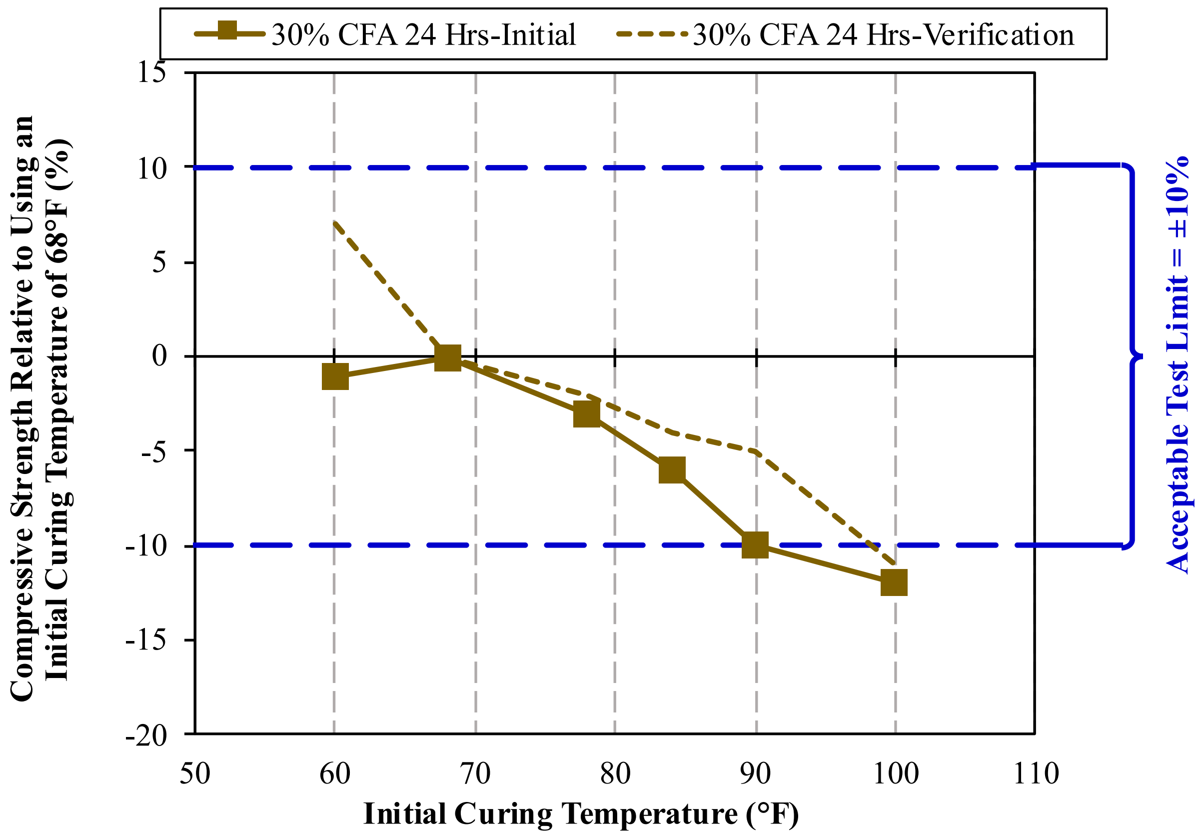


Figure B-2: Relative strength difference plot for verification of 30% Class F Fly Ash Concrete - 24 hours initial curing

