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Research Report No. 2 for ALDOT Project 930-828

EVALUATION OF IN-PLACE CONCRETE STRENGTH BY CORE TESTING

Submitted to

The Alabama Department of Transportation

Prepared by

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It was found that the amount of restraint (or degree of microcracking) influenced the strength of the core and pullout test specimens, but not the molded cylinders. The expression of ACI 209.2R (2008) is recommended to account for the effect of concrete age on the strength gain of cores. The test data indicated that the core strength is on average 87 percent of the moist-cured cylinder strength. Therefore, in accordance with the approach of ACI 318, it is recommended that the in-place concrete be deemed structurally adequate if the average of three cores equals or exceeds 85 % of the specified design strength so long as no single core has a strength below 75 % of the specified design strength. If more than three cores are tested, then the procedure of Bartlett and Lawler (2011) should be implemented to determine the acceptable minimum strength of the lowest single core.

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When the average compressive strength of molded cylinder specimens do not meet the minimum required strength for a batch of concrete, the strength of cores are used to assess the in-place compressive strength of the structural member for acceptance and payment. The overall objective of the work contained in this report is to develop an ALDOT procedure to evaluate core strength results obtained under various conditions. Since there are many factors that influence the apparent strength of cores, strength correction factors must be applied to core strengths in order to convert them to a standard which can be compared with specified 28-day design strength. The following major factors that affect the relationship between the in-place strength and the strength of molded cylinders were assessed in this project: concrete age, concrete strength level, coarse aggregate type, degree of microcracking, and difference in strength gain due to using different cementitious materials in the concrete mixture. Eight full-scale slabs were cast from which cores were recovered and tested in compression at ages of 28, 42, 91, and 365 days. Testing was conducted near the edges of the slab as well as in the interior of the slab to determine if restraint had an impact on microcracking.

It was found that the amount of restraint (or degree of microcracking) influenced the strength of the core and pullout test specimens, but not the molded cylinders. The expression of ACI 209.2R (2008) is recommended to account for the effect of concrete age on the strength gain of cores. The test data indicated that the core strength is on average 87 percent of the moist-cured cylinder strength. Therefore, in accordance with the approach of ACI 318, it is recommended that the in-place concrete be deemed structurally adequate if the average of three cores equals or exceeds 85 % of the specified design strength so long as no single core has a strength below 75 % of the specified design strength. If more than three cores are tested, then the procedure of Bartlett and Lawler (2011) should be implemented to determine the acceptable minimum strength of the lowest single core.

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Chapter 1

INTRODUCTION

1.1 BACKGROUND

Molded cylinders are most often used as a measure of quality assurance and have long been the industry standard for determining the quality of the concrete delivered to the job site. If the compressive strength of the cylinders do not satisfy the project specified design strength requirements, then it is common practice to do in-place testing on the concrete in question. The Alabama Department of Transportation (ALDOT) uses the compressive strength of cores obtained as per AASHTO T 24 (2009) from a structural member with potentially substandard strength to assess the in-place concrete strength for acceptance and payment. If the average core strength is equal to or greater than 100% of the required 28-day compressive strength (f'_c), the in-place concrete is accepted with no price adjustment. If the average core strength is (a) greater than or equal to 85% of f'_c but less than 100% of f'_c and (b) the Engineer deems the concrete to be structurally acceptable, then the concrete is accepted with the price adjustment shown in Figure 1-1. The in-place concrete represented by low core strength results is rejected if the average core strength is less than 85% of f'_c.

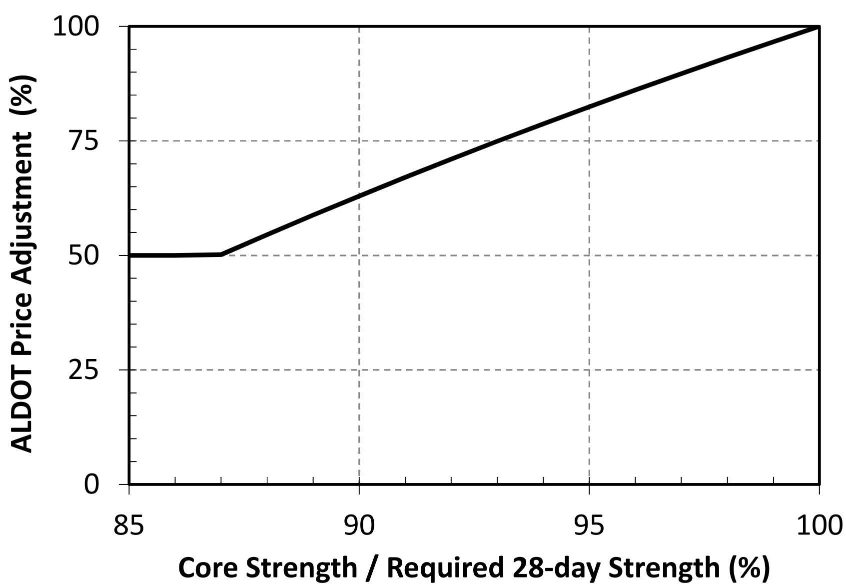

Figure 1-1: ALDOT Item 501.05 price adjustment

The ACI 318 Code (2014) allows the Engineer to accept in-place concrete if the average core strength is (a) equal to or greater than 85% of f'_c and (b) if no single core strength is less than 75% of f'_c.

There is thus a significant difference in the way core strength results are evaluated by ALDOT as compared to the ACI 318 Code. Note that the AASHTO LRFD Bridge Construction Specifications (2010) do not provide specific guidance on the assessment of core strength results relative to f'_c.

Core testing involves drilling a core from the concrete in question using an electric or gas powered core rig to cut out a cylindrical specimen. After conditioning its moisture state, the recovered specimen is then trimmed to the appropriate length, capped, and tested in compression. There are many variables to consider when assessing the core strength obtained after testing, and part of the challenge for an Engineer is how to account for the effect of these variables when comparing core strengths to the required specified concrete design strength.

Moisture conditioning has a significant effect on the apparent strength of drilled cores. In the past, general practice was to submerge the recovered cores in lime-saturated water for 40 hours prior to testing. In 2007 the post-recovery core conditioning method was revised in AASHTO T 24, to require that cores (a) be wiped off and placed in sealed plastic bags no later than 1 hour after drilling and (b) remain sealed for at least 5 days prior to testing. The intent of this conditioning method is to preserve the moisture of the recovered cores and to minimize the effect of moisture gradients introduced by wetting during drilling and specimen preparation. The presence of moisture gradients in the specimen has been shown to significantly reduce the apparent strength of the tested core (Bartlett and MacGregor 1994).

Because cores are extracted after low 28-day cylinder compression strengths are obtained, the core strength results are often obtained at a concrete age much later than 28 days. Since concrete gains strength with age, there may thus be a need to correct the core strength results back to an equivalent 28-day strength. There is also a difference in the availability of moisture for curing in-place concrete versus moist-cured specimens, and this will affect the concrete strength gain with age. It is thus important to provide guidance to ALDOT on how to account for the effect of concrete age on the in-place concrete strength relative to the specified 28-day design strength.

Concrete exposed to drying and thermal changes will develop internal microcracks. The degree of microcracking is significantly increased in concrete that is highly restrained against movement, which typically occurs in structural elements such as bridge decks. Standard-cured, molded cylinders are free to contract and are cured in a moist environment; therefore, they not contain a similar level of mircocracking when compared to in-place concrete. The presence of microcracks in in-place concrete is one of the primary reasons why core strength results may be less than the strength results obtained from standard-cured, molded cylinders.

Conflicting results have been reported regarding the effect of core orientation on strength (Carroll et al. 2016). Because of the development of the interfacial transition zone (ITZ) under mostly the larger aggregate particles (Mehta and Monteiro 2014), some researchers have found that the strength of cores drilled perpendicular to the direction of concrete casting is reduced relative to cores drilled parallel to the direction of concrete casting (Johnston 1973; Munday and Dhir 1984; Khoury et al. 2014). Note that because of how cylinders are cast, their orientation is similar to cores drilled parallel to the direction of concrete casting. There is thus uncertainty how to modify the core strength depending on how the core was extracted relative to the casting direction of the concrete.

The industry standard is to obtain cores and trim them to a length-to-diameter ratio of 2.0. If this is not possible, cores obtained from the in-place concrete are permitted to have a length-to-diameter ratio (L/D) less than 2.0 but not less than 1.0. Normal-strength specimens with a length-to-diameter ratio between 1.0 and 1.75 must have a correction factor applied to test results as specified in AASHTO T24 (2009). However, the L/D strength correction factors listed in AASHTO T 24 are only valid for concrete strengths from 2,000 to 6,000 psi. No guidance is given for when cores strength results exceed 6,000 psi. These strengths are often encountered by ALDOT when testing high-strength concrete girders or when testing cores at later ages. Guidance is needed regarding the selection of an appropriate L/D correction factor when cores strengths exceed 6,000 psi.

1.2 PROJECT OBJECTIVES

The overall objective of the work contained in this report is to develop an ALDOT procedure to evaluate core strength results obtained under various conditions. The procedure will account for the most significant variables that may affect the core strength and provide a reliable means to assess the in-place concrete strength. In addition, the following were the objectives for this project:

- Determine if there is a significant difference between core strength and molded cylinder strength and if there is, quantify this difference and provide recommendations to account for this difference,
- Determine the effect of concrete age on the strength of in-place concrete,
- Determine the effect of restraint, and thus microcracking, on concrete strength, and
- Provide guidance on how to identify outliers and how to account for the number of core strength results obtained.

1.3 RESEARCH APPROACH

The research approach consisted of two primary phases of experimental work. The first experimental phase consisted of ready-mixed concrete delivered to the Auburn University Structural Laboratory to evaluate the effects of length-to-diameter ratio on core strength. The second experimental phase was performed in the field to assess the effect of numerous variables on in-place concrete strength. This report covers the procedures and findings for the flied-testing phase of this work and provides overall implementation guidelines for this project.

Eight, 15 ft × 15 ft slab specimens with a 9½ in. depth were cast with ready-mixed concrete. In order to evaluate the effect that concrete strength level had on the relationship between in-place testing and required design strength, two high-strength concrete slabs were cast in addition to six normal-strength slabs for comparison. Three types of aggregate were used in the study: crushed limestone, uncrushed river gravel, and crushed granite. Slabs were also made with different SCM types: Class C fly ash, Class F fly ash, and slag cement. Cores, molded cylinders, and pullout specimens were tested at 28, 42, 91, and 365 days.

All experimental core drilling and testing conformed to current ALDOT practices. All the test equipment purchased matched what ALDOT used at the time the project started. Additionally, at the start of the project, a meeting was held at Auburn University with ALDOT representatives to demonstrate and ensure acceptable coring practice.

1.4 ORGANIZATION OF REPORT

A review of literature to gain more information on the effect that various variables may have on the in-place strength and core strength results is covered in Chapter 2. A summary of the practices various DOTs use to assess core strength results is also presented towards the end of Chapter 2. Chapter 3 contains the details of the experimental plan developed to help accomplish the project objectives. This includes the description of the experimental plan, proportions of concretes, raw material properties, test methods, testing schedule, and explanation of notation. The data gathered over the course of this project and the subsequent analysis of the data is covered in Chapter 4. This chapter also contains the results from the statistical analysis performed on the data collected. Chapter 5 contains all the background to the development of an ALDOT procedure to evaluate core strength results. General conclusions and recommendations are provided in Chapter 6, along with a summary of the work performed during this study.

Chapter 2

Literature Review

2.1 INTRODUCTION

For many years, engineers have tried to correlate test data from in-place strength testing to specified design strengths to determine if the strength of the in-place concrete is satisfactory. Different tests can produce different apparent strengths. This is due to both the variability of the in-place concrete as well as the test method and its variability. A challenge of interpreting in-place strength testing is being able to translate these values into meaningful information that can be compared with the specified design strength in order to determine the adequacy of the concrete for the given application. Most state departments of transportation also rely on these tests to accurately estimate the in-place concrete compressive strength and use these results to determine the payment of the contractor in cases where low strengths are obtained from molded cylinders.

2.2 FACTORS AFFECTING IN-PLACE STRENGTH

Many factors must be taken into account when conducting in-place strength testing. The manner in which testing is carried out will have a substantial effect on the apparent strength of the in-place concrete.

2.2.1 CONCRETE AGE

Age is one of the most prominent factors that affects the strength of concrete. Portland cement concrete gains strength as it hydrates. The primary gain in strength happens within the first 28 days after hydration. Mehta and Monteiro (2014) show that the rate of strength gain relative to age is dependent on many factors such as early-age temperature, cement type, and moisture conditions. Molded cylinders that are cured in accordance with AASHTO T23 (2009) are exposed to conditions that supply a constant supply of moisture to the specimens, ensuring that the hydration process continues until the specimens are tested. As stated by Price (1951), "where moisture is available for curing or where moisture contained in the concrete is not lost through drying, the strength development of the concrete will continue for a number of years." However, if moisture is not available to promote hydration, then the strength development of the concrete will be less when compare to concrete that is exposed to moisture.

Some correction factors have been developed that can take a strength obtained at any age and convert it to a 28-day strength. Yazdani and McKinnie (2004) conducted research for the Florida Department of Transportation (FDOT) in order to obtain strength correction factors based on concrete age using molded 6×12 in. cylinders that were made in accordance with ASTM C192-02 specifications.

The 6×12 in. cylinders were cured by placing them in water tanks that were maintained at 73±3°F. The concretes, which were tested in this study that contained SCMs, were made with 20% Class F fly ash and 50% Grade 100 slag cement. Yazdani and McKinnie (2004) found that the strength relationship varied depending on the type of cementing materials used in the concrete. The relationship between 28-day core strength and average core strength at a specific age as proposed by Yazdani and McKinnie (2004) can be seen in Equation 2-1. The strength conversion equations from Yazdani and McKinnie (2004) can be seen in Equation 2-2 through Equation 2-10. Note that different equations were developed for different cement types as well as different SCM types.

$$f'_c(28) = \frac{f_{core} \times 100}{F} \qquad \text{Equation 2-1}$$

Where F is defined as:

$$F = 4.4 + 39.1(lnx) - 3.1(lnx)^2 \quad \text{(Type I Cement)} \qquad \text{Equation 2-2}$$

$$F = -17.8 + 46.3(lnx) - 3.3(lnx)^2 \quad \text{(Type II Cement)} \qquad \text{Equation 2-3}$$

$$F = 48.5 + 19.4(lnx) - 1.4(lnx)^2 \quad \text{(Type III Cement)} \qquad \text{Equation 2-4}$$

Where: x = number of days since the concrete was placed

ln = natural log

Concretes with fly ash:

Cement Type I: $$f'c(28) = 0.490 * Exp(\frac{8.31}{t})^{0.276} \times f'c(t) \qquad \text{Equation 2-5}$$

Cement Type II: $$f'c(28) = 0.730 * Exp\left(\frac{2.89}{t}\right)^{0.514} \times f'c(t) \qquad \text{Equation 2-6}$$

Cement Type III: $$f'c(28) = 0.483 * Exp\left(\frac{5.38}{t}\right)^{0.191} \times f'c(t) \qquad \text{Equation 2-7}$$

Concrete with slag cement:

Cement Type I: $$f'c(28) = 0.794 * Exp\left(\frac{7.06}{t}\right)^{1.06} \times f'c(t) \qquad \text{Equation 2-8}$$

Cement Type II: $$f'c(28) = 0.730 * Exp\left(\frac{6.02}{t}\right)^{0.747} \times f'c(t) \qquad \text{Equation 2-9}$$

Cement Type III: $$f'c(28) = 0.826 * Exp\left(\frac{2.36}{t}\right)^{0.672} \times f'c(t) \qquad \text{Equation 2-10}$$

ACI 209.2R (2008) outlines a procedure for correcting compressive strength at any age back to an equivalent 28-day strength. This is done by using Equation 2-11. The value of *a*/*β* is defined as the time it takes for the concrete to reach half of its ultimate strength. Values for these constants can vary from 0.05 to 9.25 for *a* and 0.67 to 0.98 for *β*. The recommended values of the empirical constants for Equation 2-11 can be seen in Table 2-1.

$$f'_c(t) = f'_c(28) \times (\frac{t}{a+\beta\times t}) \qquad \text{Equation 2-11}$$

Where:

t = time since casting (days)

a = empirical constant from Table 2-1 (days)

β = empirical constant from Table 2-1 (unitless)

Table 2-1: Constants for ACI 209 Age Correction Equation

Cement Type	ACI 209 Empirical Constants for Equation 4-1	
	***a* (days)**	***β***
Type I	4	0.85
Type III	2.3	0.92

Bartlett and MacGregor (1996) also did a statistical analysis of a number of data points collected over a number of years in Alberta, Canada. Their analysis showed that on average, in-place strength increased approximately 25% from 28 days to one year.

2.2.2 SUPPLEMENTARY CEMENTING MATERIALS

Supplementary cementing materials (SCMs) are commonly used in today's concrete industry. They typically help decrease the overall cost of the mixture by reducing the amount of portland cement that is needed. SCMs are typically by-products of other industries and if they were not used in the concrete industry, they would be landfilled. Therefore, the use of these materials also provides a more sustainable option. The use of these materials can in some cases improve the fresh and hardened properties of the concrete.

2.2.2.1 FLY ASH

Mehta and Monteiro (2014) show that the partial replacement of portland cement with fly ash can greatly improve both the fresh and hardened properties of concrete. Fly ash is typically produced from the burning of coal in electrical power plants. Joshi and Lohtia (1997) state that fly ash is comprised of fine, spherical, glassy particles which are collected in dust collection systems located within fossil fuel power plants. Fly ash particles are oftentimes finer than portland cement particles. Bijen (1996) concluded that the pore size distribution in concretes that contain fly ash is also substantially finer than concretes containing only portland cement.

There are two main classes of fly ash used in the United States: Class C fly ash and Class F fly ash. Class C fly ash has a higher calcium oxide content and therefore has both cementing and pozzolanic characteristics. Class F fly ash typically contains lower amounts of calcium oxide and therefore acts primarily as a pozzolan. Naik, et al. (2003) noted that the rate of early-strength gain in concretes containing Class C fly ash was higher than concretes containing Class F fly ash, which is mainly due to the greater reactivity of Class C ash.

In general, Xu (1997) says that concrete containing fly ash typically has a lower 28-day strength but higher long-term strength as compared to concretes using portland cement as the only cementitious material. Naik et al. (2003) also concluded that when *moist cured*, “The long-term pozzolanic strength contribution of Class F fly ash was somewhat greater compared to Class C fly ash. Consequently, long-term compressive strengths of Class F fly ash concrete mixtures were better than that for Class C fly ash concrete mixtures.”

2.2.2.2 Slag Cement

Slag cement, also called ground-granulated blast-furnace slag, can also be used as partial replacement of portland cement when batching concrete. Mehta and Monteiro (2014) conclude that one significant advantage of using slag cement is that it decreases the amount of heat generated when concrete is batched. This characteristic is ideal when placing mass concrete. Oner and Akyuz (2007) concluded that the use of slag increased compressive strengths up to an optimal replacement percentage, which was determined to be 55-59%. From their experimental program, Oner and Akyuz (2007) also concluded that the use of slag in concrete increases workability but reduces the early-age strength of the concrete, but with proper curing, the strength increase was greater in concretes which contained slag cement because the pozzolanic reaction which converts calcium hydroxide into calcium-silicate-hydrate occurs slowly.

2.2.3 Moisture and Curing Conditions

The amount of moisture available to the concrete during the curing process has a significant effect on concrete strength and durability. When concrete is supplied with adequate moisture during curing, it allows the cement to hydrate continuously which produces higher strengths. Results from a study done by Popovics (1986) on molded cylinders can be seen in Figure 2-1. From this figure, it can be seen that the availability of moisture has a pronounced impact on the strength development of the concrete. In addition, it can be seen that continuously moist-cured cylinders perform the best in terms of long-term strength gain. It should also be noticed that when the cylinders were taken out of their moist-cured state and were kept in laboratory air, the overall strength of the cylinders decreased over this period. The availability of moisture for curing in-place concrete is thus one of the most important reasons why the strength of laboratory-cured, molded cylinder may be greater than the in-place strength.

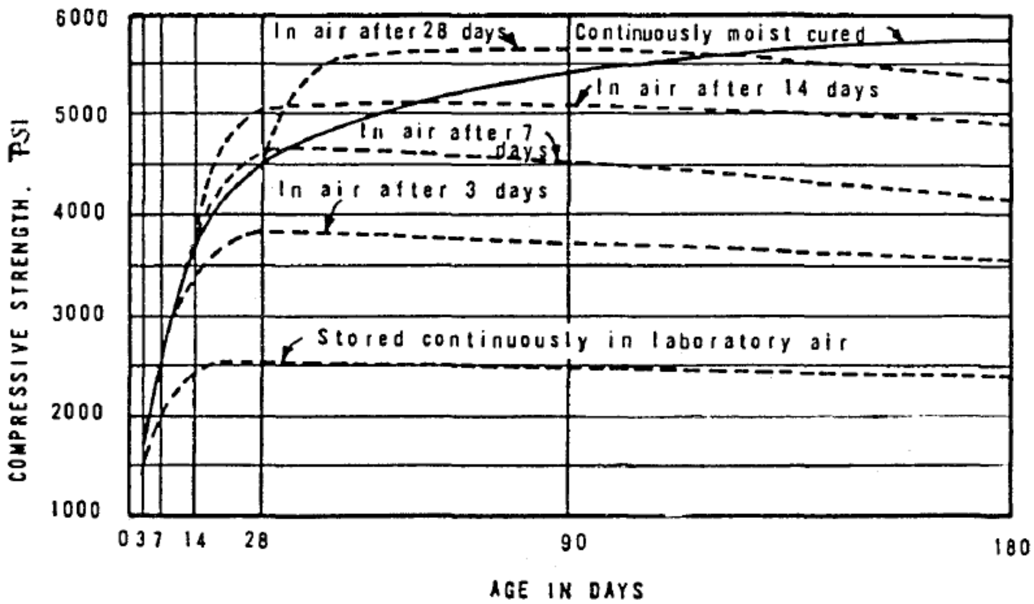

Figure 2-1: Compressive Strength vs. Age for Different Curing Conditions (Popovics 1986)

Bartlett and MacGregor (1994a) did a study on the effect of moisture conditioning of cores after drilling. Their results from their study can be seen in the regression plot in Figure 2-2. From this figure Bartlett and MacGregor (1994a) concluded that the compressive strength of a core specimen is decreased when its moisture content is uniformly increased throughout its volume after it has been cored and, conversely, that the compressive strength of a core specimen is increased when its moisture content is uniformly decreased throughout its volume. Bartlett and MacGregor (1994a) explain by stating that when cores are soaked in water, the surface of the core swells. This swelling at the surface is restrained by the interior of the core, which does not experience any moisture gain. This in turn causes residual stresses to form and lowers the overall compressive strength of the core. Conversely, when cores are left to dry, this causes shrinkage to occur on the surface of the core which causes its overall compressive strength to increase. Because of this, Bartlett and MacGregor (1994a) concluded that the most accurate estimate of in-place strength is obtained from a core specimen that contains no moisture gradient. In an attempt to eliminate a moisture gradient as much as possible, AASHTO T 24 (2009) recommends that the surface moisture of cores be wiped off and left to dry until all surface moisture has evaporated but no longer than one hour. After this, cores should be sealed in plastic bags to avoid moisture loss and therefore not create a moisture gradient within the core.

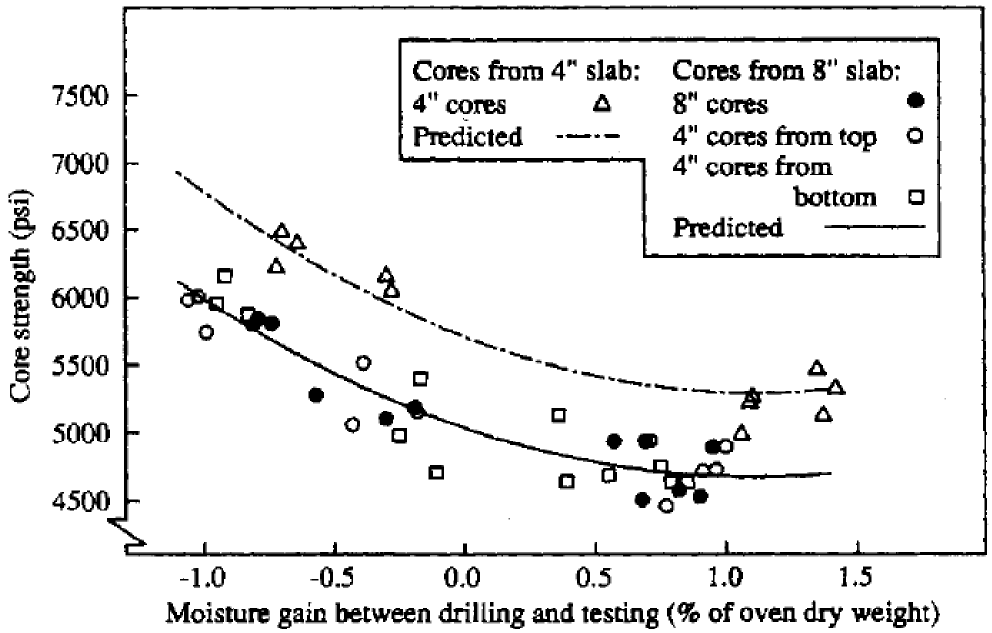

Figure 2-2: Regression Plot of Core Strength versus Moisture Gain (Bartlett and MacGregor 1994a)

2.2.4 COARSE AGGREGATE SIZE AND TYPE

Aggregate type can play an important role when assessing the strength of in-place concrete. One of the biggest reasons why aggregate size and type affect the compressive strength of a concrete specimen is because of the relationship between coarse aggregate and the interfacial transition zone (ITZ). Ollivier, Maso, and Bourdette (1995) describe the ITZ as a water-cement ratio gradient that develops around coarse aggregate which results in a different microstructure of the hydrated cement paste, which surrounds the coarse aggregate. Mehta and Monteiro (2014) explain that this happens because a film of water forms around the coarse aggregate particles, which in turn causes an increase in the water-cement ratio around the aggregate. Due to the increased amount of water gathers around the surface of the aggregate, the ettringite and calcium hydroxide particles that form are larger and therefore form a layer around the aggregate, which is weaker and more permeable. Mehta and Monteiro (2014) also conclude that the larger the coarse aggregate size, the higher the water-cement ratio in the ITZ will be, leading to a weaker and more permeable concrete. Arioz et al. (2007) showed that as the maximum aggregate size increased for cores with small diameters, the strength of the core decreased, but also noted that as core diameter increased, this effect was lessened. This means that larger specimens are impacted less by the size of the aggregate contained within them.

2.2.5 Temperature Conditions

Temperature conditions have a significant impact on the apparent strength of concrete. Mehta and Monteiro (2014) say that hot weather concreting increases slump loss, increases plastic-shrinkage cracking, and decreases the set time of freshly placed concrete. They continue to state that concrete placed under hot weather conditions has very rapid strength gain and will have greater 28-day strengths but lower long-term strengths than concrete cast at room temperature. The effect of temperature on early-age strength can be accounted for by using the maturity method.

2.3 Strength Test Methods

In the concrete industry, there have been many methods devised to evaluate the in-place compressive strength of concrete. It is very important that these testing methods produce reliable and accurate results. In this section, the following test methods are discussed: molded cylinders, cores, cast-in-place cylinders, and pullout tests.

2.3.1 Molded Cylinders

AASHTO T23 (2009), outlines the proper way to produce molded cylinders on site for compression testing. For many years, molded cylinders have been the industry standard for measuring concrete strength. Since molded cylinders are cured under controlled moisture and temperature conditions, their strength results do not reflect the in-place strength of the concrete element. Though not a good indicator of in-place strength, molded cylinders are used to measure the consistency and quality of the concrete batch that was delivered to the site. AASHTO T23 (2009) states that "the results of this test method are used as a basis for quality control of concrete proportioning, mixing, and placing operation; determination of compliance with specifications; control for evaluating effectiveness of admixtures; and similar uses."

2.3.1.1 Strength Acceptance of Molded Cylinders

Molded cylinders are often used as a method of quality control. In order for concrete to be accepted, ACI 318 (2014) states that the following requirements must be met:

1. The average of three consecutive tests ≥ f'_c
2. For f'_c ≤ 5000 psi: No result more than 500 psi below f'_c
   For f'_c ≥ 5000 psi: No result more than 0.1 x f'_c below f'_c

2.3.1.2 VARIABILITY

AASHTO T22 (2009) gives expected coefficients of variation for molded cylinders which can be seen in Table 2-2. These coefficients of variation are for cylinders made under both laboratory and field conditions and tested at the same age by the same laboratory. These coefficients are valid for 6×12 in. cylinders with compressive strengths between 2,000 and 8,000 psi and 4×8 in. cylinders with compressive strengths between 2,500 and 4,700 psi.

Table 2-2: Coefficients of Variation for Molded Cylinders (AASHTO T22 2009)

Specimen Type	Coefficient of Variation	Acceptable Range of Individual Cylinder Strengths	
		2 Cylinders	3 Cylinders
6 × 12 in. Cylinder - Laboratory Conditions	2.4 %	6.6 %	7.8 %
6 × 12 in. Cylinder - Field Conditions	2.9 %	8.0 %	9.5 %
4 × 8 in. Cylinder - Laboratory Conditions	3.2 %	9.0 %	10.6 %

When concrete is cast, two 6×12 in. cylinders are often made for quality assurance which therefore implies that the range of the compressive strengths of these cylinders under the conditions expressed by AASHTO T22 (2009) should not exceed 8.0 %.

2.3.2 CORES

When moist-cured, molded quality assurance cylinders are tested in compression and the resulting strength does not exceed the compressive design strength (f'_c) set forth by the design engineer, then in-place strength testing must be done on the in-place concrete to determine if it has adequate strength. Neville (2001) states that when concrete cylinders break low, it can be caused by a number of reasons including inadequate strength, poor consolidation, incurring damage during transit, freeze-thaw damage, improper curing, and improper testing methods. When quality assurance cylinders have an average strength below the specified compressive design strength (f'_c), core testing is most often used to assess the strength of the in-place concrete.

2.3.2.1 SUMMARY OF AASHTO T24: STANDARD METHOD OF TEST FOR OBTAINING AND TESTING DRILLED CORES AND SAWED BEAMS OF CONCRETE

When taking cores, care must be taken to ensure that as little damage as possible is inflicted upon the core, both while drilling and while transporting the core back to the laboratory for testing. When drilling, it is important that the core rig is securely fastened to the surface from which the core is being taken to ensure that there is as little core barrel wobble as possible. In addition, specimens should be secured during transportation so that they do not incur any damage to impact. AASHTO T24 (2009) specifies that a minimum core diameter of 3.75 in. should be used when retrieving cores to evaluate in-place strength. After cores have been drilled, the excess surface water should be wiped off and the surface of the core should be allowed to dry, but should not be exposed longer than one hour after drilling. Cores should then be placed in sealed plastic bags to prevent moisture loss. Cores should be kept in the sealed plastic bags at all times except during trimming and end preparation. In order to be tested, the ends of the core must not have any projections greater than 0.2 in. above the end surfaces and shall not depart from perpendicularity to the longitudinal axis by a slope of more than 1:8*d* where *d* is the average core diameter in inches. If water is used during the trimming or grinding of the core ends during trimming, these operations should be done as soon as possible after the core has been removed from the in-place concrete. After the end preparation has been completed, the core should be wiped of all excess water and allowed to let all surface water evaporate, but not be exposed for more than one hour.

AASHTO T24 (2009) also states that the length-to-diameter ratio of the obtained core should be between 1.9 and 2.1. If the length-to-diameter ratio is greater than 2.1, it must be trimmed in order to meet the specification. If a core has a length-to-diameter ratio less than 1.75, correction factors must be applied to correct its apparent strength. In addition, a core's height must be at least 95% of its diameter before capping and at least greater than or equal to its diameter after capping. Once cores are exposed to wetting due to drilling or trimming, they must be bagged in sealed plastic bags for at least five days to ensure that no moisture gradients are present in the core specimen. Once the five-day period has passed, the ends of the core must be either trimmed or ground to the required planeness or be capped in accordance with AASHTO T231 (2009). If the trimming or grinding involves exposure to moisture, this process should occur before this five-day period.

