The Application of Self-Consolidating Concrete in Full-Scale Drilled Shafts

by

Phillip Alain Gallet

A thesis submitted to the Graduate Faculty of
Auburn University
in partial fulfillment of the
requirements for the Degree of
Master of Science

Auburn, Alabama
December 13, 2010

Keywords: Self-consolidating concrete, drilled shaft,
tremie placed concrete,

Approved by

Anton K. Schindler, Chair, Associate Professor of Civil Engineering
Dan A. Brown, President of Dan Brown and Associates, PLLC
David J. Elton, Professor of Civil Engineering

Abstract

Drilled shafts are being built larger and deeper, and this has increased the challenge in creating quality drilled shafts. Due to longer placement times and more required reinforcement, problems have been discovered with the use of ordinary drilled shaft concrete in these more challenging drilled shafts. Self-consolidating concrete (SCC) is a solution to some of the concrete problems experienced. This paper summarizes two field projects that were conducted to examine the feasibility of SCC in drilled shafts. The first field project compared ordinary drilled shaft concrete to two types of SCC. A different concrete mixture in each, three shafts were constructed, exhumed, and cored. This project concluded that SCC provides a drilled shaft with a much better cover region. The second project was conducted to analyze the first ever use of SCC in production drilled shafts in the state of Alabama. This project documented problems that occurred on the site, tested the variability of the concrete arriving to the site, and analyzed the flow of the concrete through the reinforcement cage. It is concluded that high-quality drilled shafts can be created by using SCC for challenging drilled shafts.

Acknowledgments

I would first of all like to thank my advisor, Dr. Schindler, for the opportunity to conduct this research under his guidance, wisdom, and insight. A special thanks also to Dr. Brown for the opportunity to work and learn under his guidance. Thanks to Bo Canning of Kirkpatrick Concrete and Dan Green of National Cement for their help in Scottsboro. Thanks to Steve Beeler, Evelyn Pao, and the ALDOT technicians for helping me acquire data from all the tests. Thanks to my many friends, notably David Graham, David Self, and Kristin Adamson, who tirelessly reviewed some of these very chapters. Finally, a special thanks to my parents, Alain and Sandy Gallet, who have always given their love and support; and who give me great role models to live up to.

Table of Contents

List of Tables

List of Figures

Chapter 1

Introduction

1.1   Problem Statement

Drilled shafts are deep foundation structures used to support and transfer axial and lateral loads induced by structures. Using a drilled hole as the formwork, these structures are filled with concrete and form columns to support and transfer these loads (McCarthy 2002). A popular deep foundation type, drilled shafts have been increasing in size due to a growth in construction capability. Since many bridges are built over water and in areas with shallow ground water tables, it is popular to install drilled shafts through the water using the tremie method. This method uses a hollow steel pipe, referred to as a tremie, to transport the concrete, by gravity or pump truck, from the surface to a location below water. No vibration can be used to consolidate the concrete as it would be impractical to conduct and can cause defects by allowing slurry, water, or soil to mix with the concrete (O'Neil and Reese 1999). A figure of tremie placement is presented in Figure 1.1.

The concrete that has been used for drilled shaft construction must have the following characteristics (O'Neil and Reese 1999):

- Excellent workability,

- Self compaction,

- Resistance to segregation,

- Resistance to mixing with the water,

- Controlled setting,

1

- Good durability,

- Appropriate strength and stiffness, and

- Low heat of hydration.
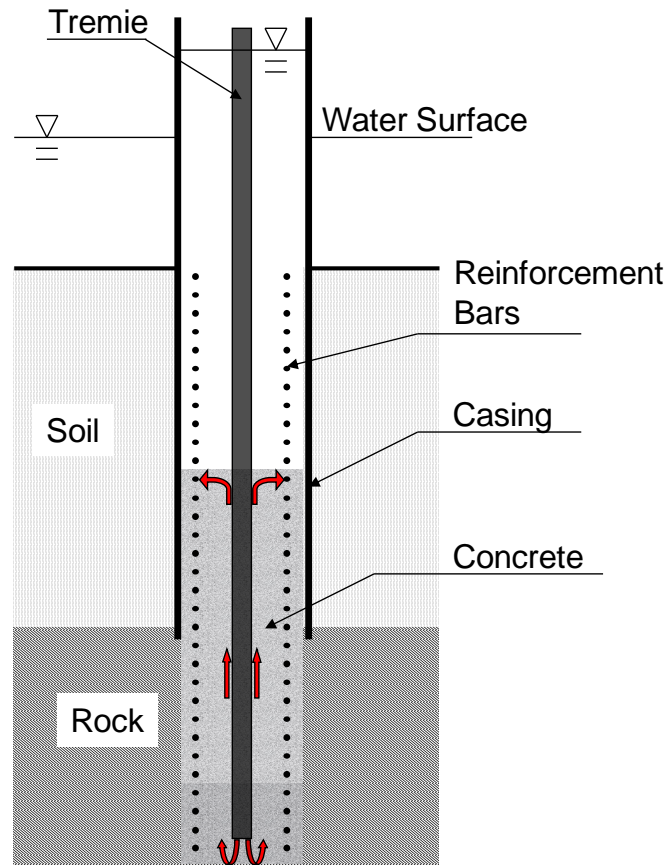
Figure 1.1: Schematic of tremie placed concrete

These large drilled shafts have congested reinforcement cages to enable the shaft to resist lateral loads such as wind, seismic, and impact forces. Problems occur when these confined cages prevent the concrete from easily flowing into the cover region and consolidating (Brown and Schindler 2007). An example of this problem is shown in Figure 1.2, where the concrete had enough workability to have a shovel pushed into it, but lacked the workability to flow through the reinforcement. Another example is shown in Figure 1.3, where the reinforcement design required two

reinforcement cages and this extra congestion prevented the concrete from

encapsulating the reinforcement. An example of poor consolidation in the cover region

is shown in Figure 1.3. In this figure, the concrete was capable of flowing through the

reinforcement, but lacked sufficient workability to consolidate under its own weight,

creating a very permeable cover region.

Problems will also occur when the concrete is not able to stay workable for the

duration of the concrete placement. Concrete that arrives at the site workable may not

have the ability to stay workable for the duration of the concrete placement. If the

concrete lacks this ability, difficulties will occur in maintaining the tremie flow. Due to

these difficulties the tremie may be lifted to improve the concrete flow. A picture of

shafts that had this problem during construction is shown in Figure 1.5.

Figure 1.2: Drilled shaft concrete without sufficient workability to flow to into the cover

region of the shaft (Brown and Schindler 2007)

Figure 1.3: Heavy congestion in the reinforcement cage prevents concrete from

encapsulating the reinforcement bars (photo by Dan Brown)

Figure 1.4: Surface of a drilled shaft with many voids caused by poor consolidation in the

cover region of the shaft (Brown 2004)

Figure 1.5: Shaft defects due to loss of workability (Photo by Dan Brown)

Recently, much research has been conducted on applying high-performance concrete (HPC) to various civil engineering structures. One type of HPC is self-consolidating concrete (SCC). SCC is defined as a "highly flowable, nonsegregating concrete that can spread into place, fill the formwork, and encapsulate the reinforcement without any mechanical consolidation" (ACI 237 2007).

SCC has shown, in previous research and some full-scale projects, that it is a viable replacement for ordinary drilled shaft concrete in demanding placement applications. However, SCC is still a relatively new product and is not routinely used in most states. Also, there is very limited documentation from projects where SCC was

used in drilled shafts. Documentation of construction methods and more experience with this high-performance concrete is required before SCC will be widely accepted for use.

A project conducted by the South Carolina Department of Transportation (SCDOT) studied integrity results on more than 400 shafts on over 42 projects. This study found that the majority of anomalies in drilled shafts were due to concrete irregularities and occurred near the top and bottom of the drilled shafts. Camp et al. (2007) believes that these anomalies may have been because of "partial segregation, probably as a result of placement through water". This study concluded that "the majority of anomalies are attributable to concrete issues…based on cores that we have observed, most anomalies are a result of segregation due to placement in water or bleeding effects." (Camp et al. 2007) Most importantly, in regard to this paper, the SCDOT study stated, "Concrete problems should be avoided through the use of appropriate mixes that are resistant to segregation yet have good workability (e.g. self-consolidating concrete) and the use of appropriate placement methods." (Camp et al. 2007)

In addition, the mechanics of concrete flow from a tremie pipe into a drilled shaft are unknown. O'Neil and Reese (1999) state that the first concrete placed into the shaft will be displaced to the surface of the shaft by the end of the concrete placement. This would mean that concrete at the surface of the shaft would be the same throughout the entire pour, and thus would be the only concrete to weaken due to mixing with the water or slurry in the shaft. However, Brown and Schindler (2005) concluded that this concrete flow does not occur and more research should be conducted. By understanding how concrete flows from a tremie pipe, one could design a concrete that would lessen the chance of mixing with slurry or water, and be able to flow into the cover regions of the shaft better.

The escape of excess water from the concrete, known as bleeding, can cause problems in shafts where this excess water is prevented from escaping, such as in cased shafts or shafts socketed into solid rock. A conventional bleed test is usually conducted on the concrete mixture to determine the amount of bleed water a concrete mixture would dispel. SCC is known for having a lesser potential to bleed (Khayat 1999), but the amount a concrete will bleed in a drilled shaft under the pressure induced by the weight of concrete and water has only recently been attempted to be tested.

1.2   Research Objectives

Limited data are available on the use of SCC in full-scale drilled shaft applications. The primary objective of this research was to assess the feasibility of using SCC for full-scale tremie placed drilled shafts. The expectation was to provide results that further the research and usage of SCC in drilled shafts.

In addition, research was conducted to better understand the mechanics of tremied concrete flow into drilled shafts, and to assess and refine a test to determine the amount of concrete bleed water induced under pressure.

Finally, an evaluation was conducted on the first use of SCC in drilled shafts in the state of Alabama, to discover any issues that may affect the use of SCC in the full-scale production of drilled shafts.

1.3   Research Methodology

To compare SCC to ordinary drilled shaft concrete, three 6-ft. diameter test shafts were created with a different drilled shaft concrete mixture in each shaft:

- One with a ordinary drilled shaft concrete (ODSC) mixture used in on an ALDOT project in North Alabama,

- One with SCC specifically proportioned for drilled shaft applications (SCC), and

- One with an experimental SCC that uses limestone powder (SCC-LP).

These shafts were exhumed and examined to compare the difference between ordinary drilled shaft concrete and SCC in full-scale shafts. In addition, colored mortar cubes were installed into each of the shafts to assess how the concrete flows out of the tremie pipe within the drilled shafts, and any differences between the flow of ordinary drilled shaft concrete and that of SCC were reported.

The second part of this research was to document the first ever placement of SCC in production shafts in the state of Alabama. These shafts were installed for a bridge across the Tennessee River in Scottsboro, Alabama. This research included the following:

- Documenting the concrete placement,

- Testing the variability of the concrete arriving to the jobsite,

- Directly assessing the concrete's ability to flow through the reinforcement by measuring the height of the concrete in the center and cover region of the shaft during concrete placement, and

- Measuring the temperature development due to hydration of the concrete.

In addition, research was conducted to develop and assess a prototype of a pressurized bleed test to determine concrete's bleeding under pressure, such as concrete that is in a drilled shaft.

1.4   Report Outline

Following the introduction chapter, a brief overview of drilled shafts, SCC, and past projects where SCC has been used in drilled shafts is provided in Chapter 2. This

includes a background of drilled shafts, current construction practice, and problems with drilled shaft concrete. Additionally, SCC is introduced and its development is summarized.

The research conducted to compare and evaluate conventional concrete and two forms of SCC in three six-foot diameter drilled shafts is discussed in Chapter 3. This included comparing the fresh concrete properties such as total air-content, unit weight, temperature, flow, segregation, bleed potential, and set times. The hardened concrete properties such as compressive strength, modulus of elasticity, permeability, and shrinkage, are also compared. Included is an evaluation of the concrete flow out of a tremie pipe into the shafts.

Research conducted during the installation of the first production drilled shafts using SCC in the state of Alabama is discussed in Chapter 4. This chapter includes the following:

- Documentation of the concrete placement,

- A study of the concrete's ability to flow through the reinforcement,

- A study of the variability of the concrete flow and stability arriving to the project site,

- A study of concrete bleed water under pressure, and

- An evaluation of the temperature due to hydration of the concrete within the drilled shafts.

Finally, an overview of the research conducted, conclusions of the projects, and recommendations developed from the research are presented in Chapter 5.

Chapter 2

Literature Review

A background to drilled shafts and the concrete designed to create them is provided in this chapter. This includes the following:

- A short history of drilled shaft construction,

- A review of drilled shaft construction methods,

- A review of drilled shaft concrete,

- A review of current drilled shaft integrity tests,

- An explanation of difficulties experienced with drilled shaft concrete placement,

- An explanation of SCC and its use in drilled shaft construction,

- A review of past projects where SCC has been used for drilled shaft construction,

- A review of the mechanism of concrete flow from a tremie, and

- A summary of the development of a pressurized bleed test.

2.1   Introduction to Drilled Shafts

A drilled shaft is a column of concrete that uses an excavated hole as concrete formwork. Reinforced or unreinforced, once cured these shafts use side friction and tip resistance to support an applied load (such as a building or a bridge). There are many

names for this deep foundation technique, such as: drilled caissons, drilled piers, cast-in-drilled-hole piles, and bored holes. (O'Neil and Reese 1999)

Drilled shafts were first introduced to the United States in cities such as Chicago and Detroit. In the late 1800's, these cities were in a need for larger buildings to accommodate the increased population and economic growth. Built higher to take up a limited amount of city ground space, these buildings were putting greater stresses on the foundation beneath. For example, in Chicago, relatively thick layers of soft to medium stiff clays exist over a deep hardpan material. To construct these buildings, workers hand dug excavations through the weak soil layers to the hardpan depth and used wood lagging or metal sheets to reinforce the sides of the excavated holes. These excavations where then filled with concrete and used to support structures. (O'Neil and Reese 1999)

In San Antonio, Texas, drilled shafts were used to bypass stiff shallow expansive material to support the structure on deeper non-expansive layers. Techniques for drilling through multiple soil and geological conditions were modified from the oil industry. These techniques include installing casings and using drilling mud to keep the holes from collapsing during excavation. (O'Neil and Reese 1999)

In other areas of the world drilled shafts were employed in a different way. O'Neil and Reese (1999) explain that "Large-diameter, straight shafts founded entirely in clay and deriving most of their support from side resistance came into common usage in Britain."

Research was conducted throughout the middle to late 1900's using full scale load tests and comparing drilled shafts to other deep foundation techniques and refining the construction process. It was not until 1977 that a drilled shaft design manual was published. This design manual was published by the Federal Highway Administration

(FHWA) and led to the latest design manual published in 1999 written by O'Neil and Reese. (O'Neil and Reese 1999)

2.2   Current Installation Practice

There are many ways of constructing a drilled shaft. These ways differ in excavation and placement methods, to the composition and properties of the concrete. This section will review the general practices and methods used to install drilled shafts.

2.2.1   Constructing the Shaft

In general, there are three methods to install a drilled shaft: dry method, cased method, and wet method (O'Neil and Reese 1999).

2.2.1.1 Dry Method

The first method can be described as the dry hole method. In this method the excavation must be able to stay open, without caving, during the drilling operation and throughout the concrete placement. In addition, ground water must not be able to penetrate into the excavation. To construct a dry shaft, first the hole is augered to its required elevation. Second, a steel reinforcement cage (if necessary) is put in place. Finally, the concrete is placed into the excavation from the surface. If the concrete comes in contact with the reinforcement of the side of the shaft before hitting the bottom of the shaft, segregation can occur. To prevent this occurrence, a drop chute can be utilized or the last chute from the concrete truck can be inverted to direct the concrete flow down the center of the shaft. (O'Neil and Reese 1999) A figure of the construction of a dry shaft using a drop chute is presented in Figure 2.1.
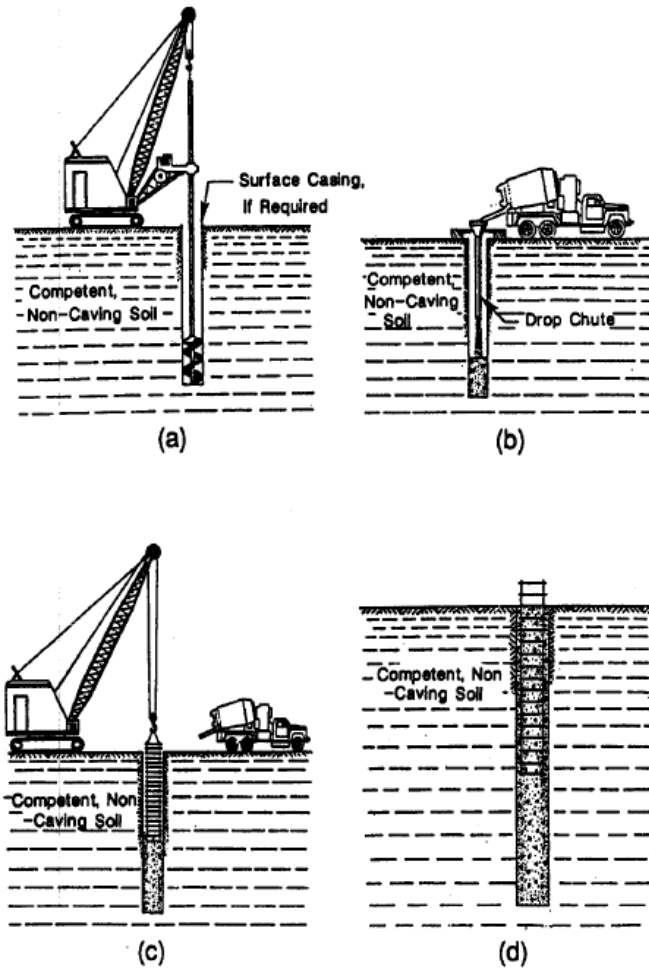
Figure 2.1: Dry method of construction: (a) initiating drilling, (b) starting concrete placement, (c) placing rebar cage, (d) completed shaft (O'Neil and Reese 1999)

2.2.1.2 Cased Method

This method is commonly used in conditions where the excavation will remain temporarily open. This method is accomplished so that the hole can be drilled and casing put in place before the excavation caves. In a shaft location where a layer of caving material is located in the subsurface, the excavation can be augered to the elevation of the caving soil. At this point, drilling slurry can be added to the shaft so that the drilling can continue through the caving soil. This drilling slurry is usually made from

the mixture of bentonite and water, or more recently a mixture of a polymer powder and water. The polymer slurry has a higher viscosity and unit weight than water and therefore induces a positive pressure to the sides of the drilled hole, thus avoiding the penetration of ground water and/or caving of the surrounding soil. The pressure applied by the slurry is diagramed in Figure 2.2. The bentonite slurry works in a similar manner, but the positive pressure forms a layer of clay on the outside of the shaft, known as a filter cake or mudcake (O'Neil and Reese 1999).
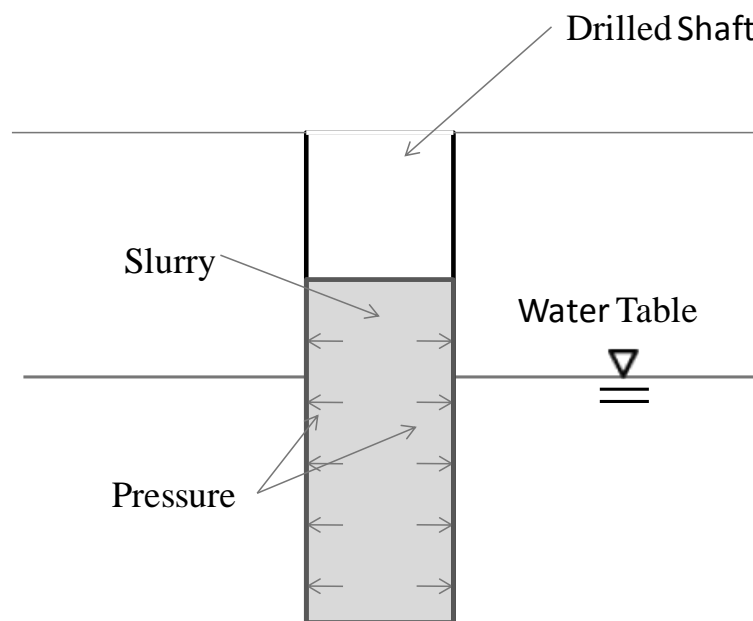
Figure 2.2: Pressure on outside of drilled shaft due to drilling slurry

When the caving layer is fully penetrated, a casing is lowered into the excavation and sealed into the firm soil beneath the caving layer. The slurry can then be removed and the excavation can be continued with a smaller auger to the required tip elevation. The concrete is then placed into the shaft from the surface. Once the concrete elevation is sufficiently above the caving soil elevation, the casing can be removed. Finally, the rest of the concrete can be placed into the excavation. O'Neil and Reese (1999) state

14

that it is common practice to either remove the steel casing immediately after construction, or leave the steel in place with the final shaft (permanent casing).

Another way to construct a cased holed is to vibrate the casing into place, so as to seal off the caving soil layer. Once the casing is at the desired elevation, an auger with a smaller diameter than the casing, can be used to excavate the soil within the casing. Once excavated, this hole can be filled like a shaft using the dry method. Once the concrete has reached an elevation sufficiently above the caving layer, the casing can be removed by vibration.

Care must be taken in the removal the temporary casing to make sure the concrete does not bind together and form an arch. This arching will cause the concrete to rise up with the steel casing. This phenomenon, known as necking, will cause voids to form. (O'Neil and Reese 1999)

2.2.1.3 Wet Method

The last drilling method can be described as the wet method, also known as the slurry displacement method (ACI 336 2001). The shaft is excavated either using drilling slurry or a casing to prevent the soil from caving. When installing a shaft below the water table, ACI 336 (2001) suggests that the slurry must stay at least 5-ft above the groundwater level. Once the shaft has been excavated to the desired elevation, the reinforcement cage is then lowered through the slurry to the bottom of the shaft. Concrete can be transported through a fluid, such as slurry or water, to the bottom of a drilled using with either gravity or a pump truck.

    o   Gravity fed tremie method

After preparing the hole, concrete can be transported to the bottom of the hole using a hollow steel pipe, known as a tremie pipe. This method, known as the gravity fed tremie method, uses gravity to force the concrete down the tremie pipe. Care must be taken to ensure the concrete within the tremie pipe does not come in contact with the

15

slurry. This can be done by using a foam plug, known as a pig (ACI 336 2001), to

separate the concrete from the fluid within the shaft at the beginning of the placement or

by putting a temporary shield on the bottom of the tremie that the concrete will force out

once placement has begun. This tremie is placed on or near the bottom of the shaft. It

is popular to slice the tremie tip so that the tremie can be set on the bottom surface of

the shaft and allow the concrete to flow. This helps because the tremie will stay in one

place without much horizontal movement. A picture of this slice is shown in Figure 2.3.

Once the tip of the tremie is fully embedded in the concrete it is required to stay

embedded 10 ft for the entire concrete placement operation (ACI 336 2001).

Figure 2.3: Cut groove at the bottom of the tremie

o   Pump method

A pump truck can also be used to pump the concrete through a pump line to the

bottom of the hole. This method, know as the pump method, uses surges of pump

pressure in a closed system to force the concrete to the bottom of the shaft. In many

cases the pump line is attached to a tremie, but in some cases the pump line itself is

used to place the concrete. Care must be taken to separate the concrete within the

pump line from the slurry within the shaft. Either a foam plug needs to be placed into the

pump line so that the concrete pressure can force the plug through the tremie, or a temporary seal must be placed on the tip of the pump line. (O'Neil and Reese 1999; ACI 336 2001)

Since this method uses a closed system, special care needs to be taken in the concrete design and pumping set up. Since the concrete within the pump line moves faster than the pump output, the concrete can be pulled apart, causing segregation (Yoa and Bittner 2007).

2.2.2   Designing the Concrete for Drilled Shaft Construction

O'Neil and Reese (1999) state that for each drilled shaft the concrete design and placement method will be unique. In the simplest case of dry hole construction, the concrete free falls to the bottom of the excavated shaft. The concrete is then forced by its own weight and fluidity to spread through the reinforcement cage.

In the most complicated case, wet or cased hole construction, the concrete is expected to do much more. The concrete must be designed so that the mixture can be fed through a tremie to the bottom of the excavated hole. Then the concrete must flow to the outsides of the excavated shaft under a force less that its own weight due to buoyancy (Yao 2007). In the presence of slurry or water, the concrete is expected to flow to the outside of the shaft, through the reinforcement cage, without the use of vibration. Excess vibration will cause mixing between the concrete and slurry, sand, ground water, soil, or any other debris trapped in the hole. (O'Neil and Reese 1999) The concrete is then expected to displace the drilling slurry (or water) within the excavation without segregating. Once installed, the final hardened properties of the concrete mixture must meet the strength and durability requirements stated in the specifications. (O'Neil and Reese 1999)

To perform this above mentioned feat, a special type of concrete must be designed. O'Neil and Reese (1999) describe that this drilled shaft concrete must have the following characteristics:

- *"Excellent workability"*: must be able to flow through the tremie and flow through the reinforcement cage to completely cover the reinforcement,
- "Self-weight compaction": must be able to consolidate without the use of external vibration,
- "Resistance to segregation": must exhibit a cohesion in order to resist segregation,
- "Resistance to leaching": must be resistant to mixing with the groundwater or drilling slurry,
- "Controlled setting": must retain flow throughout the concrete placement,
- "Good durability": must be able to resist chemical attach from the soil or groundwater,
- "Appropriate strength and stiffness": must have final hardened properties that meet the engineers specifications , and
- "Low heat of hydration for large volumes of concrete": excess temperatures caused by the heat of hydration can produce cracking.

In its simplest form, concrete is made by mixing water with three different materials: portland cement, fine aggregate, and coarse aggregate. Portland cement is a man made product created by heating up quarried limestone and clay (or shale) to temperatures of 2550 to 2900°F in a kiln. This heated mixture is rapidly cooled to form clinker. The clinker and gypsum is then ground into a powder to create portland cement. Other products with cementing characteristics (such as fly ash and slag cement) are sometimes added to the portland cement to make a cementitious mixture. These

cement-like materials are known as supplementary cementing materials (SCMs). The combination of water and the cementitious mixture (SCMs and portland cement) make what is known as the cement paste. (Mindess et al. 2003)

The fine aggregate is defined as material that will mostly pass through a No. 4 sieve. Coarse aggregate is defined as the material that is mostly retained on a No. 4 sieve. The size and gradation of the coarse aggregate is usually dependent on the purpose of the concrete. The general rule is that the largest aggregate size should be used for its given application. (Mindess et al. 2003)

Any other material added to the concrete mixture, other than fibers, is known as an admixture. These come in two different categories: mineral and chemical admixtures. Mineral admixtures, such as slag cement and fly ash, are not explained in this paper since these admixtures are popular in all types of concretes. (Mindess et al. 2003) Chemical admixtures, however, are described in greater detail later in this chapter.

In general, the strength of the concrete is dependent on the water-to-cementitious materials ratio (w/cm ratio). The lower this ratio the stronger the concrete will be. Conventional concrete has w/cm ratios of 0.35 to 0.45. High-strength concrete can have w/cm ratios below 0.35 with the use of fly ash and water reducing admixtures. (Mindess et al. 2003)

To achieve the workability required, O'Neil and Reese (1999) recommend a high w/cm ratio of 0.5 to 0.6. However, this ratio can be lowered to 0.45 or less if water-reducing admixtures are included into the mixture. This admixture is described in more detail later in this chapter.

In general, there are three ways to control the workability of the fresh concrete (Mindess et al. 2003):

- Change the coarse aggregate shape – Use smooth aggregate particles such as river gravel to allow the aggregates to move around easier,

- Change the amount of fine aggregate – the fine aggregates can act like "ball bearings" allowing the coarse aggregate to move around easier, and

- Add a chemical admixture – a water-reducing admixture can be added to the mixture to create the impression of more water in the mixture.

After workability, the next characteristic that is required from drilled shaft concrete is stability. Concrete stability is defined as the ability of the concrete to resist segregation of the cement paste from the aggregates (ASTM C 1611). The concrete must be able to flow, but concrete is a heterogeneous mixture made of materials with different specific gravities. Too much workability can cause the aggregate particles to settle from the mixture, a form of segregation. To control the stability, care needs to be taken in the proportioning and mixing process (Mindess et al. 2003). Another way to control the stability of the concrete is to add a Viscosity Modifying Admixture (VMA) (Bury and Christianson 2003). This admixture is discussed in more detail later in this chapter.

Besides the concern about setting time, high heat of hydration is a potential concern for drilled shaft concrete. Shafts larger than about 5-ft diameter have characteristics of mass concrete in which the heat of hydration can feed on itself and generate large temperatures within the shaft. Recent measurements in Florida (Mullins 2006) have shown temperatures as high as 180 °F. Concrete members made with plain portland cement that reach temperatures above 158 °F may exhibit delayed ettringite formation (DEF) (Taylor et al. 2001). DEF causes internal expansion in the cement paste and initially results in microcracking that in some instances may progress to severe cracking in the concrete. The use of sufficient amounts of fly ash or slag cement will help mitigate DEF, and temperatures up to about 178 °F can be tolerated without significant concerns (Brown and Schindler 2007). Guidelines for sufficient amounts of

SCMs to mitigate against DEF include at least 25% Class F fly ash, at least 35% Class

C fly ash, or at least 35% slag cement.

2.2.3   Concrete Field Testing

Many tests have been developed to test the quality of the fresh concrete. In

most cases, the slump of the concrete (ASTM C 143) is an important measure of the

concrete's workability in the field. The requirements stated by ACI 336 (2001) are

presented in Table 2.1.

Table 2.1: Concrete slump requirements during placement (ACI 336 2001)

Slump		Drill method
in.	mm	
4 to 6	100 to 150	Dry, uncased, or permanent casing
6 to 8	150 to 200	Temporary casing
7 to 9	180 to 230	Slurry displacement

2.2.4   Assessment of Completed Shaft Integrity

In order to make sure the quality of the constructed drilled shaft meets the

specified requirements, quality control and quality assurance procedures are conducted

throughout the construction process. It is difficult to visually inspect how the concrete

placement occurs, especially in "wet" holes where the concrete is placed below the

slurry or water surface. Many different methods have been developed in order to detect

anomalies within the completed drilled shaft. These methods can be broken up into two

different categories: destructive testing and non-destructive testing.

2.2.4.1 Destructive Testing

o   Excavation for visual inspection

This test method is conducted for the inspection of the shallow anomalies that

may have occurred on the outside of the reinforcement cage. After the completion of the

shaft, the soil surrounding the drilled shaft is removed to visually examine the quality of the concrete on the outside surface of the shaft. (O'Neil 1991)

        o   Drilling or coring

This test method is conducted by coring through the completed drilled shaft to visually inspect the cores for any anomalies. These cores can also be cut to standard sizes to test the hardened concrete properties of the in-place concrete. (O'Neil 1991)

        o   Driving completed shaft

O'Neil (1991) describes this method, stating that this method is conducted by driving the completed drilled shaft and taking force and velocity measurements at the shaft head. A defect can then be identified using stress wave theory.

2.2.4.2 Non-Destructive Testing

Non-destructive testing (NDT) is performed on drilled shafts to determine the integrity of the in-place concrete without disrupting the capacity of the shaft. Most NDT methods use hollow access tubes that are attached to the reinforcement cage to conduct the testing. Therefore, these tubes must be securely attached to the reinforcement cage before concrete placement (Mindess et al. 2003). These tubes will be discussed in further detail with each testing method that utilizes them.

        o   Pulse echo method

The pulse echo testing method, also known as the low strain integrity test (See et al. 2005), is the only non-destructive testing method described in this paper that does not require the previously mentioned access tubes. To perform this test a plastic tipped hammer is used to strike the top of the drilled shaft creating a pulse wave that will travel down and back through the shaft. The impulse wave will reflect off or any anomalies or irregularities within the concrete. The return wave is then recorded using a geophone or accelerometer. The time for the wave to travel back to the receiver will give the operator

a clue as to if an anomaly is present within the shaft. (Olsen, Aouad, and Sack 1998) A

diagram of this test is presented in Figure 2.4.
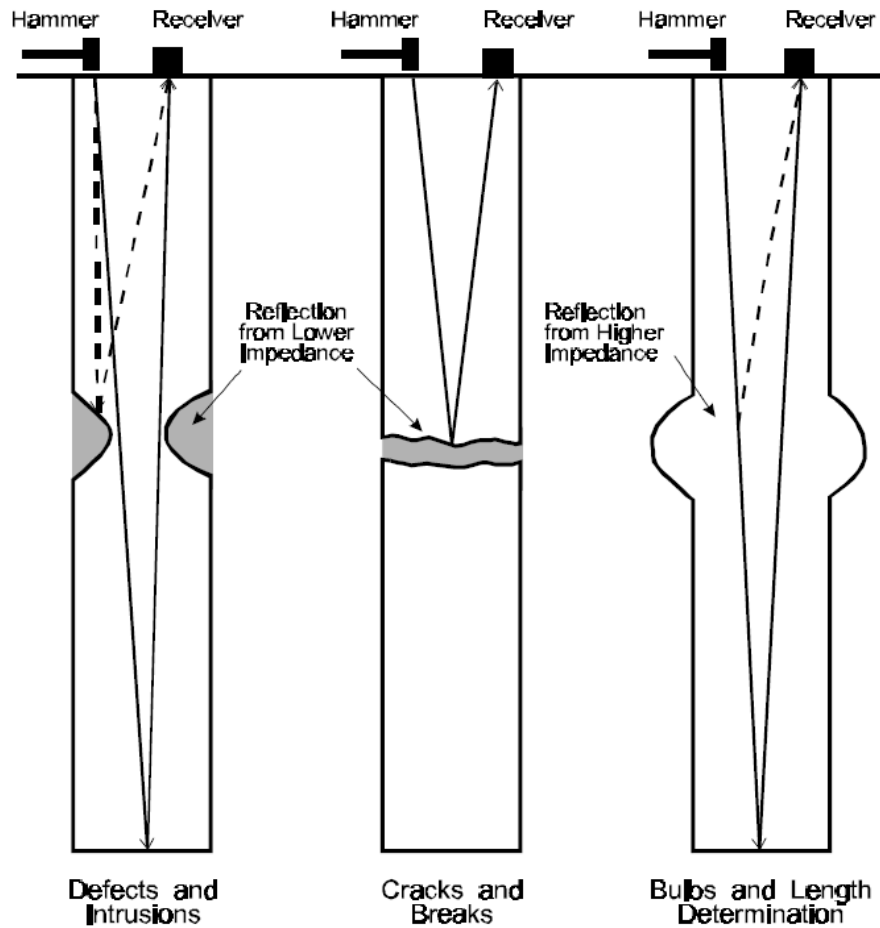
Figure 2.4: Diagram of sonic echo test (Olson, Aouad and Sack 1998)

An advantage of this test is that access tubes do not need to be previously

installed, saving time and preventing increased congestion in the reinforcement cage,

and this is a relatively fast test to conduct. However, a disadvantage of this test is that

the results can be very difficult to interpret.

o   Cross-hole sonic logging (CSL)

The CSL test is a popular drilled shaft testing method. To conduct this test the hollow access tubes (frequently known as CSL tubes) are filled with water. A ultrasonic transmitter is lowered down one hole, while a receiver is lowered down another hole. The transmitter and receiver are then raised from the bottom at the same slow rate. The time for the transmitted pulse to get to the receiver is recorded for every 0.2 inch of vertical travel up the CSL tube. The CSL tubes are assumed to be straight and therefore the distance between the tubes are known. With the pulse time and distance known, the velocity of the ultrasonic wave is calculated. This wave speed is plotted versus elevation and anomalies can be determined by a decrease or loss of wave speed. (Olsen, Aouad, and Sack 1998) A diagram of this test is presented in Figure 2.5.

An advantage of this test is that it is relatively quick and inexpensive to perform, and the results are relatively easy to interpret. The quality of the concrete can also be estimated from the wave velocities. A disadvantage of this test is that it only checks the concrete between the CSL tubes and cannot test the concrete outside of the reinforcement cage, known as the cover region. (Rausche et al. 2005)

o   Crosshole tomography

Crosshole tomography is very similar to CSL testing described previously and is usually used after a defect has been identified by CSL testing. This test uses the same equipment as the CSL test, but takes many more pulse soundings. Once the location of the anomaly has been determined using the CSL test, the receiver and transmitter are then raised and lowered at elevations around the anomaly. In this way a 3-dimensional image can be created of the unknown anomaly. A diagram of this is presented in Figure 2.6.
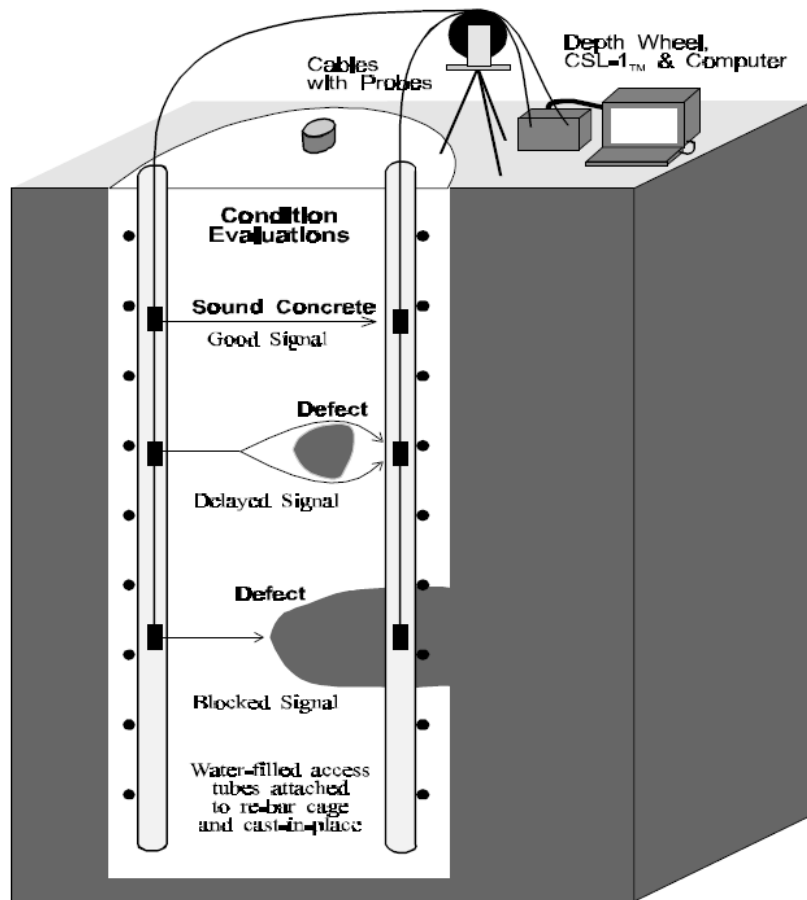
Figure 2.5: Diagram of CSL test (Olsen, Aouad, and Sack 1998)

The advantage of this test, over the other tests described in this text, is that this test can more specifically determine the shape and most importantly the size of the anomaly. However, this test, as with the CSL test, can only distinguish anomalies between the CSL tubes. Another disadvantage of this test is that it requires many calculations and must have a computer program to interpret the results. (Olsen, Aouad, and Sack 1998)
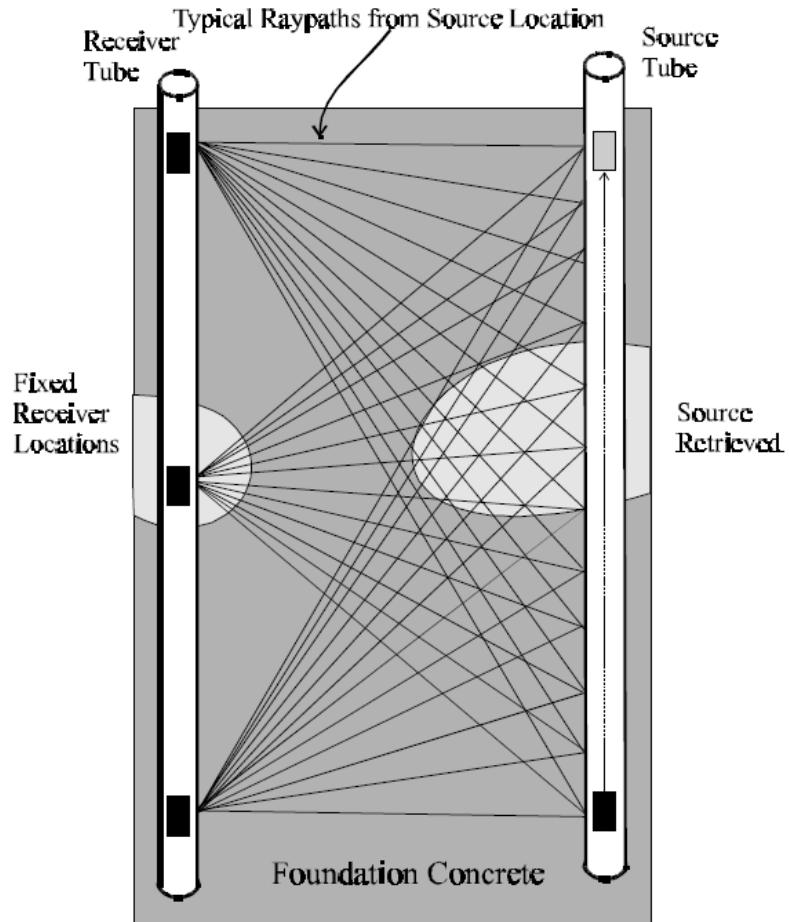
Figure 2.6: Diagram of crosshole tomography (Olsen, Aouad, and Sack 1998)

    o   Gamma-gamma testing

This method is conducted by using a radioactive source instead of a pulse or

sonic source, like the CSL and echo tests described earlier. This test is conducted by

lowering a radioactive source and reciever down each of the CSL tubes. The amount of

photons sent and received through the concrete is recorded. These results directly

relate to the density of the concrete. In this way the quality of the in-place concrete and

the location of any anomalies within the shaft can be determined.

The advantage of this test, unlike the CSL and crosshole tomography tests, is

that it can determine the quality of the concrete on the "outside" of the reinforcement

26

cage. The detection range for a probe used by the California Department of

Transportation has a range of 3-in. around the CSL tube (California Department of

Transportation 2010). This lack of penetration depth is a disadvantage of this test.

Another disadvantage is that radioactive material must be utilized in this test and

therefore this is a relatively expensive test to conduct. Because of the expense, this test

is not popular and is only known to be routinely used in California (conversation with Joe

Bailey 2009).

> o Thermal integrity testing

Thermal integrity testing works by measuring the concrete's temperature a few

days after the concrete placement, and comparing these measured temperatures to

predicted temperatures. This test is conducted by lowering temperature probes into the

CSL tubes and recording the temperature within these tubes. Based on developed

equations, the temperature within the shafts can be predicted. In areas where the

measured temperature is significantly different than the predicted temperature,

anomalies may be present. A cross-section of a test shaft with intentional anomalies

installed on the outside of the shaft is presented in Figure 2.7.

A limitation of this test is that the optimal time to perform this test is between 1

and 3 days after the concrete was placed. In shafts greater than 8 feet in diameter this

test may be conducted no later than 6 days after the concrete placement. (Mullins and

Kranc 2007)
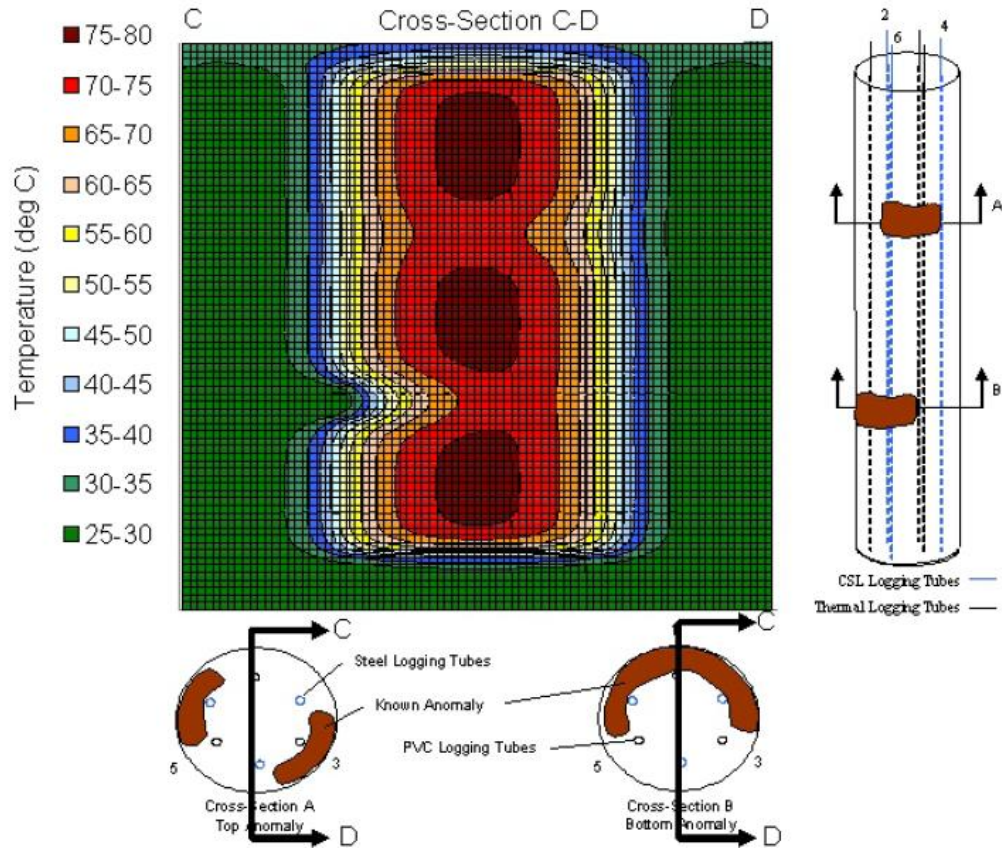
Figure 2.7: Model from thermal integrity testing showing anomalies intentionally placed

in a test shaft (Mullins and Kranc 2007)

2.2.5 Defects in Drilled Shaft Concrete

Defects can occur in drilled shafts for many different reasons. These reasons can range from issues related to constructing the shaft, problems while placing the concrete into the shaft, problems with managing the casing during the placement, problems managing the drilling slurry, or design deficiencies (O'Neil 1991). Other problems that cause defects in shafts are excess heat of hydration and excess bleed water in the concrete (Brown 2004). Due to the scope of this study, only problems caused by the drilled shaft concrete are explained.

2.2.5.1 Placement through Water or Slurry

One issue that may cause defects located at the bottom of the shaft is placing the concrete through water or slurry. That is, either free-falling concrete into a shaft that has not been totally pumped of all water, or tremie placing the concrete through water, or slurry, with the tremie pipe not near the bottom of the excavation. The water (or slurry) within the open shaft can cause the concrete to segregate and become weak mortar formed by the mixing of concrete and water, known as laitance. During tremie placement, this problem can occur anywhere in the shaft, if the tremie breaches the top of the concrete surface. This defect is a very difficult defect to detect (O'Neil 1991).

In addition, concrete is more likely to have trouble flowing through congested reinforcement cages in slurry or water filled holes because the weight of the concrete that drives the material through the cage is lessoned due to the buoyancy effects of the liquid (Yao and Bittner 2007).

2.2.5.2 Loss of Workability

If there are delays during construction, such as delays in the arrival of the concrete truck, problems with losing the concrete workability can occur. Brown (2004) notes that even without delays, if the shaft is very large it may take hours to complete the concrete placement. If the concrete starts losing its workability before the conclusion of the placement, debris can become trapped as the fresh concrete escaping the tremie pipe takes the path of least resistance and instead of displacing the old concrete, moves through the old concrete to the surface. This act can cause laitance and debris to become trapped outside of the reinforcement cage. (O'Neil and Reese 1999; Brown 2004) A diagram of this anomaly is presented in Figure 2.8. A large void caused by this anomaly is shown in Figure 2.9.

In a temporarily cased hole, loss of workability can be even more detrimental. If the concrete starts to lose workability before the casing is removed, a phenomenon

known as necking can occur. This phenomenon occurs when the stiff concrete arches

against the temporary casing so that when the casing starts to be removed the concrete

wants to move with the casing. This phenomenon can make it very difficult to remove

the casing and can cause voids in the outside of the reinforcement cage, or in the most

severe cases can cause a complete separation between layers of concrete. Even if the

casing is removed without necking, a loss of skin friction may have occurred if the

concrete was not flowable enough to fill in the gap of the removed casing. (O'Neil 1991;

Brown 2004)
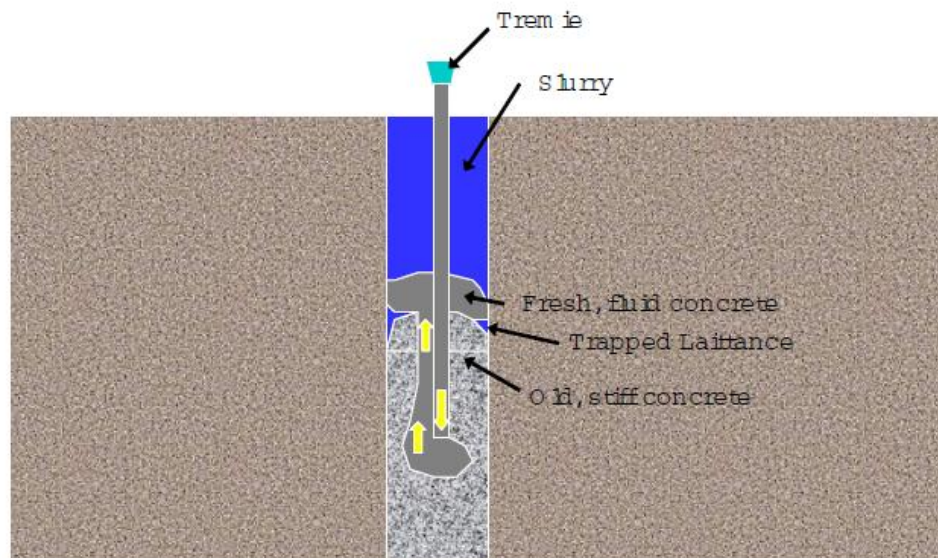
Figure 2.8: Effects of loss of workability during concrete placement (Brown and Schindler
2007)
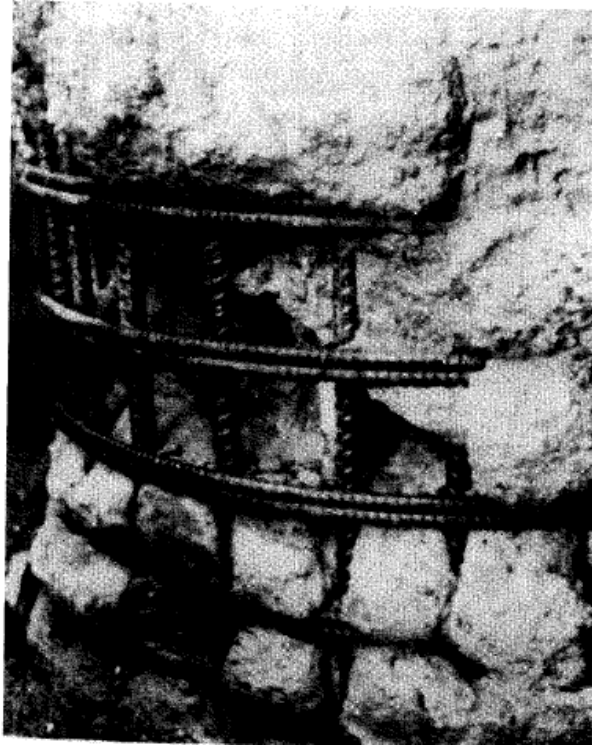
Figure 2.9: Defect in a drilled shaft caused by interruption in concrete supply during pumping (Photograph courtesy of Caltrans) (O'Neil and Reese 1999)

2.2.5.3 Suspended Solids in Slurry

O'Neil and Reese (1999) explain that care must be taken to make sure that the slurry does not have excess particles in suspension. These excess particles, or sediment, can settle to the bottom of the shaft weakening the bearing tip of the completed shaft, or can settle as the concrete is placed causing voids on the outside of the shaft. An example of voids that may be caused by sediment settling during the concrete placement is presented in Figure 2.10.

