Final Report

The Effect of Test Cylinder Size on the Compressive Strength of Sulfur Capped Concrete Specimens

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ABSTRACT

The new trend of using high-strength concrete in construction has caused a need for the use of 4 x 8 in. cylinders for assurance testing. A controlling factor that affects the size of specimen that can be tested in a compression machine is the strength of the concrete on evaluation. Some testing machines are not able to produce the force needed to break high-strength 6 x 12 in. concrete cylinders. If 4 x 8 in. cylinders are to be used in quality assurance testing, the relationship between fc4 and fc6 needs to be understood in order to ensure that concrete with sufficient strength is provided. If the average compression machine operates safely, rarely exceeding 80% of its capacity, and has a capacity of 250,000 lbs, the machine can test a 6 x 12 in. cylinder with a compressive strength of approximately 7,000 psi. The same machine can test a 4 x 8 in. cylinder of approximately 16,000 psi.

A 4 x 8 in. cylinder weighs about 9 lb compared to a 6 x 12 in. cylinder, which weighs about 30 lb. This might suggest that because 4 x 8 in. cylinders are lighter and can easily be handled, collection of quality control and assurance specimens would be easier for contractors and inspectors. The advantages of using smaller specimens are: 1) easier handling; b) less required storage space; c) less capacity required of testing machines.

This research project was born from the need to determine a correlation between the strength of the standard size 6 x 12 in. cylindrical specimen and the strength of a 4 x 8 in. cylindrical specimen made from the same batch of concrete. The objectives of this study are to review the factors that may affect the compressive strength, those that may affect the strength obtained by 4 x 8 in. and 6 x 12 in. cylinders, and the variability associated with these tests. An extensive laboratory testing program was developed to evaluate the desired goals of the project. A total of 359 4 x 8 in. and 357 6 x 12 in. cylinders were tested.

The factors that were studied to evaluate the effect of cylinder size on concrete compressive strength were aggregate size, technician, compressive strength, and age of specimen at testing. It was determined that compressive strength was the only factor significant in affecting the ratio of 4 x 8 in. cylinder strength to 6 x 12 in. cylinder strength. Compressive strength was also the only factor significant in affecting the within-test variability of each batch of concrete. It is recommended that 4 x 8 in. cylinders may be implemented for quality assurance testing if the design strength of concrete is greater than 5,000 psi and the capacity of the testing machine will not allow the testing of 6 x 12 in. cylinders based on the design strength. However, if 4 x 8 in. cylinders are used, a correlation between the 4 x 8 in. and 6 x 12 in. cylinders should be determined using a capable machine for the project.

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INTRODUCTION

1.1 BACKGROUND

Most of our current structural concrete design provisions are referenced to the compressive strength obtained from testing 6 x 12 in. concrete cylinders cured under standard laboratory controlled conditions. Unfortunately, when other cylinder sizes are used, indications are that the tested concrete strength may be affected. This opens the question how one should account for this apparent difference in tested concrete strength due to the use of a different cylinder size. The new trend of using high-strength concrete in construction has caused a need for 4 x 8 in. cylinders for assurance testing. A controlling factor that affects the size of specimen that can be tested in a compression machine is the strength of the concrete being evaluated. Some testing machines are not able to produce the force needed to break high-strength 6 x 12 in. concrete cylinders. As laboratories and testing agencies are very often equipped with testing machines having full load capacities no greater than 300,000 lbf, the maximum compressive strength of concrete that can be tested on 6 x 12 in. specimens is just over 10,000 psi when operating at full load, which is not safe on a routine basis (Aitcin et al. 1994). The required force to break a 4 x 8 in. cylinder is 44% of that required to break a 6 x 12 in. cylinder of the same strength solely based on a ratio of the two circular cross-sectional areas. This would allow machines that could not break 6 x 12 in. cylinders with strengths over 10,000 psi to break 4 x 8 in. cylinders with strengths in excess of 20,000 psi.

A 4 x 8 in. cylinder weighs about 9 lb compared to a 6 x 12 in. cylinder, which weighs about 30 lb, almost three times as much. This might suggest that because 4 x 8 in. cylinders are lighter and can easily be handled, collection and storage of quality control and assurance specimens would be easier for contractors and inspectors. One aspect of concern when using 4 x 8 in. cylinders is the size of maximum coarse aggregate used in concrete. Mixes containing a nominal maximum coarse aggregate size of 1.5 inches, or greater in some instances, are used in today's concrete industry. AASHTO T 126 (1993) states that the size of a cylinder mold shall not be smaller than 3 times the nominal maximum coarse aggregate size. This limits 4 x 8 in. cylinders to having a 1-inch nominal maximum coarse aggregate size. Also there is no standard aggregate size between 1 inch and 1.5 inches, leaving a #57 coarse aggregate the largest possible gradation for a 4 x 8 in. cylinder. The obvious advantages of using smaller specimens are: a) ease in handling and transportation; b) smaller required storage space; c) lower capacity required of testing machines; and d) the economic advantages of reduced costs for molds, capping materials, and concrete (Day and Haque 1993).

1.2 OBJECTIVES

The objective of this research project was to determine a correlation between the compressive strength of 4 x 8 in. and 6 x 12 in. cylinders and the variability associated with those results. This documentation will review the factors that may affect the compressive strength, those that may affect the strength obtained by 4 x 8 in and 6 x 12 in. cylinders, and the variability associated with these tests. Based on laboratory testing of materials used in the Alabama concrete industry, the effect of test cylinder size on concrete compressive strength will be evaluated.

1.3 REPORT SCOPE AND OUTLINE

Chapter 2 will provide a literature review of published material that is relevant to this project. The portion of Chapter 2 that reviews the factors that affect the compressive strength of concrete will serve as a general information source concerning the basic properties associated with concrete strength. The factors affecting the correlation between the strengths of 4 x 8 in. and 6 x 12 in. cylinders will be reviewed next. The variability of normal and high-strength concrete will be discussed as well as changes made to design codes based on the variability of high-strength concrete.

Chapter 3 is a summary the logic used in determining the laboratory testing program based on the conclusions from the literature review and the available materials and equipment. Information given in this chapter will consist of concrete mixture proportions and material properties. Chapter 4 provides the methods and equipment used to carry out the laboratory testing program. Fresh and hardened concrete test procedures will be explained along with their accompanying ASTM standards.

Chapter 5 is a presentation of the results of the laboratory testing program in text and graphical form. Fresh concrete properties will be given for each batch of concrete. The ratio of 4 x 8 in. and 6 x 12 in. cylinder strengths and the variability of the results will be analyzed through graphical representations and statistical analysis. The results of the laboratory testing program will be compared to the conclusions of the literature review.

Chapter 6 is a discussion of several suggested ways that 4 x 8 in. cylinders could practically be implemented for quality assurance testing. Recommendations will then be made based on the conclusions of the literature review, the results of the test data, and previously suggested implementation procedures. Chapter 7 is presentation of the comparisons between the literature review conclusions and the test results. It will also present the recommendations given in Chapter 6. Appendix A will give, in tabular form, individual test results for all cylinders.

Chapter 2

LITERATURE REVIEW

2.1 BACKGROUND

The objective of this research project was to determine a correlation between the compressive strengths of 4 x 8 in. and 6 x 12 in. cylinders and the variability associated with those results. Therefore, the main areas that are covered in this literature review are factors that affect the compressive strength of concrete, factors that affect the correlation between the strengths of 4 x 8 in. and 6 x 12 in. cylinders, the variability associated with these two cylinder sizes, and the variability of high-strength concrete. The conclusions drawn from the literature review will help determine what factors to study during the laboratory component this research project.

2.2 NOTATION

Throughout this project, the following notation was used:

$$f_{c4} = k_s \times f_{c6} \qquad \text{Equation 2.1}$$

where, f_{c4} = compressive strength of a 4 x 8 in. cylinder,
f_{c6} = compressive strength of a 6 x 12 in. cylinder, and
k_s = the strength conversion factor, correlating the 4 x 8 in. cylinder to the 6 x 12 in. cylinder strength.

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2.3 COMPARISON BETWEEN AASHTO AND ASTM STANDARDS

In the United States, most transportation agencies abide by The American Association of State and Highway Transportation Officials (AASHTO) or The American Society for Testing and Materials (ASTM) standards. The Alabama Department of Transportation uses AASHTO standards exclusively. Methods of curing, consolidating, sulfur capping, preparing, and loading specimens are identical between AASHTO and ASTM standards. The main differences between these standards include how they address the allowable specimen size and capping method.

2.3.1 Specimen Size

ASTM, AASHTO, and Canadian Standards Association (CSA) allow the use of 4 x 8 in. cylinders. CSA standards state that if non-standard cylinders are used, then strengths must be correlated to strengths of 6 x 12 in. cylinders (Day 1994 b). ASTM and AASHTO both state that "cylinders for such tests as compressive strength, Young's modulus of elasticity, creep, and splitting tensile strength may be of various sizes with a minimum of 2-in. diameter by 4-in. length. Where correlation or comparison with field-made cylinders is desired, the cylinders shall be 6 x 12 in." (ASTM C 192-00, AASHTO T 126-93). There are certain restrictions placed on the use of 4 x 8 in. cylinders. AASHTO T 126-93 and ASTM C 192-00 state that "the diameter of a cylindrical specimen or minimum cross-sectional dimension of a rectangular section shall be at least three times the nominal maximum size of the coarse aggregate in the concrete." This limits the nominal maximum size of coarse aggregate in a 4 x 8 in. cylinder to 1 inch, which effectively limits the maximum aggregate gradation to a No. 57 stone.

2.3.2 Neoprene Pads for Capping

AASHTO and ASTM both allow the use of neoprene caps as an acceptable capping method for cylindrical concrete specimens. AASHTO T 22 (1992) allows neoprene pads to be used only on 6 x 12 in. cylinders and states that ASTM C 1231 (2000) allows neoprene pads to be used on both 4 x 8 in. cylinders and 6 x 12 in. cylinders for acceptance testing as long as the compressive strength is between 1,500 and 12,000 psi. AASHTO T 22 (1992) states "neoprene caps should be considered as an acceptable substitute for sulfur-mortar caps without correction for apparent strength differences." To be acceptable, ASTM states "tests must demonstrate that at a 95 % confidence level (α = 0.05), the average strength obtained using unbonded caps is not less than 98 % of the average strength of companion cylinders capped or ground" (ASTM C 1231-2000).

2.4 FACTORS THAT AFFECT THE COMPRESSIVE STRENGTH OF CONCRETE

In general, there are many factors associated with the compressive strength of concrete, most of them being interdependent. Some of the important parameters that may affect the compressive strength of concrete are discussed in the following sections.

2.4.1 Water-Cement Ratio

Under full compaction, compressive strength is inversely proportional to the water-cement ratio as shown in Figure 2.1 and is given by the relationship developed by Duff Abrams (1919):

$$f_c = \frac{K_1}{K_2^{w/c}} \qquad \text{Equation 2.2}$$

where, f_c = concrete compressive strength,

K_1 = empirical constant,

K_2 = empirical constant, and

w/c = water to cement ratio

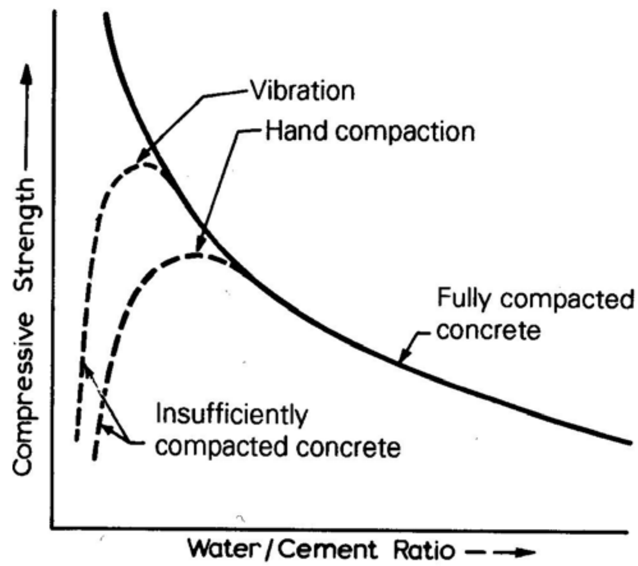

Figure 2.1: Compressive strength and water-cement ratio (Neville 1996)

The water-cement ratio is a very important factor in the determination of porosity and eventually the strength of concrete (Neville 1996). An increase in temperature increases the rate of the exothermic hydration reaction and also the development of strength with time (Neville 1996).

In practical applications it is found that the water-cement ratio is usually the most important factor with respect to strength (Neville 1996). However the situation is best summarized by Gilkey (1961) who states that "for a given cement and acceptable aggregates, the strength that may be developed by a workable, properly placed mixture of cement, aggregate, and water (under the same mixing, curing, and testing conditions) is influenced by the 1) ratio of cement to mixing water 2) ratio of cement to aggregate 3) grading, surface texture, shape, strength, and stiffness of aggregate particles 4) maximum size of aggregate."

An exception to the theory given by Abrams (1919) is the behavior of strength at very low water-cement ratio which is explained in the following discussion by Mehta and Monteiro (1993). Mehta and Monteiro (1993) stated that "in low and medium-strength concrete made with normal aggregate, both the transition zone porosity and the matrix porosity determine the strength, and a direct relation between the water-cement ratio and the concrete strength holds. This seems no longer to be the case in high-strength concretes."

2.4.2 Coarse Aggregate

The strength of concrete is dependant on size, shape, grading, surface texture mineralogy of the aggregate, strength, stiffness and the maximum size of aggregate as seen in Figure 2.2 (Gilkey 1961, Mehta and Monteiro 1993).

Mehta and Monteiro (1993) suggested that the effect of aggregate strength on the compressive strength of concrete is not considered in the case of normal-strength concrete, as it is much stronger than the transition zone and cement paste matrix. Mehta and Monteiro (1993) also explained that the transition zone and the cement paste matrix would fail before the aggregate and thus nullify the effect of the strength of aggregate. Kosmatka et al. (2002) also suggested that the aggregate strength is usually not a factor in normal strength concrete as the failure is generally determined by the cement paste-aggregate bond.

Much research has linked the bonding of the aggregate to the strength of concrete. Neville and Brooks (1987) explained that greater aggregate surface areas result in better bonding between the aggregate and the cement paste. They also observed that rough aggregates tend to exhibit better bonding than smooth aggregates. Jones and Kaplan (1957) made similar observations as Neville and Brooks (1987) but linked the surface properties to the cracking stress suggesting rough aggregates would crack at a higher stress compared to smooth aggregates.

Figure 2.2 shows the effect of water-cement ratio and the maximum aggregate size on compressive strength. It can be seen that compressive strength decreases with an increase in maximum coarse aggregate size especially for concretes with low water-cement ratios. It should be noted that the compressive strength is more sensitive to the water-cement ratio than the maximum aggregate size.

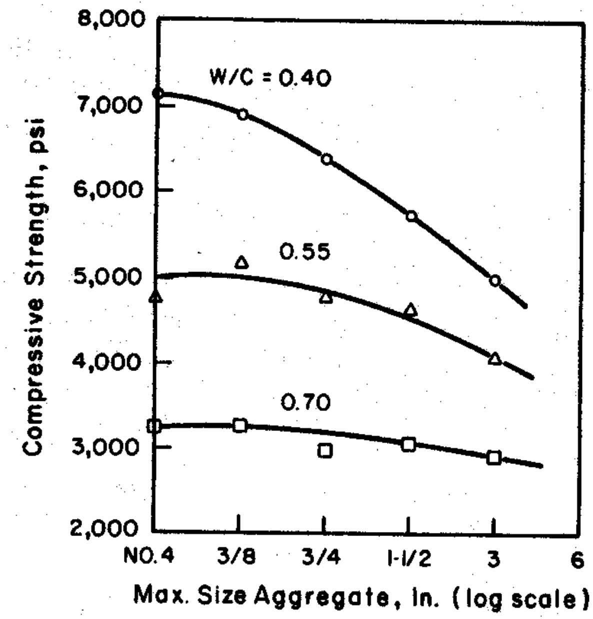

Figure 2.2: Aggregate size, w/c, and compressive strength (Cordon and Gillespie 1963)

2.4.3 Air Entrainment

Air entrainment is the incorporation of air bubbles into the concrete by either using an air-entraining admixture or air-entraining cement (Kosmatka et al. 2002). There are two forms of air found in concrete: entrapped and entrained air.

As seen from Figure 2.3 (a), entrained air causes a reduction in compressive strength at a particular water-cement ratio when compared with non-air-entrained concrete. Gilkey (1958) found that as the amount of entrained air increases, the demand for mixing water and sand reduces at a particular cement content. However, when the cement content increases the reduction in the demand for mixing water decreases (Gilkey 1958).

Thus the reduction in compressive strength associated with air-entrained concrete can be somewhat compensated by making air-entrained concrete with lower water-cement ratios (Kosmatka et al. 2002). This is applicable to moderate strength concretes as mixes with high cement contents tend to have less reduction in the demand for mixing water (Gilkey 1958).

Cordon (1946) used three different cement contents and plotted the relationships between compressive strength and air content as shown in Figure 2.3. According to the data in Figure 2.3 (b), mixes with high cement contents undergo a greater loss of strength due to the increase in the amount of entrained air. From Figure 2.3 (b), it can also be concluded that there may be a gain in strength for lower cement content mixes as the amount of entrained air increases. The workability tends to enhance with an increase in the amount of entrained air and it is a positive point in favor of air entrainment (Kosmatka et al. 2002).

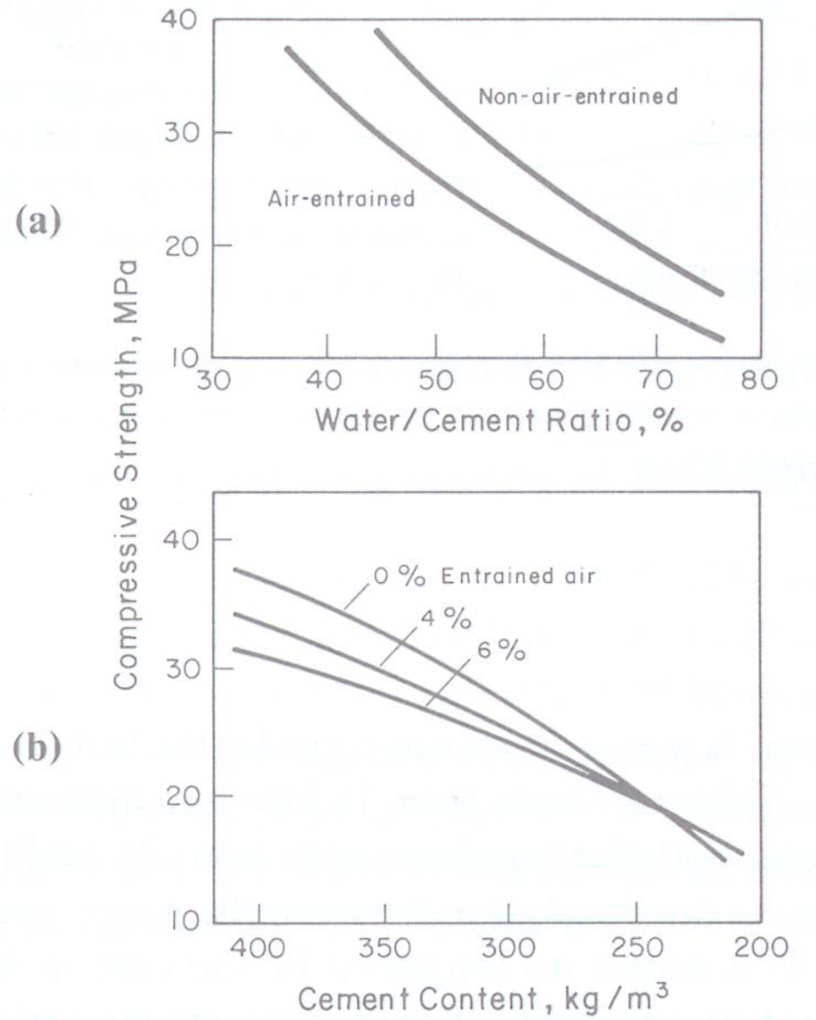

Figure 2.3: Cement content, air entrainment, w/c, and compressive strength (U.S.B.R. 1981 and Cordon 1979)

2.4.4 Curing Conditions

The reaction of water with cement is called the hydration process and the results are called the products of hydration. Curing is a process by which moisture loss is prevented at a particular temperature to enhance the hydration process of cement. The curing process not only increases strength and durability but also decreases the porosity of the concrete. To ensure that there is satisfactory development of strength during the hydration process it is necessary to prevent moisture loss (Kosmatka et al. 2002).

Neville and Brooks (1987) stated that “it must be stressed for a satisfactory development of strength it is not necessary for all the cement to hydrate and indeed this is rarely achieved in practice.” Burg (1996) observed that a higher initial curing temperature increases the rate of hydration process and early-age strength. However, high initial temperatures have been reported to produce concretes with reduced long-term strengths (Burg 1996). The curing temperature is very important with respect to concrete strength because it contributes towards the rate of hydration.