The initial length of the drilled core should be measured and recorded to the nearest 0.2 inches. If bonded caps are applied to the specimens, the length of the specimens should be recorded both before and after capping to the nearest 0.1 inch. The length of the core, which was taken after end preparation, should be used to calculate the length-to-diameter ratio of the core. The diameter of the core should also be measured and recorded to the nearest 0.01 inch. This is done by taking at least two measurements at the mid-height of the core at right angles to one another. Once these data are recorded, the cores are tested within seven days of being drilled in accordance to AASHTO T22 (2009).

2.3.2.2 Factors Affecting Apparent Strength of Cores

Many factors affect the apparent strength of the cores obtained from the in-place structure. Studies have also been conducted to determine the effects of different core diameters, length-to-diameter ratios, the amount of damage imparted on a core, core moisture conditioning, effect of reinforcement, and direction of coring relative to casting direction.

Bartlett and MacGregor (1995) proposed that the strength of a core should be converted into an equivalent in-place strength using Equation 2-12, where $f_{c,is}$ is the equivalent in-place concrete strength, $F_{l/d}$ is the strength correction factor for length-to-diameter ratio, F_{dia} is the correction factor for core diameter, F_r is the correction factor for cores containing reinforcing bars at right angles to the central axis of the core, F_{mc} is the correction factor for moisture conditions, F_d is the correction factor for core damage, and f_c is the measured strength of the core. Bartlett and MacGregor (1995) also provide a Table 2-3, which shows how these values are calculated. The factors obtained from Table 2-3 are then substituted into Equation 2-12 to calculate the equivalent core strength.

$$f_{c,is} = F_{l/d} F_{dia} F_r F_{mc} F_d f_c \qquad \text{Equation 2-12}$$

Table 2-3: Core Strength Correction Factors (ACI 214 2010)

	Factor	Mean value	Coefficient of variation V, %
$F_{\ell/d}$: ℓ/d ratio†	Standard treatment‡:	$1-\{0.130-\alpha f_{core}\}\left(2-\frac{\ell}{d}\right)^2$	$2.5\left(2-\frac{\ell}{d}\right)^2$
	Soaked 48 hours in water:	$1-\{0.117-\alpha f_{core}\}\left(2-\frac{\ell}{d}\right)^2$	$2.5\left(2-\frac{\ell}{d}\right)^2$
	Dried§:	$1-\{0.144-\alpha f_{core}\}\left(2-\frac{\ell}{d}\right)^2$	$2.5\left(2-\frac{\ell}{d}\right)^2$
F_{dia}: core diameter	2 in. (50 mm)	1.06	11.8
	4 in. (100 mm)	1.00	0.0
	6 in. (150 mm)	0.98	1.8
F_{mc}: core moisture content	Standard treatment‡:	1.00	2.5
	Soaked 48 hours in water:	1.09	2.5
	Dried§:	0.96	2.5
F_d: damage due to drilling		1.06	2.5

† Constant α equals 3(10–6) 1/psi for f_{core} in psi, or 4.3(10–4) 1/MPa for f_{core} in MPa.
‡ Standard treatment specified in ASTM C42/C42M.
§ Dried in air at 60 to 70°F (16 to 21°C) and relative humidity less than 60% for 7 days.

2.3.2.2.1 LENGTH-TO-DIAMETER RATIO

Much research has been done on the effect of length-to-diameter ratio on core testing. Bartlett and MacGregor (1994d) state that “short specimens fail at greater loads because the steel loading platens of the testing machine restrain lateral expansion throughout the specimen more effectively.” Therefore, the smaller the length-to-diameter ratio, the larger the apparent strength of the core will be. When assessing the in-place strength of concrete, AASHTO T24 (2009) defines correction factors that must be applied to cores which have length-to-diameter ratios from 1.0 to 1.75 which can be seen below in Table 2-4. These correction factors are also listed in ASTM C 42, and both AASHTO T24 and ASTM C 42 state these are only valid for concrete strengths between 2,000 and 6000 psi. Arioz et al. (2007) concluded that the effect of the length-to-diameter ratio was more significant as the diameter of the specimen decreased.

Table 2-4: Correction Factors for Different L/D Ratios For Concrete Strengths Less Than 6000 psi (AASHTO T24 2009)

Core L/D	AASHTO T24 Strength Correction Factor
1.75	0.98
1.50	0.96
1.25	0.93
1.00	0.87

AASHTO T24 (2009) does not list recommended values for length-to-diameter strength correction factors for concretes with strengths higher than 6000 psi. Similarly, AASHTO T24 (2009) notes that for strengths above 10,000 psi that correction factors may be higher that what is listed in Table 2-4, and that these factors should be applied to high-strength concretes with caution. AASHTO T24 (2009) makes no recommendation about what should be done for strength correction factors for concrete with compressive strengths between 6,000 and 10,000 psi. Similarly, Bartlett and MacGregor (1994d) state that there is some indication that as concrete strength increases, the strength correction factors for length-to-diameter ratio begin to increase, which implies that as concrete strength increases, the length-to-diameter ratio has less of an impact on apparent strength.

2.3.2.2.2 CORE DIAMETER

There are conflicting opinions when it comes to the effect of diameter on core strength. For cores with the same length-to-diameter ratio, Meininger (1968) found that the core diameter does not have an effect on the core’s apparent strength when the cores have a length-to-diameter ratio of 2.0. Bartlett and MacGregor (1994c) found that the strength of a 2-inch diameter core was approximately 94% of a 4-inch diameter core and 92% of a 6-inch diameter core. This trend can be observed in Figure 2-3. The data in Figure 2-3 has been normalized so that the standard core diameter is 4 inches.

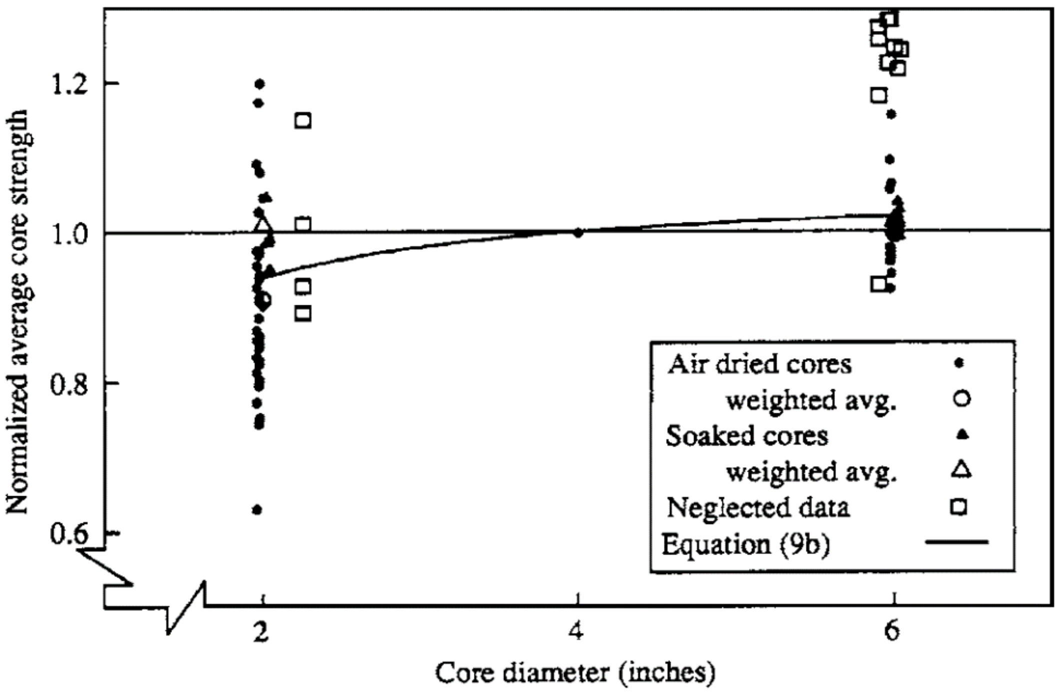

Figure 2-3: Normalized Average Core Strength versus Core Diameter (Barlett and MacGregor 1994c)

Arioz et al. (2007) concluded that as core diameter decreased, strength decreased as well. ACI 214.4R (2010) gives recommended values for correction factors for cores based on their diameters that can be seen below in Table 2-5.

Table 2-5: Suggested Strength Correction Factors for Different Core Diameters (ACI 241.4R 2010)

Diameter (in.)	Strength Correction Factors
2	1.06
4	1.00
6	0.98

Bartlett and MacGregor (1994c) also concluded that the effect of core damage increases as the size of the specimen decreases. This is especially important with respect to the effect of core diameter on apparent strength. This implies that the smaller the diameter of the specimen, the greater effect that damage will have on its apparent strength. Bartlett and MacGregor (1994c) also concluded that the variability was much larger in specimens with smaller diameters. They suggest that cores with smaller dimensions are much more susceptible to being impacted by through-thickness variation of the in-place concrete, especially in slabs.

2.3.2.2.3 Core Damage

When obtaining concrete cores, there is inherit damage that the cores are subjected to due to the destructive nature of the drilling process. As can be seen in Table 2-3 from Bartlett and MacGregor (1995), a strength correction factor of 1.06 is to be applied when a core is damaged during drilling. Bartlett and MacGregor (1994c) explain that cores can be damaged due to microcracking, cutting through coarse aggregate, and undulations at the drilled surface, but no clarification is made on what specifically constitutes enough damage for this factor of 1.06 to be applied. Arioz et al. (2007) found that strength correction factors for core damage decreased in concretes with higher strengths and hypothesized that the reason for this is that high-strength cores are subjected to less damage during the coring process.

Aggregate type also has an effect on the amount of damage imparted on the core during drilling. Khoury, Aliabdo, and Ghazy (2014) found that concrete containing river gravel are more difficult to core than concrete which contains softer aggregates such as limestone. Khoury, Aliabdo, and Ghazy (2014) also concluded that cores taken from higher strength concretes, in general, have less damage imparted on them than cores taken from lower strength concretes.

2.3.2.2.4 Casting Direction

There is some disagreement in literature over whether or not the direction of coring relative to the casting direction has an impact on the apparent strength of the core. The primary reason why there is suspicion that coring direction with respect to casting direction has an effect on the apparent strength of a core is because of the ITZ. Mehta and Monteiro (2014) write that the ITZ is most prominent around the bottom of the coarse aggregate due to bleed water that creates a plane of weakness in one direction.

Suprenant (1985) concluded that due to the plane of weakness that is formed around the bottom of the coarse aggregate relative to casting direction, the direction in which the core is drilled is significant. An illustration of this effect can be seen below in Figure 2-4. From Figure 2-4, Suprenant (1985) illustrates the plane of weakness around the bottom of the coarse aggregate. When cores that are drilled parallel to the casting direction are tested, this plane of weakness is perpendicular to the applied test load. However, if a core is drilled perpendicular to the casting direction, this plane of weakness is now parallel to the applied force when the core is tested in compression. Munday and Dhir (1984) conducted research on coring direction versus casting direction and suggest that cores taken parallel to the casting direction will have strengths approximately 8% greater than cores drilled perpendicularly to the casting direction.

There are other studies though, such as the one conducted by Bloem (1965), which have concluded that coring direction relative to casting direction does not produce statistically significant differences in apparent strengths. Bartlett and MacGregor (1994b) also conclude that there was not a significant difference in their data between cores that were drilled parallel versus perpendicular to the casting direction.

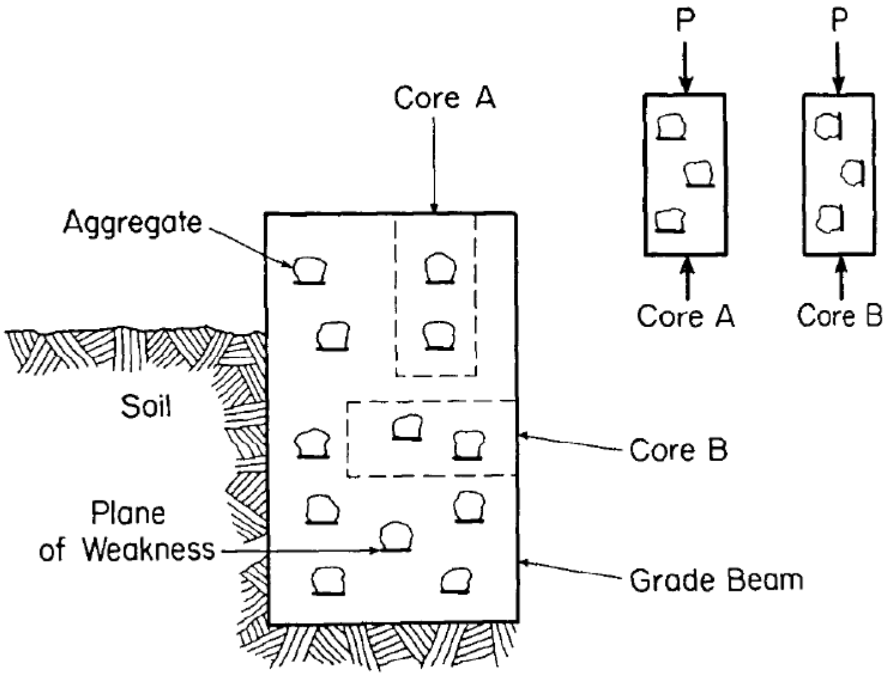

Figure 2-4: Effect of Coring Direction Relative to Casting Direction (Suprenant 1985)

2.3.2.2.5 PRESENCE OF EMBEDDED STEEL REINFORCEMENT IN CORE

Sometimes in members with a congested steel reinforcement layout, it is difficult to recover a core without hitting steel reinforcement. Because of this, guidance is necessary on what to do if a core contains steel reinforcement within it. AASHTO T24 (2009) recommends that core specimens containing embedded reinforcement not be used to determine compressive, splitting tensile, or flexural strength.

Bartlett and MacGregor (1995) recommended the correction factors shown in Table 2-6 to correct the compressive strength of a core containing steel reinforcement. It should be noted that the correction factors presented by Bartlett and Macgregor (1995) are for steel reinforcement which runs *perpendicular* to the axis of drilling. The strength correction factors shown in Table 2-6 would be used in Equation 2-11 to correct a core's compressive strength. No guidance is given on how much of the steel reinforcing bar must be contained within the core for the correction factors to be applied or if bar size has an impact on core compressive strength.

Table 2-6: Strength Correction Factors for Steel Reinforcement Present in Core Sample (Bartlett and MacGregor 1995)

Number of Reinforcing Bars Present in Core	Strength Correction Factor
1	1.08
2	1.13

2.3.2.3 Variability of Core Testing

Bartlett and MacGregor (1994c) found that the variability of cores greatly depended on the through thickness variation within the concrete itself. They also noted that “the variability of measured strengths of small-diameter cores is particularly sensitive to being inflated by the variability of the in situ strength across the dimension of the element being cored.” Therefore, if through-thickness variation develops in concrete members, the variability of cores with smaller diameters are likely to be more susceptible to this effect, but if through-thickness variation is not significantly present within a member, then the coefficient of variation will be similar for cores of all diameters. If through-thickness variability is not present, Bartlett and MacGregor (1994c) concluded that the variability of the in-situ concrete from members of moderate size from one batch of concrete is approximately 5 percent. Arioz et al. (2007) also found that variability also increased as core diameter decreased. ACI 214.4R (2010) provides Table 2-7 with expected range (in percent) for core strength test results based on the number of specimens tested.

Table 2-7: Probable Range of Core Strengths Due to Single-Operator Error (ACI 214.4R 2010)

Number of cores	Expected range of core strength as percent of average core strength	Range with 5% chance of being exceeded as percent of average core strength
3	5.4	10.6
4	6.6	11.6
5	7.2	12.4
6	8.1	12.9
7	8.6	13.3
8	9.1	13.7
9	9.5	14.1
10	9.8	14.3

2.3.2.4 Impact of Number of Cores Retrieved

In order to meet the strength acceptance based on core test results of ACI 318 (2014), the average of the *three* cores taken must be greater than 85 % of the specified design strength, while also not having a single core with a strength lower than 75 % of the specified design strength. Although it is common that three cores are taken in order to evaluate in-place strength, there are times in which more than three cores are taken. Bartlett and Lawler (2011) point out that an increase in the number of cores would not impact the mean-strength criterion of ACI 318, but it would have an impact on the requirement that no single test result could have a strength lower than 75 % of the speficied design strength. Inherently, if the amount of specimens increases, the liklihood that a specimen with a strength lower than 75 % of the specified design strength would increase as well. If the coefficient of variation, V, is known, then the graph seen in Figure 2-5 from Bartlett and Lawler (2011) can be used to determine the acceptable value of *k*, which is defined as the lowest acceptable ratio between a single core strength and specified design strength.

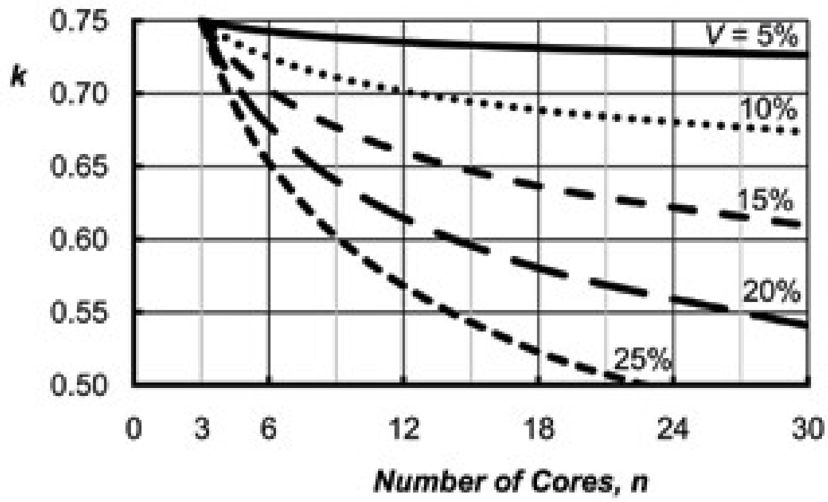

Figure 2-5: *k* versus Number of Cores Given Coefficient of Variation (Bartlett and Lawler 2011)

From the plot in Figure 2-5, it can be seen that the specified value for *k* given *V* decreases as the number of cores taken increases. This implies that the change in single core strength criterion should be taken into account when more than three core specimens are retrieved and tested. Bartlett and Lawler (2011) outline a statistical method that can be used to determine the acceptable value of *k* based on the number of cores extracted from the concrete in question. First, a value of P_1 must be specified. P_1 represents the chance that a core from a set of three cores will have a strength less than $0.75 \times f'_c$. ACI 214 (2010) recommends this value be 10 %. Once this value is established, the value of P_2 can be calculated using Equation 2-13 based on the number of cores retrieved. P_2 is defined as the corresponding probability to P_1, but with a sample size of *n* instead of three.

$$P_2 = 1 - (1 - P_1)^{3/n} \quad \text{Equation 2-13}$$

Once P_2 is calculated, then the corresponding value of c_1, which is the number of standard deviations below the mean that P_2 occurs, can be found using the standard normal distribution function. An allowable coefficient of variation, V_{ws}, is then chosen. ACI 214 (2010) contains Table 2-8, which can be used to choose an appropriate coefficient of variation. Once this is done, c_1 and V_{ws} are used in Equation 2-14 to determine the appropriate value of *k*.

$$k = 0.85 - c_1 \times V_{ws} \quad \text{Equation 2-14}$$

Table 2-8: Coefficient of Variation Due to In-Place Strength Variation Within a Structure

Structure composition		One member	Many members
One batch of concrete		7%	8%
Many batches of concrete	Cast-in-place	12%	13%
	Precast	9%	10%

2.3.3 Cast-In-Place Cylinders

With the cast-in-place cylinder testing method, a cylinder mold is held in place close to the structure's surface by a support system and filled with concrete as the member is being cast. A detailed setup for a cast-in-place cylinder can be seen below in Figure 2-6. After remaining in the structure for some time, the specimen is removed from the structure, transported to the testing lab, and tested. ACI 228.1R (2003) outlines the benefits of using cast-in-place cylinders and states that the advantages of using this method over coring include no damage being imparted on the specimen due to coring while also matching the thermal and curing history of the specimen to the in-place concrete.

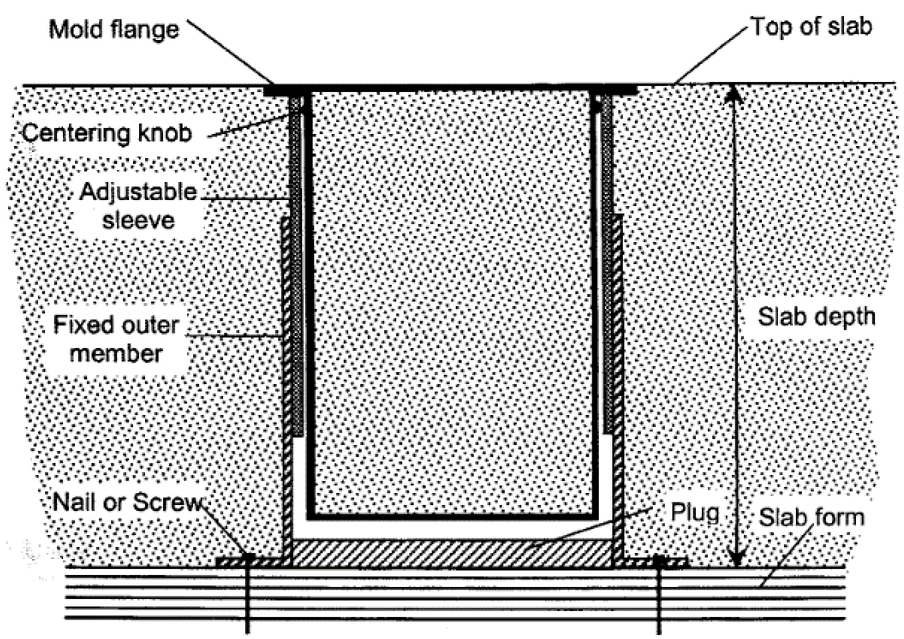

Figure 2-6: Schematic of Cast-In-Place Cylinder Mold Assembly (ASTM C873 2011)

2.3.3.1 Summary of ASTM C873

ASTM C873 (2011) specifies that the diameter of the mold must be at least 3 times the size of the nominal maximum aggregate size of the coarse aggregate gradation used in the concrete and that the mold must have a length-to-diameter ratio of at least 1.0 as well as a minimum diameter of 4 inches. Molds used to make cast-in-place cylinder specimens must conform to the water leakage specification in ASTM C470 while also being rigid enough to maintain their shape after being filled with concrete. The molds must also have a lip or ledge for the mold to sit on top of the support system as well as to seal the gap between the support system and the mold. The support members used to hold the concrete molds in place must be right circular cylinders which must have a diameter which accomodates the concrete molds while also being rigid enough to resistance deformation. Once any steel reinforcement is put in place, the support system should be secured to the formwork being used by using nails or screws. Upon installation, the top of the molds shall be even with the top of the formwork for the member. When filling the cylinder with concrete, the consolidation methods used for the member should also be used for the cast-in-place cylinders. If internal vibration is used, then the vibrator should be used externally on the specimens, briefly touching the vibrator to the outside of the support member. Internal consolidation should not be used for the specimens except under special instructions. The surface of the specimens should be finished the same as the surrounding concrete. The specimens should be exposed to the same curing conditions as the surrounding concrete.

The specimens should remain fully in place until recovered. After the specimens are removed from the slab, they must be kept at ± 10°F of the slab surface temperature at the time of removal until they are tested. Specimens must be transported back to the testing facility within 4 hours of removal from the concrete member. Caution must be used during transportation so that the specimens are not damaged. Also, insulation must be provided to prevent extreme temperature variation as well as moisture loss. Once the specimens have reached the testing facility, molds must be stripped. The average diameter of the specimen must be determined by taking the average of two measurements at mid-height of the specimen perpendicular to one another. If the specimens are to be capped, the length of the specimen should be recorded after capping. Compression testing of the members shall be done according to ASTM C39. The specimens should be testing in the moisture condition in which they were received from the field. Compressive strength should be determined using the specimen cross-sectional area obtained from using the specimen’s average diameter.

2.3.3.2 VARIABILITY OF CAST-IN-PLACE CYLINDERS

ASTM C873 (2011) states that the single-operator coefficient of variation for cast-in-place cylinder specimens is 3.5 % for concrete strengths ranging from 1500 to 6000 psi. This means that the results from two tests which were correctly performed should not differ by more than 10.0 precent of their average.

2.3.4 PULLOUT TESTING

Kierkegaard-Hansen and Bickley (1978) state that the LOK-Test pullout method was first developed in Denmark in the 1960s in order to develop a method of measuring the in-place strength of hardened concrete. A schematic of a cast-in insert can be seen in Figure 2-7. Hubler (1982) states that "in operation, the LOK-TEST device, a calibrated screw-actuated hydraulic jack, non-destructively pulls pre-positioned bolts embedded in the concrete." The force required to fail the concrete is recorded and then converted to a compressive strength. Bickley (1982) states that by using the pullout test method, "variations in the strength of in-place concrete can be measured and the minimum strength in a placement calculated by the standard statistical methods to high degrees of confidence."

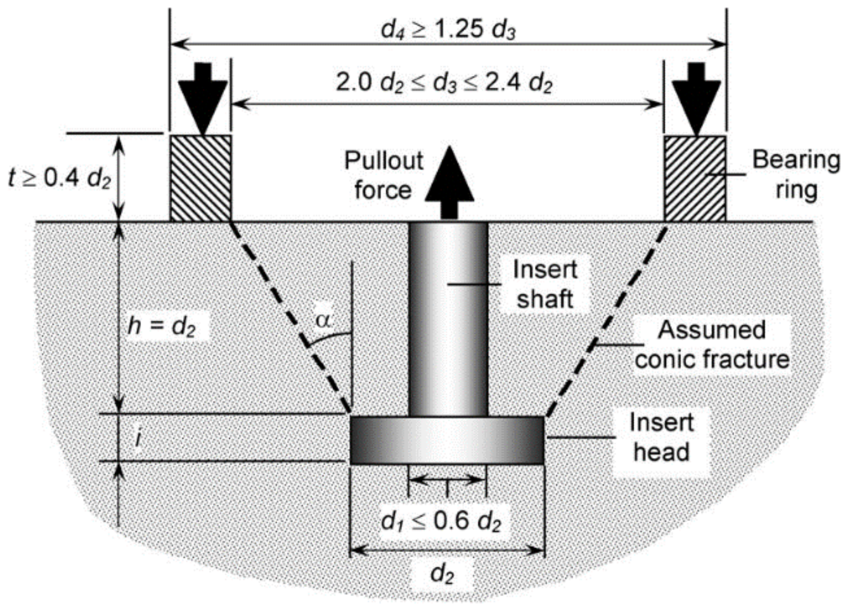

Figure 2-7: Schematic of LOK-Test Pullout Insert (ASTM C900 2007)

2.3.4.1 SUMMARY OF ASTM C900

The Standard Test Method for Pullout Strength of Hardened Concrete, ASTM C900 (2007), states that there are three items needed when doing pullout tests: a pullout insert, a loading system, and a load-measuring system. The loading system must be calibrated at least once a year in order to assure that readings collected from the testing are correctly converted to compressive strengths. Bickley (1982) recommends that the relationship between pullout force and compressive strength should be determined for each job site as well as for each type of concrete and aggregate size.

Most often times, inserts are cast into fresh concrete and then tested at specified times after the concrete has hardened. Inserts can also be inserted after the concrete has hardened. In the case of cast-in inserts, the length of the stem of the insert must be equal to the diameter of its head. Test locations must be separated by a clear space of at least seven times the diameter of the insert's head. In addition, all test locations must have a clear space of at least 3.5 times the diameter of the insert's head away from free edges of the concrete. Whenever pullout tests are used, a minimum of five tests are required. When loading the insert, a load rate of 70 ± 30 kPa/s must be used. Once failure is reached, the failure load is recorded and then converted into an equivalent compressive strength using calibration data.

2.3.4.2 FAILURE MECHANISM

Many studies through the years have tried to establish a better understanding of the failure mechanism for the pullout test method. Carino (1997) states that “the pull-out test subjects the concrete to a non-uniform, three-dimensional state of stress.” Carino (1997) continues by stating that cracking also happens in two circumferential cracking systems: the first being a system which is stable and begins at the head of the insert at approximately 1/3 of the ultimate load which spreads into the surrounding concrete at a large apex angle, and the second being the cracking system which propagates under increasing load and eventually defines the shape of the cone which is extracted. Although there is a consensus on the development cracking systems, the actual failure mechanism is still debated today.

2.3.4.3 VARIABILITY OF PULLOUT TESTING

ASTM C900 (2007) states that cast-in inserts which are embedded approximately 1-inch below the concrete surface with a maximum aggregate size of 0.75 inches have a one-operator coefficient of variation of 8 %. Therefore the range of test results should not exceed the values listed in Table 2-9. A list of coeffients of variation from various projects involving the use of pullout tests are summarized in Table 2-10 from ACI 228.1R (2003).

Table 2-9: Acceptable Pull-Out Test Range Based on Number of Tests (ASTM C900 2007)

Number of Tests	Acceptable Range (Percent of Average)
5	31%
7	34%
10	36%

Stone, Carino and Reeve (1986) conducted a test to determine the effect of apex angle and aggregate type on the nature of the relationship between pullout strength and compressive cylinder strength. Three types of aggregate, crushed limestone, river gravel, and lightweight aggregate, were used in the study. It was concluded that the coefficient of variation was much lower for the lightweight aggregate than the two sources of normal-weight aggregate. This can be explained by the different failure mechanism attributed to the particular lightweight aggregates. Since lightweight aggregates most commonly break through the aggregate and not around it, the failure load is governed by the mortar strength. On the other hand, harder aggregates, such as the limestone and river gravel, will cause failure planes which will travel around the coarse aggregate, causing the ultimate pullout force to be based on the amount of aggregate interlock that occurs. Therefore, if a large aggregate is present near a pullout insert, the resulting pullout force will be significantly higher, which would lead to a significantly higher coefficient of variation for a given number of tests. It was also concluded from the study that any apex angle within the 54 to 70 degree range will not have a significant effect on the coefficient of variation.

Table 2-10: Summary of Within-Test Coefficient of Variation of Pullout Test (ACI 228.1R 2003)

Reference	Apex angle, degrees	Embedment depth, in.	Maximum Aggregate Size, in.	Aggregate Type	No. of replicate specimens	Coefficient of variation, %	
						Range	Average
Malhotra and Carette (1980)	67	2	1	Gravel	2	0.9 to 14.3	5.3
Malhotra (1975)	67	2	1/4	Limestone	3	2.3 to 6.3	3.9
Bickley (1982)	62	1	3/8	?	8	3.2 to 5.3	4.1
Khoo (1984)	70	1	3/4	Granite	6	1.9 to 12.3	6.9
Carette and Malhotra (1984)	67	2	3/4	Limestone	4	1.9 to 11.8	7.1
	62	1	3/4	Limestone	10	5.2 to 14.9	8.5
Keiller (1982)	62	1	3/4	Limestone	6	7.4 to 31	14.8
Stone, Carino, and Reeve (1986)	70	1	3/4	Gravel	11	4.6 to 14.4	10.2
	70	1	3/4	Limestone	11	6.3 to 14.6	9.2
	70	1	3/4	Low density	11	1.4 to 8.2	6
	54	1	3/4	Gravel	11	4.3 to 15.9	10
Bocca (1984)	67	1.2	1/2	?	24	2.8 to 6.1	4.3

2.4 SUMMARY OF STATE DOTS PAYMENT REDUCTION METHODS

If cylinder breaks are low for a concrete placement, steps must be taken in order to evaluate the integrity of the in-place concrete. Most of the time, cores are taken from the structure and tested in compression to determine the in-place compressive strength. Based on these results, state DOTs have methods to assess the strength and, if deficient to some degree, to reduce the amount that is paid to the contractor for in-place concrete. These methods of price adjustment vary from state to state. The practices of the Alabama Department of Transportation (ALDOT), Tennessee Department of Transportation (TDOT), Florida Department of Transportation (FDOT), and Texas Department of Transportation (TxDOT) were examined and analyzed as part of the literature reviewed. These states were chosen because their practices for payment correction were explicitly defined within their respective highway construction practice manuals as well as for their location relative to Alabama.