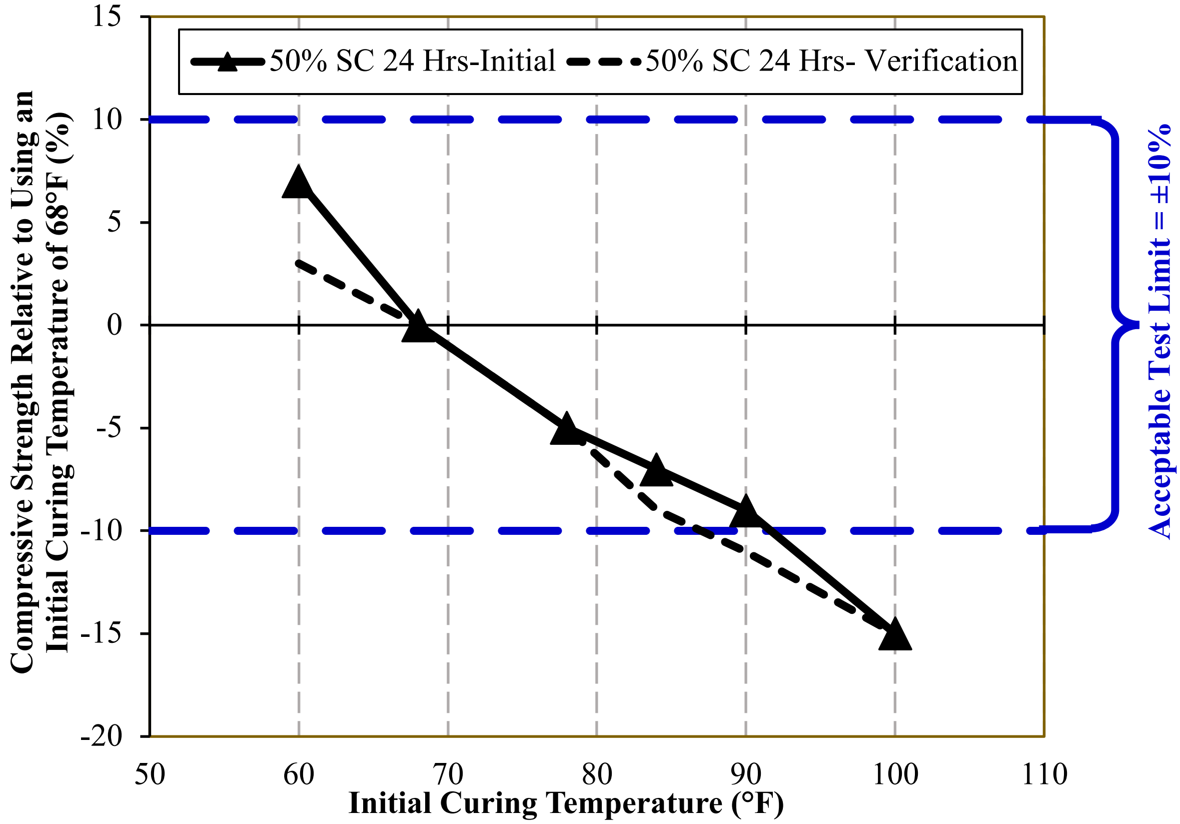


Figure B-3: Relative strength difference plot for verification of 50% Slag Cement Concrete - 24 hours initial curing