With proper curing the capillary pores get filled up with hydration products (Neville 1996) and this increases the impermeability and strength (Kosmatka et al. 2002). To maintain proper hydration during the initial stages of concrete stiffening, the internal relative humidity should be maintained at 80 percent (Kosmatka et al. 2002). Neville and Brooks (1987) explained the impermeable nature of adequately cured concrete by stating that the capillary pores inside concrete get interconnected by pores formed by the products of hydration after sufficient hydration has taken place.

2.4.5 Capping Method

A study done by Glover and Stallings (2000) at Auburn University found that compressive strengths from 4 x 8 in. cylinders with neoprene caps were 9.6 % greater than strengths from sulfur-capped 4 x 8 in. cylinders. It was also found that for 6 x 12 in. cylinders, compressive strengths from cylinders with neoprene caps were greater by 4.6 % than strengths from sulfur-capped cylinders. These values were obtained from cylinders cast at the Sherman Prestressed Concrete Plant tested at time of prestress transfer, 14 days, and 28 days; half of them were cured under a tarp with the members, the other half were from the match-cure box. Stallings and Glover also performed a study on cylinders cured with the Sure Cure™ system. When accounting for the difference between testing machines at Auburn University and Sherman, f_{c4} using neoprene pads was approximately 7 % higher than f_{c4} from sulfur-capped cylinders. This is the average value of specimens tested at time of prestress transfer, 28 days, and 56 days (Glover and Stallings 2000).

Pistilli and Willems (1993) found that cylinders capped with neoprene pads were stronger than those capped with sulfur for 6 x 12 in. cylinders over 8,000 psi and for 4 x 8 in. cylinders over 13,000 psi. There were no differences between the strengths of the two cylinder sizes when both were capped with sulfur. For cylinders tested with neoprene pads, there was no difference in strength between 4 x 8 in. and 6 x 12 in. cylinders in the range of 4,000 psi to 9,000 psi. However, 4 x 8 in. cylinders were stronger than 6 x 12 in. cylinders when in the range of 9,000 psi to 16,000 psi when both were tested with neoprene pads. 6 x 12 in. cylinders tested with sulfur caps and neoprene pads showed no difference up to strengths of 8,000 psi. 4 x 8 in. cylinders tested with sulfur caps and neoprene pads showed no difference up to strengths of 13,000 psi. Above these strengths, cylinders tested with neoprene pads had higher compressive strengths. Pisitlli and Willems (1993) also researched the effects of testing cylinders with ground end faces, but the conclusions pertain to the differences in within-test variation, not differences in compressive strength.

2.4.6 Testing Parameters

The compressive strength of concrete depends on two sets of testing parameters, i.e., specimen and loading. Specimen parameters include size, capping method, specimen shape, curing conditions and height-to-diameter ratio. Loading parameters include load rate and the different load conditions prevailing on site and in the laboratory (Mehta and Monteiro 1993).

2.4.8 Specimen Parameters

From Figure 2.4 it can be seen that as the height-to-diameter ratio increases the strength of the specimen decreases. These results will be applicable only when all the specimens are subjected to the same curing conditions because the curing conditions influence the strength of concrete (Mehta and Monteiro 1993).

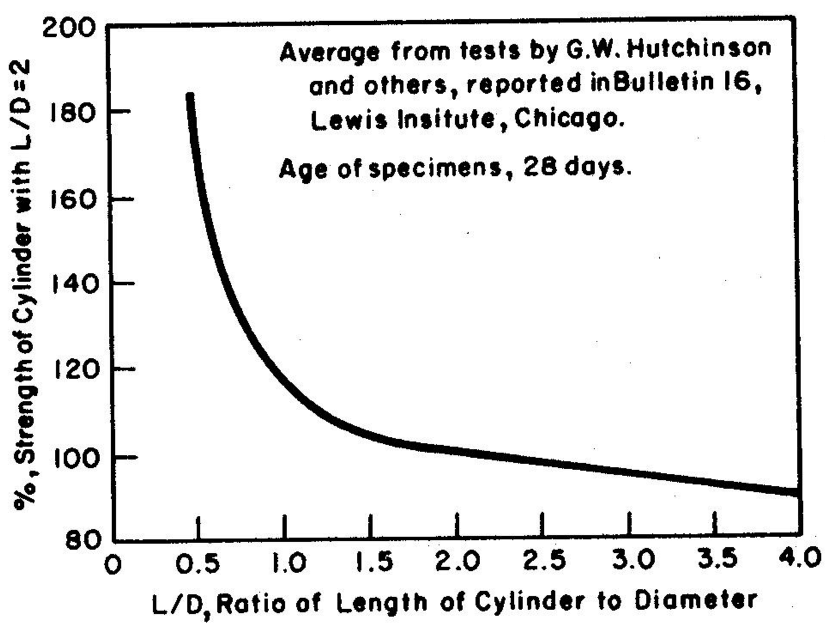

Figure 2.4: Length/Diameter ratios and compressive strength (U.S.B.R 1975)

2.4.8 Mold Material

Carrasquillo and Carrasquillo (1988) found that 6 x 12 in. cylinders made in plastic molds had a slightly lower compressive strength than those made in steel molds. They also found that 4 x 8 in. cylinders made in steel, plastic, and cardboard molds had equal compressive strengths.

2.4.9 Loading Conditions

Since the response of concrete to the applied load depends on the type of load, the compressive strengths measured under laboratory and field-testing conditions will differ (Mehta and Monteiro 1993). ASTM C 39-01 requires that the loading rate for cylindrical specimens be maintained between 20 and 50 psi/sec. Generally, the higher the rate of loading, the higher the apparent compressive strength.

2.4.10 Age

The relationship between strength and porosity is an indicator to extent which the hydration process is completed and the amount of hydration products present. Different cements require different lengths of time to achieve a particular strength and the rate of hydration is different for different types of cement (Neville 1996).

The water-cement ratio influences the rate of the hydration process and consequently the rate of strength gain. Meyer (1963) found that when low water-cement ratios are considered there is a rapid gain in early strength as compared to higher water-cement ratios. He also found that the rate of strength gain at lower water-cement ratio decreased at later ages as compared to higher water-cement ratios. Meyer (1963) also showed that the strength of concrete increases with an increase in the age of concrete.

2.5 FACTORS THAT AFFECT THE STRENGTH RATIO, k_s

The standard size specimen used for strength acceptance testing is a 6 x 12 in. cylinder. ASTM and AASHTO both allow the use of 4 x 8 in. cylinders. However, these specimens are not often used because of the uncertainty of how their strength compares to the strength of 6 x 12 in. cylinders made from the same batch of concrete. This section will review studies that have been done to correlate the strengths between a standard cylinder size and one that is smaller. Table 2.1, taken from Day (1994 a), summarizes the k_s values and ranges found from the research of others. It should be noted that there are many relationships given. Some found a specific value for k_s where others found a range. Values of k_s were given as less than 1.0, equal to 1.0, and greater than 1.0.

Table 2.1: Strength ratios from previous research given by Day (1994 a)

Reference	Relationship	Strength Range (psi)
Aitcin et al. (1992)	$f_{c4} = 1.16f_{c6}$ -1230	11,600 to 14,500
Carrasquillo and Carrasquillo (1988)	$f_{c4} = 0.93\ f_{c6}$	7,250 to 11,600
Date and Schnormeier(1984)	$f_{c4} = 1.04\ f_{c6}$	< 5,080
Day and Haque(1993)	$f_4 = f_{c6}$	< 7,250
Day (1994 b)	$f_{c4} = f_{c6}$	4,350 to 7,250
Forstie and Schnormeier (1981)	$f_{c4} = f_{c6}$	5,000 to 7,250
Forstie and Schnormeier(1981)	$f_{c4} > f_{c6}$	< 5,000
Gonnerman (1925)	$f_{c4} = 1.01\ f_{c6}$	< 4,640
Lessard and Aitcin (1992)	$f_{c4} = 1.05\ f_{c6}$	5,080 to 17,400
Malhotra (1976)	$f_{c4} = (0.85 \text{ to } 1.05)\ f_{c6}$	< 7,250
Cook (1989)	$f_{c4} = 1.05\ f_{c6}$	< 13,050
Peterman and Carrasquillo (1983)	$f_{c4} = (1.10 \text{ to } 1.15)\ f_{c6}$	7,250 to 11,600
Janak (1985)	$f_{c4} = 1.03\ f_{c6}$	< 8,120
Chojnacki and Read (1990)	$f_{c4} = (1.02 \text{ to } 1.04)\ f_{c6}$	8,410 to 14,070
Pistilli and Willems(1993)	$f_{c4} = f_{c6}$ (sulfur caps)	3,920 to 15,090
Pistilli and Willems(1993)	$f_{c4} = f_{c6}$ (polymer pads)	4,060 to 8,990
Carrasquillo et al. (1981)	$f_{c4} = 0.90\ f_{c6}$	4,350 to 11,600

2.5.1 Statistical Comparison of the Effect of Test Cylinder Size

There is discussion as to whether the results and variability obtained from 4 x 8 in. and 6 x 12 in. cylinders are statistically equivalent. ASTM C31 (2000) states that "when cylinders smaller than the standard size are used, within-test variability has been shown to be higher but not to a statistically significant degree." Some experimental studies have found that the standard deviation of the compressive strength increases with a decrease in cylinder diameter (Malhotra 1977). It has been reported that equal variability can be obtained when the number of cylinders tested is such that the summation of the cross-sectional areas of the cylinders of the two sizes are equal (Malhotra 1973). In other word, based on Malhotra's recommendation, 2.25 more 4 x 8 in. cylinders will have to be tested in order to obtain comparable variability to that obtained by testing the 6 x 12 in. cylinders. Day claims there is no reason to test more 4 x 8 in. cylinders than 6 x 12 in. cylinders (Day 1994 a). After his comprehensive statistical analysis of over 8,000 test results, Day (1994 a) states that "the coefficient of variation for 4 x 8 in. cylinders is equivalent to that of 6 x 12 in. cylinders over a broad range that encompasses normal, high, and very high-strength concrete." However, Nassar (1987) and Issa et al. (2000) found that the standard deviation increases as the diameter of the cylinder decreases. Figure 2.5 is a scatterplot Day produced after compiling data from all 22 studies. It is clear that as strength increases the deviation from the line of equity increases in favor of the 4 x 8 in. cylinders. "There is strong evidence that if one uses 4 x 8 in. cylinder plastic or steel molds, the strength obtained in the 2,900 to 14,500 psi range is expected to be 5% greater than that obtained using 6 x 12 in. cylinder molds. In the lower strength ranges, 2,900 to 8,700 psi, for example, it may be acceptable to assume from a practical perspective that strengths using 4 x 8 in. and 6 x 12 in. molds are equivalent; justification for such an assumption must be determined by standards authorities" (Day 1994 a). However, some believe that the magnitude of difference in standard deviations is great enough to require twice the number of 4 x 8 in. as 6 x 12 in. cylinders to keep an equal degree of precision (Malhotra 1976).

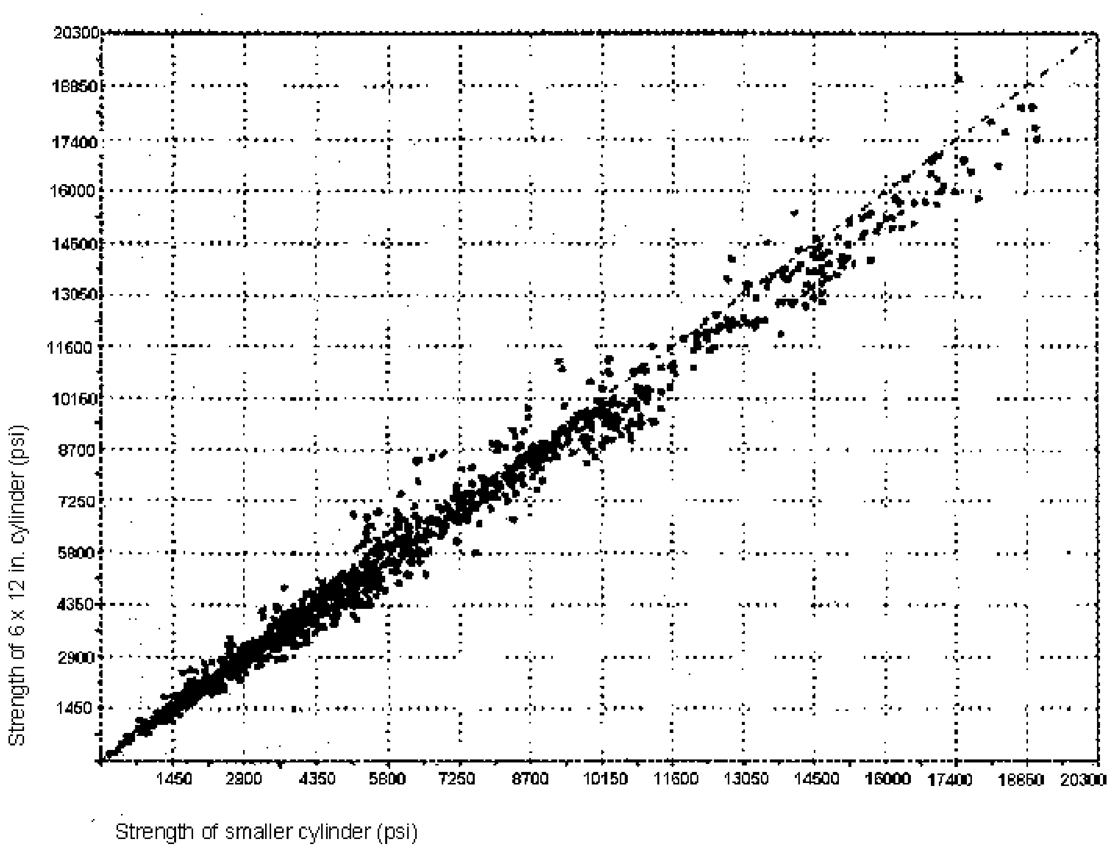

Figure 2.5: Scatterplot of f_{c6} versus f_{c4} from Day (1994 a)

There have been many studies done to find a correlation between the strength of 6 x 12 in. cylinders and smaller sized cylinders such as 4 x 8 in. or 3 x 6 in. Factors such as curing condition, compaction, capping method, and admixture content have been varied during these studies.

However, based on the results of past research efforts it seems that the main contributors to the influence the strength ratio are cylinder size and strength level. Day states that “factors such as concrete type, aggregate type, cement content, water-cement ratio, presence of supplementary cementing materials, and type of vibration appear to have no significant effect on the correlation between f_{c6} and the strength from small cylinders. On the other hand, the above factors all influence the strength of the concrete, and the level of concrete strength does appear to have an effect on differences in measured strengths from different cylinder sizes” (Day 1994 a).

There are several variables that have been introduced into experiments that did affect the strength ratio such as number of roddings per layer, number of layers per specimen, and type of curing. However, the manipulation of these variables will violate both ASTM and AASHTO standards. When 6 x 12 in. cylinders compacted with two equal layers and 25 roddings per layer are compared to 3 x 6 in. cylinders with two equal layers and decreasing number of roddings per layer, the strength ratio f_{c3}/f_{c6} decreases with decreasing number of roddings per layer for the 3 x 6 in. cylinder (Nassar and Al-Manaseer 1987). In the study done by Forstie and Schnormeier (1981) it was shown that “for 28 day strengths above 5,000 psi, the 4 x 8 in. cylinders have significantly higher strength than the same concrete tested in 6 x 12 in. cylinders. In the 7,000 psi range, the 4 x 8 in. cylinders will break about 1,000 psi higher than the corresponding 6 x 12 in. cylinder. As shown in Figure 2.6 for 28 day strengths of around 3,000 psi, both sizes of cylinders give essentially similar results” (Forstie and Schnormeier 1981).

2.5.2 Correlations at Low Strengths

In a study done by Forstie and Schnormeier (1981) 1,152 4 x 8 in. and 6 x 12 in. cylinders were tested. It was found that the general assumption of f_{c4} being greater than f_{c6} was true, but only for a certain range of strength. There was a reversal point at about 3,000 psi where f_{c6} was greater than f_{c4}. Malhotra (1976) also suggested this type of behavior occurred. “The compressive strengths of 4 x 8 in. cylinders are higher than those of 6 x 12 in. cylinders. There are, however, indications that at low strength levels the reversal may be true” (Malhotra 1976). He did not attempt to explain this phenomenon. Neville (1996) states that “the measured strength of any specimen decreases with an increase in size. Any contradiction for the smaller specimens is attributed to the ‘wall effect’. The quantity of mortar required to fill the space between the particles of the coarse aggregate and the wall of the mold is greater than that necessary in the interior of the mass and therefore in excess of the mortar available even in well-proportioned mixtures” (Neville 1996).

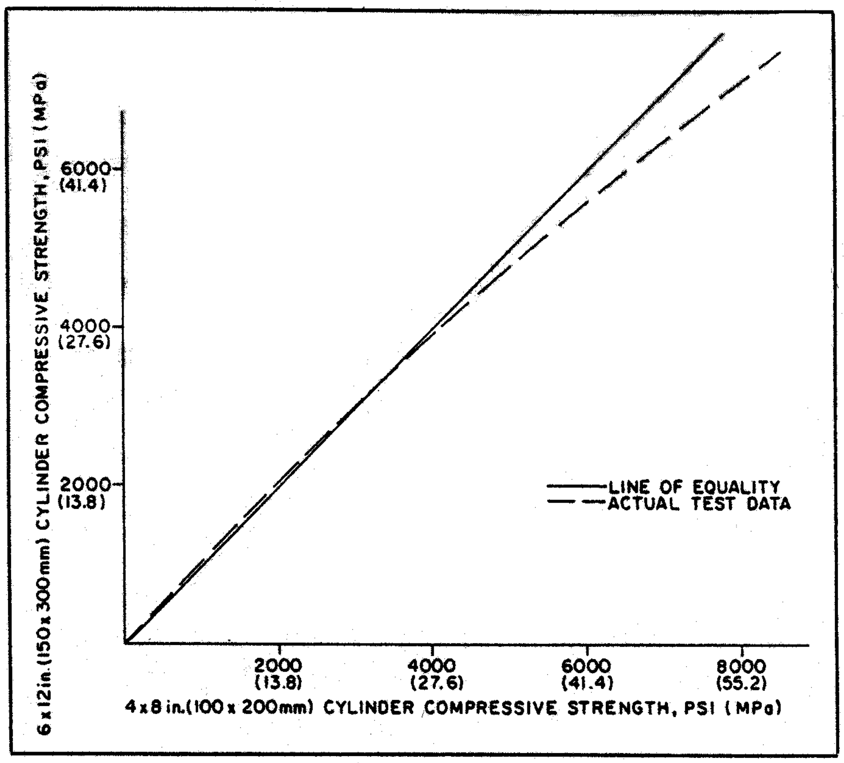

Figure 2.6: Relationship of f_{c6} versus f_{c4} from Forstie and Schnormeier (1981)

2.5.3 Age at Testing

Date and Schnormeier (1984) claim that "at any stage of curing, there is no significant difference between the strengths obtained from these cylinders." However, the results shown by Day and Haque (1993) in Table 2.2, by Aitcin et al. (1994) in Table 2.3, and by Issa et al. (2000) in Table 2.4 show that age does affect k_S.

2.5.4 Curing Conditions

It has been shown that variation from standard methods of curing conditions can affect the compressive strength of cylindrical concrete specimens. This is expected as humidities less than 100% will cause moisture loss from the cylinders and the rate of moisture loss will be different for cylinders of different size. Day and Haque (1993) performed an experiment to determine a correlation between the compressive strengths of small and standard size cylinders. In their experiment there were two cylinder sizes: 3 x 6 in. cylinders and 6 x 12 in. cylinders, and three different target strengths: 2,900 psi, 4,350 psi, and 5,800 psi. There were two types of fly ash used, and the amount of each fly ash was varied at 20, 35, and 50 percent by mass of cementitious materials. Two different types of curing methods were used. M-cured specimens were subjected to curing in a fog room at 95% ± 3% relative humidity at 68°F ± 4°F until testing. MR-cured specimens were M-cured for three days then subjected to outdoor conditions where temperatures ranged between –24°F and 73°F. All specimens were then soaked in water for two hours prior to strength testing. The following correlations in Table 2.2 are based on 8- and 29-day strength tests:

Table 2.2: Ratio of f_{c3} to f_{c6} from Day and Haque (1993).

		Age			
		8 days		29 days	
	Cure	M	MR	M	MR
Target 28 day Strength	2,900 psi	1.12	1.06	1.03	1.08
	4,350 psi	1.05	1.03	1.05	1.09
	5,800 psi	1.05	1.06	1.01	1.04

Each of these correlations was computed from data given that covers both classes of fly ash at all three percentages for one target strength and a particular testing age. Generally, the MR-cured cylinders showed a higher strength ratio than the M-cured cylinders at 29 days, but showed a lower strength ratio than the M-cured cylinders at 8 days. The strength ratio tended to decrease with increasing target strength for M-cured specimens. From the results shown in Table 2.2 and Figure 2.7, it appears that k_s may be affected by curing conditions.