2.4.1 ALABAMA DEPARTMENT OF TRANSPORTATION

During the construction of concrete structures, ALDOT requires that molded cylinders are made and tested at 28 days for quality assurance. The compressive strength from concrete cylinders is accepted when the average of two consecutive cylinder test results, obtained at the same age, equals or exceeds the specified 28-day compressive strength, and neither cylinder test result is below 95% of the specified 28-day compressive strength. ALDOT uses core testing to assess the strength of substandard concrete. If ALDOT deems it necessary to evaluate the in-place compressive strength of substandard concrete, a core investigation in accordance with ALDOT-170 is performed.

Currently, ALDOT uses the price adjustment equation shown in Equation 2-15. This relationship can be seen in graphical form in Figure 2-8. Under its current practice, the average strength of the cores that have been retrieved from the job site must be equal to or exceed the specified design strength for the contractor to receive 100 percent pay. Although ACI 318 (2014) states that the in-place concrete is structurally adequate if the average of at least 3 cores is greater than 85% of the design load, ALDOT uses a pay scale which pays only 50% of the intended construction cost if the average strength of the cores which have been obtained equal 85% of the design strength after correction factors have been applied.

$$Price\ Adjustment\ (Percent) = 100\ \times\ (1.0 - \left[\frac{f'_c - f_{c,AVG}}{0.30 \times f'_c}\right]) \qquad \text{Equation 2-15}$$

Where: f'_c = required 28-day compressive strength; and

$f'_{c,AVG}$ = average compressive strength of test cores.

Note that the price adjustment shall be rounded to the nearest tenth of a percent, and the price adjustment is valid where: 50% ≥ Price Adjustment < 100%.

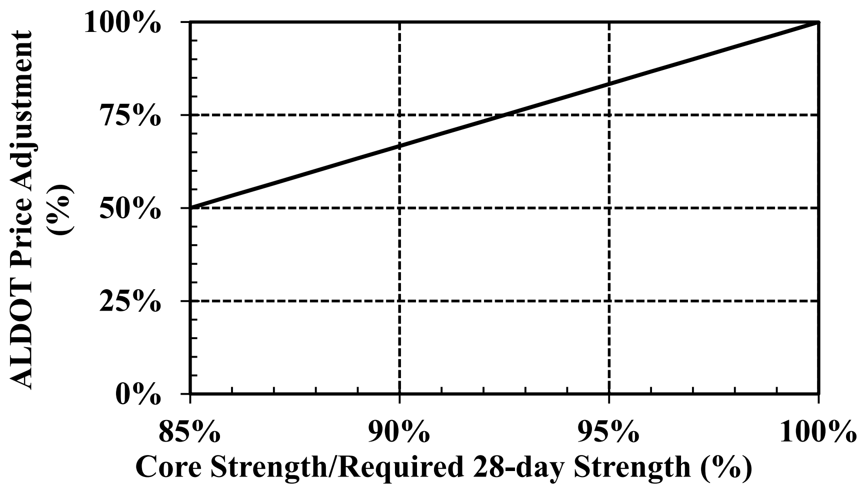

Figure 2-8: Current ALDOT Price Adjustment (ALDOT 2012)

2.4.2 TENNESSEE DEPARTMENT OF TRANSPORTATION

The Tennessee Department of Transportation (TDOT) uses specified mixtures which are grouped into classes based on their intended use. These different classes of concrete all have different specified values for minimum 28-day strength, unit weight, water-cement ratio, air content, and slump, which is shown in Table 2-11. TDOT also has a table which defines the acceptable strength of molded concrete cylinders for each class of concrete based on their age, which is shown in Table 2-12. TDOT (2006) states that if cylinder strengths do not meet the specified strength of the concrete, then cores may be taken at the expense of the contractor. If cores are taken, these strengths will become the strength of record and price adjustment will be based off of the strengths obtained from the cored specimens. Two cores are taken and the average strength of these cores becomes the strength of record. Core diameters between 3.75 in. and 4 in. must be used. Also, core lengths should be between 7.5 in. and 8 in. and should have length-to-diameter ratios from 1.9-2.1, but in no case should have a length-to-diameter ratio less than 1.0. Cores which have length-to-diameter ratios less than 1.75 after being capped will have correction factors applied to them according to AASHTO T24 (2009). Table 2-13 outlines the price reduction method used by TDOT.

It can be seen that if the cylinders or cores fail to meet the specified design strength for the age range which it was tested outlined in Table 2-12, then the price reductions in Table 2-13, which are based on the percentage below the required compressive strength that the test specimens were, will be applied to the concrete which was placed by the contractor. TDOT Division of Materials and Tests (2014) states that cores must be obtained and tested within 56 days of placement. Similarly, the Tennessee Department of Transportation (2006) states that cylinder submitted for testing after 56 days will not be accepted.

Table 2-11: Tennessee Department of Transportation Classes of Concrete (TDOT 2006)

Concrete Class	Min. 28-Day Comp. Strength (psi)	Min. Cement Content (lb/cy)	Maximum Water-Cement Ratio	Air Content % (Design ± production tolerance)	Slump (in.)
A	3000	564	0.45	6 ± 2	3 ± 1
D	4000	620	0.40	6	8 max.
L	4000	620	0.40	6	8 max.
S (Seal)	3000	682	0.47	6 ± 2	6 ± 1

Table 2-12: Speficied Acceptable Average Strength Concrete Specimens for Given Class of Concrete based on Age of Specimen (TDOT 2006)

Class of Concrete	Less than 31 Days	31 to 42 Days	43 Days or More
A, S	3000 psi	3300 psi	3500 psi
D, L	4000 psi	4400 psi	4600 psi

Table 2-13: Price Adjustment for Tennessee Department of Transportation (TDOT 2006)

Percent Below Specified Concrete Strength Specified in Table 2-10	Percent of Bid Price to be Paid
0.1 - 3.3	95
3.4 – 6.7	90
6.8 – 10.0	80
10.1 – 13.3	70
13.4 – 16.7	60
16.8 – 20.0	50
20.1 – 23.3	45
23.4 – 26.7	40
26.8 – 30.0	35
30.1 – 33.3	30
> 33.3	25

2.4.3 FLORIDA DEPARTMENT OF TRANSPORTATION

Similar to many other state DOTs, the Florida Department of Transportation (FDOT) requires that three quality assurance cylinders be made when placing concrete. FDOT (2010) states that if the average strength of the quality assurance cylinders falls more than 500 psi or 10 percent, whichever is greater, below the specified acceptable minimum compressive strength of the concrete, cores should be taken in order to determine if the in-place concrete is acceptable or if it must be removed and replaced.

Core locations must be approved by FDOT and must not induce permanent damage to the structure after the core hole is repaired. The cores are then tested by FDOT in accordance with ASTM C42 in either the wet or the dry condition, which is specified by the engineer. If the core strength results are less than 10% or 500 psi, whichever is greater, below the specified acceptable minimum compressive strength of the concrete, the concrete is deemed structurally adequate. FDOT considers concrete from which the average core strength of three specimens is more than 10% or 500 psi, whichever is greater, below the specified acceptable minimum compressive strength of the concrete structurally questionable. If this occurs, a structural analysis of the structure must be performed by the Specialty Engineer. If the analysis indicates that the concrete strength is adequate for the intended purpose of the structure, then the concrete is permitted to be left in place. Otherwise, the concrete must be removed and replaced by the contractor. Cores should not be taken if the average strength of the quality control cylinders is less than 10% or 500 psi, whichever is greater, below the specified allowable minimum strength. If cores are obtained and tested before the concrete has reached an age of 42 days, the average core strength will be taken as the 28-day strength. If cores are tested after 42 days, then the strength will be corrected for age in accordance with Equations 2-1 through 2-10 in Section 2.2.1. The formula for pay reduction can be seen below in Equation 2-16, where f'_c is the specified acceptable minimum strength of the concrete and f_c is the average strength of the core specimens retrieved from the structure.

$$Percent\ Reduction = 100 * \frac{f'_c - f_c}{f'_c} \qquad \text{Equation 2-16}$$

2.4.4 TEXAS DEPARTMENT OF TRANSPORTATION

Similar to other state DOTs, TxDOT requires that cores be taken in the event that the average strength of quality assurance cylinders does not meet the specified design strength for the project. If the average of the quality assurance cylinders meet the specified design strength and no single cylinder has a strength less than 85% of the design strength, the concrete is paid for at full price. If the average strength of the quality assurance cylinders do not meet the required strength or if one of the cylinders breaks below 85% of the required design strength, then the engineer will perform a structural analysis of the concrete structure to determine its adequacy. If cores must be taken to assess the in-place strength of the concrete, it will be done at the expense of the contractor and the engineer will test the cores. The Texas Department of Transportation (2004) specifies that "if all tested cores meet the required design strength, the concrete will be paid for at full price." If any of the cores do not meet the specified required design strength but the average strength of the cores is determined to be adequate, price reduction is done by using Equation 2-17 with the average strength of the cores being the strength of record.

$$A = [0.10 + 0.75(\frac{S_a}{S_s})^2] \times B_p \qquad \text{Equation 2-17}$$

Where:

A = Amount to be paid per unit of measure for the entire placement in question

S_a = Actual strength from cylinders or cores. Use values from cores, if taken

S_s = Minimum required strength (specified)

B_p = Unit bid price

2.4.5 Comparison of State DOT Payment Reduction Methods

From the literature discussed above, it can be seen that each state has varying ways to reduce the price paid for concrete which does not meet specified design strength according to quality assurance cylinder tests but is deemed structurally adequate. Figure 2-9 shows a graphical comparison between the various payment reduction methods of the state DOTs which were discussed in this section. From Figure 2-9, it can be seen that each state uses drastically different payment reduction methods. Of the four state DOTs which were examined, TDOT is the only one which uses a stepped function.

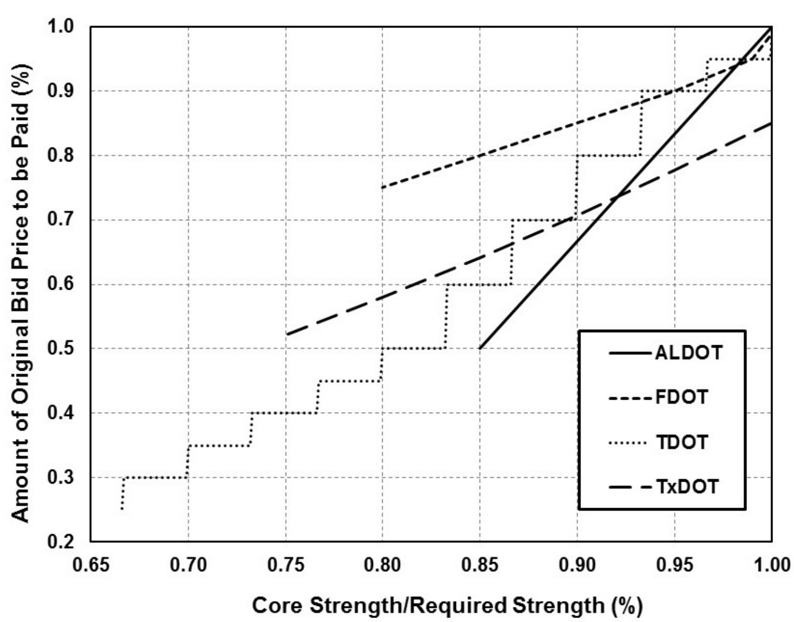

Figure 2-9: Summary of Payment Reduction Methods

2.5 Summary

Many factors affect the apparent strength of the in-place concrete. In this chapter the impact of concrete age, type of cementitious materials, curing conditions, coarse aggregate size and type, and temperature are discussed. The availability of moisture for curing in-place concrete was found to be one of the most important reasons why the strength of laboratory-cured, molded cylinder may be greater than the in-place strength.

Testing methods also have a significant effect on the apparent strength of the concrete. Molded cylinders should always be used as a method of quality assurance, but do not provide an accurate estimation of the actual in-place strength of the concrete. If cylinder test results are low, other methods of in-place testing should be used to assess the in-place strength. If cores are taken, ACI 318 (2014) states that the average strength of three specimens must exceed 85% of the design strength with no single value below 75% of the design strength for the in-place concrete to be considered structurally adequate.

When analyzing the data recovered from core testing, a number of factors must be taken into account to produce meaningful estimations of in-place strength. Factors such as core diameter, length-to-diameter ratio, moisture conditioning, core damage, coring orientation relative to casting direction, and the presence of steel reinforcement all may need to be considered to correctly assess the in-place concrete strength. Common methods used to measure the in-place strength include core testing, cast-in-place cylinders, and pullout testing. The data collected from these tests must also be analyzed carefully to draw valid conclusions about the in-place strength of the concrete.

If the average strength of the quality assurance cylinders is below the specified required strength of the concrete, most states specify that cores can or must be taken to evaluate the adequacy of the in-place concrete. If the in-place concrete strength is found to be substandard, then the contractor is most often paid on a reduced scale depending on the average strength of the core specimens. This reduced pay scale differs from state to state. If cores are taken and the strength of the in-place concrete is not accepted by the state DOT, then either the structure must be strengthened or the concrete must be removed and replaced at the contractor expense.

Chapter 3
Experimental Plan

In this chapter, the design and implementation of the experimental plan for the project is presented and discussed. Different types of in-place testing were conducted to determine the relationship between test type and apparent in-place strength as compared to standard molded cylinder strength. The effect of age, strength level, coarse aggregate type, supplementary cementing materials (SCMs), and degree of microcracking were also evaluated during the course of the project. The objective of this project was to provide ALDOT with data and recommendations for interpreting data collected from core testing as well as means to accept the in-place concrete based on core strength results.

3.1 Introduction and Problem Definition

After the completion of the literature review, the factors that were to be evaluated during the project were defined. Since multiple procedures have been developed in order to convert core compressive strength at a certain age to a representative 28-day compressive strength, it was determined that the two main procedures outlined in Chapter 2 from ACI 209.2R (2008) and Yazadani and McKinnie (2004) should be evaluated to determine which more accurately predicted the 28-day compressive strength. Since Yazdani and McKinnie (2004) recommended different equations for different types of SCM types and ACI 209.2R did not, it also needed to be known if SCM type impacts the in-place strength development as measured by cores.

Secondly, the effect of damage to the core had to be evaluated. Khoury, Aliabdo, and Ghazy (2014) found that the type of aggregate contained within concrete had an effect on the apparent strength of a core. Khoury, Aliabdo, and Ghazy (2014) also suggested that cores which were taken from high-strength concretes suffered far less damage than those taken from normal-strength concrete. Because of this, aggregate type, strength level, and the effect of microcracking were evaluated in this study.

In today's industry, one can evaluate the in-place concrete strength with many different methods. Since the goal is to assess the accuracy of core testing, other methods were used to all the research team to compare to the in-place strength results obtained from coring. One common method for predicting in-place strength throughout Europe and Canada is the pullout test. In a previous ALDOT research study, Nixon et al. (2008) successfully used pullout testing and cast-in-place cylinders to evaluate in-place strength. Because of this, pullout testing and cast-in-place cylinders were chosen as the alternative in-place testing methods to compare against core testing.

Most state DOTs have a payment reduction scale, which is used to adjust the amount which contractors are paid when cores are used to evaluate the strength of the in-place concrete. When cores are taken, their average strength is compared to the required design strength in order to determine the adequacy and strength of the in-place concrete as well as how much the contractor is to be paid for the concrete. The payment reduction practices by ALDOT, FDOT, TDOT, and TxDOT were covered in Chapter 2. In the case of all four of these DOTs, no consideration is given to the amount of damage inflicted upon a core during the drilling process, the potential difference in curing conditions, and the presence of microcracking when determining the payment reduction. Therefore, if cores do not meet the required minimum strength, then payment is reduced for the in-place concrete. In contrast, ACI 318 (2014) states that concrete strength shall be deemed structurally adequate if the average strength of three cores is greater than 85% of the specified compressive-strength (f'_c) for the project as long as no single core strength of three cores is below 75% of the required strength.

For many years, the moist-cured, molded 6×12 in. cylinder has been the standard for the quality assurance of concrete. In more recent years, it has been suggested moist-cured, molded 4×8 in. cylinders would produce similar results to 6×12 in. cylinders. Day and Haque (1993) propose that this switch would pose numerous advantages, such as easier handling during transportation, smaller required storage spaces, lower required capacity of testing machines, and the reduced costs for molds, capping materials, and concrete.

3.2 Development of Experimental Plan

First, in order to evaluate the effect of strength gain over time, different testing ages were chosen. The first in-place tests were to be conducted at 28-days in order to obtain measured values to compare with both the moist-cured, molded cylinders to be broken at 28 days as well as to compare these values with ones obtained from future testing ages. The second age that was chosen was 42 days. This is because ALDOT (2012) requires cores to be drilled and tested at the latest 42 days after placement. The long-term strength development needed to be evaluated, but it had to be within the time limitations for the project. Because of this, the testing age of 365 days was also chosen. Finally, it was determined that testing should take place at an age sometime between 42 and 365 days, so 91 days was chosen as this is three months after placement and is a common testing age used in the concrete industry. Since strength development is also dependent on SCM type, it was determined that members containing different types of SCMs should be cast. Class C fly ash, Class F fly ash, and slag cement were chosen, because these are the most commonly used SCMs in Alabama's concrete industry.

Next, variables that may contribute to core damage were considered. Since coarse aggregate type could have an effect on the apparent strength of a core due to the difficulty to cut through, three different aggregate types that are local to the state of Alabama were evaluated: uncrushed river gravel, crushed limestone, and crushed granite. Another factor which has an effect on the damage imparted on a core is the amount of microcracking which occurs within the concrete. Since microcracking is heavily impacted by the amount of restraint that the concrete is exposed to, two regions of testing were to be considered: one near the exterior edge of the member to represent the low restraint region and one in the middle of the member to represent the highly restrained region. The third variable that could possibly have an impact on the amount of damage is the concrete strength. To evaluate this, it was decided that both normal-strength and high-strength members needed to be cast in order to evaluate difference in core damage between these two concrete strengths.

There were three different types of in-place testing which were chosen: cores, cast-in-place cylinders, and pullout tests. ALDOT most frequently uses cores when determining the in-place strength of concrete. Cast-in-place cylinders are a type of in-place testing that is not widely used, but produces molded specimens with the same temperature and moisture history as the in-place concrete while still allowing the specimen to expand and contract, therefore reducing the impact of microcracking. The third type of testing used was pullout testing, which is occasionally used in Europe and Canada to evaluate in-place strength, and has been used on past ALDOT research projects (Nixon et al. 2008). Floating pullout inserts were position into the concrete surface during casting of the slab elements and then tested at the specified ages. Since only 4 in. diameter cores will be tested, it was decided to also test 4×8 in. molded cylinders to allow a direct comparison of specimens with the same diameter. Along with the different in-place testing methods, both 6×12 in. and 4×8 in. molded, moist-cured cylinders were made and tested at the same age as the in-place testing methods.

It was determined that slab specimens should be made, as this will provide enough area for all the in-place strength tests. It was determined that eight different slabs should be cast to encompass the different aggregate types, strength levels, and SCMs that need to be tested. A slab size of 15 ft × 15 ft was chosen as this represented a large enough specimen, which could be considered representative of a full-scale field specimen, but also small enough to cast using a single ready-mixed truckload of concrete. This size would also ensure that the interior and exterior testing regions had different degrees of restraint and therefore different degrees of microcracking. A slab thickness of 9 ½ in. was chosen as this is representative of the thickness of a bridge deck, and this thickness allows one to obtain a 4 in. diameter and 8 in. long core, so that it has the standard length-to-diameter ratio of 2.0.

After the testing methods and various materials were chosen, a slab layout was devised in order to satisfy all the testing requirements. In order to model the effect of axial restraint within a slab, it first needed to be known what factors have an impact on restraint. Rasmussen and Rozycki (2001) showed that it was not just the frictional force caused by the self-weight of the slab that had an effect on the axial restraint of a slab, but also the interlocking and adhesion forces between the slab and the sub-base. An illustration of this can be seen in Figure 3-1. When shrinkage and temperature change occur within a slab, these forces restrain movement. An illustration of the typical stress distribution between a slab and its sub-base can be seen in Figure 3-2.

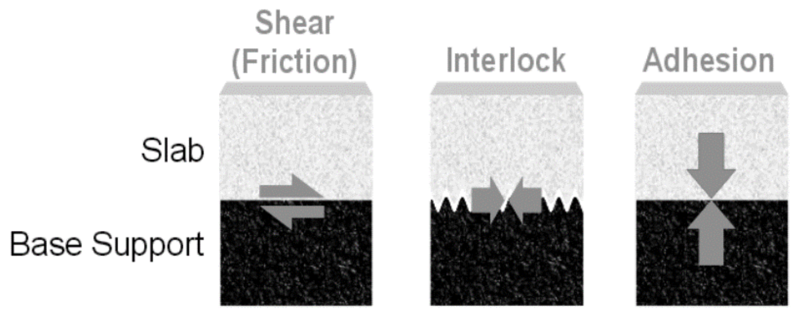

Figure 3-1: Forces Affecting the Axial Restraint of a Slab (Rasmussen and Rozycki 2001)

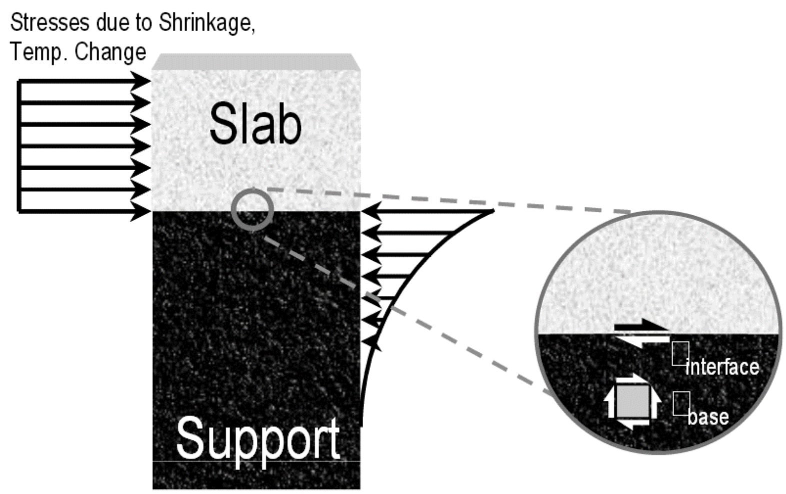

Figure 3-2: Typical Stress Distribution between Slab and Sub-Base when Exposed to Shrinkage and Temperature Change Forces (Rasmussen and Rozycki 2001)

With these modeling requirements in mind, a square, 15 ft × 15 ft slab with a 9½ in. depth was laid out and divided into 4 square quadrants. Each one of these quadrants would be tested at different ages to determine the effect of age for each testing method. The testing methods within the four quadrants were tested at 28, 42, 91, and 365 days respectively. This was done to establish a strength gain relationship for the in-place tests with respect to testing age. Care was taken to ensure that spacing requirements for the individual testing methods were met so that one testing method did not have an effect on the other surrounding tests. Figure 3-3 shows the anticipated effect that restraint would have on the slab. Testing which was done in the exterior region occurred near the area labeled “Least Restraint to Movement” while interior region testing was conducted near the area labeled “Most Restraint to Movement.” A schematic of the slab layout can be seen below in Figure 3-4. A close up view of the testing layout in a typical quadrant can be seen in Figure 3-5.

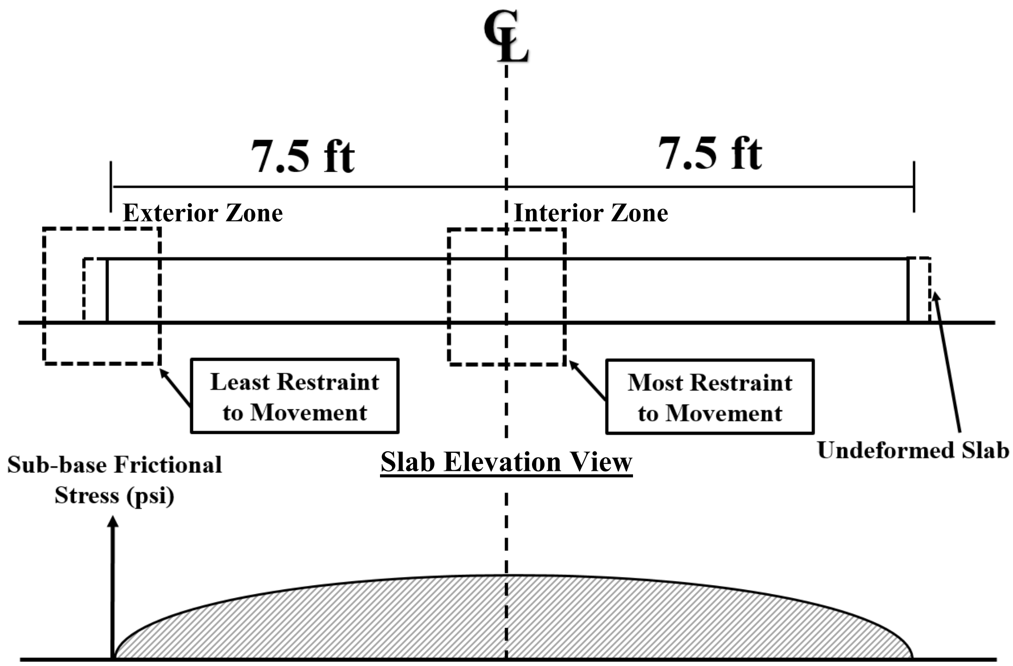

Figure 3-3: Effect of Restraint on Slab Specimens

Once the layout of the slabs were determined, concrete mixtures were selected. Considering all the variables that had to be evaluated, it was decided that six normal-strength concrete slabs and two high-strength concrete slabs were to be cast for testing purposes. Each mixture contained a different combination of aggregate type, cementing materials, and target strength. Slab identifications were developed using the abbreviations for aggregate types and SCMs found in Table 3-1 and 3-2, respectively. An example of a typical slab identification is shown in Figure 3-6. Where fly ash was used, a 20% replacement by weight of portland cement was used. Where slag cement was used, a 50% replacement by weight of portland cement was used. Where Type I portland cement is specified, the only cementing material used in the concrete was portland cement.

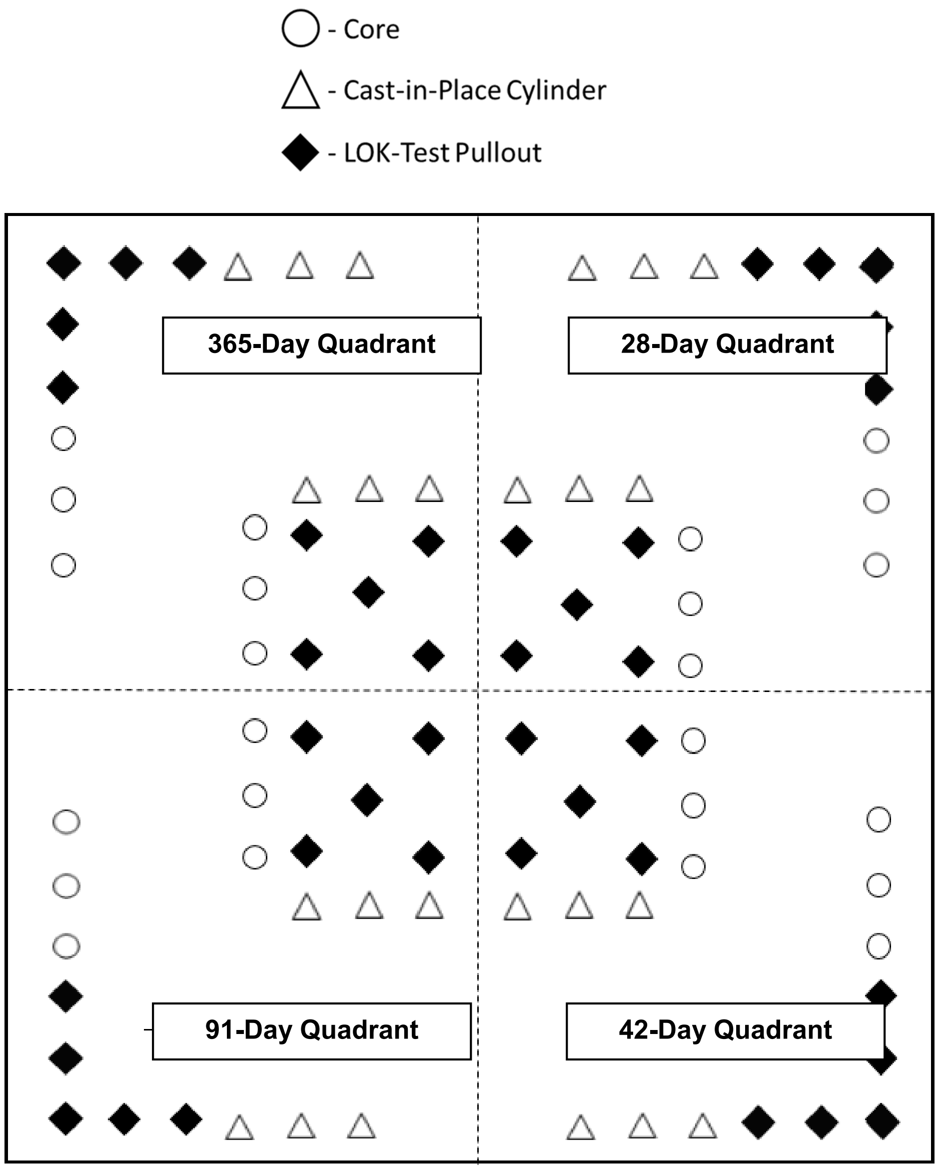

Figure 3-4: Typical Slab Layout (Not to Scale)

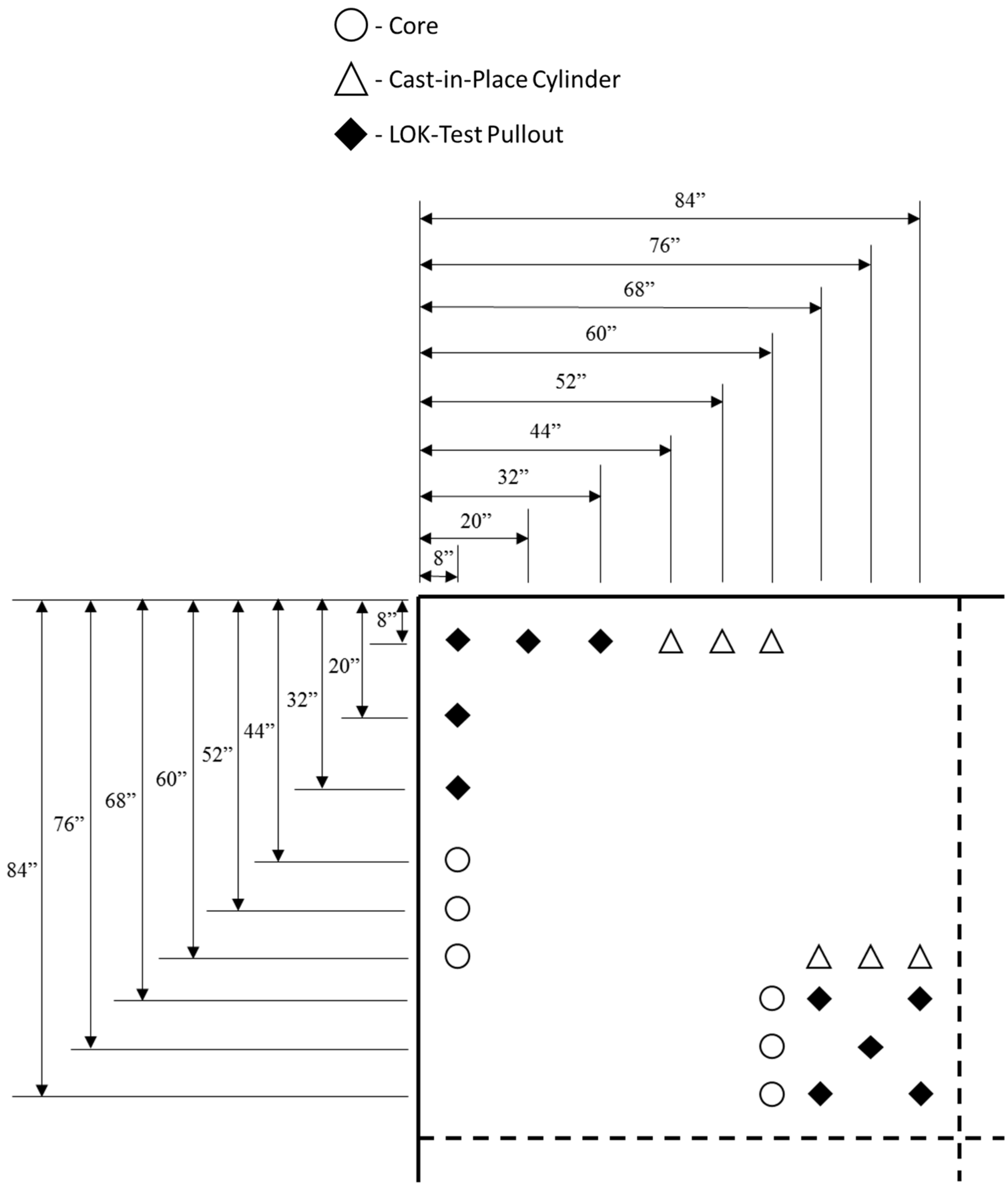

Figure 3-5: Testing Layout within a Typical Quadrant

Table 3-1: Abbreviations for Different Aggregate Types

Type	Abbreviation
Granite	GR
Limestone	LS
River Gravel	RG

Table 3-2: Abbreviations for Different Supplementary Cementing Materials

Type	Abbreviation
Type I Portland Cement	PC
Class C Fly Ash	CA
Class F Fly Ash	FA
Slag Cement	SC

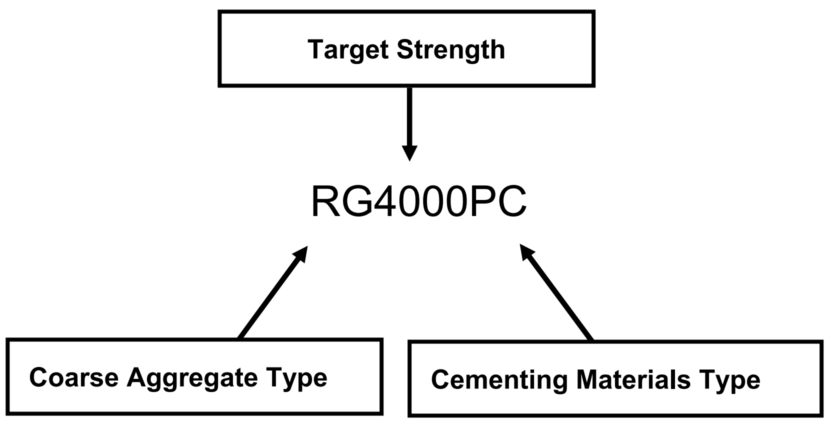

Figure 3-6: Typical Slab Identification

Mixture proportions for the normal-strength slabs were provided by Twin Cities Concrete of Sherman Industries in Auburn, AL. Mixture proportions for high-strength specimens were developed through trial batching. The target *average* strength of all the normal-strength slabs was 4,000 psi while the target strength of the two high-strength slabs was 8,000 psi. SSD batch weights per cubic yard can be seen in Tables 3-3 and 3-4.