Defects caused by the settlement of sedimentary particles are very difficult to detect except for by visual inspection of the shaft (O'Neil 1991).
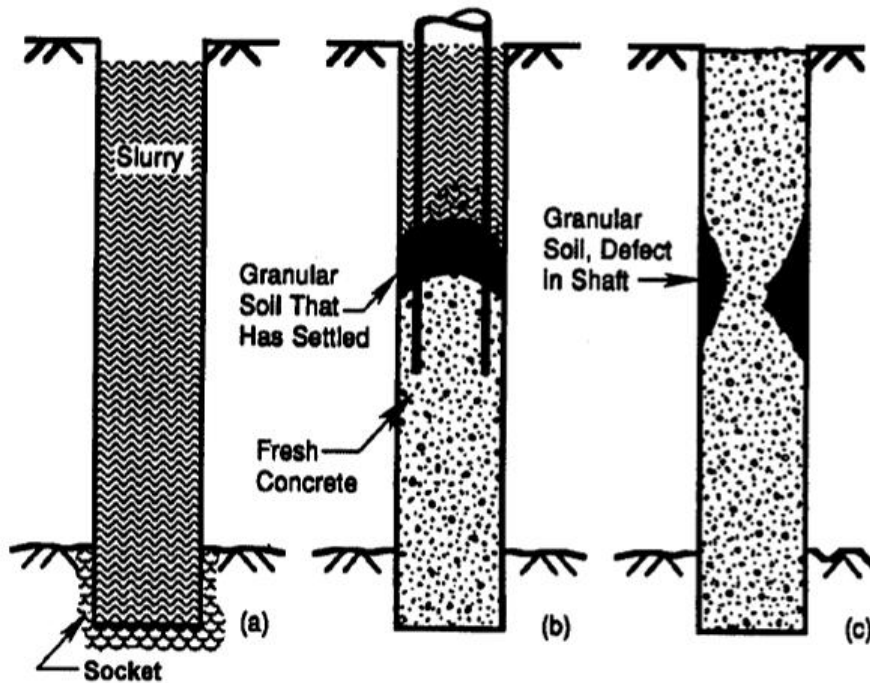
Figure 2.10: Placing concrete through heavily contaminated slurry (O'Neil and Reese 1999)

2.2.5.4 Congested Reinforcement Cages

Brown (2004) points out that design deficiencies in the shaft or the concrete will lend themselves to causing defects in the drilled shafts. He notes that large drilled shafts lend themselves to very congested reinforcement cages. Congested cages can cause defects because the flow of the concrete into the cover of the drilled shaft is greatly obstructed. It must be noted that congested reinforcement cages are not the reason for flaws. The reason for the defects is the lack of compatibility of the concrete with the reinforcement cage. The concrete must be designed to flow through the cage (Brown 2004).

When placing a shaft with incompatible reinforcement and concrete, the concrete will fill the shaft inside the reinforcement cage first, and only start to fill the cover of the

shaft when enough head has been developed to force the concrete through the cage. This elevation difference between the inside and outside of the reinforcement cage greatly increases the chances of sediment and slurry being caught in pockets along the outside of the reinforcement cage. (Brown 2004; O'Neil 1991) A diagram of how debris may be caught due to congested reinforcement cages is presented in Figure 2.11.
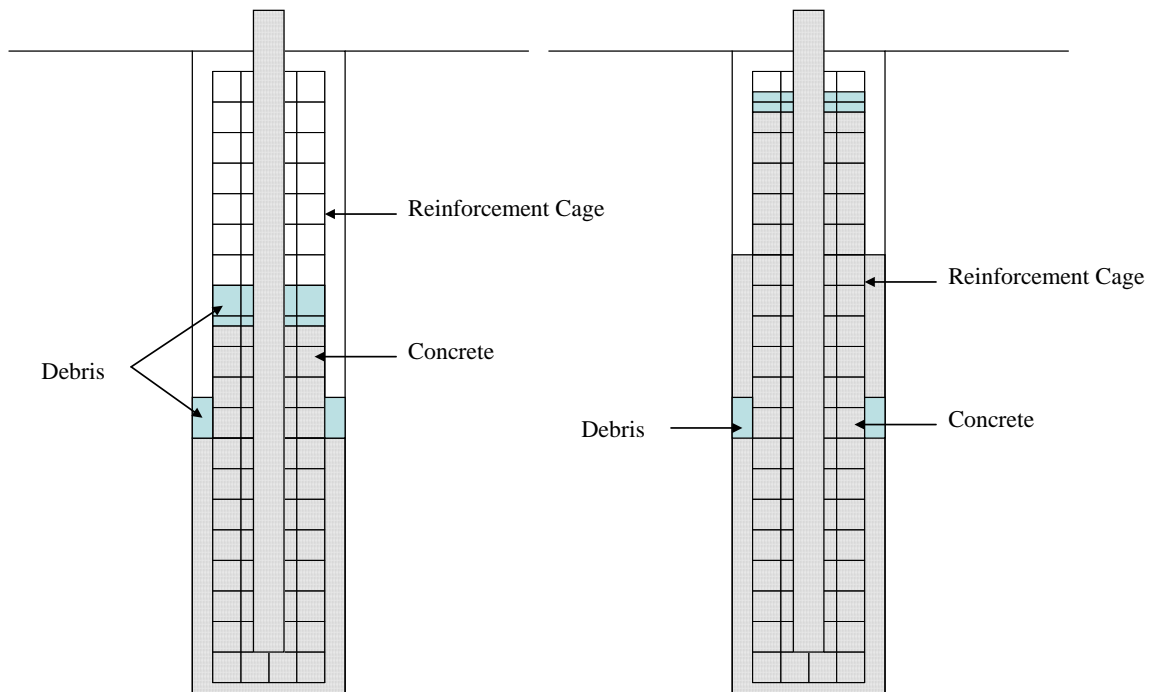
(A) Congested cage prevents adequate flow into the cover region of the shaft with debris and/or laitence floating on concrete in the cover region.

(B) The debris and/or laitence is trapped in the cover region due to the congested reinforcement cage.

Figure 2.11: Congested reinforcement cage causing concrete to trap debris (adapted from Bailey 2009)

2.2.5.5 Excessive Heat of Hydration

If the concrete starts to heat to quickly, flash setting can occur. Flash setting occurs when excessive temperatures "accelerate the rate of hydration significantly and reduce the concrete's workability" (Brown and Schindler 2007). Problems with lack of workability have been written about previously in this paper.

Besides loss of workability, high heat of hydration within the shaft can cause long term durability issues within the drilled shafts. Concretes with high volumes of fly ash or slag should be able to reach hydration temperatures of up to 178°F without long term durability problems. (Brown and Schindler 2007)

2.2.5.6 Segregation and Bleed water

In order to create a high flow concrete mixture without the addition of water reducers, the mixture must have a relatively high water-to-cementitious material ratio.

A special type of segregation occurs when excess water purges itself from the curing concrete. This anomaly is called concrete bleeding. Bleed water is the result of excess batch water escaping the concrete mixture. The excess water that is not used to hydrate the cement particles must go somewhere and therefore finds a way of escaping the concrete structure. This is not an issue when the water is allowed to harmlessly escape into the surrounding soil or shaft surface. However, this can be a problem in drilled shafts that have a permanent casing, or in shafts located in low permeability material. This casing or impermeable soil prevents the bleed water from escaping to the surface and therefore can cause voids and cracks within the curing shaft (Brown 2004).

2.3   High-Performance Drilled Shaft Concrete (HPDSC)

High-performance drilled shaft concrete is a term used in this paper to describe a highly flowable concrete that is designed to be used in a drilled shaft. (Brown and Schindler 2007) This concrete is commonly referred to in this report as SCC.

2.3.1   Self-Consolidating Concrete (SCC)

Self-consolidating concrete (SCC) is a type of high performance concrete (HPC). McCraven (2002) wrote an article of HPC stating " HPC…is concrete that meets a combination of special performance and uniformity requirements that cannot be routinely

achieved with conventional materials and practice." McCraven (2002) lists the following characteristics for HPC:

- *Ease of placement and compaction without segregation*
- *High-early strength*
- *Impermeability and high density*
- *Durability (based on exposure) and toughness*
- *Long service life (≥75 years)*
- *Low heat of hydration*
- *Volume stability (minimal shrinkage or thermal expansion)*
- *Flowability and self-leveling capability*

Goodier (2003) defines SCC as "a fresh concrete which possesses superior flowability under maintained stability (i.e. no segregation) thus allowing self-compaction – that is, material consolidation without addition of energy.*"*

To be a SCC the concrete must exhibit characteristics like those for drilled shaft concrete. The concrete must have the following three characteristics (Goodier 2003):

- The ability to flow around formwork and completely fill an area, including corners,
- The ability to pass through congested areas without segregating, and
- The ability to remain fluid and resist segregation.

2.3.1.1 History of SCC

Self-consolidating concrete, also known as self-compacting, was developed in Japan in 1988. This concrete was developed to create durable structures in a market, that at the time, had a steadily declining number of skilled workers. Goodier (2003) writes, "The removal of the need for compaction of the concrete reduced the potential for durability defects due to inadequate compaction (e.g. honeycombing)" caused by unskilled workers.

SCC started to be used in Europe in the mid to late 1990's. To explain the popularity of this product, at the time of Goodier's article in 2003, it was believed that

10% of Sweden's ready mix concrete is SCC. The use of this material in the United States was much more limited, but has been steadily growing since. (Goodier 2003)

2.3.1.2 Concrete Consistency

Self-consolidating concrete is very easy to identity due to the "flowing" characteristics of the fresh concrete mixture. The study of the deformation and flow of a material under stress is known as rheology (Mindess et al.2003). One way to describe the rheology of fresh concrete is to break the flow down into two main characteristics: yield point and plastic viscosity.

The yield point of the concrete is the point at which a force causes the concrete to start to move. SCC has a very low yield point and therefore requires a very small amount of force to move the concrete mixture. The plastic viscosity of the concrete mixture describes the ability of the concrete to flow on its own, basically the concrete's ability to resist its own internal friction. (EFNARC 2006)

An important aspect of SCC mixture proportion is the free water. The European Federation for Specialist Construction Chemicals and Concrete Systems state that variations of 1.5% moisture content (typical for aggregates) will lead to changes of 10 to 15 litres/m^3 of free water. This free water will lead to significant variations in the characteristics of flow and stability, and will cause excess bleeding. (EFNARC 2006)

2.3.1.3 Types of SCC

Bonen and Shah (2005) explain that there are two basic classifications of SCC: the powder type and the vicosity modifying admixture (VMA) type. The powder type uses large amounts of very fine powder (< 0.15 mm) to act as a lubricating medium within the concrete mixture (Khayat et al. 2006). The powder controls the plastic viscosity of the mixture while a superplasticizer (high-range water reducer) controls the yield point of the mixture. The VMA type uses a viscosity modifying admixture (VMA) to control the plastic viscosity, while the yield point is still controlled by a water reducer.

36

A third classification was described by the European Federation for Specialist Construction Chemicals and Concrete Sytems in their Guidelines for Viscosity Modifying Admixtures For Concrete. This document describes a combination type which uses powder and water reducer to enhance the flow of the concrete, as well as VMA to control the flow (EFNARC 2006).

2.3.2 Admixtures

It is popular for some forms of SCC to use admixtures added to the concrete mixture to control the performance of the fresh concrete and hardened concrete properties. Some SCC uses high-range water reducers (HRWR) or the newer synthetic high-range water reducers (SHRWR) to give the concrete its ability to flow. This is done by the HRWR giving a negative charge to all the cement particles. This negative charge causes the particles to repel each other and disperse within the mixture as presented in Figure 2.12 (Bury and Christianson, 2003).
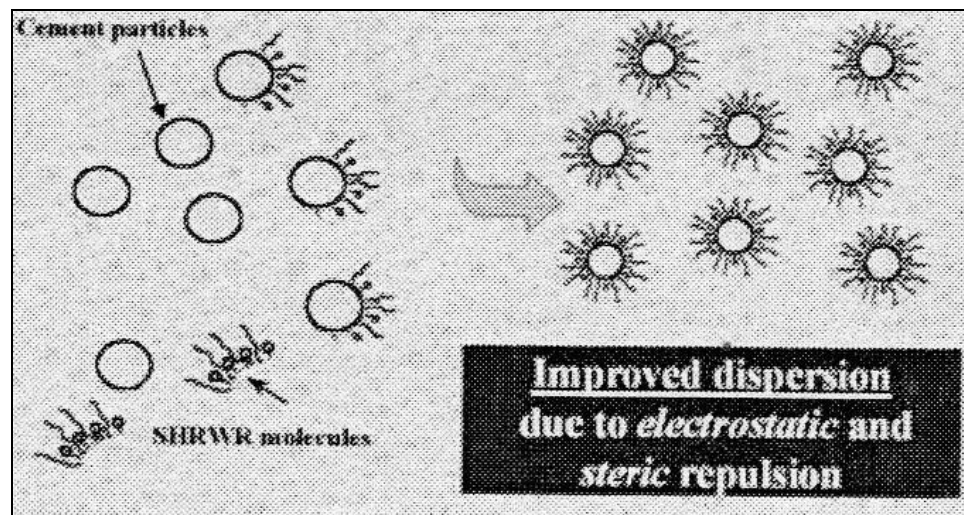
Figure 2.12: Dispersion of cement particles (Bury and Christensen 2003)

The use of a viscosity modifying admixture (VMA) will give the concrete stability. There are two different types of VMA used in SCC. The first type is VMA Thickening-Type which controls the stability of the concrete mixture by thickening the mixture and

therefore adding cohesion, which in-turn makes the concrete "more stable and less prone to segregation during and after placement." The second is the VMA binding type which controls the stability of the concrete mixture by binding the water within the mixture. By binding the water, the concrete will be less prone to bleeding, but the fresh concrete may be prone to turning into a gel when sitting still (Bury and Christianson 2003).

Water reducer and VMA are used to control how the concrete flows. HRWR is used to control the flow or spread of the concrete and VMA is used to control how fast the concrete flows.

Another popular admixture commonly used in SCC is a set-retarding admixture. This admixture decreases the rate at which the concrete will start to lose workability. In doing this, this admixture gives the concrete enough time to remain fluid until flow is no longer required. Made of lignosulfonic acids, hydrocarboxylic acids, sugars, phosphates, or salts of amphosess metals (zinc, lead, or tin); retarders slow the initial concrete reaction by slowing down the growth of crystals within the mixture. In practice, concrete truck drivers have been known to add table sugar or carbonated beverages into their concrete trucks to delay the set time and allow ample time for cleaning out the truck. When using this admixture it is expected that early strength will be reduced; however, retarding admixtures have been known to increase the ultimate compressive strength of the concrete. (Mindess et al. 2003)

2.3.2.1 Fresh Concrete Tesing

Many test methods have been developed to distinguish the quality of the fresh SCC (ACI 237 2007):

- Slump flow and visual stability index (VSI)

The slump flow test characterizes the filling ability (flow) and the VSI characterizes the stability of the fresh concrete. These tests are performed using the

38

same slump cone as ASTM C143 (2005). The specification for this test can be found in ASTM C1611 (2005). This test inverts the standard slump cone on a non-absorbent surface. The cone is filled in one continuous motion, then raised to allow the concrete to flow out the bottom of the cone and spread into a concrete patty. Once the concrete has stopped moving, two perdendicular measurements of the patty are recorded. The average of these measurements, reported to the nearest 0.5 in., is the slump flow of the sample. After this is done, a visual examination is conducted to assess the stability of the concrete patty. The index for this test has values from 0 to 3. A value of 0 signifies no visual segregation, and a value of 3 signifies complete segregation. A training manual describing this VSI test in greater detail is presented in Appendix C.

- J-Ring

This test characterized the passing ability of the fresh concrete. That is, the ability of the concrete to pass through tightly spaced reinforcement, or small openings. The specification for this test can be found in ASTM C1621 (2006). This test is performed using the same inverted slump cone and filling method, but a standard circular device with vertical bars is placed around the cone. A picture of the J-Ring test being conducted is presented in Figure 2.13. The cone is lifted so that the concrete flows out the bottom and must spread through the tightly spaced vertical bars. The diameter of the impeded flow is compared to the diameter of the unimpeded flow (slump flow test) to calculate the passing ability of the concrete.

The standard J-Ring test was considered too congested for drilled shaft applications. Because of this a modified J-Ring test was created. This modified J-Ring test is conducted in the same manner, but the modified J-Ring has 13 bars around the 12-in. diameter ring, instead of the ASTM specified 16 bars. This changed the bar spacing from 1.74-in. to 2.27-in. (Dachelet 2008).

Figure 2.13: Modified J-Ring test being conducted

- Column segregation

This test characterizes the stability of the fresh concrete. The specification for this test can be found in ASTM C1610 (2006). This test is performed using a column that can be separated into three sections: lower, middle, and upper. Intact, the column is filled with the fresh SCC. After a 10-min. period the column is then carefully taken apart in order to sieve the contents of the top and bottom sections of the column through a No. 4 sieve separately. The weight of the sieved contents of the top is compared to the weight of the sieved contents of the bottom. The percent segregation is taken as the percent difference in these weights. A picture of the segregation column is presented in Figure 2.14.

Figure 2.14: Segregation column

2.3.3 Design and Production of SCC for drilled shafts

Brown and Schindler (2007) studied problems with concrete placed in drilled shafts and describe a type of "high performance drilled shaft concrete" to use in drilled shaft applications. This high performance drilled shaft concrete uses chemical admixtures and mixture components similar to SCC.

High Performance Drilled Shaft Concrete (HPDSC) is a special type of SCC designed for use in drilled shafts placed under the water table with very congested reinforcement steel. This is a very specific use with very unusual combinations of problems. Since the shafts are placed underwater, drilling slurry must be used. The concrete is tremied or pumped through the slurry to the bottom of the shaft. The concrete must therefore be able to be pumped or temied underwater or under slurry. In addition the concrete must be able to flow through a congested reinforcement cages without the use of vibration.

To combat these difficulties, HPDSC uses a VMA type SCC or a combination SCC. It is important to include a VMA to the concrete mixture, because the VMA gives the mixture a cohesiveness or "stickiness" to prevent washout of the concrete within the water (EFNARC 2006). Bury and Christensen (2003) state "Concrete containing a VMA exhibits superior stability, even at high levels of fluidity, thus increasing resistance to segregation and facilitating easy placement."

2.4   North American SCC Drilled Shaft Projects

The following is a summarized review of literature documenting the use of SCC in drilled shafts in North America.

2.4.1   GRL and Pile Dynamics, Inc (PDI) Shaft

In March of 2003, GRL and Pile Dynamics constructed a 40-ft deep drilled shaft with four different concrete mixtures. One of these concrete mixtures was SCC. This was conducted to see if the flowability of SCC would help ensure good cover concrete for drilled shafts. (See et al. 2005) The results of this research could not be discovered by the author, but GRL sent a proposal for further SCC research to Degussa Admixtures, Inc. (now BASF) and this proposal turned into the next discussed research project.

2.4.2   Degussa Admixtures Test Shaft

Raushe et al. (2005) describe a research program that was conducted by Degussa Admixtures, currently known as BASF, with the support of GRL Engineers, Inc. to evaluate the use of SCC in drilled shaft applications.

This project consisted of testing 12 different concrete mixtures with varying slumps and slump flows. Of the 12 concrete mixtures, seven were SCC and five were conventional concrete mixtures. Set retarding admixtures were included into two of the SCC mixtures. These mixtures were placed into rectangular walls that had two hollow

steel tubes installed on either side. These tubes were placed so that the concrete wall would have a 2.75 in. concrete cover.

During the creation of the rectangular test specimens, fresh concrete tests were conducted. These include, slump (ASTM C 143) [on conventional mixtures], air content (ASTM C 231), unit weight (ASTM C 138), slump flow [on SCC mixtures], Visual Stability Index (VSI), T_{50}, U-Box, Column Segregation, IBB rheometer, and rate of hardening (ASTM C 403). Current ASTM Tests, such as slump flow, VSI, T_{50}, and column segregation had not been certified at the time of this research and cannot be verified if the current ASTM standards were used. Therefore the ASTM standard references were intentionally left off these tests.

Each test specimen was tested using CSL testing between the hollow metal tubes and low strain integrity test (also known as pulse echo test) off the top of the specimen. Compressive and modulus of elasticity tests (ASTM C 39 and ASTM C 469, respectively) were conducted on 4 in. X 8 in. cylinders at 1, 3, 7, 14, and 28 days.

During this project the concrete wave speeds received from the CSL test were used to attempt to predict the compressive strength of the concrete. The dynamic modulus of the concrete was also calculated from these results to compare with the measured concrete modulus. The following findings were reported from this study (Rausche et al. 2005):

- o "Concrete specimens of different slumps or slump flows, tested at the same curing times with either CSL or PIT [a.k.a. pulse echo test], showed no significant differences in wave speed as long as the mix design was practically identical",
- o Differing mixture designs show significant CSL and PIT testing results,
- o "Flow around the tube, and thus flow through tightly spaced rebar cage was improved with increasing slump flow", and

43

o "The SCC mixtures are fundamentally similar to the conventional concrete mixtures, with the strength – wave speed relationship, as well as dynamic modulus – measured modulus relationship, being the same for both types of concrete mixture."

2.4.3 Auburn Test Shafts

In 2003, at the Auburn Geotechnical Experimental Site in Opelika, AL five experimental test drilled shafts, 3.27 ft in diameter and 24 ft deep, were constructed to evaluate the use of self-consolidating concrete in drilled shafts (Hodgson et al. 2005).

Of the five test shafts, two were constructed with conventional drilled shaft concrete using crushed No. 57 stone, one shaft was constructed using conventional drilled shaft concrete with No. 7 uncrushed river gravel, and the last two shafts were constructed with SCC. Concrete for each shaft was tested for fresh concrete properties using the slump, L-box, slump flow, and mortar V-tunnel tests. These shafts were exhumed after four months and cut across their diameter to perform a visual inspection for any form of visible segregation. Modulus and compressive strengths taken at 28 days were used to compare hardened properties of each shaft.

The reinforcement cage in each shaft was made of 16 No. 9 reinforcement bars and No. 4 hoops at a spacing of 4 in. The reinforcement cages for the SCC mixtures were slightly different. One SCC shaft had 13 No. 9 reinforcement bars with No. 4 hoops at a spacing of 4 in. The other SCC shaft had the same longitudinal reinforcement as the ordinary shafts, but had hoops spaced at 2.25 in. Sand bags were attached to a few of the reinforcement cages to simulate debris within the shafts.

Placement was conducted using a tremie, but no slurry was used because of the use of a down-hole video camera to film the flow in the concrete within the shafts. The elevation difference between the inside and outside of the reinforcement cage was also recorded for each shaft.

For each shaft, nine 6 X 12 mm cylinders were used to test the concrete's modulus and compressive strength at 7, 28 and 91 days after the concrete placement. The following findings were reported by Hodgson et al. (2005):

- o The mortar V-tunnel test was considered impractical due to its time consumption, difficulty, and lack of precision. The slump flow, T_{50}, and L-box tests were deemed acceptable for quality control testing of SCC.

- o Rapid mixing of mixtures with HRWR results in excessive air contents within the concrete mixture.

- o The SCC flowed better through the reinforcement cage. The elevation difference for the conventional shafts was as much as 18 in. whereas the SCC shafts were as much as 4 in.

- o The SCC mixture flowed uniformly through the reinforcement cage throughout the entire placement, whereas the conventional concrete's flow through the reinforcement cage was much more erratic.

- o The SCC did not reach the required strength at 28 days; whereas, the conventional concrete was acceptable at 28 days. This was determined to be because of the increase water-to-cementitious ratio and the high amount of supplementary cementing materials of the SCC mixtures.

- o The conventional concrete with the crushed No. 57 stone was determined to have many more instances of honeycombing and did not cover the artificial debris as well as the other mixtures. The conventional mixture with No. 7 river gravel displayed similar results as the SCC mixtures, with no visible honey-combing and good flow around the artificial debris.

- o Concrete mixtures with the No. 7 river gravel appeared to have a better aggregate distribution than the concrete mixtures with crushed limestone.

- o Each shaft with the river gravel seemed to have an even amount of aggregate throughout the length of the shaft.
- o Air-voids of 0.04 to 0.08 in. were visible in the SCC shafts. The fresh SCC concrete also had unusually high air contents. The high air contents of these mixtures were concluded to be caused by rapid on-site mixing after the addition of additional HRWR.

2.4.4   South Carolina Bridge Project

In 2005, at Lumber River, South Carolina, Auburn University conducted a project to evaluate the use of self-consolidating concrete for drilled shaft applications. For this project Auburn University developed a SCC mixture from an extensive laboratory-testing program. (Brown and Schindler 2005; Holley et al. 2005)

This SCC mixture was developed to compare to an experimental drilled shaft mixture considered by the South Carolina Department of Transportation (SCDOT), this mixture was known as the SC Coastal mixture. The aggregates and cementitious materials used for the SCC and SC Coastal mixtures were all from sources located in South Carolina. Both the SCC and SC Coastal mixtures used a blend of No. 789 and No. 67 gravel. The SC Coastal mixture used water reducers to increase the flow of the concrete (greater than that of conventional concrete).

The SCC mixture had a high sand-to-total aggregate ratio and a higher fly ash content than most drilled shaft mixtures. This mix had the higher cementitious content, but the lowest content of portland cement. This mixture also used a viscosity modifying admixture to increase the stability of the mixture.

For this project four 6-ft diameter experimental shafts were constructed as well as two bridge foundations. Of the four experimental shafts, two of the shafts were designed to be exhumed with a length of 30 ft, the other two shafts were to be load tested and had a length of 72 ft. One of each of the exhumed and load test shafts was

to be constructed of SCC and SC Coastal mixtures, respectively. The two bridge foundations included a smaller bridge foundation that required six shafts to be constructed using the SCC mixture, and a larger bridge foundation that required 20 shafts to be constructed using the SC Coastal mixture.

The reinforcement cage was constructed of No. 14 bars at a 6-in. spacing as well as No. 5 bar hoops at a 6-in. spacing. In the top 12 ft of each shaft the hoops were spaced on 3-in. centers. In addition, six hollow metal tubes (CSL tubes) were attached to the longitudinal bars. A second reinforcement cage was put inside the first for the top 12 ft of the shaft. This cage was made of No. 11 bars at 5-in. spacing as well as No. 5 hoops at a 6-in. spacing.

The 30 ft shafts were constructed using a temporary casing. The 72 ft shafts were constructed with a permanent casing. A 12 in. diameter tremie pipe was used to place concrete. Color-dye was used in the 30 ft shafts to evaluate the concrete flow. The first concrete placed into the hole was dyed black with grey and red following sequentially. Approximately 4 yd^3 of black, 16 yd^3 of grey, and 4 yd^3 of red concrete were used. After the addition of the dyed concrete the tremie was raised 10 ft. An intentional 30 min. delay was caused to simulate delays that occur in the field after 24 yd^3 were placed.

The SC Coastal mixture's slump varied between 10 in. and 10.5 in. The SCC slump flow varied between 24 in. and 27 in. Both concrete mixtures had significant amounts of bleed water on the surface of the shafts. This amount equaled approximately 6 in. to 10 in. over the entire cross-section. It was observed on the following day that the "centers of the shafts were depressed from the reduction in volume." The 28-day compressive strengths of both concrete were greater than 6,200 psi.

The CSL results showed both shafts had good quality concrete except for an anomaly that was seen in the SCC shaft at a depth of 13 ft. After excavation and cutting the shaft this anomaly was determined to be a small soil inclusion lodged on the side of one of the CSL tubes, a defect that would have occurred in any concrete, not an effect of the SCC.

Upon visual analysis, the outside surface of both shafts did not show any surface irregularities. At the bottom corners of each shaft, some irregularities were noted. Brown et al. (2005) and Holley et al. (2005) reported the following findings from this project:

- o Both SCC and the SC Coastal mix performed well in difficult construction conditions.
- o "Good performance can be obtained with relatively modest attention to quality control and inspection."
- o Conventional CSL test results may exaggerate the magnitude of potential defects.
- o The use of greater amounts of cementitious material in the SCC mixture did not cause a increase in the in-place concrete temperatures.
- o Concrete in the cover region of the shafts was determined to be of acceptable quality with regard to the concrete in the interior of the shafts.
- o The SCC mixture is an acceptable drilled shaft mixture and may "prove especially useful where seismic detailing requirements result in congested reinforcement."

2.4.5   South Carolina DOT

A study was conducted by the South Carolina Department of Transportation (SCDOT) to "evaluate the integrity of the majority of drilled shafts installed on state bridge projects." (Camp et al. 2007). This project comprised of more than 400 shafts on over 42 projects. This study found that the majority of anomalies were due to concrete

irregularities and occurred near the top and bottom of the drilled shafts. Camp et al.

(2007) believes that these anomalies may have been because of "partial segregation,

probably as a result of placement through water". This study concluded that "the

majority of anomalies are attributable to concrete issues…based on cores that we have

observed, most anomalies are a result of segregation due to placement in water or

bleeding effects." (Camp et al. 2007) Most importantly, in regard to this paper, the

SCDOT study stated, "Concrete problems should be avoided through the use of

appropriate mixes that are resistant to segregation yet have good workability (e.g. self-

consolidating concrete) and the use of appropriate placement methods." (Camp et al.

2007)

2.4.6   The New Minneapolis I-35W Bridge

Western et al. (2009) wrote about the building of the new Minneapolis I-35W

bridge and the fact that this bridge was designed an built in only 11 months. Not only is

this bridge famous because of the original bridge's catastrophic collapse, but because of

the speed at which this bridge was constructed. The bridge construction challenge was

to build a bridge with a minimum service life of 100 years as fast as possible. To do this

the design build team implemented many types of HPC throughout the bridge

construction, including SCC in the drilled shafts.

The drilled shafts were seven to eight feet in diameter with depths up to 95 feet.

Western et al. (2009) stated that "This was the first large scale use of cast-in-place SCC

for Mn/DOT [Minnesota Department of Transportation]." The mixture included large

amounts fly ash and slag to reduce the heat of hydration by approximately 50%. The

specified 28-day design strength was 5,000 psi and the test cylinders had 28-day

compressive strengths up to 10,000 psi. "The performance of the SCC mix used in the

drilled shafts exceeded expectations." (Western et al. 2009)

Dr. Dan Brown was used as a consultant on this project, and explained that some instances of anomalies were seen in the CSL results for this bridge. After coring the shafts in question, it was found that sand was entrapped in some spaces within the shaft. These spaces were very small and the shafts were deemed sufficient. The reason for the sand in the shafts may have been from excess sand settling out of the slurry onto the top of the shaft. (Brown 2010)

2.4.7 New Jersey DOT project (Nassif 2008)

The New Jersey Department of Transportation has sponsored a research project, conducted by Hani Nassif of Rutgers University and Husam Najm of Florida International University, to research SCC. This project was made up into two phases. Phase One was to develop SCC mixture designs to use in pre-cast structures and evaluated the use of suppimentary cementing materials (SCM's). Phase two evaluated the use of SCC in drilled shaft construction.

It should be noted that phase one concluded that self-consolidating concrete with slump flows values greater than 24 inches "indicate good flowability as well as good ability to self consolidate without segregation." (Nassif 2008).

Phase Two consisted of the construction of five drilled shafts: three of these shafts were constructed with self-consolidating concrete of differing mixtures. Strain and temperature gauges were installed onto the cages of the SCC drilled shafts. Twenty 4 in. X 8 in. cylinders were taken from the second truck of each SCC drilled shaft mixture for 3-, 7-, 14-, and 28-day compressive strength testing. Three additional 6 in. X 12 in. cylinders were taken as well, to check the compressive strengths of the smaller cylinders. Crosshole Sonic Logging (CSL) testing was performed on the completed shafts to determine the integrity of the in-place structures.

An issue occurred with the first of the SCC mixtures to be placed in a shaft. The cylinders made to assess the concrete strengths were found to have 0.25 in. of

hardened paste on the top surface of each cylinder, a clear observation of segregation. This weakened spot lowered the compressive strengths of the cylinders, but the strengths were still above the specified limit. This problem was fixed by lowering the slump flows of the two remaining SCC drilled shaft mixtures.

The mixture of the last SCC drilled shaft to be constructed had a slump flow range between 19 in. and 21.5 in.; however, this is similar to the target slump flow used by Brown et al. (2007). This range was below the specified NJDOT specification which has slump flows of 24 in. to 28 in. The L-box and J-ring tests conducted showed that blocking may be a problem for this shaft. However, none of the drilled shafts showed any anomalies from the CSL tests.

Nassif et al. (2008) reported the following findings from this project:

- o It was recommended that the L-box or J-ring test supplement the slump flow test to ensure adequate resistance to segregation,
- o For drilled shaft applications it was recommended that the slump flow test and J-ring test be used to assess the quality of the fresh concrete,
- o "It was observed that there is a need to examine the various mixes for segregation by applying the Visual Stability Index (VSI) as a screening tool",
- o A J-ring test is an essential fresh concrete test when the SCC mixture has only HRWR to control the flow and a high aggregate content, and
- o The performance of SCC in drilled shafts was found to be "satisfactory".

2.5 Concrete Flow within Drilled Shafts

Gerwick and Holland (1986) performed tests on concrete placed under water by tremie. This concrete was flowable enough to flow and consolidate under its own weight, but was still thick enough to limit the amount of laitance. Their study determined

51

that the concrete would flow in either a bulging flow pattern or layered flow pattern. A schematic of "bulging flow" and "layered flow" are presented in Figure 2.15. It was concluded that bulging flow was the most desirable to limit the amount of laitance. (Gerwick and Holland 1986)

As explained in Section 2.4.4, a research project was conducted in South Carolina on SCC in drilled shafts. Part of this project involved using dyed concrete to determine the flow of gravity fed tremied concrete within the drilled shaft. A picture of this dyed concrete is presented in Figure 2.16. With the tremie located on the bottom of the shaft, the first load of concrete placed was dyed black and the fourth load of concrete was dyed red. The second and third loads were not dyed. This project concluded that the first load placed will fill the bottom of the shaft, and the proceeding loads will travel upwards around the tremie. However, only a small layer of grey concrete was seen between the black and red concrete. Therefore the red concrete must have displaced the grey concrete up the shaft. (Holley et al. 2005)

One longitudinal cut was made and the project budget would not allow additional longitudinal cuts to be made shafts. A cross-sectional cut is presented in Figure 2.17. The red concrete flowed much tighter around the tremie and unlike with the SC coastal drilled shaft mixture, grey concrete can be seen around the red concrete. A cross-sectional cut was also taken 13 ft from the bottom, near the location where the tremie was moved to for the rest of the concrete flow. At this location the red concrete was pushed to areas near the reinforcement cage of the shaft. A picture of this location is shown in Figure 2.18. This project concluded that, "the SCC exhibited similar flow direction to the conventional mix. The lowest slump concrete (also the first load placed and the one dyed black) from both mixes appeared to remain at the bottom of the shaft. Subsequent loads appeared to flow up around the tremie pipe, displacing the surrounding concrete out laterally." (Holley et al. 2005)
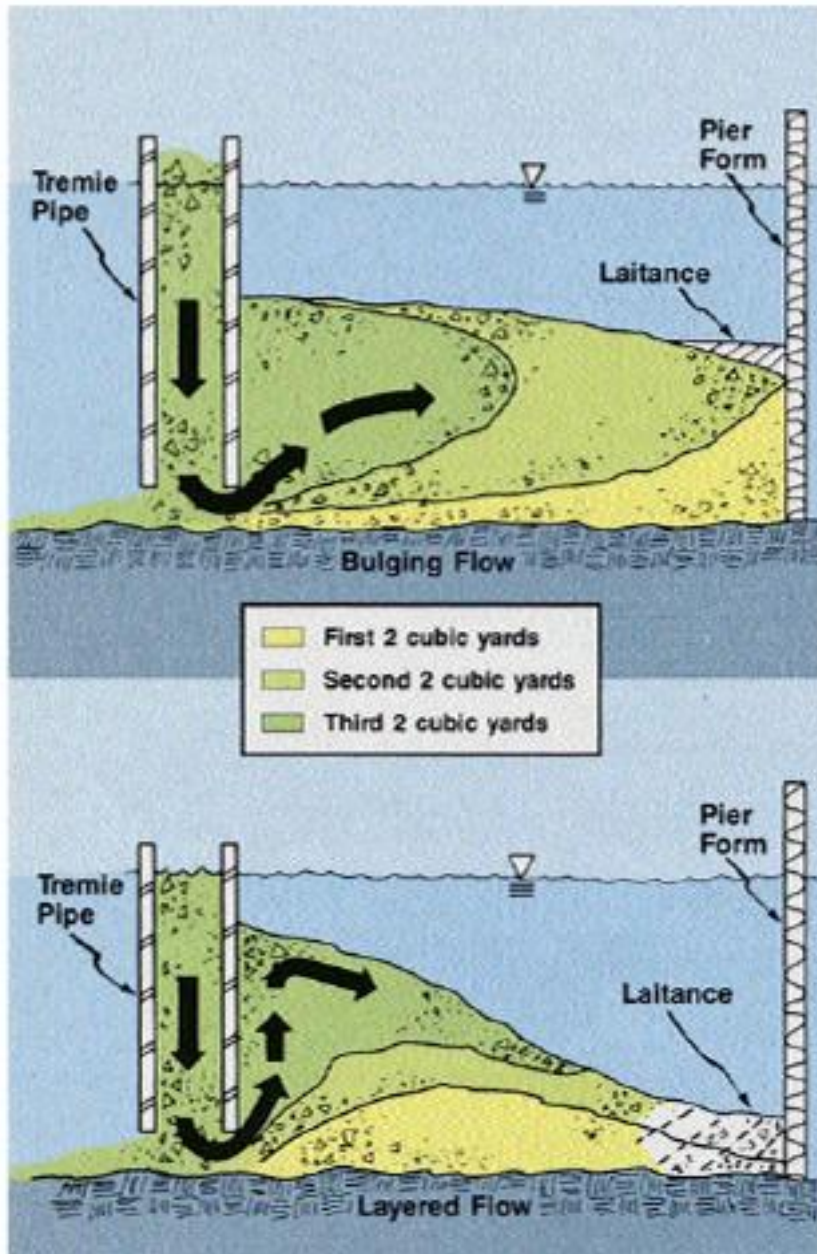
Figure 2.15: Bulge flow versus layered flow (Gerwick and Holland 1986)

Figure 2.16: Dyed concrete showing the first concrete in the shaft staying near the

bottom, filling the bottom corners of the shaft (Holley et al. 2005)

Figure 2.17: SC SCC: cross-sectional cut 6 ft from bottom (Holley et al. 2005)

Figure 2.18: SC SCC: cross-section 13 ft from bottom (Holley et al. 2005)

2.6 Pressurized bleed test

A pressurized bleed test, also known as the forced bleed test, was developed by Kamal H. Khayat in order to test the bleeding of grout and concrete mixtures under pressure. This device was created to test the effect of using rheology-modifying admixtures and high-range water reducers in combination to improve grout flow, while limiting the amount the grout will bleed under pressure. (Khayat and Yahia 1997) In 2002, Khayat again measured the force bleed of grouts using this test to determine the effects of thixotropy modifying admixtures (Khayat et al. 2002). Khayat discusses conducting this test on grout mixtures in a paper discussing the effects of using VMAs and HRWRs together. In this paper, Khayat refers to this test as the "baroid filtration test." (Saric-Coric et al. 2003)

This "baroid filtration test" is a current test conducted on bentonite slurries in order to assess the bleedability of the drilling slurry (Ball et al. 2006). A schematic of this test is presented in Figure 2.19.

Khayat used this test by taking a 6.7 fl. oz. sample and used nitrogen gas to apply a 80 psi pressure. The bleed was monitored over a 10-min. time period and calculated as a percent of total water in the sample. (Saric-Coric et al. 2003; Khayat et al. 2002; Khayat and Yahia 1997)
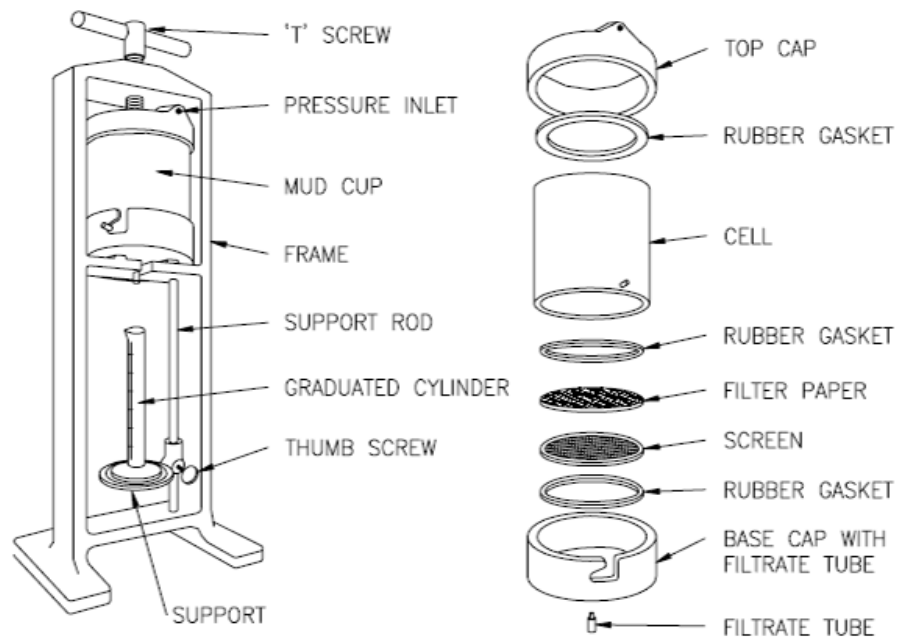
Figure 2.19: Standard Filter Press (Ball et al. 2006)

Chapter 3

Experimental Test Shafts

3.1   Introduction

This chapter is a summation of the work conducted to install and analyze three experimental drilled shafts. The primary purpose of the field study was to evaluate the use of self-consolidating concrete (SCC) as a viable material for use in drilled shaft construction. Using practiced construction methods, this field study compared self-consolidating concrete to ordinary drilled shaft concrete with regards to:

- Fresh concrete properties,

- Hardened concrete properties, and

- Overall completed shaft integrity.

In addition to the study, an analysis was conducted on the experimental shafts to evaluate the concrete flow within the drilled shaft.

3.1.1   Chapter Outline

A brief discussion of the plan for the work is presented in Section 3.2. This plan was changed slightly throughout the project and these changes are noted. Following the proposed study is a summary of the materials and mixture proportions used in the field study (Section 3.3), as well as an overview of the actual shaft construction (Section 3.4) and the results obtained from the study (Section 3.5). The following discussions include, but are not limited to:

- Details of the three test shafts,

- Fresh concrete property testing,

- Hardened concrete property testing,

- Placement monitoring,

- Temperature measurement,

- Cross-hole sonic logging testing,

- Exhuming of shafts,

- Testing of cores from exhumed shafts, and

- Analysis of exhumed shafts.

3.1.2 Project Location

The location of the field study was on AL-35 in Scottsboro, Alabama. As

presented in Figure 3.1, the field study was located on the north bank of the proposed

southbound lane of the "new" B.B. Comer Bridge. Three, 7-ft diameter 25-ft long, drilled

test shafts were prepared on the crest of a hill in the median of the existing AL-35. A

picture of the drilled shafts under construction at this location is presented in Figure 3.2.

3.2 Experimental Plan

This experimental plan is based on the proposed experimental study designed by

Dachelet (2008) in his thesis "The effectiveness of self-consolidating concrete (SCC) for

drilled shaft construction". Future tense is used in this section to explain the proposed

study. Most of this study was conducted as explained in this section; however, some

changes were made and are explained in the following sections.

★ Project location

http://www.alabama-map.org/alabama-road-map.gif
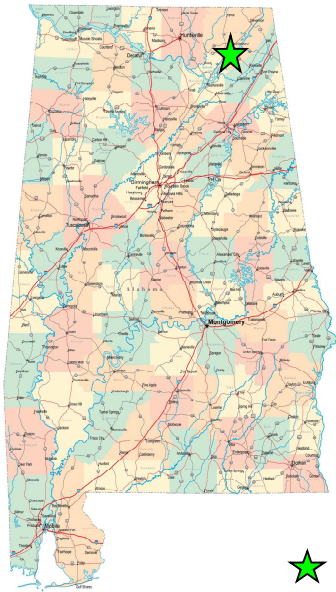
Google Maps

Figure 3.1: Project location

Figure 3.2: In-place test shafts (river can be seen in background) (photo by D. Brown)

3.2.1 Test Shafts

The plan was to construct, test, and exhume three test shafts made of three different concrete mixtures. These shafts will be exhumed 28 days, or later, after placement for visual inspection and testing. Each shaft will be constructed using a sono-tube casing with loose sand backfill around the outside of this casing. Loose sand will be used to ease the removal of the cast shaft. The casing will be filled with polymer slurry and the concrete will be pumped through the slurry-filled shaft. A polymer slurry is to be used because it is often used in drilled shaft placement below the water table. For the same reason, tremie placement was the selected placement method. A diagram showing this proposed scheme is presented in Figure 3.3. A cross-section of the proposed reinforcement cage is presented in Figure 3.4.

The following three concrete mixtures were to be used:

- **Ordinary Drilled Shaft Concrete (ODSC):** One 6.0-ft diameter X 25-ft deep test shaft made with ordinary drilled shaft concrete: water-cementious ratio (w/cm) equal to 0.40, sand-to-aggregate ratio (S/Agg) equal to 0.36, and No. 4 hoops at 4-in. on center.

- **Self-Consolidating Concrete (SCC):** One 6.0-ft diameter X 25-ft deep test shaft made with SCC: w/cm = 0.40, S/Agg = 0.49, and No. 4 hoops at 4-in. on center.

- **Self-Consolidating Concrete with Limestone Powder (SCC-LP):** One 6.0-ft diameter X 25-ft deep test shaft made with SCC with a calcium carbonate filler resulting in a w/cm = 0.44, water-to-powder ratio (w/p) equal to 0.40, S/Agg = 0.49, and No. 4 hoops at 4-in. on center.

3.2.2 Assessment of Concrete Flow during Placement

To assess the flow of the concrete within the drilled shaft, colored mortar cubes will be used. To ensure these cubes do not float or settle, mortar cubes were used as they have a similar specific gravity to the concrete mixture.

Colored mortar cubes have been made for use during construction of the experimental castings. These cubes are ½ x ½ x ½ in. square, and approximately 4,000 red cubes and 2,000 for each of the following colors blue, green, yellow and orange cubes have been made. A picture of a sample of these cubes is presented in Figure 3.5. These cubes will be added to the tremie-placed concrete at selected locations. After the exhumed shafts are cut, the location of these colored cubes will be evaluated to determine the flow characteristics of the concretes during placement.

Enough cubes have been made for use in all three experimental castings. The blocks have a 28-day compressive strength of more than 7,900 psi as tested by Dachelet (2008). Cubes of this size were made as this is smaller than the nominal maximum size of a No. 57 and 67 gradation, but large enough to be observed on a cut concrete cross section.

These cubes will be added to the concrete while the concrete is placed into the back of the pump truck. These cubes are to be placed into the concrete at a time when the concrete depth is known. Each color will be used at a different depth.
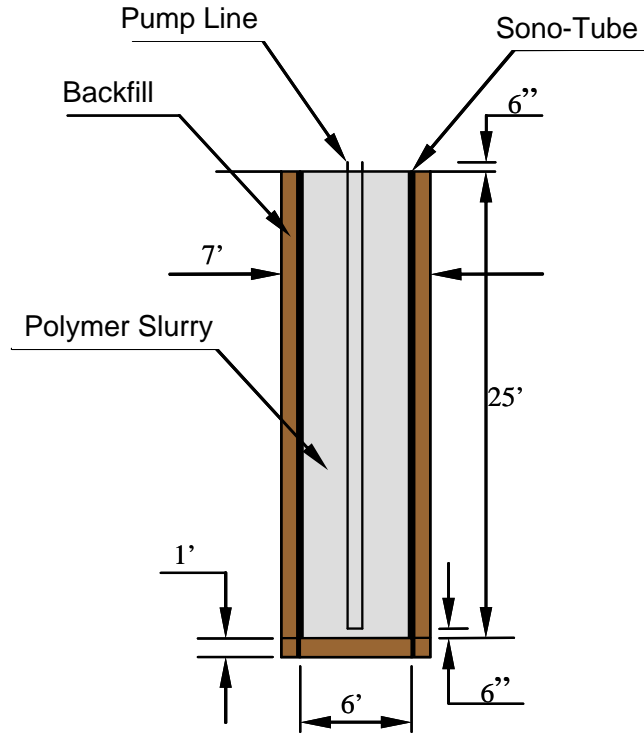
Figure 3.3: Longitudinal section of shaft (Dachelet 2008
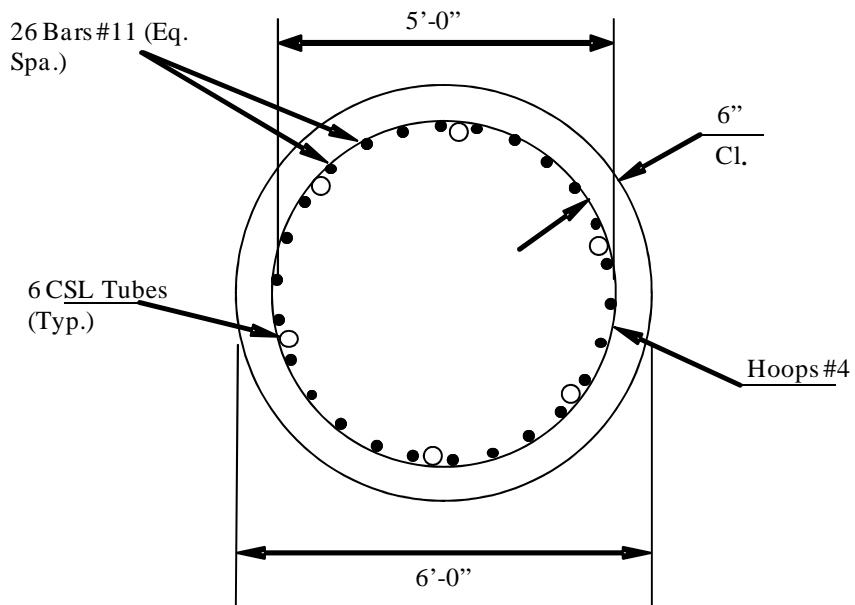
Figure 3.4: Cross section of shaft (Dachelet 2008)

62

Figure 3.5: Color mortar cubes for use in experimental shafts

The first two buckets of mortar cubes placed into the shaft will be red. Twice as many red cubes were made so that a better estimate of the initial concrete flow out of the tremie can be determined. The proposed placement of each cube color is presented in Figure 3.6. If the concrete flows in a perfectly laminar manner, as described by O'Neil and Reese (1999), the location of the cubes can be predicted, as shown in Figure 3.7.