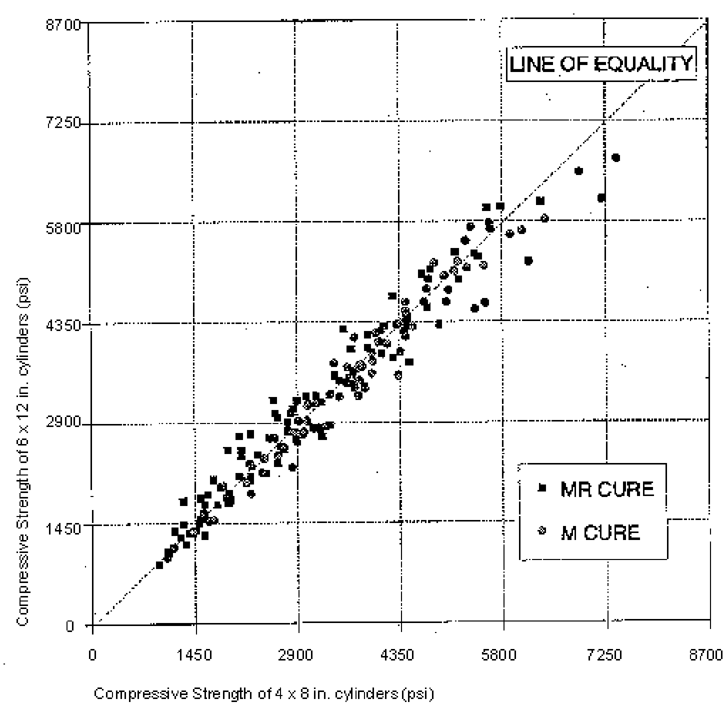

Figure 2.7: Scatter plot of M-cured and MR-cured specimens from Day (1994 b)

Aitcin et al. (1994) also studied the effect of curing conditions on varying cylinder sizes. Three different cylinder sizes were used: 4 x 8 in. cylinders, 6 x 12 in. cylinders, and 8 x 16 in. cylinders. Three different strengths were evaluated: 5,000 psi, 13,000 psi, and 17,500 psi. Air, water, and sealed curing were the three types of curing studied. However, the 4 x 8 in. cylinders were subjected to the three different curing conditions. 6 x 12 in. and 8 x 16 in. cylinders were only air cured (except for 1 day old specimens, where all specimens were cured in their mold). Their results are summarized in Table 2.3 and are based on strength ratios for the compressive strengths of air, water, and seal-cured 4 x 8 in. cylinder strengths to the air-cured 6 x 12 in. cylinder strengths at 7 and 28 days for all three strength levels. Each ratio is the mean of three 4 x 8 in. cylinder strengths to the mean of three 6 x 12 in. cylinder strengths.

Table 2.3: Ratios of f_{c4} to f_{c6} from Aitcin et al. (1994).

Strength Ratio	Strength Range					
	5,000 psi		13,000 psi		17,500 psi	
	7 days	28 days	7 days	28 days	7 days	28 days
f_{c4air} / f_{c6air}	1.06	1.05	0.99	1.02	0.99	0.94
$f_{c4sealed}$ / f_{c6air}	0.99	1.02	1.05	1.1	1.02	0.96
$f_{c4water}$ / f_{c6air}	1.03	1.09	1.07	1.19	0.98	1.05

Even though standard moist room curing will be the method of curing for this project, it is interesting to note how the effect of different methods of curing for each size cylinder affects the strength ratio. The biggest difference was found when the 4 x 8 in. cylinders are cured in water and the 6 x 12 in. cylinders are cured in air. This is an expected result, due to the fact that water curing allows the least amount of moisture loss from a specimen whereas air curing allows the most amount of moisture loss. These results are not extremely valuable to the current project since the correlation between 4 x 8 in. and 6 x 12 in. cylinders both cured in a moist room was not studied. In this research project, the effect of different curing conditions will not be evaluated as these conditions are controlled by the procedures outlined in AASHTO and ASTM standards.

2.5.7 Effect of Aggregate Size

A study was done by Issa et al. (2000) to determine the effect of aggregate size along with specimen size on the compressive strength of concrete. Four different sizes of cylinders were evaluated: 6 x 12 in., 4 x 8 in., 3 x 6 in., and 2 x 4 in. The nominal maximum aggregate size was varied between sizes of No. 4, 0.375 in., 0.75 in., 1.5 in., and 3 in. The 6 x 12 in. cylinders were not made with No. 4 aggregate. Issa et al. found that the coefficient of variation of compressive strength increased as the nominal maximum size aggregate increased. It was also found that for different size cylinders with the same maximum nominal aggregate size, the coefficient of variation increased as the cylinder size decreased. These results were concluded for concretes having a 28 day compressive strength between 5,000 psi and 7,000 psi, with coefficients of variation ranging between 1.75% to 5.2% for 6 x 12 in. cylinders and 3.1% to 6.1% for 4 x 8 in. cylinders. The 6 x 12 in. cylinders made with a 0.75-in. maximum aggregate size had an average 28 day compressive strength of 6497 psi and a coefficient of variation of 3.2%. The 4 x 8 in. cylinders made with a 0.75-in. maximum aggregate size had an average 28-day compressive strength of 6,597 psi and a coefficient of variation of 4.8%. Table 2.4 shows the changes in correlation as nominal maximum aggregate size varies.

Table 2.4: Ratios of f_{c4} to f_{c6} from Issa et al. (2000).

Max. Agg. Size (in.)	Age	
	7 days	28 days
0.375	1.05	1.06
0.75	1.08	1.02
1.5	1.06	1.04
3	0.98	0.97

From the data collected by Issa et al. (2000), it can be seen that the strength ratio, based on 7-day and 28-day strengths, generally was found to decrease with increasing maximum nominal aggregate size for 4 x 8 in. and 6 x 12 in. cylinders made from the same batch.

2.5.6 Mold Material

Day (1994 a) performed a study to determine the effect on concrete compressive strength when the type of mold and its size were varied. He used 6 x 12 in. plastic and cardboard molds, 4 x 8 in. plastic molds, and 3 x 6 in. plastic and cardboard molds. It should be noted that it is not common practice to use cardboard molds in the Alabama concrete industry. It can be seen from the box plot graph shown in Figure 2.8 that no matter what mold material or strength range was used, the strength ratio k_s is approximately 1.05 based on the mean line on each individual box plot.

Even though the mean lines of the box plots are approximately 1.05, there is a wide range of values for each category. Day (1994 a) explains that for the box and whisker plots shown in Figure 2.8, only 3.1% of the data are outliers, which are represented as lines or dots extending from the box. This means that the range of the box represents 96.9% of all data points. It can been that several of the boxes range from a k_s value of 1.0 to 1.10 with outliers having ranges approximately between 0.85 and 1.25. The legend of Figure 2.8 is difficult to read. In each strength range, the mold material from left to right is plastic, steel, and tin. Plastic cylinder molds are most commonly used in the Alabama concrete industry. For concrete cylinders made with plastic molds in the strength range of

2,900 psi to 8,700 psi, k_s ranges between 0.93 and 1.16. The results of the research done by Carrasquillo and Carrasquillo (1988) found that when considering cylinders made from plastic, steel, and cardboard molds, k_s was equal to 0.93 for strengths ranging between 7,000 psi and 12,000 psi.

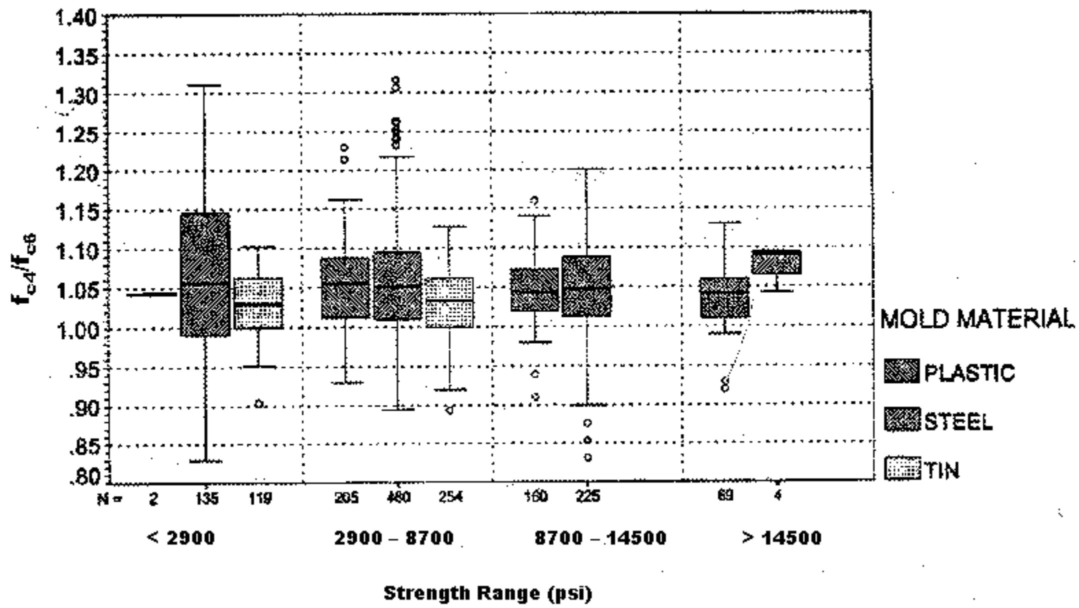

Figure 2.8: Dependence of k_s on Strength Range and Mold Material from Day (1994 a)

2.6 VARIABILITY ASSOCIATED WITH CONCRETE COMPRESSIVE STRENGTH TESTING

When dealing with concrete quality control and assurance, controlling the variability of concrete is vitally important. Strict guidelines and specifications have been developed over the years to determine the required average strength of concrete based on the design strength needed, and accounts for the inherit variability associated with the concrete that is produced. These requirements are based on the 28-day strength of standard cured 6 x 12 in. cylinder specimens. Also, these requirements were initially developed for all strength concretes. Recent research has shown that high-strength concrete is more variable than low- and normal-strength concrete (Cook 1989). This has led to the modification of certain specifications to account for this new knowledge. Because 4 x 8 in. cylinders come from the need to test high-strength concrete, the variability of 4 x 8 in. cylinders should be compared to that of 6 x 12 in. cylinders.

2.6.1 Variability of High-Strength Concrete

Table 3.5 of ACI 214-77 gives limit values for standard deviation and coefficient of variation to determine whether the control was excellent, very good, good, fair, or poor. For general construction testing, a standard deviation below 400 psi is considered excellent and above 700 psi is considered poor. Cook (1989) suggests “ACI 214 may not be a fair evaluation for the higher strength concretes.” Neville (1996) agrees with Cook (1989) by saying “the recommendations of ACI 214-77 are based on concretes used up to the mid-1970’s, and such concretes did not often have a cylinder strength in excess of 35 MPa (5,000 psi). It is, therefore, questionable whether the approach of ACI 214-77 necessarily applies to high strength concrete with a 28-day compressive strength in excess of 80 MPa (12,000 psi), let alone in the region of 120 MPa (17,000 psi).” It should be known that ACI 214-77 was reapproved in 1989 and again in 1997.

ACI 363.2R-98 states that “in the case of high-strength concrete, defining quality control categories based on absolute dispersion may be misleading, since some standard deviations greater than 700 psi are not uncommon for 10,000 psi concrete on well controlled projects.” Table 3.5 of ACI 214-77 gives standards of quality control in terms of standard deviation for overall variation and in terms of coefficient of variation for within-test variation. Table 5.1.1 of ACI 363.2R-98, a modification of Table 3.5 of ACI 214-77, gives standards of quality control in terms of coefficient of variation for both overall variation and within-test variation. This is due to research by Cook (1989) and Anderson (1985) suggesting that the coefficient of variation is a better estimate of variability. Cook (1989) claims that “the coefficient of variation is a unitless standard deviation expressed as a percentage of the average strength. This value will be less affected by the magnitude of the strengths obtained and is more useful in comparing the degree of control between higher strength concretes and lower strength concretes.”

Table 2.5: Standards of concrete control (Table 3.5 from ACI 214-77).

Overall variation					
Class of operation	Standard deviation for different control standards, psi				
	Excellent	very good	good	fair	poor
General construction testing	< 400	400 to 500	500 to 600	600 to 700	> 700
Laboratory trial batches	< 200	200 to 250	250 to 300	300 to 350	> 350
Within-test variation					
Class of operation	Coefficient of variation for different control standards, %				
	Excellent	very good	good	fair	poor
Field control testing	< 3	3 to 4	4 to 5	5 to 6	> 6
Laboratory trial batches	< 2	2 to 3	3 to 4	4 to 5	> 5

Table 2.6: Standards of concrete control (Table 6 from Cook 1989).

Overall variation					
Class of operation	Coefficient of variation for different control standards, %				
	Excellent	very good	good	fair	poor
General construction testing	< 8	8 to 10	10 to 12	12 to 15	> 15
Laboratory trial batches	< 4	4 to 6	6 to 8	8 to 10	> 10
Within-test variation					
Class of operation	Coefficient of variation for different control standards, %				
	Excellent	very good	good	fair	poor
Field control testing	< 3	3 to 4	4 to 5	5 to 6	> 6
Laboratory trial batches	< 2	2 to 3	3 to 4	4 to 5	> 5

Table 2.7: Standards of concrete control (Table 5.1.1 from ACI 363.2R-98).

Overall variation					
Class of operation	Coefficient of variation for different control standards, %				
	excellent	very good	good	fair	poor
General construction testing	< 7	7 to 9	9 to 11	11 to 14	> 14
Laboratory trial batches	< 3.5	3.5 to 4.5	4.5 to 5.5	5.5 to 7	> 7
Within-test variation					
Class of operation	Coefficient of variation for different control standards, %				
	excellent	very good	good	fair	poor
Field control testing	< 3	3 to 4	4 to 5	5 to 6	> 6
Laboratory trial batches	< 2	2 to 3	3 to 4	4 to 5	> 5

ACI Committee 318 produces the Building Code Requirements for Structural Concrete and Commentary. ACI 318-02 differs from ACI 318-99 in Section 5.3.2. Section 5.3.2.1 in ACI 318-02 separates the required average compressive strength of concrete when a standard deviation can be established into two categories: one for concretes with a specified strength less than or equal to 5000 psi, and one for concretes with a required strength greater than 5000 psi. This division is not given in ACI 318-99. Equations 5-1 and 5-2 remain the same for both codes; however, there is an Equation 5-3 for the newest code. Equation 5-3 from ACI 318-02 is

$$f'_{cr} = 0.9 f'_c + 2.33s \qquad \text{Equation 2.3}$$

where, f'_{cr} = required average compressive strength

f'_c = required compressive strength

s = standard deviation

In both the 1999 and 2002 codes, Section 5.3.2.2 gives f'_{cr} when a standard deviation cannot be established. It has three categories: one for concretes with f'_c less than 3000 psi, for when f'_c is between 3000 and 5000 psi, and for when f'_c is greater than 5000 psi. The change occurs in the third category with the expression becoming:

$$f'_{cr} = 1.1 f'_c + 700 \qquad \text{Equation 2.4}$$

These two changes have been made to accommodate the increase in variability for high-strength concrete. These modifications to the code were suggested by Cook (1989). Cook gives two equations, one of which is Equation 5-1 in ACI-318 and another which ends up being Equation 5-3 in ACI 318-02. He describes the latter as "a modified version of Equation 4-2 of ACI 318-83 since the code was established on the basis of concrete strengths in the range of 3000 to 6000 psi." It is generally accepted that as the strength of concrete increases, the variability of test results will increase as well. Hester (1980) reports that differences between compressive strengths of concrete from laboratories using the same mix design can reach 10%. Cole (1966) reports that the coefficient of variation of tests performed on similar concrete can be as low as 3% for one testing laboratory but as high as 9% for another laboratory. He contributes this difference in results mainly to improper machine calibration. Cole also reports that differences in reported strengths of the same batch of concrete can reach as high as 18%. It should be noted that the shape of specimens studied by Cole was a cube. Kennedy et al. (1995) report that within-laboratory and between-laboratory standard deviations increased as the average compressive strength increased as shown in Figure 2.9.

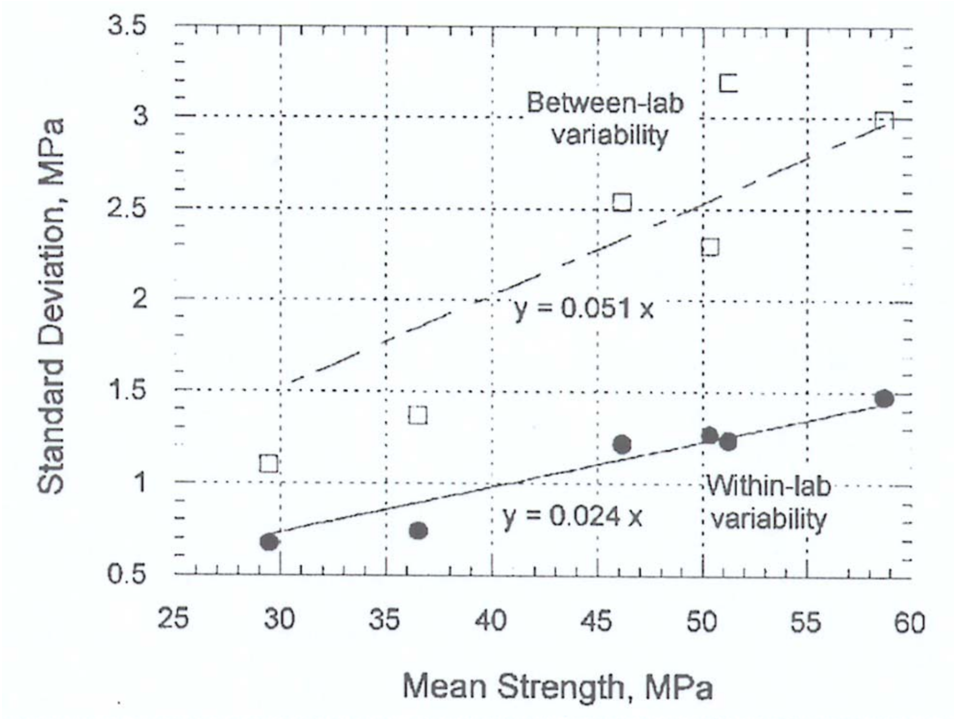

Figure 2.9: Between-lab and within-lab variability from Kennedy et al. (1995) (1 MPa = 145 psi)

Cook (1989) investigated the variability of high-strength concrete and presented his results in a paper titled “10,000 psi Concrete” which considered over 4,000 test specimens. Cook discusses the trial mixing done starting in 1981 for the InterFirst Plaza in Dallas, TX (now Bank of America Plaza) which started construction in 1983. Of the total 84,700 yd^3 of concrete used to construct the building, 20,560 yd^3 of concrete had a design strength of 10,000 psi and 1,800 yd^3 had a design strength of 8,000 psi (Cook 1989). This is an enormous amount of concrete; quality control for a project of this scale required great attention to detail and planning. This is the reason why trial mixing for this project was started two years before actual construction began. Table 2.8 shows the comparison of the test results from trial batching at a commercial laboratory and a producer’s laboratory. It can probably be assumed that with the nature of the project in mind, both laboratories were practicing testing procedures with as much attention to detail as possible. At the time of the testing, these results would be judged by ACI 214-77 Table 3.5 and would be categorized as fair to poor since the standard deviation is greater than 700 psi. The results would also violate the maximum coefficient of variation of 2.37% given in ASTM C 39 (1996). Therefore it could be concluded that the cause of the high variability was the fact that the concrete produced was of very high strength.

Cook (1989) states that “a 10,000 psi designed concrete with a standard deviation of 800 psi has the same degree of control as 3,000 psi designed concrete with a standard deviation of 240 psi” based on the two concretes having the same coefficient of variation. It should also be noted that the coefficient of variation decreases as the age of specimens increased.

Table 2.8: Summary of statistics from Cook (1989).

	Commercial Laboratory				Producer Laboratory			
Age (days)	n	Avg. f_c (psi)	Std-Dev (psi)	C.O.V.	n	Avg. f_c (psi)	Std-Dev (psi)	C.O.V.
3	386	7,373	980	13.3	112	7,463	689	9.2
7	421	9,059	721	8.0	139	9,063	596	6.6
28	419	11,149	855	7.7	139	11,192	678	6.1
56	411	12,068	850	7.0	139	12,082	682	5.6
180	377	13,397	791	5.9	138	13,462	724	5.4

2.6.2 Effects of Cylinder End Condition on Within-Test Variation

ASTM states that when cylinders smaller than the standard size are used, within-test variability has been shown to be higher but not to a statistically significant degree (ASTM C 31 2000). Pistilli and Willems (1993) conducted extensive research concerning the effects of cylinder size and cylinder end conditions on within-test variability. The average range and standard deviation of many groups of two cylinders were the values that defined the variability in their study. The variability of f_{c4} and f_{c6} were studied for sulfur caps in the range of 2,000 psi to 15,000 psi and for polymer pads in the range of 2,000 psi to 19,000 psi. Their results showed that cylinder size did not affect within-test variability. Based on a 95% confidence level, they show that the variations for 4 x 8 in. and 6 x 12 in. cylinders are the same when capped with sulfur and in the range of 2,000 psi and 15,000 psi. For strengths greater than 6,000 psi, the end condition of cylinder was shown to have a great effect on within-test variability. Pistilli and Willems (1993) claim that "sulfur caps appeared inadequate for accurate compressive strength measurement for strengths above 13,000 psi."