Table 3-3: SSD Batch Weights for High-Strength Slabs

Component	RG8000PC	LS8000PC
Coarse Aggregate SSD (lb/yd^3)	1800	1800
Fine Aggregate SSD (lb/yd^3)	1130	1130
Type I Portland Cement (lb/yd^3)	950	950
Water (lb/yd^3)	266	266
Water-Cement Ratio	0.28	0.28
Glenium 7500 HRWR (oz/cwt)	6	5
Delvo Stabilizer (oz/cwt)	-	6

Table 3-4: SSD Batch Weights for Normal-Strength Slabs

Component	RG4000PC	LS4000PC	GR4000PC	RG4000CA	RG4000FA	RG4000SC
Coarse Aggregate SSD (lb/yd^3)	1880	1880	1880	1900	1900	1900
Fine Aggregate SSD (lb/yd^3)	1225	1225	1225	1105	1105	1105
Type I Portland Cement (lb/yd^3)	560	560	560	461	461	290
Class C Fly Ash (lb/yd^3)	-	-	-	115	-	-
Class F Fly Ash (lb/yd^3)	-	-	-	-	115	-
Grade 100 Slag (lb/yd^3)	-	-	-	-	-	290
Water (lb/yd^3)	280	280	280	268	268	268
Water-Cementitious Materials Ratio	0.50	0.50	0.50	0.46	0.46	0.46
WRA (oz/cwt)	3	3	3	3	3	3
Air (oz/cwt)	0.5	0.5	0.5	0.3	0.3	0.3

From the proportions in Tables 3-3 and 3-4, it can be seen that the water-to-cement ratio for normal-strength slabs containing only portland cement was 0.50 while the water-to-cementing materials ratio for normal-strength slabs containing SCMs was 0.46. These two water-to-cementitious materials ratios are higher than what ALDOT would typically allow, because the goal was to obtain concrete with low strengths typical of elements that would be cored when ordering 4,000 psi concrete. The water-to-cement ratio for both of the high-strength slabs was 0.28. The concrete was batched at a plant and delivered to site in a ready-mixed concrete truck.

3.3 SITE PREPARATION

On the selected site, two areas measuring 15 ft × 60 ft were selected and cleared of all brush, trees, and debris. Each area was designed to accommodate 4 slabs measuring 15 ft ×15 ft each. Number 57 crushed limestone aggregate was laid as a sub-base.

Treated pine 2x10s were then used as forms for each casting. Wooden stakes were used on the exterior of the forms to keep them from bowing during casting due to the fluid pressure of the concrete. The cast-in-place cylinder molds were placed in the correct spots within the forms and secured with number 3 standard rebar. Form release agent was applied to the inside of the forms to make form removal easier. Forms were removed from the slabs at 7 days after concrete placement along with all curing materials. A typical setup before casting a slab can be seen in Figure 3-7. Since slabs were cast directly adjacent to each other, fiberboard was bonded to the shared edge of two slabs to prevent bond and restraint between two slabs.

Figure 3-7: Typical Setup before Placement

3.4 IBUTTON TEMPERATURE SENSORS

iButton temperature sensors were used to record the temperature history of all slabs as well as moist-cured cylinders. For each concrete placement, a total of eight iButton temperature sensors were used. Two sensors were placed near the edge of the slab to collect temperature data to reflect the temperature of the area near the exterior testing locations. The sensors were tied to wooden stakes that were hammered securely into the ground. One sensor was placed at approximately the mid-height of the slab while the second was placed near the surface of the slab. Two more sensors were placed in the interior of the slab in a similar fashion.

The other four temperature sensors were placed in cylinders to monitor their temperature development. Two sensors were placed in a 6×12 in. cylinder while the other two sensors were placed in a 4×8 in. cylinder. Similar to the slabs, each of the two cylinders had one sensor placed near its mid-height and the other sensor placed near its surface. These cylinders were used strictly to collect temperature data for the molded cylinders from the placement and were not tested in compression.

For the first seven days after casting, temperature data were recorded every 15 minutes. Temperature data was collected from 7 to 91 days using a time interval of 90 minutes. After 91 days, the data were collected every 255 minutes until the slab and cylinders were 365 days old.

3.5 CASTING

Ready-mixed concrete from Twin City Concrete (ALDOT Vendor Code 408) located in Auburn, Alabama was used for all casts. For slabs having average targeted strengths of 4,000 psi, mixture proportions supplied by Twin City Concrete were used. For slabs using high-strength concrete, mixture proportions were supplied to Twin City Concrete for batching. Once the truck arrived on site, the slump and total air content were measured. If necessary, admixtures were added to the mixture to achieve slump within a target range of 3 to 5 in. for normal-strength concrete and 5 to 9 in. for high-strength concrete. Once these values were recorded, casting began. A wheelbarrow was filled with concrete for the making of the molded cylinders and cubes. As concrete was placed in the forms, the cast-in-place cylinders were filled by shovel to ensure uniformity between the surrounding concrete and the cast-in-place specimens. Internal vibration was applied to the slab using a conventional vibrator.

Cast-in-place cylinders were vibrated externally by touching the vibrator to the exterior of the support system, which held the molds in place. Once the slab forms were filled, the top of the slab was struck off. After this, the surface was finished with a bull float. After final finishing was applied, the pullout inserts were placed at their appropriate locations within the slab. Blemishes around the inserts were smoothed out with handheld metal trowels. A completed slab prior to the application of curing can be seen in Figure 3-8.

Figure 3-8: A Completed Slab

3.6 FINISHING AND CURING METHODS

After the concrete had reached initial set, wet cotton mats were used to cover the slab. Soaker hoses were connected to a large water tank through a timer, which allowed water to flow slowly through the soaker hoses every 12 hours for a 1-hour interval to provide adequate moisture to the cotton mats during curing. The cotton mats with soaker hoses can be seen in Figure 3-9 and the entire curing system can be seen in Figure 3-10. Large sheets of plastic were placed over the cotton mats and soaker hoses to avoid as much evaporation as possible. After seven days, the plastic, soaker hoses, cotton mats, and forms were removed from the slab.

Figure 3-9: Curing Mats and Soaker Hoses

Figure 3-10: Complete Curing System

3.7 MOLDED CYLINDERS

For each cast, sixteen 6×12 in. and sixteen 4×8 in. cylinders were made and tested in compression. Both sizes of molded cylinders were cast in order to determine if there was a difference in apparent strength due to cylinder size. Three cylinders of each size were tested at 7, 28, 42, 91, and 365 days. Temperature sensors were placed in separate cylinders of each size in order to determine the temperature history of the cylinders. Molded cylinders were made in accordance with AASHTO T23 (2009).

After the molded cylinders had been made, they were placed in temperature-controlled containers filled with water for at least 24 hours, but no more than 48 hours. During this period, the cylinders were kept within the required temperature range stated in AASHTO T23 (2009). For specimens with anticipated 28-day strength less than 6000 psi, this required temperature range is 60-80°F; for specimens with anticipated 28-day strength greater than 6000 psi, this required range is 68-78°F. An example of the cylinders during their initial-curing state is shown in Figure 3-11.

Figure 3-11: Initial Curing of Molded Cylinders

After the initial curing stage, the specimens were retrieved from the test site and brought back to the Auburn University Civil Engineering Materials Laboratory. Care was taken so that the specimens were not damaged while being transported back to the laboratory. Special cylinder holders were made in order to ensure that the specimens were transported safely. Cylinders were then removed from their molds, labeled, and placed in the moist-curing room where they remained until they were sulfur capped and tested. The cylinders were sulfur capped the day before being tested for compressive strength. After being capped, the cylinders were again placed in the moist-curing room until tested in compression in accordance with AASHTO T22 (2009).

3.8 PULLOUT CALIBRATION CUBES

A test method was developed to validate the calibration table for the LOK-Test pullout inserts. Cubes (8×8×8 in.) were constructed, and pullout inserts were attached to the four inside vertical walls of the cubes using small bolts, which threaded into the pullout inserts. As per the manufacturer's recommendation, Type L-45 inserts were used for concrete with expected 28-day strength of 4000 psi, while Type L-46 inserts were used for high-strength concrete. The cubes were formed using the same procedure as molded cylinders. The cubes were made using three equal lifts. The number of rods per lift was determined using the same rod-to-area ratio as molded cylinders. Each lift was rodded 56 times and tapped with a rubber mallet.

After initial set, the cubes were placed inside a shed with a window-mounted air conditioner to control the curing temperature. Wet burlap was placed over the cubes and then covered with plastic sheeting to prevent moisture loss. After a waiting period of 24-48 hours, the cubes were transported back to the Auburn University Materials Laboratory along with the molded 6×12 in. and 4×8 in. cylinders. After being removed from their respective molds, the cubes were labeled accordingly and placed into the moist-cure room until being tested at their specified ages. During testing, a constant load rate of 70 ± 30 kPa/s was applied to each insert with a pullout testing machine as specified by ASTM C900 (2007). The pullout insert was loaded until failure. Once failure occurred, the maximum gage reading was recorded. The pullout calibration tests were conducted at 28, 42, and 365 days. In order to evaluate the accuracy of the tables supplied by the manufacturer, 6×12 in. and 4×8 in. molded, moist-cured cylinders were tested in compression at the same age as when the calibration pullout tests were conducted.

3.9 CORES

For each slab, 4 in. diameter cores were retrieved and tested in compression to establish a relationship between molded, moist-cured cylinder strength and in-place strength as measured by core testing. Six cores were taken from each age-specific quadrant of each slab. Three cores were taken a distance of 8 in. on-center from the edge of the slab, which represented the low restraint condition, and three cores were recovered from the interior region of the slab that represented the high restraint condition. A core being extracted from the exterior zone of the slab can be seen in Figure 3-12.

Cores were retrieved from their slabs one week prior to testing. Once cores were drilled, they were removed from the slab using a core snap. After removal, the cores were wiped dry, and all surface water allowed to evaporate. After initial lengths were recorded and the surface water had evaporated, the cores were placed inside two sealable plastic bags, secured with rubber bands as required in AASHTO T24 (2009). The cores were then transported back to the Auburn University Structures Laboratory and trimmed to a length-to-diameter ratio of 2.0 using a wet saw. After being trimmed, the cores were again wiped off to remove all excess surface water. The cores were then re-sealed back in plastic bags and wrapped with rubber bands after all surface water had evaporated as per AASHTO T24 (2009). The cores were kept sealed in their plastic bags for 6 days until sulfur capped. After being sulfur capped, the cores were placed back in sealed plastic bags and wrapped with rubber bands until tested in compression the following day. AASHTO T231 (2009) requires that the sulfur caps of specimens whose expected strength is less than 5,000 psi must harden for at least two hours before the specimen is tested. AASHTO T231 (2009) also requires that the sulfur caps of specimens whose expected strength is greater than or equal to 5,000 psi must harden for at least 16 hours before the specimen is tested. Testing occurred 7 days after removal from the slab, which meets requirements for both concrete strength levels set forth in AASHTO T24 (2009). Before being tested, the average diameter and capped length were recorded. Following the test, the time of the test and peak compressive force were recorded, and compressive strength was calculated based on the cross-sectional area of the core calculated from the average diameter. Cores were tested at concrete ages of 28, 42, 91, and 365 days.

Figure 3-12: Drilling of a Core from the Exterior Region of a Slab

3.10 CAST-IN-PLACE CYLINDERS

Each slab also contained cast-in-place cylinders, which measured 4 in. in diameter and 8 in. in height, that were retrieved from the slab and tested in compression. The purpose of cast-in-place cylinders is to obtain a compression strength from a molded specimen that has not been damaged due to coring and microcracking; however, these specimens have similar temperature and moisture conditioning as the in-place concrete. Cast-in-place cylinders were tested at 28, 42, 91, and 365 days. A relationship was then established relating the strengths measured from the cast-in-place cylinders to the strengths measured from the corresponding molded cylinders. A set-up had to be developed in order to hold the cylinders in place during casting. To do this, 2x6 boards were cut to specified lengths. A hole-saw was used to cut a series of 4 in. diameter holes approximately 1 in. into the surface of the board. For the first three concrete placements, sheet metal was cut and then wrapped into cylinders, which fit into the 2×6 and held the plastic cylinders molds in place. It was determined that sheet metal was not ideal for this purpose because the confining pressure of the surrounding concrete made it extremely difficult to remove the molds from the hardened slab. Because of this, a new system was developed for the remaining five slabs. Steel pipe with an outside diameter of 4½" and an inside diameter of 4 3/8" was used. This provided a more rigid system, which was less deformable under the confining pressure of the surrounding concrete. After being cut to the correct length, the pipe was placed in the same 2×6 board system described above. The cylinders were filled manually by shovel as the slab was being cast. During casting, consolidation was achieved by touching the vibrator to the outside of the support system as per ASTM C873 (2011). After casting, the cast-in-place cylinders were retrieved from their slabs one week before being tested in compression.

To ensure the same moisture conditions, the same procedure was followed for the cast-in-place cylinders as was used for the cored specimens. Once retrieved from their specified locations within the slab, the cast-in-place cylinders were stripped from their plastic molds, placed in two sealed plastic bags, and wrapped with rubber bands. At this point, the cast-in-place cylinders were transported back to the Auburn University Materials Laboratory to be trimmed. Only the tops of the cast-in-place cylinders that had been exposed while within the slab were trimmed. Only a minimal amount was trimmed from the top to ensure that the length-to-diameter ratio was as close to 2.0 as possible. After being trimmed, the cast-in-place cylinders were again placed in two sealed plastic bags and wrapped in rubber bands for six days until being sulfur capped. On the day before testing, the cast-in-place cylinders were removed from their bags and sulfur capped. AASHTO T231 (2009) requires that the sulfur caps of specimens whose expected strength is less than 5,000 psi must harden for at least two hours before the specimen is tested. AASHTO T231 (2009) also requires that the sulfur caps of specimens whose expected strength is greater than or equal to 5,000 psi must harden for at least 16 hours before the specimen is tested. Testing occurred 7 days after removal from the slab, which meets requirements for both strength levels as per AASHTO T24 (2009). Once capped, the specimens were again placed back in their bags and wrapped with rubber bands. Before being tested, the average diameter and capped length were recorded. After compression testing, the time at failure as well as the peak compressive failure load were recorded. The strength of the specimen was based on the area calculated using the average measured diameter.

3.11 PULLOUT INSERTS

LOK-Test pullout inserts were placed into each slab just after bull floating of the fresh concrete. As recommended by the manufacturer, L-49 floating inserts were used in slabs with an expected 28-day strength of 4000 psi while L-50 floating inserts were used in high-strength concrete slabs. L-50 inserts are used for testing high-strength concrete because they have a thicker head to withstand a higher pullout force. Typical L-49 and L-50 inserts can be seen in Figure 3-13, and the placement of L-49 inserts into the exterior region of a slab can be seen in Figure 3-14.

Figure 3-13: Typical L-50 (Left) and L-49 (Right) Inserts

Figure 3-14: Placement of L-49 LOK-Test Pullout Inserts Into a Slab

Ten pullout inserts were placed in each quadrant of the slab for a total of 40 inserts per slab. In each zone, five of the ten inserts were placed 8 in. on-center from the edge of the slab in the exterior region, which has the least amount of restraint to movement. The remaining five inserts were placed in the interior region of the slab where the restraint to movement was much higher than the exterior region. During testing, a constant load rate of 70 ± 30 kPa/s was applied by the pullout machine as specified by ASTM C900 (2007). The concrete was loaded until failure. Once failure occurred, the maximum gage reading was recorded. An example of a pullout test being performed can be seen in Figure 3-15.

Figure 3-15: Pullout Test Being Performed

3.12 Raw Materials Summary

3.12.1 Coarse Aggregate and Fine Aggregate

Over the course of the project, three different types of aggregates were used: crushed limestone, uncrushed river gravel, and crushed granite. All aggregates used were a No. 67 gradation. The crushed limestone came from APAC Midsouth in Opelika, Alabama (ALDOT ID Number 1604), while the uncrushed river gravel was from the Foley Materials Company in Shorter, Alabama (ALDOT ID Number 1481), and the crushed granite was from the Columbus Quarry, LLC in Fortson, Georgia (ALDOT ID Number 0135). All natural sand used for the project was from the Foley Materials Company in Shorter, AL (ALDOT ID Number 1481). The properties of the coarse and fine aggregates are summarized in Table 3-5. Gradations were performed on all coarse aggregate sources to verify that all were within the requirements for No. 67 gradation specified in AASHTO M 43 (2009). The coarse aggregate gradation results can be seen in Figure 3-16. All coarse aggregates used throughout the duration of the project met the gradation requirements of AASHTO M43 (2009).

Table 3-5: Coarse and Fine Aggregate Properties

Aggregate	Bulk Specific Gravity (SSD)	Absorption Capacity (%)
Crushed Limestone	2.84	0.20
Uncrushed River Gravel	2.63	0.40
Crushed Granite	2.68	0.40
Sand	2.64	0.40

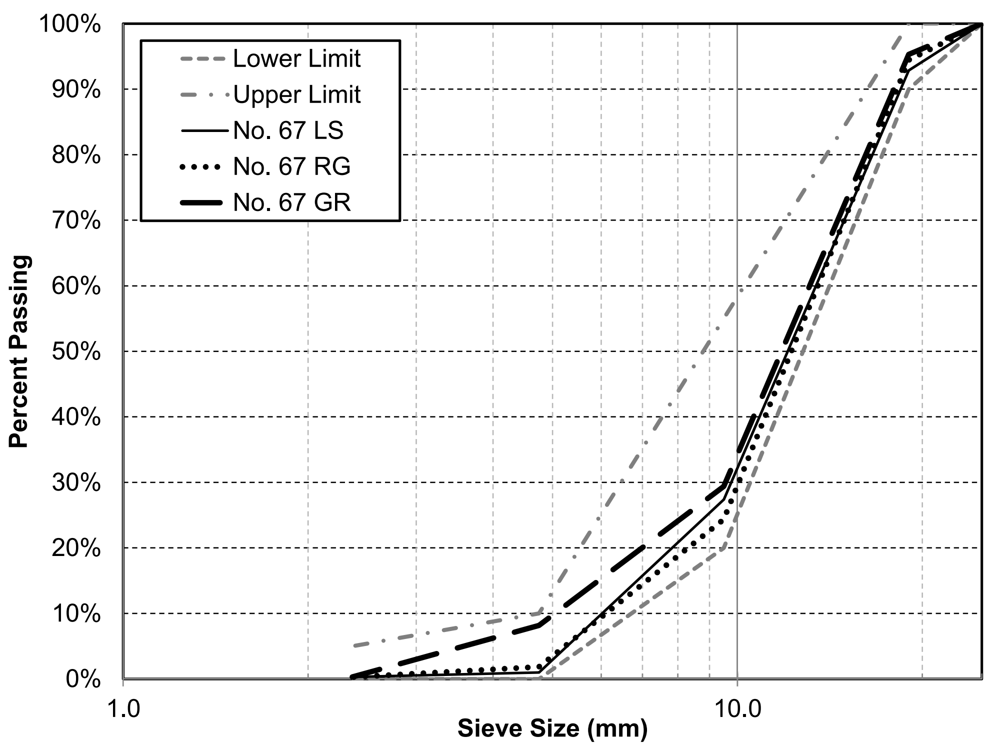

Figure 3-16: Coarse Aggregate Gradations

3.12.2 CEMENT AND SUPPLEMENTARY CEMENTING MATERIALS

In today's concrete industry, it is very common that supplementary cementitious materials (SCM) are used in concrete production in order to reduce cost, increase performance, and/or provide a more sustainable option. In this study, several different SCMs were used in casting the experimental slabs in order to determine if a difference in strength gain was present.

3.12.2.1 TYPE I CEMENT

Type I cement was used in all the slabs of this project. The source of the cement was Lehigh Portland Cement Company located in Leeds, Alabama (ALDOT Vendor Code 140).

3.12.2.2 CLASS C FLY ASH

Class C fly ash was used in the RG4000CA slab as a 20% replacement for portland cement. The source of the Class C fly ash was Holcim in Quentin, Alabama.

3.12.2.3 CLASS F FLY ASH

Class F fly ash was used in the RG4000FA slab as a 20% replacement for the portland cement. The source of the Class F fly ash was Boral Industries in Cartersville, Georgia.

3.12.2.4 SLAG CEMENT

Slag cement was used in the RG4000SC slab as a 50% replacement for portland cement. The source of the slag cement was Holcim in Birmingham, Alabama.

Chapter 4
Presentation of Results

4.1 INTRODUCTION

Throughout the course of the project, many variables were evaluated to determine their effect on the in-place strength of concrete. In addition, different methods were used to assess the in-place strength. Compressive strength data were collected on each of the eight test slabs over the course of one year in order to determine these effects. In-place testing was performed at 28, 42, 91, and 365 days in order to develop a relationship between the age of the concrete being tested and in-place strength. The factors examined in this field project were the effect of concrete strength level, concrete age, restraint (mircocracking), supplementary cementing materials, aggregate type, and strength test type. Six of the slabs were normal-strength concrete and the other two slabs were made with high-strength concrete.

Relationships were evaluated between in-place testing methods and 6×12 in. molded moist-cured cylinders. In addition to developing relationships between in-place testing and 6×12 in. cylinder strength, 4×8 in. molded cylinders were also tested as this matches the dimensions of the cores tested in this project. The cast-in-place cylinder data from casts LS4000PC, RG4000PC, and RG4000CA were not considered in the analysis due to the difficulties encountered while removing some of these specimens from the slabs. All statistical analyses were performed with a 95% confidence level unless otherwise specified.

4.2 TEMPERATURE DATA

It has long been known that the temperature of the freshly placed concrete has a significant impact on the hardened properties of the concrete. Increased temperatures during placement can provide numerous challenges to the party placing the concrete, such as decreased workability, decreased set time, and drying and thermal shrinkage cracking. Temperature also has a significant impact on the rate of strength gain within concrete. Mehta and Monteiro (2014) state that the higher the placement temperature, the more rapid the gain of strength which, if placed correctly, typically increases early-age strength, but has a negative effect on long-term strength. During this project, the placement temperature was monitored for each cast. The fresh concrete temperature was taken upon arrival of the ready-mixed concrete truck using a thermometer and recorded, while the ambient temperature data for the day was collected from the Auburn-Opelika Airport from Weather Underground (2014). These temperature data can be seen in Table 4-1.

Table 4-1: Temperature Data for All Casts

Cast	Date	Maximum Air Temperature (°F)	Mean Air Temperature (°F)	Low Air Temperature (°F)	Fresh Concrete Temperature (°F)
RG4000CA	7/2/2013	84	76	68	93
LS4000PC	7/11/2013	81	76	70	86
RG4000PC	8/1/2013	88	80	72	89
RG4000SC	9/3/2013	88	80	72	88
GR4000PC	9/12/2013	88	77	66	86
RG8000PC	9/24/2013	70	67	64	86
RG4000FA	10/15/2013	73	65	57	85
LS8000PC	7/17/2014	82	72	61	94

4.3 EFFECT OF CYLINDER SIZE

Since only 4 in. diameter cores were tested for this project, 4×8 in. molded cylinders were tested in addition to 6×12 in. molded cylinders. Testing both to 4×8 in. and 6×12 in. molded cylinders allowed the research team to determine if the strength of the smaller cylinders (and thus cores) any different from the strength of the larger 6×12 in. molded cylinders.

In this study, three cylinders of each size were tested at 28, 42, 91, and 365 days for each slab that was placed to determine if cylinder size affected average cylinder strength. The average cylinder strength for each set of molded, moist-cured cylinders was calculated. The relationships between the 6×12 in. and 4×8 in. cylinder strengths from the normal-strength concrete and high-strength concrete were compared against each other to see if there was a statistically significant difference. To do this, the average compressive strength of the 6×12 in. molded cylinders was divided by the average compressive strength of the 4×8 in. molded cylinders to obtain a ratio of the average compressive strengths. A t-test assuming equal variance was then conducted between the ratios of the normal-strength and high-strength cylinders. This resulted in a P-value of 0.084. Since this value is greater than 0.05, this indicates that there was not a statistically significant difference in the relationship between the 6×12 in. and 4×8 in. cylinders for normal-strength and high-strength concrete at a confidence level of 95 %.

Once this was known, the data sets were combined to analyze if there was a statistically significant difference between the average strengths of the 6×12 in. and the 4×8 in. cylinders. A plot for the average compressive strengths of the 6×12 in. versus the 4×8 in. cylinders can be seen in Figure 4-1. A paired t-test was performed comparing the average strength of the 6×12 in.

cylinders versus the average strength of the 4×8 in. cylinders. This resulted in a P-value of 0.633. Since this value is higher than 0.05, it was concluded that there is no statistical difference between the compressive strengths obtained from 6×12 in. and 4×8 in. molded, moist-cured cylinders at a 95 % confidence level.

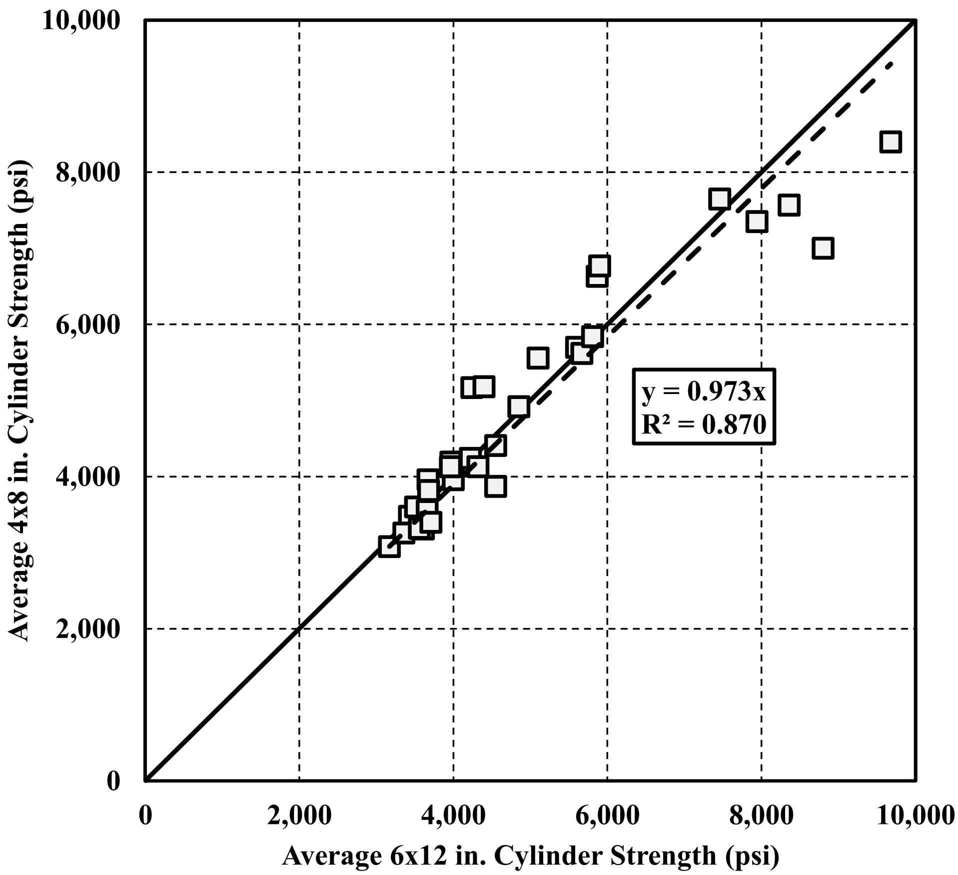

Figure 4-1: Comparison of the Average Strengths of 6×12 in. Cylinders versus 4×8 in. Cylinders

It can be seen in Figure 4-1 that there is a clear trend between the average strengths of the 6×12 in. and 4×8 in. molded, moist-cured cylinders. This indicates that cylinder size does not have an effect on the average strength produced by the test. This is in agreement with the findings of Day and Haque (1993). This also indicates that when the strengths of cores and molded, moist-cured 6×12 in. cylinders are compared, any difference that is found in strength cannot be attributed to the physical difference in size of the specimens.

4.4 VERIFICATION OF PULLOUT TABLE PROVIDED BY GERMANN INSTRUMENTS

For each time the LOK-Test pullout machine is calibrated, Germann Instruments provides a table that gives values for compression strength based on the pullout force. This table converts the pullout force in kilonewtons (kN) and equates it to a specified compressive force in pounds per square inch (psi). In order to be assured of the accuracy of the table, testing was done to evaluate if the table provided an accurate conversion from pullout strength to equivalent cylinder strength. In order to do this, six cubes measuring 8×8×8 in. were cast with four pullout tests in each cube as described in Section 3.10. A total of eight calibration pullout tests were performed at each testing age of 28, 42, and 365 days. The eight pullout readings were each converted into an equivalent compressive strength. These equivalent compressive strengths were then averaged to obtain an average pullout compressive strength. Similar to the analysis between the 6×12 in. and 4×8 in. cylinders, it was desired to combine the data into one set containing data from both the normal-strength and high-strength cubes. To do this, the average 6×12 in. molded cylinder strength was divided by the average compressive strength of the cube pullouts. A t-test assuming equal variance was then used to determine if there was a difference in the relationship between the 6×12 in. cylinders and the pullout calibration cubes for the normal-strength and high-strength concretes. This t-test resulted in a P-value of 0.414, which indicates that there was no difference between the cylinder-cube relationship and that the two data sets could be combined. A scatter plot of average calibration pullout strength versus 6×12 in. cylinder strength was created in order to fit a trend line through the data points to verify the accuracy of the calibration table provided by Germann Instruments. Figure 4-2 shows a plot of average 6×12 in. molded cylinder strength versus average compressive strength obtained from the calibration pullout tests.