Twenty-eight days, or later, after completing the shaft, the shaft shall be removed and cut longitudinally down its center. The exposed surface will then be cleaned and shellacked to allow visual examination of the cube locations.
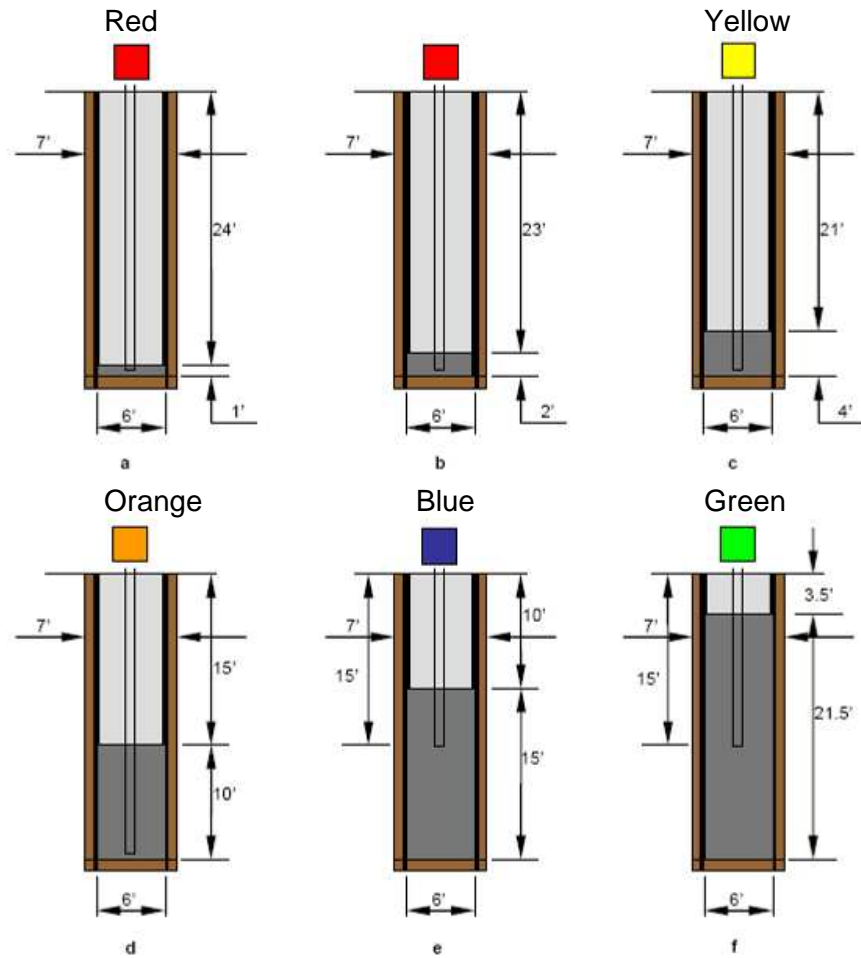
Figure 3.6: The placement order for each mortar cube (Dachelet 2008)

3.2.3 Assessment of Fresh Concrete Behavior

In order to test a representative sample of concrete from the concrete trucks, concrete samples will be taken from the trucks after they have discharged approximately half of their load. The truck will be sent to the testing area at this time in order to fill wheelbarrows with concrete for property testing. The following tests will be conducted:
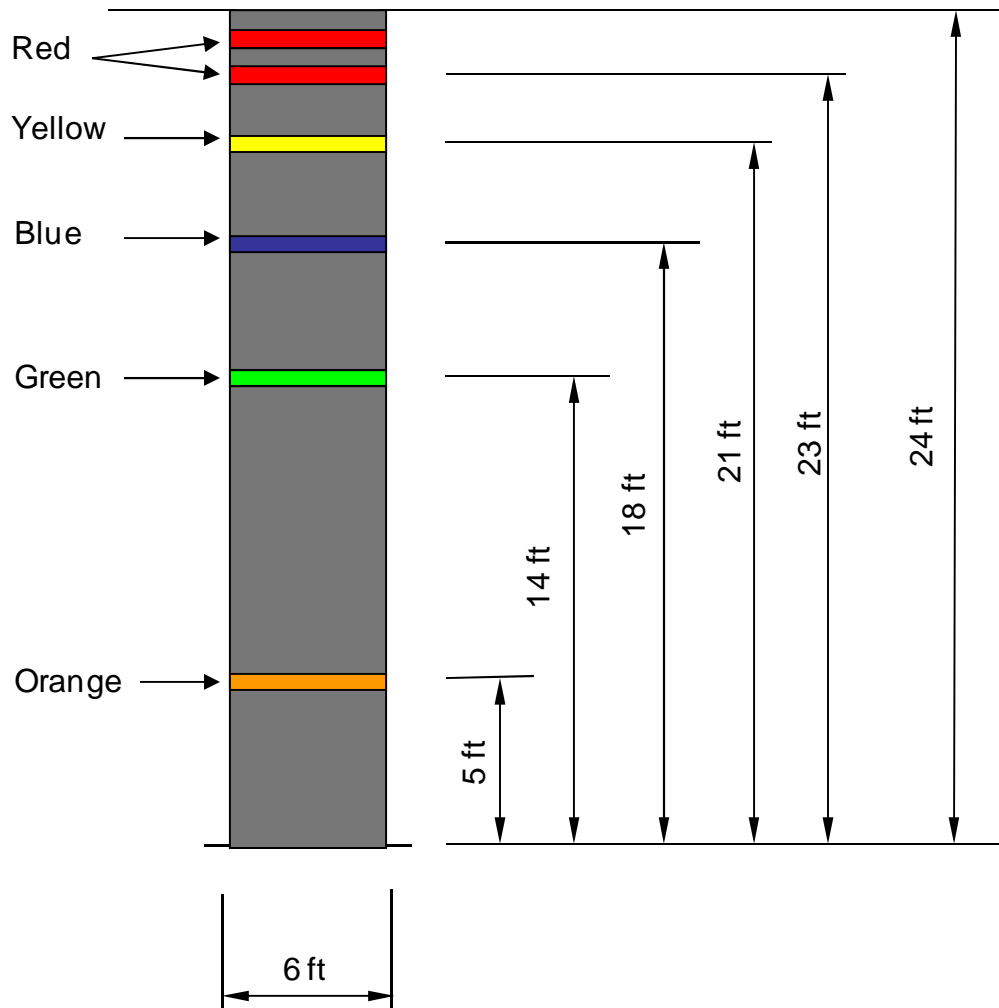
Figure 3.7: Hypothetical cube locations based on laminar flow (adapted by Dachelet 2008)

- Slump test

To measure the consistency of the ODSC, a slump test will be performed on all concrete batches at the time of placement. This test will be conducted as specified in ASTM C 143 (2005). The slump of the ODSC, at the time of placement, must be between six to nine inches to meet the proposed specification (Appendix A). These samples shall be taken from the middle of every truck. Also, to measure the consistency of the concrete over time, this test will be performed every 30 min. for a 6-hr period on

the sample taken from the first truck. To meet the project specifications, the slump shall be no less than four inches after six hours from the time of placement.

- Slump flow test

To measure the consistency of the SCC and SCC-LP shafts, a slump flow test will be performed. This test will be conducted as specified in ASTM C 1611 (2005). The slump flow of the SCC mixtures, at the time of placement, must be 21 ± 3 in. to meet the proposed specification (Appendix A). These samples shall be taken from the middle of every truck. Also, to measure the fluidity of the concrete over time, this test will be performed every 30 min. for a 6-hr period on the sample taken from the first truck. To meet the project specifications, the slump reading must be no less than six inches after six hours from the time of placement.

- Total air content and unit weight

To determine the total air content and unit weight of the concrete, a pressure meter will be used. These tests will be conducted as specified in ASTM C 138 (2005). These tests will be performed on samples from all concrete batches. To meet the project specifications, the air content must be four percent ± two percent (Appendix A). These samples shall be taken from the middle of a single truck.

- Modified J-Ring test

To test the concrete's ability to flow through the reinforcement cage, a modified J-Ring Test will be performed on the SCC and SCC-LP batches at the time of placement. This test will be conducted as specified by ASTM C 1621 (2005). However, the standard J-Ring was considered to congested for drilled shaft applications. The modified J-Ring to be used for this test has 13 bars around the 12-in. diameter ring, instead of the ASTM specified 16 bars. This changes the bar spacing from 1.74 in. to 2.27 in. (Dachelet 2008) This sample shall be taken from the middle of a single truck.

- Segregation column

To assess the static segregation of the concrete, a segregation column test will be performed on the SCC and SCC-LP batches at the time of placement. This test will be conducted as specified by ASTM C1610 (2005), but the wait time will be extended from 10 min. to 1 hr. This additional wait time is used for drilled shaft applications because of the extended placement times typically used for large shafts. This test will be performed on one sample of each SCC and SCC-LP batches at the time of placement. This sample shall be taken from the middle of a single truck.

- Bleed test

To assess the concretes ability to bleed, a bleed test will be performed on a sample of each SCC and SCC-LP batches at the time of placement. This test will be conducted as specified by ASTM C 232 (2005). With this method, the bleeding of a concrete sample is determined at standard atmospheric pressure. This sample shall be taken from a truck single truck for each mixture.

- Pressurized bleed test

Pressure is applied to the fresh concrete placed into a drilled shaft due to the weight of the concrete and the weight of water, or slurry, above. This test method is being developed to assess the concrete's ability to bleed under this pressure. The data were collected to assist with the development of this new test method. This test will be performed on a sample of each SCC and SCC-LP batches at the time of placement.

This test is based on a forced bleed test, developed by Khayat, to test the ability of grout to bleed in pre-stressed applications (Khayat and Yahia 1997). To perform this test a sample of concrete is placed in a 6-in. diameter by 12-in. tall piston chamber. The SCC is then poured into the chamber using one steady motion. The top of the chamber is then struck off to remove any excess concrete. Next, the cap is placed on the chamber. This chamber cap has a metal screen filter, filter paper, and a steel plate with

67

holes to prevent the concrete and paste from leaving the chamber. A picture of the piston cap is presented in Figure 3.8. The bottom of the chamber is a piston that is actuated by a rubber air spring. This air spring is pressurized with an adjustable air-compressor. A picture of the pressurized bleed test chamber is presented in Figure 3.9. The assembled pressurized bleed test is presented in Figure 3.10.

Before the air compressor is connected to the apparatus, water is added to the beaker located on the top of the cap to fill the air voids located in the cap. This water is added until water begins coming out of the bleed valve (located next to the beaker). Once air stops exiting the bleed valve, this valve is shut and the amount of water in the beaker is recorded. The air compressor is then attached to the apparatus and slowly turned up to 30 psi. The chamber is kept at this pressure for 30 min. taking readings every 10 min. After 30 min. the pressure is increased to around 75 psi for 30 min. and readings are recorded every 10-min. The increase and wait is then continued for pressures of 165, 240, and 300 psi taking readings every 30 min. and waiting for 60 min. before increasing the pressure. The intent was to have the pressures at 10, 25, 55, 80, and 100 psi; however, it was later discovered that the pressure being applied to the air-spring was one third the pressure experienced in the chamber.

- Concrete set time

To test the time for the concrete to reach its initial and final set times, the penetration resistance test will be used on a concrete sample from the first truck of each shaft. This test will be performed in accordance with ASTM C 403 (2005) from a sample of mortar wet-sieved from each concrete mixture.

Figure 3.8: Piston cap with metal filter, filter paper, and a metal plate with holes to

prevent aggregate and paste from leaving the piston chamber

Figure 3.9: Pressurized bleed test chamber and air compressor

Figure 3.10: Constructed pressurized bleed test apparatus

3.2.4 Assessment of Hardened Concrete Behavior

Hardened concrete properties will be determined from concrete samples taken from the third truck of each shaft placement. The following properties will be assessed:

- Compressive strength and elastic modulus

To assess the compressive strength and modulus of elasticity of the concrete mixtures, three, 6-in. diameter by 12-in. molded specimens will be cast and tested per testing age for each mixture. The compressive strength shall be tested in accordance with ASTM C 39 (2005). The modulus of elasticity shall be tested in accordance with ASTM C 469 (2005). The curing of the specimens will be conducted in accordance with ASTM C 31. The specimens will be removed from the molds no earlier than two times the initial set time. The specimens will be tested at ages of 7, 28, 56, and 91 days.

After being cast, the samples will be placed into a temperature controlled water-filled curing tank located in a trailer at the jobsite. These cylinders will be moved to the

Auburn University curing room at the conclusion of the field project. The test will be

conducted by the author in the Auburn University concrete testing laboratory.

- Drying shrinkage

To assess the shrinkage of the concrete mixtures, three, 3-in. by 3-in. by 12-in.

molded specimens will be cast per mixture. These specimens will be tested in

accordance with ASTM C 157 (2005). The specimens, known as shrinkage prisms, will

be removed from the molds no earlier than two times the initial set time. The shrinkage

prisms will be placed in a lime-saturated bath for the first seven days, as specified by the

ASTM specification. This bath will be located at the jobsite, but will be moved to Auburn

University at the completion of the field project. Afterwards, the specimens will be

removed from the lime bath and placed in air storage at Auburn University. The length

of the specimens will be measured at 1, 2, 3, 7, 14, 28, 56, 91, 180, and 365 days after

removal from lime-saturated water bath.

- Resistance to chloride ion penetration

To assess the concrete's ability to resist chloride ion penetration, three, 4-in.

diameter x 8-inch molded specimens will be cast per testing age for each mixture.

These samples will be tested in accordance with ASTM C 1202 (2005). The specimens

will be removed from the molds no earlier than two times the initial set time. The

specimens will be put into water filled curing tanks, located at the jobsite, after casting.

At the conclusion of the field project, these cylinders will be moved to the Auburn

University laboratory where the mold will be removed and the specimens will be placed

in the curing room. The cylinders will be cut to 2-in. slices within a week of testing. The

2-in. specimens shall be cut using a water-cooled diamond saw and a sanding block

shall be used to smooth blemishes around the circumference on the sample. The

testing will be conducted by the author in the Auburn University concrete testing

laboratory. The specimens will be tested 91 and 365 days after casting.

3.2.5 Placement Monitoring

To directly assess the ability of the concrete mixtures to flow through the reinforcement cage, the elevation difference between the inside and outside of the steel reinforcement cage will be measured periodically during concrete placement. This monitoring will be conducted by the use of plumb-bobs attached to a nylon measuring tape.

3.2.6 Assessment of Shaft Integrity

To assess the quality of the in-place concrete, cross-hole sonic logging (CSL) will be performed when concrete had exceeded an age of seven days. Six metal tubes, with inside diameters of approximately 1.75 in., will be attached to the transverse reinforcement to provide access for CSL testing.

3.2.7 Assessment of In-Place Concrete Properties

To assess the in-place concrete properties, the shafts will be removed, cut, cored and visually examined. All shafts will be exhumed at an age no earlier than 28 days after placement. The exhumed shafts are to be laid on their sides to allow a longitudinal cut to be made down the center of each shaft. After cutting, the surface of this longitudinal cut will be pressure washed, allowed to dry, and shellacked to allow visual examination of the in-place concrete and analyze the location of the colored mortar cubes. One-half of the longitudinal slice will then be cut at its cross-section, either at locations 7 ft and 20 ft from the top of the shaft, or at a location deemed important to analyze further. Visual assessment of aggregate distribution, colored mortar cube location, and defects will be conducted for each cross-sectiosn. A diagram of the expected cut planes and cube locations is presented in Figure 3.11.
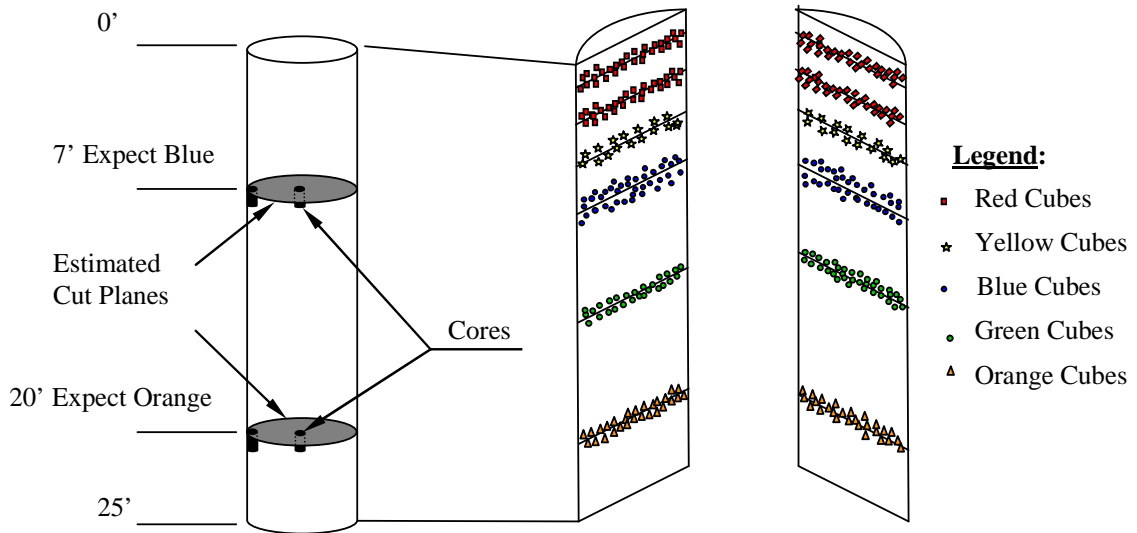
Figure 3.11: Cutting and coring locations of exhumed shafts (Dachelet 2008)

To assess the properties of the in-place concrete, cores will be taken from the cross-sectional cuts. The cores will be taken from locations in the shaft cover region (between the shaft surface and the steel reinforcement cage) and near the center of the shaft for each cross-sectional cut. Six cores will be tested from each elevation to determine the in-place concrete's hardened concrete properties (compressive strength, modulus of elasticity, and permeability). A diagram of the proposed core locations for each cross section is presented in Figure 3.12.
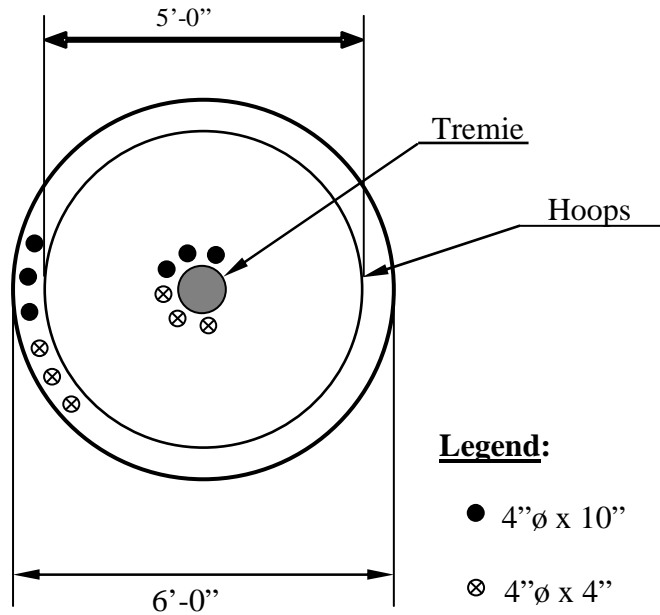
Figure 3.12: Coring detail of exhumed shafts (Dachelet 2008)

- Compressive strength and elastic modulus

To determine the compressive strength and elastic modulus of the in-place concrete, three core specimens, 4-inch diameter x 10-inch (Testing Size: 4-inch diameter x 8-inch), will be acquired. The compressive strength will be tested in accordance with ASTM C39 (2005) and the modulus of elasticity will be tested in accordance with ASTM C469 (2005). These cores will be taken from the bottom side of each cross-sectional cut. After exhuming, the concrete samples will be placed into sealed plastic bags. Each bag will be sealed in another bag so as to ensure an air-tight seal. The outer bag will then be taped shut and labeled. The samples will be removed from the bags no earlier than two days before testing. The samples will be sliced using a water-cooled diamond saw to a length of 8-in. These cut samples will then be put back into plastic bags and prepared by capping with sulfur mortar in accordance with ASTM C 617 (2003). The capped samples will be placed back into the plastic bags for

at least two hours before testing. The cores shall be tested 56 days after shaft

placement.

- Permeability

To assess the permeability of the in-place concrete, three core specimens, 4-in.

diameter X 4-in. disks (Testing Size: 4-in. diameter X 2-in. disks), will be acquired for

testing in accordance to ASTM C 1202 (2005). These cores will be taken from the

bottom side of each cross-sectional cut. After exhuming, the concrete samples will be

placed into sealed plastic bags. Each bag will then be sealed in another bag so as to

ensure an air-tight seal. The outer bag will then be taped shut and labeled. The

samples will be removed from the bags no earlier than two days before testing. The

samples will be sliced using a water-cooled diamond saw to a length of 2 in. and a

sanding block used to smooth blemishes around the circumference on the sample. The

cores will be tested 91 days after shaft placement.

3.3 Materials and Mixture Properties for Test Shafts

The Ordinary Drilled Shaft Concrete (ODSC) mixture is the standard mixture the

Alabama Department of Transportation (ALDOT) is currently using in their drilled shafts

on the Scottsboro bridge project. Two SCC mixtures were designed by Auburn

University for evaluation. The first SCC mixture is designated as SCC in this report. The

second mixture includes limestone powder in the mixture; therefore, this mixture is

designated as SCC-LP. The mixture proportions for each concrete are presented in

Table 3.1.

Table 3.1: Concrete Mixture Proportions

Item	Mixture Type		
	Conventional Alabama DOT (ODSC)	SCC	SCC-LP
Type I cement content (lb/yd^3)	605	473	473
Class F fly ash content (lb/yd^3)	145	202	135
Water Content (lb/yd^3)	292	268	270
No. 67 coarse aggregate, SSD (lb/yd^3)	1,875	0	0
No. 789 coarse aggregate, SSD (lb/yd^3)	0	1,505	1,548
Fine aggregate content, SSD (lb/yd^3)	1,050	1,462	1,493
Limestone Powder (lb/yd^3)	0	0	69
Water-to-cementitious material ratio	0.39	0.40	0.44
Sand-to-total aggregate ratio	0.36	0.49	0.49
Water Reducer / Retarder admixture (oz/cwt)	4	0	0
High-Range Water Reducer admixture (oz/cwt)	0	10	8.5
Hydration-Stabilizing admixture (oz/cwt)	0	2.5	2.5
Air-entraining admixture	0.5	0	0

- Type I portland cement: The portland cement used for this project was manufactured by National Cement Co. in Ragland, Alabama. This cement is a general purpose cement commonly used in general construction as well as drilled shaft construction. The chemical composition of this cement is presented in Table 3.2.

Table 3.2: Chemical composition of the cement

Chemical Analysis	
Item	Result (%)
SiO_2	21.08
Al_2O_3	4.36
Fe_2O_3	3.64
CaO	64.87
MgO	2.76
SO_3	2.86
LOI	1.32
Na_2OEq	0.54
Insoluble Residue	0.35
CO_2	0.79
Limestone	1.88
$CaCO_3$ in limestone	95.68
Potential Compounds	
C_3S %	57.1
C_2S %	17.3
C_3A %	5.4
C_4AF %	11.1
$C_4AF+2(C_3A)$	21.9
$C_3S+4.75C_3A$	82.8

- Class F fly ash: The class F fly ash used for this project was provided by SEFA, Inc. and was manufactured in Cumberland, Tennessee. Fly ash is a by-product formed from the burning of coal. This ash is a fine material than provides cementing properties when mixed with water and cement. This material is less expensive than portland cement and is required by most industries and state Department of Transportation's because of its cementing and filling benefits, as well as its sustainability, since it would otherwise be placed in a landfill.

- Coarse aggregate: The coarse aggregate used for this project was quarried by Vulcan Materials Co. in Scottsboro, Alabama. The gradations used for this project were No. 67 for the ODSC mixture and No. 78 for the SCC and SCC-LP mixtures. The nominal maximum aggregate size for the No. 67 and No. 78

gradation is 0.75 in. and 0.5 in., respectively. The smaller aggregate size was selected for the SCC and SCC-LP mixtures in order to increase the flowing and passing ability of the concrete.

- Fine aggregate: The fine aggregate used for this project was supplied by Madison Materials in Summit, Alabama.

- Limestone powder: Betocarb®, an OMYA product, was used as an additional powder in the SCC-LP mixture. This product is a finely ground limestone powder (2-10 micron diameter) that is put into the mixture design to minimize the bleed water within the concrete. The use of this material was also expected to increase the mixtures resistance to segregation (Khayat et al. 2006).

- Water reducing / retarding admixture: The ODSC mixture used WRDA® 64. WRDA® 64 is a water reducer that was dosed in order to act as a set retarder as well as a water reducer. This admixture is a polymer based aqueous solution.

- High-range water reducing (HRWR) admixture: The SCC and the SCC-LP mixtures used ADVA® 380 as the HRWR admixture. HRWR admixtures are also known as superplasticizers and work the same as any water reducer, but with a much higher potency.

- Hydration-stabilizing admixture: Recover® was used as the Hydration Stabilizing admixture in both the SCC and the SCC-LP mixtures. This admixture extends the time for the concrete to reach its initial set.

- Air-entraining admixture: Daravair® 1000 was used as the air-entraining admixture in the ODSC mixture. Air-entraining admixtures are used to chemically entrain small pockets of air voids in the concrete mixtures.   These entrained air voids enhance the durability of the concrete, increasing its resistance to freeze-thaw cycles in the environment.

3.4 Overview of Construction

The following section gives an overview of the construction process used to construct and test the test shafts.

3.4.1 Shaft Condition upon Arrival of Research Staff on Site

During August 11, 2008 through August 13, 2008 concrete was placed into the three test shafts. A different concrete mixture was placed into a different shaft on each day. Upon the arrival of the Auburn University personnel the shafts were already excavated. A corrugated steel casing was placed into each hole (instead of the proposed sono-tube). Approximately one foot of concrete was already placed within each test shaft as presented in Figure 3.13.

The test shafts were located on the top of a hill in the median of AL-35. The open shafts were approximately 15 ft apart, in a line running east to west. The easternmost shaft was designated the ODSC shaft and was filled with concrete first. The middle shaft was designated the SCC shaft and was filled second. The final shaft filled was the westernmost shaft, and this shaft was designated the SCC-LP shaft.

3.4.2 Steel Reinforcement Cages

Each shaft had identical steel reinforcement cages. Each cage consisted of twenty-six, No. 11 bars running longitudinally, six CSL tubes equally spaced, and No. 4 hoops at 4-inch spacing. The CSL tubes had an inside diameter of approximately 1.75 in. Four of the No. 11 bars were threaded and extended a few feet above the shaft. These bars were located across the shaft from one another and were used for exhuming the shafts once they were cured. A diagram of the cross-section, showing the reinforcement cage, is presented in Figure 3.14.
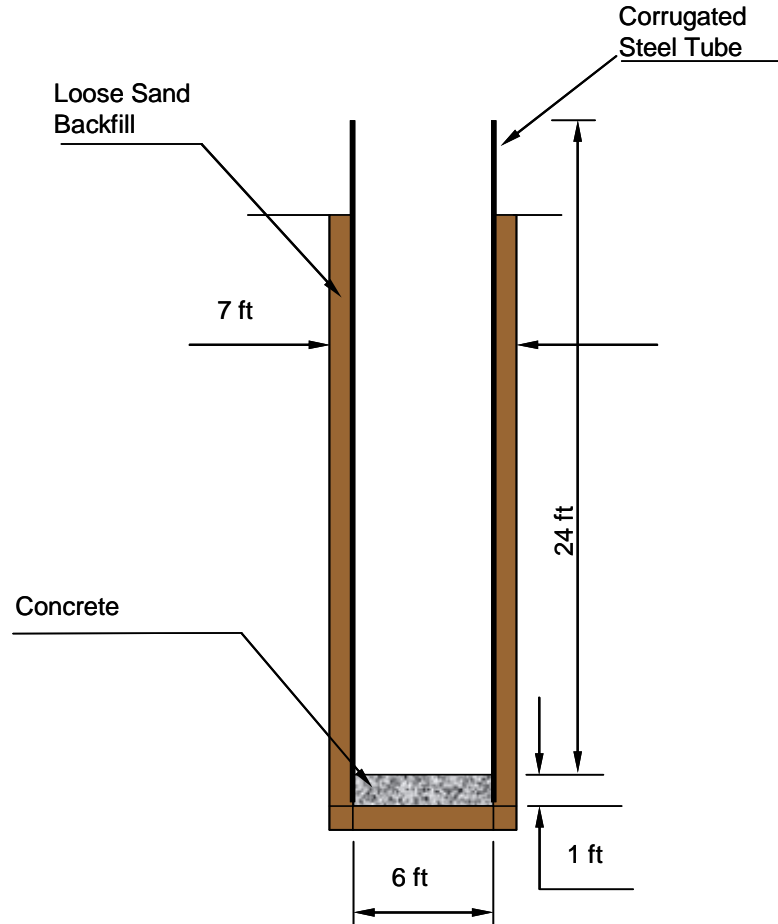
Figure 3.13: Schematic of open shaft before test concrete placement

3.4.3   Slurry Mixing

The test shafts were neither below the water table nor capable of collapsing; however, drilling slurry was used to simulate the placement methods used in current production drilled shafts.

For this project Poly-Bore[TM] polymer slurry was used. This is a dry powder-like substance that, when added to water, becomes a viscous fluid that is used for bore-hole stabilization. The dosage used was approximately one pound per 100 gallons of water. This slurry was used for each shaft. While the concrete was being placed into one shaft the slurry was being pumped into the next shaft. During the concrete placement of the last shaft, the slurry was pumped into a container for disposal.
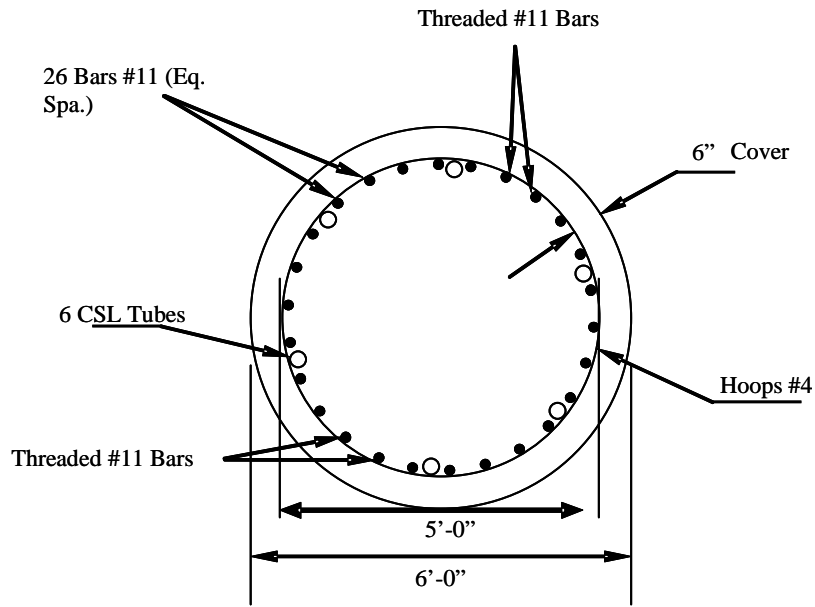
Figure 3.14: Cross section of shaft

3.4.4   Addition of Sand and Shale

In order to make the concrete placement more realistic, imperfections such as sand and pieces of shale were added to the shafts. The shale pieces were dropped from the surface into each shaft at random locations throughout the pour. Five gallons of sand (approximately 0.5% by volume) was placed into the slurry while the slurry was mixing in the first shaft prior to any concrete placement. Sand was not added to the slurry at any other time during the concrete placement.

3.4.5   Overview of the Ordinary Drilled Shaft Concrete (ODSC) Placement

This shaft was poured on August 11, 2008. The average temperature for the day was 74.2 °F with a maximum of 89.6 °F. The skies were clear to partly cloudy (Yankee Publishing Inc. 2009).

The pour was delayed because a water truck was not present to mix the dry slurry. A water truck was brought from Birmingham, AL. After the slurry was sufficiently mixed in the shaft, the concrete batch plant was notified to send the first truck.

A concrete pump truck was utilized to place the concrete within each of the drilled shafts. This truck was parked on the hill at an elevation slightly lower than the top of the drilled shafts. The steel reinforcement cage was lowered into the shafts using a crane. A picture of the pump truck and crane is presented in Figure 3.15. The end of the pump line was attached to an 8-in. diameter straight steel pipe, referred to in this paper as the tremie. The bottom of this tremie had a cut out on its side to allow the concrete to initially flow out of the tremie while the tremie is firmly placed on the bottom of the shaft. A picture showing the bottom of the tremie is presented in Figure 3.16. A foam plug was fed into the pump line before the concrete was pumped to prevent the concrete from mixing with the slurry in the shaft while traveling through the tremie.

Figure 3.15: Pump truck location on hill with the drilled shafts

Figure 3.16: Bottom of tremie pipe

The first concrete truck arrived on site at approximately 4:15 p.m. As planned, the fresh concrete property tests (i.e., slump, air content, unit weight, and temperature) were performed. The first truck finished placement at 4:54 p.m.

The second truck arrived on site at approximately 4:50 p.m. All concrete cylinders and shrinkage prisms were made from a concrete sample taken from this truck. Auburn University personnel tested the fresh concrete properties of the concrete in this truck, acquired a sample to determine the setting times, and conducted a slump retention test on a concrete sample from this truck. The second truck finished placement at 5:18 p.m.

The third and final truck arrived on site at approximately 5:50 p.m. Placement of the final truck was concluded at 6:05 p.m. Batch and placement times are summarized Table 3.3.

Table 3.3: ODSC batch and placement times

Truck No.	Batch Time	Placement Time	
		Start	End
1	4:00 p.m.	4:15 p.m.	4:54 p.m.
2	4:32 p.m.	4:50 p.m	5:18 p.m
3	5:33 p.m.	5:50 p.m.	6:05 p.m.

During concrete placement the tremie was not manually moved, but it rose throughout the placement due to the force of the concrete being discharged from its bottom end. The tremie rise was recorded and is presented in Figure 3.17.

3.4.6 Overview of the Self-Consolidating Concrete (SCC) Placement

The SCC was placed into its shaft on August 12, 2008. The average temperature for the day was 71.5 °F with a maximum of 82.4 °F. The skies were cloudy with an occasional light rain shower (Yankee Publishing Inc. 2009)

On this date, the concrete placement was delayed due to the arrival time of the water truck and the slurry test kit. The slurry test kit measures the thickness and viscosity of the slurry mixture (O'Neil and Reese 1999). ALDOT personnel recorded the results from this test. These results have not yet been acquired by the author and appear to have been misplaced.

The first truck was batched at 11:30 a.m., but was rejected because the mixture lacked sufficient filling ability due to its low slump flow. Subsequent truck arrival and placement times are summarized in

Table 3.4. The same concrete placement process as the ODSC shaft was used for this shaft.
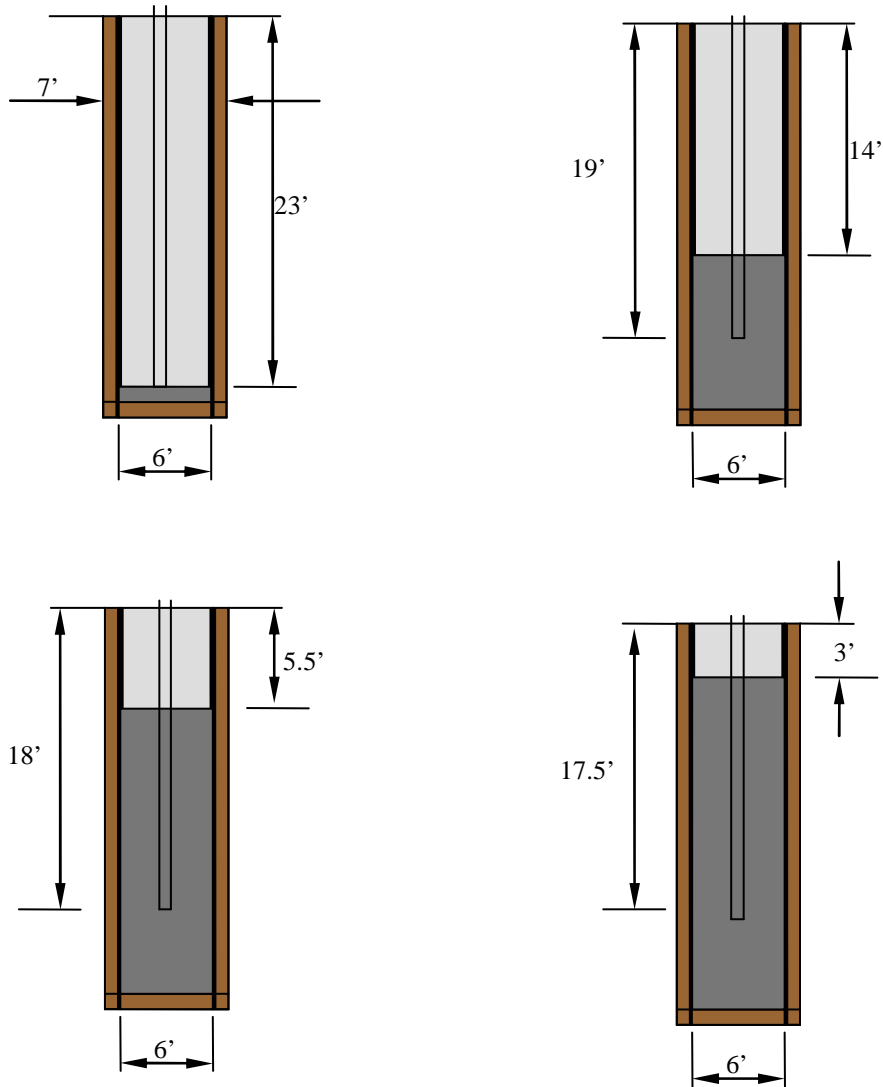
Figure 3.17: ODSC tremie movement

Table 3.4: SCC batch and placement times

Truck No.	Batch Time	Placement Time	
		Start	End
1	12:29 p.m.	12:50 p.m.	1:37 p.m.
2	1:45 p.m.	2:25 p.m.	2:49 p.m.
3	2:30 p.m.	3:15 p.m.	3:20 p.m.
4	3:20 p.m.	4:00 p.m.	4:05 p.m.

For the SCC, a slump flow test was used instead of the standard slump test.

During the slump flow test the stability and viscosity of the mixture were also determined.

These properties were determined by running the Visual Stability Index (VSI) test and the T_{50} test. Both of these tests are described in the appendix of ASTM C 1611.

All concrete cylinders and shrinkage prisms were made from a concrete sample from the second truck. Auburn University staff tested the fresh concrete properties of this truck, and acquired a sample to determine the concrete's setting times. The following tests were also performed on a sample of concrete from this truck:

- Bleed Test,

- Pressurized Bleed Test,

- Segregation Column,

- Modified J-Ring, and

- Slump Flow Retention.

The third truck did not completely fill the shaft, so a fourth truck was ordered with a three cubic yard load to finish the placement. ALDOT personnel performed all the fresh concrete testing for the third and fourth truck.

During concrete placement, the tremie was not manually moved, but it rose throughout the pour due to the force of the concrete being discharged from its bottom end. The tremie's rise was recorded and is presented in Figure 3.18.

3.4.7   Overview of the SCC with Limestone Powder Placement

The self-consolidating concrete with limestone powder (SCC-LP) was placed on August 13, 2008. The average temperature for the day was 73.2 °F with a maximum of 86.0 °F. The skies were cloudy with an occasional light rain shower (Yankee Publishing Inc. 2009).
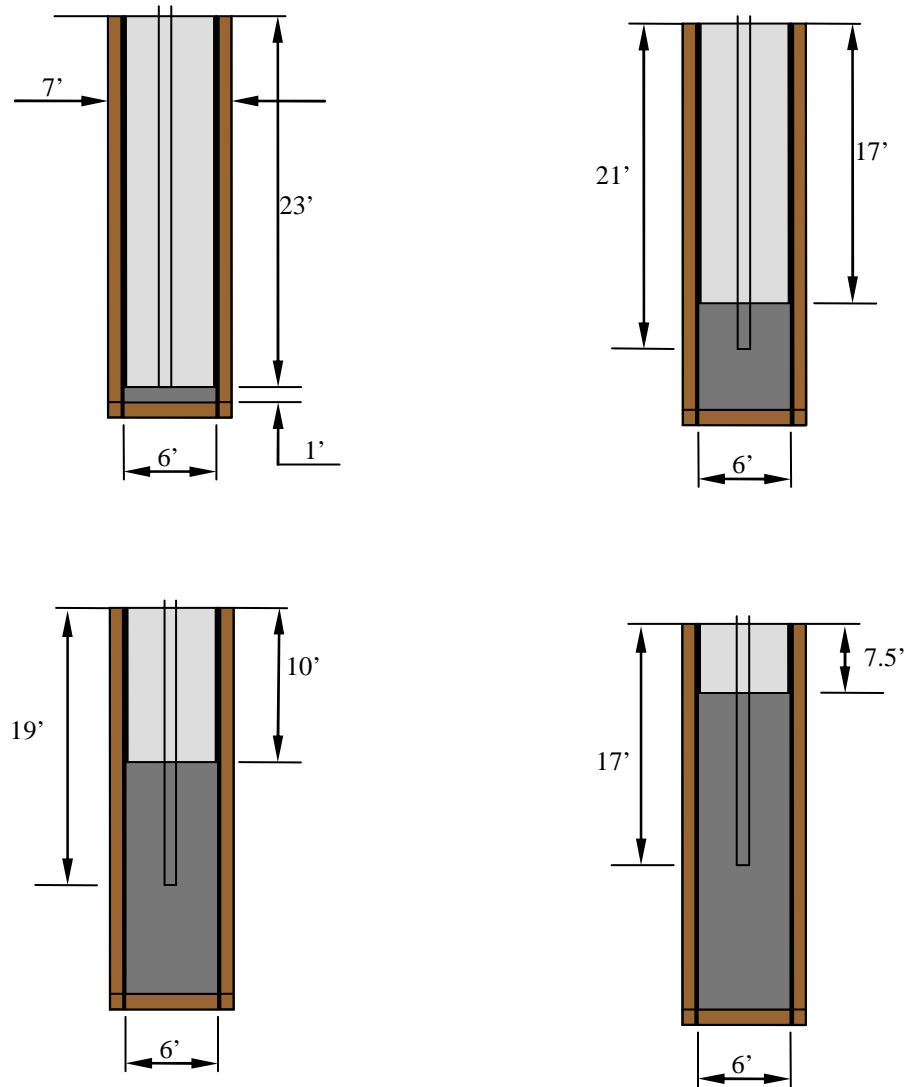
Figure 3.18: SCC tremie movement

The first truck was batched at 9:35 a.m. with only three and a half cubic yards of concrete. However, placement of this truck was delayed until much later due to a decision made in the field to wait until the second truck was on its way. The second truck batched was rejected because the slump flow was considerably lower than acceptable. The third truck to arrive on site, now designated as Truck No. 2, required four attempts to get the concrete mixture's slump within the specified range. The fourth truck to arrive on site, now designated as Truck No. 3, had the same slump flow problem

as those experienced by Truck No. 2. Arrival and pour times are summarized in Table

3.5. It should be noted that no trial batches were made for the SCC-LP mixture and this

is probably why difficulties were experienced to produce this mixture.

Table 3.5: SCC-LP batch and placement times

Truck No.	Batch Time	Placement Time	
		Start	End
1	9:35 a.m.	11:55 a.m.	12:02 p.m.
2	11:15 a.m.	12:04 p.m.	2:12 p.m.
3	12:35 p.m.	2:00 p.m.	2:20 p.m.
4	2:05 p.m.	4:05 p.m.	4:15 p.m.

All concrete cylinders and shrinkage prisms were made from a concrete sample

obtained from Truck No. 3. Auburn University staff tested the fresh concrete properties

of this truck, and acquired a sample to determine the concrete setting times. The

following are the other tests performed on a sample of concrete from this truck:

- Bleed Test,

- Pressurized Bleed Test,

- Segregation Column,

- Modified J-Ring, and

- Slump Flow Retention.

The concrete plant did not have enough limestone powder to compete the shaft, and

consequently was not able to mix any limestone powder into the last concrete truck. The

SCC mixture design was used for this truck.

During concrete placement the tremie was purposely not moved, but it rose

throughout the pour due to concrete being discharged from its bottom end. The tremie

rise was recorded and can be presented in Figure 3.19.

3.4.8 Addition of Cubes

The time at which the cubes were placed was determined by the estimated amount of concrete placed in each shaft. The placement of the cubes relative to the tremie location for each shaft is presented in Figures 3.20, 3.21, and 3.22.

3.4.9 Assessment of Concrete's Ability to Flow through the Reinforcement

During construction, the elevation difference between the outside and inside of the reinforcement cages was carefully recorded. The recorded elevations plotted against shaft depth are summarized in Figure 3.23. The measured differences between the cover elevation and interior elevation are summarized in Figure 3.24.

The placement of the ODSC concrete occurred in the least amount of time. The only delay was an intentional one-hour long delay that occurred after the second truck. The SCC concrete placement took approximately two hours with one intentional delay. This delay was longer than one hour and occurred after the first truck. The SCC-LP concrete placement occurred over the longest time period with extended time delays between the concrete trucks. The delays with the SCC-LP shaft were not purposeful. These delays were due to difficulties in getting the fresh concrete properties within the specification limits at the plant and due to a lack of trucks available for this project.

Before a concrete truck completely emptied, the pump truck operator would stop pumping to keep the hopper of the pump truck filled with concrete. During the delays between concrete trucks, the pump truck operator would pump this excess concrete very slowly into the tremie so as to prevent clogging of the pump line. The effects of this slow pumping can be seen in Figure 3.23 as the parts in the graph where there is a slight increase in elevation over a time of 30 minutes or more.
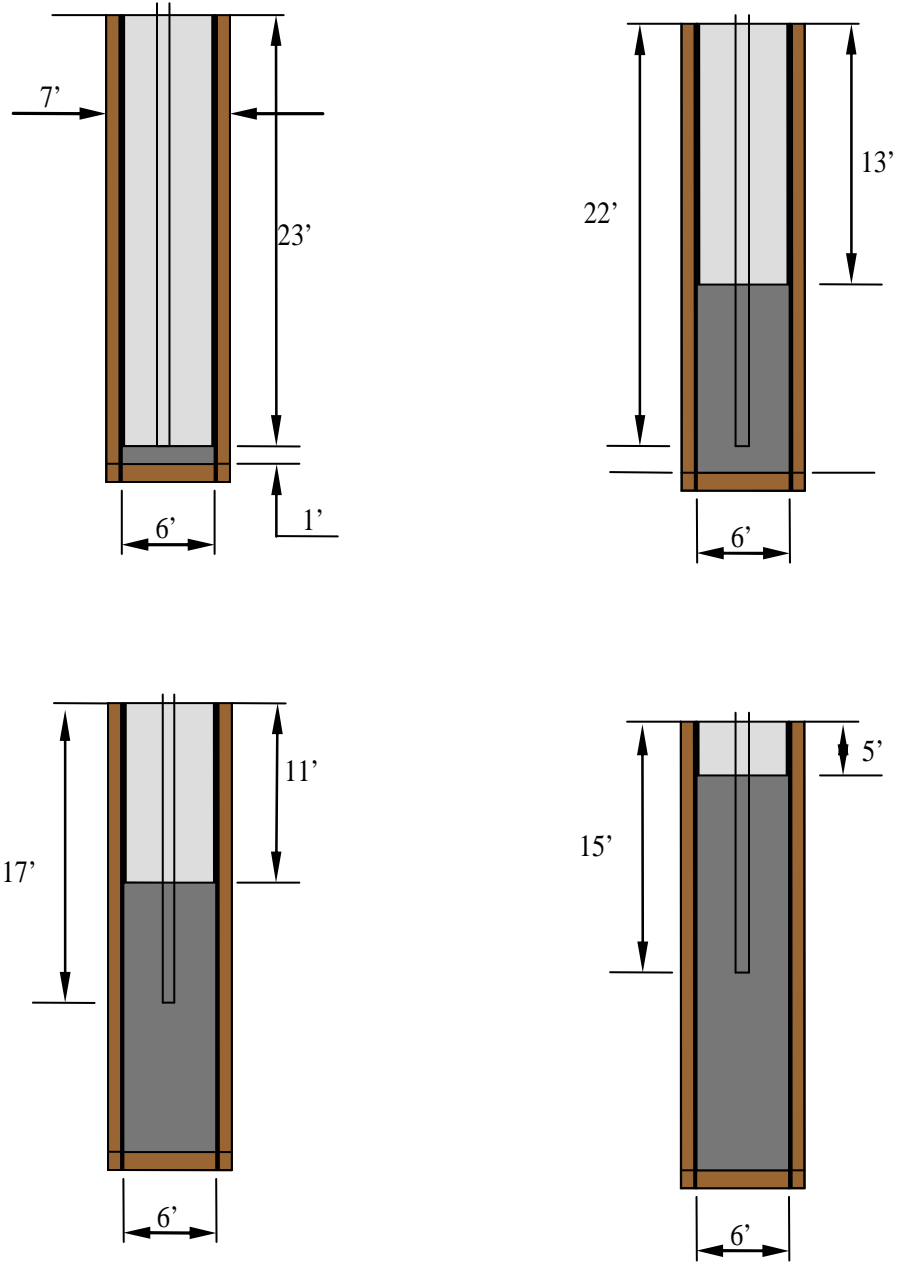
Figure 3.19: SCC-LP tremie movement
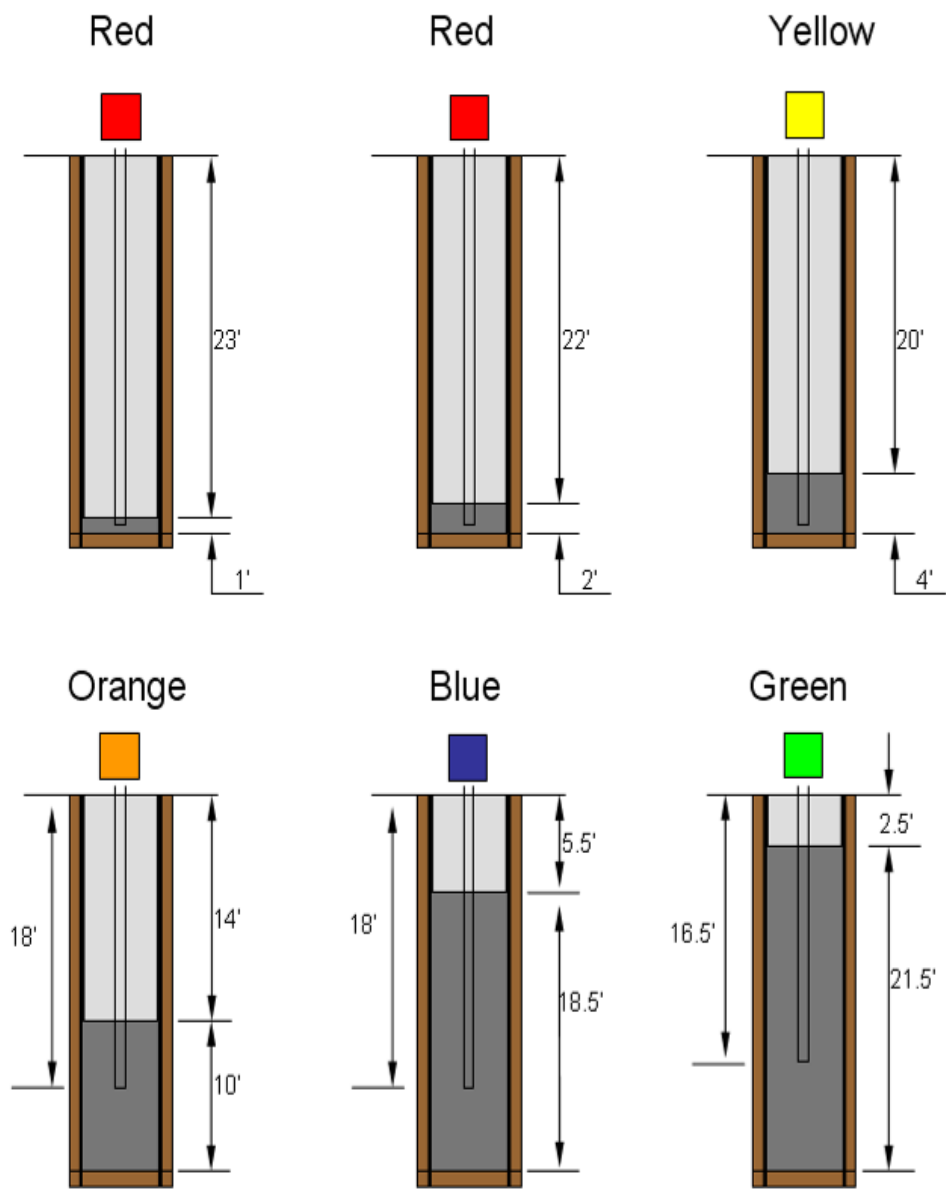
Figure 3.20: Addition of cubes during ODSC shaft placement

Red — 23', 1'

Red — 22', 2'

Yellow — 20', 4'

Orange — 21', 14', 10'

Blue — 19', 5.5', 18.5'

Green — 17', 2.5', 21.5'
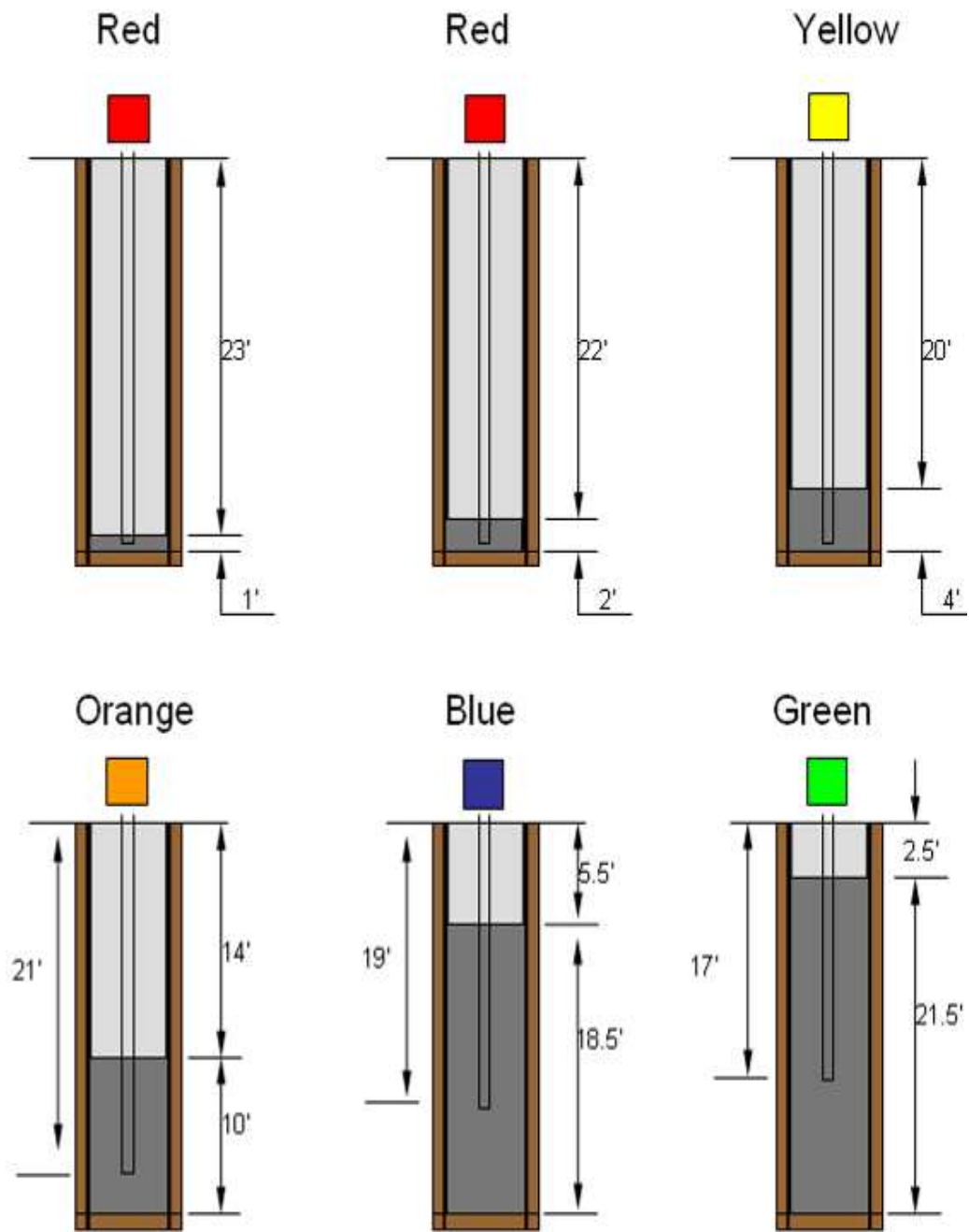
Figure 3.21: Addition of cubes during SCC shaft placement
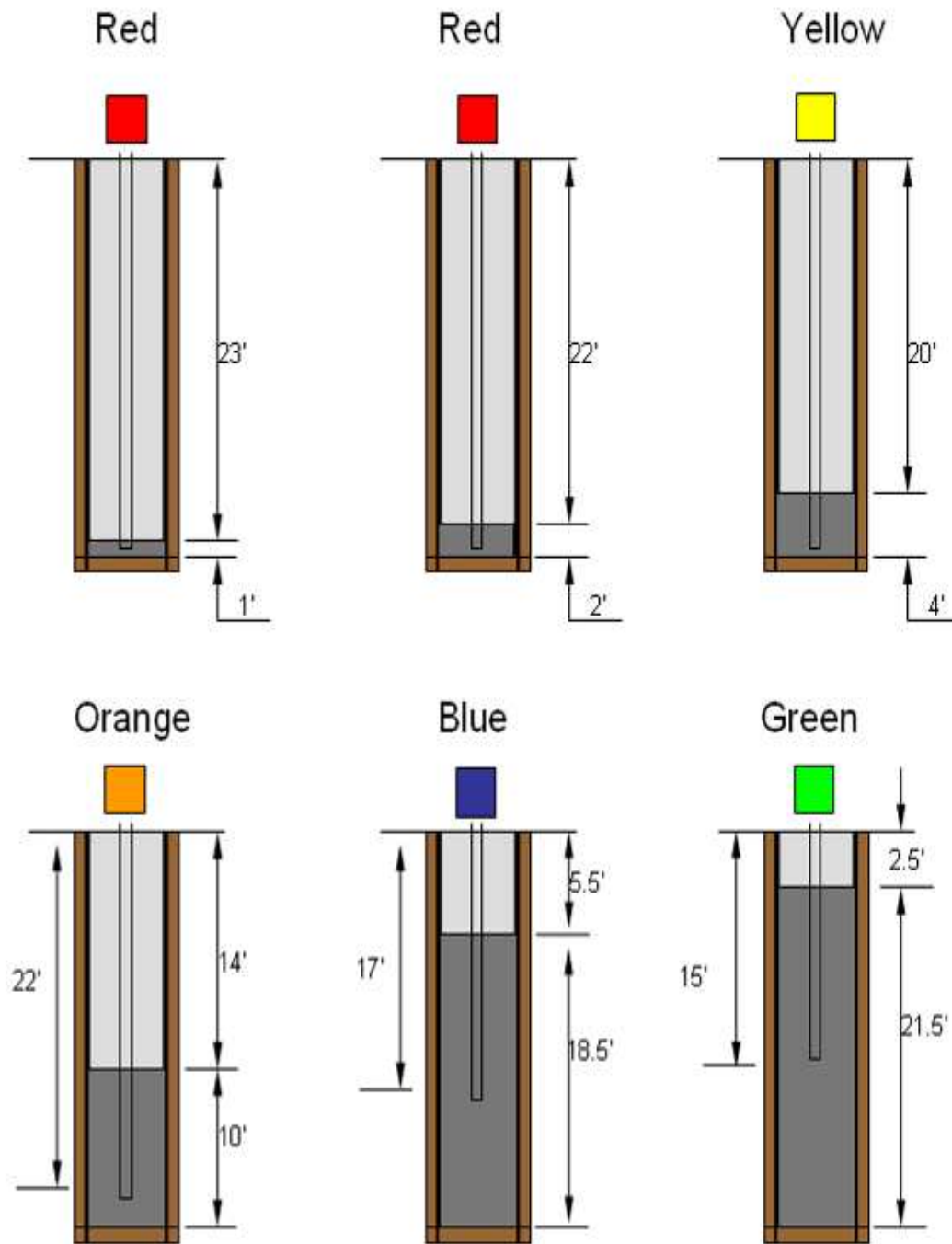
Figure 3.22: Addition of cubes during SCC-LP shaft placement

In the ODSC and SCC-LP shafts, the differences between the inside and outside of the reinforcement cage were as high as 12 in. and 11 in., respectively. In the SCC

shaft this difference was limited to 4 in., indicating that the concrete in the SCC shaft maintained a more uniform elevation that the ODSC shaft. By flowing more uniformly upwards, the SCC is less likely to form voids, honeycomb, or entrap floating debris during the concrete placement (Brown 2004).