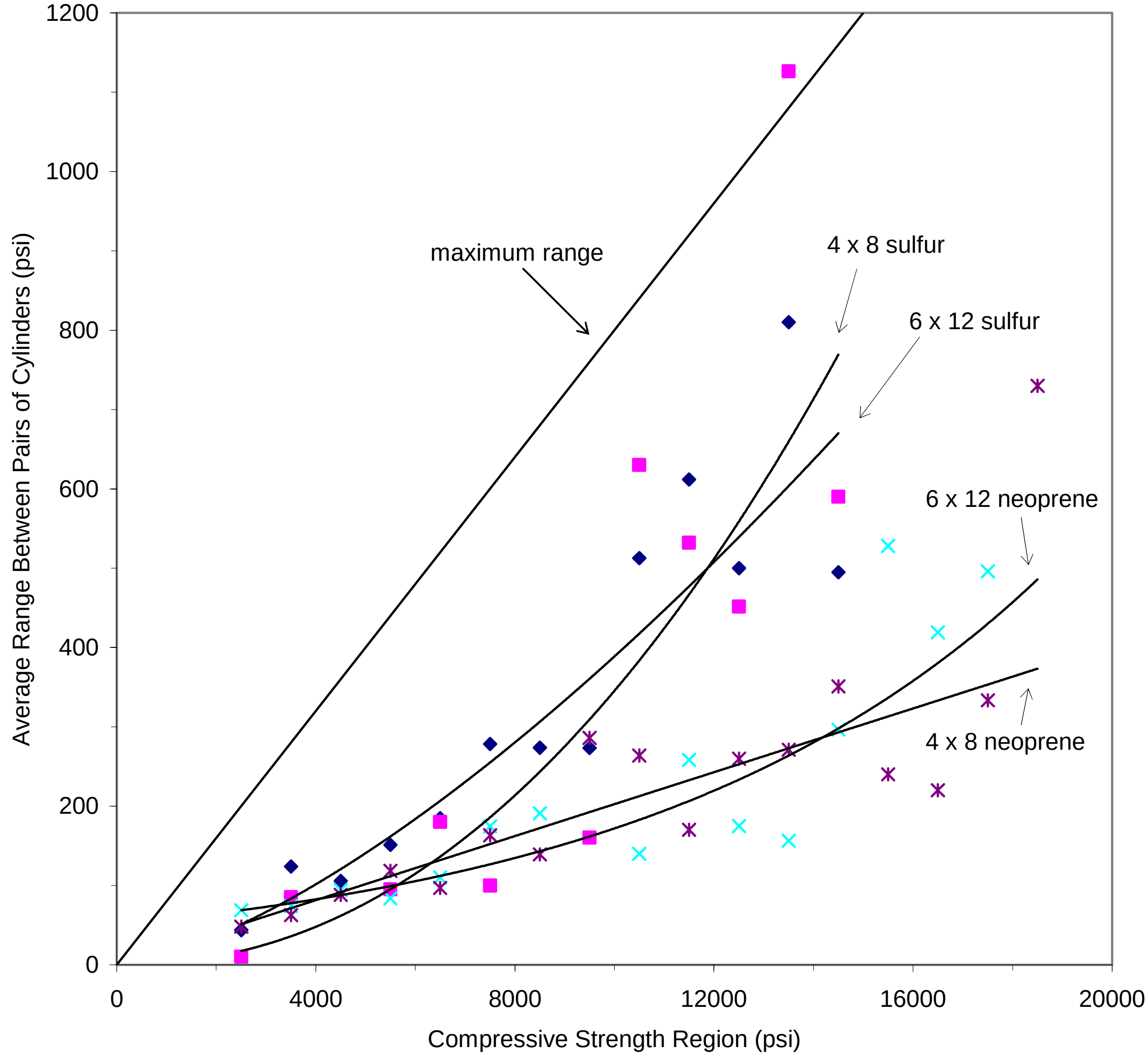

Figure 2.10: Within-test range for pairs of cylinders using sulfur and neoprene pads from Pistilli and Willems (1993)

Figure 2.10 is a replication of Figure 2 from Pistilli and Willems (1993) with a line plotting the maximum acceptable range between two cylinders based on the ASTM C39 (2001) which is 8%. The line was calculated by multiplying compressive strength by this 8%. Even though Pistilli and Willems (1993) claim that using sulfur caps for test specimens with compressive strengths over 13,000 psi is inadequate, it can be seen that the average ranges for each strength region are well below the maximum acceptable range. This might lead one to assume that using sulfur caps and neoprene pads are adequate at all strengths.

Carrasquillo and Carrasquillo (1988) found that within-test variation of cylinders tested with unbonded caps was less than that of cylinders tested with high-strength mortar caps for concretes with strengths between 6,000 psi and 17,000 psi.

2.7 Conclusions

In the literature review have discussed factors affecting the compressive strength as well as the factors affecting the correlation between the strengths of 4 x 8 in. and 6 x 12 in. cylinders. Since the smaller cylinder size provides ease in transportation and construction, it is gaining popularity and is widely used. There is a need to investigate if a correlation between the strengths of the two cylinder sizes can be established. Table 2.9 shows which factors, based on the literature review, affect the compressive strength of concrete, and which of those factors appear to also affect the ratio of f_{c4} and f_{c6}.

Table 2.9: Strength factors vs. Correlation factors

Factors affecting concrete compressive strength	**Factors affecting the correlation between f_{c4} and f_{c6}**
strength level	strength level
age of specimen	age of specimen
aggregate size/gradation	aggregate size/gradation
specimen size/shape	
capping method	capping method*
mold material	mold material*
consolidation method	consolidation* method
curing conditions	curing conditions*
water/cementitous ratio	
air content	
mix proportions	
admixtures	
cement type	
loading conditions	

Note: * indicates that factor is set standard by specifications

The factors that affect the compressive strength as well as the strength ratio are aggregate size, strength level, and age of specimen. Using different types of mold material varied the strength ratio, but not to a significant amount. Age of specimen, strength level, and aggregate size were the three factors that were shown to have the greatest affect on the strength ratio. There are factors such as compaction and curing conditions that can be varied that will affect the strength ratio. However, varying these factors will violate AASHTO and ASTM standards. It was found from Day's study of over 8,000 compiled specimen strengths that within the strength range of 2,900 psi and 14,500 psi, f_{c4} is expected to be 5% higher than f_{c6}. However, in the lower strength range of 2,900 psi to 8,700 psi, f_{c4} and f_{c6} can be assumed equal. Day also found that there is no need to test more 4 x 8 in. cylinders than 6 x 12 in. cylinders due to the fact that "the coefficient of 4 x 8 in. cylinders is equivalent to that of 6 x 12 in. cylinders over a broad range that encompasses normal, high, and very high-strength concrete" (Day 1994 a). There have been studies done that revealed that f_{c6} was larger than f_{c4}. Carrasquillo and Carrasquillo (1988) found that k_s was equal to 0.93. Also, Forstie and Schnormeier (1981) and Malhotra (1976) claim that at low strength ranges f_{c6} could possibly be higher than f_{c4}.

It has been shown by Cook (1989) that standard deviations and coefficients of variation of high-strength concrete test results can reach quantities much higher than that of normal or low strength concretes. He also showed that high-strength concretes can have the same degree of control as low-strength concrete, as long as coefficient of variation is the standard of control, not standard deviation. Based on the research and suggestions by Cook (1989), ACI 318-02 has modified its requirements for f'_{cr} and ACI 363.2R-98 has modified Table 3.5 of ACI 214-77 to account for the increased variability of high-strength concrete.

Chapter 3

LABORATORY TESTING PROGRAM

3.1 GENERAL

The objective of this research was to develop a correlation between the compressive strengths of 4 x 8 in. cylinders and 6 x 12 in. cylinders made from the same batch of concrete. Based on the literature review, it was determined that the main variables that could potentially affect the strength ratio were strength range, aggregate type, and testing age. Therefore, these were the factors selected to be varied within the experimental phase of this research. 7- and 28-day strengths were selected due to the standard age at testing for quality assurance and quality control testing. No.57 and No.67 gradations were selected because they are the most commonly used. Three target strength ranges were selected: 4,000 psi, 6,000 psi, and 8,000 psi.

Table 3.1: Cylinder quantities for the experimental program

28-Day Compressive Strength	Coarse Agg. Size	4 x 8 in. Cylinders		6 x 12 in. Cylinders	
		7 Day	28 Day	7 Day	28 Day
4,000 psi	No. 57	30	30	30	30
	No. 67	30	30	30	30
6,000 psi	No. 57	30	30	30	30
	No. 67	30	30	30	30
8,000 psi	No. 57	30	30	30	30
	No. 67	30	30	30	30

3.2 MIXTURE AND BATCH DESIGNS

A batch size of 6.5 ft^3 was established based on the practical mixing capacity of a 12 ft^3 concrete mixer. This would produce twenty 4 x 8 in. cylinders, twenty 6 x 12 in. cylinders, 0.25 ft^3 for the pressuremeter test, and 20% extra volume for waste. Each row in Table 3.1 represents one of the six mixture designs. Each mixture design was batched and mixed three times. Therefore, there were 18 total batches. Each batch produced ten 4 x 8 in. cylinders for 7-day strengths tests, ten 4 x 8 in. cylinders for 28-day strength tests, ten 6 x 12 in. cylinders for 7-day strength tests, and ten 6 x 12 in. cylinders for 28-day strength tests. Since there were two technicians conducting this research, each technician made an equal number of 4 x 8 in. and 6 x 12 in. cylinders during cylinder construction. Also, to maintain consistency, each technician handled only the cylinders made by that technician throughout the entire process from cylinder construction until testing.

Table 3.2: Concrete mixture proportions

Property	Strength Range (psi)					
	4,000	**4,000**	**6,000**	**6,000**	**8,000**	**8,000**
Coarse Agg. Size	57	67	57	67	57	67
Water (pcy)	289	289	267	267	225	225
Type I Cement (pcy)	564	564	658	658	0	0
Type III Cement (pcy)	0	0	0	0	752	752
Class F Fly Ash (pcy)	141	141	0	0	0	0
Coarse Agg. (pcy)	1824	1754	1874	1824	1950	1900
Fine Agg. (pcy)	1098	1130	1199	1209	1149	1157
Target Air (%)	5.0	5.0	5.0	5.0	5.0	5.0
Glenium 3000 NS (oz/cy)	0	0	0	0	67.68	60.20
Pozzolith 100 XR (oz/cy)	42.60	28.20	19.70	19.70	22.60	22.60
MB AE 90 (oz/cy)	21.20	21.20	5.00	5.00	5.00	5.00
w/c	0.51	0.51	0.41	0.41	0.30	0.30
w/cm	0.41	0.41	0.41	0.41	0.30	0.30

3.3 NOTATION

In this research project, the goal was to obtain 720 individual test results. A specific specimen identification system was developed to keep track of the data. A cylinder's identity was named in the order of aggregate, strength range, batch, technician, and age. The three batches of each mixture were labeled A, B, and C. For example, a cylinder with the identity 67-8-C-T1-7 would be a cylinder made from the third batch of the 8000 psi mixture with #67 coarse aggregate by technician 1 and tested after 7 days.

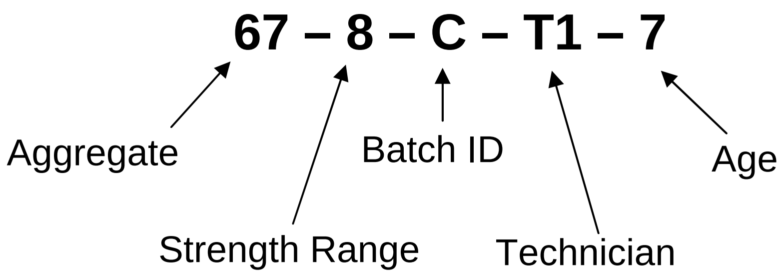

Figure 3.1: Example specimen identification system

3.4 RAW MATERIAL SOURCES

The following raw material sources were used for this project:

- Types I and III Portland Cement - Both types of portland cement were manufactured by Lafarge at their Atlanta, GA plant.
- Aggregates - Fine aggregate, No.57 coarse aggregate, and No.67 coarse aggregate were stocked and supplied by Twin City Concrete. Both coarse aggregates were crushed limestone. The No.57 crushed stone was obtained from Martin Marietta Materials in Auburn, AL. The No.67 crushed stone was obtained from Martin Marietta Materials in O'Neal, AL. The aggregates were tested to determine their gradation, and these results are summarized in Figures 3.1, 3.2, and 3.3. All aggregates were in accordance with ASTM C 33 (2002).
- Chemical Admixtures – High-range water reducer Glenium 3000 NS, mid-range water reducer and retarder Pozzolith 100XR, and air entraining agent MB AE 90 were supplied by Master Builders.

Table 3.3 gives the absorption capacities and the bulk specific gravities (saturated surface dry condition) for the aggregates and cementitious materials used. Figure 3.1 shows the gradation of the fine aggregate and the upper and lower limits provided by ASTM C 33 (2002). Figure 3.2 shows the gradation of the No.57 coarse aggregate and the upper and lower limits provided by ASTM C 33 (2002). Figure 3.3 shows the gradation of the No.67 coarse aggregate and the upper and lower limits provided by ASTM C 33 (2002).

Table 3.3: Specific gravities and absorption capacities for raw materials

Raw Material	Specific Gravity	Absorption Capacity (%)
Fine Aggregate	2.63	0.68
#57 Crushed Limestone	2.81	0.60
#67 Crushed Limestone	2.75	0.77
Class F Fly Ash	2.29	-
Type I Cement	3.15	-
Type III Cement	3.15	-

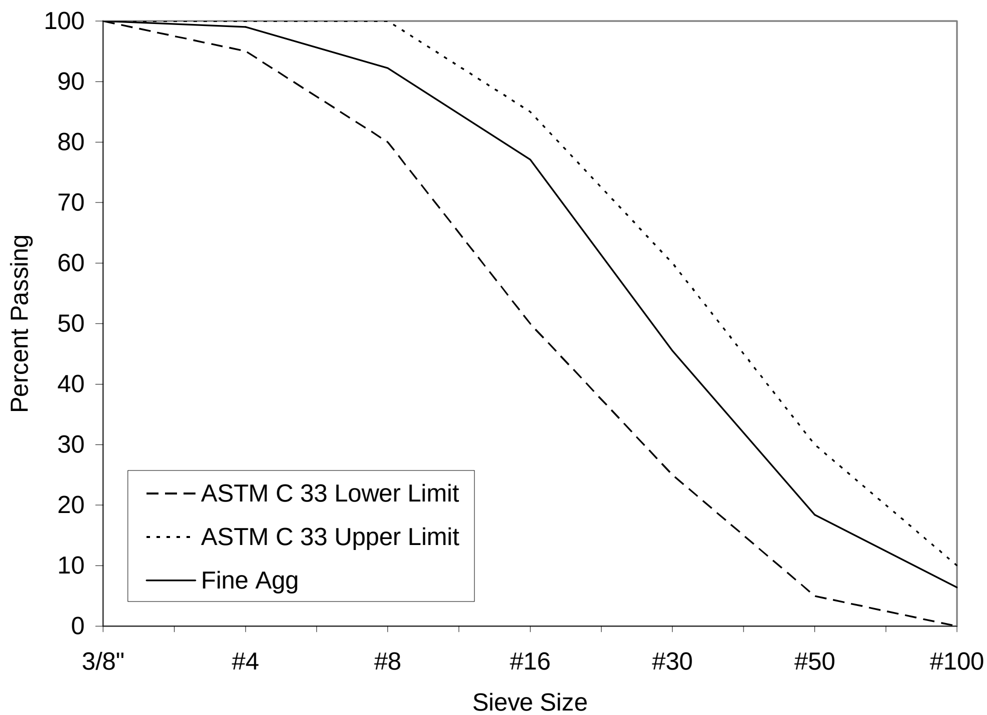

Figure 3.1: Fine aggregate gradation

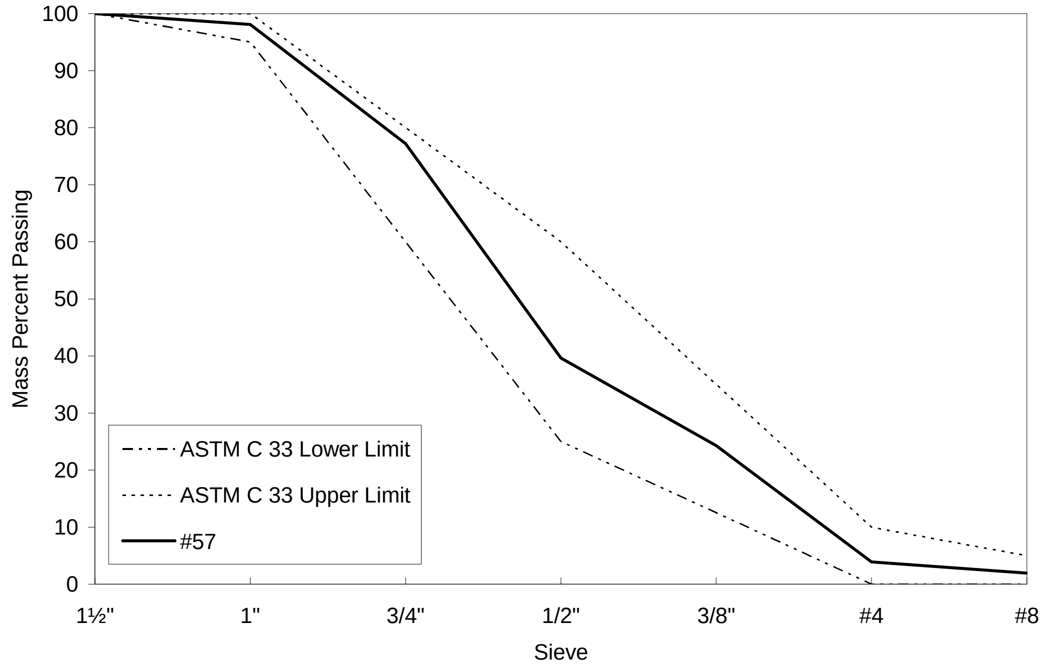

Figure 3.2: No.57 coarse aggregate gradation.

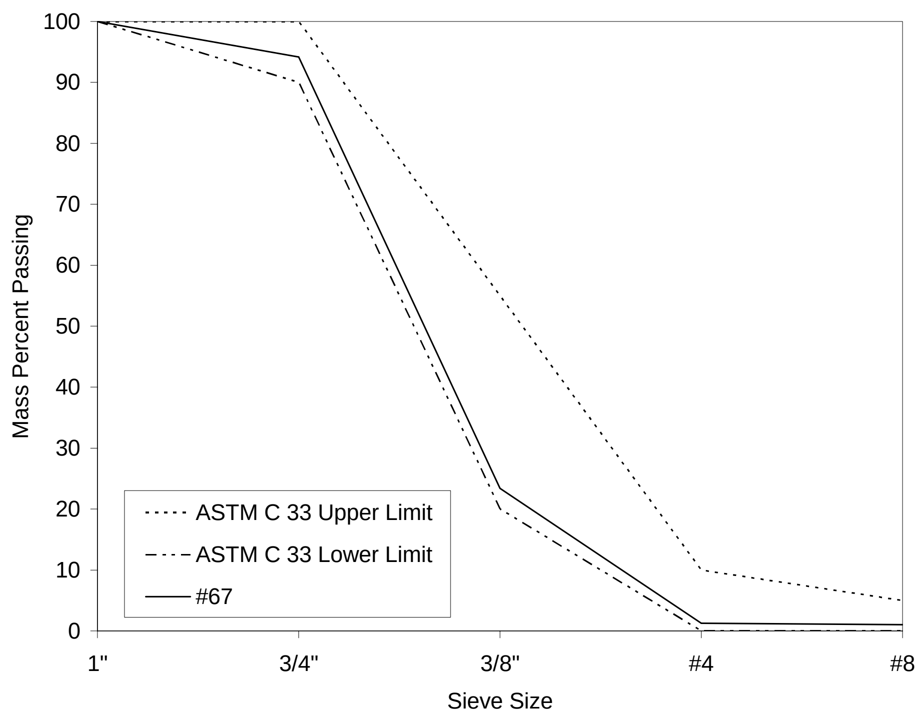

Figure 3.3: No.67 coarse aggregate gradation

Chapter 4

LABORATORY EQUIPMENT, SPECIMENS, AND PROCEDURES

4.1 GENERAL

Before the summer of 2003, concrete mixing at Auburn University was done in an outside environment subject to temperature and moisture variations that followed ambient conditions. In May of 2003, a new facility was built inside the Harbert Engineering Center of Auburn University's Civil Engineering Department. This new facility is indoors, eliminating the temperature and moisture variations associated with ambient conditions. Also, aggregates were kept in sealed 55 gallon drums with liners instead of outside wooden storage bins. All concrete mixing for this program was done in the new facility.

This facility consists of an elevated platform with access by stairs or by a ramp built for easy material handling. Figure 4.1 depicts this new facility. The platform was constructed of wood, covered with a conveyor belt material, and sealed with silicon to provide a water-tight area. All wastewater was collected in a drainage tub with a volume of approximately 200 gallons. Prior to mixing, the tub was filled approximately halfway to provide enough water to dilute the wastewater. Five to ten pounds of sugar was added to the tub to prevent any setting of cementitious materials. The tub was constructed with galvanized steel, a spray-on abrasion resistant liner, and a valve. All wastewater was treated with concentrated phosphoric acid to neutralize the pH. The pH was measured with a pH meter to ensure that the wastewater's pH was between 6.0 and 8.0 before discharging. The wastewater was then passed through a screen before being released out of the tank, into a flexible hose, and into the storm sewer.