Once the data were combined, a paired t-test was done between the average strength of the 6×12 in. cylinders and the pullout calibration cubes. This paired t-test resulted in a P-value of 0.104. This meant that the pullout calibration chart supplied by Germann Instruments could be used without any adjustments to the values. As can be seen in Figure 4-2, the average of the cube pullout strengths was approximately 97% of that of the 6×12 in. cylinders. This was determined to be within the range of error for the pullout tests and therefore the strengths provided in the calibration table from Germann Instruments were used.

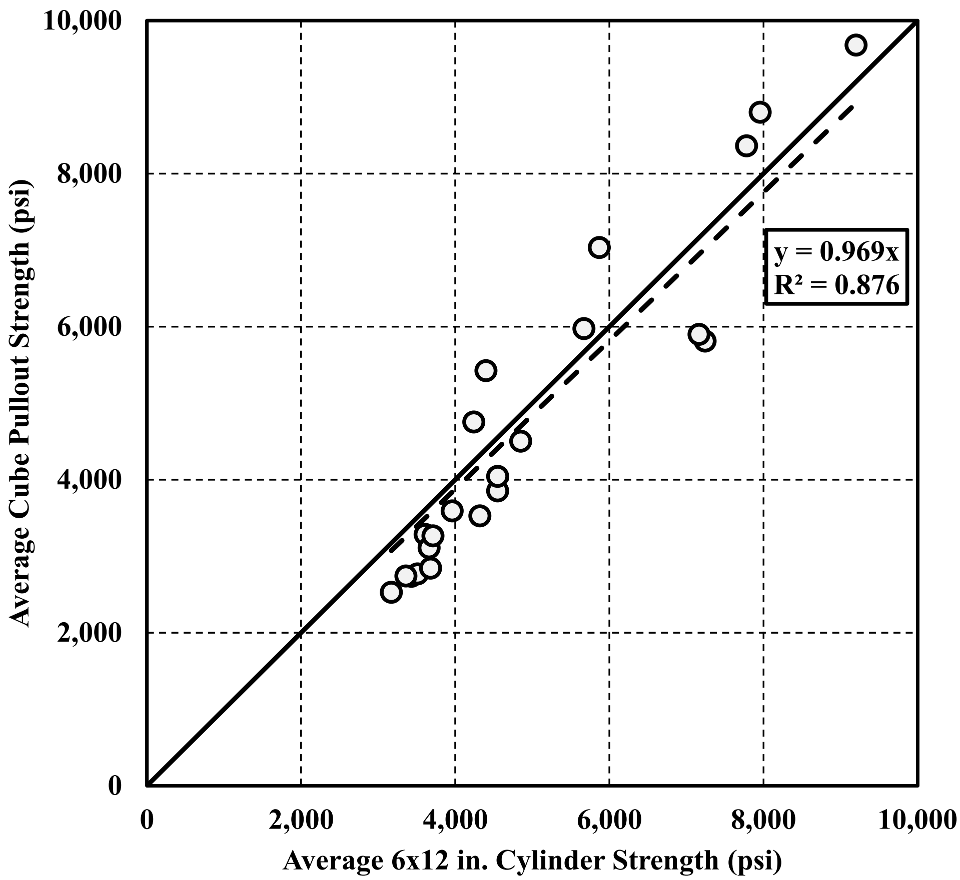

Figure 4-2: Average 6×12 in. Cylinders versus Average Calibration Pullout Cube Strength

4.5 EFFECT OF AGGREGATE TYPE

In today's concrete industry, many different types of coarse aggregate are used. Most of the time, aggregate selection is most influenced by local availability. In east Alabama, several aggregate types are locally available. In this study, slabs containing uncrushed river gravel, crushed limestone, and crushed granite were used. A No. 67 gradation was used for all aggregates. The purpose of using different aggregate types was to determine if aggregate type influenced the apparent strength of cored specimens relative to the other means used to assess the strength. Specimens from exterior and interior regions were kept separate when performing the statistical analysis to determine if aggregate type influenced the difference in strength between in-place strength and cylinder strength for different restraint conditions.

4.5.1 EFFECT OF AGGREGATE TYPE ON STRENGTH OF 6 × 12 IN. VERSUS 4 X 8 IN. CYLINDER

In order to determine if aggregate type had a significant impact on the strength of 6×12 in. and 4x8 in. cylinders, a statistical analysis was performed. The average strength of 6×12 in.

cylinders from a given age and cast was divided by the average strength of the corresponding 4×8 in. cylinders in order to obtain a normalized value which would be comparable without being influenced by strength gain over time. After this was done, values were separated into groups based on coarse aggregate type. An analysis of variance (ANOVA) analysis was then performed on the data to determine if coarse aggregate type had an impact on the relationship between 6×12 in. cylinders versus 4×8 in. cylinders. The ANOVA analysis yielded a P-Value of 0.142. Since this value is less than 0.05, this shows that coarse aggregate type did not have a significant impact on the relationship between the compressive strengths of 6×12 in. versus 4×8 in. molded, moist-cured cylinders at a 95% confidence level.

4.5.2 EFFECT OF AGGREGATE TYPE ON IN-PLACE STRENGTH TEST METHOD

A statistical analysis was completed in order to determine if aggregate type influenced the apparent strength of different in-place testing methods. For this analysis, each in-place specimen strength was divided by the average 6×12 in. molded cylinder strength of the same age to normalize the strength gain present in a slab over time. After this, the normalized values were separated by aggregate type and an ANOVA analysis was done for each in-place test type. Specimens from exterior and interior regions were kept separate during this analysis in order to ensure that restraint did not have an effect on the outcome of the ANOVA analysis. A summary of the P-values from the ANOVA analyses are presented in Table 4-2.

Table 4-2: Summary of P-values from ANOVA Analysis for In-Place Testing

Test Method and Location	P-value From ANOVA Analysis
Exterior Cores	9.45×10^{-5}
Interior Cores	0.023
Exterior Pullouts	0.009
Interior Pullouts	1.80×10^{-7}

Since all P-values were less than 0.05, it was found in the normal-strength concretes that aggregate type was significant for both the exterior and interior specimens for cores as well as the pullout tests at a 95 % confidence level. An analysis was not performed on the cast-in-place cylinders as these data were not available for each type of aggregate used.

After this, the average ratios of in-place strength to molded cylinder strength were calculated for each test method. The results for normal-strength specimens can be seen below in Table 4-3.

Table 4-3: Summary of In-Place Strength to Molded Cylinder Strength Ratios by Coarse Aggregate Type for Normal-Strength Concrete

Core Type	RG	LS	GR
Exterior Cores	0.972	0.852	0.908
Interior Cores	0.919	0.861	0.837
Exterior Pullout	1.071	1.083	0.896
Interior Pullout	0.964	1.093	0.724

From this table, it can be seen that aggregate type had a different effect on each type of testing. It can be seen that the river gravel had the highest core-to-molded cylinder strength ratio while the limestone slab had the highest pullout-to-molded cylinder ratio. It should also be noticed that the in-place specimens from the granite slab had the lowest overall relative strength to their molded cylinder counterparts of the three aggregate types used in the study. Since only one granite and one normal-strength limestone slab were tested during the project, it would be recommended that more testing be done on similar limestone and granite specimens to obtain a more conclusive result.

4.6 Effect of Restraint

The location of an in-place test with respect to the member from which it is being performed has a significant effect on the amount of restraint that the concrete in that region experiences. In theory, concrete that is close to the edge of a member is less restrained against movement and therefore less prone to developing microcracks within the microstructure of the concrete. Conversely, concrete that is not near an edge of a member is more restrained and theoretically more likely to develop microcracks. Microcracking increases permeability and lowers concrete strength. Once microcracks are formed, they can develop into larger cracks, which can lead to failure. From this, it can be assumed that microcracking can have an effect on the apparent in-place strength. Therefore, if the restraint that the concrete is subjected to causes significant microcracking, then the amount of restraint present will have an effect on in-place strength.

In order to determine if restraint has an effect on in-place strength, in-place testing was conducted near the edge of each slab as well as in the middle of each slab. The testing near the edge of the slab represented the low restraint condition while testing at the middle of each slab represented the higher level of restraint condition. Since the cast-in-place cylinders were contained within metal sleeves which were used as the support system, it was anticipated that level of restraint would play a less significant role in the apparent strength of these specimens simply because the cast-in-place specimens had room to freely expand and contract.

A statistical analysis was completed on the collected data to determine if restraint had a significant effect on apparent in-place strength. In order to normalize all the data, all in-place test strengths were divided by the average molded 6×12 in. cylinder strength from that cast at the age it was tested. Once these ratios were obtained, equal variance t-tests were performed to determine if there was a difference in strength between the normal-strength and high-strength specimens with respect to the effect of restraint. To do this, separate equal-variance t-tests were conducted on the cylinder-to-in-place test strength ratios for both the exterior and interior specimens. The P-values for the equal-variance t-tests can be seen in Table 4-4.

Table 4-4: P-values for Strength Level t-test

Specimen Type	P-values for t-test Comparing in-place Specimens of Normal-Strength and High-Strength Concrete	
	Exterior	**Interior**
Cores	0.341	0.978
Cast-in-Place Cylinders	0.936	0.980
Pullout Tests	0.278	6.67×10^{-5}

From Table 4-4, it can be seen that all the P-values except one was above 0.05. Because of this, it was determined that the strength level did not have a significant impact on the effect of restraint. Because of this, the data from the normal-strength and high-strength casts were combined within each respective test type.

After this was done, an analysis was performed to determine if restraint influenced the relationship between apparent in-place strength and 6×12 in. molded, moist-cured cylinder strength. This was done by doing an equal-variance t-test between the in-place-to-molded cylinder strength ratios for each in-place testing type. The P-values for the t-tests can be seen in Table 4-5.

Table 4-5: P-values for t-tests determining the Effect of Restraint

Specimen Type	P-values for t-test
Cores	0.005
Cast-in-Place Cylinders	0.502
Pullout Tests	0.040

From the results of the equal-variance t-tests, it can be seen that both the cores and the pullout tests were effected by restraint while the cast-in-place cylinders were not. This result is as expected, because the cast-in-place cylinders have some room to freely expand and contract inside the sleeves that hold them in-place. Plots containing exterior versus interior strength data for cores and pullouts were also constructed to better show the relationship between the interior and exterior specimen results and can be seen in Figures 4-3 through 4-5.

From Figures 4-3 through 4-5, it can be seen that restraint has an impact on the apparent strength of the slabs for the cores as well as the pullout tests, but not for the cast-in-place cylinders. The strength of the interior cores averaged to be approximately 94.8 % of that of the exterior cores. Similarly, the strength of the interior pullouts averaged about 96.9 % of the exterior pullout strength. This was as expected because the middle specimens have a higher degree of restraint and therefore had a higher degree of microcracking, which owers the overall compressive strength of these in-place specimens. In an actual constructed element, the degree of restraint and thus microcracking would vary and it is plausible that more microcracking could be present that what was encountered in this research project.

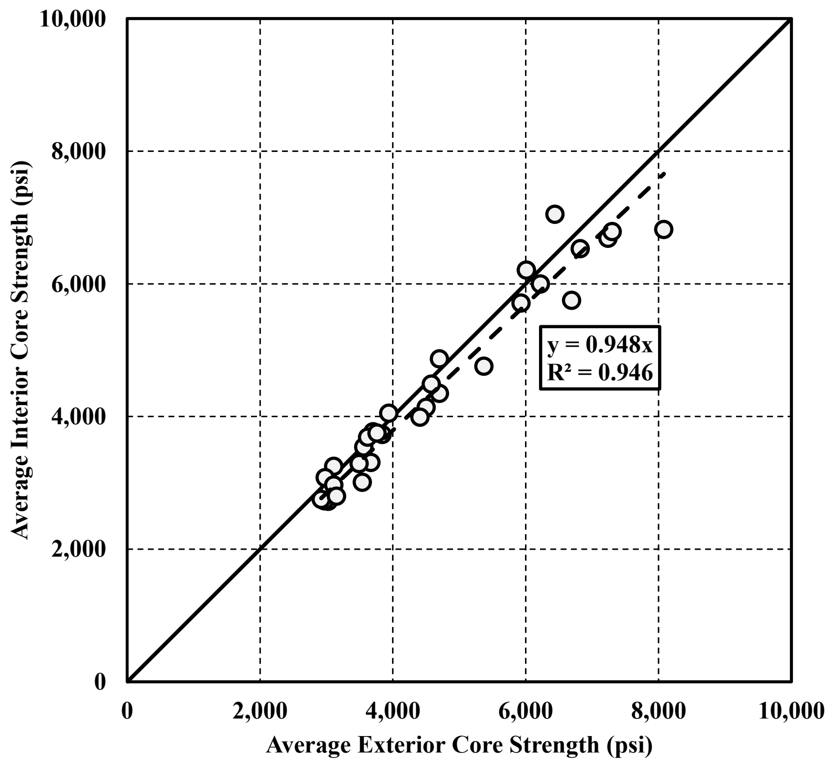

Figure 4-3: Average Exterior Core Strength versus Average Interior Core Strength

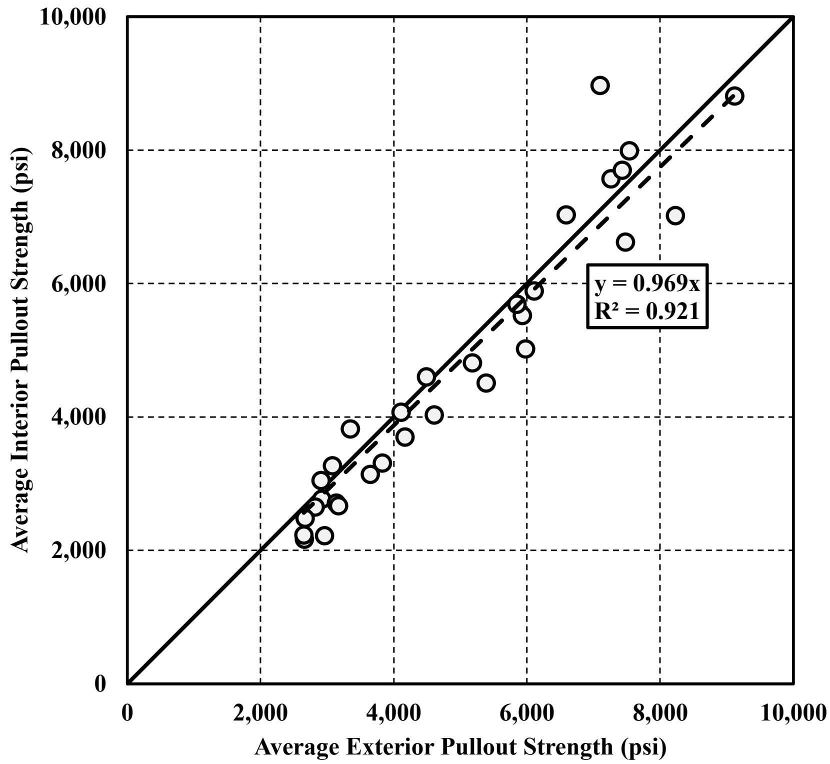

Figure 4-4: Average Strength of Exterior Pullout Tests versus Average Strength of Interior Pullout Tests

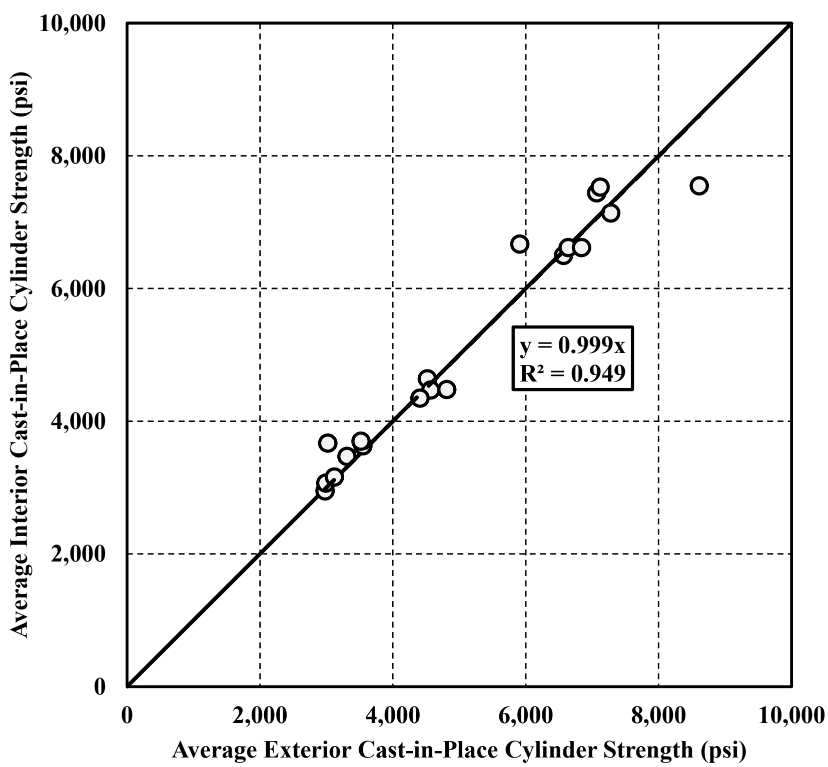

Figure 4-5: Average Strength of Exterior Cast-in-Place Cylinders versus Average Strength of Interior Cast-in-Place Cylinders

4.7 EFFECT OF SUPPLEMENTARY CEMENTING MATERIALS AND STRENGTH GAIN OVER TIME

Partial replacement of cement with SCMs is a common practice in today's concrete industry. Since these SCMs may have an effect on the strength development of in-place concrete, relationships must be developed in order to establish age correction factors that can be applied to in-place specimens test results at any age to convert them to a representative 28-day strength. In order to evaluate the in-place strength from core testing, the core strength must be converted to a 28-day core strength, and then be compared with 6×12 in. molded cylinder strength.

ACI 209.2R (2008) also presents a method for correcting concrete strength results to account for strength gain over time. This method uses Equation 4-1 with differing constant values for different cement types. The constants for different cement types can be seen in Table 4-6. No consideration is given to partial replacement of cement using SCMs. Yazdani and McKinnie (2004) developed different equations to correct for the strength gain in in-place specimens based on *moist-cured*, molded cylinders, which can be seen in Section 2.2.1. These equations, which are now used in practice by the Florida DOT, produce age-correction factors for different concretes that contain different SCMs as well as different cement types.

$$f'_c(t) = f'_c(28) \times \left(\frac{t}{a+\beta\times t}\right) \qquad \text{Equation 4-1}$$

Where:

t = time since casting (days)

a = empirical constant from Table 4-5 (unitless)

β = empirical constant from Table 4-5 (unitless)

Table 4-6: Constants for ACI 209 Age Correction Equation

Cement Type	**ACI 209 Empirical Constants for Equation 4-1**	
	a	**β**
Type I	4	0.85
Type III	2.3	0.92

As an example of the time dependent data, the strength development results for the slag cement slab is shown in Figure 4-6. Since it was found that restraint does have an impact on in-place strength, only the interior specimen trend lines are shown in Figure 4-6. The strength gain model of ACI 209.2R (2008) as well as the strength gain the model for slag cement developed by Yazdani and McKinnie (2004) are also shown on Figure 4-6. It is clear from this figure that the strength gain of the cores are significantly different from the molded cylinders that were moist-cured in the laboratory.

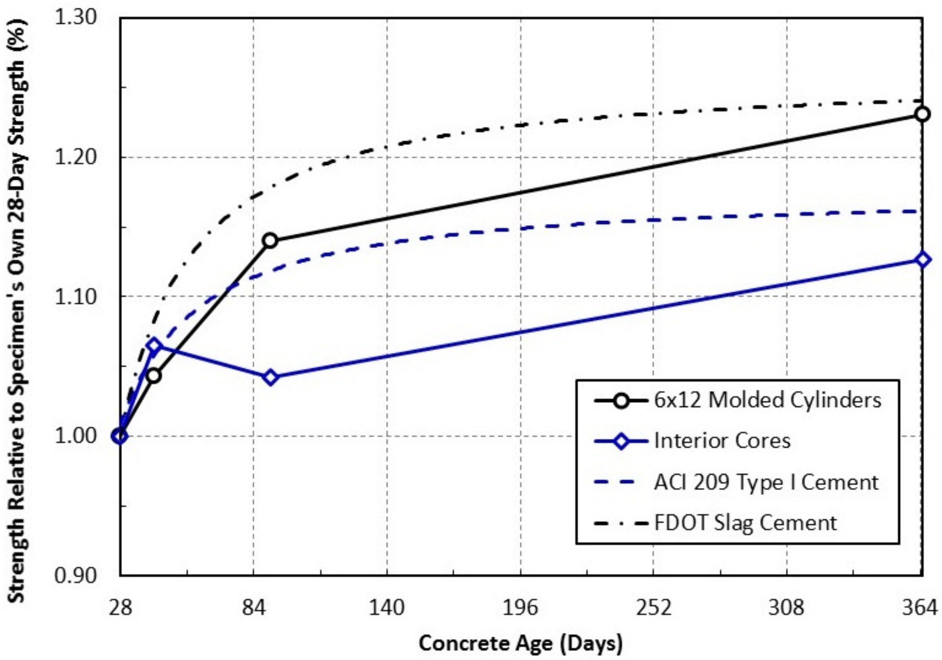

Figure 4-6: Strength Gain of 6×12 in. Cylinders and Cores of 50% Slag Cement Concrete versus the Strength Gain Predicted by ACI 209 and FDOT

A statistical analysis was performed on the data to determine the effect that the SCMs had on the overall strength gain over the first year after placement. For both the cores and pullouts as well as the 6×12 in. molded cylinders, an ANOVA analysis at a 95% confidence level was completed to determine if the difference in cementing materials and moist versus field curing caused a difference in strength gain over time. The results of the ANOVA analyses can be seen in Table 4-7. It can be seen from the ANOVA analysis results that since none of the P-values are less than 0.05, the post 28-day strength gains of all the different types of cementing materials were not significantly different from one another.

Table 4-7: Summary of P-Values from ANOVA Analysis of Strength Gain for Different SCMs

Specimen Type	P-Value from ANOVA Analysis
6x12 Cylinders	0.804
Interior Cores	0.229
Interior Pullouts	0.755

After the ANOVA analyses were run comparing all the cementing materials against one another, three separate analyses were done to see if there was a statistical difference present between the slab that contained only portland cement and each of the slabs that contained

SCMs. This was done in order to determine if there was a statistical difference between portland cement and each individual type of SCM. To do this, a paired t-test was conducted on the ratios of strength at a given time to the strength at 28 days. The results from these paired t-tests can be seen in Table 4-8. Since none of these P-Values was less than 0.05, it was concluded that the data collected for the project show that strength gain was not statistically different between the slab containing only Type I portland cement and the other slabs that contained SCMs. Because of this, one model could be used to represent the strength gain regardless of cementing material type.

Table 4-8: P-Values of Paired t-tests Conducted on Portland Cement Specimens Versus Specimens Containing Supplementary Cementing Materials

Test Type	Comparison	P-Value from t-test
6x12 Cylinder	Cement vs. C Ash	0.224
	Cement vs. Slag	0.806
	Cement vs. F Ash	0.415
Interior Cores	Cement vs. C Ash	0.315
	Cement vs. Slag	0.412
	Cement vs. F Ash	0.312

The data collected from this study were then compared against two different models for predicting strength gain in hardened concrete: the ACI 209.2R (2008) equation and the Florida DOT equations developed by Yazdani and McKinnie (2004). The strength gain for each in-place testing method as well as each size of molded cylinders were evaluated separately in order to obtain error values for each test method. Since it was determined that the SCM type does not statistically affect strength gain at a 95% confidence level, all specimens of the same test type were grouped together. For example, molded 6×12 in. cylinders from casts RG4000PC, RG4000CA, RG4000FA, and RG4000SC were analyzed together in order to produce a more conclusive result. In order to do an unbiased statistical analysis, ratios of test average at a certain age were divided by the average strength of that test at 28 days in order to produce a value which solely represented strength gain within the testing method for a given cast. In order to determine which function represented the best fit, the collected strength gain ratios were statistically analyzed against strength correction factors obtained from ACI 209 and FDOT. The unbiased estimate of the standard deviation was found for each of the two relationships with respect to the collected data. This was found using Equations 4-2 and 4-3 from McCuen (1985).

$$S_j = \sqrt{\frac{1}{n-1} * \sum_i^n {\Delta_i}^2} \qquad \text{Equation 4-2}$$

Where,

S_j = the unbiased estimate of the standard deviation (percent),

n = the number of data points (unitless), and

Δ_i = the absolute error (percent).

$$\Delta_i = \frac{S_{i,e} - S_{i,m}}{S_{i,m}} \times 100 \qquad \text{Equation 4-3}$$

Where,

Δ_i = the absolute error (percent),

$S_{i,e}$ = the estimated value of the strength gain factor (unitless), and

$S_{i,m}$ = the measured value of the strength gain factor (unitless).

For each type of test, an individual analysis was completed to determine if different test methods produced different strength gain relationships. The errors for each age were summed and the unbiased estimate of the standard deviation was calculated for each test method. A summary of the results can be seen below in Table 4-9. The average strength gain of all molded 6×12 in. cylinders and cores versus the strength gain predicted by ACI 209 and FDOT are compared in Figure 4-7.

Table 4-9: Summary of the Unbiased Estimate of the Standard Deviation for Strength Gain in Normal-Strength Concrete

Test Method	**FDOT Unbiased Estimate of the Standard Deviation (%)**	**ACI 209 Unbiased Estimate of the Standard Deviation (%)**
Molded 6×12 in. Cylinders	5.99	6.69
Interior Cores	17.56	12.24
Interior Pullouts	18.05	13.20
All Data	14.51	10.77

From Table 4-9, it can be seen that the FDOT equations provided a better strength gain estimation for the molded cylinders while the ACI 209 equation provided a much more accurate fit for both the core and pullout tests. From Figure 4-7, it can visually be seen that the FDOT equations provided a better strength gain estimation for the molded cylinders while the ACI 209 equation provided a much more accurate fit for the core tests. The values in Table 4-8 which are listed as “All Data” are the estimates of the standard deviation of the error for the molded 6×12 in. cylinders, interior cores, and interior pullouts combined. Since an updated pay scale for ALDOT is needed for core strengths, it is recommended that ALDOT use the ACI 209 equation to account for the effect of age on the in-place strength. Therefore, recommendations were based on this equation.

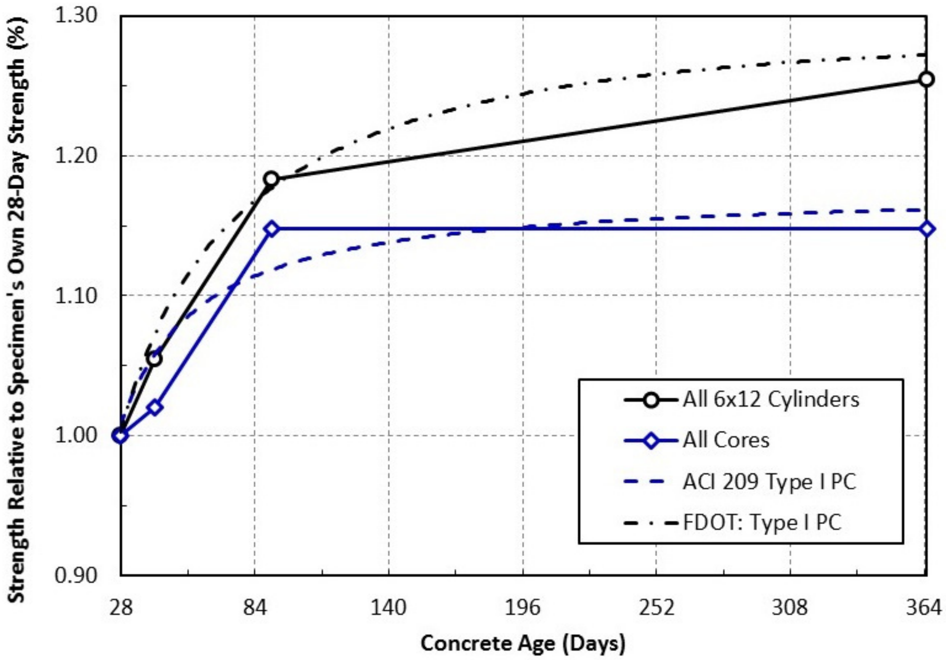

Figure 4-7: Average Strength Gain of All Molded 6×12 in. Cylinders and Cores versus the Strength Gain Predicted by ACI 209 and FDOT

After this, measured values obtained during the study were compared solely with the estimates produced by using the ACI 209 equation. For Type I cement, ACI 209 recommends using the values of a = 4 and β = 0.85. Using the measured strengths obtained during the study, a regression analysis was done to determine the best-fit curve to solve for the values for a and β in order to minimize the unbiased estimate of the standard deviation. The results for this analysis are presented in Table 4-10.

Table 4-10: Adjusted a and β Values for Different Testing Methods

Test Method	a Value	β Value
Molded 6×12 in. Cylinders	6.87	0.781
Interior Cores	6.15	0.853
Interior Pullout	3.18	0.907

It can be seen from Table 4-10 that all the adjusted values of a fall between 0.05 and 9.25 and all the β fall between 0.67 and 0.98 as specified within ACI 209.2R (2008). After the adjusted a and β values were calculated, these values were used to provide an estimate of strength for each testing method. Using these adjusted a and β values, the estimated standard deviations were again calculated and the results are shown in Table 4-11.

Table 4-11: Comparison of Adjusted and Unadjusted ACI 209 Values of the Unbiased Estimate of the Standard Deviation

Test Method	Adjusted ACI 209 Unbiased Estimate of the Standard Deviation (%)	Unadjusted ACI 209 Unbiased Estimate of the Standard Deviation (%)
Molded 6×12 Cylinders	4.43	6.69
Interior Cores	11.58	12.24
Interior Pullout	8.79	13.20

As can be seen from Table 4-11, the use of adjusted ACI 209 values for *a* and *β* do not significantly improve the unbiased estimate of the standard deviation. In order to compare the values further, the estimated versus recorded strength values for molded 6×12 in. cylinders and interior cores were plotted against one another for both the unadjusted and adjusted values of *a* and *β* and can be seen in Figures 4-8 through 4-11. Error bars were placed at ±15 percent due to the variability of the core testing and molded cylinders over multiple batches.