This finding matches the conclusion determined from Hodgson et al. (2003), where the ordinary drilled shaft concrete had a measured difference as high as 18.4 in. and the SCC had a maximum measured difference of only 4 in. It was concluded in this study that this uniform upward flow of the SCC should prevent debris from being entrapped against the side of the shaft (Hodgson et al. 2003).
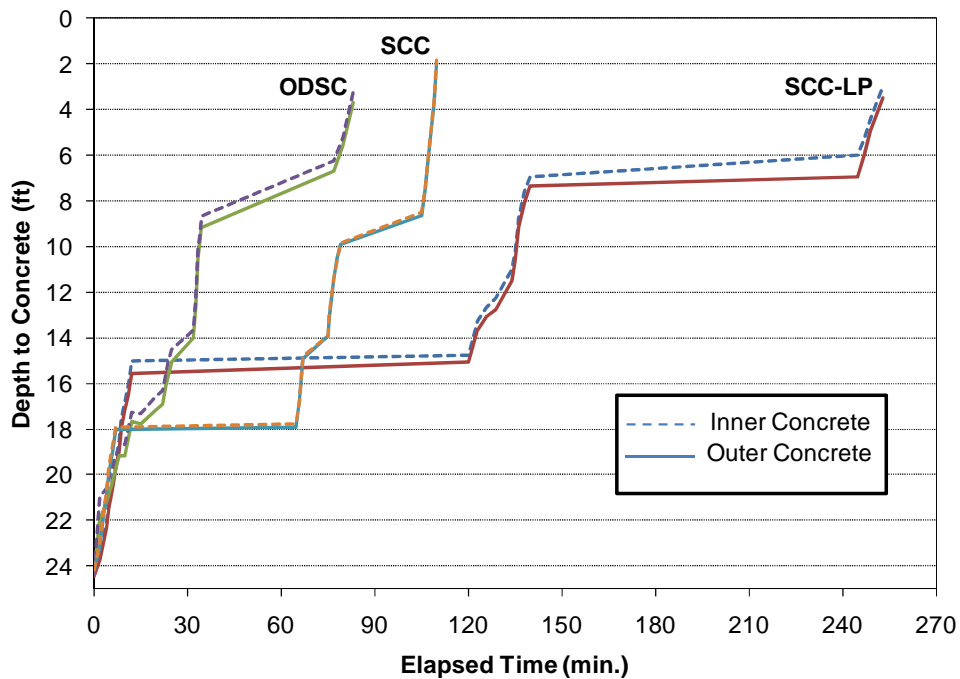
Figure 3.23: Concrete height measured throughout the pour
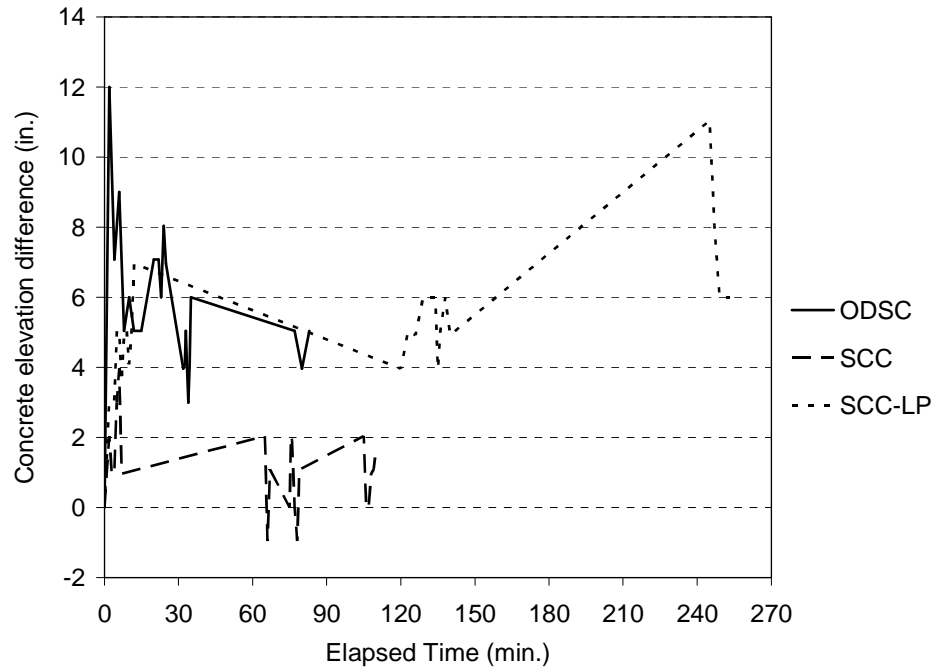
Figure 3.24: Elevation differences during concrete placement

3.4.10 Shaft Integrity Testing

Crosshole sonic logging (CSL) of the test shafts was conducted on September 8, 2008 (four weeks after placement). The CSL test set-up is presented in Figure 3.25. The CSL tubes were filled completely with water. A hydrophone was lowered down one tube, while a receiver was lowered down another tube. Once both devices were lowered to the bottom of the shaft they were pulled up at a constant rate. An ultra-sonic pulse was sent from the geophone to the receiver approximately every 0.2 ft of rise (Robertson and Bailey 2008). The time for the pulse to start from the geophone and end at the receiver was measured and divided by the distance between the two devices. This calculation approximately determines the wave velocity of the pulse through the material. This wave velocity was used to distinguish the integrity of the concrete material between the access tubes.

Figure 3.25: CSL testing conducted on-site

3.4.11 Exhuming of Test Shafts

On September 10, 2008 the test shafts were exhumed. In order to remove the shafts, a crane assisted workers to attach a steel frame to the threaded reinforcement bars.   Pressurized water was then used to remove the loose sand that was located outside the steel casing of each shaft. This was performed by using a long steel rod with holes throughout. This rod was pushed into the ground outside of the steel casing, and then the high-pressure water was used to "blow" the loose material away from the in-place shaft. This removal process is presented in the Figure 3.26. After the loose material was removed, a crane was utilized to raise the shaft as presented in the Figure 3.27. After removal from the ground, each exhumed shaft was set on its side as presented in Figure 3.28.

Figure 3.26: Loose sand removal with pressurized water

Figure 3.27: Exhuming of ODSC shaft

Figure 3.28: Exhumed shafts ready for cutting

3.4.12 Cutting and Coring of Test Shafts

Cutting and coring were done from October 28 through November 4, 2008 The SCC shaft was cut and cored first, followed by the ODSC shaft, and finally the SCC-LP shaft.

A diamond wire was used to perform the cuts. Three cuts were made on each shaft: one complete longitudinal cut and two cross-sectional cuts across one side of one longitudinal section. These cuts are presented in Figures 3.29 and 3.30.

The cross-sectional cuts were made at approximately seven foot and twenty foot from the top of the shaft, respectively. Six cores were obtained from the cross section outside the reinforcement cage and another six cores were removed from the cross section inside the reinforcement cage. Coring of SCC shaft at approximately seven feet from the top of the shaft is presented in Figure 3.31.

Figure 3.29: Longitudinal cut

Figure 3.30: Diamond wire ready for cross-sectional cut

Figure 3.31: Coring SCC concrete approximately seven feet from top of shaft

A picture of a cross section after coring was completed is presented in Figure 3.32. This figure shows that the cores taken to test the concrete inside the rebar cage were not taken from the exact center of the shaft; however, they were from the region inside the steel reinforcement hoops. These core locations were selected by the concrete cutting technicians to accelerate the core recovery process.

3.5   Results and Discussion

3.5.1   Fresh Concrete Properties

A wheelbarrow sample of concrete was taken from the middle of each truck for fresh concrete testing. For the ODSC, the unit weight, air content, temperature and slump were determined from this concrete. The SCC and SCC-LP fresh concrete batch testing included these tests as well as a slump flow (instead of slump), T_{50}, and VSI tests.

Figure 3.32: Cored section of SCC shaft (20-ft from top)

One truck was selected for each mixture (second, second, and third trucks for the ODSC, SCC, and SCC-LP, respectively) to have extra tests conducted. These extra tests include the slump loss (or slump flow loss), setting by penetration resistance, and conventional bleed tests. Segregation column and pressurized bleed tests were also conducted on the SCC and SCC-LP mixtures.

The fourth truck of the SCC-LP shaft did not have any limestone powder in the mixture. Therefore, the SCC mixture design was used and the following test results reflect this change in mixtures for the SCC-LP shaft.

3.5.1.1 Air Content and Unit Weight of the Fresh Concrete

Results from the total air content test are presented in Figure 3.33. The air content of the ODSC samples stayed within specifications (2.5% to 6%). There was no specification for the air content of either SCC mixtures. The measured air contents of the SCC mixture were very consistent with the exception of Truck No. 2, which was slightly lower than the rest of the loads. The air content of the SCC-LP mixture was much more inconsistent with a maximum measured value of 11% on Truck No. 3. It

should be noted that all the concrete cylinders and prisms for the SCC-LP shaft were produced from the concrete in Truck No. 3. These results from the concrete cylinders for the SCC-LP shaft show the effect of the elevated air content in this concrete.

The unit weight results are presented in Figure 3.34. The unit weight of the SCC-LP is very low for Truck No. 2 and Truck No. 3 because of the high air contents in these batches.

The temperature of the concrete was measured on a sample of concrete from each concrete truck. Results of the fresh concrete temperature tests are presented in Figure 3.35. The ODSC and the SCC mixtures began placement around 12:00 P.M., approximately the hottest part of the day.
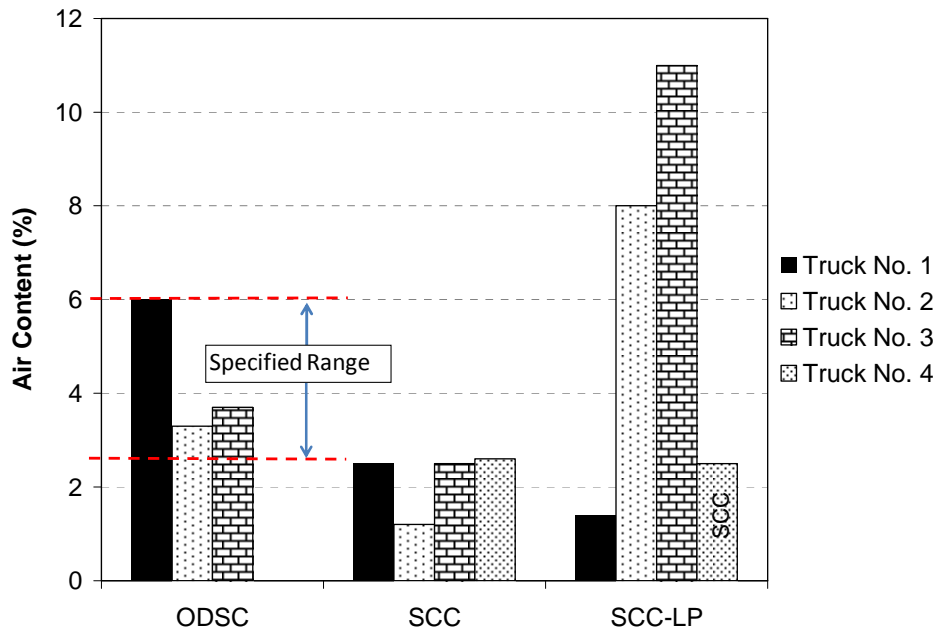
Figure 3.33: Air content test results
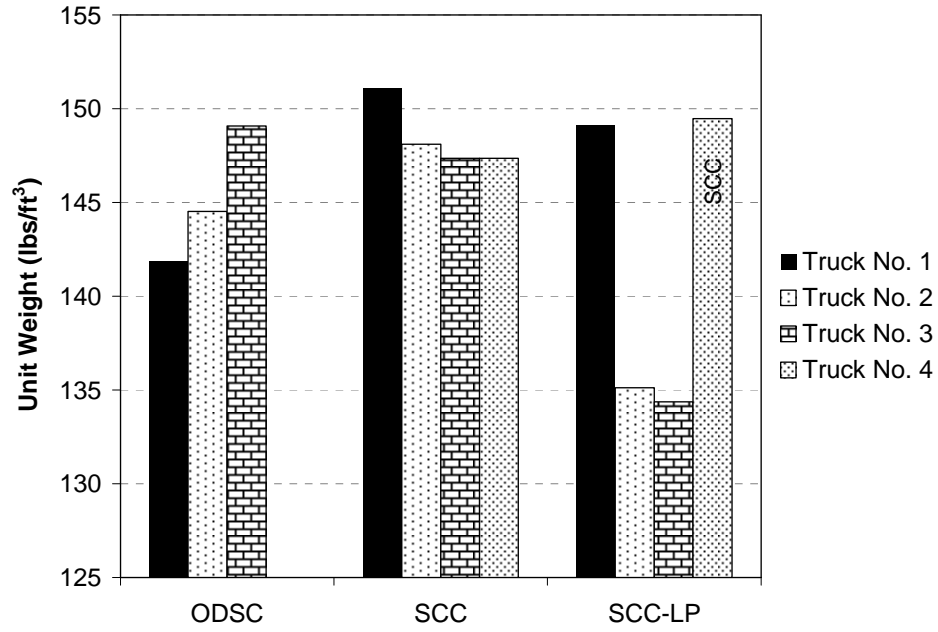
Figure 3.34: Unit weight test results
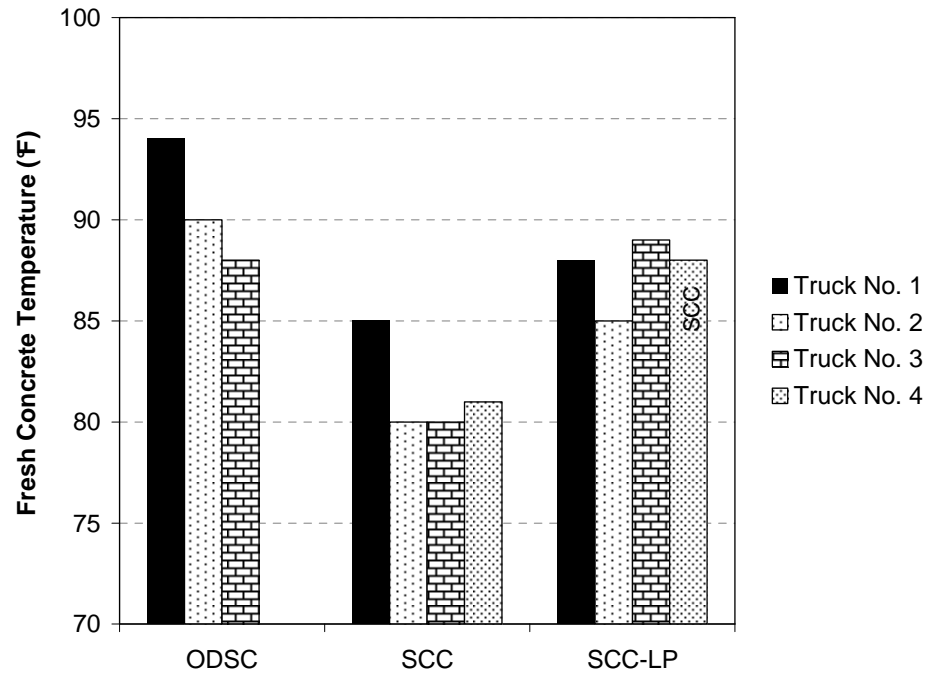
Figure 3.35: Fresh concrete temperature results

3.5.1.2 Consistency of the Fresh Concrete

The recorded slump and slump flow data are presented in Figure 3.36. The ODSC mixture had a very consistent slump for each truck poured. The SCC mixture's slump flow varied throughout the concrete placement, and was the only mixture to have concrete placed that exceeded the project slump flow specification.

The result of this high slump flow is evident in the bleed and segregation tests conducted. However, no problems were apparent in the analysis of the final product of the SCC shaft. The SCC-LP concrete stayed relatively consistent throughout the day.
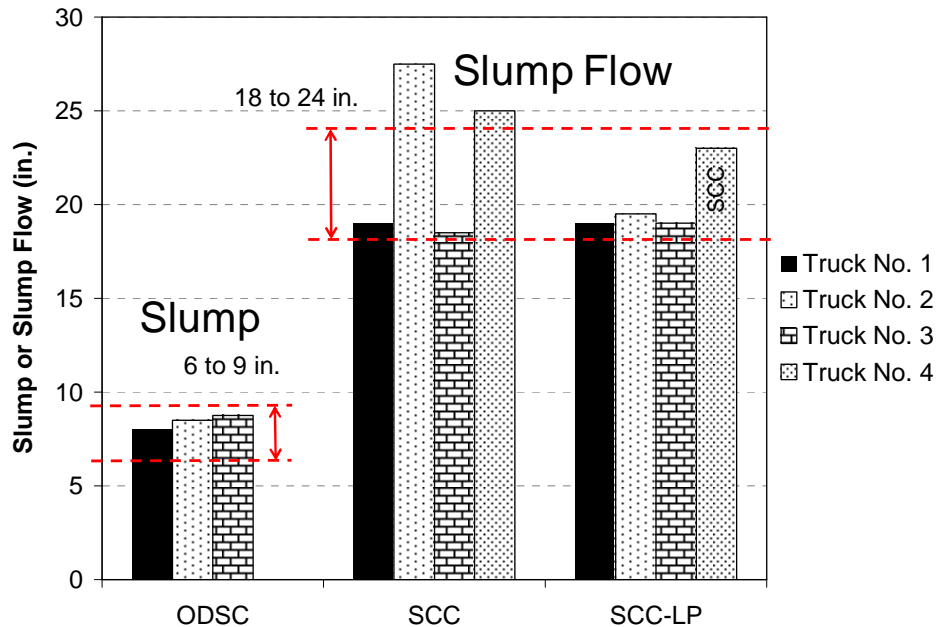
Figure 3.36: Slump and slump flow results

3.5.1.3 Assessment of Concrete's Ability to Flow

Data for the modified J-Ring test were obtained from a sample of concrete from the second and third truck load of each SCC and SCC-LP mixtures, respectively. Results from this test are presented in Table 3.6.

The passing ability of the samples was calculated by subtracting the modified J-Ring results from the slump flow results (ASTM C 1621 2005). The passing ability of the SCC and SCC-LP mixtures are compared in Figure 3.37. Even though the modified J-Ring test performed had wider bar spacing than the specified ASTM test, the blocking assessment was conducted in accordance with the ASTM C 162 (2005) specification. The blocking assessment table from this ASTM specification is presented in

Table 3.7. The SCC-LP mixture was determined to have minimal to noticeable blocking; whereas, the SCC mixture had noticeable to extreme blocking.

These results do not correspond with the results gathered in the field from the elevation measurements inside and outside of the rebar cage. As shown in Figure 3.15, the SCC mixture did not have any problems flowing through the rebar cage of the test shaft. Since the spacing between the modified J-Ring's bars was increased, it is uncertain how Table 3.7 applies to drilled shaft applications on the modified J-Ring. More research is required to develop a blocking assessment for the modified J-Ring in drilled shaft applications.

Table 3.6: Modified J-Ring and slump flow results

Shaft	Slump Flow (inches)	Mod. J-Ring (inches)
SCC	27.5	22.5
SCC-LP	19	17.25
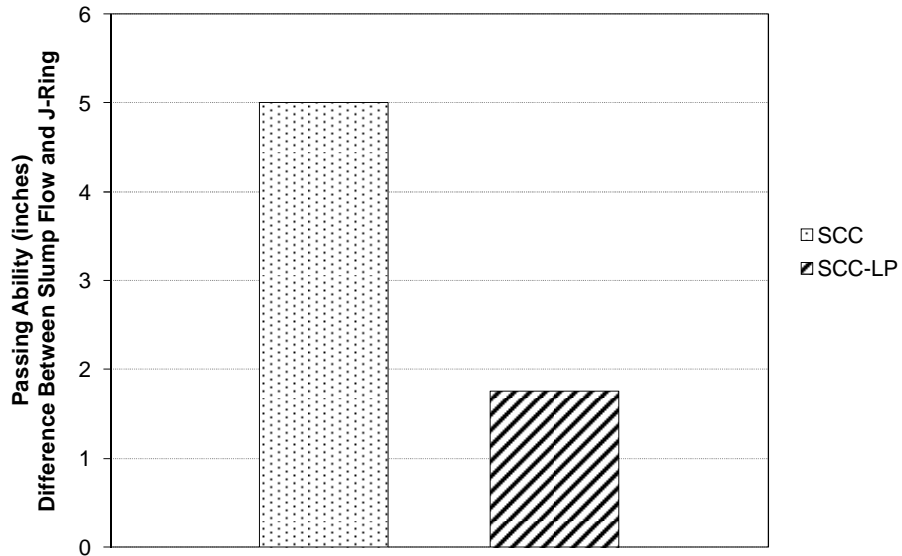
Figure 3.37: Passing ability of SCC and SCC-LP mixtures

Table 3.7: Blocking assessment of concrete mixture (ASTM C 1621 2006)

Difference Between Slump Flow and J-Ring Flow	Blocking Assessment
0 to 1 in. [0 to 25 mm]	No visible blocking
>1 to 2 in. [>25 to 50 mm]	Minimal to noticeable blocking
>2 in. [>50mm]	Noticeable to extreme blocking

3.5.1.4 Assessment of Concrete Stability

The segregation column test was performed on a sample of concrete from the second and third truck load of the SCC and SCC-LP mixtures, respectively. The sampled concrete was allowed to stand for one hour in the segregation column as presented in Figure 3.38. The segregation column test results are presented in Figure 3.39.

The SCC batch had a static segregation index of 15.5%, whereas the SCC-LP batch had a static segregation index of 3.1%. The SCC mixture's segregation is not considered acceptable by the comments offered by ACI Committee 237 (2007), which

106

states that the percent static segregation should be below 10%. However, this test was conducted from the same batch as the sample that had a slump flow result greater than the maximum specified slump flow value specified. It may be concluded from the high slump flow and poor consolidation results that too much water was added to this batch of concrete.

Auburn University previously conducted laboratory segregation tests on similar mixture designs and recorded segregation percentage values of 6% and 7% for the SCC and SCC-LP mixtures, respectively (Dachelet 2008).

Figure 3.38: Segregation column
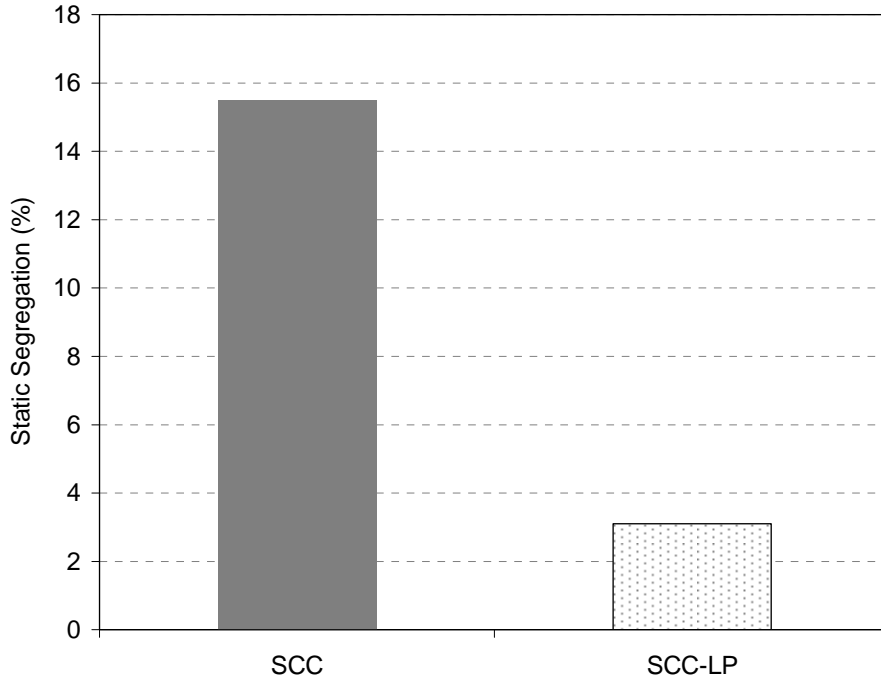
Figure 3.39: Segregation column results

In addition to the segregation column test used to assess the concretes static
stability, the visual stability index (VSI) test was conducted to assess the dynamic
stability of the concrete. This test was conducted in accordance with the appendix of
ASTM C 1611 (2005), and was performed by visually inspecting the concrete patty left
from the slump flow test. The criteria for this test are presented in Table 3.8. The
specification for this project states that the SCC and SCC-LP mixtures must have VSI
ratings less than or equal to 1.5 (Appendix A).

The values recorded in the field are presented in Table 3.9. It should be noted
that these values are subjective and are based on visual observation. For this project all
observations were conducted by the same technician to minimize the error in VSI
results.

The SCC and SCC-LP mixtures were all considered to be stable with VSI ratings
between 0 and 1.5, meeting the projects specification.

Table 3.8: Visual stability index values (ASTM C 1611 2005 Appendix)

VSI Value	Criteria
0 = Highly Stable	No evidence of segregation or bleeding.
1 = Stable	No evidence of segregation and slight bleeding observed as a sheen on the concrete mass.
2 = Unstable	A slight mortar halo $\leq$ 0.5 in.($\leq$ 10 mm) and/or aggregate pile in the of the concrete mass.
3 = Highly Unstable	Clearly segregating by evidence of a large mortar halo > 0.5 in. (> 10 mm) and/or a large aggregate pile in the center of the concrete mass.

Table 3.9: Recorded VSI Values

Shaft	Truck No.	VSI
SCC	1	0.0
	2	1.5
	3	0.0
	4	1.5
SCC-LP	1	0.0
	2	0.0
	3	1.0
	4	0.0

In addition to the segregation column and the VSI test, a measurement of how fast the concrete flows during the slump flow test was conducted. The time required for the concrete to flow to a twenty inch diameter (50 cm) is referred to as the T_{50} time. This time was measured for each of the SCC and SCC-LP batches and is presented in Figure 3.40. The T_{50} test was not conducted on the first truck of the SCC shaft due to miscommunication on the job site.

The SCC mixture had one batch with a relatively long T_{50} time, which corresponds with the lowest slump flow tested for this mixture. However, the slump flow for this high T_{50} test is only slightly lower than the first three SCC-LP slump flows tested. Thus, at similar slump flows, the SCC-LP flowed faster than the SCC, but at a higher

slump flow the SCC flowed faster to a greater distance maintaining its stability.

Therefore, when produced within the specification the SCC will do a better job flowing

horizontally and filling congested areas.
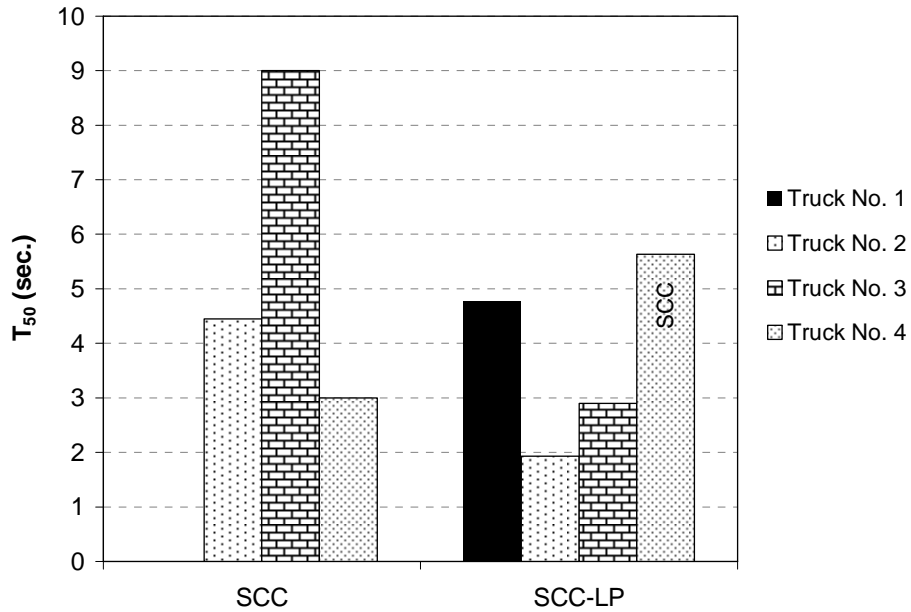
Figure 3.40: Recorded T_{50} times

3.5.1.5 Assessment of Concrete's Workability Retention

Setting tests by the penetration resistance method was performed on a sample of

concrete from the second truck of the ODSC mixture and the third truck of the SCC and

SCC-LP mixtures. These results are presented in Figure 3.41. The initial and final

setting times are presented in

Table 3.10. It should be noted that the ODSC mixture took longer than expected

to reach its initial set. The contractor on the jobsite stated that for this mixture the usual

set time was approximately ten hours (visually assessed with no testing). This delay

may have been caused by extra retarder added to the mixture to account for the

relatively high temperatures experienced.

Not only are the ODSC mixture set times higher than the lab tested values, but the SCC and SCC-LP tests were higher as well. The previous research conducted in the Auburn University laboratory on similar mixture designs recorded initial set times of 21.3 and 12.3 hours for the SCC and SCC-LP mixtures, respectively (Dachelet 2008). The laboratory mixtures did not contain as much hydration stabilizing admixtures and this may explain the difference in the set times.
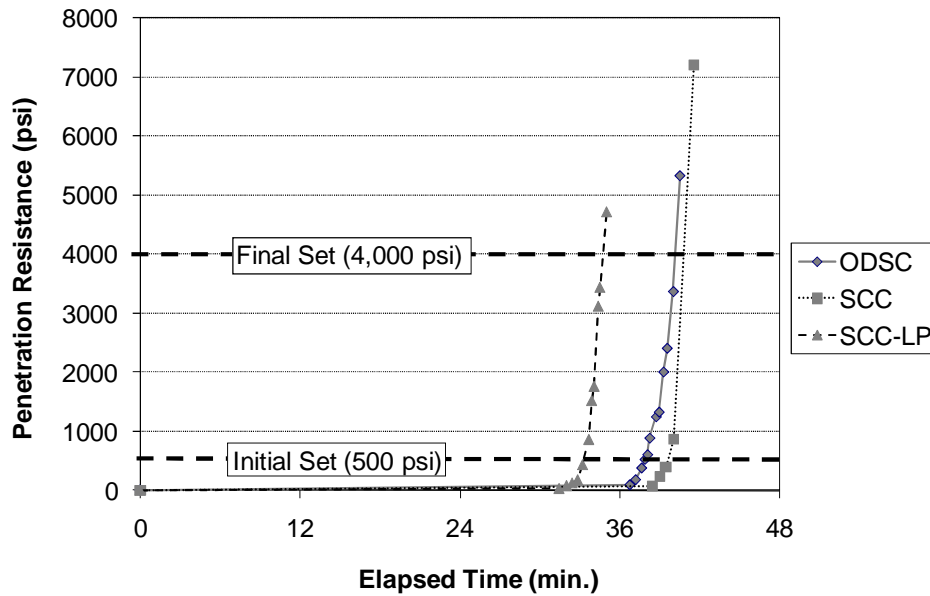
Figure 3.41: Setting by penetration resistance results

Table 3.10: Initial and final set times

	Elapsed Time (hrs)	
	Initial Set (500psi)	Final Set (4,000psi)
ODS	37.9	40.2
SCC	39.5	40.8
SCC-LP	33.3	34.7

The workability retention test was conducted on a sample of concrete from one truck for each concrete mixture. (Second, second, and third trucks of the ODSC, SCC, and SCC-LP mixtures, respectively). Results from this test are presented in Figure 3.42.

The ODSC concrete mixture met the project specifications to maintain a minimum four-inch slump for the duration of the placement. A specification was not placed on the SCC and SCC-LP mixtures, but at the conclusion of the concrete placement, these mixtures had slump flows of 17 in. and 17.5 in., respectively.
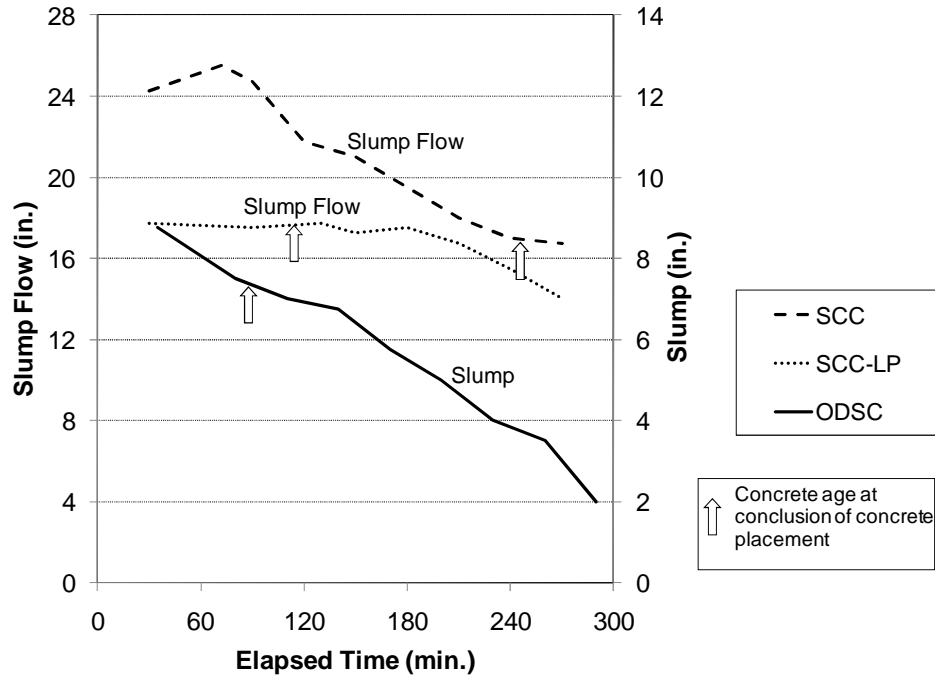
Figure 3.42: Slump/slump flow retention results

3.5.1.6 Bleeding of the Concretes

- Conventional bleed test

The bleed test was conducted on a sample of concrete from the second truck of the ODSC mixture and the third truck of the SCC and SCC-LP mixtures. This test is performed under prevailing atmospheric pressure conditions. Data from this test are presented in Figure 3.43. The total amount of bleed water that was recorded is presented in Table 3.11.

The SCC mixture clearly exhibited the most total bleed water. However, only minimal bleed water was recorded in this mixture until 40 minutes had elapsed. Before this time interval, a glossy film was observed on the surface of the exposed concrete. After 40-minutes the film disappeared and a large amount of bleed water was released. A picture of this bleed water after 80 min. is presented in Figure 3.44.

The ODSC mixture consistently bled water to reach its maximum bleed water of 118 mL. No bleed water was recorded from the SCC-LP mixture. Since there was no change, the SCC-LP bleed test was stopped after two hours had elapsed.

The SCC-LP results are not surprising as the addition of the limestone powder was expected to reduce the bleed water of the mixture (Khayat et al. 2006). The amount of ODSC bleed water was also as expected. However, the SCC bleed water was surprising in that the bleed water within the SCC shaft was expected to have been less than that of the ODSC mixture. Extra water may have been added to the concrete batch, increasing the w/cm ratio and causing this excessive bleeding. This would also explain the high slump flow and poor consolidation results from this batch.

- Pressurized bleed test

This experimental test was performed on a batch of concrete from the second, second, and third trucks of the ODSC, SCC, and SCC-LP mixtures, respectively. The pressurized bleed test being conducted is presented in Figure 3.45.

The results from this test are presented in Figure 3.46 and Figure 3.47. The amount of bleed shown on the right hand side of the graph is in percent of free water batched.

113
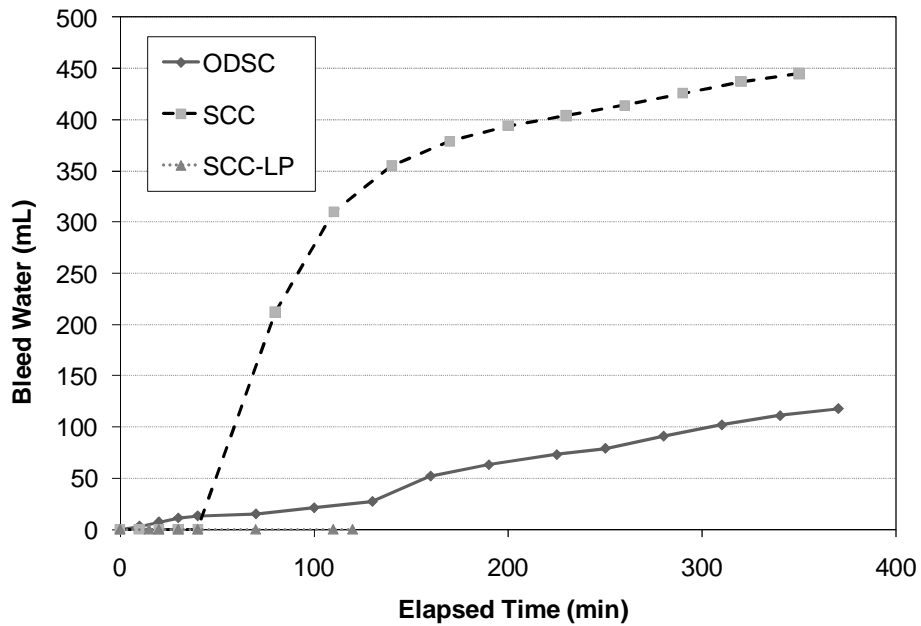
Figure 3.43: Conventional bleed test results

Table 3.11: Total bleed water

Mixture	Elapsed Time (hrs)	Total Bleed Water (mL)
ODSC	6.2	118
SCC	5.8	445
SCC-LP	2.0	0

The intention was to apply pressure slowly to the piston to simulate the conditions within the drilled shaft. However, the pressures could not be increased slowly or precisely due to the imprecision of the adjuster knob on the air compressor. When the target pressure was exceeded, the pressure at which the gauge reached was recorded and the chamber was kept at this recorded pressure for its duration. For this reason, each test result is presented on different figures with the actual applied chamber pressures at different time intervals.

114

Figure 3.44: SCC bleed water after 80 minutes
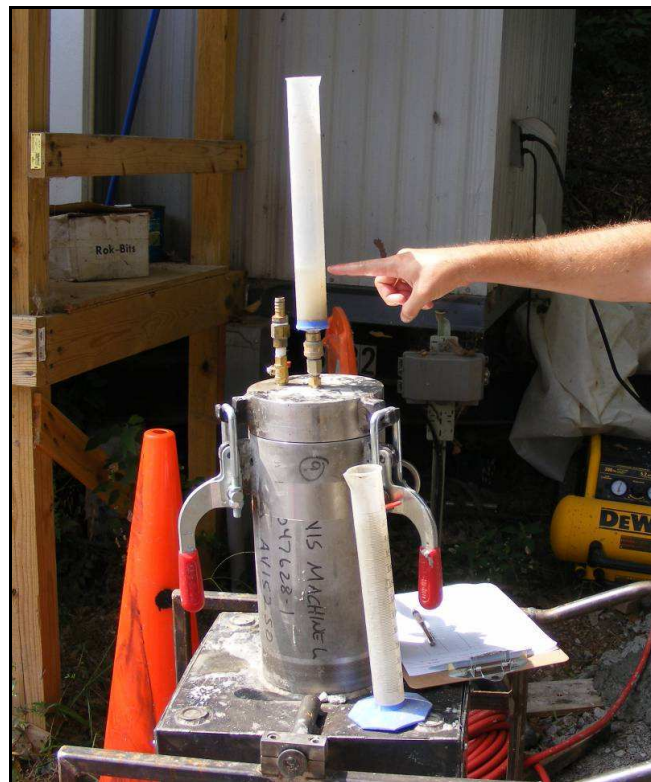
Figure 3.45: Pressurized bleed test being conducted

Although the pressures vary for each test, it is apparent that the excess water appears to be easily pushed out of the ODSC and SCC mixtures in a relatively short period of time. The SCC-LP mixture had a much slower recorded bleed rate. The author believes that this slow bleeding was due to an unclean apparatus. Therefore the results for the SCC-LP mixture are believed to be false and have not been included in this document. However, the SCC-LP mixture did bleed under pressure, but the actual pressure that was on the sample to cause this bleed water is unknown.
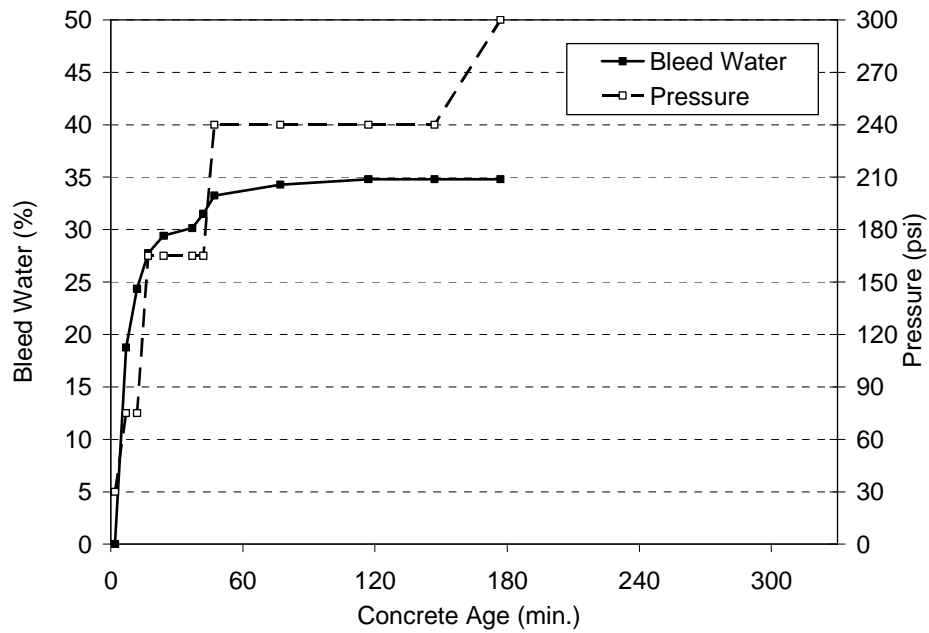
Figure 3.46: ODSC Pressure Bleed Results
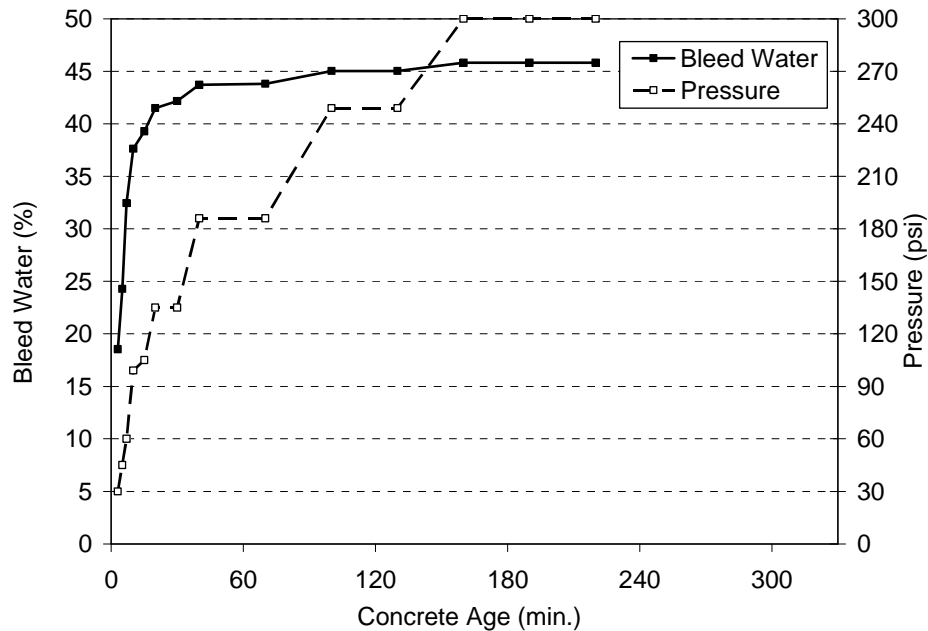
Figure 3.47: SCC pressurized bleed test results

3.5.2 Hardened Concrete Properties of Molded Specimens

The results in the following sections were obtained from testing molded cylinders made from the concrete placed into each shaft.

3.5.2.1 Compressive Strength and Modulus of Elasticity Results

The compressive strengths and modulus of elasticity of the concrete, presented in Figure 3.48 and Figure 3.49, respectively, are averages of three cylinders per testing age.

The SCC mixture had the highest compressive strength for each testing age. The SCC-LP had the lowest compressive strength at each testing age. This low strength can be related to the high air content of the batch the cylinder's were formed from. A better comparison between the concrete mixtures may be obtained from the core testing data presented later in this report (Section 3.5.6.1).