Moisture corrections were conducted on fine and coarse aggregates for every batch using a small digital scale and a microwave. Batching was done using 5 gallon buckets and a large digital scale. Batching was done either the day before or the morning of mixing day. When batching was one the day before mixing, lids were securely placed on the 5 gallon buckets to prevent moisture loss or gain.

In order to efficiently batch and mix all the concrete for this project, two technicians were involved. Both technicians attended and successfully completed the "Level I – Concrete Field Testing Technician" certification as offered by the American Concrete Institute. Throughout this project, each specimen was made, transported, capped, and tested by the same technician as this allowed the research team to evaluate the effect of different technicians on the concrete strength. The results will be presented in terms of that obtained by "Technician 1" and "Technician 2."

Figure 4.1: Indoor mixing room

4.2 MIXING PROCEDURES

The original mixing procedure was as follows:

1. Add coarse aggregate and fine aggregate, mix for 3 minutes.
2. Add 50% of the water, mix for 3 minutes.
3. Add all cementitious materials at once, mix for 3 minutes.
4. Add remaining 50% of water and admixtures.
5. Rest for 3 minutes.
6. Mix for 3 minutes.
7. Perform fresh property tests and make cylinders.

This mixing procedure proved to be ineffective in thoroughly mixing the concrete. Cement and fine aggregate tended to collect in the back of the mixer. A second mixing procedure was established. The steps are as follows:

1. Add coarse aggregate and fine aggregate, mix for 3 minutes.
2. Add 100% of water and admixtures, mix for 3 minutes.

3. Add cementitious materials one bucket at a time, mixing for 30 seconds before adding another bucket.
4. Mix for 3 minutes.
5. Rest for 3 minutes.
6. Mix for 3 minutes.
7. Perform fresh property tests and make cylinders.

This new mixing procedure eliminated the problem of cement and fine aggregate collecting in the back of the mixer and helped in achieving the desired slump with greater ease. The estimated total elapsed time between when cementitious materials first had contact with water and when the last cylinder was made was about 45 minutes to one hour. Figure 4.2 shows a picture of the drum mixer used for this project.

Figure 4.2: 12-ft^3 concrete mixer

4.3 FRESH CONCRETE PROPERTY TESTING

Fresh property tests performed on each batch of fresh concrete were slump, air content by pressure meter, unit weight, and temperature. Slump tests were carried out according to ASTM C143 (2000). Unit weight tests were carried out according to ASTM C138 (2001) using the 0.25 ft^3 container from the pressure meter. Temperature tests were carried out according to ASTM C1064 (2001). Air content tests were carried out according to ASTM C231 (1997) using the concrete from the unit weight test, a pressure meter, and bulb syringe.

4.4 MAKING AND CURING SPECIMENS

All specimens were made and cured according to ASTM C192 (2000). All 6 x 12 in. cylinders were rodded with a 5/8-in. tamping rod, 25 times per layer, for three layers of equal height. All 4 x 8 in. cylinders were rodded with a 3/8-in. tamping rod, 25 times per layer, for two layers of equal height. After strike-off, all specimen molds were capped with a tightly sealed plastic lid and left to set. Cylinders were moved from mixing room to curing room on average of 30 hours after making. Curing conditions were held constant a 73^{o}F and at a 100% relative humidity in a moist-cure room. Cylinders were cured for the entire time until testing except for the time required to sulfur cap the cylinder ends.

4.5 CAPPING OF THE SPECIMENS

All specimens were capped according to ASTM C 617 (1987) with a molten sulfur-based compound. The sulfur compound and capping molds were manufactured by Forney. In this process, a hardened sulfur compound is melted at approximately 265^{o}F. The molten sulfur is then poured into the molds shown in Figure 4.3. The cylinder is slowly lowered into the mold displacing the molten sulfur. After 15-30 seconds, the sulfur will harden and the cylinder is removed from the mold.

Figure 4.3: Molds used for sulfur capping

4.6 COMPRESSIVE STRENGTH TESTING

The only hardened concrete property that was tested was compressive strength. All specimens were tested according to ASTM C39 (2001) with a Forney 400 kip compression machine. The load rate used for both size cylinders was 35 psi/sec. This is the middle value from the specified range of 20–

50 psi/sec. 35 psi/sec for a 6 x 12 in. cylinder is about 60,000 lbs/min and 26,000 lbs/min for a 4 x 8 in. cylinder. Each specimen was loaded in the compression machine shown in Figure 4.5 until complete failure occurred, and at this stage the peak load was recorded.

Figure 4.4: 400-kip Forney compression testing machine

Chapter 5

PRESENTATION OF RESULTS

5.1 GENERAL

During the conceptual stage of this research, the experimental program was designed to have three different strength ranges and three water-cementitious ratios. However, there ended up being three strength ranges with two water-cementitious ratios. The 6,000 psi mixes were attempted first. Air entrainment admixture dosages were taken from the middle of the recommended dosage given by the Master Builders website. However, this proved to be too high of a dosage. Due to the increased air content, the two mixes that were designed to give a 28 day strength of approximately 6,000 psi actually gave strengths of 4,000 psi on average. Since a 4,000 psi range of data was needed, all specimens made from the original 6,000 psi mix were relabeled as 4,000 psi data, and a new 6,000 psi mix was designed. Also, the over dosage of air-entrainer caused extreme segregation between the coarse aggregate and the cement paste in the trial batches of the 8,000 psi mixes. This was another reason to drop the air-entrainer dosage significantly. The problem was corrected before re-attempting the 6,000 psi and 8,000 psi mixes. It can be seen from Table 3.2 in Chapter 3 that the 6,000 psi and 8,000 psi mixes have an air-entrainer dosage one fourth that of the 4,000 psi mixes.

Table 5.1 summarizes the fresh concrete properties that were obtained for each batch during this testing program. It can be seen from Table 5.1 how the high air-entrainer dosage had an effect on the fresh concrete properties of the 4,000 psi mixes. Because of the high air content, the slump was high and the unit weight was low. Due to the low water-cement ratio of the 8,000 psi mixes, a high-range water reducer was needed to improve the workability of the concrete. There were several 8,000-psi batches that had to be repeated due to the low quality of cylinders.

Large “bugholes” and defects on the surfaces of the cylinders were caused by the inability to properly consolidate the concrete in the cylinders due to the decreased workability of the concrete over the time required to make all 40 cylinders. The approximate time required to make all 40 cylinders from a batch, after the concrete was thoroughly mixed, was approximately 45 minutes. This was enough time to maintain good workability for the 4,000 psi and 6,000 psi mixes, but not enough time for the initial 8,000 psi mixes. Therefore, the dosage of high-range water reducer was increased, and a new mixing process described in Section 4.2 was adopted. These corrections produced a concrete that maintained desired workability throughout the entire cylinder making period, which in turn resulted in good quality cylinders.

Table 5.1: Fresh concrete properties for each batch

Batch Title	Slump (in.)	Air Content (%)	Temp. (^{o}F)	Unit Weight (pcf)
57-4000-A	4.75	5.00	75	143.4
57-4000-B	5.00	6.25	74	143.4
57-4000-C	8.00	8.00	75	138.0
67-4000-A	7.00	6.75	75	138.1
67-4000-B	6.50	6.00	78	137.4
67-4000-C	6.50	6.75	75	135.1
57-6000-A	4.50	4.00	75	149.6
57-6000-B	3.25	5.00	75	148.5
57-6000-C	3.00	4.50	75	148.9
67-6000-A	2.25	4.00	76	148.7
67-6000-B	2.00	3.00	77	150.4
67-6000-C	2.00	3.00	78	150.6

57-8000-A	7.50	2.50	77	156.8
57-8000-B	7.00	7.00	77	146.2
57-8000-C	9.50	6.00	76	147.5
67-8000-A	8.50	6.50	75	146.0
67-8000-B	7.50	5.00	76	151.7
67-8000-C	8.50	7.00	78	143.9

5.2 ANALYZING k_s THROUGH GRAPHICAL REPRESENTATIONS

As discussed in Chapter 2, k_s is the ratio of 4 x 8 in. cylinder strength to 6 x 12 in. cylinder strength. Figures 5.1 and 5.2 show the normal distributions for the results of the three strength ranges. 4 x 8 in. cylinder normal distributions are plotted on the horizontal axis and 6 x 12 in. cylinder normal distributions are plotted on the vertical axis. Lines representing the mean of each distribution extend outward until they intersect with the mean line of the opposite cylinder size for the same strength range. This was done to plot the intersections of the means of each distribution against a 45^o line of equality. It can be seen that the mean compressive strengths for each strength range increased with age. However, the intersections of the means of the two cylinders sizes for a strength range do not change in relation to the equality line from 7 day to 28 day results. This graphical evaluation indicates that age does not affect the strength ratio k_s. The intersection of the means in an indicator as to the magnitude of k_s. If the intersection is below the line of equality, then the strength is in favor of the 4 x 8 in. cylinders and k_s is greater than 1.0. If the intersection is on the line of equality, then the strengths are equal and k_s is equal to 1.0. If the intersection is above the line of equality, then the strength is in favor of the 6 x 12 in. cylinders and k_s is less than 1.0. From Figures 5.1 and 5.2, it can be concluded from test data collected during this study that below 6,000 psi, k_s is greater than 1.0, and that above 6,000 psi, k_s is less than 1.0. Also, by examining Figures 5.1 and 5.2, it can be seen that the normal distributions for the 8,000 psi mixes have a much greater range than the 4,000 and 6,000 psi mixes indicating a higher degree of variability.

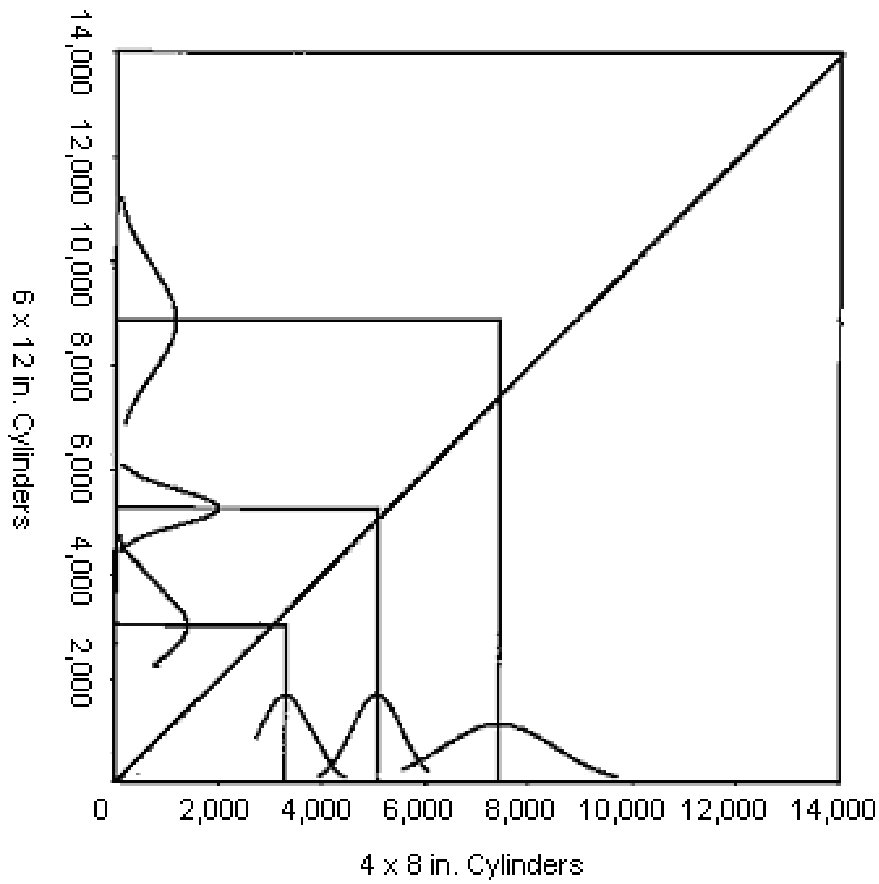

Figure 5.1: Normal distributions for 7-day results

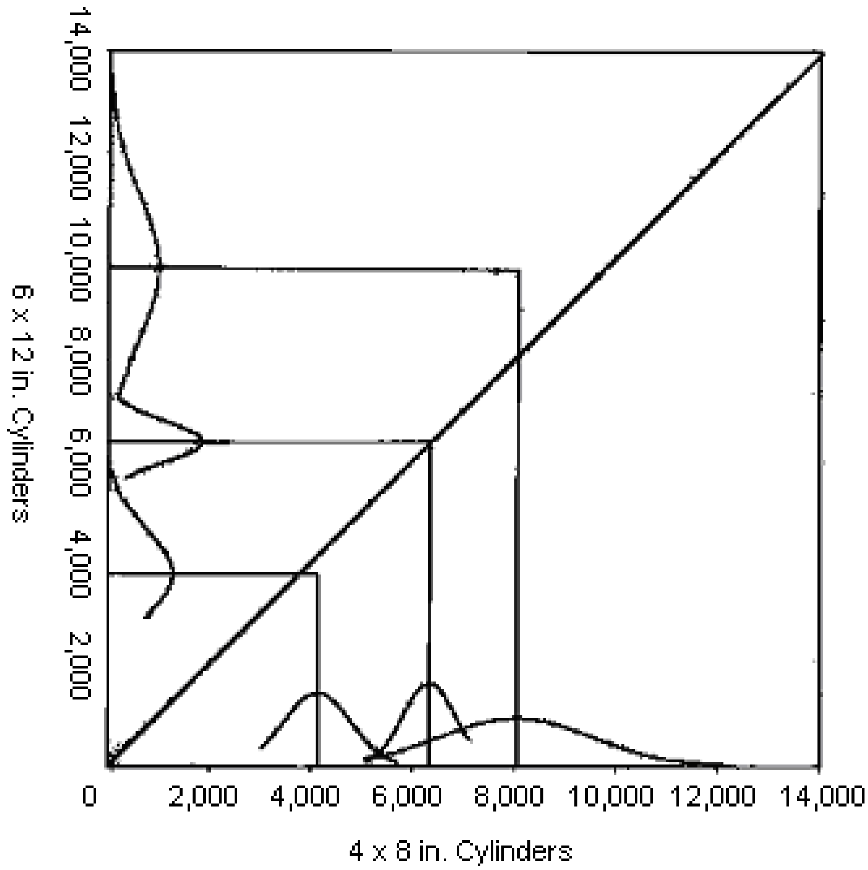

Figure 5.2: Normal distributions for 28-Day results

Figures 5.3, 5.4, and 5.5 are scatter plots for 7 day results, 28 day results, and the two ages superimposed on each other. Each point represents one 4 x 8 in. cylinder and one 6 x 12 in. cylinder. In order to plot the data in this fashion, test results were categorized into the most specific specimen designation of aggregate, strength range, batch, technician, and age. Then the data was sorted in order of strength and paired with a cylinder of the other size. From Figures 5.3 and 5.4, the variability can be assessed by the closeness of the data clusters. The 4,000 psi data showed very close clusters which indicates a high degree of consistency. The 6,000 psi data is less consistent with clusters that are slightly more dispersed than the 4,000 psi data. The 8,000 psi data shows a very low degree of consistency with clusters that are very dispersed. This would suggest that the variability of concrete test results increases with an increase in compressive strength. The trend of the data from Figures 5.3, 5.4, and 5.5 follows the trend of the intersections of means from Figures 5.1 and 5.2. For both 7- and 28-day scatter plots, the 4,000 psi data is below the equality line, the 6,000 psi data is on the equality line, and the 8,000 psi data is above the equality line. This trend would suggest that k_s decreases with increasing compressive strength.

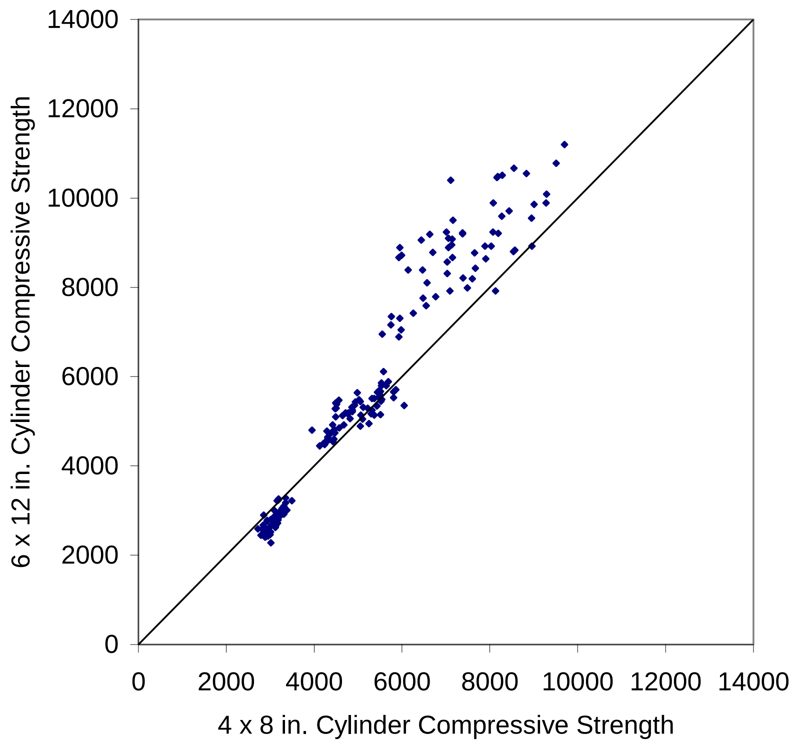

Figure 5.3: Scatter plot of 7-day strengths

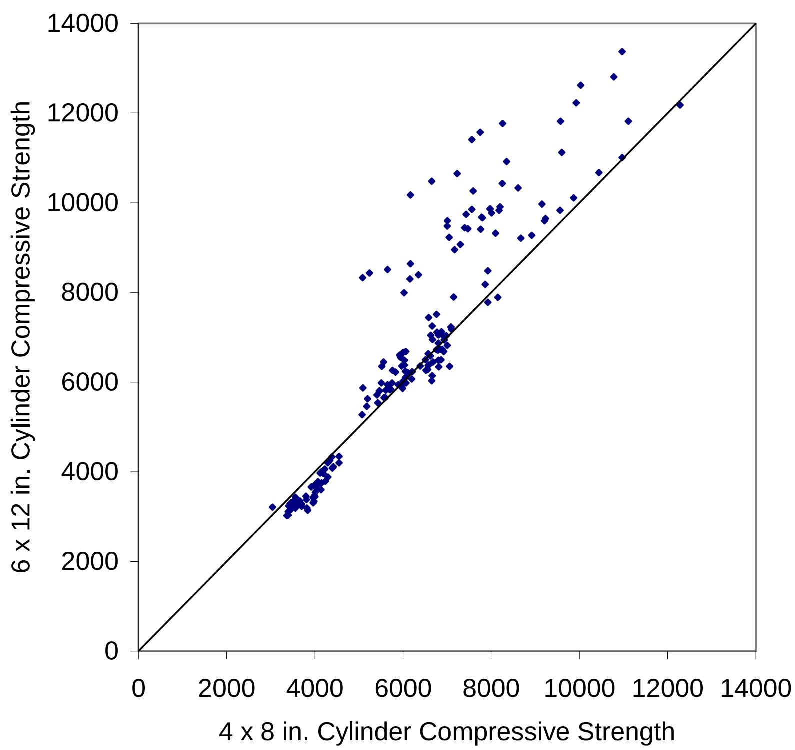

Figure 5.4: Scatter plot of 28-day strengths

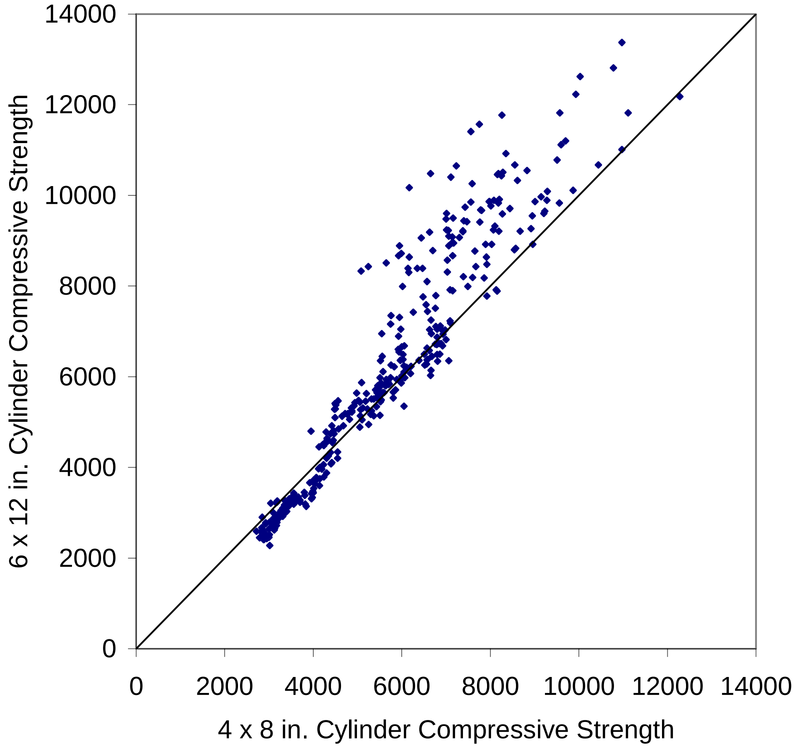

Figure 5.5: Scatter plot of 7- and 28-day strengths

Figures 5.6, 5.7, and 5.8 are bar graphs showing the individual k_s value for a batch at 7 and 28 days for technicians 1, 2, and both technicians. A horizontal line is plotted on all three figures showing a k_s value of 1.0. It can be seen from Figures 5.6, 5.7, and 5.8 that there is a trend for each strength range. The k_s values for the 4,000 psi batches tend to be above 1.0. The k_s values for the 6,000 psi batches tend to be about 1.0. The k_s values for 8,000 psi batches tend to be below 1.0. These trends follow the trends discussed in Figures 5.1 through 5.5. It is also worth noting that k_s does not appear to be affected by the aggregate size, age of the concrete, or technician.