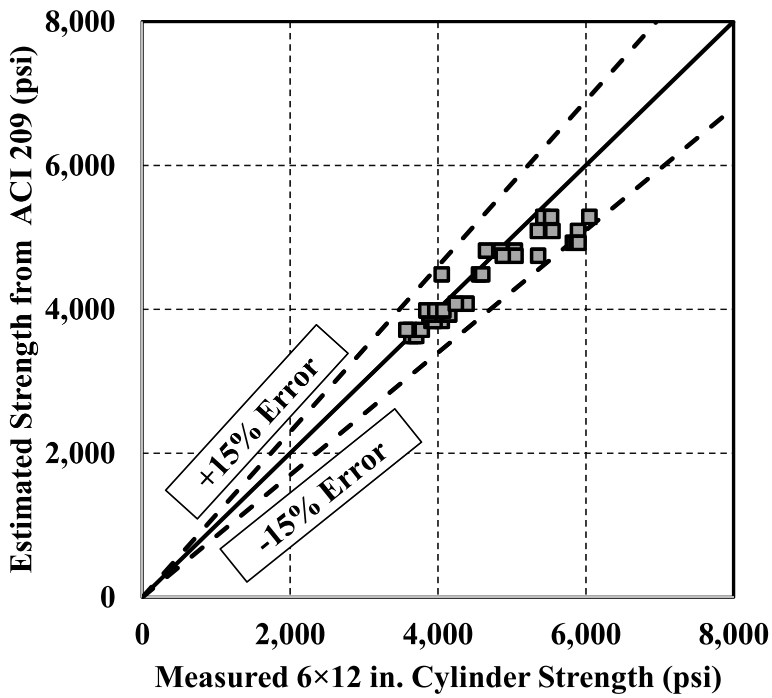

Figure 4-8: Measured Strength Versus Estimated Strength Using ACI 209 for Molded 6×12 in. Cylinders with Unadjusted *a* and *β* Values

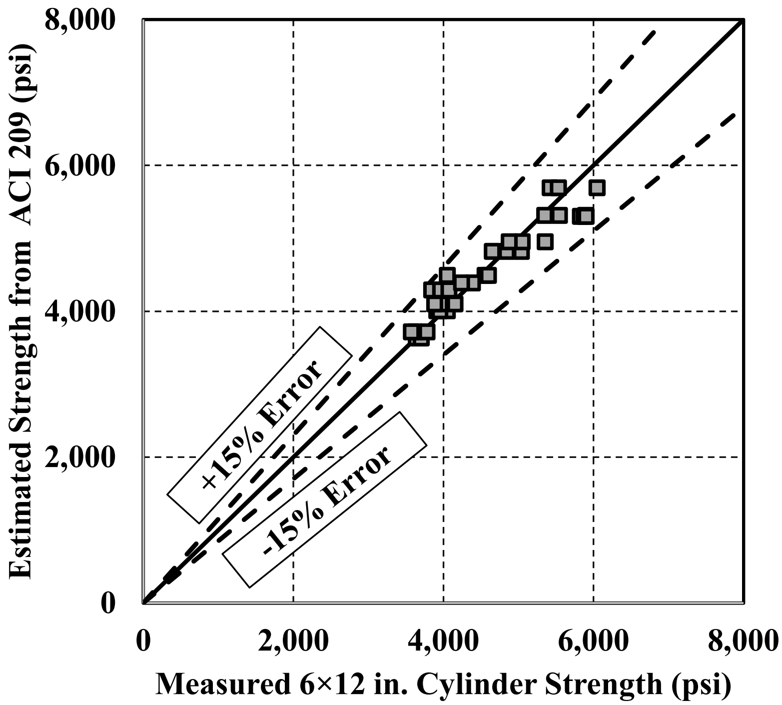

Figure 4-9: Measured Strength Versus Estimated Strength Using ACI 209 for Molded 6×12 in. Cylinders with Adjusted *a* and *β* Values

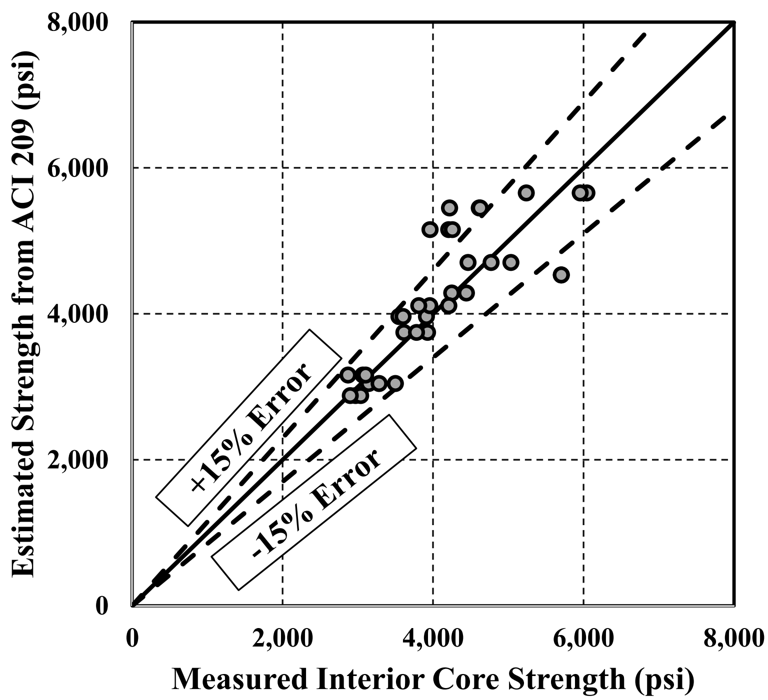

Figure 4-10: Measured Strength Versus Estimated Strength Using ACI 209 for Interior Cores with Unadjusted *a* and *β* Values

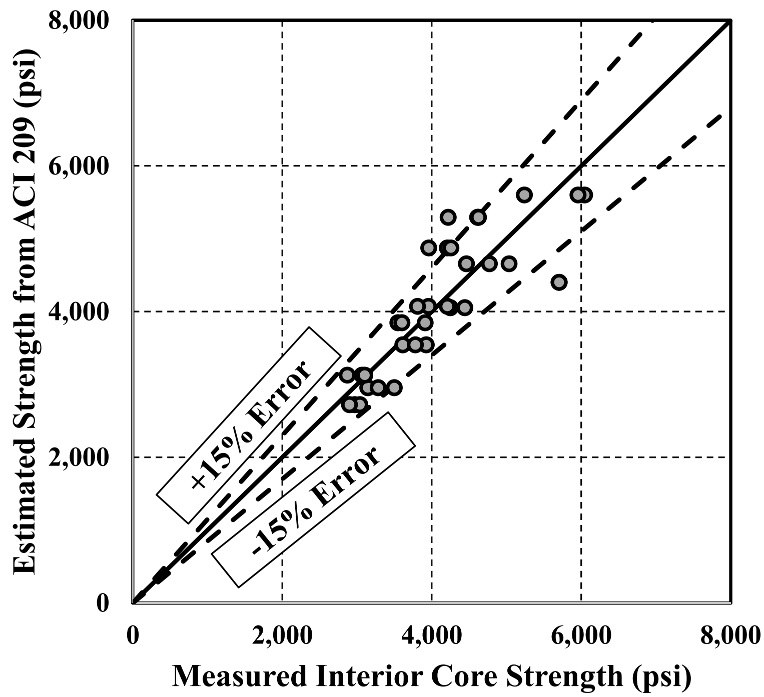

Figure 4-11: Measured Strength Versus Estimated Strength Using ACI 209 for Interior Cores with Adjusted *a* and *β* Values

From the plots shown in Figures 4-8 through 4-11 and the data shown in Table 4-10, it can be seen that there is little improvement using the adjusted values of *a* and *β* in the ACI 209 equation. Therefore, the recommended ACI 209 values of *a* = 4 and *β* = 0.85 were validated as providing a sufficiently accurate estimate of the effect of concrete age on the strength. The use of the ACI 209 values of *a* = 4 and *β* = 0.85 are therefore recommended for ALDOT use.

4.8 COMPARISON OF CORE TO MOLDED CYLINDER STRENGTHS

In the concrete industry, the standard measure of quality control and assurance has been the strength of moist-cured, molded 6×12 in. cylinders for many years. In the United States, the most common in-place strength testing method is core testing. When molded 6×12 in. cylinders are tested and do not meet the specified minimum strength for the project, cores are typically extracted and tested to evaluate the in-place strength of the concrete. Most state DOTs use a pay reduction scale based on the strengths obtained from core testing relative to the specified design strength. In most cases, no consideration is given to the amount of damage inflicted upon a core during the drilling process as well as the difference in curing conditions and the presence of microcracking during the payment reduction. Therefore, if cores do not meet the required minimum strength, then payment is reduced for the in-place concrete. ACI 318 (2014) states that concrete strength shall be deemed structurally adequate if the average strength of three cores is greater than 85% of the specified compressive-strength (f'_c) for the project as long as no single core strength of three cores is below 75% of the required strength.

One of the objectives of the project was to determine though testing if there was a statistical difference between core strength and molded, moist-cured cylinder strength and, if there was this relationship should be established. In order to do this, a procedure had to be developed in order to convert core strength at a given age back to an equivalent core strength at 28 days. The reason why a 28-day core strength is desired is to compare this value with the specified design strength of the concrete in question. After this, a relationship must be established between core strength and cylinder strength to assess the adequacy of the in-place concrete and therefore create a pay scale that is appropriate for both the state DOT as well as the contractor. From theory, since the interior cores represented the most restrained condition, this meant that it also represented the worst-case scenario (lowest values) in terms of compressive strength for the data collected in this study. In actual structures, the in-place restraint could exceed what was present in this study, especially in very large structural elements. Based on this, comparisons were made between the strength from both exterior and interior core specimens versus molded 6×12 in. cylinders, but conclusions were based on the comparison of *interior* core strength to cylinder strength to determine if adjustments should made on how to assess when a comparable level of strength is obtained from the core and cylinder samples.

In order to determine the relationship between core and cylinder strength, it first had to be determined if data from normal-strength and high-strength slabs could be analyzed together. To do this, average core strengths from all ages were divided by the average strength of their corresponding 6×12 in. molded cylinders. These ratios were then compared for both interior and exterior cores. A student t-test was performed between the normal-strength ratios and the high-strength ratios for both exterior and interior cores. The results can be seen in Table 4-12. From the P-values produced from the statistical analysis, it can be seen that there was not a statistical difference for the core-cylinder relationship for normal-strength and high-strength concrete. This means that only one relationship needed to be established for relating core strength to cylinder strength regardless of strength level for the strength range tested. Because of this, both sets of data were combined in order to form one trend from which conclusions could be drawn and recommendations made.

Table 4-12: Summary of P-Values Comparing Normal- vs. High-Strength Cores

Specimen Type	P-Value
Exterior Cores vs. 6x12" Molded Cylinders	0.431
Interior Cores vs. 6x12" Molded Cylinders	0.630
All Cores vs. 6x12" Molded Cylinders	0.375

Once it was determined that the normal-strength and the high-strength data could be combined, an analysis was done to determine if there was a statistical difference between the strength results from cores and the results from the 6×12 moist-cured, molded cylinders. To do this, two paired t-tests were conducted between the average strengths of the cores and the cylinders for both exterior and interior cores. A third t-test was done to compare all the core strengths combined regardless of restraint type against the cylinders. The results of the t-tests can be seen in Table 4-13. From Table 4-13, it can be concluded that at a 95 % confidence level there is a statistical difference between core strength and molded, moist-cured cylinder strength at all testing ages. The trend between core strength and cylinder strength are shown in Figures 4-12 through 4-14.

Table 4-13: P-values from Core versus Molded Cylinder t-test

Specimen Type	P-value from t-test
Exterior Cores vs. 6 × 12 in. Molded Cylinders	0.002
Interior Cores vs. 6 × 12 in. Molded Cylinders	3.65×10^{-5}
All Cores vs. 6 × 12 in. Molded Cylinders	2.68×10^{-5}

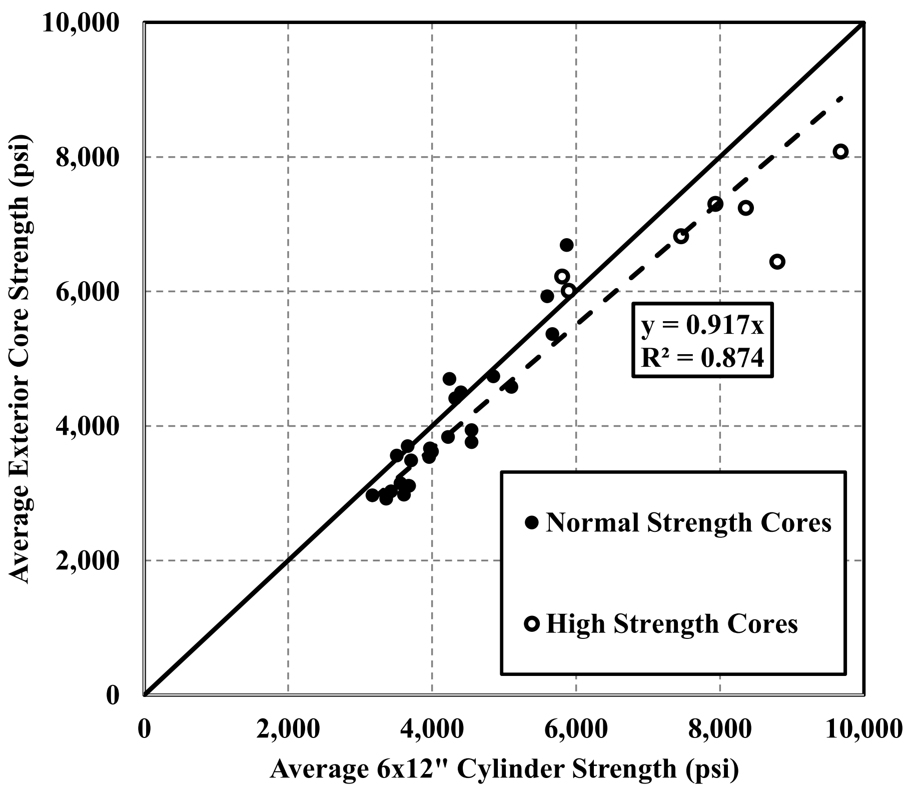

Figure 4-12: Average 6×12 in. Molded Cylinder Strength versus Average Exterior Core Strength

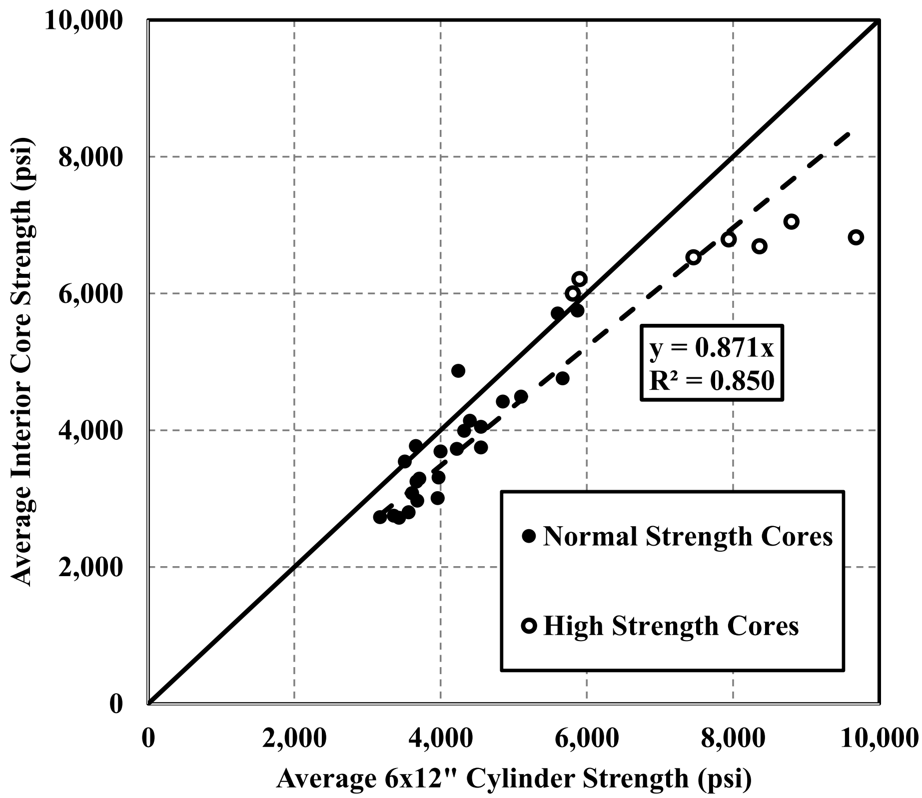

Figure 4-13: Average 6×12 in. Molded Cylinder Strength versus Average Interior Core Strength

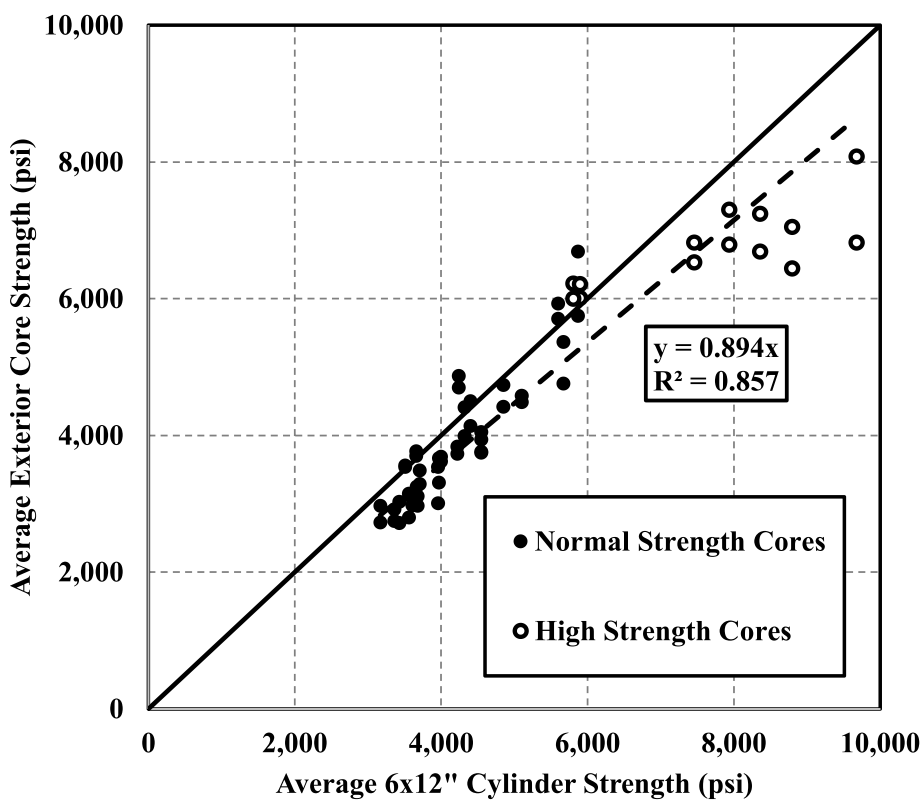

Figure 4-14: Average 6×12 in. Molded Cylinder Strength versus Average Core Strength

From ACI 318 (2014), it is specified that an average core strength of 85% of the specified compressive strength (*f'c*) constitutes satisfactory in-place strength for the concrete in question. For Figures 4-12 through 4-14, linear trend lines were fitted through the data that run through the origin. The relatively high R^2 values indicate a very well defined trend. It can be seen in the Figure 4-13 that interior core strength averaged approximately 87% of the average molded cylinder strength. It can also be seen that exterior core strength averaged approximately 92% of the average molded cylinder strength, while overall core strength averaged about 89% of the average molded cylinder strength. These differences in core strength and molded, moist-cured cylinder strength was expected due in part to the damage imparted on a core from drilling through aggregate, the difference in curing methods, and the presence of microcracking, especially in the interior core specimens. It was expected that the interior cores would have a reduced strength compared to the exterior cores. This trend agrees with the principles presented by Rasmussen and Rozycki (2001) that the friction, interlock, and adhesion forces between the slab and the sub-base have a significant impact on the amount of axial restraint within a slab.

Since the slabs only measured 15 ft×15 ft, it was determined that cores taken from the interior of even larger members could be considered more restrained and therefore could have an even greater reduction in strength than the interior cores examined in the study. Based on the findings that the interior core strength averaged approximately 87% of the average molded cylinder strength and the recommendations of ACI 318 (2014), it is recommended that an average core strength of 85% or greater of the design strength be deemed structurally adequate by ALDOT.

Chapter 5
Development of Implementation Guidelines for Core Testing

5.1 INTRODUCTION

An objective of this project was to develop a draft ALDOT procedure to evaluate the in-place strength by core testing. Many factors that influence core strength results were evaluated during the study including the following: length-to-diameter ratio, core diameter, effect of restraint, aggregate type, aggregate size, direction of drilling, SCM type, concrete age, and effect of curing. This chapter provides recommendations for a draft ALDOT procedure.

5.2 CURRENT ALDOT CORING AND EVALUATION PRACTICE

Currently, when the average compressive strength of standard quality assurance cylinders do not equal or exceed the required design strength, cores are taken from the in-place concrete in question at the expense of the contractor in order to evaluate the concrete that was placed at the site. It is specified by ALDOT (2013) that cores must be taken, cured, and tested within 42 days of concrete placement. Also, it is stated that "Core specimens will be measured, cured, and tested in accordance with AASHTO T 24. Proper strength correction factor will be applied to cores having length-to-diameter (L/D) ratio less than two." If the average strength of the cores extracted from the in-place concrete do not meet or exceed the specified design strength, the contractor is paid a reduced price for the in-place concrete based on the average strength of the extracted cores. The price of the concrete is adjusted using Equation 2-13. The pay scale can be seen in Figure 2-8. The current pay scale is valid only if the average strength of the cores is greater than or equal to 85 % of the required design strength. The current pay scale does not take into account the difference in core strength due to damage imparted on the core by cutting through coarse aggregate, the effect of different curing methods, and the presence of microcracking within a core.

5.3 EVALUATION OF VARIABLES AFFECTING CORE STRENGTH

Throughout the project, numerous variables were evaluated in order to determine their impact on core strength. In this section, justification will be provided to determine the equivalent, 28-day in-place compressive strength with Equation 5-1. The core strength correction factors are defined in the remainder of this section.

$$f_c(28) = (F_{L/D}F_{dia}F_{age}F_{dir})f_{core}(t) \qquad \text{Equation 5-1}$$

Where,

$f_c(28)$ = the equivalent, 28-day in-place compressive strength (psi),

$F_{L/D}$ = length-to-diameter strength correction factor,

F_{dia} = diameter strength correction factor,

F_{age} = age strength correction factor,

F_{dir} = casting direction strength correction factor and

$f_{core}(t)$ = measured core compressive strength at any time (psi).

5.3.1 CORE LENGTH-TO-DIAMETER RATIO

AASHTO T 24 (2009), ACI 214 (2010), and Bartlett and MacGregor (1995) all list methods to correct core strength due to different length-to-diameter ratios. Research that was conducted during this project and detailed by Carroll et al. (2016) evaluated the effect of length-to-diameter ratio on core specimens. It was recommended by Carroll et al. (2016) that cores that have diameters greater than or equal to 3.75 in., as per AASHTO T 24 (2009), should be corrected for length-to-diameter ratio by using the strength correction function shown in ACI 214 (2010), which was presented in Table 2-3 and is shown in Equation 5-2. This length-to-diameter strength correction factor is applicable for concrete that has a compressive strength between 2,000 and 14,000 psi. Carroll et al. (2016) recommended that these length-to-diameter strength correction factors not be applied to cores with a diameter of 3 in.; therefore, cores with a diameter less than 3.75 in. shall only be used if they have a length-to-diameter ratio of 2.0 at the time of testing (i.e. after trimming).

$$F_{L/D} = 1 - \left(0.130 - \frac{3}{1{,}000{,}000} f_{core}\right)(2 - L/D)^2 \qquad \text{Equation 5-2}$$

Where,

$F_{L/D}$ = length-to-diameter strength correction factor,

f_{core} = measured core compressive strength (psi),

L = core length measured after capping (in.), and

D = core diameter measured before capping (in.).

5.3.2 CORE DIAMETER

AASHTO T 24 (2009) states that core diameter shall not be less than 3.75 in. Despite this, there are many occasions when it is not possible retrieve a core with a diameter of 3.75 in.

due to the presence of embedded steel reinforcement. ACI 214 (2010) recommends strength correction factors for core diameter, as shown in Table 2-3 and again in Table 5-1. Research conducted for this project and detailed by Carroll et al. (2016) indicated that cores having a diameter of less than 3.75 in. should only be used if a length-to-diameter ratio of 2.0 can be obtained. This was due to the high variability of the 3 in. diameter cores that were tested. This variability showed that the strength correction function for length-to-diameter ratio recommended by ACI 214 (2010) is not accurate for 3 in. diameter cores with length-to-diameter ratios less than 2.0. For 3 in. diameter cores having a length-to-diameter ratio of 2.0, Carroll et al. (2016) recommends that a strength correction factor of 1.03 should be applied to account for the effect that core diameter has on strength. This strength correction factor was obtained by interpolating values suggested by ACI 214 (2010) and these values can be used for concrete having a compressive strength between 1,440 and 13,400 psi.

Table 5-1: Core Diameter Correction Factor

Core Diameter	F_{dia}
2 in.	1.06
3 in.	1.03
4 in.	1.00
6 in.	0.98

5.3.3 Coring Direction Relative to Casting Direction

As detailed by Suprenant (1985), the diretion of drilling with respect to the direction in which the concrete was placed creates a difference in the planes of weakness found within a core. Research conducted in this project and detailed by Carroll et al. (2016) investigated the effect of coring direction with respect to casting direction. It was found the specimens which were cored perpendicular to the casting direction had a relative strength of 96 % as compared to those cores which were taken parallel to the casting direction. Because of this, Carroll et al. (2016) recommends the casting direction strength correction factors as listed in Table 5-2 be used. Unusual directions neither parallel or perpendicular to the casting direction shall be measured and rounded to the nearest 90° to determine the case that is closest to the directions listed in Table 5-2.

Table 5-2: Casting Direction Strength Correction Factor

Casting Direction	F_{dir}
Cores drilled **parallel** to casting direction	1.00
Cores drilled **perpendicular** to casting direction	1.04

5.3.4 AGGREGATE SIZE

AASHTO T 24 (2009) specifies that cores which are obtained from concrete with a nominal maximum aggregate size (NMAS) greater than 1 ½ in. should preferably have a diameter of at least three times the NMAS and must have a diameter of at least two times the NMAS. Due to this recommendation, research was conducted during this project on the relationship between NMAS and core diameter and is detailed by Carroll et al. (2016). Based on the findings of the study, it was concluded that the length-to-diameter ratio strength correction factors in ACI 214 (2010) were not valid for 3 in. diameters cores that contain No. 67 aggregate and larger. Carroll et al. (2016) also concluded that coarse aggregate sizes of No. 67 and 57 do not have a significant impact on the strength of cores made with different length-to-diameter ratios.

5.3.5 MOISTURE CONDITIONING

AASHTO T 24 (2009) recommends a standard treatment for cores which prevents the formation of moisture gradients within a core after removal from a slab. It is highly recommended that this procedure be followed so that moisture gradients have no effect on the apparent strength of a core. If different moisture conditioning is applied to cores once removed from a specimen, it is recommended that the moisture conditioning correction factors recommended by ACI 214 (2010), which can be seen in Table 2-3, be applied to the test results.

5.3.6 DAMAGE

ACI 214 (2010) recommends a strength correction factor of 1.06 which compensates for excessive damage imparted on a core during drilling. Bartlett and MacGregor (1994c) says that this factor accounts for microcracking caused by drilling as well as undulation of the core barrel during drilling and aggregate pop outs which may occur during testing. No advice is given on to what degree the damage must be for the factor to be applied. ACI 214 (2010) states that this factor is valid for normal weight concretes which have a compressive strength between 2,000 and 13,400 psi. If a core is extracted that would require the use of this excessive damage factor, it is recommended that another core specimen be drilled and the damaged core be discarded instead of testing the damaged core and applying the strength correction factor of 1.06.

5.3.7 AGGREGATE TYPE

Research conducted by Khoury, Aliabdo and Ghazy (2014) concluded that concrete containing river gravel are more difficult to core than concrete which contains aggregates such as limestone and that because of this, a higher amount of damage was inflicted on concretes which contain natural aggregates. Since there are many coarse aggregate types available to the concrete industry in the state of Alabama, slabs were cast in this study that contained different types of coarse aggregate to evaluate the effect that it had on core strength. A statistical analysis was completed on the data collected to see if coarse aggregate type had an effect on the relationship between in-place strength and moist-cured, molded cylinder strength. This analysis produced mixed results for each type of test type. It is recommended that no strength correction factor due to aggregate type should be used. This is because the difference in apparent strength due to aggregate type is most likely caused by damage imparted on the core due to aggregate type, which is accounted for by deeming that cores that have an average strength of greater than or equal to 85 % of the required design strength be considered structurally adequate.

5.3.8 EFFECT OF RESTRAINT

In this project it was found that there was a statistically significant difference between the strength of cores recovered from the low restrained exterior region of the slab and cores taken from the highly restrained interior region of the slab. There is no specific strength correction factor recommended the amount of restraint, becsause this varies from project to project. However, as was the case in Section 5.3.7, high restraint and microcracking may impact the in-place concrete strength, and this is accounted for deeming that cores that have an average strength of greater than or equal to 85 % of the required design strength be considered structurally adequate.

5.3.9 CONCRETE AGE

Several different age correction models have been developed in order to convert a concrete compressive strength at any age to an equivalent 28-day strength. One of these models was developed by Yazdani and McKinnie (2004) using moist-cured, molded cylinders. Different strength equations were developed based on different cement types and SCM types. These strength correction equations can be seen in Equations 2-1 through 2-10. ACI 209 (2008) provides Equation 2-11 as a way to convert concrete compressive strength at any age to an equivalent 28-day strength. Both of these methods were evaluated, and it was determined that the ACI 209 equation provided the best estimation of equivalent 28-day strength for all concrete types and this expression is thus presented in Equation 5-3. It is recommended that this equation be used with the given constants to convert the average strength of three cores back to an equivalent 28-day strength which can then be compared with the specified design strength.

$$F_{age} = \left(\frac{4+0.85\times t}{t}\right) \quad \text{Equation 5-3}$$

Where,

F_{age} = age strength correction factor, and

t = concrete age at time of core casting (days).

5.3.10 PRESENCE OF STEEL REINFORCEMENT

AASHTO T 24 (2009) recommends that no core which contains steel reinforcement should be tested unless necessary. Though no data were collected during this study to evaluate the effect of steel reinforcement contained with cores, Bartlett and MacGregor (1995) have some recommendations presented in Table 2-6 that could be used. However, these recommendations are difficult to apply because the number, portion, and orientation of steel reinforcement can all vary. Since ALDOT currently has the ability to use ground-penetrating radar (GPR) to detect reinforcement, the most practical approach in accordance with AASHTO T 24 (2009) is that no core containing embedded reinforcement should be tested and used for in-place concrete strength assessment.

5.3.11 RELATIONSHIP BETWEEN CORE AND CYLINDER STRENGTH

The current pay reduction method used by ALDOT does not reflect differences in core strength and molded, moist-cured cylinder strength due to damage, differences in curing methods, and the presence of increased microcracking within core. Because of these factors, ACI 318 (2014) recommends that if the average strength of a set of three cores equals or exceeds 85 % of the specified design strength and no single core has a strength below 75 % of the specified design strength, the in-place concrete be deemed structurally adequate. This study showed that there is a statistically significant difference between core strength and molded, moist-cured cylinder strength. The cores that were obtained from the interior region of the slabs averaged approximately 87 % of the strength of the molded, moist-cured cylinder counterparts. Since cores are taken from structures that might have more restraint than the 15 ft ×15 ft slabs that were cast in this study, it is recommended that the in-place concrete be deemed structurally adequate if the average of *three* cores equals or exceeds 85 % of the specified design strength so long as no single core has a strength below 75 % of the specified design strength. If more than *three* cores are tested, then the procedure of Bartlett and Lawler (2011) should be used to determine the acceptable minimum strength of the lowest single core.

5.4 RECOMMENDED PROCEDURE FOR CORRECTING CORE STRENGTH

It is recommended that at least four cores be recovered in the event that in-place strength has to be evaluated due to the low strength of quality assurance cylinders. This is so that if one core is damaged or found to be an outlier, then an average of the remaining three cores is still useful to determine the structural adequacy of the in-place concrete.

From the findings in this study, it is recommended that strength correction factors be applied to cores for the following: length-to-diameter ratio, core diameter, concrete age, and casting direction and as expressed in Equation 5-1.

Once equivalent 28-day core strengths are obtained with Equation 5-1, the core strengths should be checked for outliers. This can be done using the method from ASTM E178 (2008). This is done by using Equations 5-4 and 5-5. The output of Equations 5-4 and 5-5 must be less than the $T_{critical}$ that is listed in Table 5-3, which depends on the number of core strength results in the sample.

$$\frac{f_{c,average} - f_{c,low}}{\sigma} \leq T_{critical} \qquad \text{Equation 5-4}$$

$$\frac{f_{c,high} - f_{c,average}}{\sigma} \leq T_{critical} \qquad \text{Equation 5-5}$$

Where,

$f_{c,average}$	=	average compressive strength of all cores (psi)
$f_{c,low}$	=	lowest compressive strength of all cores (psi),
$f_{c,high}$	=	highest compressive strength of all cores (psi),
σ	=	standard deviation of all core strengths (psi), and
$T_{critical}$	=	critical value for a given number of cores tested at a 5% significance level.