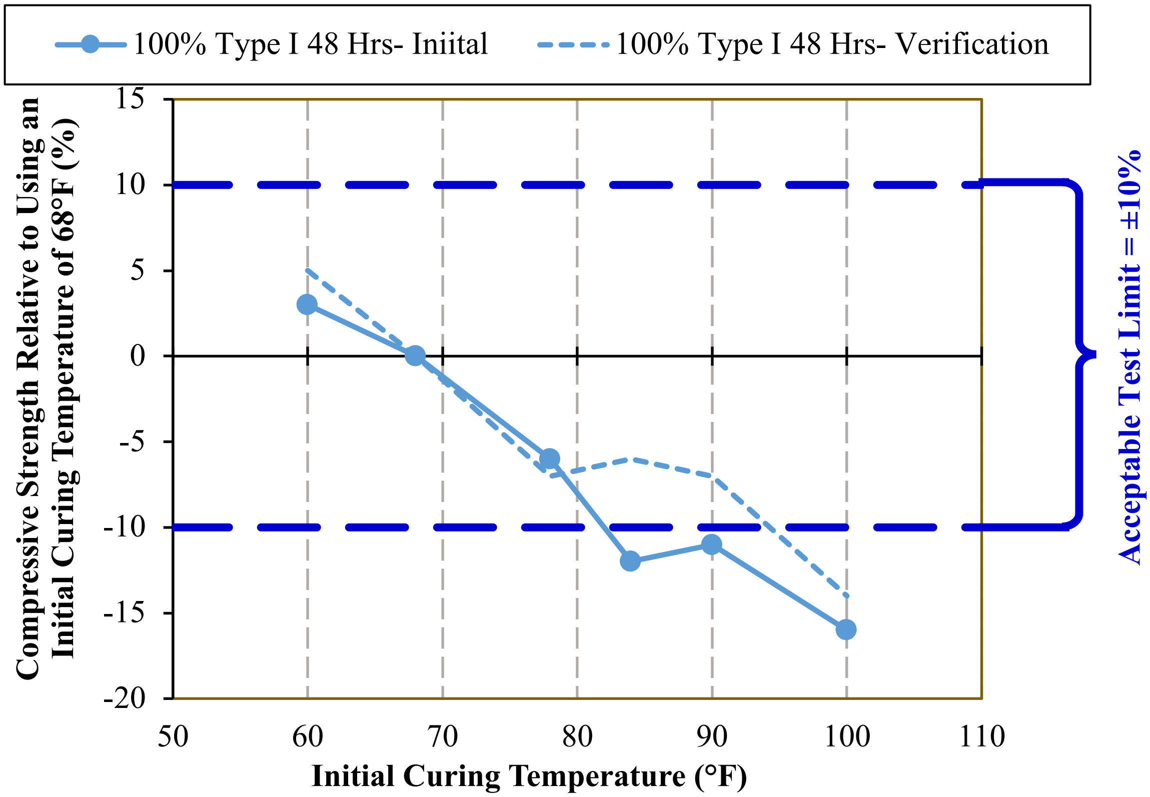


Figure B-4: Relative strength difference plot for verification of 100% Type I PCC - 48 hours initial curing

Appendix C: Batching Procedures

C.1 Mixing Procedure for Conventional-Slump Concrete

Mix the concrete in the laboratory in accordance with ASTM C 192 (2019):

1. "**Butter**" the mixer by using cement, sand, and water to produce a mortar with similar proportions as the concrete to coat the mixer.
2. Drain mortar from the mixer.
3. Add **all coarse and fine aggregates** (alternate buckets of coarse and fine aggregates to help with proper mixing.)
4. Add approximately **80% of water**.
5. Add all air-entraining admixture (**AEA**) while mixer is running.
   Mix the material thoroughly for 3 minutes.
6. Add **all cementitious material** with the mixer running.
7. Disperse **all admixtures in the remaining mixing water (20%)**, and add the solution to the mixer with the mixer running.
8. After all ingredients are added, mix for **3 minutes**.
9. Rest for **3 minutes**.
10. Mix for **2 minutes**.
11. **Sample** concrete to test fresh properties, if acceptable = **Done**.
12. If any additional water-reducing admixtures are needed to adjust consistency:
    mix for 1 minute, rest for 2 minutes, and mix for 1 minute. Then, re-sample and test fresh properties.

Notes:

1. Cover the open end of the mixer during mixing, the rest period, and when stationary to prevent evaporation.
2. A different mixing procedure is need when using **<u>silica fume</u>**.

C.2 Mixing Procedure for Concrete with Silica Fume

Mix the concrete in the laboratory in accordance with ASTM C 192 (2019):

1. "**Butter**" the mixer by using cement, sand, and water to produce a mortar with similar proportions as the concrete to coat the mixer.
2. Drain mortar from the mixer.
3. Add **all coarse aggregates**
4. Add approximately **80% of water**.
5. Add **silica fume** slowly into the revolving mixer
6. Mix for **3 minutes**.
7. Add **all fine aggregates**
8. Add all air-entraining admixture (**AEA**) while mixer is running.
   Mix the material thoroughly for 3 minutes.
9. Add **all cementitious material** with the mixer running.
10. Mix for **1.5 minutes**.
11. Disperse **all admixtures in the remaining mixing water (20%)** and add the solution to the mixer with the mixer running.
12. After all ingredients are added, mix for **3 minutes**.
13. Rest for **3 minutes**.
14. Mix for **2 minutes**.
15. **Sample** concrete to test fresh properties, if acceptable = **Done**.
16. If any additional water-reducing admixtures are needed to adjust
    a. consistency: mix for 3 minutes, rest for 3 minutes, and mix for 2 minutes.
    b. Then, re-sample and test fresh properties.

Notes:

1. Cover the open end of the mixer during mixing, the rest period, and when stationary to prevent evaporation.

2. When using **silica fume**, a significant amount of **high-range water-reducing admixture** is needed to obtain workable concrete.