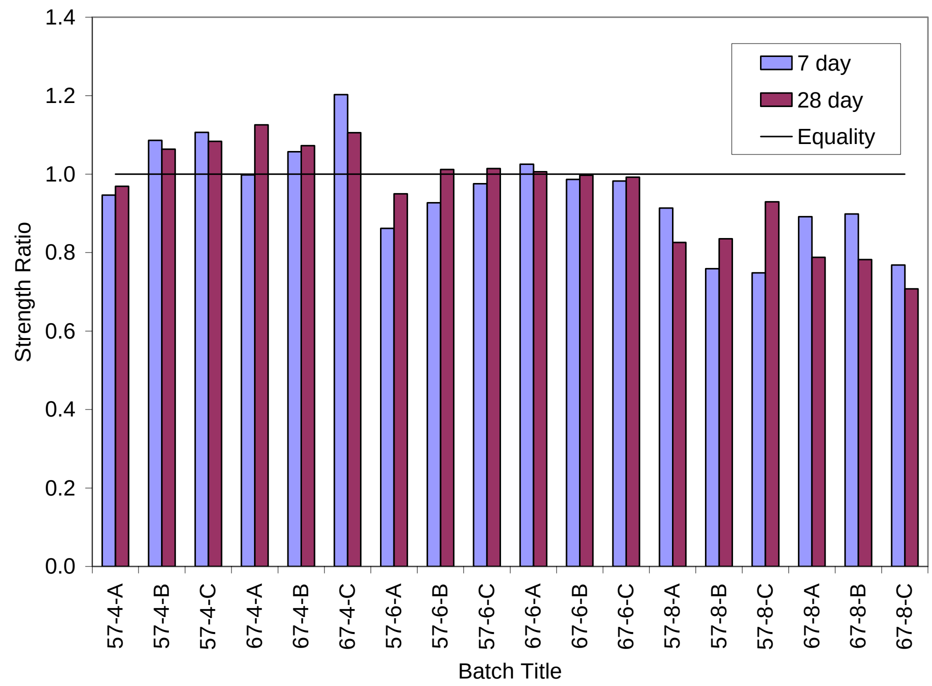

Figure 5.6: k_s values for Technician 1

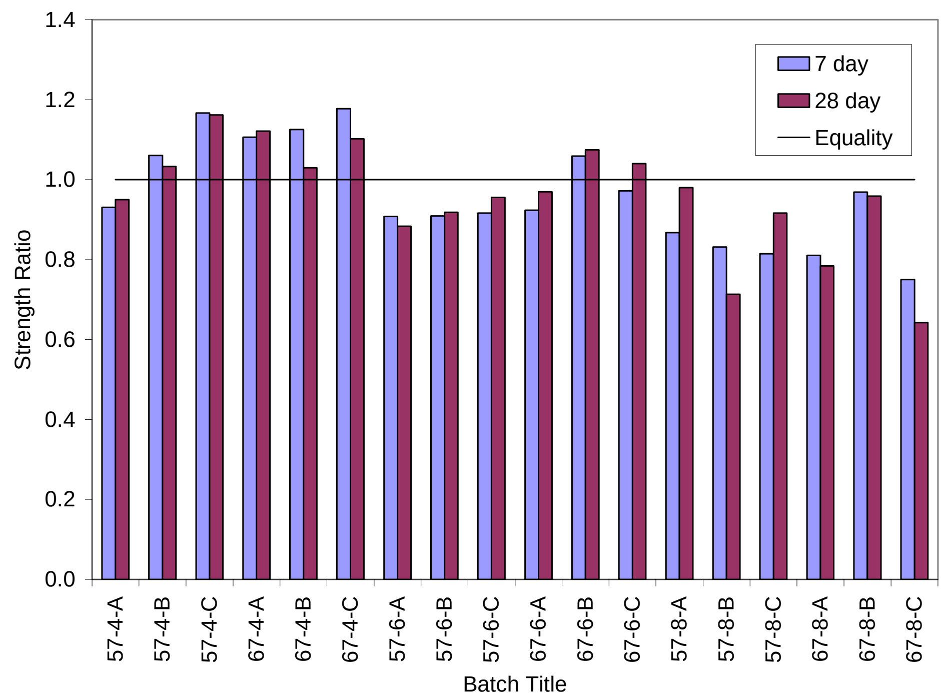

Figure 5.7: k_s values for Technician 2

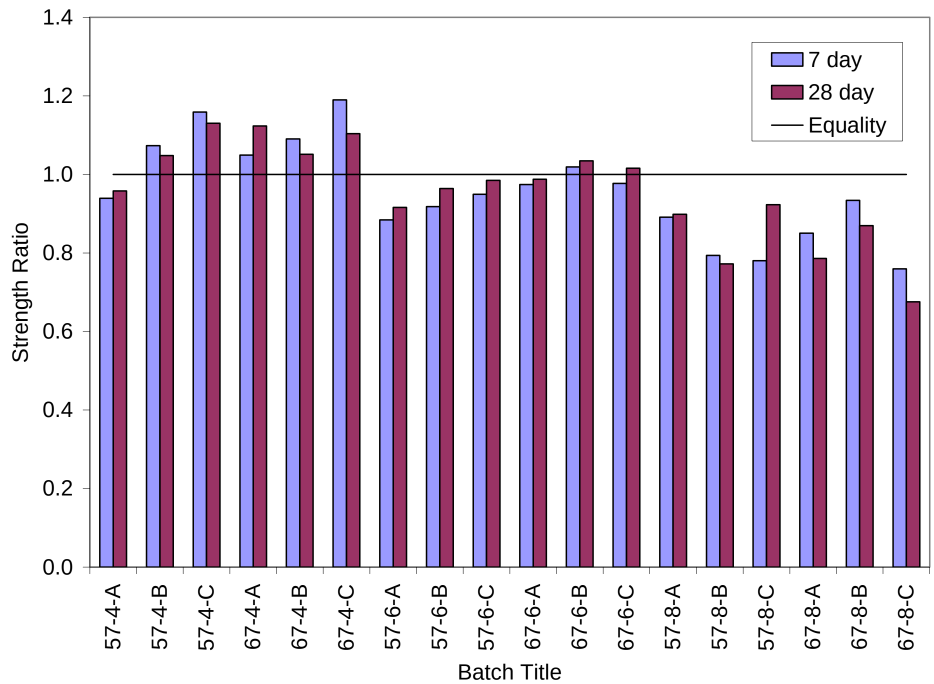

Figure 5.8: k_s values for both Technicians

5.3 TYPICAL VALUES OF k_s

A k_s value was computed for each of the 18 batches at concrete ages of 7 and 28 days using test results from both technicians. This gives a total of 36 k_s values. The ranges of these values can be seen in Table 5.2 for each strength range and overall. Each one of the 36 k_s values is computed from ten 4 x 8 in. cylinders and ten 6 x 12 in. cylinders.

Table 5.2: Ranges of values for k_s

Strength Range	Min	Average	Max
4,000 psi	0.94	1.08	1.19
6,000 psi	0.88	0.97	1.03
8,000 psi	0.68	0.83	0.93
all strengths	0.68	0.96	1.19

5.4 WITHIN-TEST VARIABILITY

ASTM C39 (2001) Section 10 states that the maximum coefficient of variation for 6 x 12 in. cylinders made under laboratory conditions should be 2.37% for concretes with compressive strengths between 2,000 psi and 8,000 psi. This is the maximum value for the coefficient obtained from a sample of test results that are statically sufficient to allow the calculation of a standard deviation. Table 1 in ASTM C670 provides multiplier values to use if the number of test results is between two and ten. If this is the case, then the value given for a certain number of test results is multiplied by the maximum coefficient of variation in ASTM C39-01. Doing this will give the maximum acceptable range of results within a batch and accounts for the limited number of test results available per batch. This is explained in Section 3.3.2 of ASTM C670-96. This upper limit can be seen as the dotted black line in Figures 5.9 through 5.14. It is labeled "C39/C670" because the value was obtained from the coefficient of variation given in ASTM C39-01 and the multipliers given in ASTM C670-96. The actual range of results within a batch, which is referred to as the "percent difference" in this report, is the numerical difference between the maximum and minimum test results divided by their average.

Table 3.5 of ACI 214-77 (Table 2.5 of this report) states that for cylinders made in laboratory trial batches, a coefficient of variation less than 2.0% is excellent, between 2.0 and 3.0% is very good, between 3.0 and 4.0% is good, between 4.0 and 5.0% is fair, and above 5% is poor. If the upper limits of each of these categories are multiplied by the multipliers given in Table 1 of ASTM C670-96, this will give the maximum acceptable ranges for excellent, very good, good, and fair. These upper limits can be seen as the four other horizontal lines on Figures 5.9 through 5.14. They are labeled "214/C670" because the values were obtained from the coefficient of variation given in ACI 214-77 and the multipliers given in ASTM C670 (1996).

Figures 5.9 through 5.14 summarize the within-test variability obtained from this study as these graphs show the actual percent difference for each batch. All actual percent differences shown were computed from the multipliers for five test results for each technician and ten test results for both technicians. Exceptions to this are the actual percent differences computed for 6 x 12 in. cylinders labeled 57-4-A-T2-7, 57-4-A-T2-28, 67-6-B-T2-7, and the 4 x 8 in. cylinders labeled 57-6-C-T2-7. These were computed from the multipliers for 4 tests results for each technician and 9 test results for both technicians. This is because out of 720 proposed specimens in this research project, a total of 716 cylinders were tested and used as test results.