If one of the strength values from the data set is determined to be an outlier, then this core strength shall be discarded and not used in the calculation of the average or minimum compressive strength that is used to determine if the in-place concrete is structurally adequate. After an outlier is discarded, the remaining data set shall again be checked for outliers using the same procedure. Once all outliers have been removed, the average compressive strength shall be computed again without the outlying strength values. This average compressive strength shall then be used to determine the structural adequacy of the in-place concrete. A minimum of three-core strength results that all meet the outlier check requirements are required to determine if the in-place concrete strength is structurally adequate.

Table 5-3: $T_{critical}$ Values for Given Number of Cores in a Sample (ASTM E178 2008)

n	$T_{critical}$	n	$T_{critical}$	n	$T_{critical}$
3	1.153	19	2.532	35	2.811
4	1.463	20	2.557	36	2.823
5	1.672	21	2.58	37	2.835
6	1.822	22	2.603	38	2.846
7	1.938	23	2.624	39	2.857
8	2.032	24	2.644	40	2.866
9	2.11	25	2.663	41	2.877
10	2.176	26	2.681	42	2.887
11	2.234	27	2.698	43	2.896
12	2.285	28	2.714	44	2.905
13	2.331	29	2.73	45	2.914
14	2.371	30	2.745	46	2.923
15	2.409	31	2.759	47	2.931
16	2.443	32	2.773	48	2.94
17	2.475	33	2.786	49	2.948
18	2.504	34	2.799	50	2.956

Note: All listed $T_{critical}$ values were determined at an upper 5% significance level

The in-place concrete shall be considered structurally adequate when both the following two criteria are met:

1) The average compressive strength of the cores shall equal or exceed 85 % of the specified 28-day design strength (f'_c).
2) No single core compressive strength shall be less than $k \times f'_c$, with the *k*-value as defined in Table 5-4. The k-value is dependent on the number of cores extracted from the concrete in question. If the sample size is three cores, then the value of *k* is 0.75.

If the average core strength is lower than 85 % of the specified design strength or the data set contains a core strength lower than $k \times f'_c$, then a structural analysis must be done to determine if the in-place concrete is satisfactory, needs to be strengthened, or must be removed and replaced.

The k-value is used in Criterion 2, because Bartlett and Lawler (2011) point out that an increase in the number of cores would not impact the mean-strength criterion, but it would have an impact on the requirement that no single test result could have a strength lower than 75 % of the speficied design strength. The k-values in Table 5-4 were calculated in accordance with the findings of Bartlett and Lawler (2011) using a P_1 of 10% as well as an allowable value of V_{ws} of 7.8 %. This value comes from Table 2-8 which is from ACI 214 (2010). This value of V_{ws} was chosen because it lies between the specified values of 7 % for a single batch of concrete and 12 % for multiple batches of concrete for a single member while also producing a *k* value of 0.75 for three cores.

Table 5-4: Values of *k* based on the Number of Cores Extracted

n	***k***	***n***	***k***	***n***	***k***
3	0.75	**15**	0.69	**27**	0.67
4	0.74	**16**	0.69	**28**	0.67
5	0.73	**17**	0.69	**29**	0.67
6	0.72	**18**	0.69	**30**	0.67
7	0.72	**19**	0.68	**31**	0.67
8	0.71	**20**	0.68	**32**	0.67
9	0.71	**21**	0.68	**33**	0.67
10	0.70	**22**	0.68	**34**	0.67
11	0.70	**23**	0.68	**35**	0.67
12	0.70	**24**	0.68	**36**	0.66
13	0.70	**25**	0.68	**37**	0.66
14	0.69	**26**	0.67	**38 to 50**	0.66

Chapter 6

Summary, Conclusions, and Recommendations

6.1 Summary

Due to the number of factors that may affect the apparent strength of cores, core strength results can be challenging to analyze and interpret. Since there are many factors that influence the apparent strength of cores, strength correction factors must be applied to core strengths in order to convert them to a standard which can be compared with specified 28-day design strength. The following major factors that affect the relationship between the in-place strength and the strength of molded cylinders were assessed in this project: concrete age, concrete strength level, coarse aggregate type, degree of microcracking, and difference in strength gain due to using different cementitious materials in the concrete mixture.

Eight, 15 ft × 15 ft slab specimens with a 9½ in. depth were cast with ready-mixed concrete. In order to evaluate the effect that concrete strength level had on the relationship between in-place testing and required design strength, two high-strength concrete slabs were cast in addition to six normal-strength slabs for comparison. Three types of aggregate were used in the study: crushed limestone, uncrushed river gravel, and crushed granite. Slabs were also made with different SCM types: Class C fly ash, Class F fly ash, and slag cement. Testing was conducted near the edges of the slab as well as in the interior of the slab to determine if restraint had an impact on microcracking. Different types of in-place testing were conducted to determine the relationship between test type and apparent in-place strength as compared to standard molded cylinder strength. The in-place strength was assessed with cores, cast-in-place cylinders, and pullout tests. Four in. diameter cores were tested along with 6×12 in. and 4×8 in. molded, moist-cured cylinders. All strength specimens were tested at 28, 42, 91, and 365 days.

6.2 Conclusions

Conclusions from the work document in this report are as follows:

- There was no statistical difference between the compressive strength results of 6×12 in. and 4×8 in. molded cylinders for both normal-strength and high-strength concrete.
- It was found that the amount of restraint (or degree of microcracking) influenced the strength of the core and pullout test specimens. For example, the strength of cores exposed to the highest restraint averaged to be approximately 94.8 % of that of the cores exposed to the least restraint. However, the strength of the cast-in-place cylinders were

not affected by their location, as their cylinder molds provide room for them to expand and contract within the support system which holds them in place.

- Equation 5-3 in Chapter 5 from ACI 209.2R (2008) was determined to provide a sufficiently accurate estimate of strength gain of cores with concrete age. The type of SCM did not have a statistically significant impact on the post-28-day rate of strength gain on in-place specimens.
- Aggregate type affected the relationship between in-place strength and molded cylinder strength. In-place specimens from the granite slab had the lowest overall relative strength to their molded cylinder counterparts of the three aggregate types used in the study.
- At a 95 % confidence level, there is a statistical difference between core strength and molded, moist-cured cylinder strength at all testing ages. The average core strength from the concrete with the highest restraint and thus most microcracking, was approximately 87 % of the average molded cylinder strength. This difference in core strength and molded, moist-cured cylinder strength was expected due in part to the damage imparted on a core from drilling through aggregate, the difference in curing methods, and the presence of microcracking.

  The result that the average core strength from the concrete with the highest restraint was approximately 87 percent of the average molded, moist-cured cylinder strength are in agreement with the approach used by ACI 318 (2014). ACI 318 (2014) uses an approach in that concrete should be deemed structurally adequate if the average compressive strength of the cores is equal to or greater than 85 percent of the specified 28-day design strength (with the additional requirement that no single core of three tested, having a compressive strength less than 75 percent of the required design strength).

6.3 RECOMMENDATIONS

The following recommendations are offered based on the work documented in this report:

- When taking cores in the field, the compressive strength of each core should be converted back to an equivalent core strength at 28 days and then compared with the specified design compressive strength using Equation 5-1. Strength correction factors for length-to-diameter ratio, core diameter, concrete age, and casting direction should be applied.
- Once equivalent 28-day core strengths are obtained with Equation 5-1, the core strengths should be checked for outliers by using the method from ASTM E178 (2008) as outlined in Section 5.4.

- It is recommended that at least four cores be recovered in the event that in-place strength has to be evaluated. This is so that if one core is damaged or found to be an outlier, then an average of the remaining three cores is still useful to determine the structural adequacy of the in-place concrete.
- In accordance with AASHTO T 24 (2009), it is recommend to only recover and test cores with a diameter of 3.75 in. or greater. Cores smaller than this size have high variability in compressive strength results, and the length-to-diameter strength correction factor recommended by ACI 214 (2010) is not accurate for 3 in. diameter cores.
- The standard treatment for cores (sealed in plastic bags for at least five days after last coming in contact with moisture from either drilling or sawing) in AASHTO T 24 (2009) should be followed in all cases where cores are recovered to assess the in-place strength. This procedure minimizes moisture gradients within a core after removal from the structure.
- Since ALDOT currently has the ability to use ground-penetrating radar (GPR) to detect reinforcement, the most practical approach in accordance with AASHTO T 24 (2009) is that no core containing embedded reinforcement should be tested and used for in-place concrete strength assessment.
- This study showed that there is a statistically significant difference between core strength and molded, moist-cured cylinder strength due to factors such as coring damage, differences in curing methods, and the presence of increased microcracking within core. It is recommended that the in-place concrete be deemed structurally adequate when both the following two criteria are met:
  1) The average compressive strength of the cores shall equal or exceed 85 % of the specified 28-day design strength (f'_c).
  2) No single core compressive strength shall be less than $k \times f'_c$, with the k-value as defined in Table 5-4. The k-value is dependent on the number of cores extracted from the concrete in question. If the sample size is three cores, then the value of k is 0.75.

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Appendix A

Pullout Calibration Table from Germann Instruments

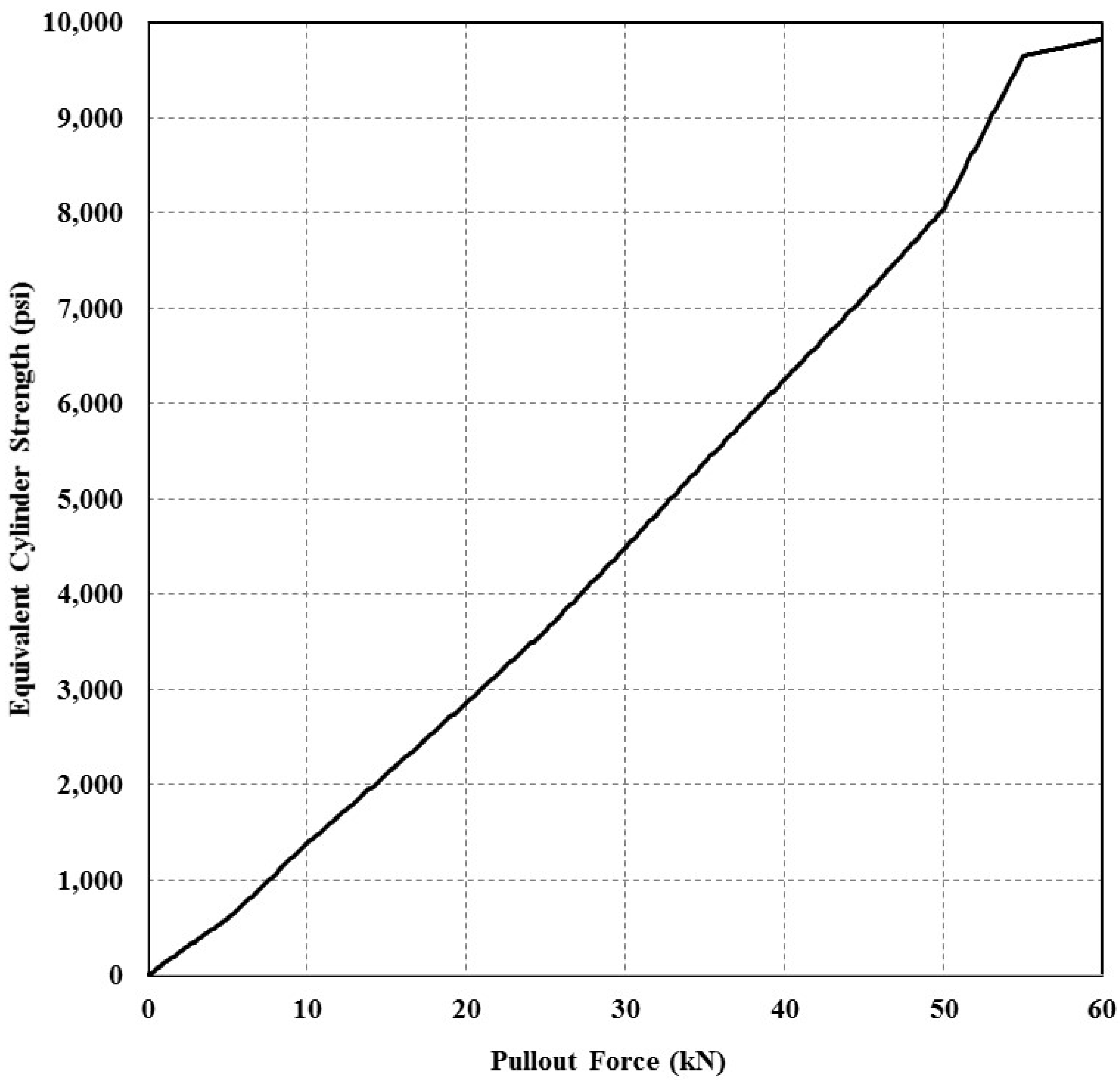

Figure A-1: Pullout Force vs. Predicted Molded Cylinder Strength for 6/4/2013-6/4/2014

Table A-1: Pullout Table from Germann Instruments for 6/4/2013-6/4/2014

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
0	0	0
0.1	0.1	12
0.2	0.2	24
0.3	0.3	37
0.4	0.4	49
0.5	0.5	61
0.6	0.6	73
0.7	0.7	85
0.8	0.8	98
0.9	0.9	110
1	1	122
1.1	1.1	134
1.2	1.2	147
1.3	1.3	159
1.4	1.4	171
1.5	1.5	183
1.6	1.6	195
1.7	1.7	208
1.8	1.8	220
1.9	1.9	232
2	2	244
2.1	2.1	256
2.2	2.2	267
2.3	2.3	279
2.4	2.4	290
2.5	2.5	302
2.6	2.6	314
2.7	2.7	325
2.8	2.8	337
2.9	2.9	348
3	3	360
3.1	3.1	372
3.2	3.2	383
3.3	3.3	395
3.4	3.4	406
3.5	3.5	418
3.6	3.6	429
3.7	3.6	441

Reading (kN)	Actual Pull Force (kN))	Cylinder Strength (psi)
3.8	3.7	453
3.9	3.8	464
4	3.9	476
4.1	4	487
4.2	4.1	499
4.3	4.2	510
4.4	4.3	522
4.5	4.4	534
4.6	4.5	545
4.7	4.6	557
4.8	4.7	568
4.9	4.8	580
5	4.9	592
5.1	5	604
5.2	5.1	620
5.3	5.2	636
5.4	5.3	652
5.5	5.4	667
5.6	5.5	683
5.7	5.6	699
5.8	5.7	715
5.9	5.8	731
6	5.9	747
6.1	6	762
6.2	6.2	778
6.3	6.3	794
6.4	6.4	810
6.5	6.5	826
6.6	6.6	842
6.7	6.7	857
6.8	6.8	873
6.9	6.9	889
7	7	905
7.1	7.1	921
7.2	7.2	936
7.3	7.3	952
7.4	7.4	968
7.5	7.5	984

Table A-1: Pullout Table from Germann Instruments for 6/4/2013-6/4/2014 (continued)

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)	Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
7.6	7.6	1000	11.4	11.5	1584
7.7	7.7	1016	11.5	11.6	1598
7.8	7.8	1031	11.6	11.7	1613
7.9	7.9	1047	11.7	11.8	1627
8	8	1063	11.8	11.9	1642
8.1	8.1	1079	11.9	12	1657
8.2	8.2	1095	12	12.1	1671
8.3	8.4	1111	12.1	12.2	1686
8.4	8.5	1126	12.2	12.3	1700
8.5	8.6	1142	12.3	12.4	1715
8.6	8.7	1158	12.4	12.5	1729
8.7	8.8	1174	12.5	12.5	1744
8.8	8.9	1190	12.6	12.6	1759
8.9	9	1206	12.7	12.7	1773
9	9.1	1221	12.8	12.8	1788
9.1	9.2	1237	12.9	12.9	1802
9.2	9.3	1253	13	13	1817
9.3	9.4	1269	13.1	13.1	1832
9.4	9.5	1285	13.2	13.2	1846
9.5	9.6	1300	13.3	13.3	1861
9.6	9.7	1316	13.4	13.4	1875
9.7	9.8	1332	13.5	13.5	1890
9.8	9.9	1348	13.6	13.6	1904
9.9	10	1364	13.7	13.7	1919
10	10.1	1380	13.8	13.8	1934
10.1	10.2	1394	13.9	13.9	1948
10.2	10.3	1409	14	14	1963
10.3	10.4	1423	14.1	14.1	1977
10.4	10.5	1438	14.2	14.2	1992
10.5	10.6	1452	14.3	14.3	2007
10.6	10.7	1467	14.4	14.4	2021
10.7	10.8	1482	14.5	14.5	2036
10.8	10.9	1496	14.6	14.6	2050
10.9	11	1511	14.7	14.7	2065
11	11.1	1525	14.8	14.8	2079
11.1	11.2	1540	14.9	14.9	2094
11.2	11.3	1555	15	15	2109
11.3	11.4	1569	15.1	15.1	2124

Table A-1: Pullout Table from Germann Instruments for 6/4/2013-6/4/2014 (continued)

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength(psi)
15.2	15.2	2139
15.3	15.3	2154
15.4	15.4	2169
15.5	15.5	2184
15.6	15.6	2199
15.7	15.7	2214
15.8	15.8	2229
15.9	15.9	2244
16	16	2259
16.1	16.1	2274
16.2	16.2	2289
16.3	16.3	2304
16.4	16.4	2319
16.5	16.4	2334
16.6	16.5	2348
16.7	16.6	2363
16.8	16.7	2378
16.9	16.8	2393
17	16.9	2408
17.1	17	2423
17.2	17.1	2438
17.3	17.2	2453
17.4	17.3	2468
17.5	17.4	2483
17.6	17.5	2498
17.7	17.6	2513
17.8	17.7	2528
17.9	17.8	2543
18	17.9	2558
18.1	18	2573
18.2	18.1	2588
18.3	18.2	2603
18.4	18.3	2618
18.5	18.4	2633
18.6	18.5	2648
18.7	18.6	2663
18.8	18.7	2678
18.9	18.8	2693

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength(psi)
19	18.9	2708
19.1	19	2723
19.2	19.1	2738
19.3	19.2	2753
19.4	19.3	2768
19.5	19.4	2783
19.6	19.5	2798
19.7	19.6	2813
19.8	19.7	2828
19.9	19.8	2843
20	19.9	2858
20.1	20	2874
20.2	20.1	2889
20.3	20.2	2904
20.4	20.3	2919
20.5	20.4	2934
20.6	20.5	2950
20.7	20.6	2965
20.8	20.7	2980
20.9	20.8	2995
21	20.9	3011
21.1	21	3026
21.2	21.1	3041
21.3	21.2	3056
21.4	21.3	3072
21.5	21.4	3087
21.6	21.5	3102
21.7	21.6	3117
21.8	21.7	3133
21.9	21.8	3148
22	21.9	3163
22.1	22	3178
22.2	22.1	3193
22.3	22.2	3209
22.4	22.3	3224
22.5	22.4	3239
22.6	22.5	3254
22.7	22.6	3270

Table A-1: Pullout Table from Germann Instruments for 6/4/2013-6/4/2014 (continued)

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
22.8	22.7	3285
22.9	22.8	3300
23	22.9	3315
23.1	23.1	3331
23.2	23.2	3346
23.3	23.3	3361
23.4	23.4	3376
23.5	23.5	3391
23.6	23.6	3407
23.7	23.7	3422
23.8	23.8	3437
23.9	23.9	3452
24	24	3468
24.1	24.1	3483
24.2	24.2	3498
24.3	24.3	3513
24.4	24.4	3529
24.5	24.5	3544
24.6	24.6	3559
24.7	24.7	3574
24.8	24.8	3589
24.9	24.9	3605
25	25	3620
25.1	25.1	3636
25.2	25.2	3654
25.3	25.3	3671
25.4	25.3	3688
25.5	25.4	3706
25.6	25.5	3723
25.7	25.6	3740
25.8	25.7	3758
25.9	25.8	3775
26	25.9	3792
26.1	26	3810
26.2	26.1	3827
26.3	26.2	3845
26.4	26.3	3862
26.5	26.4	3879

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
26.6	26.5	3897
26.7	26.6	3914
26.8	26.7	3931
26.9	26.8	3949
27	26.9	3966
27.1	27	3983
27.2	27.1	4001
27.3	27.2	4018
27.4	27.3	4035
27.5	27.4	4053
27.6	27.5	4070
27.7	27.6	4087
27.8	27.6	4105
27.9	27.7	4122
28	27.8	4140
28.1	27.9	4157
28.2	28	4174
28.3	28.1	4192
28.4	28.2	4209
28.5	28.3	4226
28.6	28.4	4244
28.7	28.5	4261
28.8	28.6	4278
28.9	28.7	4296
29	28.8	4313
29.1	28.9	4330
29.2	29	4348
29.3	29.1	4365
29.4	29.2	4382
29.5	29.3	4400
29.6	29.4	4417
29.7	29.5	4435
29.8	29.6	4452
29.9	29.7	4469
30	29.8	4487
30.1	29.9	4505
30.2	30	4523
30.3	30.1	4541

Table A-1: Pullout Table from Germann Instruments for 6/4/2013-6/4/2014 (continued)

Reading (kN)	Actual Pull Force (kN))	Cylinder Strength (psi)
30.4	30.2	4559
30.5	30.3	4577
30.6	30.3	4595
30.7	30.4	4613
30.8	30.5	4631
30.9	30.6	4649
31	30.7	4667
31.1	30.8	4685
31.2	30.9	4703
31.3	31	4721
31.4	31.1	4739
31.5	31.2	4757
31.6	31.3	4775
31.7	31.4	4793
31.8	31.5	4811
31.9	31.6	4828
32	31.7	4846
32.1	31.8	4864
32.2	31.9	4882
32.3	32	4900
32.4	32.1	4918
32.5	32.2	4936
32.6	32.3	4954
32.7	32.4	4972
32.8	32.5	4990
32.9	32.6	5008
33	32.7	5026
33.1	32.8	5044
33.2	32.9	5062
33.3	33	5080
33.4	33.1	5098
33.5	33.2	5116
33.6	33.3	5134
33.7	33.4	5152
33.8	33.5	5170
33.9	33.6	5188
34	33.7	5206
34.1	33.8	5224

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
34.2	33.9	5242
34.3	34	5260
34.4	34.1	5278
34.5	34.2	5296
34.6	34.3	5314
34.7	34.4	5332
34.8	34.5	5350
34.9	34.6	5368
35	34.7	5386
35.1	34.8	5404
35.2	34.9	5421
35.3	35	5438
35.4	35.1	5455
35.5	35.2	5473
35.6	35.3	5490
35.7	35.4	5507
35.8	35.5	5525
35.9	35.6	5542
36	35.7	5559
36.1	35.8	5576
36.2	35.9	5594
36.3	36	5611
36.4	36.1	5628
36.5	36.1	5646
36.6	36.2	5663
36.7	36.3	5680
36.8	36.4	5697
36.9	36.5	5715
37	36.6	5732
37.1	36.7	5749
37.2	36.8	5767
37.3	36.9	5784
37.4	37	5801
37.5	37.1	5818
37.6	37.2	5836
37.7	37.3	5853
37.8	37.4	5870
37.9	37.5	5888

Table A-1: Pullout Table from Germann Instruments for 6/4/2013-6/4/2014 (continued)

Reading (kN)	Actual Pull Force (kN))	Cylinder Strength (psi)
38	37.6	5905
38.1	37.7	5922
38.2	37.8	5939
38.3	37.9	5957
38.4	38	5974
38.5	38.1	5991
38.6	38.2	6008
38.7	38.2	6026
38.8	38.3	6043
38.9	38.4	6060
39	38.5	6078
39.1	38.6	6095
39.2	38.7	6112
39.3	38.8	6129
39.4	38.9	6147
39.5	39	6164
39.6	39.1	6181
39.7	39.2	6199
39.8	39.3	6216
39.9	39.4	6233
40	39.5	6250
40.1	39.6	6268
40.2	39.7	6285
40.3	39.8	6303
40.4	39.9	6320
40.5	40	6337
40.6	40.1	6355
40.7	40.2	6372
40.8	40.3	6389
40.9	40.3	6407
41	40.4	6424
41.1	40.5	6441
41.2	40.6	6459
41.3	40.7	6476
41.4	40.8	6493
41.5	40.9	6511
41.6	41	6528
41.7	41.1	6545

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
41.8	41.2	6563
41.9	41.3	6580
42	41.4	6598
42.1	41.5	6615
42.2	41.6	6632
42.3	41.7	6650
42.4	41.8	6667
42.5	41.9	6684
42.6	42	6702
42.7	42.1	6719
42.8	42.2	6736
42.9	42.3	6754
43	42.4	6771
43.1	42.5	6788
43.2	42.5	6806
43.3	42.6	6823
43.4	42.7	6840
43.5	42.8	6858
43.6	42.9	6875
43.7	43	6893
43.8	43.1	6910
43.9	43.2	6927
44	43.3	6945
44.1	43.4	6962
44.2	43.5	6979
44.3	43.6	6997
44.4	43.7	7014
44.5	43.8	7031
44.6	43.9	7049
44.7	44	7066
44.8	44.1	7083
44.9	44.2	7101
45	44.3	7118
45.1	44.4	7137
45.2	44.5	7155
45.3	44.6	7174
45.4	44.7	7192
45.5	44.8	7210

Table A-1: Pullout Table from Germann Instruments for 6/4/2013-6/4/2014 (continued)

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
45.6	44.9	7229
45.7	45	7247
45.8	45.1	7266
45.9	45.2	7284
46	45.3	7303
46.1	45.4	7321
46.2	45.5	7340
46.3	45.6	7358
46.4	45.7	7377
46.5	45.8	7395
46.6	45.9	7413
46.7	46	7432
46.8	46.1	7450
46.9	46.2	7469
47	46.3	7487
47.1	46.4	7506
47.2	46.5	7524
47.3	46.6	7543
47.4	46.7	7561
47.5	46.8	7580
47.6	46.9	7598
47.7	47	7616
47.8	47.1	7635
47.9	47.2	7653
48	47.3	7672
48.1	47.4	7690
48.2	47.5	7709
48.3	47.6	7727
48.4	47.7	7746
48.5	47.8	7764
48.6	47.9	7783
48.7	48	7801
48.8	48.1	7819
48.9	48.2	7838
49	48.3	7856
49.1	48.4	7875
49.2	48.5	7893
49.3	48.7	7912

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
49.4	48.8	7930
49.5	48.9	7949
49.6	49	7967
49.7	49.1	7986
49.8	49.2	8004
49.9	49.3	8023
50	49.4	8041
50.1	49.5	8073
50.2	49.7	8105
50.3	49.9	8137
50.4	50.1	8169
50.5	50.2	8202
50.6	50.4	8234
50.7	50.6	8266
50.8	50.8	8298
50.9	51	8330
51	51.1	8362
51.1	51.3	8394
51.2	51.5	8426
51.3	51.7	8458
51.4	51.8	8491
51.5	52	8523
51.6	52.2	8555
51.7	52.4	8587
51.8	52.6	8619
51.9	52.7	8651
52	52.9	8683
52.1	53.1	8715
52.2	53.3	8747
52.3	53.4	8780
52.4	53.6	8812
52.5	53.8	8844
52.6	54	8876
52.7	54.1	8908
52.8	54.3	8940
52.9	54.5	8972
53	54.7	9004
53.1	54.9	9036

Table A-1: Pullout Table from Germann Instruments for 6/4/2013-6/4/2014 (continued)

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
53.2	55	9069
53.3	55.2	9101
53.4	55.4	9133
53.5	55.6	9165
53.6	55.7	9197
53.7	55.9	9229
53.8	56.1	9261
53.9	56.3	9293
54	56.5	9325
54.1	56.6	9358
54.2	56.8	9390
54.3	57	9422
54.4	57.2	9454
54.5	57.3	9486
54.6	57.5	9518
54.7	57.7	9550
54.8	57.9	9582
54.9	58	9614
55	58.2	9647
55.1	58.2	9650
55.2	58.3	9654
55.3	58.3	9657
55.4	58.3	9661
55.5	58.3	9664
55.6	58.3	9668
55.7	58.4	9671
55.8	58.4	9675
55.9	58.4	9679
56	58.4	9682
56.1	58.4	9686
56.2	58.5	9689
56.3	58.5	9693
56.4	58.5	9696
56.5	58.5	9700
56.6	58.5	9703
56.7	58.6	9707
56.8	58.6	9711
56.9	58.6	9714
57	58.6	9718
57.1	58.6	9721
57.2	58.7	9725
57.3	58.7	9728
57.4	58.7	9732
57.5	58.7	9735
57.6	58.7	9739
57.7	58.8	9743
57.8	58.8	9746
57.9	58.8	9750
58	58.8	9753
58.1	58.8	9757
58.2	58.9	9760
58.3	58.9	9764
58.4	58.9	9767
58.5	58.9	9771
58.6	58.9	9775
58.7	58.9	9778
58.8	59	9782
58.9	59	9785
59	59	9789
59.1	59	9792
59.2	59	9796
59.3	59.1	9799
59.4	59.1	9803
59.5	59.1	9807
59.6	59.1	9810
59.7	59.1	9814
59.8	59.2	9817
59.9	59.2	9821
60	59.2	9824

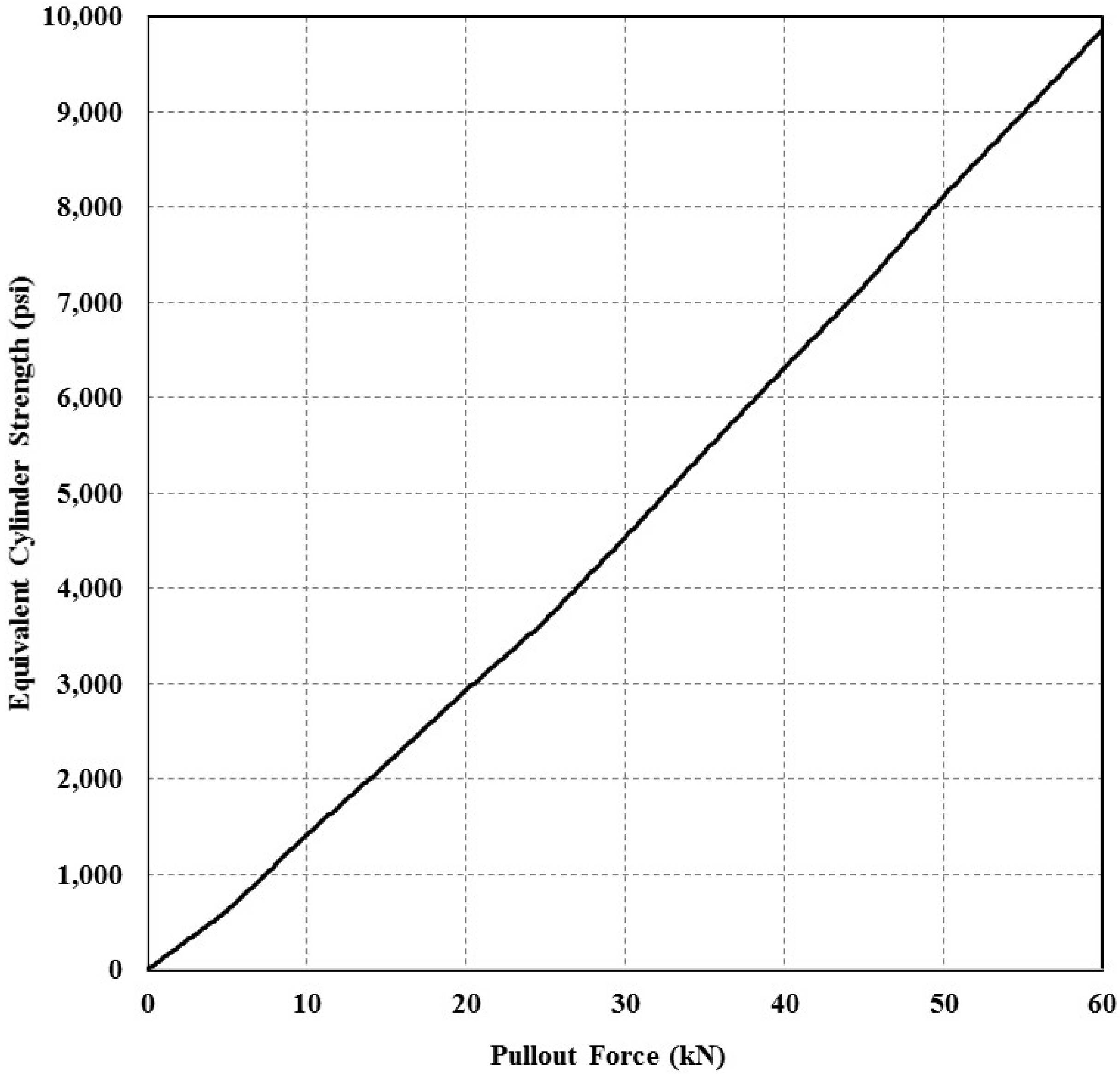

Figure A-2: Pullout Force vs. Predicted Molded Cylinder Strength for 6/5/2014-6/5/2015