117
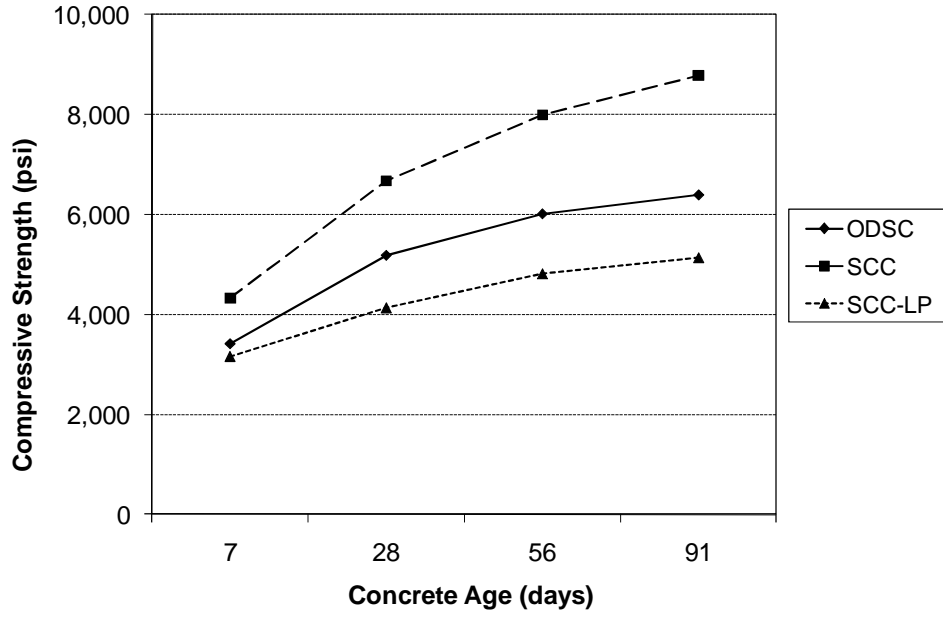
Figure 3.48: Molded cylinder compressive strength results
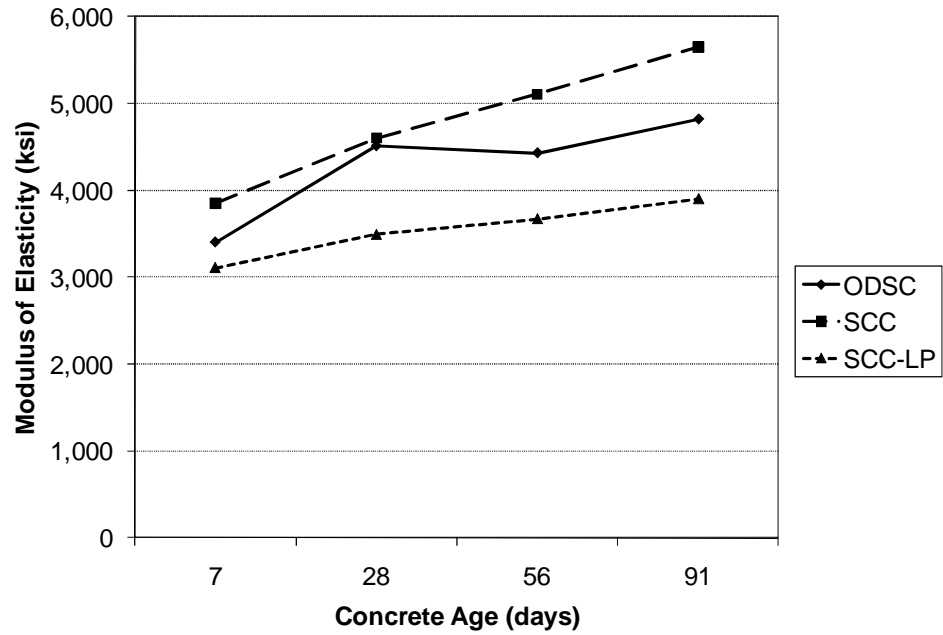
Figure 3.49: Molded cylinder modulus of elasticity results

118

3.5.2.2 Resistance to Chloride Ion Penetration

The results from the chloride ion penetration resistance test are presented in Figure 3.50. The ASTM specification states that the "variation of a single test result has been found to be 12.3%" (ASTM C 1202). Using 12.3 % as a limit to compare various results these test results are similar. Therefore, the chloride ion penetrability of each of the concrete mixtures is approximately equal. The ASTM C 1202 (2005) testing standard for this test is presented in

Table 3.12. Using this table it can be concluded that the permeability of the samples are very low.
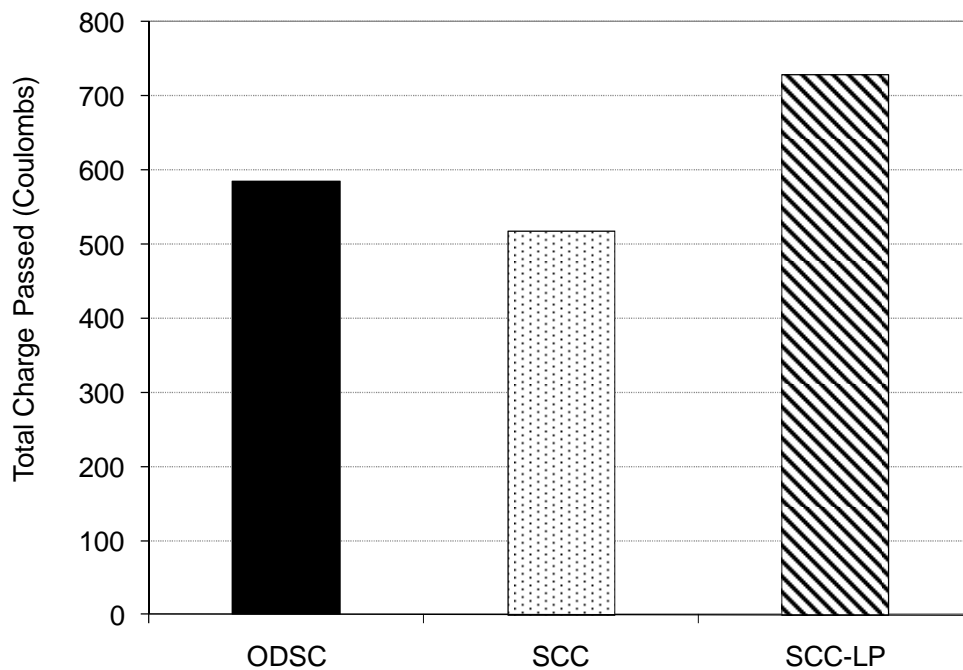
Figure 3.50: Molded cylinder 180-day chloride ion penetration results

Table 3.12: Chloride ion penetrability based on charge (ASTM C 1202 2005)

Charge Passed (coulombs)	Chloride Ion Penetrability
>4,000	High
2,000–4,000	Moderate
1,000–2,000	Low
100–1,000	Very Low
<100	Negligible

3.5.2.3 Drying Shrinkage

The results from this test are presented in Figure 3.51. The measured drying

shrinkage appears similar for all the mixtures.
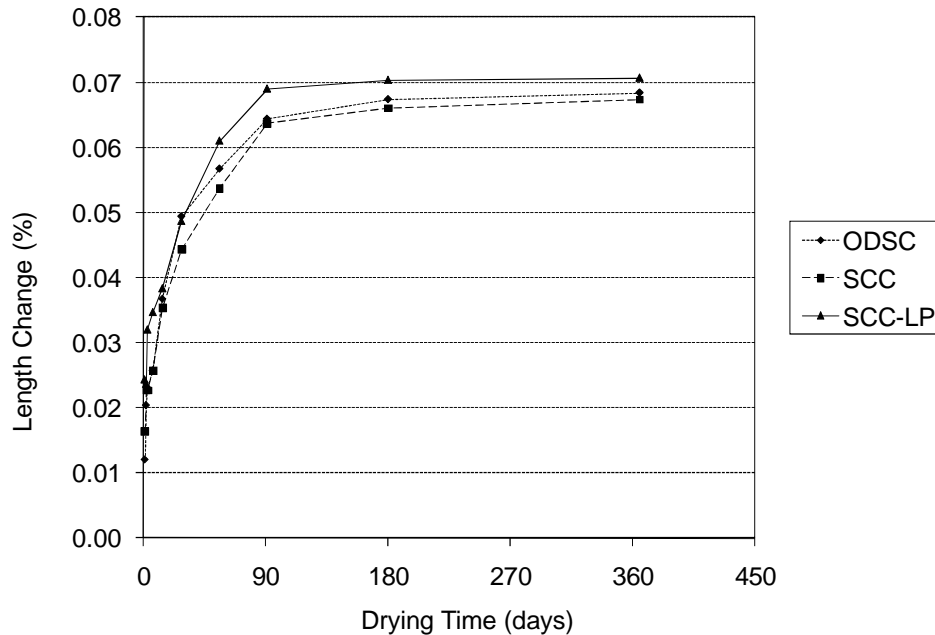
Figure 3.51: Shrinkage of molded specimens results

3.5.3   In-Place Shaft Integrity

Crosshole sonic logging (CSL) was conducted on the test shafts on September

8, 2008. Applied Foundation Testing (AFT) performed the CSL tests and supplied a

report for each shaft (Robertson and Bailey 2008). In this section, the findings of each

120

report will be summarized. It should be noted that sand was intentionally added to the slurry and pieces of shale were intentionally dropped into the shaft during construction.

The CSL testing logs numbered each CSL tube as presented in Figure 3.52. References to shaft imperfections were conducted by stating the tube pair and the depth from the surface.
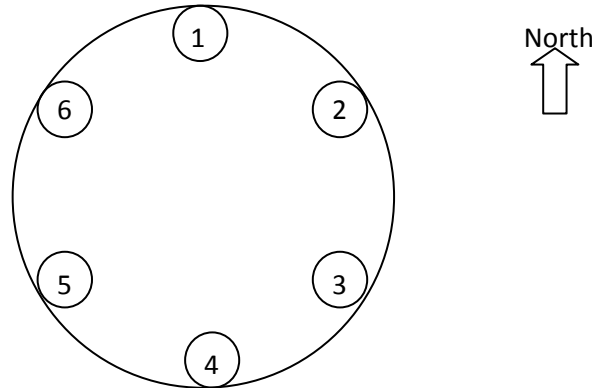
Figure 3.52: CSL tube numbering

3.5.3.1 ODSC Shaft Results

No severe imperfections were observed in the ODSC shaft; however, slight imperfections were observed. These imperfections are not large enough to cause any alarm, but for research purposes were noted. The following tube pairings were highlighted as areas of interest:

- Tube Pair 1-2 from 13.0 to 13.5 feet: slight decrease in pulse velocity (<10%),

- Tube Pair 2-3 from 7.8 to 8.0 feet: slight decrease in pulse velocity (<10%),

- Tube Pair 5-6 from 10.4 to17.4 feet: slight decrease in pulse velocity (<10%), and

- Tube Pair 1-4 from 0.0 to 2.0 feet: slight decrease in pulse velocity (<10%) with minor reduction in energy.

3.5.3.2 SCC Shaft Results

No severe imperfections were observed in the SCC shaft; however, a slight imperfection was discovered in the shaft at tube pair 2-5 from 20.4 feet to the bottom. This pair showed a slight decrease in pulse velocity with a minor reduction in energy (<10%). It was noted however, that this zone would normally not be deemed problematic and was only noted for research purposes.

3.5.3.3 SCC-LP Shaft Results

No severe imperfections were observed in the SCC-LP shaft; however, slight imperfections were discovered in the shaft. These imperfections are not large enough to cause any alarm, but for research purposes were noted. The following tube pairings were highlighted as areas of interest:

- Tube Pair 1-4 from 0.0 to 3.6 feet: slight decrease in pulse velocity (10 to 13%),

- Tube Pair 1-5 from 0.0 to 3.6 feet: slight decrease in pulse velocity (10 to 13%), and

- Tube Pair 2-4 from 0.0 to 3.6 feet: slight decrease in pulse velocity (10 to 13%).

AFT also noted that "several other tube pair combinations exhibited a similar decrease in concrete pulse velocity in the upper 3.0 to 3.6 feet of the drilled shaft". However, these tube pairings had pulse velocity decreases less than 10%.

3.5.3.4 Summary of CSL Results

The cross-hole sonic logging did not identify any major imperfections in the test shafts. AFT noted in each of their reports that "the CSL data indicated no anomalous zones within the tested tube pairs that would be considered problematic to the overall shaft integrity" (Robertson and Bailey 2008).

It should be noted that the shale pieces, added to the shafts purposely, were visible in the cut sections of the SCC shaft, but did not show up in the CSL results. However, a bleed channel was noted in one of the cores of the SCC shaft at the elevation that the disturbance was noted in the CSL results.

3.5.4 Evaluation of Exhumed Shafts

In this section, the visual quality of the in-place concrete and any imperfections discovered are noted. Additionally, results of testing the cores recovered from the shafts are presented.

3.5.4.1 Outer Surface of the Shafts

As the surface of each shaft was corrugated steel, the outer surface does not reflect what would actually occur in an actual production shaft. However, since this was an impermeable surface, the addition of sand to the slurry simulates what happens when particulate debris is in the slurry within a shaft that is bearing into a rock socket or cased. With nowhere to go, the sand must either become trapped in the shaft or be displaced upwards out of the shaft with the drilling slurry. The sand deposits on the outside region of the shaft are presented in Figure 3.53. The buildup sand on the outer surface of the shafts, specifically the bottom two feet of the ODSC shaft, show that sand will settle out of the drilling slurry and become trapped on the outside of the shaft.

The ODSC shaft had a much larger accumulation of sand on the outer wall of the shaft. This build up may be due to the fact that the sand was added to the slurry in only this drilled shaft and assumed to flow with the slurry into the next shafts. Therefore, it would be expected that the sand accumulation would be less for each shaft regardless of the viscosity of the concrete. Since the same amount of sand could not have been in the slurry for each shaft, limited conclusions can be determined by looking at the sand build up on the outer surfaces of the shafts. However, the presence of sand on the

outside surface of each shaft is evidence that the settled sand will get pushed to the outside of the shaft and get trapped along the side walls of the shaft.

3.5.4.2 Voids, Bleed Channels, and Anomalies

In general, the majority of the visible bleed channels were located within the SCC shaft. However, the ODSC shaft contained a number of sand filled voids on the surface of the shaft and voids due to lack of consolidation along the reinforcement cage. Some of the core samples in the SCC shaft had to be redone because of bleed channels. Many of the cores in the ODSC shaft had to be redone because of voids present in this shaft. A picture of cores with voids and a bleed channel are presented in Figure 3.54.

The ODSC shaft contained voids located in the cover of the shaft that appear to be due to lack of concrete consolidation. That is, voids were visible under many of the hoops and along one of the longitudinal reinforcement bars. This long void can be seen for approximately 15 ft of the longitudinal cut as presented in Figure 3.55. A possible reason for this void is that the concrete was not able to fully encapsulate the longitudinal reinforcement possibly causing this long void. This anomaly can be shown using the J-Ring test, as presented in Figure 3.56.

The SCC-LP shaft had very few bleed channels, most of which were located near the bottom corners of the shaft (see Section 3.5.4.3). However, an anomaly was visible near the top of the SCC-LP shaft. This anomaly is presented in Figure 3.57. It is unknown what caused this anomaly, but this may have been caused by the tremie pipe when it was pulled from the shaft or from a cement ball of poorly mixed concrete that was mixed in the concrete.

ODSC

SCC

SCC-LP

Figure 3.53: Outside surface of bottom of the shafts

(a)   (b)

Figure 3.54: a) ODSC core with a sand filled void b) SCC core with a bleed channel

Figure 3.55: Long void located within the ODSC shaft

126

Figure 3.56: J-Ring test showing possible reason for poor consolidation on the cover of the ODSC shaft

Figure 3.57: Anomaly observed near the top of the SCC-LP shaft

3.5.4.3 Condition of Shaft Bottoms

It should be noted that the bottom of the shaft referred to in this section is the bottom of the experimental shafts within the corrugated pipe and not the actual bottom of the drilled shaft, as one foot of ordinary concrete was placed in the bottom of each drilled shaft. For all the shafts, the interface between the bottom of the shaft and the initial concrete was covered with a film of sand and slurry. The ODSC shaft had weak pockets of slurry and sand outside the reinforcement cage at the bottom of the shaft. These pockets are presented in Figure 3.58.

The bottom of the SCC shaft was in the best condition with few visible channels and no visible voids (see Figure 3.59). The SCC-LP shaft did not have any visible voids but the corner was damaged and showed signs of poor concrete. This chipped concrete may have been caused by a problem in moving the shaft around after excavation or by poor-quality concrete. A few bleed channels were also visible near the corners of the shaft (see Figure 3.60).

Sand and slurry filled pockets

Figure 3.58: Bottom of ODSC Shaft with sand and slurry filled voids

Bleed channels

SCC Shaft bottom interface

Bleed channels
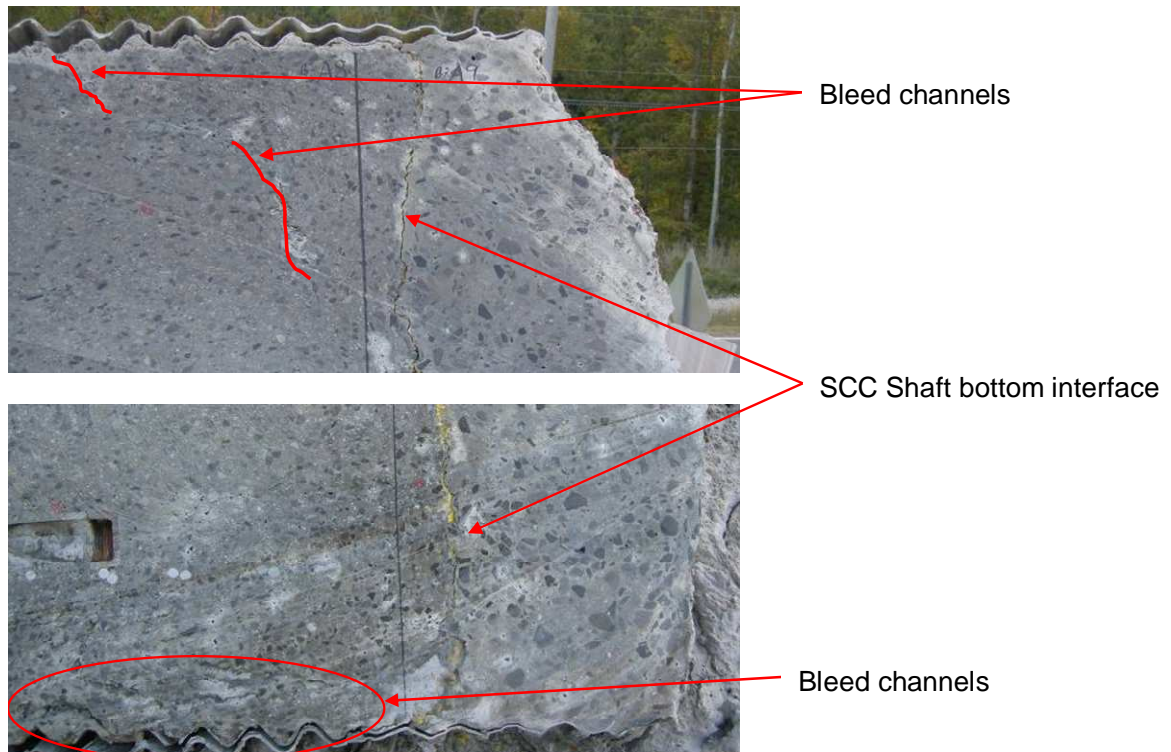
Figure 3.59: SCC bottom surface

3.5.5   Visual Evaluation of Concrete

3.5.5.1 Concrete Flow Analysis

After each cut was made, each section was shellacked to improve the concrete's surface appearance for inspection. A one-foot by one-foot grid was drawn on each longitudinal cut. Colored mortar cubes were counted and mapped within each of these grids. The counted cubes were then plotted based on their elevation in relation to the top of the shaft. The results of this cube mapping are presented in Figures 3.61, 3.62, and 3.63. The predicted location of the cubes when laminar flow is assumed to occur is presented on the left side of the figure. The approximate tremie tip elevation at the time the color cubes were discharged is presented in the middle of the figures. Finally, on the right side of the graph is the number of cubes counted at each elevation.
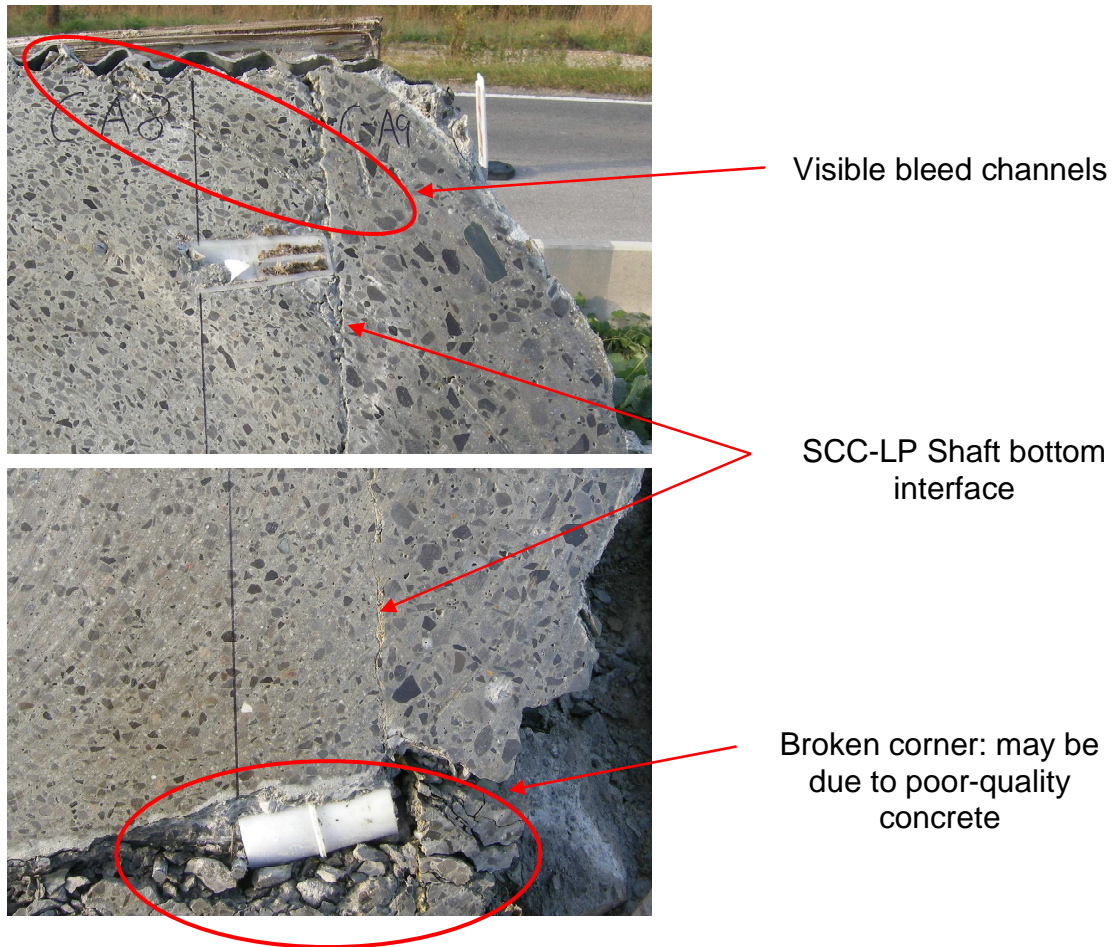
Figure 3.60: SCC-LP bottom surface

The current FHWA drilled shaft manual (O'Neil and Reese 1999) states that "The concrete that arrives first at the top of the shaft [during the concrete placement] is normally that which was placed first." For this to occur, that concrete must have a laminar flow as predicted by Dachelet (2008). From the figures it can be concluded that the concrete did not flow in a perfectly laminar state as predicted. This laminar flow would be ideal, because it would mean that only a small portion of concrete would be in contact with the slurry mixture during the entire pour. Twice as many red cubes were added to the concrete at the beginning of the shafts to increase the chances of

discovering how the initially discharged concrete flows. If the red cubes are found at the top the statement made by O'Neil and Reese (1999) will be confirmed.

However, Gerwick and Holland (1986) performed tests on concrete tremie flow under water and determined that the concrete would not flow in this laminar state, but it rather flowed in either a bulging or layered manner. The bulging flow and layered flow as determined by Gerwick and Holland (1986) is presented in Figure 3.64.

It was concluded by Gerwick and Holland (1986) that bulging flow was the most desirable to limit the amount of laitance. This research was conducted on conventional under-water concrete where the concrete is allowed to laterally flow and not on SCC where the concrete is confined in a shaft.

A previous research project was conducted in South Carolina on SCC in drilled shafts. A conventional-slump concrete mixture, called SC Coastal, was used in comparison with a SCC mixture (Brown et al. 2005, Holley et al. 2005). Part of this project involved using dyed concrete to predict the flow of the concrete within the drilled shaft. A picture of this dyed concrete in the SC Coastal shaft is presented in Figure 3.65. With the tremie located on the bottom of the shaft, the first load of concrete placed was dyed black and the fourth load of concrete was dyed red. The second and third loads were not dyed. It was concluded that the first load placed will fill the bottom of the shaft, and the proceeding loads will travel upwards around the tremie. However, only a small layer of grey concrete was seen between the black and red concrete. Therefore the red concrete must have displaced the grey concrete up the shaft (Holley et al. 2005).
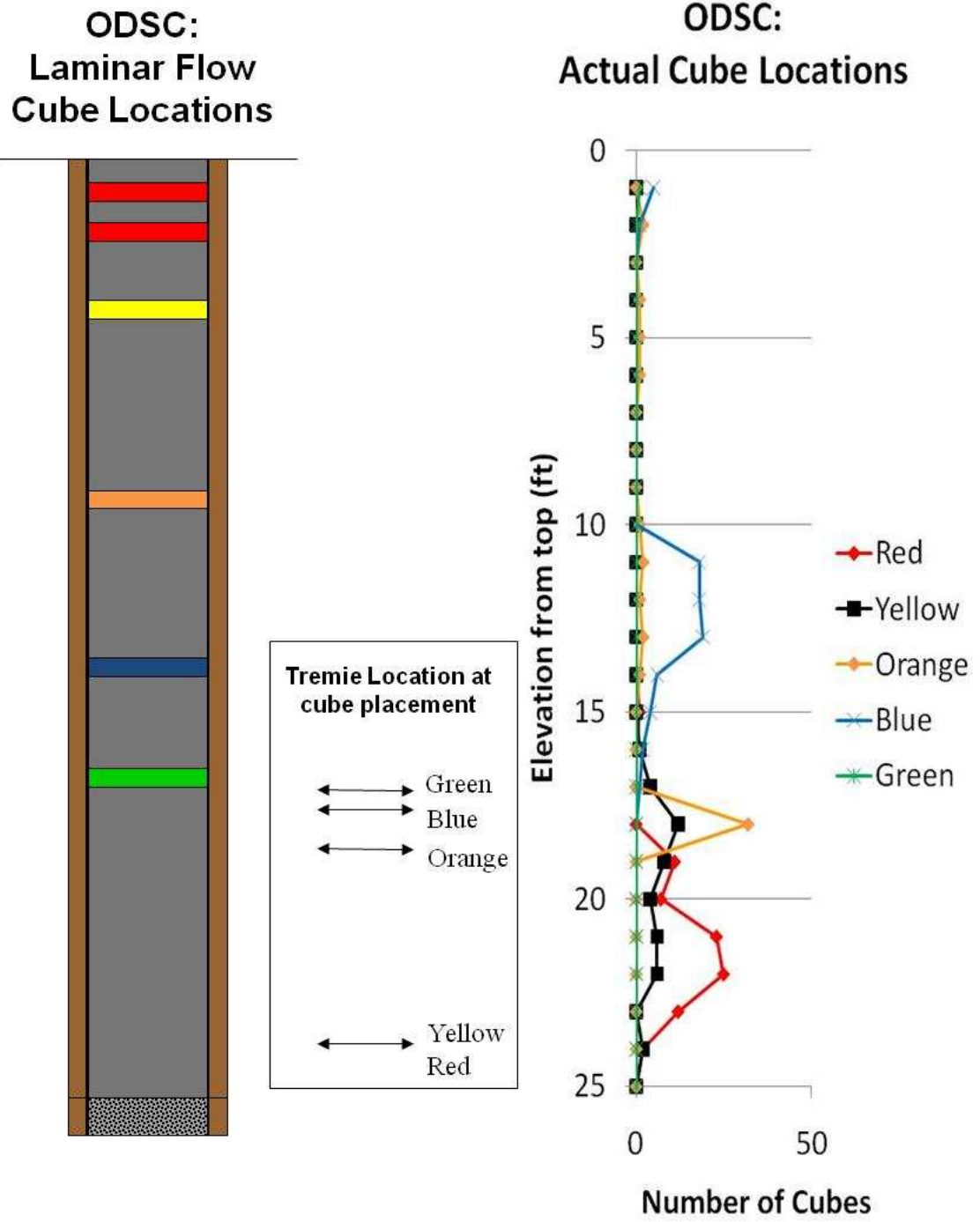
Figure 3.61: ODSC actual versus laminar cube location
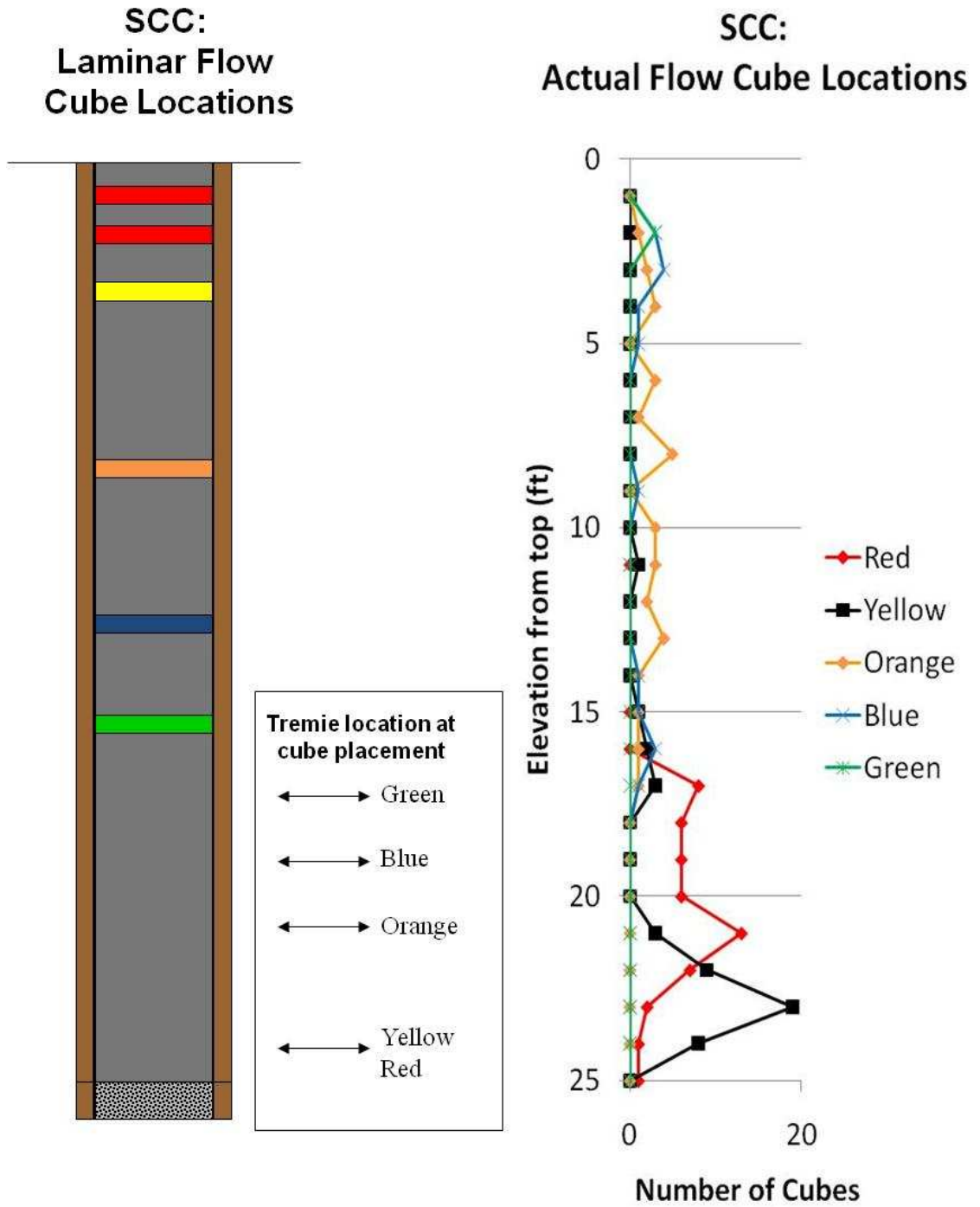
Figure 3.62: SCC actual versus laminar cube location
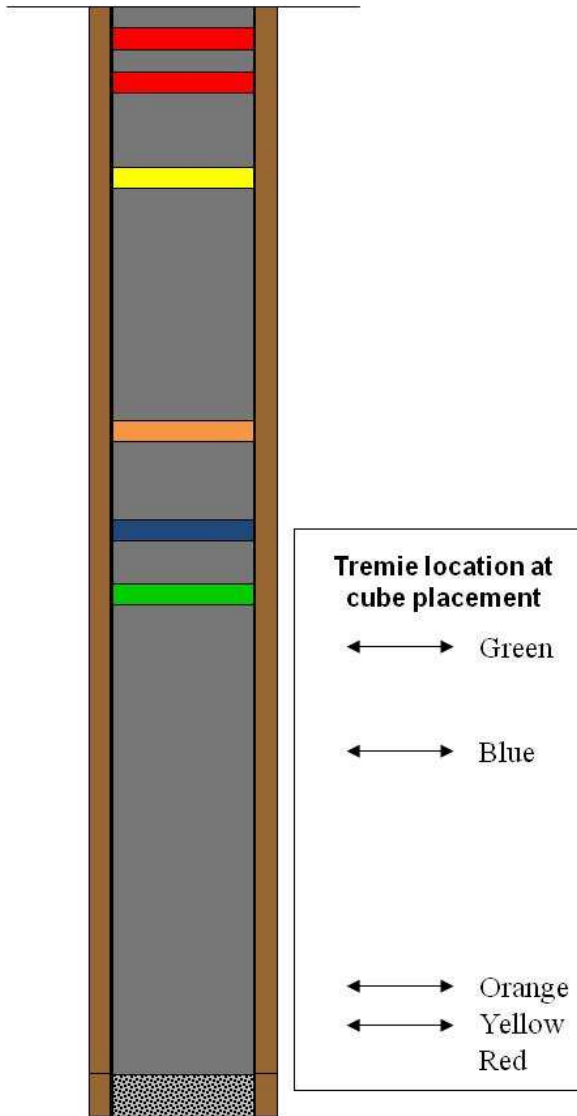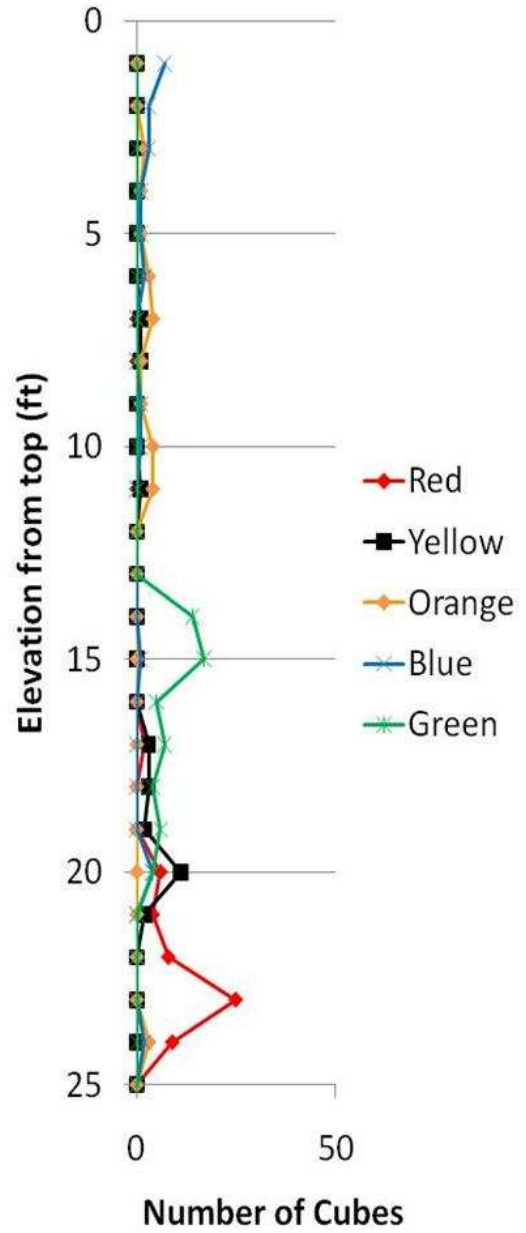
Figure 3.63: SCC-LP actual versus laminar cube location
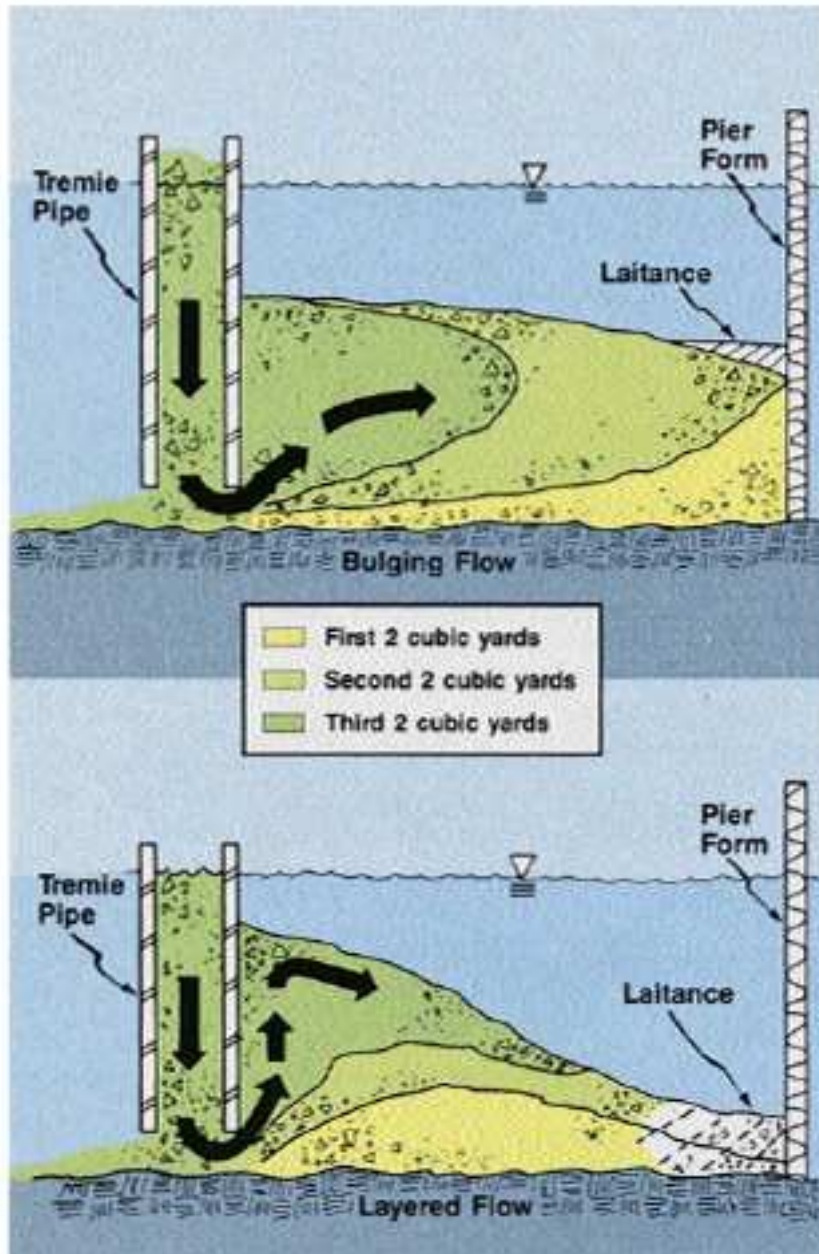
Figure 3.64: Bulging flow versus layered flow (Gerwick and Holland 1986)

Figure 3.65: Dyed concrete in the bottom of the SC Coastal shaft (Holley et al. 2005)

A longitudinal cut could not be performed on the bottom of the SCC shaft for the South Carolina project due to the projects budget. The cross-sectional cut made in the SCC shaft is presented in Figure 3.66. The red concrete flowed much tighter around the tremie and unlike with the ordinary drilled shaft mixture, grey concrete can be seen around the red concrete. A cross-sectional cut was also made 13 ft from the bottom, near the location the tremie was moved to for the rest of the concrete flow. At this location the red concrete was pushed to areas near the reinforcement cage. A picture of this location is presented in Figure 3.67. This project concluded that "…the SCC exhibited similar flow direction to the ordinary mix. The lowest slump (also the first load placed and the one dyed black) concrete from both mixes appeared to remain at the bottom of the shaft. Subsequent loads appeared to flow up around the tremie pipe, displacing the surrounding concrete out laterally." (Holley et al. 2005)

Figure 3.66: South Carolina SCC: cross-sectional cut 6 ft from bottom (Holley et al. 2005)

Figure 3.67: South Carolina SCC: cross-section 13 ft from bottom (Holley et al. 2005)

To understand the flow within the drilled shafts and compare this flow with the South Carolina data, a graph was created to plot the number of cubes that appear horizontally away from the center of the shaft versus the elevation from the top. Cubes at the same elevation, on either side of the longitudinal centerline were added to quantify the number of cubes that spread from the center. These plots are presented in Figures 3.68, 3.69, and 3.70.

The ODSC mixture appeared to flow in a layered manner near the outside of the shaft, since the majority of the yellow cubes were located above the majority of the red cubes. Near the center of the shaft, however, the yellow and red cubes are mixed and do not show a clear pattern. The orange cubes seemed to stay clumped near the center of the shaft at the same elevation that they were dispensed. Therefore, the concrete placed between the orange and blue cubes must have layered onto the orange cubes. The majority of the blue cubes were located between 12 ft and 14 ft, 5 ft to 7 ft higher than the elevation that the cubes were discharged. At the time the blue cubes were discharged, there was only 5.5 ft between the top of concrete and the top of the shaft. Therefore, some of the concrete with blue cubes must have been displaced upward around the tremie in order to be located 7 ft above its discharged depth. A few blue cubes were also located in the center and cover region of the shaft within one foot of the top of the completed shaft. Therefore, some of the concrete must have traveled up the tremie, much like in the South Carolina project. No green cubes were observed on the cut of this shaft.
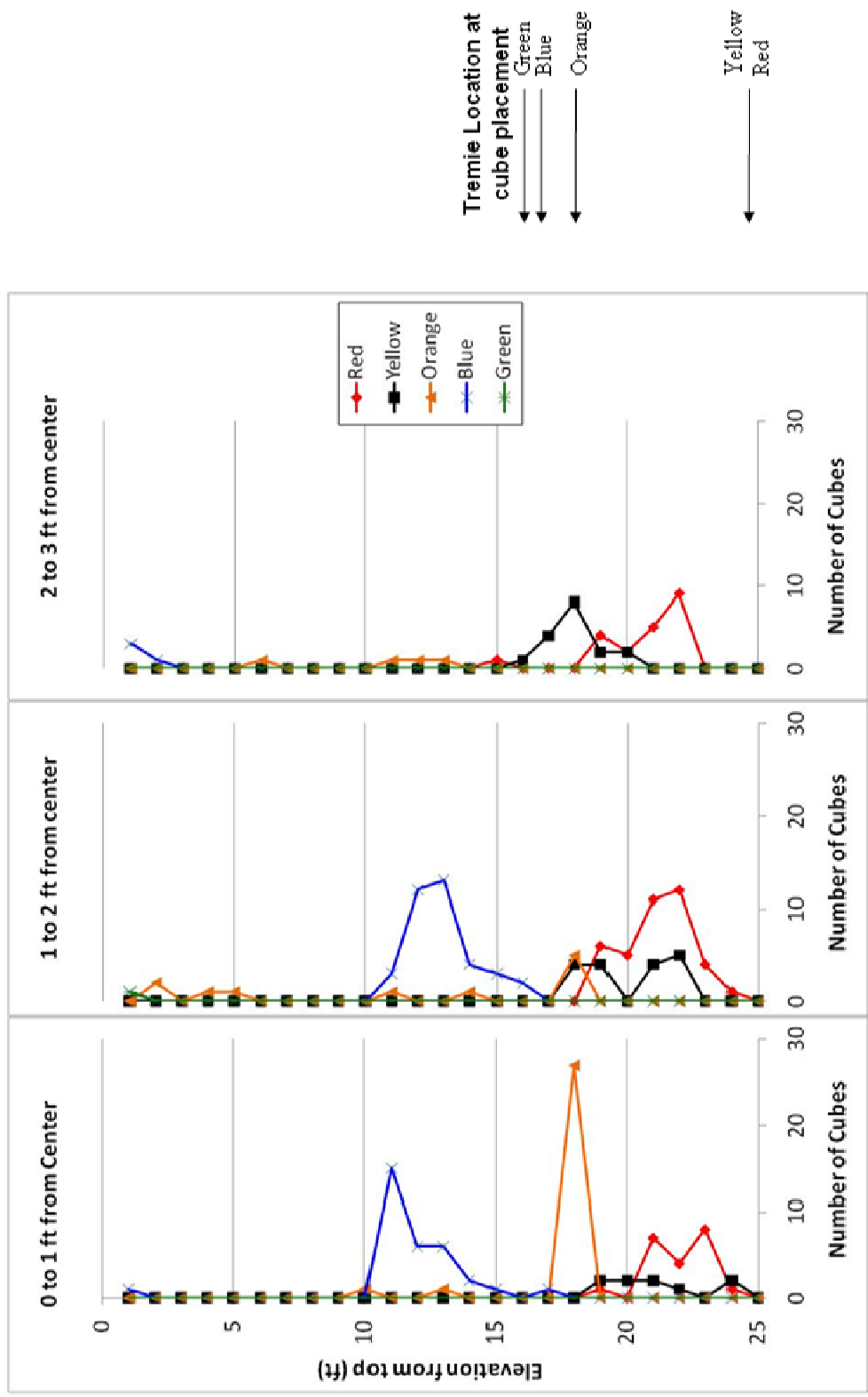
Figure 3.68: ODSC shaft cube locations from center
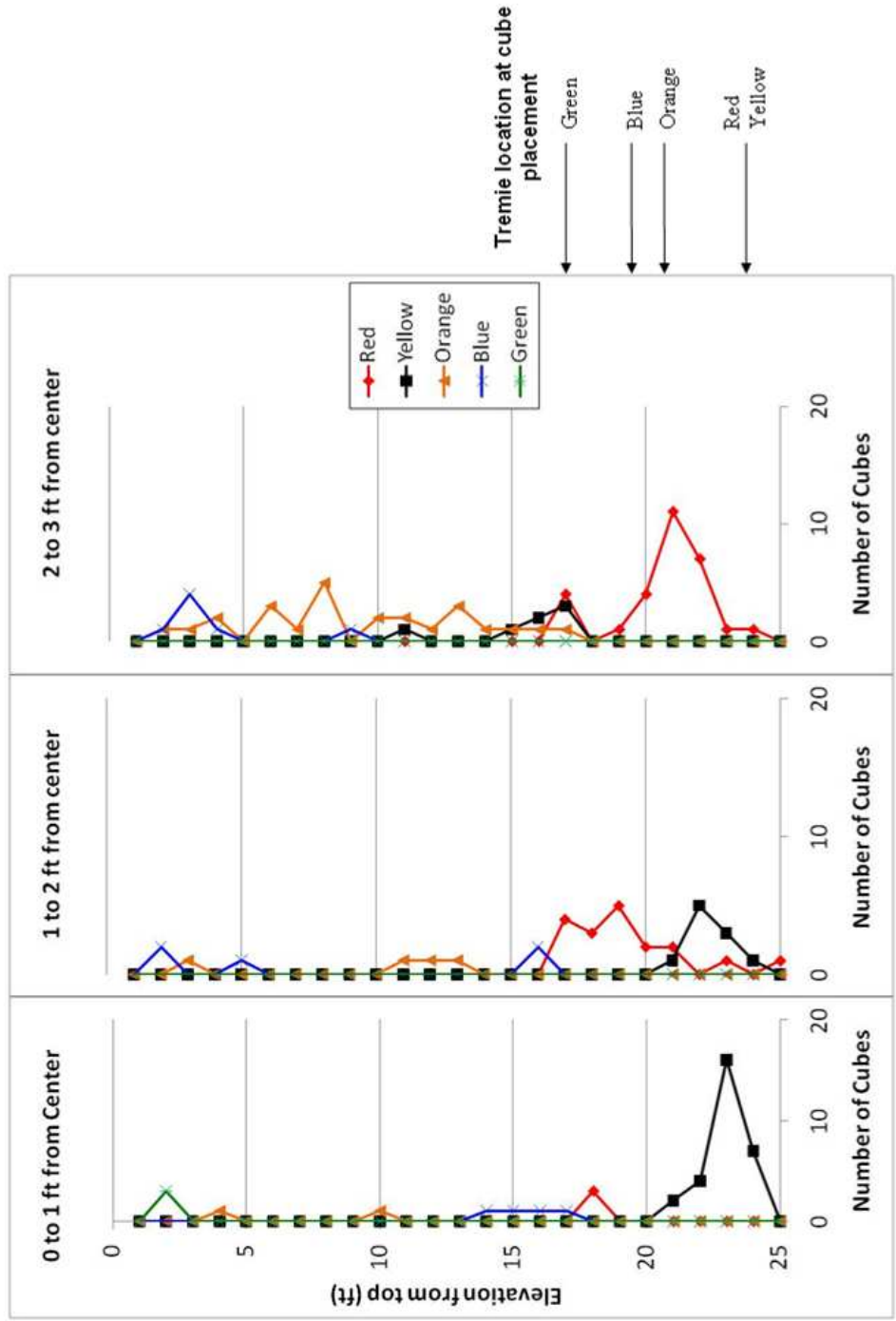
Figure 3.69: SCC shaft cube locations from center
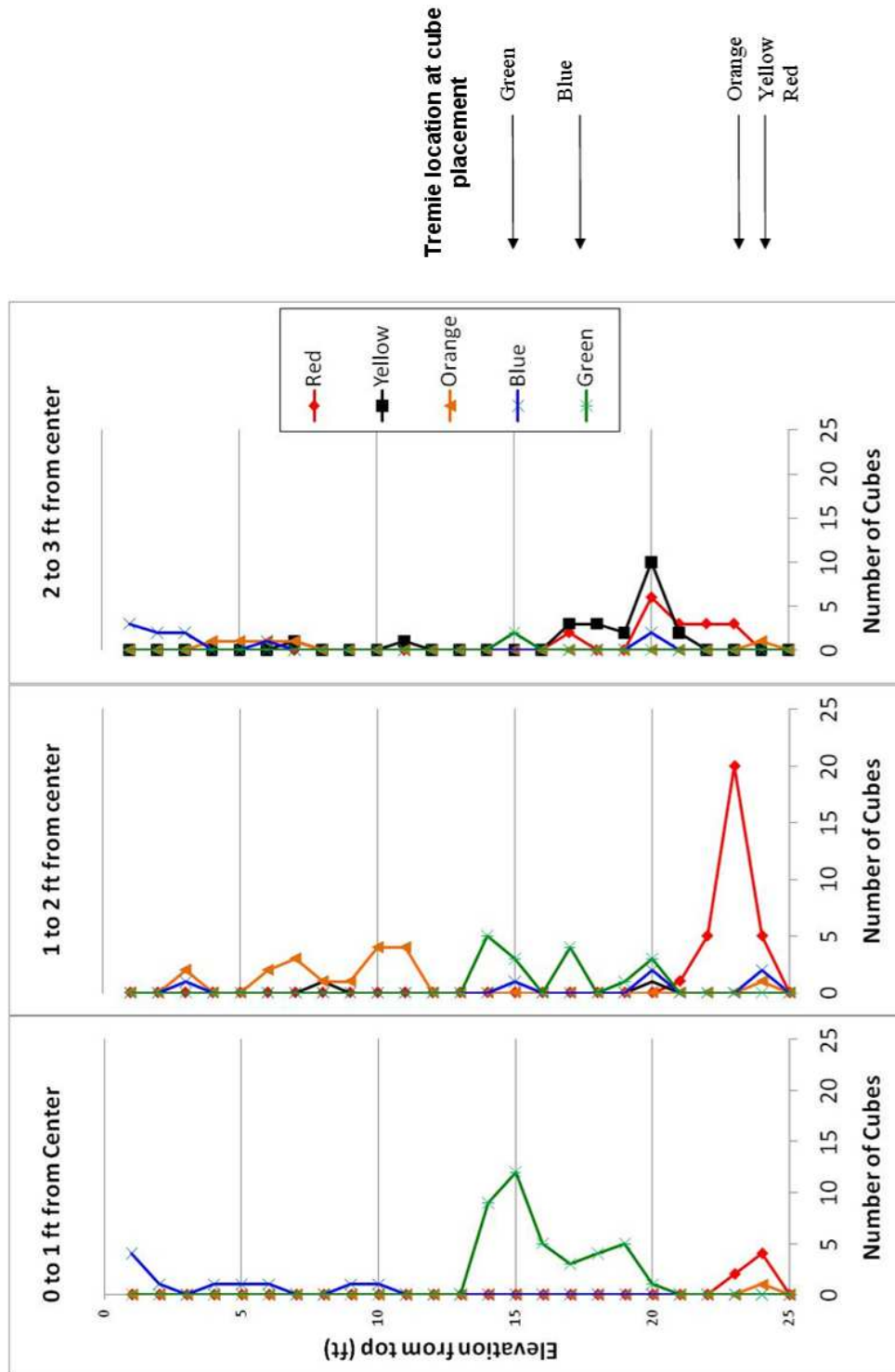
Figure 3.70: SCC-LP shaft cube locations from center

The SCC shaft seems to have flowed in a mixed manner. The red and yellow cubes stayed at the bottom, with the yellow cubes appearing to be bulging into the red cubes and displaced some of the red cubes upward. The orange cubes ended up scattered near the cover region of the shaft for almost the entire shaft length from their discharged location. Similar to the ODSC shaft, most of the blue cubes appeared near the top of the shaft and no green cubes were observed.