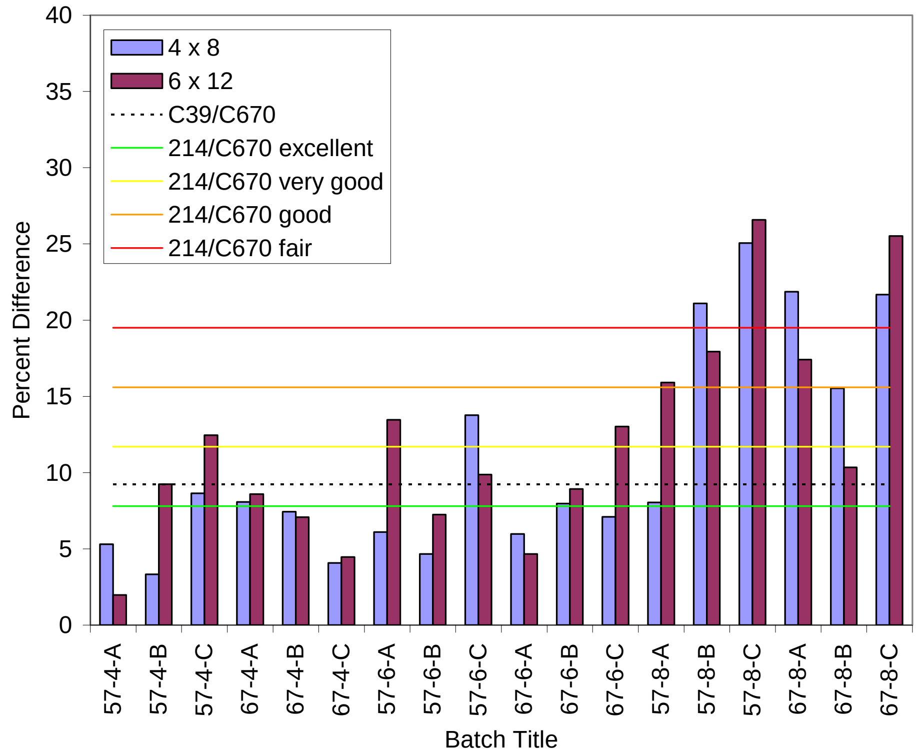

Figure 5.9: Percent difference for Technician 1: 7-day results

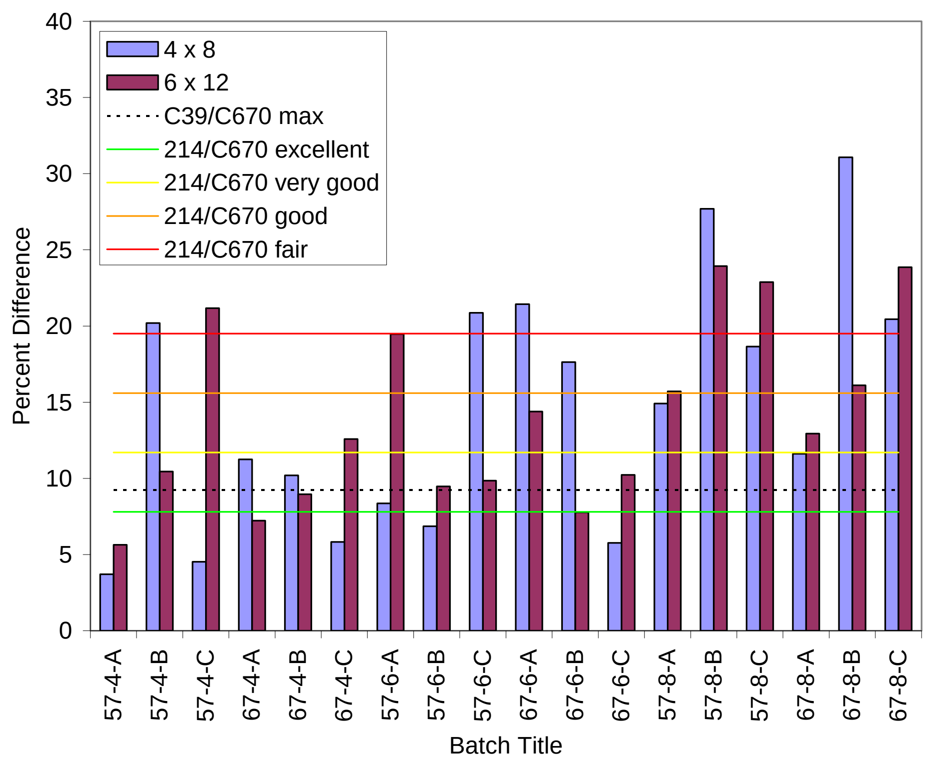

Figure 5.10: Percent difference for Technician 2: 7-day results

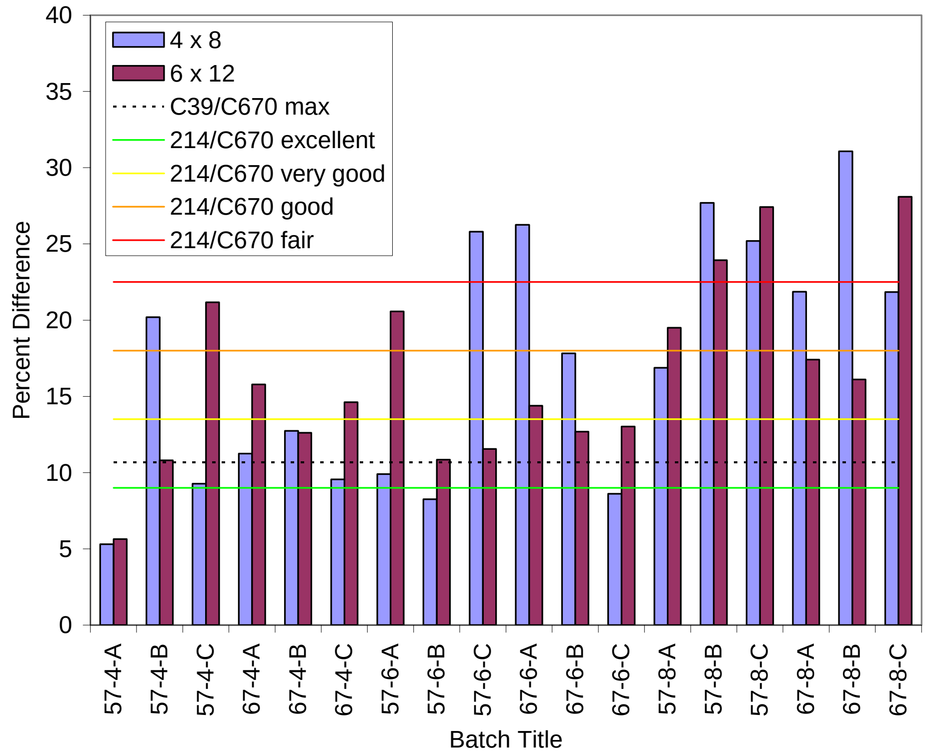

Figure 5.11: Percent difference for both Technicians: 7-day results

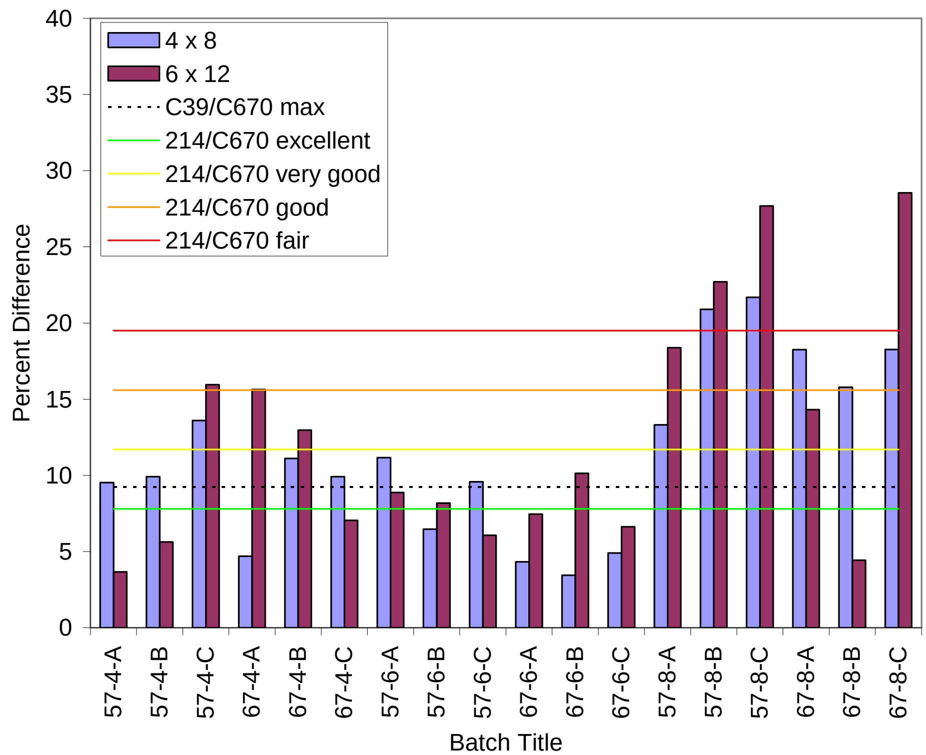

Figure 5.12: Percent difference for Technician 1: 28-day results

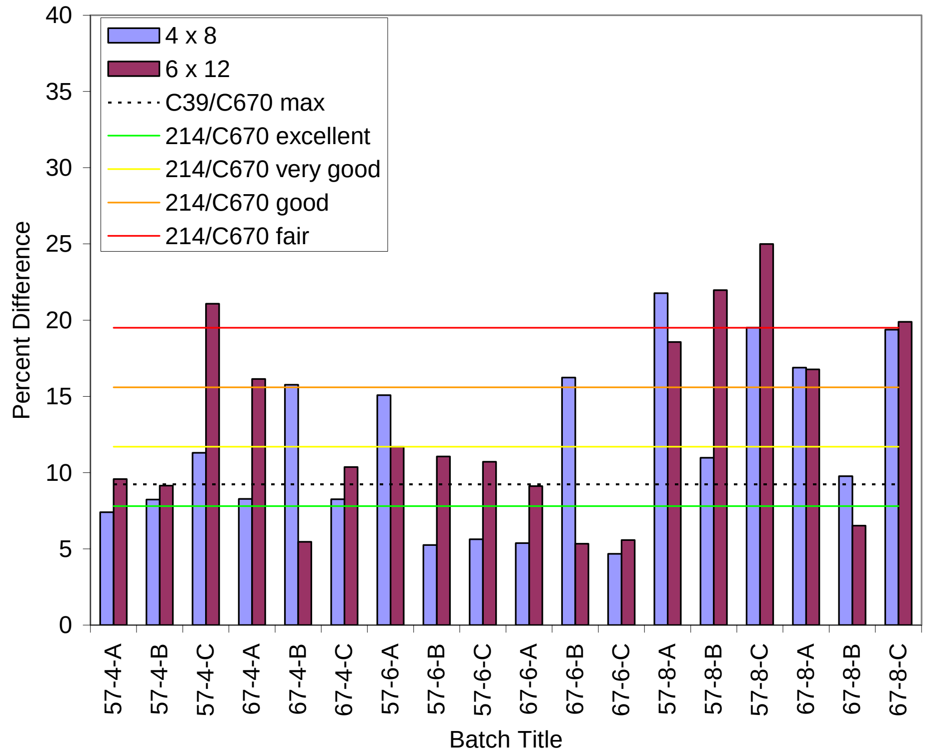

Figure 5.13: Percent difference for Technician 2: 28-day results

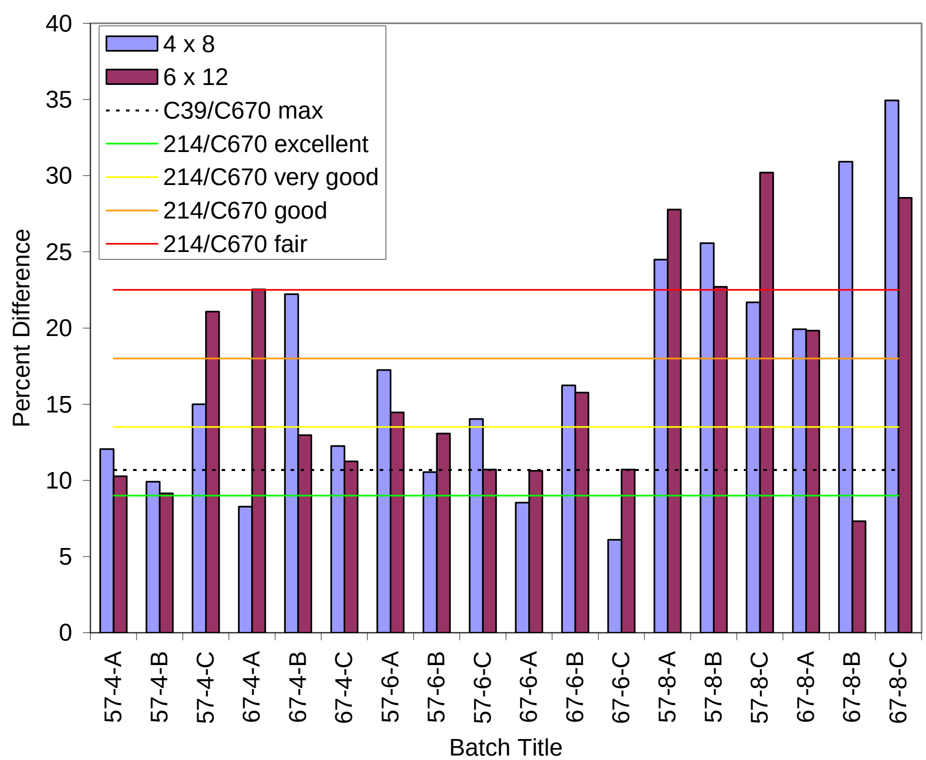

Figure 5.14: Percent difference for both Technicians: 28-day results

It can be seen from Figures 5.9 through 5.14 that the 8,000 psi data showed the most within-batch variability, which exceeded the “ASTM C39 / ASTM C670” maximum limit. The 4,000 and 6,000 psi data showed percent differences that fell within all the categories based on the “ACI 214 / ASTM C670” limits: excellent, very good, good, fair, and poor. In Figures 5.9 through 5.14, the solid horizontal lines represent the upper limits of the “ACI 214 / ASTM C670” categories and the dashed horizontal line represents the “ASTM C39 / ASTM C670” maximum limit. The order from top to bottom on the graph is as follows: 214/C670 fair, 214/C670 good, 214/C670 very good, C39/C670 maximum, 214/C670 excellent.

5.5 ANOVA ANALYSIS

Until this point, the determination of which variables affected k_s and within-test variability has been based on graphical interpretations of the data. In order to statistically determine which variables affects k_s and the within-test variability, an analysis of variance (ANOVA) statistical test was performed on the data. ANOVA is a statistical procedure to test the null hypothesis that the means of two or more populations are equal (El Mogazhy 2001). The values computed by the ANOVA test are the sum of squares, F value, P value in percent, and the critical F value for a given confidence level. If the computed F value is less than the critical F value, then the null hypothesis is accepted. If the computed F value is more than the critical F value, then the null hypothesis is rejected. The critical F value is a function of the degree of freedom and the confidence level. The P value in percent indicates the probability associated with the decision to accept or reject the null hypothesis (El Mogazhy 2001). The number of degrees of freedom associated with a particular test is one less than the number of population means tested. In order to determine which of the factors will reject the hypothesis, an ANOVA analysis was performed for every possible combination of factors. The factors studied for k_s were age, aggregate, technician, and strength range. The factors studied for the within-batch variability were age, aggregate, technician, strength range, and cylinder size. To prepare the data for the ANOVA analysis, the data was arranged to its most specific description described in Figure 3.1. All possible combinations of variables were determined and the data were then rearranged into the categories. The number of possible combinations by selecting r variables from a set of n variable is given by the factorial expression:

$$\frac{n!}{r!(n-r)!} \qquad \text{Equation 5.1}$$

Fifteen total possible combinations for k_s and 31 total possible combinations for within-test variability were considered. Tables 5.3 and 5.4 give the degree of freedom, computed sum of squares, computed F value, computed P percentage, and critical F value at 90%, 95%, and 99% confidence levels for all possible combinations of factors for k_s and within-test variability. It can be seen from Tables 5.3 and 5.4 that the computed F value for all categories containing strength range as a factor is much greater than all three critical F values for the confidence levels. The computed sum of squares for all categories containing strength range as a factor is much larger than the sum of squares of categories not containing strength range. This would indicate that there was a higher degree of error when comparing the means of populations with strength range as a factor. Also, the computed P percentage for categories containing strength range is extremely low compared to that of categories without strength range. Recall that the P value as a percent is associated with the probability of accepting or rejecting the null hypothesis (El Mogahzy 2001). The extremely low P percentages for categories with strength range, suggests that there is a very small probability in accepting the null hypothesis. Recall that the null hypothesis of ANOVA is that the mean of two or more populations are equal. In this analysis, the common factor in the categories where null hypothesis was rejected is the significant factor in affecting k_s and within-test variability.

Therefore it is determined that strength range is the only factor among all tests that rejected the null hypothesis for k_s and within-batch variability at the 90%, 95%, and 99% confidence levels. Therefore it is also determined that aggregate size/gradation, age at testing, and technician are not significant in affecting k_s. It is also be determined that these factors along with cylinder size are not significant in affect within-test variability. Compressive strength is the only significant factor in affecting k_s and within-test variability and proved to be statistically significant at a 99% confidence level.

Table 5.3: ANOVA analysis results for k_s

Model	D.O.F.	Sum of Squares	F value	P value (%)	$F_{critical}$ α=0.01	$F_{critical}$ α=0.05	$F_{critical}$ α=0.1
age	1	0.0001	0.003	95.5	7.01	3.98	2.78
technician	1	0.0003	0.016	90.0	7.01	3.98	2.78
aggregate	1	0.0163	1.039	31.2	7.01	3.98	2.78
strength range	2	0.7293	65.5	1.12E-14	4.93	3.13	2.38
technician / age	3	0.0008	0.016	99.7	4.08	2.74	2.17
technician /aggregate	3	0.0179	0.371	77.4	4.08	2.74	2.17
aggregate / age	3	0.0268	0.559	64.4	4.08	2.74	2.17
technician / strength range	5	0.7334	25.5	3.19E-12	3.31	2.35	1.94
aggregate / strength range	5	0.7805	31.0	4.40E-14	3.31	2.35	1.94
strength range / age	5	0.7369	25.9	2.37E-12	3.31	2.35	1.94
technician / age / aggregate	7	0.0303	0.256	96.8	2.93	2.16	1.81
technician / strength range / age	11	0.7428	10.9	1.00E-08	2.56	1.95	1.68
technician / strength range / aggregate	11	0.7885	13.2	2.49E-10	2.56	1.95	1.68
strength range / age / aggregate	11	0.8086	14.5	4.07E-11	2.56	1.95	1.68
technician / aggregate / age / strength range	23	0.8262	6.0	8.90E-06	2.22	1.76	1.55

Table 5.4a: ANOVA analysis results for percent difference

Model	D.O.F.	Sum of Squares	F value	P value (%)	$F_{critical}$ α=0.01	$F_{critical}$ α=0.05	$F_{critical}$ α=0.1
age	1	16.78	0.378	54.0	6.82	3.91	2.74
technician	1	93.28	2.127	14.7	6.82	3.91	2.74
aggregate	1	77.63	1.766	18.6	6.82	3.91	2.74
cylinder size	1	18.95	0.427	51.4	6.82	3.91	2.74
strength range	2	2782.66	55.5	1.70E-16	4.76	3.06	2.34
age / technician	3	167.12	1.268	28.8	3.92	2.67	2.12
age / aggregate	3	114.37	0.860	46.3	3.92	2.67	2.12
age / cylinder size	3	42.43	0.315	81.4	3.92	2.67	2.12
technician / aggregate	3	172.05	1.306	27.5	3.92	2.67	2.12
technician / cylinder size	3	115.17	0.866	46.0	3.92	2.67	2.12
aggregate / cylinder size	3	140.98	1.065	36.6	3.92	2.67	2.12
strength range / age	5	2937.20	24.0	2.67E-15	3.15	2.28	1.89
strength range / aggregate	5	2890.23	23.3	6.76E-15	3.15	2.28	1.89
strength range / cylinder size	5	2808.19	22.1	3.31E-14	3.15	2.28	1.89
strength range / technician	5	2936.19	24.0	2.73E-15	3.15	2.28	1.89

Table 5.4b: ANOVA analysis results for percent difference

Model	D.O.F.	Sum of Squares	F value	P value (%)	$F_{critical}$ α=0.01	$F_{critical}$ α=0.05	$F_{critical}$ α=0.1
Technician / age / aggregate	7	266.01	0.854	54.5	2.77	2.08	1.76
Technician / age / cylinder size	7	203.80	0.647	71.6	2.77	2.08	1.76
Technician / aggregate / cylinder size	7	276.67	0.890	51.7	2.77	2.08	1.76
Age / aggregate / cylinder size	7	188.65	0.598	75.7	2.77	2.08	1.76
Technician / age / strength range	11	3147.81	11.9	2.86E-	2.38	1.86	1.62
Age / aggregate / strength range	11	3096.74	11.5	7.65E-	2.38	1.86	1.62
Age / cylinder size / strength range	11	2978.85	10.7	6.92E-	2.38	1.86	1.62
Technician / aggregate / strength range	11	3051.67	11.2	1.80E-	2.38	1.86	1.62
Technician / cylinder size / strength range	11	2982.46	10.7	6.48E-	2.38	1.86	1.62
Aggregate / cylinder size strength range	11	2990.14	10.8	5.63E12	2.38	1.86	1.62
Technician / age / aggregate / cylinder size	15	402.44	0.580	88.6	2.18	1.75	1.54
Technician / age / aggregate / strength range	23	3317.37	5.8	5.57E-	1.97	1.62	1.46
Age / aggregate / cylinder size / strength range	23	3227.98	5.4	2.46E-	1.97	1.62	1.46
Technician / aggregate / cylinder size / strength range	23	3234.91	5.5	2.19E-	1.97	1.62	1.46
Technician / age / cylinder size / strength range	23	3218.97	5.4	2.84E-	1.97	1.62	1.46
Technician / age / aggregate / cylinder size/ strength	47	3563.31	2.6	2.93E-	1.76	1.49	1.37

5.6 COMPARISON OF TEST RESULTS TO RESULTS FOUND IN THE LITERATURE REVIEW

In this research project, it was found that 4 x 8 in. cylinders were generally stronger than 6 x 12 in. cylinders in compression when strengths were less than 6,000 psi and 4 x 8 in. cylinders were generally weaker than 6 x 12 in. cylinders when strengths were greater than 6,000 psi. The strength ratio k_s in this project ranged from a low of 0.68 to a high of 1.19 with an overall average of 0.96. Refer to Table 5.2 for the minimum, average, and maximum k_s value for the three strength ranges and for the average obtained for the overall project.

The literature review found that 4 x 8 in. cylinders in some cases were generally stronger in compression than 6 x 12 in. cylinders with increasing compressive strength. Day (1994 a) found that for the range of 2,900 psi and 14,500 psi, f_{c4} is expected to be 5% higher than f_{c6}. However, in the lower strength range of 2,900 psi to 8,700 psi, f_{c4} and f_{c6} can be assumed equal. Malhotra (1976) suggests that 4 x 8 in. cylinders could be weaker than 6 x 12 in. cylinders at low strengths and reported a k_s range of 0.85 to 1.05. R. L. Carrasquillo has researched the effect of test cylinder size on compressive strength on several occasions and reported the following k_s values: Carrasquillo et al. (1981) k_s = 0.90, Peterman and Carrasquillo (1983) k_s = 1.10 to 1.15, Carrasquillo and Carrasquillo (1988) k_s = 0.93. Forstie and Schnormeier (1981) state that k_s is greater than 1.0 for concretes with compressive strengths greater than 5,000 psi. Other k_s values and ranges can be seen in Table 2.1. The literature showed that even though average k_s values were equal to or greater than 1.0, the range of k_s value could range from 0.85 to 1.15.

ASTM C31 (2000) states that "when cylinders smaller than the standard size are used, within-test variability has been shown to be higher but not to a statistically significant degree." Pistilli and Willems (1993) conducted extensive research concerning the effects of cylinder size and cylinder end conditions on within-test variability. Their results showed that cylinder size did not affect within-test variability. Based on a 95% confidence level, they show that the variations for 4 x 8 in. and 6 x 12 in. cylinders are the same when capped with sulfur and in the range of 2,000 psi and 15,000 psi. The results concluded by Pistilli and Willems (1993) are confirmed in this research project. Based on the test results in this project and a 99% confidence level, it was concluded that cylinder size was not significant in affecting the within-test variability through the ANOVA statistical analysis.

Based on the materials tested in this project and the ANOVA analysis, age of concrete and aggregate size had no effect on k_s. This agrees with Date and Schnormeier (1984) who found that k_s remained constant "for commonly used concrete, at any stage of curing." Day and Haque (1993), Aitcin et al. (1994), and Issa et al. (2000) showed results that suggested k_s was affected by age. However, Issa et al. (2000) found that k_s was affected by aggregate size.

Cook (1989) showed that concretes with compressive strengths over 10,000 psi can be produced with quality even though the variability far exceeds that of normal strength concrete. The suggestions made by Cook (1989) concerning how to properly quantify the acceptable variability of high-strength concrete have been implemented in ACI 318-02 and ACI 363.2R-98. Kennedy et al. (1995) report that within-laboratory and between-laboratory standard deviations increased as the average compressive strength increased as shown in Figure 2.9. Pistilli and Willems (1993) showed the increase in the average range of pairs of two cylinders with increasing compressive strength. Similar results were found as well in this research project. Recall that the percent difference of test results of a batch is the numerical difference between the maximum and minimum strengths divided by the average of the two. This percent difference was the basis for defining within-test variability. As seen from Figures 5.9 through 5.14, the percent differences for the 8,000 psi batches were far greater than the percent differences for the 4,000 psi and 6,000 psi batches which is similar to the results found in the literature review. This was also determined by the ANOVA analysis, concluding that strength range was the only factor significant in affecting within-test variability.

Chapter 6

PRACTICAL IMPLEMENTATION OF RESULTS

6.1 GENERAL

When considering the use of 4 x 8 in. cylinders in the current concrete industry, there are three main questions that arise:

1. What is the relationship between the strengths of 4 x 8 in. cylinders and 6 x 12 in. cylinders?
2. Is the variability of strength of the two cylinder sizes comparable?
3. How should 4 x 8 in. cylinders be used during quality assurance testing?

Based on the literature reviewed and the results presented, the questions will be answered in order to provide some guidelines towards the use of 4 x 8 in. cylinders for quality control purposes.

6.2 IMPLEMENTATION BASED ON THE LITERATURE REVIEW

Malhotra (1976) states that “the standard deviation of the compressive strength of test cylinders increases with decrease in the cylinder diameter as indicated by Tucker’s ‘summation-strength’ theory. However, the magnitude of this increase is such that considerably more than twice the number of 4 x 8 in. cylinders will have to be tested for each cylinder to obtain the same degree of precision.” Tucker’s theory gets more specific by stating that the number of specimens needed to obtain the same degree of precision is the ratio of the cross-sectional areas of two different specimen sizes, but that the strengths of the two specimen sizes are equal. This would require 2.25 more 4 x 8 in. cylinders to be tested than 6 x 12 in. cylinders. Current quality control testing requires that two 6 x 12 in. cylinders be used. The criteria given by Malhotra (1976) would suggest that five 4 x 8 in. cylinders be used.

Hester (1980) is not in favor of using 4 x 8 in. cylinders due to research stating that smaller specimens result in higher strengths and increased variability. He suggests the exclusive use of 6 x 12 in. cylinders. However, he does state that “if other specimen shapes or sizes are used, meticulous attention must be given to minimization of testing errors, and an increased number of specimens should be tested.” From his extensive study of high-strength concrete, Cook (1989) found that 4 x 8 in. cylinders were approximately 5% higher in compressive strength than 6 x 12 in. cylinders. He says that “a laboratory trial batch program seems to be the most effective method of determining concrete properties and establishing mixture proportions for high-strength concrete. Specimen size used for strength measurements of laboratory trial batches should be consistent with the size used by the specifier for acceptance on the project.”

Day (1994 a) compiled data from all possible resources concerning the correlation between 4 x 8 in. and 6 x 12 in. cylinders. From these compiled resources, he was able to analyze over 8,000 test results. Based on his study, Day concluded that “due to the equivalence of coefficients of variation, there is little justification for future specifications to require the testing of 3 rather than 2 cylinders when 4 x 8 in. plastic molds are used.” Day (1994 a) states that “the coefficient of variation of 4 x 8 in. cylinders is equivalent to that of 6 x 12 in. cylinders over a broad range that encompasses normal, high, and very high-strength concrete.” This is contrary to the views of Malhotra (1976), Tucker (1945), and Hester (1980) who all claim that more cylinders would have to be tested for cylinders sizes smaller than 6 x 12 in. However, it should be noted that Malhotra (1976), Tucker (1945), and Hester (1980) are basing their conclusions using standard deviation as a control standard. Day (1994 a) and Cook (1989) both use coefficient of variation as their standard of control.

The Canadian Standards Association, CSA, has implemented in the specification CSA A23.1 the requirement that f_{c4} be reduced by 5% to be compared with f_{c6}. ACI 363.2R-98 states that “when 4 in. diameter cylinders have been used for QA/QC testing in the U.S.A., strength reductions have not been applied to the measured strength.” ACI 363.2R-98 also states that “regardless of specimen size, the size used to evaluate trial mixture proportions should be consistent with the size specified for acceptance testing, and should be acceptable to the Architect/Engineer. If necessary, the relationship between the compressive strengths of the two specimen sizes can be determined at the laboratory or field trial stage using the testing machine that will be used for the project.” It can be concluded from Day (1994 a), Cook (1989), and ACI 363.2R-98 that 4 x 8 in. cylinders can be used for QA/QC testing in the same quantities as 6 x 12 in. cylinders and as long as 4 x 8 in. cylinders remain as the consistent size used on a single project.

Section 450-4 of the Florida Department of Transportation 2004 Standard Specifications for Road and Bridge Construction states that “when the maximum nominal size of the aggregate does not exceed 1 inch, 4 x 8 in. test cylinders may be used for compressive strength tests of concrete. The use of 4 x 8 in. test cylinders requires that the approved mix design contains compressive strength data for both 6 x 12 in. and 4 x 8 in. test cylinders. For the QA tests the Engineer will use the same size test cylinders that are used for the QC sampling and testing. Obtain the same compressive strength specified in the Contract Documents for 6 x 12 in. cylinders for 4 x 8 in. cylinders or obtain proportionally adjusted specified strength when the mix design correlation data indicates higher compressive strength for 4 x 8 in. cylinders.” This process is described in a personal correspondence between Ghulam Mujtaba, the Florida State Prestressed Concrete Engineer, and Sergio Rodriguez, the Alabama State Concrete Engineer. In this correspondence, Mr. Mujtaba states the following process in implementing 4 x 8 in. cylinders in concrete compressive strength testing:

1. The maximum nominal size of the aggregate in the concrete mix shall not exceed 1 inch.
2. For each designed concrete mix the results of the compressive strength of concrete using 4 x 8 in. and 6 x 12 in. cylinders shall be correlated to each other.
3. Based on the results of the established correlation, the strength of concrete using 4 x 8 in. test cylinders shall be proportionally adjusted. The adjustment is described in the following example.

Example: The specified 28-day strength based on 6 x 12 in. cylinders is 6,000 psi. Correlation data indicates that the average compressive strength of concrete using 4 x 8 in. cylinders is 5% higher than the compressive strength using 6 x 12 in. cylinders. Based on the results of the correlation data of the concrete mix, the required 28-day strength shall be at least 6,300 psi when using 4 x 8 in. cylinders.

It should be noted that the FDOT Specification 450 does not refer to the case when the compressive strength of 4 x 8 in. cylinders is less than that of 6 x 12 in. cylinders made from the same batch of concrete.