Table A-2: Pullout Table from Germann Instruments for 6/5/2014-6/5/2015

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)	Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
0	0	0	3.8	3.9	466
0.1	0.1	12	3.9	4	479
0.2	0.2	25	4	4.1	491
0.3	0.3	37	4.1	4.2	503
0.4	0.4	49	4.2	4.3	515
0.5	0.5	62	4.3	4.4	527
0.6	0.6	74	4.4	4.5	540
0.7	0.7	86	4.5	4.6	552
0.8	0.8	99	4.6	4.7	564
0.9	0.9	111	4.7	4.8	576
1	1	123	4.8	4.9	588
1.1	1.1	136	4.9	5	601
1.2	1.2	148	5	5.1	613
1.3	1.3	160	5.1	5.2	631
1.4	1.4	173	5.2	5.3	647
1.5	1.5	185	5.3	5.4	663
1.6	1.6	197	5.4	5.5	679
1.7	1.7	210	5.5	5.6	695
1.8	1.8	222	5.6	5.7	711
1.9	1.9	234	5.7	5.8	727
2	2	247	5.8	5.9	743
2.1	2.1	259	5.9	6	759
2.2	2.2	271	6	6.1	775
2.3	2.3	283	6.1	6.2	791
2.4	2.4	295	6.2	6.3	807
2.5	2.5	308	6.3	6.4	823
2.6	2.6	320	6.4	6.6	839
2.7	2.7	332	6.5	6.7	855
2.8	2.8	344	6.6	6.8	871
2.9	2.9	356	6.7	6.9	887
3	3.1	369	6.8	7	903
3.1	3.2	381	6.9	7.1	919
3.2	3.3	393	7	7.2	935
3.3	3.4	405	7.1	7.3	951
3.4	3.5	417	7.2	7.4	967
3.5	3.6	430	7.3	7.5	983
3.6	3.7	442	7.4	7.6	999
3.7	3.8	454	7.5	7.7	1015

Table A-2: Pullout Table from Germann Instruments for 6/5/2014-6/5/2015 (continued**)**

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
7.6	7.8	1031
7.7	7.9	1047
7.8	8	1063
7.9	8.1	1079
8	8.3	1095
8.1	8.4	1111
8.2	8.5	1127
8.3	8.6	1143
8.4	8.7	1159
8.5	8.8	1175
8.6	8.9	1191
8.7	9	1207
8.8	9.1	1223
8.9	9.2	1239
9	9.3	1255
9.1	9.4	1271
9.2	9.5	1287
9.3	9.6	1303
9.4	9.7	1319
9.5	9.8	1335
9.6	9.9	1351
9.7	10.1	1367
9.8	10.2	1383
9.9	10.3	1399
10	10.4	1415
10.1	10.5	1430
10.2	10.6	1445
10.3	10.7	1460
10.4	10.8	1474
10.5	10.9	1489
10.6	11	1504
10.7	11.1	1519
10.8	11.2	1534
10.9	11.3	1548
11	11.4	1563
11.1	11.4	1578
11.2	11.5	1593
11.3	11.6	1608

Reading (kN)	Actual Pull Force (kN))	Cylinder Strength (psi)
11.4	11.7	1623
11.5	11.8	1637
11.6	11.9	1652
11.7	12	1667
11.8	12.1	1682
11.9	12.2	1697
12	12.3	1711
12.1	12.4	1726
12.2	12.5	1741
12.3	12.6	1756
12.4	12.7	1771
12.5	12.8	1786
12.6	12.9	1800
12.7	13	1815
12.8	13.1	1830
12.9	13.2	1845
13	13.3	1860
13.1	13.4	1874
13.2	13.5	1889
13.3	13.6	1904
13.4	13.7	1919
13.5	13.8	1934
13.6	13.9	1949
13.7	14	1963
13.8	14.1	1978
13.9	14.2	1993
14	14.3	2008
14.1	14.4	2023
14.2	14.5	2037
14.3	14.6	2052
14.4	14.7	2067
14.5	14.8	2082
14.6	14.9	2097
14.7	15	2112
14.8	15.1	2126
14.9	15.2	2141
15	15.3	2156
15.1	15.4	2171

Table A-2: Pullout Table from Germann Instruments for 6/5/2014-6/5/2015 (continued)

Reading (kN)	Actual Pull Force (kN))	Cylinder Strength (psi)
15.2	15.5	2187
15.3	15.6	2202
15.4	15.7	2218
15.5	15.8	2233
15.6	15.9	2249
15.7	16	2264
15.8	16.1	2280
15.9	16.2	2295
16	16.3	2310
16.1	16.4	2326
16.2	16.5	2341
16.3	16.6	2357
16.4	16.7	2372
16.5	16.8	2388
16.6	16.9	2403
16.7	17	2418
16.8	17.1	2434
16.9	17.2	2449
17	17.3	2465
17.1	17.4	2480
17.2	17.5	2496
17.3	17.6	2511
17.4	17.7	2527
17.5	17.8	2542
17.6	17.9	2557
17.7	18	2573
17.8	18.1	2588
17.9	18.2	2604
18	18.3	2619
18.1	18.4	2635
18.2	18.5	2650
18.3	18.6	2665
18.4	18.7	2681
18.5	18.9	2696
18.6	19	2712
18.7	19.1	2727
18.8	19.2	2743
18.9	19.3	2758

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
19	19.4	2774
19.1	19.5	2789
19.2	19.6	2804
19.3	19.7	2820
19.4	19.8	2835
19.5	19.9	2851
19.6	20	2866
19.7	20.1	2882
19.8	20.2	2897
19.9	20.3	2913
20	20.4	2928
20.1	20.5	2943
20.2	20.6	2957
20.3	20.7	2972
20.4	20.8	2987
20.5	20.9	3001
20.6	21	3016
20.7	21.1	3030
20.8	21.2	3045
20.9	21.3	3060
21	21.4	3074
21.1	21.5	3089
21.2	21.5	3104
21.3	21.6	3118
21.4	21.7	3133
21.5	21.8	3148
21.6	21.9	3162
21.7	22	3177
21.8	22.1	3191
21.9	22.2	3206
22	22.3	3221
22.1	22.4	3235
22.2	22.5	3250
22.3	22.6	3265
22.4	22.7	3279
22.5	22.8	3294
22.6	22.9	3309
22.7	23	3323

Table A-2: Pullout Table from Germann Instruments for 6/5/2014-6/5/2015 (continued)

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
22.8	23.1	3338
22.9	23.2	3353
23	23.3	3367
23.1	23.4	3382
23.2	23.5	3396
23.3	23.6	3411
23.4	23.7	3426
23.5	23.8	3440
23.6	23.9	3455
23.7	24	3470
23.8	24.1	3484
23.9	24.2	3499
24	24.3	3514
24.1	24.4	3528
24.2	24.5	3543
24.3	24.6	3557
24.4	24.6	3572
24.5	24.7	3587
24.6	24.8	3601
24.7	24.9	3616
24.8	25	3632
24.9	25.1	3649
25	25.2	3667
25.1	25.3	3684
25.2	25.4	3702
25.3	25.5	3719
25.4	25.6	3736
25.5	25.7	3754
25.6	25.8	3771
25.7	25.9	3788
25.8	26	3806
25.9	26.1	3823
26	26.2	3840
26.1	26.3	3858
26.2	26.4	3875
26.3	26.5	3893
26.4	26.6	3910
26.5	26.7	3927

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
26.6	26.8	3945
26.7	26.9	3962
26.8	27	3979
26.9	27.1	3997
27	27.1	4014
27.1	27.2	4031
27.2	27.3	4049
27.3	27.4	4066
27.4	27.5	4083
27.5	27.6	4101
27.6	27.7	4118
27.7	27.8	4135
27.8	27.9	4153
27.9	28	4170
28	28.1	4188
28.1	28.2	4205
28.2	28.3	4222
28.3	28.4	4240
28.4	28.5	4257
28.5	28.6	4274
28.6	28.7	4292
28.7	28.8	4309
28.8	28.9	4326
28.9	29	4344
29	29.1	4361
29.1	29.2	4378
29.2	29.3	4396
29.3	29.3	4413
29.4	29.4	4430
29.5	29.5	4448
29.6	29.6	4465
29.7	29.7	4483
29.8	29.8	4500
29.9	29.9	4517
30	30	4535
30.1	30.1	4553
30.2	30.2	4571
30.3	30.3	4589

Table A-2: Pullout Table from Germann Instruments for 6/5/2014-6/5/2015 (continued)

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)	Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
30.4	30.4	4607	34.2	34.2	5292
30.5	30.5	4625	34.3	34.3	5310
30.6	30.6	4643	34.4	34.4	5328
30.7	30.7	4661	34.5	34.5	5346
30.8	30.8	4679	34.6	34.6	5364
30.9	30.9	4697	34.7	34.7	5382
31	31	4715	34.8	34.8	5400
31.1	31.1	4733	34.9	34.9	5418
31.2	31.2	4751	35	35	5436
31.3	31.3	4769	35.1	35.1	5454
31.4	31.4	4787	35.2	35.2	5471
31.5	31.5	4805	35.3	35.3	5489
31.6	31.6	4823	35.4	35.4	5506
31.7	31.7	4841	35.5	35.5	5524
31.8	31.8	4859	35.6	35.6	5541
31.9	31.9	4877	35.7	35.7	5559
32	32	4895	35.8	35.8	5576
32.1	32.1	4913	35.9	35.9	5594
32.2	32.2	1931	36	36	5611
32.3	32.3	4949	36.1	36.1	5629
32.4	32.4	4967	36.2	36.2	5646
32.5	32.5	4985	36.3	36.2	5664
32.6	32.6	5003	36.4	36.3	5682
32.7	32.7	5021	36.5	36.4	5699
32.8	32.8	5039	36.6	36.5	5717
32.9	32.9	5057	36.7	36.6	5734
33	33	5076	36.8	36.7	5752
33.1	33.1	5094	36.9	36.8	5769
33.2	33.2	5112	37	36.9	5787
33.3	33.3	5130	37.1	37	5804
33.4	33.4	5148	37.2	37.1	5822
33.5	33.5	5166	37.3	37.2	5839
33.6	33.6	5184	37.4	37.3	5857
33.7	33.7	5202	37.5	37.4	5874
33.8	33.8	5220	37.6	37.5	5892
33.9	33.9	5238	37.7	37.6	5909
34	34	5256	37.8	37.7	5927
34.1	34.1	5274	37.9	37.8	5945

Table A-2: Pullout Table from Germann Instruments for 6/5/2014-6/5/2015 (continued)

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)	Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
38	37.9	5962	41.8	41.5	6622
38.1	38	5980	41.9	41.6	6639
38.2	38.1	5997	42	41.7	6656
38.3	38.2	6015	42.1	41.8	6673
38.4	38.3	6032	42.2	41.9	6691
38.5	38.4	6050	42.3	42	6708
38.6	38.5	6067	42.4	42.1	6725
38.7	38.6	6085	42.5	42.2	6742
38.8	38.7	6102	42.6	42.3	6759
38.9	38.8	6120	42.7	42.4	6776
39	38.9	6137	42.8	42.5	6794
39.1	39	6155	42.9	42.6	6811
39.2	39.1	6172	43	42.7	6828
39.3	39.2	6190	43.1	42.8	6845
39.4	39.2	6207	43.2	42.9	6862
39.5	39.3	6225	43.3	43	6879
39.6	39.4	6243	43.4	43.1	6897
39.7	39.5	6260	43.5	43.1	6914
39.8	39.6	6278	43.6	43.2	6931
39.9	39.7	6295	43.7	443.3	6948
40	39.8	6313	43.8	43.4	6965
40.1	39.9	6330	43.9	43.5	6983
40.2	40	6347	44	43.6	7000
40.3	40.1	6364	44.1	43.7	7017
40.4	40.2	6381	44.2	43.8	7034
40.5	40.3	6399	44.3	43.9	7051
40.6	40.4	6416	44.4	44	7068
40.7	40.5	6433	44.5	44.1	7086
40.8	40.6	6450	44.6	44.2	7103
40.9	40.7	6467	44.7	44.3	7120
41	40.8	6484	44.8	44.4	7137
41.1	40.9	6502	44.9	44.5	7154
41.2	41	6519	45	44.6	7171
41.3	41.1	6536	45.1	44.7	7190
41.4	41.2	6553	45.2	44.8	7209
41.5	41.3	6570	45.3	44.9	7228
41.6	41.3	6588	45.4	45	7247
41.7	41.4	6605	45.5	45.1	7266

Table A-2: Pullout Table from Germann Instruments for 6/5/2014-6/5/2015 (continued)

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
45.6	45.2	7285
45.7	45.3	7304
45.8	45.4	7323
45.9	45.5	7342
46	45.6	7361
46.1	45.7	7380
46.2	45.8	7399
46.3	45.9	7417
46.4	46	7436
46.5	46.1	7455
46.6	46.2	7474
46.7	46.3	7493
46.8	46.4	7512
46.9	46.6	7531
47	46.7	7550
47.1	46.8	7569
47.2	46.9	7588
47.3	47	7607
47.4	47.1	7626
47.5	47.2	7644
47.6	47.3	7663
47.7	47.4	7682
47.8	47.5	7701
47.9	47.6	7720
48	47.7	7739
48.1	47.8	7758
48.2	47.9	7777
48.3	48	7796
48.4	48.1	7815
48.5	48.2	7834
48.6	48.3	7853
48.7	48.4	7871
48.8	48.5	7890
48.9	48.6	7909
49	48.7	7928
49.1	48.8	7947
49.2	49	7966
49.3	49.1	7985

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
49.4	49.2	8004
49.5	49.3	8023
49.6	49.4	8042
49.7	49.5	8061
49.8	49.6	8080
49.9	49.7	8098
50	49.8	8117
50.1	49.9	8135
50.2	50	8152
50.3	50.1	8169
50.4	50.2	8187
50.5	50.3	8204
50.6	50.4	8222
50.7	50.5	8239
50.8	50.6	8256
50.9	50.6	8274
51	50.7	8291
51.1	50.8	8308
51.2	50.9	8326
51.3	51	8343
51.4	51.1	8360
51.5	51.2	8378
51.6	51.3	8395
51.7	51.4	8412
51.8	51.5	8430
51.9	51.6	8447
52	51.7	8464
52.1	51.8	8482
52.2	51.9	8499
52.3	52	8517
52.4	52.1	8534
52.5	52.2	8551
52.6	52.3	8569
52.7	52.4	8586
52.8	52.5	8603
52.9	52.6	8621
53	52.7	8638
53.1	52.8	8655

Table A-2: Pullout Table from Germann Instruments for 6/5/2014-6/5/2015 (continued)

Reading (kN)	Actual Pull Force (kN)	Cylinder Strength (psi)
53.2	52.8	8673
53.3	52.9	8690
53.4	53	8707
53.5	53.1	8725
53.6	53.2	8742
53.7	53.3	8760
53.8	53.4	8777
53.9	53.5	8794
54	53.6	8812
54.1	53.7	8829
54.2	53.8	8846
54.3	53.9	8864
54.4	54	8881
54.5	54.1	8898
54.6	54.2	8916
54.7	54.3	8933
54.8	54.4	8950
54.9	54.5	8968
55	54.6	8985
55.1	54.7	9003
55.2	54.8	9020
55.3	54.9	9038
55.4	55	9056
55.5	55.1	9073
55.6	55.2	9091
55.7	55.3	9109
55.8	55.4	9126
55.9	55.4	9144
56	55.5	9161
56.1	55.6	9179
56.2	55.7	9197
56.3	55.8	9214
56.4	55.9	9232
56.5	56	9250
56.6	56.1	9267
56.7	56.2	9285
56.8	56.3	9303
56.9	56.4	9320
57	56.5	9338
57.1	56.6	9356
57.2	56.7	9373
57.3	56.8	9391
57.4	56.9	9408
57.5	57	9426
57.6	57.1	9444
57.7	57.2	9461
57.8	57.3	9479
57.9	57.4	9497
58	57.5	9514
58.1	57.6	9532
58.2	57.7	9550
58.3	57.8	9567
58.4	57.9	9585
58.5	58	9602
58.6	58.1	9620
58.7	58.2	9638
58.8	58.3	9655
58.9	58.4	9673
59	58.5	9691
59.1	58.6	9708
59.2	58.7	9726
59.3	58.8	9744
59.4	589.9	9761
59.5	59	9779
59.6	59	9796
59.7	59.1	9814
59.8	59.2	9832
59.9	59.3	9849
60	59.4	9867

Appendix B

Collected Strength Data

Table B-1: Molded Cylinder Strengths from Cast RG4000CA

Age (days)	6x12 Cylinder Strength (psi)	4x8 Cylinder Strength (psi)
7-Day	3500	3720
	3820	3830
	Outlier	3550
28-Day	4710	3620
	4480	3860
	4450	4120
42-Day	4850	4770
	5040	5130
	4650	4870
91-Day	5350	5860
	5550	5330
	5900	5900
365-Day	6050	5960
	5430	5360
	5530	5540

Table B-2: Molded Cylinder Strengths from Cast LS4000CT

Age (days)	6x12 Cylinder Strength (psi)	4x8 Cylinder Strength (psi)
7-Day	2920	2690
	2880	3040
	2730	2760
28-Day	3750	4000
	3520	3890
	3760	4000
42-Day	3550	3360
	3570	3190
	3710	3380
91-Day	4280	4340
	4240	4200
	4130	4180
365-Day	4500	4180
	4700	4480
	4440	4560

Table B-3: Molded Cylinder Strengths from Cast RG4000CT

Age (days)	6x12 Cylinder Strength (psi)	4x8 Cylinder Strength (psi)
7-Day	2690	2740
	2480	2780
	2490	NC
28-Day	3460	3480
	3470	3560
	3380	3400
42-Day	3680	3900
	3640	3820
	3710	3750
91-Day	3910	4140
	4050	4170
	3950	4270
365-Day	3850	4170
	3970	4180
	4080	4030

Table B-4: Molded Cylinder Strengths from Cast RG4000SC

Age (days)	6x12 Cylinder Strength (psi)	4x8 Cylinder Strength (psi)
7-Day	2450	2620
	2550	2670
	2610	2710
28-Day	3400	3420
	3550	3640
	3580	3740
42-Day	3620	3600
	3570	3480
	3780	3570
91-Day	3960	4000
	4160	3670
	3880	4220
365-Day	4390	3920
	4240	4340
	NC	4130

Table B-5: Molded Cylinder Strengths from Cast GR4000CT

Age (days)	6x12 Cylinder Strength (psi)	4x8 Cylinder Strength (psi)
7-Day	2300	2160
	2300	1970
	2260	2040
28-Day	3190	3060
	3180	3030
	3160	3140
42-Day	3450	3250
	3300	3290
	3330	3250
91-Day	3700	3230
	3580	3410
	3410	3330
365-Day	3690	3410
	3710	3470
	3710	3330

Table B-6: Molded Cylinder Strengths from Cast RG4000FA

Age (days)	6x12 Cylinder Strength (psi)	4x8 Cylinder Strength (psi)
7-Day	2960	3710
	3170	3800
	3190	NC
28-Day	4110	5130
	4300	5250
	4330	5150
42-Day	4050	5160
	4560	4940
	4600	5440
91-Day	5050	5440
	4880	5850
	5360	5380
365-Day	5830	7210
	5880	6910
	5900	5770

Table B-7: Molded Cylinder Strengths from Cast RG8000CT

Age (days)	6x12 Cylinder Strength (psi)	4x8 Cylinder Strength (psi)
7-Day	7100	6900
	8030	7190
	7980	6110
28-Day	8380	6880
	9100	6850
	8930	7270
42-Day	8650	7790
	7960	7560
	8470	7370
91-Day	8100	7330
	7630	7370
	8090	NC
365-Day	9310	8500
	9610	8310
	10120	8400

Table B-8: Molded Cylinder Strengths from Cast LS8000CT

Age (days)	6x12 Cylinder Strength (psi)	4x8 Cylinder Strength (psi)
7-Day	4960	5270
	4830	5340
	4280	4960
28-Day	5740	5800
	5650	6030
	6030	5700
42-Day	5660	7090
	5690	6690
	6350	6520
91-Day	7440	7410
	7630	7800
	7330	7740
365-Day	NC	NC
	NC	NC
	NC	NC

Table B-9: Cast-In-Place Cylinder Strengths from Cast RG4000SC

Age (days)	Exterior CIP Strength (psi)	Interior CIP Strength (psi)
28-Day	3830	3330
	3320	3810
	3490	3760
42-Day	3240	3410
	3120	3910
	2710	3690
91-Day	3260	3540
	3430	3370
	3230	3490
365-Day	4490	4560
	4330	4130
	NC	NC

Table B-10: Cast-In-Place Cylinder Strengths from Cast GR4000CT

Age (days)	Exterior CIP Strength (psi)	Interior CIP Strength (psi)
28-Day	2840	2910
	2940	2900
	3160	3020
42-Day	3110	3260
	2920	3070
	2950	2870
91-Day	3070	3070
	2950	3310
	3350	3110
365-Day	3510	3690
	3540	3660
	NC	3740

Table B-11: Cast-In-Place Cylinder Strengths from Cast RG4000FA

Age (days)	Exterior CIP Strength (psi)	Interior CIP Strength (psi)
28-Day	4790	4450
	4430	4810
	4340	4660
42-Day	4650	4650
	5080	4650
	4690	4140
91-Day	4810	4470
	4550	4730
	4360	4210
365-Day	7080	6620
	6680	6590
	5960	6300

Table B-12: Cast-In-Place Cylinder Strengths from Cast RG8000CT

Age (days)	Exterior CIP Strength (psi)	Interior CIP Strength (psi)
28-Day	7130	7280
	Outlier	Outlier
	7020	7600
42-Day	5750	6320
	6080	7020
	Outlier	Outlier
91-Day	6210	7270
	7020	5960
	7290	Outlier
365-Day	8600	7050
	8090	8060
	9120	Outlier

Table B-13: Cast-In-Place Cylinder Strengths from Cast LS8000CT

Age (days)	Exterior CIP Strength (psi)	Interior CIP Strength (psi)
28-Day	6690	6600
	6400	6570
	6820	6680
42-Day	7200	7780
	7190	6750
	7450	6900
91-Day	6830	7000
	6680	7650
	7860	7950
365-Day	7390	7220
	6820	7230
	6730	7570

Table B-14: Core Strengths from Cast RG4000CA

Age (days)	Exterior Core Strength (psi)	Interior Core Strength (psi)
28-Day	3690	4500
	4150	3840
	4000	3810
42-Day	5060	4440
	4630	4250
	4420	NC
91-Day	6000	5710
	6080	5710
	5710	NC
365-Day	5320	5040
	5390	4780
	5410	4470

Table B-15: Core Strengths from Cast LS4000CT

Age (days)	Exterior Core Strength (psi)	Interior Core Strength (psi)
28-Day	3080	3040
	3120	3350
	3130	3350
42-Day	3110	3250
	2850	3030
	2970	2960
91-Day	3740	3670
	4120	3740
	3650	3790
365-Day	3720	4040
	3760	3660
	3780	3540

Table B-16: Core Strengths from Cast RG4000CT

Age (days)	Exterior Core Strength (psi)	Interior Core Strength (psi)
28-Day	3290	2580
	2860	2980
	2940	2610
42-Day	2960	2970
	3270	3040
	3100	2900
91-Day	3750	3150
	3650	3500
	3610	3290
365-Day	3270	2870
	3720	3070
	3620	3100

Table B-17: Core Strengths from Cast RG4000SC

Age (days)	Exterior Core Strength (psi)	Interior Core Strength (psi)
28-Day	3810	3650
	3510	3510
	3360	3480
42-Day	3580	3610
	3820	3930
	3710	3780
91-Day	3600	3910
	3750	3550
	3530	3600
365-Day	4360	4210
	4430	3960
	4440	3810

Table B-18: Core Strengths from Cast GR4000CT

Age (days)	Exterior Core Strength (psi)	Interior Core Strength (psi)
28-Day	2920	2800
	2950	2710
	3050	2660
42-Day	2980	2710
	2870	2800
	2910	2740
91-Day	3260	2780
	2970	2850
	3210	2760
365-Day	3420	3610
	3520	3070
	3530	3180

Table B-19: Core Strengths from Cast RG4000FA

Age (days)	Exterior Core Strength (psi)	Interior Core Strength (psi)
28-Day	4670	4660
	4720	4940
	4710	4990
42-Day	4590	3960
	4440	4210
	4490	4260
91-Day	4730	4220
	4410	4620
	4610	4630
365-Day	6470	6050
	6800	5240
	6810	5960

Table B-20: Core Strengths from Cast RG8000CT

Age (days)	Exterior Core Strength (psi)	Interior Core Strength (psi)
28-Day	6570	7200
	6320	6900
	Outlier	NC
42-Day	7270	6330
	7350	Outlier
	7110	7040
91-Day	7180	6970
	7300	6840
	7420	6570
365-Day	8290	7150
	8280	6490
	7660	NC

Table B-21: Core Strengths from Cast LS8000CT

Age (days)	Exterior Core Strength (psi)	Interior Core Strength (psi)
28-Day	6490	6070
	6240	6140
	5940	5790
42-Day	5770	6240
	5980	6210
	6290	6170
91-Day	6900	6310
	7050	6720
	6500	6550
365 -Day	6870	6380
	7840	6250
	6270	NC

Table B-22: Pullout Strengths from Cast RG4000CA

	Exterior Pullouts		**Interior Pullouts**	
	Pullout Force (kN)	**Equivalent Compressive Strength (psi)**	**Pullout Force (kN)**	**Equivalent Compressive Strength (psi)**
28-Day	37.4	5801	47.1	7506
	44.4	7014	31.1	4685
	36.9	5715	37.4	5801
	34.6	5314	33.4	5098
	37.4	5801	30	4487
42-Day	37.1	5749	NC	NC
	36.8	5697	33.5	5116
	37.1	5749	33.4	5098
	38.7	6026	32.5	4936
	42.5	6684	32.5	4936
91-Day	40.4	6320	41	6424
	42.6	6702	30.7	4613
	28.2	4174	41.7	6545
	36.4	5628	42.9	6754
	40.9	6407	27.9	4122
365-Day	43.6	6931	NC	NC
	34	5256	40.6	6416
	38.8	6102	37.2	5822
	44.1	7017	37.5	5874
	34	5256	35	5436

Table B-23: Pullout Strengths from Cast LS4000CT

	Exterior Pullouts		Interior Pullouts	
	Pullout Force (kN)	**Equivalent Compressive Strength (psi)**	**Pullout Force (kN)**	**Equivalent Compressive Strength (psi)**
28-Day	24	3468	24.6	3559
	25.6	3723	24.7	3574
	20.4	2919	24.2	3498
	21.6	3102	28.5	4226
	24.3	3513	28.6	4244
42-Day	33.2	5062	27.2	4001
	20.5	2934	26.9	3949
	30.6	4595	21.5	3087
	27.7	4087	25.9	3775
	NC	NC	NC	NC
91-Day	25	3620	27.4	4035
	31	4667	30.5	4577
	27.3	4018	27.5	4053
	31.5	4757	25.7	3740
	24.1	3483	26.8	3931
365-Day	28.1	4205	26.8	3979
	31.2	4751	32.3	4949
	30.3	4589	29.6	4465
	29.2	4396	34.4	5328
	NC	NC	28.5	4274

Table B-24: Pullout Strengths from Cast RG4000CT

	Exterior Pullouts		Interior Pullouts	
	Pullout Force (kN)	**Equivalent Compressive Strength (psi)**	**Pullout Force (kN)**	**Equivalent Compressive Strength (psi)**
28-Day	19.1	2723	19.4	2768
	19.4	2768	19.4	2768
	22	3163	19.6	2798
	18.5	2633	18.3	2603
	22.9	3300	20.5	2934
42-Day	24.2	3498	18.1	2573
	21.8	3133	16.1	2274
	22	3163	22.2	3193
	20.8	2980	19.7	2813
	20.3	2904	NC	NC
91-Day	21.5	3087	16.9	2393
	18.1	2573	20	2858
	23.9	3452	24.5	3544
	18.8	2678	20.6	2950
	19.3	2753	24.2	3498
365-Day	23.6	3455	16	2310
	26.5	3927	22.6	3309
	26.7	3962	21.3	3118
	23.9	3499	21.4	3133
	23.3	3411	26	3840

Table B-25: Pullout Strengths from Cast RG4000SC

	Exterior		Interior	
	Pullout Force (kN)	**Equivalent Compressive Strength (psi)**	**Pullout Force (kN)**	**Equivalent Compressive Strength (psi)**
28-Day	18.8	2678	23	3315
	21.6	3102	13.9	1948
	19.7	2813	20.8	2980
	22.1	3178	15.3	2154
	16.3	2304	20	2858
42-Day	22	3163	22.2	3193
	21.2	3041	20.6	2950
	19.9	2843	24.7	3574
	23	3315	20.5	2934
	21.1	3026	25.5	3706
91-Day	15.1	2124	17	2408
	18.3	2603	16.9	2393
	21.5	3087	17.6	2498
	18.2	2588	17.5	2483
	20.5	2934	18.4	2618
365-Day	30.2	4571	20.7	3030
	26.9	3997	22.6	3309
	27.6	4118	21.8	3191
	21.7	3177	19.3	2820
	22.4	3279	28	4188

Table B-26: Pullout Strengths from Cast GR4000CT

	Exterior Pullouts		Interior Pullouts	
	Pullout Force (kN)	**Equivalent Compressive Strength (psi)**	**Pullout Force (kN)**	**Equivalent Compressive Strength (psi)**
28-Day	16.6	2348	15.6	2199
	18.4	2618	12.7	1773
	18.2	2588	13.2	1846
	20	2858	17.8	2528
	20.3	2904	17.6	2498
42-Day	22.2	3193	16.3	2304
	19.3	2753	14.7	2065
	21.2	3041	13.5	1890
	19.4	2768	16.6	2348
	21.3	3056	17.4	2468
91-Day	24	3468	18.3	2603
	20.5	2934	16	2259
	20.3	2904	20.4	2919
	16.5	2334	12.3	1715
	11.6	1613	12	1671
365-Day	20.6	3016	16.7	2418
	22.1	3235	17.9	2604
	23.3	3411	NC	NC
	NC	NC	16.4	2372
	20.6	3016	22.4	3279

Table B-27: Pullout Strengths from Cast RG4000FA

	Exterior		Interior	
	Pullout Force (kN)	**Equivalent Compressive Strength (psi)**	**Pullout Force (kN)**	**Equivalent Compressive Strength (psi)**
28-Day	35.4	5455	26.5	3879
	35.3	5438	27.2	4001
	35	5386	NC	NC
	38.5	5991	35.3	5438
	31.2	4703	31.2	4703
42-Day	34.3	5260	22.8	3285
	31.5	4757	27.8	4105
	29.9	4469	31.6	4775
	32.2	4882	32.3	4900
	25.3	3671	21.6	3102
91-Day	NC	NC	41	6424
	28.9	4296	25.6	3723
	28.6	4244	NC	NC
	34.8	5350	25.6	3723
	43.3	6823	34.9	5368
365-Day	50	8117	41.8	6622
	47	7550	45	7171
	57	9338	NC	NC
	42.9	6811	40.4	6381
	35.7	5559	39.9	6295