The SCC-LP mixture appears to have flowed in a turbulent or mixed manner. Where mixed manner describes a combination between layered and bulged flow. Initially the concrete appeared to flow in a layered manner since most of the yellow cubes were observed toward the outside of the shaft above the red cubes. The orange cubes were observed 13 ft to 19 ft higher than their discharged location. Blue cubes were discovered 3 ft below their discharged elevation and some green cubes were discovered 5 ft below their discharged elevation. Since these cubes were located well below their discharged elevation, turbulence or mixing must have occurred.

There was a problem in distinguishing the orange cubes from the red cubes in the concrete. However, since these cubes were placed at different times they usually ended up in very different areas and therefore could be distinguished. Rocks within the SCC and SCC-LP shafts; however, when cut in half sometimes had similar coloring to the orange cubes. Therefore, some of the scatter of the orange cubes, seen in the SCC and SCC-LP shafts, may have been a result of mislabeling a rock fragment as a cube.

Based on the results from the South Carolina project and the results from this project, a hypothesis was created. The higher viscosity concretes, such as the ODSC mixture and the ordinary mixture in South Carolina, fill the bottom of the shaft first because there is no confining stress on tip of the tremie. Once the tremie tip is immersed, the next concrete to flow out of the tremie stays close to the tremie due to the confining pressure. This confining pressure will cause some of the concrete to travel up

143

around the tremie, but also cause the concrete to displace the previously placed concrete upward. This will occur until the confining stress is greater than the stress that the pump truck causes to displace the concrete. When that occurs, as seen during the project, the pressure causes the tremie to move upwards. Since the most of each layer flows up around the tremie, the concrete seems to flow in a layered manner. A diagram of this theory is presented in Figure 3.71.
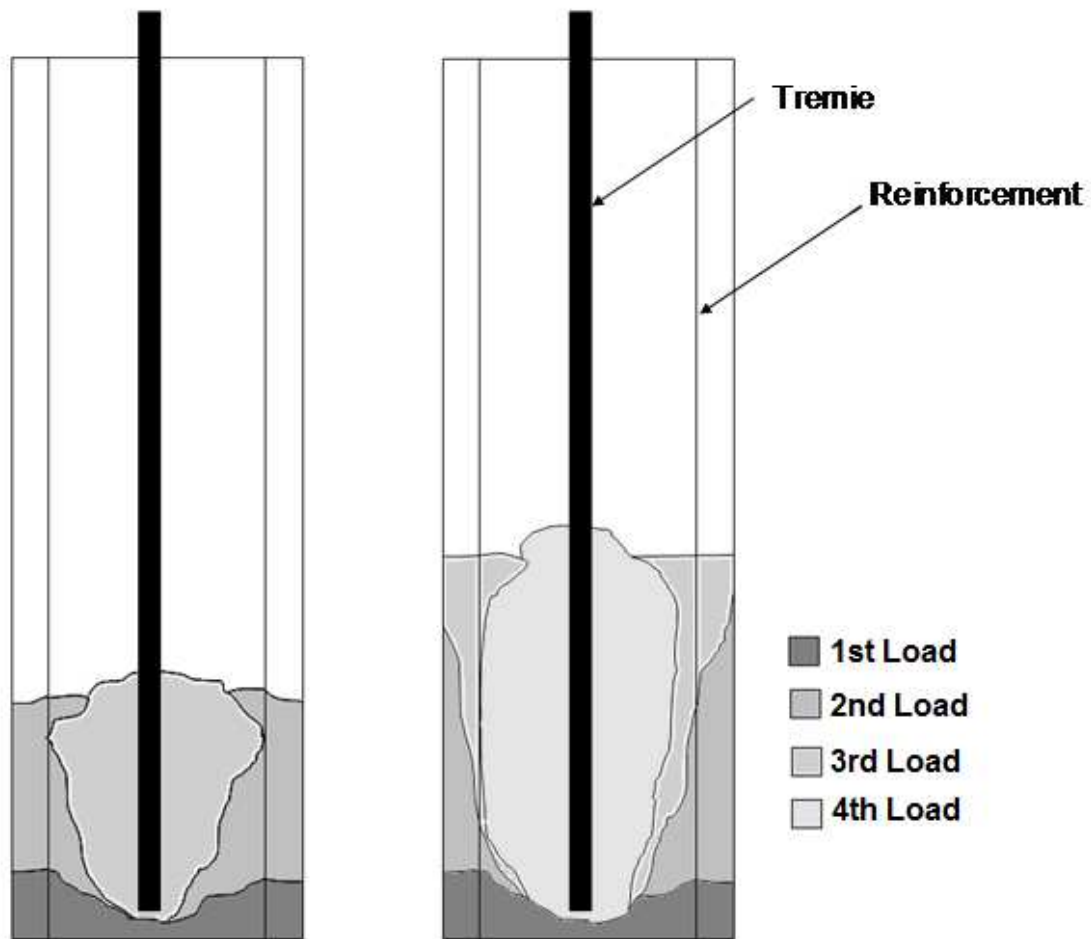
Figure 3.71: Hypothetical movement of high-viscosity concrete (such as ODSC)

The lower viscosity concretes, such as SCC, are affected by the confining stress in the same way. However, less of this concrete travels up around the tremie causing more of the previous concrete to rise and spread out into the shaft. At the tremie tip

elevation the concrete bulges into the previous concrete, then when the next concrete enters the shaft it pushes some of this bulge to the outside, upward and mixes, but little flows up the outside of the tremie. Therefore, most of the concrete stays near or just above the elevation that it is placed. A diagram with this theory is presented in Figure 3.72.
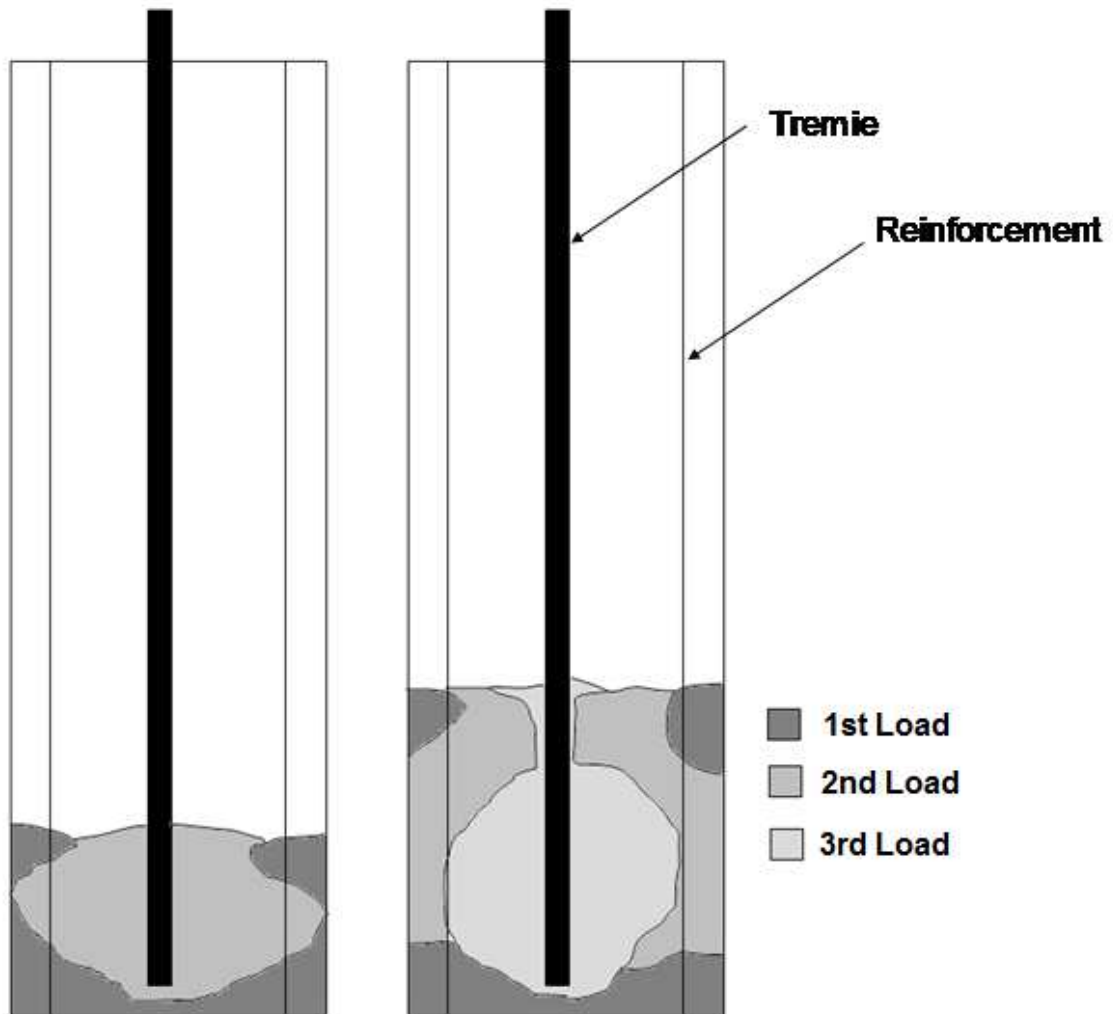
Figure 3.72: Hypothetical movement of low-viscosity concrete (such as SCC)

3.5.5.2 Flow of Imperfections

The pieces of shale, dropped in the shaft to simulate imperfections, were visible in the cut section of the SCC shaft. A picture these shale pieces are presented in Figure

3.73. These imperfections were located at elevations of approximately six and eight feet below the top of the shaft. If laminar flow occurred, the shale would have stayed on top of the concrete during the entire pour. This is further proof that laminar flow did not occur. Also, the pieces were dropped near the center of the shafts during the concrete placement. In order for the shale to end up located near the reinforcement cage, and in the cover region, the concrete must have flowed from around the tremie to the outside of the shaft.
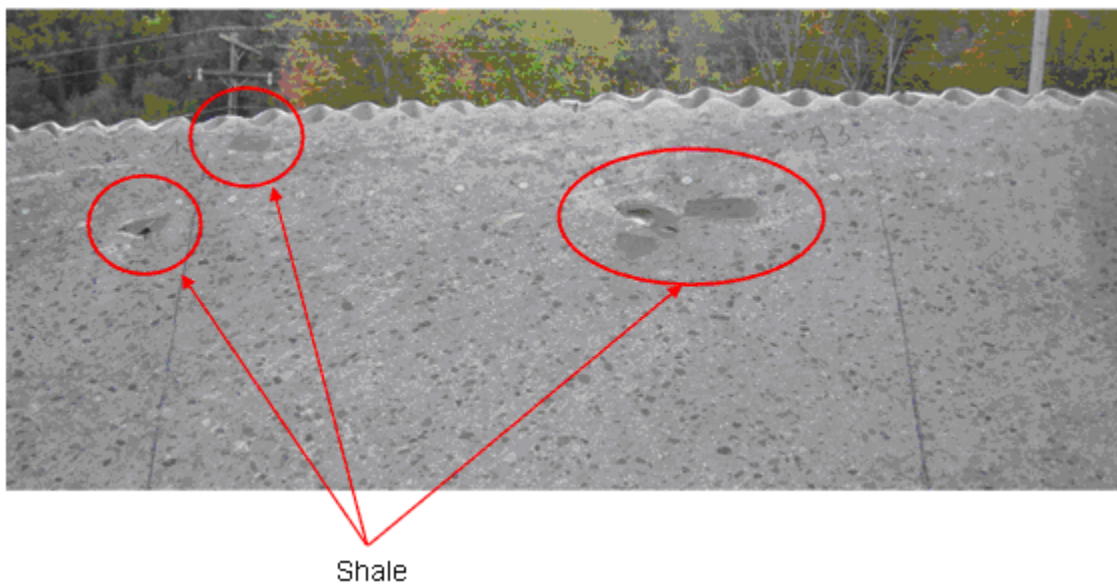
Figure 3.73: Shale pieces within the SCC shaft six to nine feet below the top

3.5.6   In-place Concrete Properties

To determine the properties of the in-place concrete, 72 cores were taken from the shafts. The cores were taken from the cross-sectional cuts. The results from the laboratory tests performed on these cores are presented below.

3.5.6.1 Compressive Strength and Modulus of Elasticity of Cores

The cores were acquired at elevations 7 ft and 20 ft from the top of the shaft. As described earlier, at each elevation cores were acquired from inside and outside of the

reinforcement cage. The results from the compressive strength of the cores and the 28-day molded cylinder results are presented in Figure 3.74. On the bottom of this figure, the labels refer to the depth from the top of the shaft followed by the location relative to the reinforcement cage.

To better show the differences in the compressive strengths inside versus outside of the reinforcement cage, the results from the outer cores were divided into the cores from inside the cage. The results of this are presented in Figure 3.75.

The results from the modulus of elasticity test are presented in Figure 3.76. The modulus of elasticity results show similar finding to the compressive strength data discussed above.

The ODSC cores had a significant difference between the strength of the concrete inside the reinforcement cage in comparison to the concrete strength outside the reinforcement cage. For this shaft, it appears that the reinforcement bars obstructed the flow and inhibited consolidation of the concrete.

The SCC core strengths were not affected by the reinforcement cage. For this mixture the cores acquired from the lower elevation were stronger.

The SCC-LP cores seem to be affected by the reinforcement cage, but not to the same extent as the ODSC cores. The apparent obstruction provided by the longitudinal reinforcement bars does not match the results from the modified J-Ring test. This test results concluded that the reinforcement cage would have a minimal effect on the concrete flow. The J-Ring test results from the SCC mixture concluded that heavy blocking would occur. It is apparent that this heavy blocking did not occur in the SCC shaft as its mechanical properties were similar within and outside of the reinforcement cage.
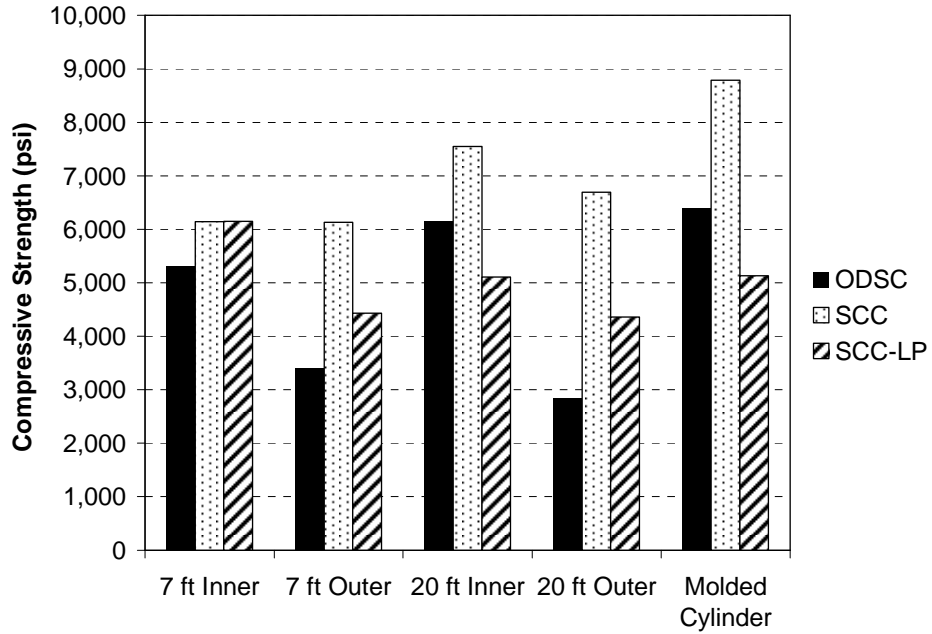
Figure 3.74: Compressive strengths of cores versus the molded cylinders
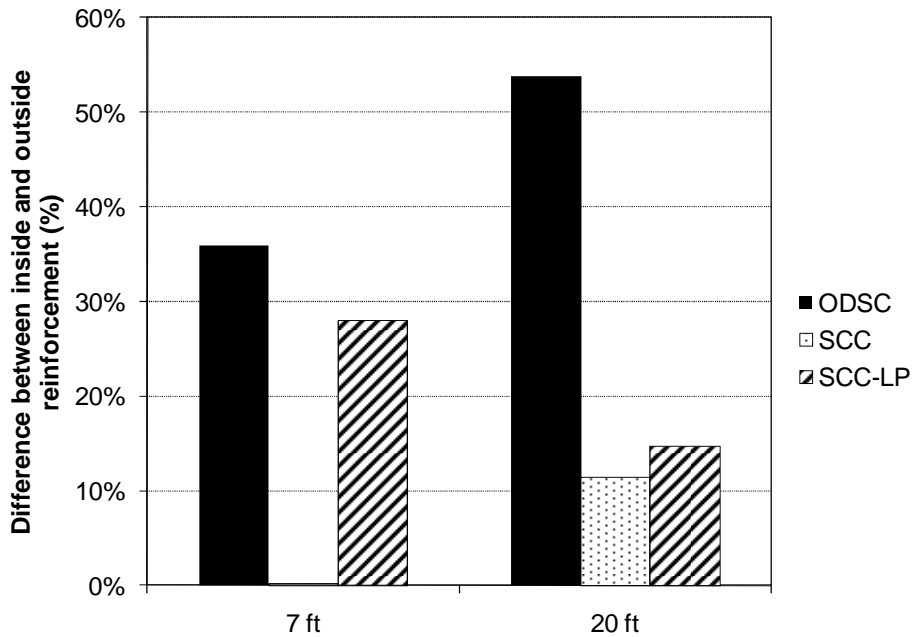
Figure 3.75: Difference in concrete compressive strength between the inside and the

outside of the reinforcement cage
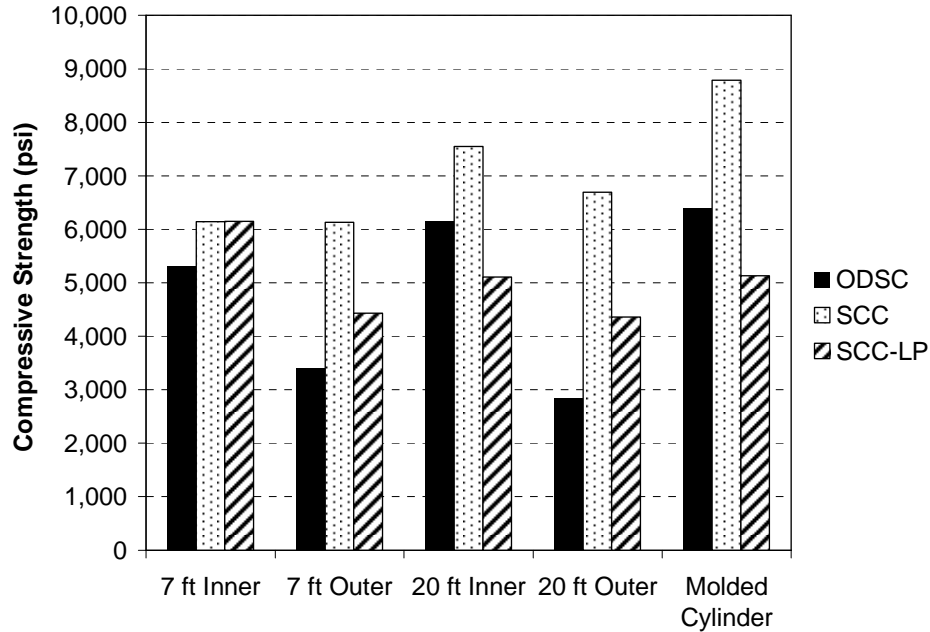
Figure 3.76: Modulus of elasticity of the cores compared to the molded cylinder

3.5.6.2 Chloride Ion Penetration Resistance of Cores

The results from the chloride ion penetration resistance test conducted on the cores are presented in Figure 3.77. For clarity, the table showing the value of the test results from the AASHTO code is repeated in Table 3.13.

The ODSC cores were variable in their resistance to chloride ion penetration. All the cores taken inside the reinforcement cage were sound with very low chloride ion penetration results. Many of the cores taken from outside the reinforcement cage of this shaft had to be cut multiple times to acquire an intact sample that would not leak during the test. The highest penetration result was recorded on a sample taken from this shaft.

The SCC cores showed the least amount of variance in comparison to the ODSC and the SCC-LP concrete. The SCC shaft's cores taken from inside the reinforcement cage were all of high quality with very low chloride ion penetration results. Cores from outside the reinforcement cage had low penetrability readings as well, with the highest

penetrability reading from a core that was located 7 ft from the top of the shaft, outside
the reinforcement cage.

The SCC-LP core results had low overall chloride ion penetration values with
very low permeability values for all the cores except for the outside cores acquired from
an elevation 20 ft from the top.
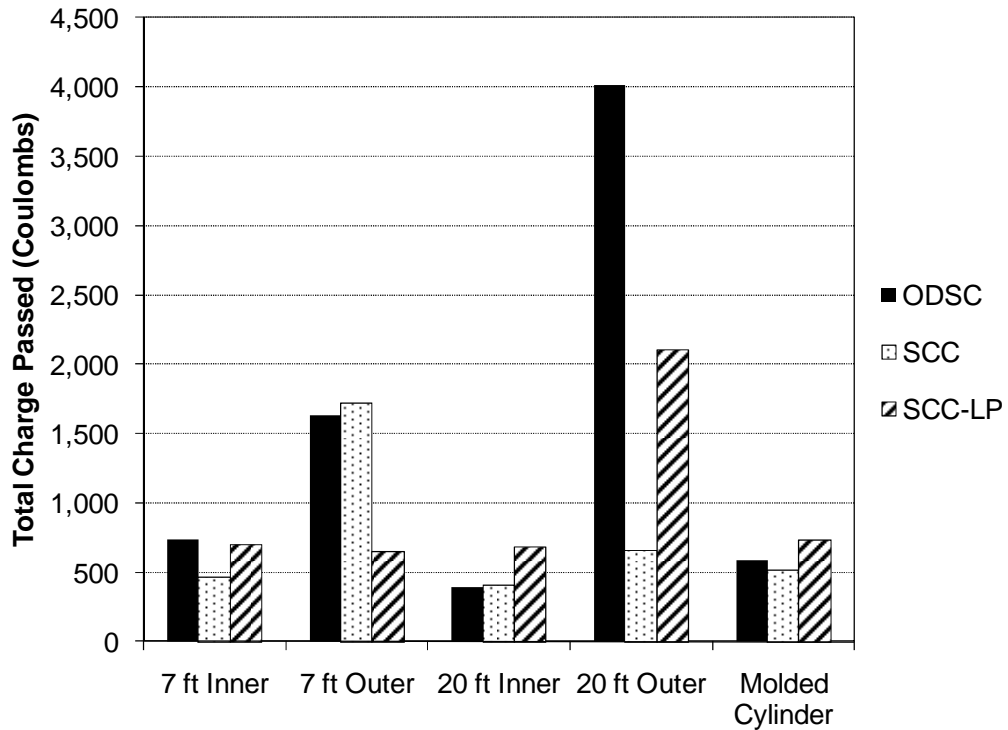
Figure 3.77: Chloride ion penetration test results of the cores compared to molded
cylinders

Table 3.13: Chloride ion permeability based on charge (AASHTO T 277)

Charge Passed (coulombs)	Chloride Ion Penetrability
>4,000	High
2,000–4,000	Moderate
1,000–2,000	Low
100–1,000	Very Low
<100	Negligible

3.6 Summary and Conclusions

These experimental shafts were constructed, tested, and exhumed to compare three different concrete mixtures for drilled shaft applications. An ordinary drilled shaft concrete mixture (ODSC) was the standard mixture currently used on the production bridge project. This mixture was compared to two different self-consolidating concrete mixtures. One mixture was a self-consolidating mixture designed by Auburn University and was designated SCC. The other self-consolidating mixture was an experimental mixture designed by Auburn University to minimize bleed water within the concrete. This SCC mixture was created by adding fine limestone powder to the mixture. This mixture was designated SCC-LP.

The SCC shaft had the best in-place properties of the three, by all measures. The cores taken from this shaft had the most consistent compressive strength and modulus of elasticity values throughout the cross sections. The visual inspection of the cut concrete revealed the least amount of imperfections, and the CSL results determined the least amount of minor disturbances in this shaft. In addition, during the construction of the drilled shaft, the top of the SCC maintained a horizontal surface during the placement of the entire shaft, unobstructed by the reinforcement cage. However, the fresh concrete property tests conducted showed different results. The SCC mixture had a higher static segregation as measured by the segregation column than the SCC-LP mixture and showed the most bleeding during the conventional bleed test. Bleed channels were observed within this shaft mostly in areas located on the inside of the reinforcement cage. It should be noted that the sample acquired for the segregation and bleed test had a slump flow value greater than the maximum allowed in the project specification. This concrete passed the VSI test, and therefore was allowed to be placed to note any disturbances that may occur from using this concrete. The only note that

may be made about the final product of the SCC shaft is the bleed channels that were noted, and these may have been caused by using this batch of concrete that was outside of the project specification.

The ODSC shaft appeared to have the worst overall condition. Large amounts of sand were built up on the outside of the shaft. Large voids were observed in the bottom corners of the shaft exposing the reinforcement cage to the outside surface of the shaft. Voids caused by poor consolidation were also observed in the cover region of the shaft. The compressive strengths of the concrete cores had the largest difference between the inner cores and the outer cores and indicated that poor-quality concrete was present in the cover region.

Overall, the SCC-LP shaft had the least amount of visual bleed channels. The bottom of this shaft had more bleed channels and indications of weak concrete than the SCC shaft but was still in better condition than the ODSC shaft. The SCC-LP mixture was the most difficult to produce at the concrete plant and many trucks were sent back from the jobsite.

Colored mortar cubes were placed into the shaft to determine how the concrete flows out of a tremie into the drilled shaft. Based on the cube locations and findings by Holley et al. (2005), it was hypothesized in this paper that the higher viscosity concrete, such as ODSC, flow in a layered manner inside the shaft, as shown in Figure 3.71. That is, the concrete layers on top of the previously placed concrete rather than displacing the concrete around the tremie and up the shaft. Less viscous concrete, such as SCC, flow in a bulging manner with less of the concrete flowing up around the tremie and instead displacing the previous layers up the shaft, as shown in Figure 3.72. More research is required to further evaluate this hypothesis and the actual flow patterns within a drilled shaft. This research project determined that self-consolidating concrete is a viable choice for drilled shaft applications.

Chapter 4

Evaluation of the Construction of Full-Scale Shafts in

Scottsboro, Alabama

4.1   Overview

Due to the success of the SCC mixture during the test shafts, the Alabama Department of Transportation (ALDOT) decided to require SCC for the drilled shafts on all the shafts of Phase II of the AL-35 Southbound bridge project.

The purpose of this phase of this project was to document the placement of SCC in large-scale production drilled shafts. This included written summaries of each concrete pour (Appendix D), based on photo and video documentation, as well as ALDOT and the concrete contractor's notes and reports. In addition, testing was conducted by the author to evaluate the following:

- Flow of the concrete within the shaft,

- Degree of concrete flow and VSI variability,

- Amount of concrete bleed water, and

- Evaluation of the use of a pressurized bleed test.

In evaluating the fresh and hardened concrete properties, as well as the procedure used to produce the drilled shafts, it is believed that a better understanding of SCC in drilled shafts can be acquired. Therefore, a educated decision may be made to require SCC in all challenging drilled shafts constructed in Alabama.

4.2   Construction Plan

4.2.1   Contributing Companies

Kirkpatrick Concrete, Russo Corporation, Scott Bridge Co., and ALDOT made up the team that produced and constructed the drilled shafts for this project. Kirkpatrick Concrete was the concrete contractor and created a SCC mixture specifically for this project. Russo Corporation was the drilled shaft contractor and performed the drilling and installation of the drilled shafts. ALDOT supplied technicians to perform the quality assurance testing. Scott Bridge Co. was the bridge contractor, performing surveying and moving barges for the drilled shaft contractor. GMS Testing was hired by the drilled shaft contractor to conduct the CSL tests to verify the completed shaft integrity. For research purposes, Applied Foundation Testing (AFT) was hired by Auburn University to perform specialized, non-destructive tests on selected drilled shafts.

4.2.2   Description of Production Shafts

This phase of the bridge project consisted of three piers to be installed over water (Piers No. 7, No. 8, and No. 9). Each of these piers consist of five, 8-ft diameter, drilled shafts. A figure of the drilled shaft locations for each pier is presented in Figure 4.1. The length of the shafts was determined by the quality of the rock encountered during drilling. Each shaft was constructed beneath approximately 40 ft of water into the Tuscumbia Limestone formation (Irvin and Dinterman 2009).

Each shaft was reinforced with a 7-ft diameter cage with 47 No. 11 bars around the diameter and No. 4 bar hoops at a 12-inch spacing. Eight CSL access tubes were tied onto the cage at an even spacing. A schematic of the cross section is presented in Figure 4.2. A picture of a typical reinforcement cage being lowered into an open shaft is presented in Figure 4.3.

Existing B.B Comer Bridge

1○   ○ 4
3○
2○   ○ 5

North

Existing N. Bound AL-35 Bridge

Figure 4.1: Location of drilled shafts for each pier (not to scale)

47 Bars #11 (Eq. Spa.)

7 ft

6 in. Cover

6 CSL Tubes

Hoops #4

8 ft
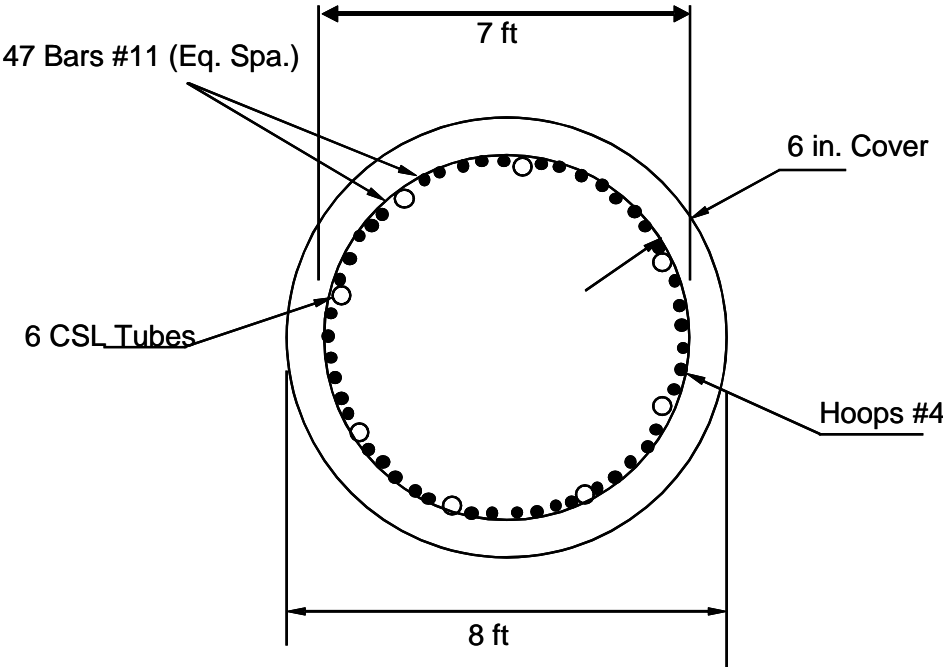
Figure 4.2: Cross section of reinforcement cage

Figure 4.3: Drilled shaft reinforcement cage

The drilled shafts were excavated and installed using methods determined by the drilled shaft contractor.

4.2.3 Testing Fresh Concrete Properties

To ensure quality concrete is installed into the drilled shafts, ALDOT technicians performed the following tests on concrete sampled from one truck of every 50 yd^3 of concrete placed:

- Air content,

- Unit Weight,

- Slump flow, and

- VSI.

To study and further analyze the placement of the concrete, an Auburn University representative performed the following tests on selected drilled shafts:

- Slump flow and VSI on every truck during shaft construction,

- Direct measurement of concrete flow through reinforcement,

- Conventional bleed test, and

- Pressurized bleed test.

In addition, Auburn University hired a specialty testing firm to perform the following on selected shafts:

- Installation of temperature probes,

- Additional CSL testing,

- Crosshole tomography testing (if necessary), and

- Gamma gamma integrity testing.

4.3   ALDOT Fresh Concrete Testing

4.3.1   SCC Fresh Concrete Training

Since the ALDOT technicians on this project never had experience of working with or testing SCC, two training days were set up to teach them how to perform the slump flow (ASTM C 1611 2005) and VSI (ASTM C 1611 2005: Appendix) tests. In addition to these training days, the author created a VSI manual to help explain the VSI test, this manual is presented in Appendix C. Included in this manual is a flow chart to help each technician make a fair judgment of the stability of the concrete. The training days were conducted at the Kirkpatrick batch plant near the project site. The author showed the invited ALDOT technicians how to perform the slump flow and VSI tests. Then each technician was allowed to perform these tests under the supervision of the author.

The author observed that the technicians had no problem repeating the slump flow test as instructed. However, since the VSI is a subjective assessment of the concrete's stability, the VSI test results varied between the technicians. In addition, the technicians seemed to put a relationship on the speed at which the concrete flowed and the stability of the concrete. If the concrete reached the slump flow value quickly, the technicians were likely to give the sample a higher VSI, without regard to the appearance of the concrete patty, resulting in a false test result.

The training helped ALDOT technicians to be able to perform the slump flow and VSI tests to the ASTM standards.

4.3.2 ALDOT Testing Area

To perform the fresh concrete tests and mold the concrete cylinders, ALDOT technicians set up a testing area located on the North side of the existing North bound AL-35 Bridge. An overhead picture depicting this area is presented in Figure 4.4. This area included a slump flow table, a flat shaded area to mold the cylinders, and a job trailer with temperature-regulated curing tanks.

4.4 Research Fresh Concrete Testing

4.4.1 Slump Flow and VSI Variability Testing

To test the variability of the slump flow and VSI of the concrete arriving at the jobsite, a sample of concrete from each truck was tested. Note that ALDOT only tested one truck for every 50 yd^3 of concrete delivered to the site. Each sample was taken after the concrete truck had dispensed half of its load into a 3 yd^3 bucket. The samples were acquired directly from the concrete chute into a 5-gallon plastic bucket and immediately carted to the testing area.

To perform these tests, a testing area was created on the existing North bound AL-35 Bridge. This area was located approximately 100 ft from the location where the

concrete trucks discharged their load. A picture of this testing area is presented in

Figure 4.5.
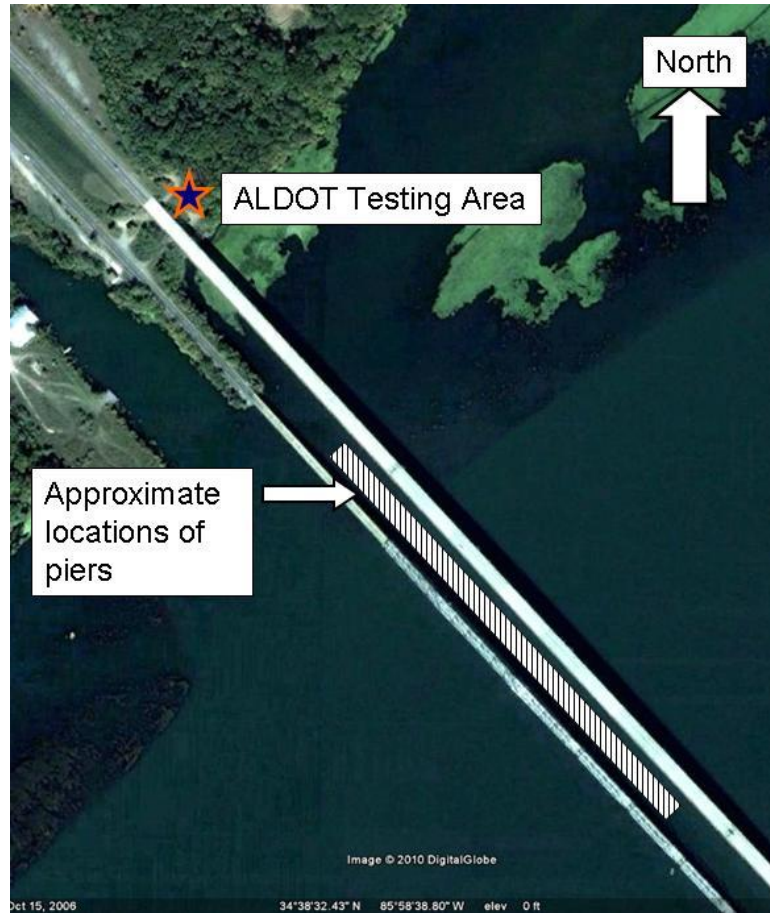
Figure 4.4: Location to ALDOT testing area in relation to pier locations (adapted

from Google Earth 2010)

Figure 4.5: Picture of slump flow and VSI variability testing area

4.4.2   Direct Measurement of Concrete Flow through Reinforcement

To test the ability of the concrete to flow through the reinforcement cage, the elevation to the top of the concrete outside of the reinforcement cage was measured. To determine this, a weighted measuring tape was lowered through the water to the concrete surface on the outside of the reinforcement cage. This measurement was taken while the depth of the concrete near the tremie was being measured. This depth to concrete in the center of the shaft was usually measured by a representative from the drilled shaft contractor after every other bucket was placed into the shaft. Due to the distance from Auburn, Alabama to Scottsboro, Alabama, it was difficult to recruit volunteers to take this measurement. Therefore, this test was conducted when possible during this project.

4.4.3 Bleed Test

To assess the concrete's ability to bleed, a bleed test was performed on concrete samples taken during the placement of selected shafts. This test was conducted in accordance with ASTM C 232 (2004). The samples were acquired from the batch truck after it had filled a 3-yd^3 bucket that was used to fill the shafts. The concrete was placed

into two 5-gallon buckets, capped with a plastic lid, and then transported by truck to the ALDOT testing area.

To conduct this test, a steel bucket (now referred to as bleed test bucket) 10-in. in diameter and 12-in. tall was partially filled with the sampled concrete. The concrete was placed into the bucket in one continuous motion to fill the bucket 1-in. from the top. A rubber mallet was used to strike the outside of the bucket 20 times (five times for each direction, North, South, East, and West). Then, a plastic lid was laid on the bleed test bucket. Every 10 min, a wooden block was placed under one side of the bleed test bucket to tilt the bucket. The bucket was kept tilted for two minutes. After this time, the plastic lid was removed from the bucket, and any bleed water visible was removed using an eyedropper and added to a beaker and recorded. The wooden block was then removed from the side of the bucket and the bucket was capped and left for another 10-min. period. This process was repeated every 10 min. for the first 40 min. and every 30 min. thereafter, until the concrete ceased to bleed. If no bleed water was visible after the first 90 min., the test was ended.

4.4.4   Pressurized Bleed Test

To approximate the amount of bleed water concrete generates under pressure, a pressurized bleed test was conducted. Initially, this test was conducted in the same manner as explained in Chapter 3; however, for the first series of tests conducted during this project the chamber pressures were changed. These pressures were decreased for this new series of testing. The new pressures were 60 psi over the first 10 min. and then 90 psi after an hour.

The testing apparatus was modified part-way through the project to better simulate the conditions in the shaft. This modification consisted of adding a pressurized beaker in place of the previous beaker. Pictures of this modification are presented in Figures 4.6 and 4.7. This was done to apply a back pressure that would simulate the

161

water pressure in the shaft. The apparatus was set up in the same manner, but a back

pressure of 20 psi was applied to the top of the sample. This back pressure was kept

constant for the entire test. In 10 min. intervals, the piston pressure was increased in the

following increments: 0, 15, 30, 45, and 60 psi. After this first hour, the pressure was left

unchanged for the next 60 min. The amount of water in the beaker was recorded every

5 min. for the first hour and every 15 min. thereafter.

Figure 4.6: Pressurized beaker installed on the pressurized bleed chamber

Figure 4.7: Pressurized bleed test being conducted with backpressure

4.5 Assessment of In-Place Concrete Integrity

To assess the integrity of the cured drilled shaft, the project specification, shown in Appendix B, requires CSL tests to be performed on each drilled shaft. For research purposes, some of the shafts had temperature probes installed. Additional research integrity testing was planned, but had not been conducted by July 6, 2010.

4.5.1 Installation of Temperature Sensors

To assess the heat of hydration of the in-place concrete, temperature sensors were installed onto the reinforcement cage of selected drilled shafts. To install the sensors, additional reinforcement bars were attached to the cage at three different cage elevations. Three sensors were attached at each of the following elevations:

- 4 ft from the bottom,
- ½ the shaft length from the bottom, and

- ¾ the shaft length from the bottom.

At each elevation, the sensors were attached at the following locations:

- One sensor onto the reinforcement cage,

- One sensor near the center of the cage on the reinforcement bar, and

- One sensor approximately 3-in. into the cover region of the shaft.

A schematic of the temperature probe locations is presented in Figure 4.8.

4.5.2   Concrete Integrity Testing

To assess the quality of the cured drilled shaft, the project specification, presented in Appendix B, states that every shaft must have CSL tests performed no earlier than 48 hrs and no later than 20 days after concrete placement.

Note: 3 sensors per bar, 9 sensors per shaft.
L = length of the concrete shaft
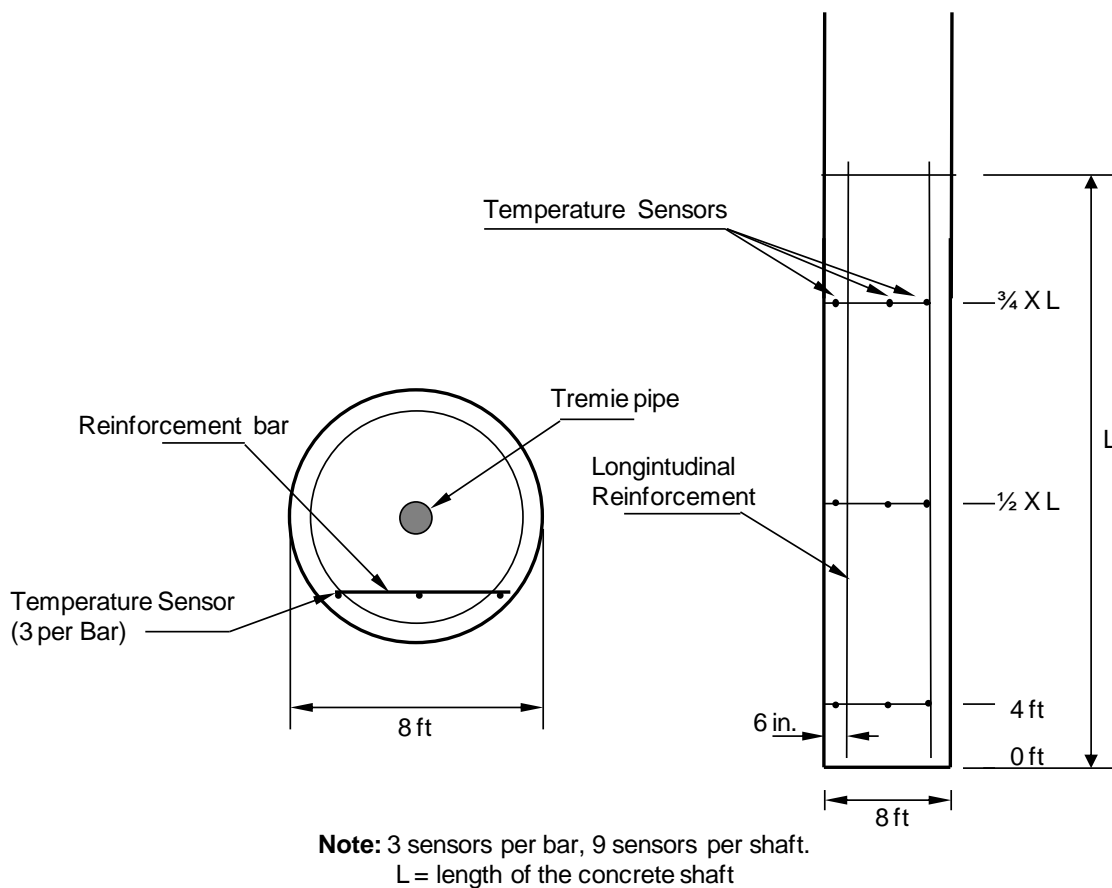
Figure 4.8: Location of temperature probes

4.6 Materials and Proportions

The mixture proportions for the concrete developed by the concrete contractor are presented in Table 4.1.

- Type I/II portland cement: The portland cement used for this project was manufactured by National Cement Co. in Ragland, Alabama.

- Class F fly ash: The Class F fly ash used for this project was provided by SEFA, Inc. and was manufactured in Cumberland, Tennessee.

- Coarse aggregate: The coarse aggregate used for this project was quarried by Vulcan Materials Co. in Scottsboro, Alabama. The gradation used for this was No. 78 stone, having a nominal maximum aggregate size of 0.75 inch.

Table 4.1: The drilled shaft SCC mixture proportions

Item	Mixture (SCC)
Type I/II cement content (lb/yd^3)	494
Class F fly ash content (lb/yd^3)	210
Water content (lb/yd^3)	282
No. 67 coarse aggregate, SSD (lb/yd^3)	1480
Fine aggregate content, SSD (lb/yd^3)	1390
Water-to-cementitious material ratio	0.40
Sand-to-total aggregate ratio	0.48
Hydration stabilizing admixture (oz/yd^3)	58.0
High-range water reducing admixture (oz/yd^3)	58.0
Viscosity modifier admixture (oz/yd^3)	28.0
Air-entraining admixture (oz/yd^3)	3.0

- Fine aggregate: The fine aggregate used for this project was supplied by Madison Materials in Summit, Alabama.

- Hydration stabilizing admixture: The mixture used DELVO® STABILIZER a BASF product. This admixture retards setting time by controlling the hydration of the cementitious materials.

- High-range water reducing (HRWR) admixture: The mixture used Glenium®
  7000, a BASF product, as the HRWR admixture.

- Viscosity modifier admixture: RHEOMAC® VMA 362, a BASF product, was
  used as the viscosity modifying admixture in this mixture.

- Air-entraining admixture: MB-AE^{TM} 90, a BASF product, was used as the air-
  entraining admixture in this mixture.

4.7   Quality Control and Quality Assurance of Concrete Placement

To ensure quality concrete is placed into the shafts, quality control and quality
assurance measures were taken.

4.7.1   Quality Control at Batch Plant

The Kirkpatrick concrete batch plant was located nine miles from the jobsite. A
map showing the concrete truck's travel distance is presented in Figure 4.9. To make
sure quality concrete gets delivered to the jobsite, a series of steps were taken by the
concrete batch plant. At the beginning of each day, a moisture sample was acquired
from the fine aggregate to make sure the moisture sensor was working properly. Once
this sensor was verified, an empty truck was backed under the batch plant and filled with
the aggregates, cement, fly ash, and water. The truck then spun the drum at a high rate
for 60 revolutions. After this time, the chemical admixtures were added to the truck.
This sequence was used to limit the amount of cement balls in the concrete mixture. To
also limit the cement balls, each truck was filled with 6 yds^3 of concrete. In this way
each truck could fill two 3 yd^3 concrete buckets at the jobsite. Both the late addition of
the chemicals and the use of 6 yd^3 trucks were changes that were implemented after the
placement of the first shaft. On this shaft, numerous cement balls were visible in most of
the ready mix trucks.

4.7.2 Quality Assurance at the Jobsite

At the jobsite, the quality assurance testing was conducted by ALDOT technicians in their testing area. The ALDOT selected truck was stopped at this area before heading to the bridge. In this area the truck placed the concrete from the very back of the truck into 5-gallon buckets. These buckets were then loaded into the back of a pick-up truck and driven a short distance to the area where the quality control testing was conducted.
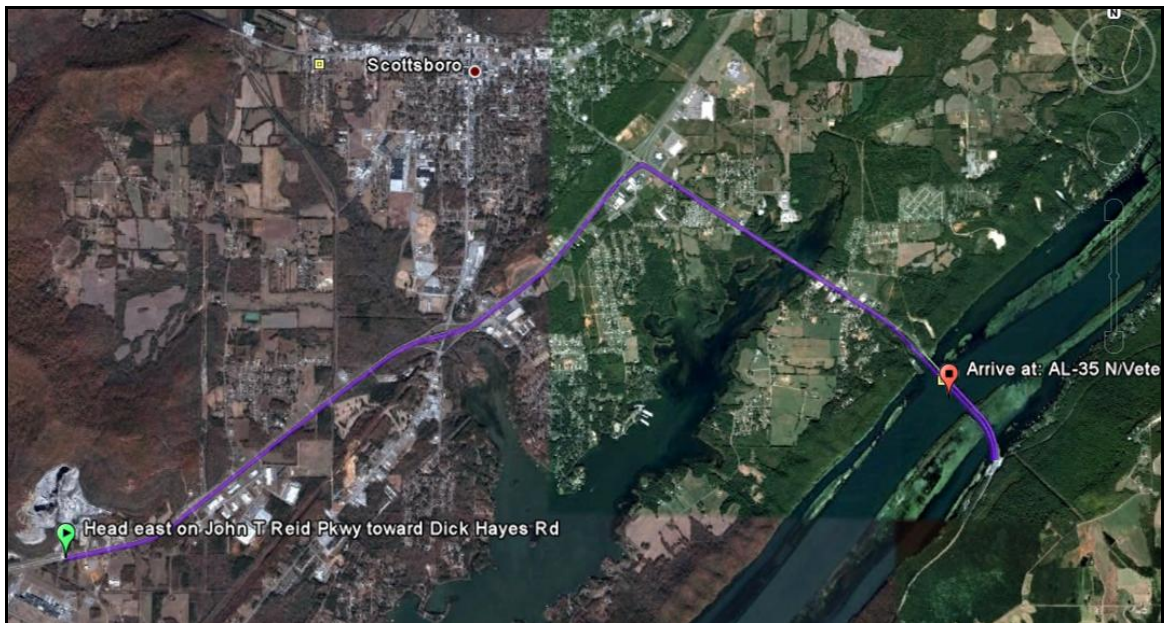
Figure 4.9: Route from batch plan to project location (adapted from Google Earth 2010)

Starting with the second shaft of Pier No. 8 (the seventh shaft to be installed), the drum of the batch truck was rotated at a high rate in order to mix the concrete before sampling. If the sampled concrete met the required specifications, it was allowed to proceed to the bridge. The VSI of the concrete should have been tested per the project specification (Appendix B); however, to the author's knowledge the ALDOT technicians did not perform the VSI test.

During the concrete placement, an ALDOT technician measured the depth to the top of the concrete after every other concrete bucket discharged into the tremie hopper. This measurement allowed the technician to plot a graph of actual concrete volume versus the theoretical concrete volume. This graph is known as a concrete curve (O'Neil and Reese 1999). An example of a concrete curve from is presented in Figure 4.10.
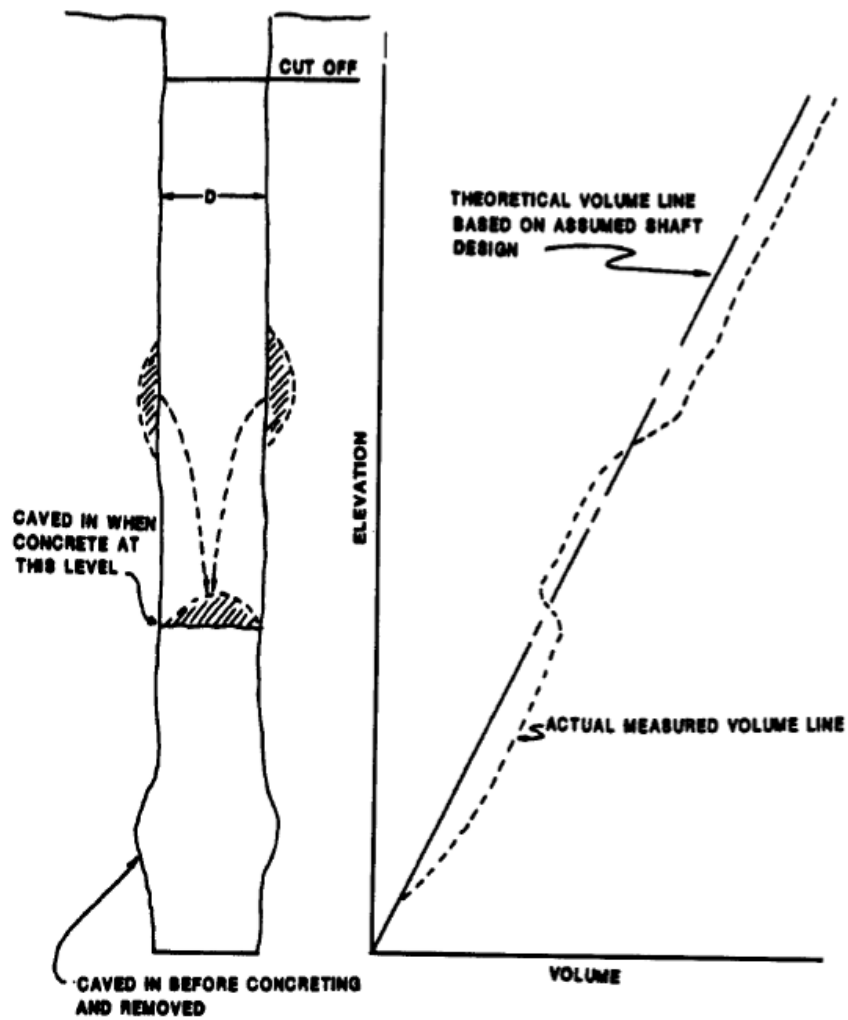
Figure 4.10: Comparison of actual amount of concrete to theoretical amount of concrete (ADSC/DFI 1989)

4.8   Production Process

The drilled shafts were constructed using the wet method; with either a full length permanent casing, or a socketed temporary casing. For Pier No. 7, a permanent full-length casing was installed for each shaft socketed into the limestone bedrock. The limestone in this geology had fractures in the rock, so it was decided to keep the cased holes full of water and place the concrete by tremie through the water.