6.3 DISCUSSION OF THE REQUIRED AVERAGE COMPRESSIVE STRENGTH OF TEST SPECIMENS

To review, Section 5.3.2 of ACI 318-02 discusses the required average compressive strength, f'_{cr}, of test specimens in order to determine the design strength of concrete, f'_c. Table 5.3.2.1 of ACI 318-02 gives equations to determine f'_{cr} from f'_c when a standard deviation of the concrete strength is available. According to ACI 318-02, the minimum number of test specimens needed to compute a standard deviation is 30. When the number of test specimens is between 15 and 30, a modification factor is used to increase the known standard deviation. It is most likely that when mixing trial batches of concrete, less than 15 cylinders will be available. Therefore Equation 2.4, which is the equation given in Table 5.3.2.2 of ACI 318-02 for computing the design strength of concrete over 5,000 psi, will most likely be used when performing trial batches.

$$f'_{cr} = 1.1 f'_c + 700 \qquad \text{Equation 2.4}$$

It was stated by Aitcin et al. (1994) that as laboratories and testing agencies are very often equipped with testing machines having full load capacities rarely in excess of 300,000 lbf, the maximum compressive strength of concrete that can be tested on 6 x 12 in. specimens is just over 10,000 psi when operating at full load, which is not safe on a routine basis. Assuming that the average compression machine currently available in the Alabama concrete industry has a maximum capacity of 250,000 lbf and safe operation does not exceed 80% of that value, 200,000 lbf is the maximum load available for the average compression machine. This means that the upper limit of safe compressive strength testing for a 6 x 12 in. cylinder on the average compression machine is approximately 7,000 psi. Assuming this value as f'_{cr}, and using Equation 2.4, the maximum f'_c available is approximately 5,800 psi. However, design strengths of concrete used in high-rise buildings and prestressed applications are well above this value, which again illustrates the potential need to use 4 x 8 in. cylinders when testing high-strength concrete.

6.4 Suggested Implementation Procedure Based on Test Results

Based on the results presented in Chapters 2 and 5, it can be concluded that no universal k_s value could be recommended for use on all concretes. By nature, concrete is a variable material and can be affected by numerous factors. Assigning a universal k_s value applicable to all concretes would not be good practice as it may be unconservative in some cases and overly conservative in others. A better approach would be to determine the appropriate k_s value for each individual concrete mixture. It is recommended, for assurance purposes, that the required 28 day compressive strength, f'_c, should be determined based on tests performed on 6 x 12 in. cylinders. A suggested procedure to implement this approach is as follows:

1. Perform a trial batch of sufficient size to be representative of typical production of the desired concrete mixture. Make three 4 x 8 in. and three 6 x 12 in. cylinders from the same batch of concrete.
2. Test the compressive strength of the 6 x 12 in. cylinders (f_{c6}). Confirm that f_{c6} meets the f'_{cr} required so that the concrete exceeds the required f'_c.
3. If f_{c6} does not exceed the f'_{cr} required, repeat the trial batching in steps 1 and 2 until f_{c6} is equal or greater than f'_{cr} required.
4. If f_{c6} exceeds the f'_{cr} required, test the compressive strength of the 4 x 8 in. cylinders.
5. Compute the k_s for the mixture.
6. From this k_s value, the f'_{cr} for 4 x 8 in. cylinders can be computed from the f'_{cr} of 6 x 12 in. cylinders. For example, if the f'_{cr} is 8,000 psi for 6 x 12 in. cylinders and the k_s computed for that mix is 1.05, then f'_{cr} for 4 x 8 in. cylinders is 8,400 psi.

In order to implement 4 x 8 in. cylinders into quality assurance testing in this fashion, all parameters must remain constant. Concrete mixture proportions must not vary. If it is desired to change to concrete mixture, a new k_s value must be evaluated through the process previously described. The testing parameters must remain unchanged. The need to test 4 x 8 in. cylinders comes from the inability to test 6 x 12 in. cylinders based on the compression machine capacity. It would be optimal to confirm k_s on the same machine that will be used for quality assurance testing. However, if the machine used for quality assurance testing cannot test the 6 x 12 in. cylinders, k_s should be confirmed on a machine that has the capacity to test cylinders of such strength. Both machines should be well calibrated as to limit differences and errors due to using two compression machines. It would be good practice to reconfirm the k_s value at intervals as the concrete is supplied to the project. Since all standards and specifications are based on the compressive strength of 6 x 12 in. cylinders, this will be the base of the process previously described. All strengths computed from 4 x 8 in. cylinders will eventually be evaluated as equivalent 6 x 12 in. cylinder strengths.

Since testing of 4 x 8 in. cylinders comes from the need to test high-strength concrete, Section 5.3.2 ACI 318-02 divides the equations of f'_{cr} for concretes with design strengths less than or equal to 5,000 psi and greater than 5,000 psi, and the maximum practical design strength of concretes that can be tested on the average compression machine is 5,800 psi, it is recommended that 4 x 8 in. cylinders only be used when 6 x 12 in. cylinders cannot be tested due to limitations of testing capacity. Based on the similarities of the divisions of ACI 318-02 Section 5.3.2 and the maximum practical design strength of concretes that can be tested on the average compression machine, it is recommended that 4 x 8 in. cylinders only be used when f'_c is greater than 5,000 psi. If the capacity of the compression machine that will be used on a particular project is high enough to safely test 6 x 12 in. cylinders, then 4 x 8 in. cylinders should not be used.

Chapter 7

CONCLUSIONS AND RECOMMENDATIONS

The new trend of using high-strength concrete in construction has caused a need for the use of 4 x 8 in. cylinders for assurance testing. A controlling factor that affects the size of specimen that can be tested in a compression machine is the strength of the concrete on evaluation. Some testing machines are not able to produce the force needed to break high-strength 6 x 12 in. concrete cylinders. If 4 x 8 in. cylinders are to be used in quality assurance testing, the relationship between f_{c4} and f_{c6} needs to be understood in order to ensure that concrete with sufficient strength is provided. This was the primary objective of this research project: to find a correlation between f_{c4} and f_{c6}. The scope includes testing 6 different mixture proportions which varied in aggregate and strength range. A total of 716 test cylinders were tested. Age at testing and technician were evaluated as well to determine their effects on k_s. All test specimens were batched, mixed, cured, and tested under controlled laboratory conditions.

7.1 CONCLUSIONS

The literature review reveals prior research done to find a correlation between the strengths of the standard 6 x 12 in. cylinder to that of a smaller cylinder. Throughout this report, the parameter k_s was used to denote the ratio of 4 x 8 in. cylinder strength to 6 x 12 in. cylinder strength. It was found in the literature review that specimen size/shape, aggregate size/gradation, age at testing, and strength range were the main factors affecting k_s. Day (1994 a) performed a statistical analysis of over 8,000 compiled specimen strengths. He found that within the strength range of 2,900 psi and 14,500 psi, f_{c4} is expected to be 5% higher than f_{c6}. However, in the strength range of 2,900 psi to 8,700 psi, f_{c4} and f_{c6} can be assumed equal. The general trend that was found from the literature review was that k_s increases with increasing compressive strength. This was not the case for the results of this research project. It was found that each strength range had its own range of k_s values and that k_s decreased with increasing compressive strength ranges. Day states that “factors such as concrete type, aggregate type, cement content, water-cement ratio, presence of supplementary cementing materials, and type of vibration appear to have no significant effect on the correlation between f_{c6} and the strength from small cylinders. On the other hand, the above factors all influence the strength of the concrete, and the level of concrete strength does appear to have an effect on differences in measured strengths from different cylinder sizes" (Day 1994 a). This statement was also the conclusion of this research project. An analysis of variance (ANOVA) was performed on the data and based on a 99% confidence level, it was concluded that strength was the only factor that significantly affected k_s.

Day (1994 a) also found that there is no need to test more 4 x 8 in. cylinders than 6 x 12 in. cylinders due to the fact that "the coefficient (of variation) of 4 x 8 in. cylinders is equivalent to that of 6 x 12 in. cylinders over a broad range that encompasses normal, high, and very high-strength concrete" (Day 1994 a). Cook (1989) concluded that "a 10,000 psi designed concrete with a standard deviation of 800 psi has the same degree of control as 3,000 psi designed concrete with a standard deviation of 240 psi". Because of this, Cook (1989) made a suggestion that coefficient of variation be the standard of control for concrete testing instead of standard deviation. Suggestions made by Cook (1989) were later implemented in ACI 318-02 and ACI 363.2R-98 to account for the high variability of high-strength concrete. Kennedy et al. (1995) and Pistilli and Willems (1993) concluded that within-test variability increased with increasing compressive strength and that cylinder size did not affect within-test variability. This was also found to be true for this research project. Based on the test results in this project and a 99% confidence level, it was concluded that cylinder size was not significant in affecting the within-test variability through the ANOVA statistical analysis. The within-in test variability of the 8,000 psi batches was far greater than the 4,000 psi and 6,000 psi batches. An ANOVA analysis was performed to determine which factors have a significant effect on the variability. Based on a 99% confidence level, it was found that the variability was significantly affected only by the strength of the concrete.

7.2 RECOMMENDATIONS

Based on the results of this research and the implementation procedures suggested in the literature, recommendations can be given about how to use 4 x 8 in. cylinders in quality assurance testing. A suggested procedure to implement the use of 4 x 8 in. cylinders is as follows:

1. Perform a trial batch of sufficient size to be representative of typical production of the desired concrete mixture. Make three 4 x 8 in. and three 6 x 12 in. cylinders from the same batch of concrete.
2. Test the compressive strength of the 6 x 12 in. cylinders (f_{c6}). Confirm that f_{c6} meets the f'_{cr} required so that the concrete exceeds the required f'_c.
3. If f_{c6} does not exceed the f'_{cr} required, repeat the trial batching in steps 1 and 2 until f_{c6} is equal or greater than f'_{cr} required.
4. If f_{c6} exceeds the f'_{cr} required, test the compressive strength of the 4 x 8 in. cylinders.
5. Compute the k_s for the mixture.

6. From this k_s value the f'_{cr} for 4 x 8 in. cylinder can be computed from the f'_{cr} of 6 x 12 in. cylinders. For example, if the f'_{cr} is 8,000 psi for 6 x 12 in. cylinders and the k_s computed for that mix is 1.05, then f'_{cr} for 4 x 8 in. cylinders is 8,400 psi.

In order to implement 4 x 8 in. cylinders into quality assurance testing in this fashion, all parameters must remain unchanged. All strengths computed from 4 x 8 in. cylinders will eventually be evaluated as equivalent 6 x 12 in. cylinder strengths. It is recommended that 4 x 8 in. cylinders only be used when f'_c is greater than 5,000 psi and the compression machine used for testing is not sufficient to safely break 6 x 12 in. cylinders at f'_{cr}.

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APPENDIX A

TEST DATA COLLECTED DURING

COMPRESSIVE STRENGTH TESTING

Table A.1a: Compressive strengths of individual specimens (psi)

Sample ID	7 DAY STRENGTHS		28 DAY STRENGTHS	
	6 x 12	4 x 8	6 x 12	4 x 8
57-4-A-T1	4600	4450	5840	5720
57-4-A-T1	4580	4330	5820	5610
57-4-A-T1	4550	4280	5650	5600
57-4-A-T1	4530	4270	5650	5570
57-4-A-T1	4510	4220	5630	5200
57-4-A-T2	4740	4390	5800	5460
57-4-A-T2	4640	4310	5540	5430
57-4-A-T2	4600	4310	5460	5180
57-4-A-T2	4480	4240	5270	5070
57-4-A-T2		4230		5070
57-4-B-T1	3170	3350	4200	4550
57-4-B-T1	3060	3330	4110	4420
57-4-B-T1	3050	3270	4080	4400
57-4-B-T1	2970	3250	4060	4230
57-4-B-T1	2890	3240	3970	4120
57-4-B-T2	3220	3490	4340	4550
57-4-B-T2	3100	3310	4330	4390
57-4-B-T2	3040	3290	4250	4340
57-4-B-T2	2960	3200	4200	4300
57-4-B-T2	2900	2850	3960	4190
57-4-C-T1	2730	3140	3790	4240
57-4-C-T1	2640	3000	3780	4070
57-4-C-T1	2570	2890	3720	4070
57-4-C-T1	2510	2890	3660	3920
57-4-C-T1	2410	2880	3230	3700
57-4-C-T2	2820	3160	3880	4300
57-4-C-T2	2770	3120	3640	4030
57-4-C-T2	2690	3090	3340	3980
57-4-C-T2	2670	3040	3310	3960
57-4-C-T2	2280	3020	3140	3840

Table A.2b: Compressive strengths of individual specimens (psi)

Sample ID	7 DAY STRENGTHS		28 DAY STRENGTHS	
	6 x 12	**4 x 8**	**6 x 12**	**4 x 8**
67-4-A-T1	3280	3350	4000	4150
67-4-A-T1	3260	3190	3600	4140
67-4-A-T1	3230	3180	3540	4010
67-4-A-T1	3220	3160	3440	4000
67-4-A-T1	3010	3090	3420	3960
67-4-A-T2	3010	3380	3750	4150
67-4-A-T2	2920	3310	3730	4060
67-4-A-T2	2920	3250	3720	4010
67-4-A-T2	2880	3110	3470	3990
67-4-A-T2	2800	3020	3190	3820
67-4-B-T1	2780	2930	3450	3800
67-4-B-T1	2770	2930	3430	3560
67-4-B-T1	2760	2930	3370	3550
67-4-B-T1	2670	2840	3240	3410
67-4-B-T1	2590	2720	3030	3400
67-4-B-T2	2680	3090	3390	3560
67-4-B-T2	2610	2870	3320	3500
67-4-B-T2	2580	2820	3310	3460
67-4-B-T2	2470	2820	3280	3440
67-4-B-T2	2450	2790	3210	3040
67-4-C-T1	2520	3010	3380	3810
67-4-C-T1	2490	2990	3280	3700
67-4-C-T1	2460	2970	3250	3590
67-4-C-T1	2430	2950	3240	3470
67-4-C-T1	2410	2890	3150	3450
67-4-C-T2	2790	3180	3350	3660
67-4-C-T2	2720	3170	3190	3560
67-4-C-T2	2660	3130	3160	3460
67-4-C-T2	2620	3120	3110	3400
67-4-C-T2	2460	3000	3020	3370

Table A.3c: Compressive strengths of individual specimens (psi)

Sample ID	7 DAY STRENGTHS		28 DAY STRENGTHS	
	6 x 12	**4 x 8**	**6 x 12**	**4 x 8**
57-6-A-T1	5470	4560	6240	6050
57-6-A-T1	5380	4510	6220	5830
57-6-A-T1	5280	4480	5940	5650
57-6-A-T1	4920	4420	5800	5470
57-6-A-T1	4780	4290	5710	5410
57-6-A-T2	5410	4490	6600	5920
57-6-A-T2	5100	4490	6450	5560
57-6-A-T2	4740	4470	6350	5520
57-6-A-T2	4550	4430	5980	5510
57-6-A-T2	4450	4130	5870	5090
57-6-B-T1	5440	5050	6360	6390
57-6-B-T1	5430	4940	6230	6210
57-6-B-T1	5360	4920	6070	6200
57-6-B-T1	5250	4870	5980	6070
57-6-B-T1	5060	4820	5860	5990
57-6-B-T2	5640	4980	6680	6060
57-6-B-T2	5310	4860	6660	6000
57-6-B-T2	5190	4780	6550	5940
57-6-B-T2	5170	4770	6260	5760
57-6-B-T2	5130	4650	5980	5750
57-6-C-T1	5310	5120	6280	6560
57-6-C-T1	5050	5100	6260	6520
57-6-C-T1	4890	5050	6210	6110
57-6-C-T1	4850	4570	6100	6050
57-6-C-T1	4810	4460	5910	5960
57-6-C-T2	5220	4870	6490	6030
57-6-C-T2	5190	4720	6380	6030
57-6-C-T2	4920	4680	6360	5970
57-6-C-T2	4800	3950	5940	5890
57-6-C-T2	4730		5830	5700

Table A.4d: Compressive strengths of individual specimens (psi)

Sample ID	7 DAY STRENGTHS		28 DAY STRENGTHS	
	6 x 12	**4 x 8**	**6 x 12**	**4 x 8**
67-6-A-T1	5710	5860	7230	7090
67-6-A-T1	5660	5810	6940	6930
67-6-A-T1	5530	5810	6730	6890
67-6-A-T1	5490	5540	6730	6850
67-6-A-T1	5450	5520	6710	6790
67-6-A-T2	6110	5580	7120	6870
67-6-A-T2	5670	5510	7110	6770
67-6-A-T2	5650	5440	7040	6630
67-6-A-T2	5480	5020	6630	6570
67-6-A-T2	5290	4500	6500	6510
67-6-B-T1	5620	5480	7050	6800
67-6-B-T1	5540	5470	6950	6670
67-6-B-T1	5510	5370	6580	6620
67-6-B-T1	5250	5320	6380	6580
67-6-B-T1	5140	5060	6370	6570
67-6-B-T2	5350	6050	6350	7060
67-6-B-T2	5150	5510	6340	6810
67-6-B-T2	5140	5370	6140	6660
67-6-B-T2	4950	5250	6030	6650
67-6-B-T2		5070	6020	6000
67-6-C-T1	5890	5690	7180	7100
67-6-C-T1	5800	5640	7030	6980
67-6-C-T1	5580	5510	7020	6920
67-6-C-T1	5510	5320	6870	6800
67-6-C-T1	5170	5300	6720	6760
67-6-C-T2	5860	5530	6820	7000
67-6-C-T2	5800	5530	6680	6920
67-6-C-T2	5690	5480	6500	6860
67-6-C-T2	5340	5430	6490	6800
67-6-C-T2	5290	5220	6450	6680

Table A.5e: Compressive strengths of individual specimens (psi)

Sample ID	7 DAY STRENGTHS		28 DAY STRENGTHS	
	6 x 12	4 x 8	6 x 12	4 x 8
57-8-A-T1	11200	9700	13370	10970
57-8-A-T1	10090	9290	12810	10780
57-8-A-T1	9890	9280	12620	10030
57-8-A-T1	9860	9010	12230	9930
57-8-A-T1	9550	8950	11120	9600
57-8-A-T2	10780	9510	12180	12280
57-8-A-T2	10550	8830	11820	11110
57-8-A-T2	9710	8440	11010	10970
57-8-A-T2	9590	8270	10670	10440
57-8-A-T2	9210	8190	10110	9870
57-8-B-T1	10510	8280	11820	9570
57-8-B-T1	10480	8180	9970	9150
57-8-B-T1	10460	8160	9770	8010
57-8-B-T1	10400	7110	9670	7800
57-8-B-T1	8780	6700	9410	7760
57-8-B-T2	10670	8550	11770	8260
57-8-B-T2	9890	8080	11570	7750
57-8-B-T2	8920	8030	11410	7560
57-8-B-T2	8770	7650	9850	7560
57-8-B-T2	8390	6470	9440	7400
57-8-C-T1	9080	7140	9830	8180
57-8-C-T1	8890	7060	7890	8150
57-8-C-T1	8720	5990	7780	7920
57-8-C-T1	8670	5930	7510	6760
57-8-C-T1	6950	5550	7440	6580
57-8-C-T2	8670	7150	9320	8100
57-8-C-T2	8570	7030	8480	7920
57-8-C-T2	8390	6140	8180	7860
57-8-C-T2	7050	5980	7900	7150
57-8-C-T2	6890	5930	7250	6660

Table A.6f: Compressive strengths of individual specimens (psi)

Sample ID	7 DAY STRENGTHS		28 DAY STRENGTHS	
	6 x 12	**4 x 8**	**6 x 12**	**4 x 8**
67-8-A-T1	9240	8070	10330	8610
67-8-A-T1	8920	7890	10260	7590
67-8-A-T1	8190	7600	9740	7430
67-8-A-T1	7990	7490	9070	7300
67-8-A-T1	7760	6480	8950	7170
67-8-A-T2	9220	7380	10920	8350
67-8-A-T2	9200	7380	10430	8250
67-8-A-T2	8950	7130	9860	7970
67-8-A-T2	8310	7030	9420	7470
67-8-A-T2	8100	6570	9230	7050
67-8-B-T1	8640	7910	9910	8200
67-8-B-T1	8430	7670	9860	7980
67-8-B-T1	8210	7390	9680	7780
67-8-B-T1	7920	7090	9600	7010
67-8-B-T1	7790	6770	9480	7000
67-8-B-T2	8920	8960	9830	9560
67-8-B-T2	8830	8570	9650	9230
67-8-B-T2	8800	8540	9600	9210
67-8-B-T2	7920	8130	9270	8920
67-8-B-T2	7590	6550	9210	8670
67-8-C-T1	9500	7160	10650	7230
67-8-C-T1	9240	7010	10480	6650
67-8-C-T1	9190	6630	8390	6350
67-8-C-T1	7420	6260	8300	6160
67-8-C-T1	7350	5760	7990	6020
67-8-C-T2	9100	7060	10170	6170
67-8-C-T2	9060	6440	8640	6170
67-8-C-T2	8890	5950	8510	5650
67-8-C-T2	7310	5950	8430	5240
67-8-C-T2	7160	5750	8330	5080