A design change, unrelated to this research effort, was implemented after the completion of the shafts for Pier No. 7. Due to this design change, Pier No. 8 was constructed with shorter drilled shafts that did not have full length casing, in order to increase the skin friction of the shaft.

To produce the drilled shafts, many steps had to be taken. Before the concrete was batched, the drilled hole was prepared. To prepare the hole, the following steps were generally used on this project:

- o   After the driller reached the required depth, the debris was cleared from the bottom of the shaft,
- o   The assembled reinforcement cage was spliced to a less congested cage that made sure the CSL tubes were straight the entire length of the shaft, as shown in Figure 4.11,
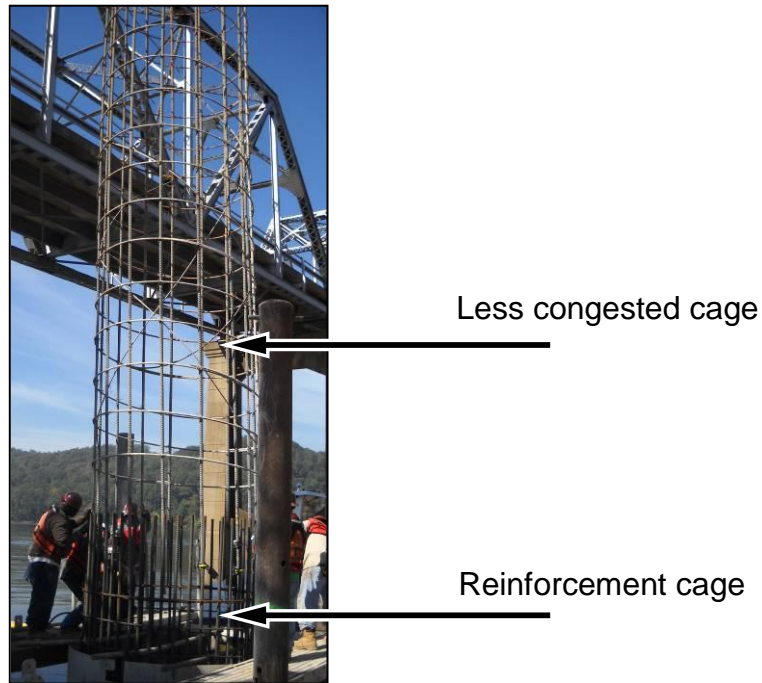
Figure 4.11: Less congested cage spliced to the reinforcement cage

- The assembled cage was then lowered into the hole to allow the CSL tubes to be lowered and put into place, then a crane lifted the cage slowly out of the hole to allow the CSL tubes to be attached to the entire length of the cage,

- As the cage was lowered back into the hole, spacers were installed to make sure the cage stayed in the center of the hole during the concrete placement,

- A 10-in. tremie pipe, with a tremie hopper on top, was then lowered into the hole, and placed on the bottom of the shaft, as shown in Figure 4.12,
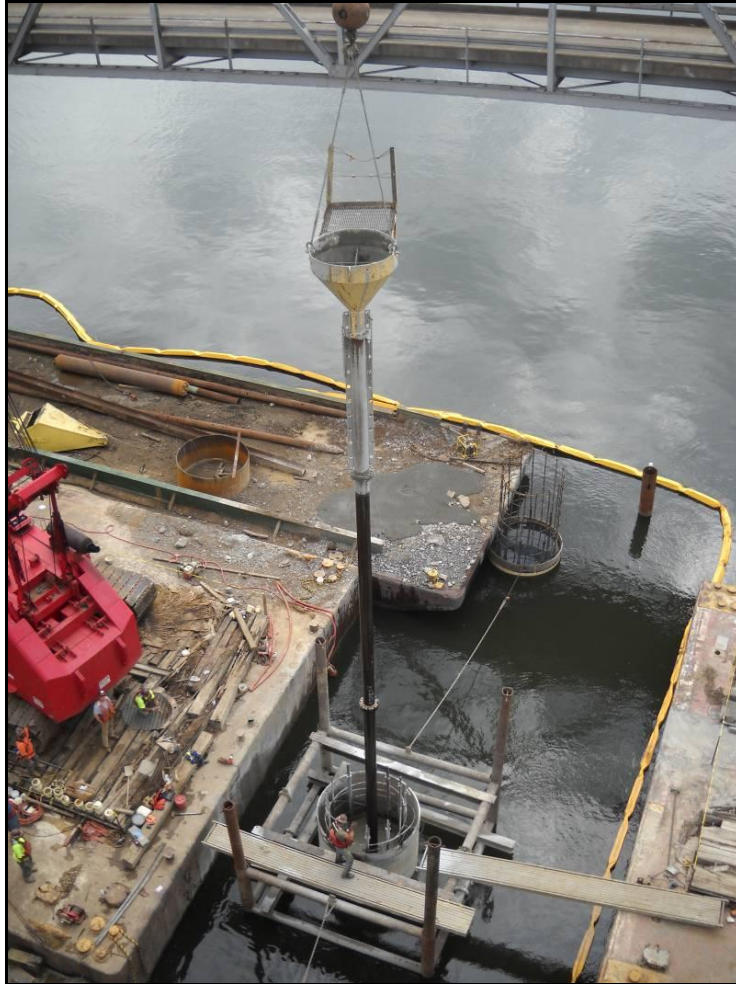
Figure 4.12: Tremie and hopper being lowered into the shaft

o The crane then lifted the hopper slightly and steel bars are slid under the hopper to make sure the hopper was secure and the tremie was just slightly above the bottom of the shaft, and

o Just before the concrete is discharged into the hopper, a wet foam plug, known as a pig, was pushed into the top of the tremie; a picture of this plug is presented in Figure 4.13.

Figure 4.13: Foam plug to separate the water in the tremie from the initial concrete

placed into the tremie

While the hole was being prepared, traffic control markers were set to block off the

left hand lane of the North bound AL-35 bridge. This blocked lane was used by the

ready mix trucks to discharge the concrete into the concrete buckets. Once prepared,

the batch plant was contacted to send the concrete trucks to the jobsite.

Once the concrete was deemed suitable at the batch plant, the ready mix concrete

truck was sent to the jobsite. The first truck would pull into ALDOT's testing area to have

its properties checked for compliance with the project specifications. The trucks would

then proceed to the bridge. On the bridge, the truck would discharge its load into one of

four, 3-yd^3 concrete buckets. A picture of one of these buckets waiting to be filled is

shown in Figure 4.14. Since the trucks were filled with 6 yd^3 of concrete, one truck

would fill two buckets.

Figure 4.14: Three-cubic yard bucket waiting to be filled on the bridge

These buckets were controlled by two cranes that were located on the barges. Each crane would take turns moving an empty bucket from the barge to the bridge to be filled, and then back to the barge until all four buckets were filled. Once all four buckets were filled, and the ALDOT technicians state the concrete was acceptable, each crane moved one bucket near the tremie hopper. With both buckets near the hopper, one bucket was selected to discharge into the hopper. As soon as this bucket was empty, the other bucket was moved into position over the hopper and discharged. This continued until all of the full buckets were discharged into the shaft. At this time one crane and one bucket were used to continue the concrete placement. Pictures of the buckets being discharged into the hopper are presented in Figures 4.15 and 4.16.

Figure 4.15: Three cubic yard buckets being discharged into the tremie hopper of Shaft

No. 2 of Pier No. 7
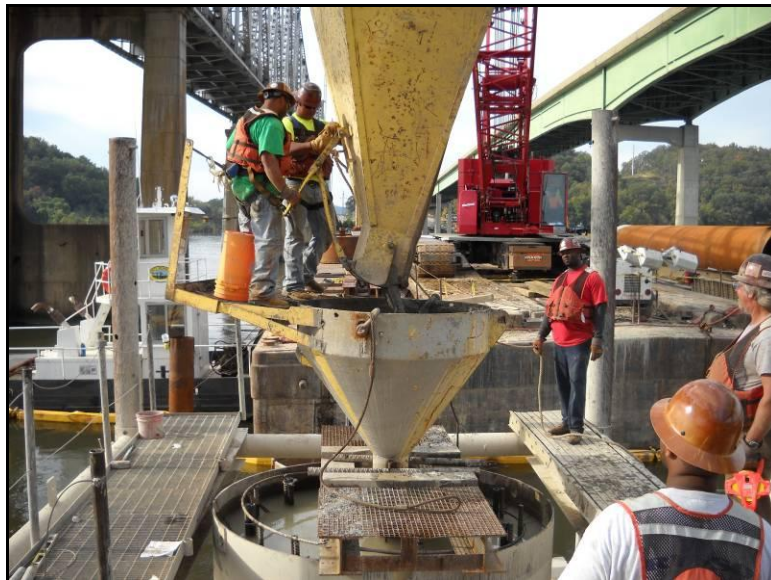
Figure 4.16: Three-cubic yard bucket being discharged into the tremie hopper of Shaft

No. 1 of Pier No. 7

The concrete placement was completed when the depth to the concrete at the center of the cage was a few feet above the required depth. This was done to take into account the few feet of weak concrete that mixed with the water, known as laitance (O'Neil and Reese 1999).

4.8.1   Summary of First Shaft's Concrete Placement

The first shaft constructed on this project was Shaft No. 4 of Pier No. 7. The concrete was placed into this shaft using a pump truck and pump line attached to a tremie. The pump truck was positioned on the left hand lane on the existing North bound bridge on U.S. Hwy 35. The placement of concrete into this shaft did not occur smoothly. The tremie pipe clogged multiple times, the pump lines became disconnected on the barge multiple times due to pressure in the line, and numerous cement balls were visible in the concrete from the batch trucks. A picture of the cement balls on the pump truck grate is shown in Figure 4.17. A picture of the pump line configuration is shown in Figure 4.18. A more detailed account of the concrete placement for this shaft is in the daily logs found in Appendix D.

Figure 4.17: Cement balls on the pump truck grate

Figure 4.18: Pump line and tremie configuration

The CSL logs from this shaft showed low quality concrete in some locations. Cores were taken from the shaft and confirmed the CSL results. Therefore, micropiles were installed to provide additional support to the shaft filled with defects.

The cause of all the problems that occurred on this date are unknown. The test shafts described in the Chapter 3 were installed using a pump truck without the problems described on this shaft. However, since the test shafts, the concrete contractor had changed the SCC proportions from those used for the test shafts. Also, a different drilled shaft crew was used to place the concrete for the production shafts. Finally, for the test shafts the pump truck for the test shafts was located at an elevation slightly below the top of the shafts; unlike on the production shafts, where the pump truck was located on the bridge many feet above the top of the shaft. This shaft shows that a test shaft should be conducted for all projects to test the materials and construction methods used.

4.8.2 Summary of Tremie Leak

After the problems that occurred with the production of Shaft No. 4 of Pier No. 7, it was decided to use a gravity fed tremie to place the concrete for the remaining drilled shafts on this project.

The next shaft was Shaft No. 5 of Pier No. 7. During the placement of the concrete in this shaft, the concrete stopped flowing out of the tremie hopper after approximately 3 yd^3 of concrete were placed into the hole. Due to lack of concrete flow down the tremie pipe, the concrete placement was cancelled on the first day this shaft was attempted. The hole was cleaned out before the concrete set and another attempt was made to place the concrete in this shaft. During the second concrete placement of this shaft, the concrete flow ceased again when approximately 3 yd^3 were placed into the shaft. The placement was ceased at this time to determine the causes of the placement problems.

Small stones were dropped into the top of the tremie hopper and splashing was heard in the tremie pipe, suggesting water was somehow entering into the tremie pipe. To determine how the water was entering the shaft, the pipe was lifted out of the shaft and a steel plate was welded to the bottom of the tremie. During this time the concrete placed into the hole was removed. The tremie pipe was then lowered into the hole and submerged for approximately 7 min. After this time, the tremie was lifted out of the hole. While removing the tremie from the hole, water was seen leaking from a gasket on the tremie pipe, as shown in Figure 4.19.

Figure 4.19: Water leaking from gasket on Shaft No. 5 of Pier No. 7

With the location of the leak determined, the gasket was replaced that afternoon and bolts were installed in every bolt hole around the gasket, instead of every other bolt like previously used. To determine the condition of the fixed tremie pipe, on the next day, a plate was welded to the bottom of the tremie and the tremie was lowered into the hole for a few minutes. After removal no leaks were seen. The placement of the shaft using the fixed tremie pipe did not have any delays or problems.

4.8.3 Discussion of Concrete Placement

Other than the problems discussed in Section 4.8.1 and Section 4.8.2, no other problems or major delays occurred during the placement of the concrete. The following is a summary of notes taken during the concrete placement:

- Each successful concrete placement took approximately two to three hours to complete,

- The drilled shaft contractor seemed to like the SCC better at lower slump flows because the concrete looked and acted more stable,

- The concrete appeared to flow freely through the tremie and in no instance was the tremie moved to help the concrete flow,

- Even without moving during the entire construction process, the tremie did not have any problems being removed from the shaft, and

- No problems occurred that would make one believe that the concrete started setting before the shaft was completely filled.

The following occurrences did not delay or cause problems, but were noted during construction:

- Some cement balls were still visible within some batch trucks, but did not cause any problems, and

- Each shaft was over filled a few feet with concrete due to the possible presence of laitance.

4.9 Fresh Concrete Testing Results

The results of the tests conducted by the author for this project are presented in this section.

4.9.1 Slump Flow and VSI

o Shaft No. 4 of Pier No. 8

To test the variability of the concrete that arrives to the jobsite, slump flow and VSI tests were performed. The results of these tests taken during the installation of Pier No. 8, Shaft No. 4 are presented in Figure 4.20. The statistics calculated from these data are presented in Table 4.2. Two of the trucks on this date had their fresh concrete properties tested by ALDOT. These tests passed, but four of the trucks that were tested by the author were below the project specification for slump flow values (18 in. to 24 in.). However, every sample passed the VSI test. Most samples tests received a VSI value of 0, while Load 8 and Load 13 received VSI values of 0.5.

The maximum slump flow value was well within the specification; however, the minimum slump value was in some cases well below the minimum specified value. It should be noted that all the trucks tested by ALDOT met the project specification.
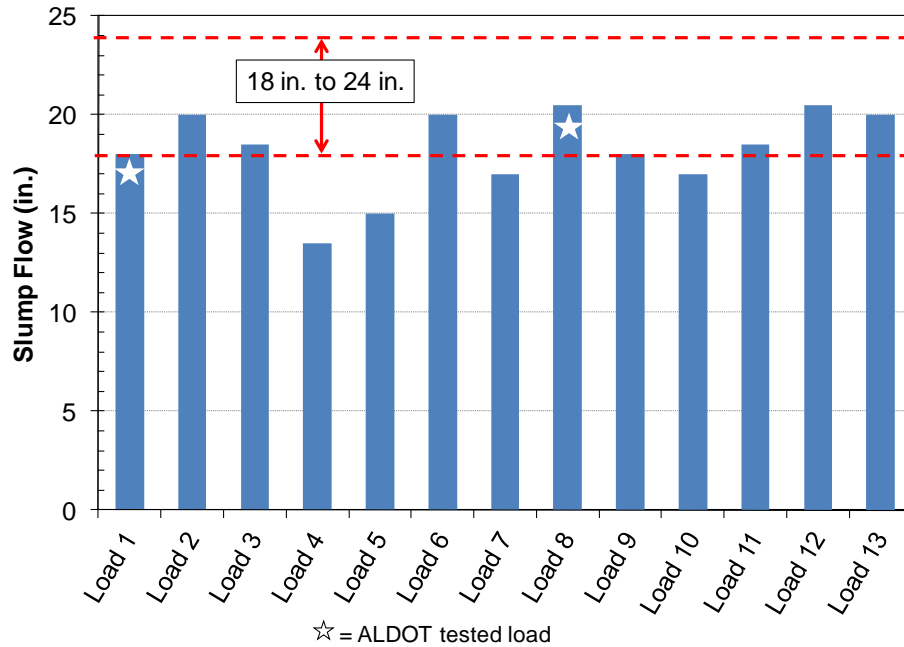
Figure 4.20: Slump flow results from Shaft No. 4 of Pier No. 8

Table 4.2: Slump flow statistics from shaft No. 4 of Pier No. 8

Slump Flow Results	Value (in.)
Average	18.0
Maximum	20.5
Minimum	13.5
Standard Deviation	2.2
Range	7.0

o Shaft No. 5 of Pier No. 8

The results of the tests of Pier No. 8, Shaft No. 5 are presented in Figure 4.21. The statistics calculated from this data are presented in Table 4.3. As with the previous shaft, two trucks were sampled to test the fresh concrete properties. These tests

passed, but six of the trucks tested on the bridge were below the project specification.

The first truck tested and passed by ALDOT, was tested on the bridge and its slump flow

was just below 18 in. and this did not meet the project specifications.

The average slump flow for this day was below the minimum project specified

value. The concrete was very stable and all the loads tested on this date obtained a VSI

value of 0.
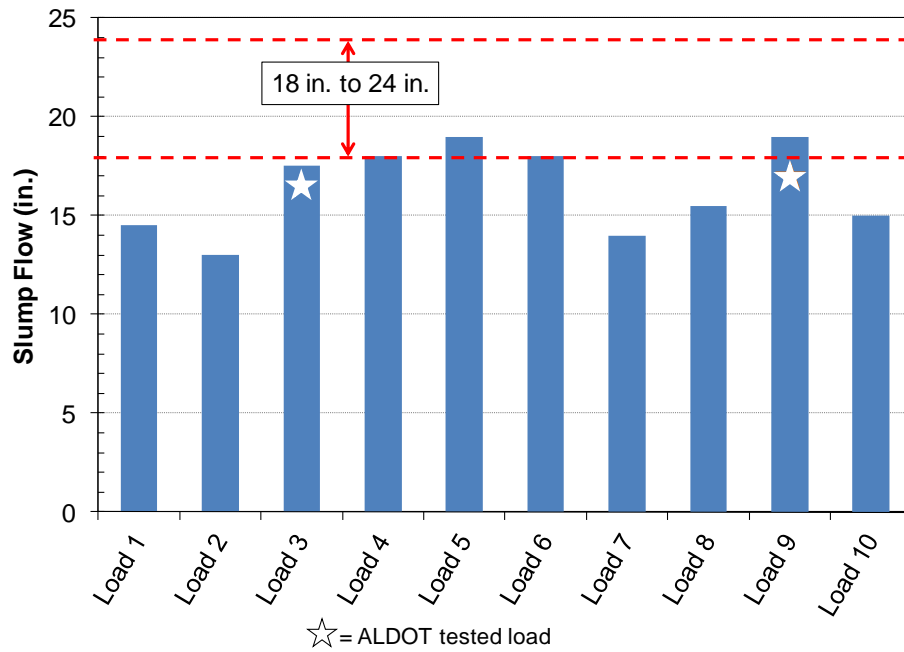
Figure 4.21: Slump flow results from Shaft No. 5 of Pier No. 8

Table 4.3: Slump flow statistics from Shaft No. 5 of Pier No. 8

Slump Flow Results	Value (in.)
Average	16.5
Maximum	19.0
Minimum	13.0
Standard Deviation	2.2
Range	6.0

o   Shaft No. 3 of Pier No. 8

The results of the tests of Pier No. 8, Shaft No. 3 are presented in Figure 4.22. The statistics calculated from this data are presented in Table 4.4. Load 5 was tested by ALDOT and had a slump flow that exceeded the project specifications. This truck was sent back to the batch plant and not poured into the shaft.
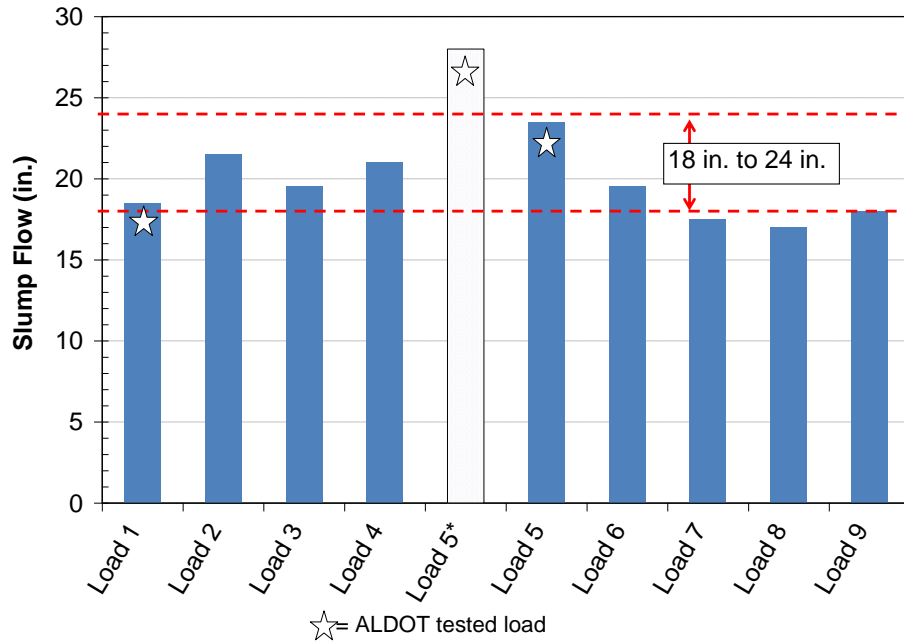
Figure 4.22: Slump flow results from Shaft No. 3 of Pier No. 8

Table 4.4: Slump flow statistics from Shaft No. 3 of Pier No. 8

Slump Flow Results	Value (in.)
Average	20.5
Maximum	23.5
Minimum	17.0
Standard Deviation	2.2
Range	6.5

4.9.2 Direct Measurement of Concrete Flow through Reinforcement Cage

These measurements were acquired twice during the project. The measurements acquired from Shaft No. 1 or Pier No. 7 are presented in Figure 4.23. The measurements acquired from Shaft No. 3 of Pier 8 are presented in Figure 4.24. The differences in the concrete depth in the inside versus the cover region for these two shafts are presented in Figure 4.25.

The concrete in Shaft No. 1 of Pier 7 appeared to flow unobstructed by the reinforcement cage, and in some cases the elevation in the cover region of this cage was a few inches higher than the concrete measured in the center. These measurement varied between -3.5 in. and 5 in. during the concrete placement. On the contrary, the concrete in Shaft No. 3 of Pier 8 did seem to be obstructed by the reinforcement cage with a maximum elevation difference of 22 in. and at no time was the elevation in the cover region higher than the elevation in the center of the shaft.
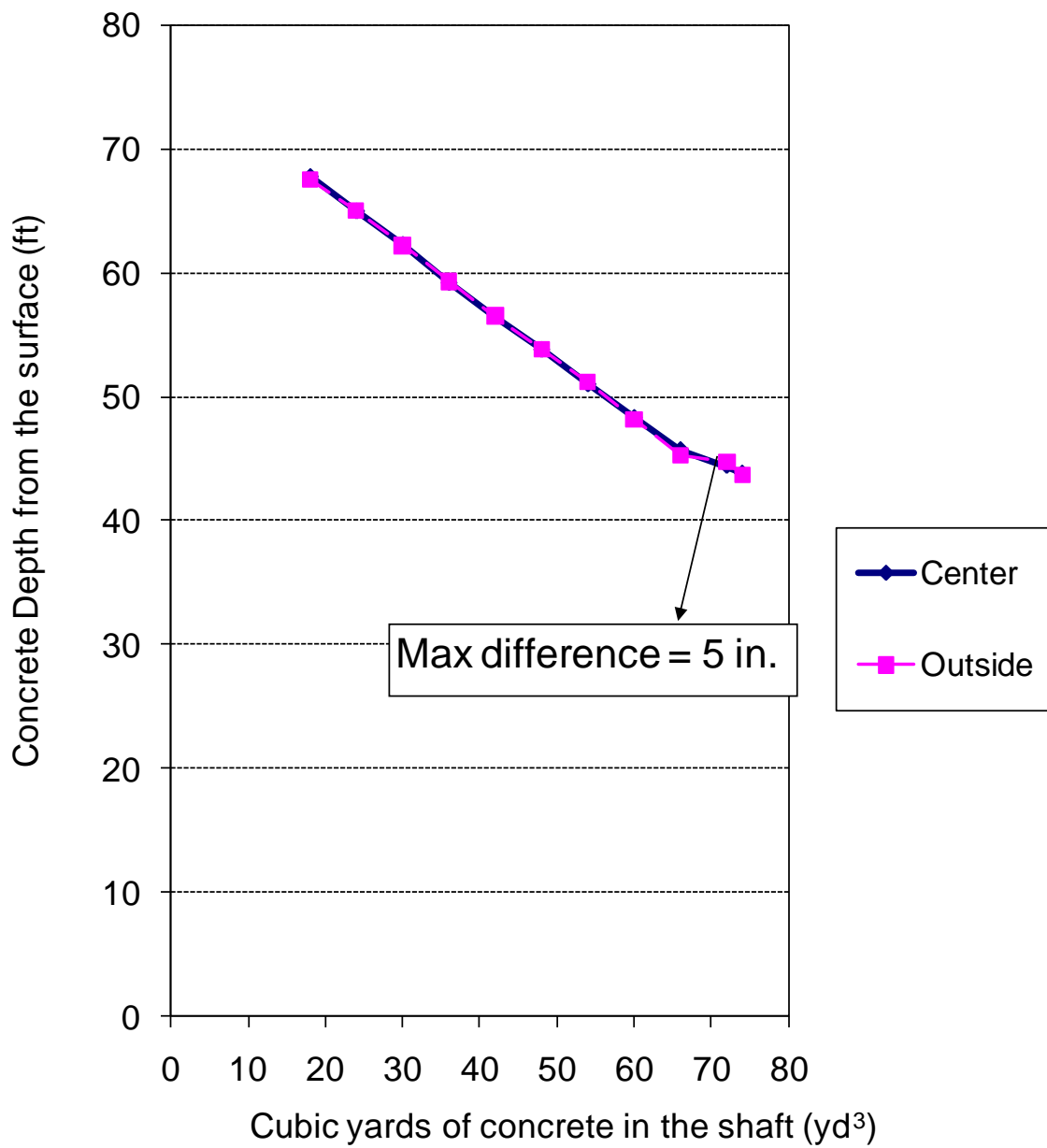
Figure 4.23: Depth of concrete outside the reinforcement cage compared to the depth of

concrete taken near the center of the shaft from Shaft No. 1 of Pier No. 7
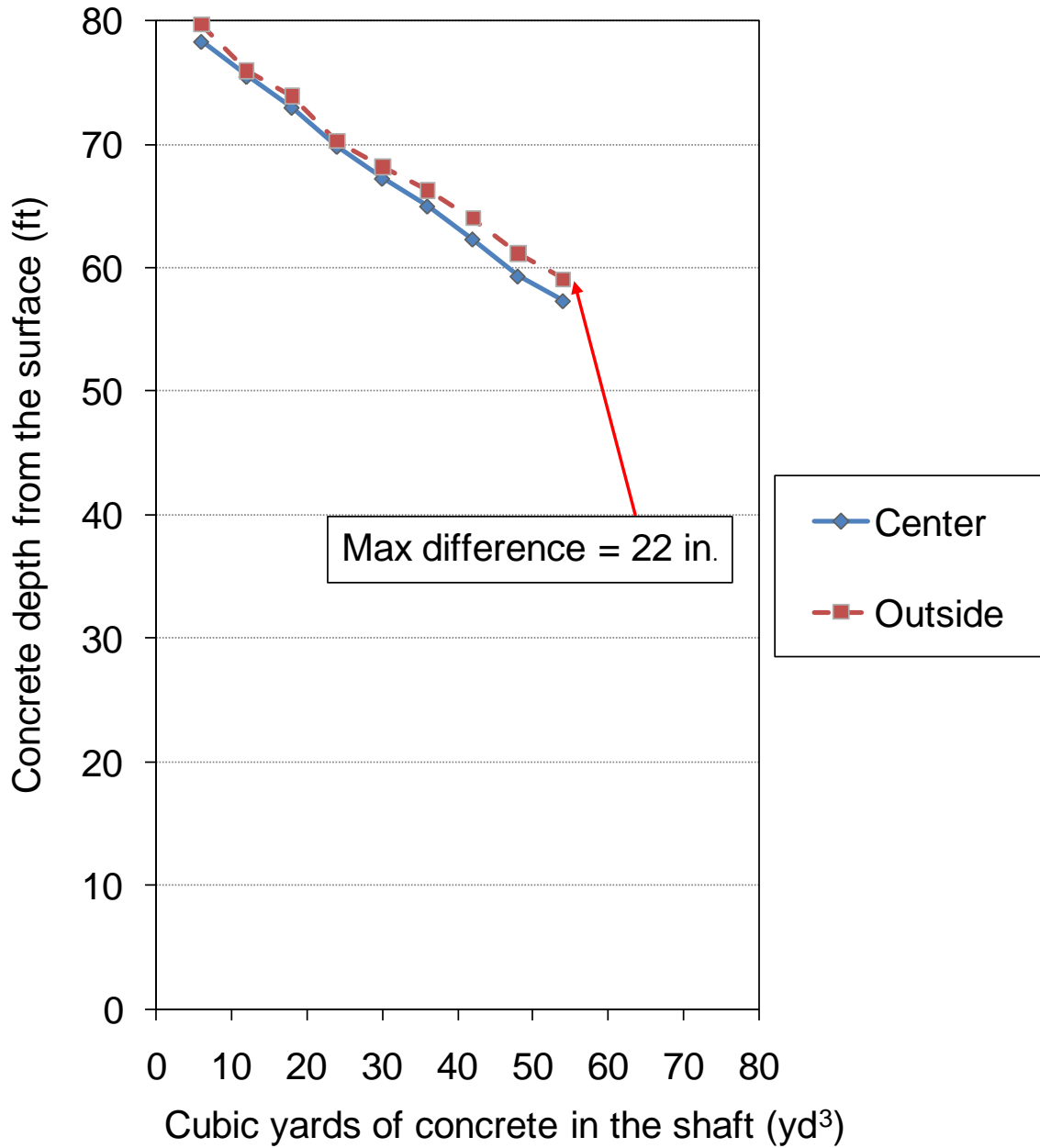
Figure 4.24: Depth of concrete outside the reinforcement cage compared to the depth of

concrete taken near the center of the shaft from Shaft No. 3 of Pier No. 8
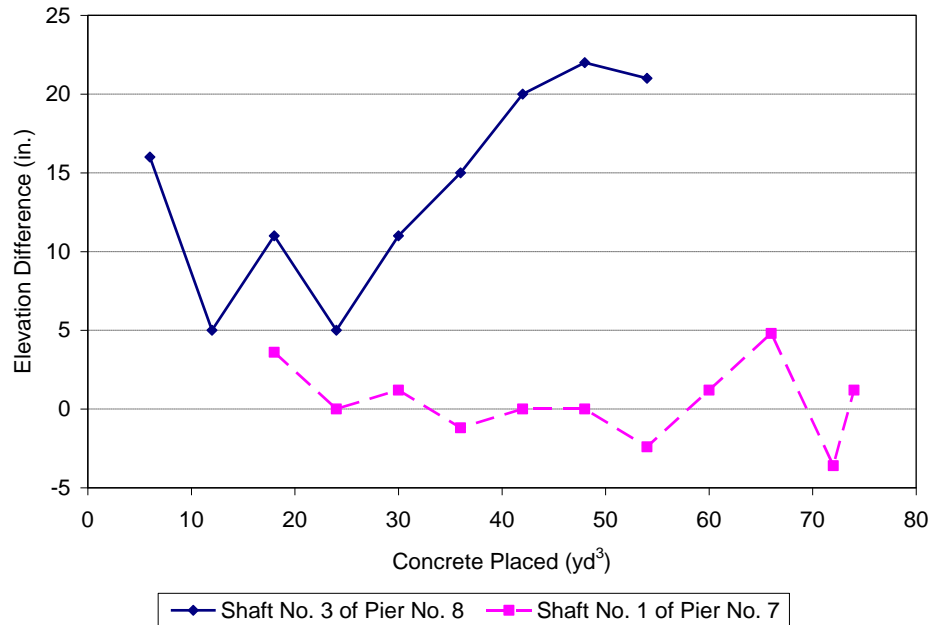
Figure 4.25: Elevation measurements compared

4.9.3 Conventional Bleed Test and Pressurized Bleed Test

Bleed tests were conducted on Shaft No. 4, No. 5, and No. 3 of Pier No. 8. The results from these tests, as well as the total results from the pressurized bleed tests, are presented in Table 4.5. Every conventional bleed test was conducted for at least 1.5 hours and every test had a result of 0.0 percent bleeding.

The pressurized bleed test was performed on the same drilled shafts as the conventional bleed test. The full results from Shaft No. 4 of Pier No. 8 are not shown due to a flawed test. The piston was believed to have been dirty and therefore the pressure applied to the piston was unknown. This being said, the concrete from this test did exhibit bleeding under the unknown pressure; whereas, the results from the conventional bleeding test showed no bleed water. The results from Shaft No. 3 and Shaft No. 5 are presented in Figure 4.26 and Figure 4.27, respectively.

The test method conducted on the concrete from Shaft No. 3 included the addition of backpressure to the piston. The pressure shown on the right hand axis is the upward piston pressure minus the initial 20 psi back pressure.

Table 4.5: Results from conventional and pressurized bleed tests

Pier No.	Shaft No.	Conventional Bleed Result (%)	Pressurized Bleed Result (%)
8	4	0.0	25.6*
8	5	0.0	31.7
8	3	0.0	26.4

*Result from unknown pressure
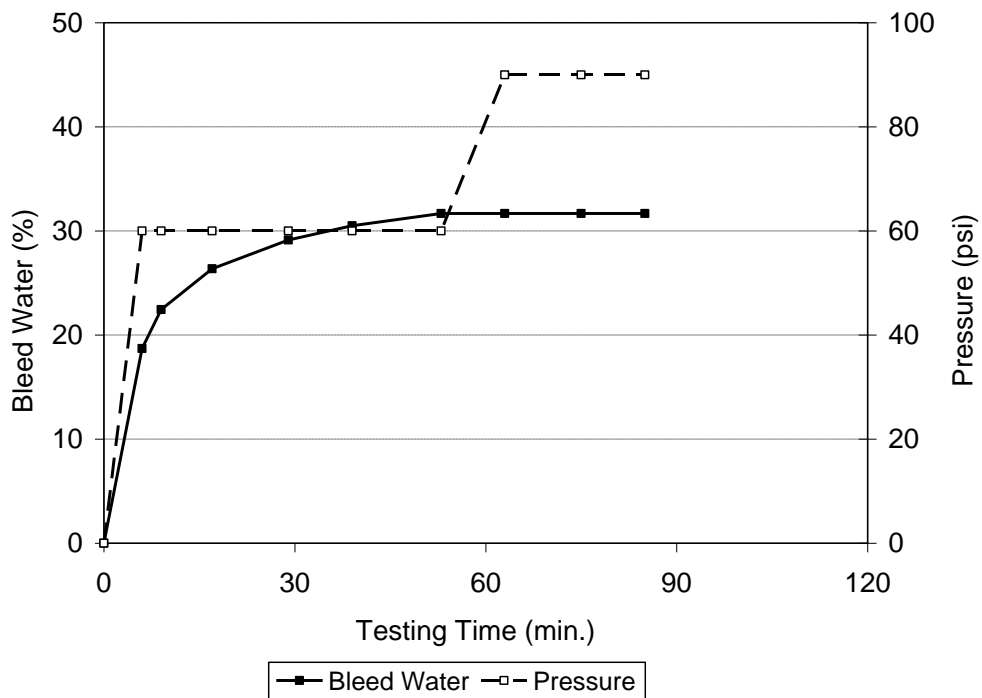
Figure 4.26: Results from the pressurized bleed from Shaft No. 5 of Pier No. 8
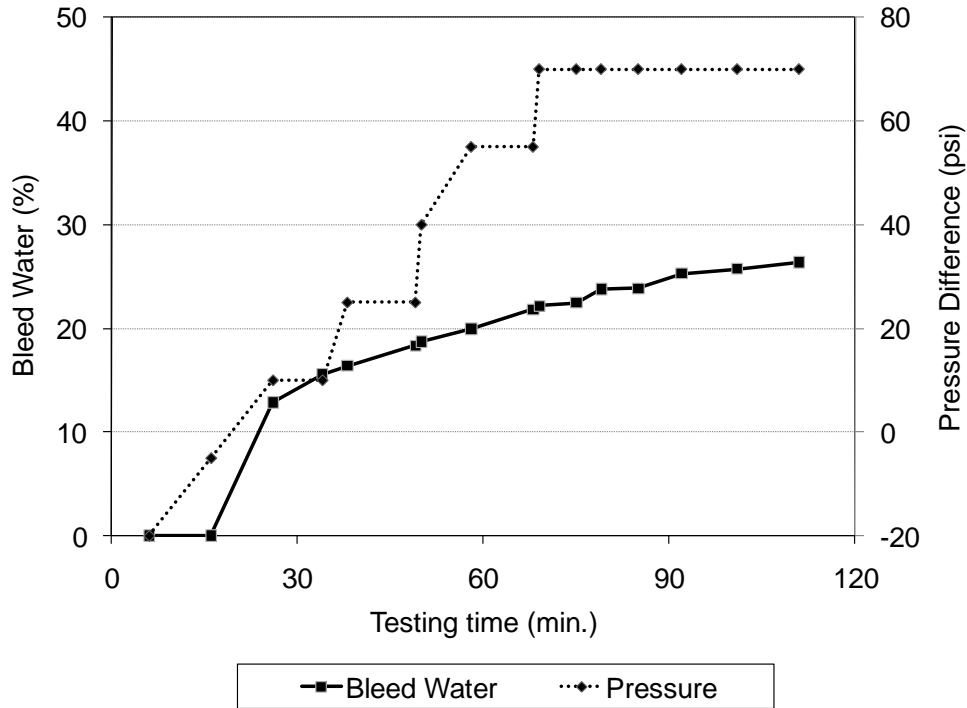
Figure 4.27: Results from the pressurized bleed test with backpressure from Shaft No. 3

of Pier No. 8

4.9.4 Drilled Shaft Temperature

Results from the temperature probes installed onto Shaft No. 3 of Pier No. 8 are presented in Figures 4.28, 4.29, and 4.30. Seven of the nine sensors installed into the shaft recorded temperature data. The probes installed on the outside at 12 ft from the bottom and in the center at 4 ft from the bottom were faulty. A diagram showing the location of the maximum temperature and the location of the faulty sensor is shown in Figure 4.31. The two faulty probes seemed to have been caused by a leak in the coating and were not able to collect temperature measurements.

The maximum temperature recorded was 170 °F, located 12 ft from the bottom of the shaft near the center of the cross section. At this elevation, the probe attached to the inside of the reinforcement cage peaked at a temperature of 146 °F. This elevation was believed to be the center of the shaft. At the elevation 4 ft from the bottom, the

temperature probes were located just a few inches apart and peaked at temperatures of

131 °F and 121 °F for the probes inside and outside the reinforcement cage,

respectively. The elevation that was believed to be three quarters of the way up the

shaft was 17.5 ft. At this elevation, data from all of the probes were acquired. At this

elevation the temperature within the shaft peaked at 118 °F, 132 °F, and 167 °F for the

locations outside the reinforcement, inside the reinforcement and near the center of the

cross section, respectively.
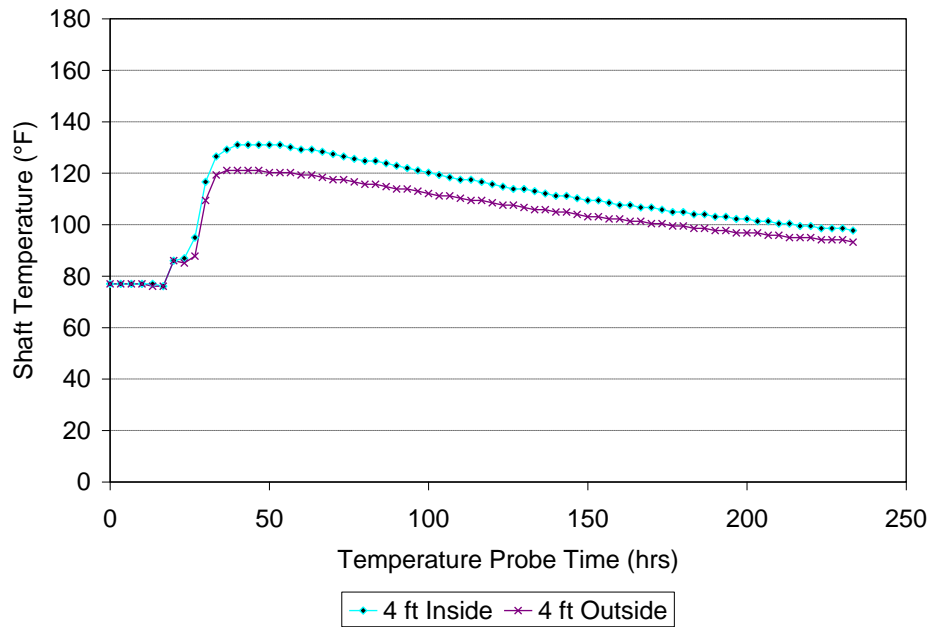
Figure 4.28: Temperature measurements acquired from Shaft No. 3 of Pier No. 7

at an elevation 4 ft above the bottom of the shaft
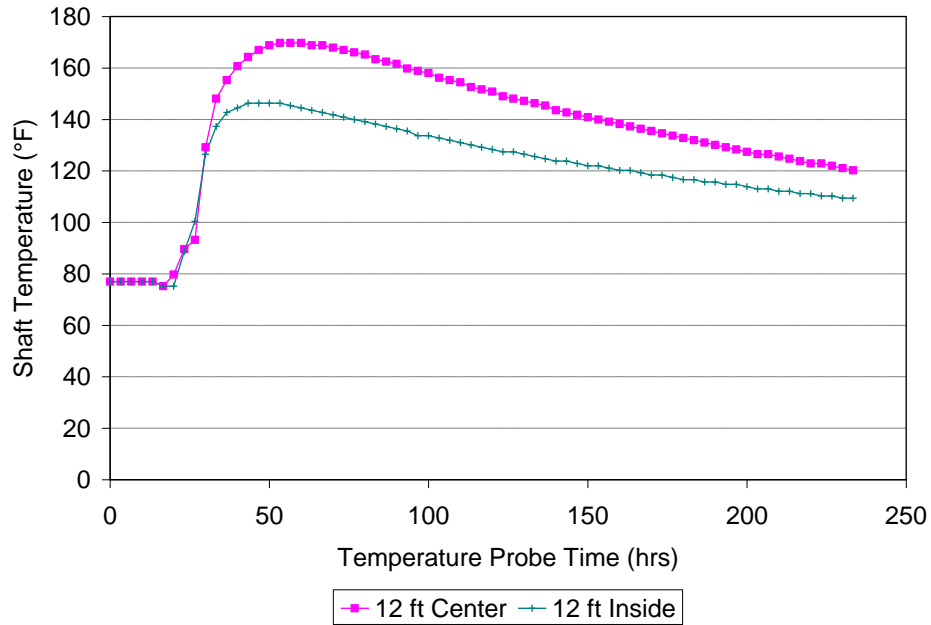
Figure 4.29: Temperature measurements acquired from Shaft No. 3 of Pier No. 7

at an elevation 12 ft above the bottom of the shaft
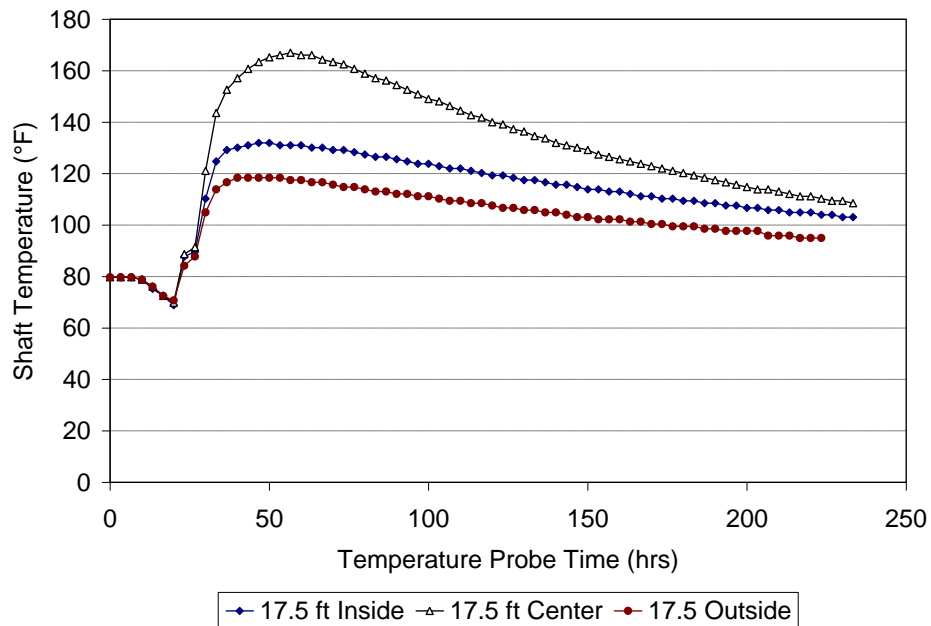
Figure 4.30: Temperature measurements acquired from Shaft No. 3 of Pier No. 7

at an elevation 17.5 ft above the bottom of the shaft
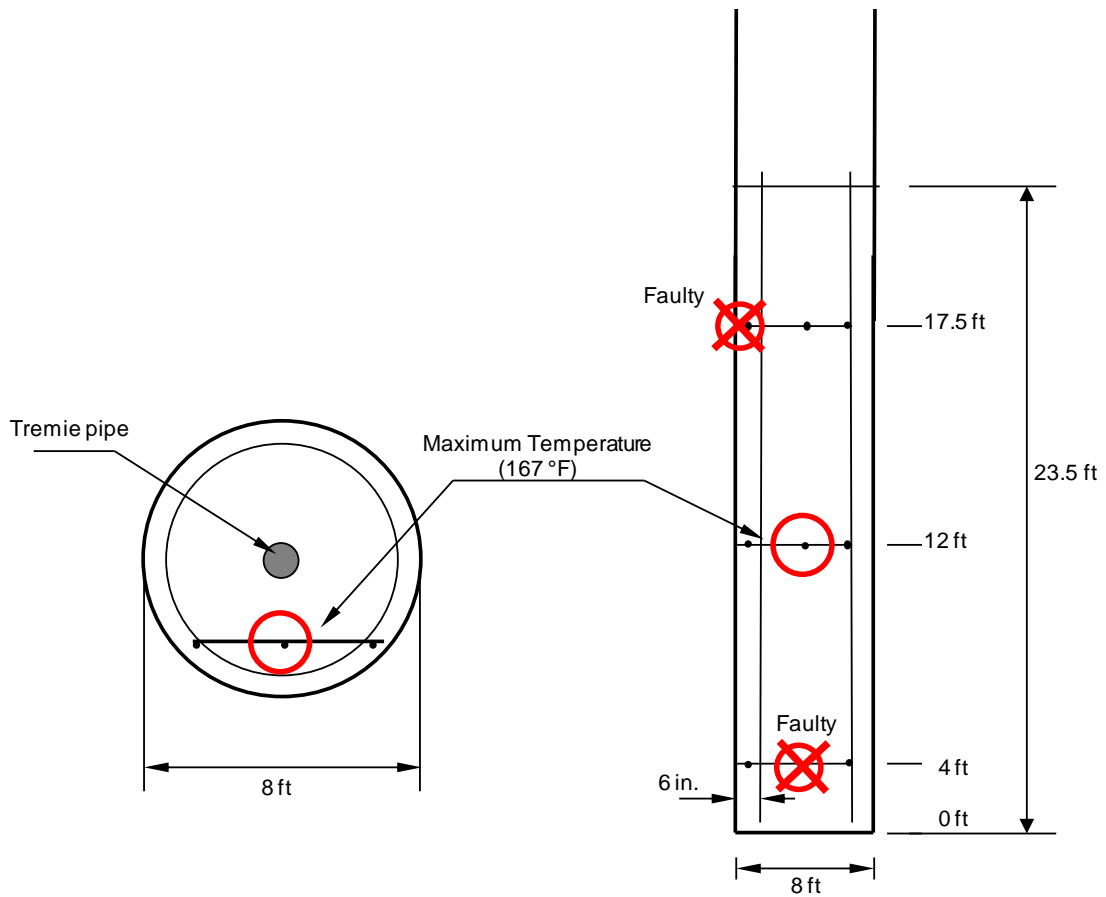
Figure 4.31: Diagram showing location of faulty sensors and the maximum recorded

temperature

4.10 CSL Results

As of June 24, 2010, results were acquired from all of the shafts in Pier No. 7 and

Shaft No. 4 of Pier No. 8.   The results indicated the following:

- Pier No. 7

  o Shaft No. 4, as previously discussed in Section 4.8.1, had significant

    anomalies detected that required further investigation.

  o Shaft No. 5, tested 17 days after placement, had debonding in the top

    7.5 ft of most tube pairs except 4-5, 1-3, and 2-4, where debonding

    was visible in only the top 2.5 ft; however, the rest of the shaft had an

    acceptable result of less than 10% velocity loss through the shaft.

191

Debonding at the top occurs often due to the effect of debonding and this shaft was deemed to be acceptable

- o Shaft No. 1 had most of the shaft resulting in less than 10% velocity loss, but this shaft had one anomaly, referred to as a "pipe joint", between tube pair 7-8.

- o Shaft No. 3 had less than 10% velocity loss throughout the shaft.

- o Shaft No. 2, testing occurred later than 94-days after placement, had less than 10% velocity loss throughout most of the shaft, but had variations between tube pairs 2-3, 7-8, and 8-1; this shaft also had debonding at the top 4.5 ft of the shaft.

- ▪ Pier No. 8

  - o Shaft No. 4 had less than 10% velocity loss throughout the shaft.

4.11 Discussion

The findings from the test results are discussed in this section.

4.11.1 Discussion of Slump Flow of Concrete

Slump flow tests were performed to test the variability of the concrete arriving at the jobsite. The variability in the concrete, distinguished by the calculated standard deviation, did not seem to be an issue. However, the average slump flow for each day was at or below the lower limit of the project specification.

The tendency for the concrete producer to produce the SCC closer to the lower limit of the slump flow range may have been due to the problem experienced during the placement of the first shaft, i.e. Shaft No. 4 of Pier No. 7. It should be noted that it is typical practice for SCC specification for precast applications to allow a ± 2.0 in. range around the target slump flow (PCI 2003). In this project specification, a range of ± 3.0 in. around the target slump flow of 21 in. was specified to provide the concrete producer some extra room to allow for the additional variations inherent to the ready mix concrete

industry as compared to the precast concrete industry. Note that the measured ranges

listed in Tables 4.2, 4.3, and 4.4, were 7.0 in., 6.0 in., and 6.5 in., respectively. Note that

these values are close to the 6.0 in. tolerance specified. It is recommended that

producers of ready mix SCC sample their aggregate moisture states more often in order

to keep the slump flow values within a range of ± 3.0 in. around the target slump flow. It

is clear from these results that it would be problematic for producers of ready mix SCC to

meet a specification that only allows a ± 2.0 in. range around the target slump flow.

The use of a slump flow that is less than the lower limit specified in the project is

not recommended. Producing and using this concrete may have the following results:

- o Based on the research conducted on the test shafts, concrete with a
  slump of between 6 in. and 9 in. may still have issues consolidating in the
  cover region of the reinforcement cage.
- o SCC is more expensive to make per cubic yard due to the addition of the
  chemical admixtures. Therefore, it is not economical to use this more
  expensive concrete if concrete with a workability similar to conventional
  drilled shaft concrete is expected.

4.11.2 Discussion of the Concrete Flow through the Reinforcement

The flow of the concrete varied widely between the two shafts where the

difference between the inner and outer concrete depths were measured. However, it is

uncertain that a maximum elevation difference of 22-in. in an 8-ft. diameter drilled shaft

is significant.

However, the last three truck loads placed into the shaft had slump flow values

below or at the minimum allowed slump flow. These results correspond well with the

highest elevation difference measured. Therefore, concrete placed at or below the

minimum specified slump flow may exhibit higher elevation differences. The variability of

the slump flow and VSI was not conducted on Shaft No. 1 of Pier No. 7, and therefore

could not be verified. It is recommended that more data from field studies be collected to further evaluate the significance of various height differences between the inside and outside of the reinforcing cage.

4.11.3 Discussion of Bleed Tests

The results of the conventional bleed test and pressurized bleed test show that concrete that does not show any potential to bleed at atmospheric pressure may still bleed under pressure, such as concrete in a drilled shaft. A modification to the bleed test apparatus primarily consisted of applying a constant back pressure to the bleed water collection cylinder. This modification proved to work well as there was a gradual release in the amount of bleed water as the pressure increased as shown in Figures 4.26 and 4.27. However, the author only performed two tests with this new configuration. Therefore, future research is recommended to evaluate this bleed test apparatus under controlled-laboratory conditions with various concrete mixtures.

4.11.4 Discussion of Drilled Shaft Temperature

It is known that temperatures greater than 178 °F can reduce the long-term durability of concrete with greater than 25% Class F fly ash (Brown and Schindler 2007). The maximum temperature acquired near the center of Shaft No. 3 of Pier No. 8 was slightly below this value. Since this measurement was not taken at the exact center, and the heat in the center would be expected to be higher, high heat of hydration may be a cause for concern with this mixture design. However, in this shaft, the high temperature would only affect a small volume of concrete near the center. This concern increases if this mixture design is selected on shafts of larger diameters where higher temperatures would be expected.

This high heat of hydration is a common problem in mass concrete placements and in drilled shafts. This is not cause for concern specific to SCC, but concern for large diameter drilled shafts in general (Mullins et al. 2009).

4.12 Future Drilled Shaft Integrity Research

To assess the quality of the available drilled shaft integrity tests, pending ALDOT's approval, further integrity testing will be conducted on selected production shafts. These tests will be conducted in addition to the CSL test and will include the following:

- Crosshole tomography – uses the same principles as the CSL test, but moves the probes to different elevations to create a three-dimensional picture of an anomaly.

- Gamma-gamma – uses a radioactive source and a Geiger counter lowered down each of the CSL tubes. The amounts of photons sent and received through the concrete correspond to the density and indirectly the quality of the concrete.

To conduct this study, three production shafts will be selected for additional testing. These shafts will have the standard CSL test performed on the shaft as specified in the project specification. After this, another CSL test will be conducted for research purposes. If any anomalies are discovered in these CSL tests, crosshole tomography will be performed to examine the anomalies more closely. Finally, gamma-gamma testing will be performed on the same selected drilled shafts. The purpose of this study will be to determine if there is a need for further research to evaluate if any testing in addition to the standard CSL integrity testing should be considered.

4.13 Conclusions

- Every project should have a test shaft included into the budget to check the concrete mixtures suitability with the method and procedure of the shaft installation.

- From the training day, it was concluded that field technicians have a tendency to assign a high VSI due to concrete that flows at a high speed out of the slump cone,

- Field technicians require guidance to ensure all tests required by the specification are conducted in the field,

- The slump flow of the SCC arriving to the jobsite does not vary significantly; however, field tests may need to be conducted more frequent than once every 50 yd^3 to make sure the concrete placed meets the project specifications,

- Concrete that shows no potential to bleed at atmospheric pressure, may still produce bleed water when placed into a drilled shaft, where it will experience significant pressures,

- SCC placed with slump flow values below the minimum slump flow value may contribute to high elevation differences between the inside and outside of the reinforcement cage, lending itself to entrapped laitance or voids (Brown and Schindler 2007), and

- Based on the CSL results, SCC seems to produce in-place shafts that meet the integrity requirements of the ALDOT.

Chapter 5

Summary, Conclusions, and Recommendations

5.1   Research Summary

The objective of this research was to determine the effectiveness of self-consolidating concrete (SCC) in drilled shafts placed under full-scale production conditions. The research took place in Scottsboro, Alabama, where three test shafts were created and tested. A different drilled shaft concrete mixture was used in each shaft:

- One with a ordinary drilled shaft concrete (ODSC) mixture used in on an ALDOT project in North Alabama,

- One with SCC specifically proportioned for drilled shaft applications (SCC), and

- One with an experimental SCC that uses limestone powder (SCC-LP).

The concrete for these shafts was placed using a pump truck, and an elevation measurement was taken inside and outside the reinforcement cage to assess the concrete's ability to flow through the reinforcement cage. Fresh concrete properties such as slump, total air content, unit weight, bleeding, and temperature were determined for the ODSC mixture. Fresh concrete properties such as slump flow, total air content, unit weight, bleeding, temperature, and segregation were determined for both the SCC mixtures. These shafts were exhumed 30 days after the concrete was placed into the shafts. After exhuming, the shafts were cut to visually examine the quality of the

hardened concrete. Cores were taken as well to assess the hardened properties of the in-place concrete, both inside and outside of the reinforcement cage.

The results from the test shafts convinced the ALDOT representatives to decide that the drilled shafts of the next phase of the AL-35 Southbound Bridge project near Scottsboro, Alabama be constructed with SCC. During the construction of the production shafts, tests were conducted to assess the following:

- The variability of the concrete arriving to the jobsite,

- The concrete's ability to flow through the reinforcement cage, and

- The concrete's ability to bleed under pressure.

An additional part of the test shaft project assessed how the concrete flows out of the tremie pipe into a drilled shaft. To conduct this study, thousands of colored mortar cubes were added to the concrete at different intervals. Once hardened and removed, the shafts were cut in half, cleaned, shellacked, and surveyed to show the final location of the mortar cubes. A grid was drawn onto the cut surfaces and the location of each visible cube on the cut surface was located and logged. These survey results, along with results from previous research projects (Holley et al. 2005, Gerwick and Holland 1986), were used to create a hypothesis of how concrete flows from a tremie pipe within the drilled shafts.

In addition to both the test shafts and the production shafts, a pressurized bleed test was developed to assess the concrete's ability to lose water (or bleed) under pressure. This test uses a device designed by Auburn University. Tests were conducted during this research to determine the best process and methods for performing the test. A modification to the bleed test apparatus was conducted part-way through this study. This modification primarily consisted of applying a constant back pressure to the bleed water collection cylinder. This modification proved to work well as

there was a gradual release in the amount of bleed water as the pressure increased as shown in Figures 4.26 and 4.27.

5.2 Conclusions

The results of this study support the following conclusions:

5.2.1 Test Shaft Conclusions

- The SCC and SCC-LP shafts showed significantly better consolidation in the cover region of the drilled shafts than in the ODSC shaft.

- The surface of the SCC flowed upwards in a horizontal manner unobstructed by the reinforcement cage during the entire placement, as shown in Figure 3.23.

- The addition of limestone powder lowers the bleed potential of the concrete.

- Drilled shafts, with congested reinforcement cages, constructed with ordinary drilled shaft concrete (slump 7 to 9 in.) have a significantly lower quality concrete in the cover region as compared to shafts constructed with SCC type mixtures (slump flow 18 to 24 in.).

- The ODSC appeared to have the worst overall condition. Large amounts of sand were built up on the outside of the shaft, large voids were observed in the bottom corners of the shaft exposing the reinforcement cage. The compressive strengths of the ODSC cores had the largest difference between the inner and outer locations indicating poor quality concrete was present in the cover region.

- The SCC shaft was in the best condition of the three. The cores taken from this shaft had the most consistent compressive strength and

modulus of elasticity values throughout the cross sections, the visual

inspection of the cut concrete showed the fewest imperfections, and its

CSL results revealed the fewest amount of minor defects in this shaft.

5.2.2 Drilled Shaft Concrete Flow Conclusions

- Concrete forced out of the bottom of a pump line does not flow in a

  perfectly laminar fashion, because the first concrete that covers the

  tremie tip is not displaced to the top of the shaft.

- The lower the viscosity of the concrete, the more mixing that occurs

  between different layers of concrete in the drilled shaft.

- Higher viscosity concrete, such as ordinary drilled shaft concrete, flows in

  a layered manner inside the shaft, whereas less viscous concrete, such

  as SCC, flows in a bulging manner with less of the concrete flowing up

  around the pump line and instead displacing the previous layers up the

  shaft, as shown in Figures 3.70 and 3.71.

5.2.3 Production Shaft Findings

- Problems were experienced during the first two production drilled shafts.

  Due to these problems, every project should have a test shaft included

  into the budget to check the concrete mixture's suitability with the

  concrete placement methods and shaft installation procedures.

- Concretes with high powder contents and low water-to-cementitious

  materials ratios, like SCC, have a tendency to form cement balls in the

  mixture; this phenomenon can be avoided by changing the sequence of

  the batching process.

- Numerous concrete loads did not meet the specified slump flow requirements. Concrete arriving to the jobsite needs to be tested more often to make sure the concrete placed meets the project specifications.

- SCC placed with slump flow values below the minimum slump flow value may contribute to high elevation differences between the inside and outside of the reinforcement cage, which may lead to entrapped laitance or voids on the sides of the shaft (Brown and Schindler 2007).

- SCC is a viable product for use in drilled shafts to help ensure the quality of the cover region of the shafts.

- Properly designed and installed SCC can produce high-quality drilled shafts with little to no integrity defects.

5.2.4 Pressurized Bleed Test Conclusions

- Concrete that shows no potential to bleed at atmospheric pressure may still produce bleed water when placed into a drilled shaft, where it will experience significant pressures.

5.3 Recommendations

Additional research is needed to further evaluate the use SCC in drilled shafts. The following are recommendations and suggestions based on the author's experiences and research:

- More research needs to be conducted to create a SCC mixture that bleeds less that conventional drilled shaft concrete.

- Future research is recommended to evaluate the bleed test apparatus developed for this project under controlled-laboratory conditions with various concrete mixtures.

- Since the spacing between the modified J-Ring's bars were increased, it is uncertain how Table 3.7 applies to drilled shaft applications. More research is required to develop a blocking assessment for the modified J-Ring for drilled shaft applications.

- Computer modeling needs to be conducted on non-Newtonian fluids to further assess how concrete moves during tremie placement.

- Research needs to be conducted to determine a less subjective fresh concrete test than the Visual Stability Index (VSI) to rapidly assess the stability of the concrete in the field.

- More research needs to be conducted to assess the acceptable limits of the column segregation test, as determined by ACI 237 (2007).

- The addition of limestone powder lowers the concrete's ability to bleed. However, this research determined that concretes made with this product had variable fresh concrete properties when leaving the batch plant. Since the addition of limestone powder may also be beneficial to lower heat of hydration in large shafts, more research needs to be conducted on the addition of limestone powder to concrete mixtures for full-scale production.

References

AASHTO T 277. 2008. Standard Method of Test for Electrical Indication of Concrete's
Ability to Resist Chloride Ion Penetration. *Standard Specifications for
Transportation Materials and Methods of Sampling and Testing*, 28th Edition.

ACI Committee 237. 2007. Self-Consolidating Concrete. *American Concrete Institute,*
Farmington Hills, Michigan.

ACI Committee 336. 2001. Specification for the Construction of Drilled Piers. *American
Concrete Institute*, Farmington Hills, Michigan.

ADSC/DFI. 1989. *Drilled Shaft Inspector's Manual*. Drilled Shaft Committee of the
ADSC: The International Association of Foundation Drilling and DFI: The Deep
Foundations Institute, Dallas, Texas.

ASTM C 39. 2005. Standard Test Method for Compressive Strength of Cylindrical
Concrete Specimens. *Annual Book of ASTM Standards* 2005. Section 04, V.
04.02: Concrete and Aggregates.

ASTM C 138. 2005. Standard Test Method for Density, Yield, and Air Content of
Concrete. *Annual Book of ASTM Standards* 2005. Section 04, V. 04.02: Concrete
and Aggregates.

ASTM C 143. 2005. Standard Test Method for Slump of Hydraulic-Cement Concrete.
Annual Book of ASTM Standards 2005. Section 04, V. 04.02: Concrete and
Aggregates.

ASTM C 157. 2005. Standard Test Method for Length Change of Hardened-Cement
Mortar and Concrete. *Annual Book of ASTM Standards* 2005. Section 04, V.
04.02: Concrete and Aggregates.

ASTM C 192. 2005. Standard Test Method for Making and Curing Concrete Test
Specimens in the Laboratory. *Annual Book of ASTM Standards* 2005. Section
04, V. 04.02: Concrete and Aggregates.

ASTM C 403. 2005. Standard Test Method for Time of Setting of Concrete Mixtures by
Penetration Resistance. *Annual Book of ASTM Standards* 2005. Section 04, V.
04.02: Concrete and Aggregates.

ASTM C 469. 2005. Standard Test Method for Static Modulus of Elasticity and Poisson's
Ratio of Concrete in Compression. *Annual Book of ASTM Standards* 2005.
Section 04, V. 04.02: Concrete and Aggregates.

ASTM C 1202. 2005. Standard Test Method for Electrical Indication of Concrete's
Ability to Resist Chloride Ion Penetration. *Annual Book of ASTM Standards* 2005.
Section 04, V. 04.02: Concrete and Aggregates.

ASTM C 1610. 2005. Standard Test Method for Static Segregation of Self-Consolidating
Concrete Using Column Technique. *Annual Book of ASTM Standards* 2005.
Section 04, V. 04.02: Concrete and Aggregates.

ASTM C 1611. 2005. Standard Test Method for Slump Flow of Self-Consolidating
Concrete. *Annual Book of ASTM Standards* 2005. Section 04, V. 04.02: Concrete
and Aggregates.

ASTM C 1621. 2005. Standard Test Method for Passing Ability of Self-
ConsolidatingConcrete by J-Ring. *Annual Book of ASTM Standards* 2005.
Section 04, V. 04.02: Concrete and Aggregates

Ahamd, I. and S. Azhar. 2004. Temperature variation in high slump drilled shaft concrete and its effect on slump loss. *Cement and Concrete Research.* Vol. 34 No. 2. pp 207-217.

Bailey, J.D. 2005. An Evaluation of the Use of Self-Consolidating Concrete (SCC) for Drilled Shaft Application. M.S. Thesis, Auburn University.

Ball, D.J., M.T. Hutchinson, S.A. Jefferis, P.G. Shotten, L. Stamsfoeld and A. J. Wills. 2006. *Bentonite Support Fluids in Civil Engineering.* Federation of Piling Specialists. 2nd Edition. http://www.fps.org.uk/fps/guidance/bentonite.pdf (accessed Feb. 22)

Bonen, D. and S.P. Shah. 2005. Fresh and hardened properties of self-consolidating concrete. *Progress in Structural Engineering Materials: Concrete Construction.* Vol. 7. Is. 1. pp 14-26.

Brown, D. A. 2004. Recipe for Success with Drilled Shaft Concrete. *Foundation Drilling Magazine.* Nov. pp 16-24.

Brown, D.A. 2004. *Zen and the Art of Drilled Shaft Construction: The Pursuit of Quality* Invited keynote lecture and paper, Geotechnical Special Publication No. 124, ASCE, pp. 19-33. Geo-Institute International Conference on Drilled Foundations, Orlando.

Brown, D.A. and A.K. Schindler. 2005. "The Use of Self-Consolidating Concrete for Drilled Shaft Construction: Preliminary Observations from the Lumber River Bridge Field Trials", Proc. of ADSC GEO3 QA/QC Conference, Dallas, TX.

Brown, D.A. and A.K. Schindler, 2007. High Performance Concrete and Drilled Shaft Construction. *Geotechnical Special Publication No. 158.* Geo-institute GeoDenver 2007 Conference, http://scitation.aip.org/getpdf/servlet/GetPDFServ-

let?filetype=pdf&id=ASCECP000221040902000031000001&idtype=cvips&pro-g=normal. (accessed Jan. 25, 2010).

Brown, D.A., A.K. Schindler, J. Bailey, A. Goldberg, W. Camp III, and D. Holley. 2007. Evaluation of Self-Consolidating Concrete for Drilled Shaft Applications at Lumber River Bridge Project, South Carolina. *Transportation Research Record: Journal of the Transportation Research Board.* No. 2020: pp 67-75.

Brown, D.A. 2010. High Performance Concrete for Drilled Shaft Foundations. *Alabama Highway Conference: February 22-23, 2010.* Presentation.

Brown, D.A. 2010. Personal Communication

Bury, M.A. and B.J. Christensen. 2003. The Role of Innovative Chemical Admixtures in Producing Self-Consolidating Concrete. *First Northat American Conference on the Design and Use of Self-Consolidating Concrete 12-13 November 2002,* edited by Surendra Shah, Joseph Daczko, and James Lingdcheit. Addison, Illinois: Hanley-Wood, pp 137-140.

California Department of Transportation. *Gamma-Gamma Logging (GGL).* http://www.dot.ca.gov/hq/esc/geotech/ft/gamma.htm (accessed January 20, 2010).

Dachelet, D.O. 2008. The Effectiveness of Self-Consolidating Concrete (SCC) for Drilled Shaft Construction. M.S. Thesis, Auburn University

Domone, P. 1997. The Slump Flow Test for High-Workability Concrete. *Cement and Concrete Research.* Vol. 28. No. 2. pp 177-182.

Gerwick Jr., B. and T. Holland. 1986. *Concrete placement under water: How does it flow and where does it go?* The Aberdeen Group, Publication No. C860943, ftp://imgs.ebuild.com/woc/C860943.pdf (accessed May 15, 2009)

Goodier, C.I. 2003. Development of self-compacting concrete. *Proceedings of the Institution of Civil Engineers: Structures and Buildings.* Vol. 156. Is. SB4.

EFNARC. 2006. *Guidelines for Viscosity Modifying Admixtures For Concrete*. 2006

    European Federation for Specialist Construction Chemicals and Concrete

    Sytems. ENC 179VMA r12. September. http://www.efnarc.org/pdf/Sand

    GforSCC.PDF. (Accessed Jan. 25, 2010)

Google Earth. 2010. http://maps.google.com/ (accessed May 18, 2010).

Hodgson III, D., A.K. Schindler, D.A. Brown, and M. Stroup-Gardiner. 2005. Self

    Consolidating Concrete for Use in Drilled Shaft Applications. *Journal*

    of Materials in Civil Engineering. Vol. 17, No. 3, pp 363-369.

Holley, Daniel W., William M. Camp III, and Aaron D. Goldberg. 2005. Project Summary

    Report: *Self Consolidating Concrete (SCC) in Drilled Shafts Research Project*.

    Prepared for Kimley-Horn and Associates, Inc. Raleigh, North Carolina. S&ME,

    Inc. Mt. Pleasant, South Carolina.

Khayat, K.H. 1999. Workability, Testing, and Performance of Self-Consolidating

    Concrete. *ACI Materials Journal.* Vol. 96. No. 3 pp 346-353.

Khayat, K.H. and A. Yahia. 1997. Effect of Welan Gum-High-Range Water Reducer

    Combinations on Rheology of Cement Grout.. *ACI Materials Journal.* Vol. 94 No.

    5. pp 365-372.

Khayat, K.H., O. Bonneau, and F. Tchieme. 2006. Precast, Prestressed SCC made with

    Betocarb 3. *Sherbrooke University Report No. 3*. Quebec, Canada. (April).

Khayat, K.H. M. Saric-Coric and F. Liotta. 2002. Influence of Thixotropy on Stability

    Characteristics of Cement Grout and Concrete*. ACI Materials Journal.* V. 99. No.

    3. May-June 2002. pp 234-24.

McCarthy, D.F. 2002. *Essentials of Soil Mechanics and Foundations: Basic*

    Geotechnics. 6th edition. Prentice Hall. Upper Saddle River, New Jersey.

McCracken, S.C. 2002. *High-Performance Concrete Today: Nothing Routine*.

>Publication No. C02F029. Copyright 2002 Hanley-Wood, LLC.

>ftp://imgs.ebuild.com/woc/C02F029.pdf (accessed July 29, 2009).

Mindess, S.J., F. Young, and D. Darwin. 2003. *Concrete.* Second Edition. Prentice Hall.

>Upper Saddle River, NJ.

Mullins, G. and S. Kranc. 2007. *Thermal Integrity Testing of Drilled Shafts – Final*

>*Report*. University of South Florida. Prepared for Florida Department of

>Transportation. FDOT Grant No. BD544-20. http://www.dot.state.fl.us/research-

>center/Completed_Proj/Summary_SMO/FDOT_BD544_20_rpt.pdf (Accessed

>May 15, 2010)

Mullins, G., D. Winters, and K. Johnson. 2009. *Attenuating Mass Concrete Effects in*

>*Drilled Shafts*. Florida Department of Transportation. Report No. BD-544-39.

Nassif, H., H. Najm and K. Aktas. 2008. *Self-Consolidting Concrete (Phase I & II).*

>Federal Highway Administration. Project No. NJ 2007-021.

Olsen, L.D. and M.F. Aoud and D.A. Sack. 1998. *NDT Diagnosis of Drilled Shaft*

>*Foundations*. Transportation Research Record: Journal of the Transportation

>Research Board. Vol 1663. pp 120-127.

O'Neil, M.W. and L.C. Reese. 1999. *Drilled Shafts: Construction Procedures and Design*

>*Methods,* FHWA Report No. FHWA-IF-99_025. Dallas, Texas: ADSC: The

>International Association of Foundation Drilling.

O'Neil, M.W. 1991. *Construction Practices and Defects in Drilled Shafts*. Transporation

>Research Record 1331. pp 6-14.

Precast/Prestressed Concrete Institute (PCI). 2003. Interim Guidelines for the Use of Self-Consolidating Concrete in Precast/Prestressed Concrete Institute Member Plants. 1st Edition. Precast/Prestressed Concrete Institute, Chicago, Illinois.

Rausche, F. ,H. See, Robinson, B., M. Kurtz, and E. Attiogbe. 2005. *Quality Assurance for Drilled Shafts Using Self-Consolidating Concrete.* GEO3 Construction Quality Assurance/Quality Control Technical Conference: Dallas/Ft. Worth, TX; pp 425-436.

Robertson, D. and J. Bailey. 2008. *Report of Crosshole Sonic Logging, B.B. Comer Bridge.* AFT Project No.208102. September 15, 2008.

Saric-Coric, M. K.H. Khayat and A. Tagnitt-Hamou. 2003. *Performace characteristics of cement grouts made with various combinations of high-range water reducer and cellulose-based viscosity modifier.* Cement and Concrete Research 33. pp 1999-2008.

Schindler, A.K. 2010. Personal Communication

See, H. T. and M. A. Kurtz, Joseph A. Daczko, Emmanuel K. Attibogbe, Brent Robinson, Frank Rausche, and Eric Mack. 2005. *Evaluation of SCC for Drilled Shaft Applications*, Report No. TS-ES-044. Cleveland, Ohio.

Taylor, H.F.W., C. Famy, and K.L. Scrivener. 2001. Delayed ettringite formation, *Cement and Concrete Research*, Vol. 31, pp. 683-693.

Turner, J.P. 1992. Constructability for Drilled Shafts. *Journal of Construction Engineering and Management.* Vol. 118. No. 1, March.

Western, Kevin L., Alan R. Phipps and Christopher J. Burgess. 2009. The New Minneapolis I-35W Bridge: Designed and Built in Only 11 months. *STRUCTURE magazine.* April. pp 32-34.

Yankee Publishing. 2009. *The Old Farmers Almanac.* www.almanac.com. Accessed Jan. 15, 2009.

Yoa, Sam X. and Robert B. Bittner. 2007. "Underwater Concrete in Drilled Shafts: the key Issues and Case Histories". Geotechnical Special Publication No. 158. Proceedings Of Sessions Of Geo-Denver, February 18-21, 2007. Denver, Colorado.

Irvin, D. and P. Dinterman. (2009) Geologic Map of the Hollywood 7.5-Minute Quadrangle. Jackson Co. Alabama. http://www.ogb.state.al.us/documents/pubs/onlinepubs/geo_maps/QS52/QS52_Plate.pdf (accessed: June 18, 2010)

Appendix A: Test Shaft Specifications

ALABAMA DEPARTMENT OF TRANSPORTATION

STANDARD SPECIFICATIONS

2006

CONTRACT FORMS

PROJECT NO(s). BRF-0035(502)

SPECIFICATIONS, PROPOSAL, CONTRACT AND BOND FOR CONSTRUCTION OF:

0.277 MILE BRIDGE REPLACEMENT (PARTIAL, PHASE 1)

KNOWN AS FEDERAL AID PROJECT NO(s) BRF-0035(502)

LOCATED: ON SR-35 AT THE TENNESSEE RIVER IN SCOTTSBORO

IN: JACKSON COUNTY

TYPE: BRIDGE REPLACEMENT (PARTIAL, PHASE 1)

I HEREBY CERTIFY THAT THIS IS A TRUE AND CORRECT COPY OF THE ORIGINAL CONTRACT NOW ON FILE IN THE OFFICE OF THE ALABAMA DEPARTMENT OF TRANSPORTATION IN MONTGOMERY, ALABAMA

ALABAMA DEPARTMENT OF TRANSPORTATION

BY: _____

OFFICE ENGINEER

ALABAMA DEPARTMENT OF TRANSPORTATION

DATE: February 9, 2007

Special Provision No. <u>06-0420</u>

SUBJECT: Drilled Shaft Construction, Project Number IBRCP-0035(502),
Jackson County .

Alabama Standard Specifications, 2006 Edition, shall be amended by the replacing SECTION 506 with the following:

SECTION 506
DRILLED SHAFT CONSTRUCTION

506.01 Description.

This work shall consist of all labor, materials, equipment and services necessary to perform all operations to complete a drilled shaft installation in accordance with these Specifications and the details and dimensions shown on the plans.

This work shall also consist of the excavation, exhumation, backfilling of the excavation and handling of designated drilled shafts for research purposes.

506.02 Materials.

(a) GENERAL.

All materials shall conform to requirements set forth in Division 800, Materials. The requirements provided for Structural Portland Cement Concrete, Section 501, shall apply in all respects to drilled shafts, except where otherwise indicated by specific requirements given hereinafter in this Section or noted by plan details.

Portland cement concrete used in construction of drilled shafts shall designated as either "Class DS1", "Class DS2", "Class DS3" or "Class HPDS" concrete. The specific class of concrete that is required will be shown in the Pay Item Description for Drilled Shaft Construction.

(b) CONCRETE (DS1, DS2 AND DS3).

For Class DS1, DS2, and DS3 concrete, the concrete producer shall establish the proportion of materials for each class of drilled shaft concrete following the guidelines described in ALDOT-170, *"Method of Controlling Concrete Operations for Structural Portland Cement Concrete"*, except that, instead of the reference to the Master Proportion Table, the concrete producer shall use the criteria outlined hereinafter in this Subarticle. The concrete supplier shall submit for approval the proposed concrete mix design to the State Materials and Test Engineer following the requirements in ALDOT-170. The distribution of the approved concrete mix design and re-approval of concrete mix designs will be as per ALDOT-170 respectively. Any changes of the materials and/or proportions of the mix design will require a concrete mix resubmittal.

1. Criteria applicable to Class DS1, Class DS2 and Class DS3 concrete:

Minimum Compressive Strength at 28 days shall be 4000 psi {30 MPa}.

The amount of cementitious material shall be a minimum of 600 pounds {360 kg} and a maximum of 800 pounds per cubic yard {475 kg per cubic meter} of concrete.

An air-entraining admixture is required in the concrete mix; the range of total air content shall be 2.5 % to 6.0 % by volume.

The maximum water to total cementitious material ratio shall be 0.40.

Slump requirements:

The allowable range of consistency slump during concrete placement shall be from 6 inches to 9 inches {150 mm to 230 mm}.

The minimum consistency slump for all of the concrete placed in an individual shaft shall be no less than 4 inches {100 mm} at the end of the concrete placement in that shaft.

The temperature of the concrete, at the time of placement in the shaft, shall not be less than 50 °F {10 °C} nor more than 95 °F {35 °C}.

Gradation of the coarse aggregate used shall meet the requirements for either ALDOT Size No. 57, No. 67 or No. 7.

All materials used in manufacturing the concrete shall conform to the requirements of the Specifications.

2. Additional criteria applicable to Class DS1 concrete:

Either Type I or Type II cement shall be used.

The cementitious content may be composed of up to 30% by weight {mass} substitution of either Class C or Class F fly ash additive. In lieu of fly ash, ground granulated blast furnace slag may be substituted for cement up to a minimum substitution rate of 25% and a maximum substitution rate of 50% by weight {mass}.

3. Additional criteria applicable to Class DS2 concrete:

Type II cement shall be used.

The cementitious content shall be composed of no less than 20% nor more than 30% by weight {mass} of Class F fly ash additive. In lieu of fly ash, ground granulated blast furnace slag may be substituted for cement up to a minimum substitution rate of 35% and a maximum substitution rate of 50% by weight {mass}.

4. Additional criteria applicable to Class DS3 concrete:

Type II cement shall be used.

The cementitious content shall be composed of 20% by weight {mass} of Class F fly ash and 10% by weight {mass} of microsilica additives. In lieu of the percentages of fly ash and microsilica, the cementitious content may be composed of 50% by weight {mass} substitution of ground granulated blast furnace slag and 5% by weight {mass} addition of microsilica additives.

(c) CONCRETE (HPDS).

The concrete supplier for the HDPS concrete shall have an ALDOT certified concrete laboratory, and shall have an established quality assurance plan.

The required physical properties and mix proportion of the HPDS concrete are given in the following table. A proposed mix design will not be required for the HPDS concrete.

-------------- HPDS CONCRETE --------------		
-- Physical Properties --		
Property	Value	Test
Minimum Compressive Strength	Minimum 4000 psi at 28 days	Section 501
Slump	Minimum 4 Inches two hours after completion of concrete placement	Section 501
Slump Flow	21 inches +/- 3 inches at concrete placement	ASTM C 1611
Visual Stability Index	1.5 or less during concrete placement	ASTM C 1611
-- Approximate Concrete Mixture Proportion --		
Component	Type or Source	Amount
Portland Cement	Type I Cement (National Cement Company)	494 pcy
Fly Ash	Class F Fly ash (SEFA Group)	210 pcy
Fine Aggregate	Natural Sand from Madison Materials	1,469 pcy (SSD)
Coarse Aggregate	Crushed Limestone from Vulcan, Scottsboro	1,480 pcy (SSD)
Water Content	Section 807	280 pcy
Additive	Recover	6 to 8 oz/cwt (Note 1)
Additive	ADVA CAST 555	6 to 8 oz/cwt (Note 2)
Additive	V-MAR 3	2 oz/cwt
Additive	Daravair 1000	0.5 oz/cwt (Note 3)

Note 1: This dosage should be determined to provide a 4 inch slump, two hours after the completion of concrete placement in the shaft.
Note 2: This dosage should be determined to meet the slump flow requirements at the time of placement.
Note 3: This dosage should be determined to produce a total air content of 2.5 to 6 %.
Note 4: Water reducing admixtures may only be added at the concrete batch plant.

(d) SLURRY.

When use of slurry is either shown to be required in the contract documents or selected by the Contractor, mineral slurries shall be used unless another type of slurry is proposed for use by the Contractor and approved by the Engineer. The following minimum requirements apply to material components used in slurries:

1. APPROVED MINERALS.

Sodium Bentonite or Attapulgite shall be used as the principal mineral constituents of slurry. The Engineer may approve use of other minerals upon receipt of demonstrated proof that the requested alternate mineral insures shaft stability at the applicable shaft construction site.

2. MIXING WATER.

Mixing water shall be capable of meeting drinking water standards as outlined in Section 807.

3. SAND.

Clean, locally available sand meeting the requirements of Section 802 (not to exceed four (4) percent by volume) may be mixed in drilling slurries.

4. ADDITIVES.

At the Contractors discretion, additives may be used to control the consistency and/or yield of slurries subject to the limitation that the type and amount of additives used shall not exceed the recommendation(s) of the principal mineral manufacturer.

(e) CASING.

When use of casing is either specified by the contract documents or selected by the Contractor, casings shall be smooth, non-corrugated, clean, watertight steel of ample strength to withstand both handling and driving stresses and the pressures of concrete and the surrounding earth materials. Where permanent casing is required, serviceable used casing may be installed with the approval of the Engineer.

The Contractor is responsible for insuring that all casing, new or used, is capable of withstanding the aforementioned stress and pressure requirements.

(f) STEEL REINFORCEMENT.

Unless otherwise noted on the contract documents, all steel reinforcement shall be Grade 60 {420} billet steel meeting the requirements of Section 502, sized and installed in accordance with the contract plans as applicable. Welding of the reinforcing steel will not be permitted without the written approval of the Bridge Engineer. Welding to the main vertical reinforcing steel will not be permitted.

506.03 Construction Methods and Equipment.

The Contractor shall perform excavations required for shafts through whatever materials are encountered, to the dimensions and elevations shown in the plans or otherwise required by the specifications and special provisions. The Contractor's methods and equipment shall be suitable for the site conditions and materials encountered. The permanent casing method shall be used only at locations shown on the plans or authorized by the Bridge Engineer.

Actual cores recovered from the test borings are available for inspection at the Bureau of Materials and Tests.

(a) GENERAL REQUIREMENTS.

1. CONTRACTOR QUALIFICATIONS.

The Contractor shall submit descriptions of the drilled shaft construction projects completed in the last three years to serve as evidence of the capability to construct drilled shafts. The descriptions of the drilled shaft projects shall contain names and telephone numbers of owners' representatives who can verify the Contractor's participation on those projects. These descriptions shall be submitted with the Installation Plan and will be evaluated by the Engineer.

The evaluation of the Contractor's capability for constructing drilled shafts will have a bearing on the decision by the Engineer to require the construction of a Trial Drilled Shaft.

2. INSTALLATION PLAN.

a. Installation Plan Requirements.

No later than 30 days after the date of the Notice to Proceed, the Contractor shall submit an installation plan for review by the Engineer. This plan shall provide information on the following items as applicable:

- Name and experience record of the drilled shaft superintendent in charge of drilled shaft operations for this project;

- List of proposed equipment to be used including cranes, drills, augers, bailing buckets, final cleaning equipment, desanding equipment, slurry pumps, core sampling equipment, tremies, concrete pumps, casing, etc.;

- Details of the overall anticipated construction operation sequence and the proposed sequence of shaft construction;

- Details of planned shaft excavation methods;

- Details of the methods to be used to insure shaft stability (i.e. prevention of caving, bottom heave, etc., using temporary casing, slurry or other means) during excavation and concrete placement. This shall include a review of method suitability to the anticipated site and

subsurface conditions. If casings are proposed or required, casing dimensions and detailed procedures for permanent casing installation, and temporary casing installation and removal shall be provided.

      - When use of slurry is required or proposed, details of the methods for mixing, circulating and desanding slurry;

      - Details of methods to clean the shaft excavation;

      - Details of reinforcement placement including support and centralization methods;

      - Details of concrete placement method required or proposed including operational procedures for free fall, tremie or pumping as appropriate; and

      - The method used to fill or eliminate all voids between the plan shaft diameter and excavated shaft diameter, or between the shaft casing and surrounding soil, if permanent casing is specified.

      - Details of the material, equipment, and procedures proposed to accomplish the required load testing.

      - It shall be noted in the procedure for the construction of a shaft that will be exhumed for research purposes that the concrete shall be allowed to cure for least 16 hours before any disturbance is allowed near the shaft. After the concrete in each drilled shaft (that will be exhumed) has been placed, adjacent shafts shall not be excavated, adjacent piles shall not be driven, and equipment wheel loads and vibration from equipment shall not be allowed to occur at any point within a 25 foot radius of the drilled shaft.

    b. Evaluation of Installation Plan.

      The Engineer will evaluate the drilled shaft installation plan for conformance with the plans and specifications. Within 15 days following receipt of the installation plan, the ALDOT Construction Engineer will return the plan for corrections, distribute the plan for construction inspection, or contact the Contractor to establish a mutually agreeable date and time for a meeting to discuss the installation plan. If a meeting is held to discuss the installation plan the Contractor and his drilled shaft project superintendent shall be in attendance. The Contractor will be notified of changes in the submitted installation plan deemed necessary by the ALDOT Construction Engineer within seven days after the aforementioned meeting. Shaft construction shall not begin until the installation plan has been distributed by the ALDOT Construction Engineer for construction inspection. Distribution of the installation plan for construction inspection shall not relieve the Contractor of the responsibility to satisfactorily complete the work as detailed on the plans and in the specifications.

    c. Modification of Installation Plan.

      Any proposed modification of the installation plan during construction shall be submitted to the Construction Engineer for review and distribution.

    3. EXHUMATION PLAN FOR THE REMOVAL OF SHAFTS FOR RESEARCH PURPOSES. — *Not Received*

      The Contractor shall submit a drilled shaft exhumation plan to the Engineer for review. This submittal shall be included as a part of the drilled shaft installation plan.

      All information that is necessary for the exhumation, backfilling of the excavation, treatment and final disposition of the drilled shafts that are required to be exhumed shall be included in the exhumation procedure. All proposed modifications of the exhumation plan during construction shall be submitted to the Engineer for review and approval.

    4. PROTECTION OF EXISTING STRUCTURES AND UTILITIES.

      The Contractor shall control his operations to prevent damage to existing structures and utilities as outlined in Article 107.12. Preventive measures shall include, but are not limited to, selecting construction methods and procedures that will prevent caving of the shaft excavation, monitoring and controlling the vibrations from construction activities such as the driving of casing or sheeting, drilling of the shaft, or from blasting, if permitted.

    5. CONSTRUCTION SEQUENCE.

    a. Excavation to the bottom of shaft elevation shall be completed before shaft construction begins unless otherwise noted in the contract documents or approved by the Engineer. Any disturbance caused by shaft installation to a planned drilled shaft area shall be repaired by the Contractor prior to the shaft construction.

b. When drilled shafts are to be installed in conjunction with embankment placement, the Contractor shall construct drilled shafts after the placement of fills unless shown otherwise in the contract documents or approved by the Engineer.

c. Substructure concrete shall not be placed on a drilled shaft until the concrete in the shaft reaches a minimum of 80% of the required 28-day compressive strength and until all CSL test results (when required) are accepted and the CSL tubes have been dewatered and grouted.

(b) METHODS OF CONSTRUCTION.

1. DRY METHOD.

The dry construction method shall be used only at sites where the groundwater level and soil conditions are suitable to permit construction of the shaft in a relatively dry excavation, and where the sides and bottom of the shaft may be visually inspected by the Engineer prior to placing reinforcement and concrete. The dry method consists of drilling the shaft excavation, removing accumulated water and loose material from the excavation, placing the reinforcing cage, and concreting the shaft in less than 3 inches of water.

2. WET METHOD.

The wet construction method may be used at sites where a dry excavation can not be maintained for placement of the shaft concrete. This method consists of using water or mineral slurry to maintain stability of the hole perimeter while advancing the excavation to final depth, placing the reinforcing cage, and concreting the shaft. Where drilled shafts are located in open water areas, exterior casings shall be extended from above the water elevation into the ground to protect the shaft concrete from water action during placement and curing of the concrete. The casing shall be installed in a manner that will produce a positive seal at the bottom of the casing so that no seepage of water or other materials occurs into or from the shaft excavation.

3. CASING FOR DRY OR WET CONSTRUCTION METHODS.

Permanent or temporary casing may be used when shown on the plans or at sites where the dry or wet construction methods are inadequate to prevent caving or excessive deformation of the hole. In this method the casing may be either placed in a predrilled hole or advanced through the ground by twisting, driving or vibration before being cleaned out. Casing which is going to be installed by predrilling and permanently left in rock for the purpose of shielding voids, shall be installed in not more than a 2 inch {50 mm} oversized drill hole. When downsizing of permanent casing is required, no more than six feet of overlap of casing will be allowed.

When the casing method is required but not shown on the plans, the Contractor shall submit details of the proposed casing method (including casing lengths and diameters) and the proposed procedures of casing installation to the Bridge Engineer for review in the installation plan. If the need is determined after work on the shafts has begun, a revised plan proposing this method must be submitted for review.

(c) EXCAVATION PROCEDURES.

1. EXCAVATION LOCATION, COORDINATION AND TIME CONSTRAINTS.

Shaft excavations shall be made at locations and to the elevations, geometry and dimensions shown in the contract documents or as directed by the Engineer.

A shaft shall not be excavated as long as an adjacent shaft in the same substructure unit is open unless authorized in writing by the Construction Engineer. Blasting and vibrating casings in place will not be allowed until the concrete in adjacent shafts has reached 80 % of the required 28-day compressive strength.

Once the excavation of a shaft has been started, the excavation shall be conducted in a continuous operation until the excavation is completed.

When an excavation is performed with any type of drilling fluid (i.e. slurry, water, etc.) used to stabilize the excavation, the placement of concrete shall begin within 36 hours from the start of excavation and within 12 hours from the start of the excavation of the bottom 5 feet {1.5 m} of the shaft. If the Contractor exceeds these time limits, additional work may be required to insure that the condition of the excavation is adequate to result in an acceptable load carrying capacity in the completed drilled shaft. The Contractor may be required to over ream the entire depth of excavation

(or the bottom 5 feet {1.5 m} if the 12 hour time limit is exceeded), increase the depth of the excavation or perform other work that may be required by the Engineer to provide an acceptable excavation. There will be no compensation for this additional work.

The minimum width of over reaming shall be 1/2 inch {13 mm} and the maximum width shall be 3 inched {75 mm}.

2. EXCAVATION LOG.

The Contractor shall maintain an excavation log during shaft excavation. The log shall contain information such as: the description and approximate top and bottom elevation of each soil or rock material encountered during shaft excavation, elevations at which seepage or groundwater flow are encountered, and remarks. The type of tools used for the excavation shall be shown on the log. All changes in the type of tools used for excavation shall be shown on the log. The Engineer will monitor these operations and the logs will be used as a basis of measurement for payment. The Contractor shall resolve all discrepancies on the log noted by the Engineer at the end of each work day. Two copies of the legible, final log shall be furnished to the Engineer within 24 hours after a shaft excavation is completed and accepted.

3. HANDLING EXCAVATED MATERIAL.

Excavated materials which are removed from shaft excavations shall be disposed of by the Contractor in accordance with Subarticle 215.03(g).

4. EXCAVATION SAFETY.

The Contractor shall not permit workers to enter the shaft excavation for any reason unless: suitable casing has been installed, the water level has been lowered and stabilized below the level to be occupied, and adequate safety equipment and procedures have been provided to protect workers entering the excavation. The Contractor is responsible for compliance with applicable State and Federal safety regulations.

(d) TYPES OF DRILLED SHAFT EXCAVATION.

1. DRILLED SHAFT EXCAVATION.

The excavation of the shaft using conventional earth drilled shaft excavation tools will be designated as "drilled shaft excavation".

2. SPECIAL DRILLED SHAFT EXCAVATION.

The excavation of the shaft requiring rock tools and/or procedures to accomplish hole advancement will be designated as "special drilled shaft excavation". This excavation will be for the removal of rock or other hard material within the planned shaft.

(e) EXCAVATING AND DRILLING EQUIPMENT.

1. GENERAL.

Excavation and drilling equipment shall have adequate capacity including power, torque and down thrust to excavate a hole of both the maximum specified diameter and to a depth of twenty (20) percent beyond the depths shown on the plans when operated at rated capacity.

2. ROCK TOOLS AND EQUIPMENT.

When the material encountered cannot be drilled using conventional earth drilling tools and equipment, the Contractor shall provide rock drilling equipment including air tools, approved blasting materials, and other equipment as necessary to construct the shaft excavation to the size and depth required. Concurrence of the Engineer shall be obtained prior to switching from earth to rock drilling tools and equipment. Approval of the Engineer is required before excavation by blasting is permitted.

3. OVERREAMING.

a. Sidewall overreaming shall be required when the sidewall of the hole is determined to have either softened due to excavation methods, swollen due to delays in concreting, or degraded because of slurry cake buildup. Overreaming thickness shall be a minimum of 1/2 inch {13 mm} and a maximum of 3 inches {75 mm}.

b. Overreaming may be accomplished with a grooving tool, overreaming bucket or other approved equipment. The thickness and extent of sidewall overreaming shall be as directed by the

Engineer. The Contractor shall bear all costs associated with both sidewall overreaming and additional shaft concrete placement.

 4. LOST TOOLS.

 Drilling tools which are lost in the excavation shall not be considered obstructions and shall be promptly removed by the Contractor without compensation. All costs due to lost tool removal shall be borne by the Contractor including costs associated with correcting hole degradation due to removal operations and time delays.

(f) EXPLORATORY SHAFT EXCAVATION.

 1. GENERAL.

 The Contractor will be required to perform some type of exploratory shaft excavation (soil samples, rock cores or drilling or probing) below the bottom elevations shown on the plans unless this requirement is noted on the plans as being deleted. The Contractor shall extend drilled shaft tip elevations when the Engineer determines that the material encountered during this exploratory excavation is unsuitable and/or differs from that anticipated in the design of the drilled shaft.

 2. ROCK CORES AND SOIL SAMPLES.

 The Contractor shall take 2.0 inch {51 mm} minimum diameter rock cores and/or soil samples at locations as designated on the plans or as directed by the Engineer to determine the character of the material directly below the completed shaft excavation. The soil samples shall be extracted with a split spoon sampler or undisturbed sample tube in accordance with AASHTO T 206 and T 207. The methods and equipment used for the rock coring shall be those given in Subarticle 506.10(b) for the core drilling of drilled shaft concrete. The cores and/or soil samples shall be taken to a minimum of 10 feet {3 m} below the bottom of the drilled shaft excavation unless otherwise noted on the plans or directed by the Geotechnical Engineer. The Engineer may require this depth to be extended up to a total depth of 20 feet {6 m} below the bottom of the shaft. The Contractor may choose to take these cores and/or soil samples prior to excavating for the drilled shafts, however, payment will only be considered for that portion of the cores taken below the bottom elevation of the shafts shown on the plans.

 Rock core and soil test samples shall be measured, visually identified and described on the Contractor's log. The samples shall be placed in suitable containers, identified by shaft location, elevation and project number and delivered to the Central Laboratory in Montgomery with the Contractor's field log within 24-hours after the exploration is completed. The Engineer will inspect the samples/cores and determine the final depth of required excavation based on his evaluation of the sampled materials suitability.

 3. DRILLING OR PROBING.

 At all drilled shaft locations where rock cores and/or soil samples are not designated, the Contractor will be required to drill or probe an exploratory hole below the bottom elevation of the shaft to determine if any voids or crevices are present. The exploratory hole shall be taken to a depth of 10 feet {3 m}, unless noted otherwise on the plans. Exploratory drilling or probing will not be required if it is noted on the plans that this requirement is not necessary. No direct payment will be made for this operation.

(g) OBSTRUCTION REMOVAL.

 Surface and subsurface obstructions at drilled shaft locations shall be removed by the Contractor. Such obstructions may include man-made materials such as old concrete foundations and natural materials such as boulders. Special procedures and/or tools shall be employed by the Contractor in the event the hole cannot be advanced using conventional augers fitted with soil or rock teeth, drilling buckets and/or underreaming tools. Special procedures/tools may include but are not limited to: chisels, boulder breakers, core barrels, air tools, hand excavation, temporary casing, and increasing the hole diameter. Blasting shall not be permitted unless specifically approved in writing by the Engineer. Removal of obstructions will be classified as "special drilled shaft excavation".

(h) TRIAL DRILLED SHAFT INSTALLATION.

1. GENERAL.

The Engineer will require the construction of a trial shaft if the submittal of descriptions of previous drilled shaft construction projects does not, in the opinion of the Engineer, substantiate the Contractor's capability for constructing the drilled shafts on this project. The Engineer may also require the construction of a trial shaft to verify the adequacy of unusual construction methods and/or equipment proposed for use in the construction of the production shafts.

The trial drilled shaft shall be constructed if required by special note on the plans.

2. LOCATION AND DEPTH.

The trial shaft(s) shall be positioned as indicated on the plans or as directed by the Engineer. Unless otherwise indicated, shafts shall be drilled to the maximum depth of any production shaft shown on the plans.

3. FAILURE TO DEMONSTRATE ABILITY.

Failure of the Contractor to demonstrate the adequacy of his equipment, methods and/or expertise shall be reason for the Engineer to require alterations necessary to eliminate unsatisfactory results. Additional trial shafts required to demonstrate correction of deficiencies shall be at the Contractor's expense.

4. TRIAL SHAFT APPROVAL.

Once approval has been given to construct production shafts, no changes will be permitted in the personnel, methods or equipment that were used to construct the satisfactory trial shaft without written approval of the Engineer.

5. SITE RESTORATION.

Unless otherwise shown in the contract documents, the trial shaft holes will be filled with non-reinforced concrete in the same manner that production shafts will be constructed. The concreted trial shafts shall be cutoff 2 feet {600 mm} below finished grade or at the mudline if in water. The disturbed areas at trial shaft holes shall be restored as nearly as practical to their original condition. No direct payment will be made for cutting off the top of the trial shaft or for the site restoration.

506.04 Encased Excavations.

(a) GENERAL.

The outside diameter of casings shall not be less than the specified shaft size. No extra compensation will be allowed for concrete required to fill an oversized casing or excavation. All casings, except permanent casing, shall be removed from shaft excavations.

(b) TEMPORARY CASING.

1. GENERAL.

All casing shall be considered temporary unless specifically shown as permanent casing in the contract documents. The Contractor will be required to remove temporary casing before completion of concreting the drilled shaft. Telescoping, predrilling with slurry, and/or overreaming to beyond the outside diameter of the casing may be required to install casing.

2. SIZE SUBSTITUTION.

If the Contractor elects to remove a specified diameter or length of casing and substitute a longer or larger diameter casing through caving soils, the excavation shall be either stabilized with slurry or backfilled before the new casing is installed. Other methods, as approved by the Engineer, may be used to control the stability of the excavation and protect the integrity of the foundation soils.

3. BOUND OR FOULED CASINGS.

Temporary casings which become bound or fouled during shaft construction and cannot be practically removed shall constitute a defect in the drilled shaft. The Contractor shall be responsible for correcting such defective shafts to the satisfaction of the Engineer. Correction may consist of, but is not limited to: removing the shaft concrete and extending the shaft deeper to compensate for loss of frictional capacity in the cased zone, providing straddle shafts to compensate for capacity loss, or

providing a replacement shaft. All corrective measures including redesign of shafts caused by defective shafts shall be done to the satisfaction of the Engineer without compensation or an extension of the completion date of the project. In addition, no compensation will be paid for casing remaining in place.

4. REMOVABLE CASING.

When the shaft extends above ground or through a body of water, the portion exposed above ground or through a body of water may be formed with suitable, removable casing except when permanent casing is specified. Removable casing shall be stripped from the shaft in a manner that will not damage the concrete. Casings can be removed when the concrete has attained a compressive strength of not less than 2500 psi {20 MPa} as determined from concrete cylinder breaks provided: curing of the concrete is continued for the full period in accordance with specifications and the shaft concrete is not exposed to salt water or moving water for seven days.

(c) PERMANENT CASINGS.

1. GENERAL.

Permanent casing shall be used when shown in the contract documents. The casing shall be continuous between top and bottom elevations prescribed in the plans. After installation is complete, the permanent casing shall be cut off at the prescribed elevation and the shaft completed by installing necessary reinforcing steel and concrete in the casing.

Exterior surfaces of permanent casings shall be cleaned and coated with the prime coat only of a System 1A Coating in accordance with the requirements given in Section 521 and as shown on the plans. The exterior surfaces shall be coated prior to the installation of the casings. After the installation of the casings, all damage to the coated surfaces of the casings exposed to the air shall be repaired by a repeated application of the same prime coat. When not shown in the contract documents, permanent casing may be used if determined to be necessary by the Engineer and if approved by the Bridge Engineer.

2. MULTIPLE CASINGS.

In cases where special temporary casings are shown on the plans or authorized in writing by the Engineer, the Contractor shall maintain alignment of both the temporary outer and permanent inner casing, and a positive, watertight seal between the two casings during excavation and concreting operations.

506.05 Use of Slurry.

(a) GENERAL.

Slurries shall have a mineral grain size that will remain in suspension and sufficient viscosity and gel characteristics to transport excavated material to a suitable screening system. The percentage and specific gravity of the material used to make the suspension shall be sufficient to maintain stability of the excavation and allow proper concrete placement.

(b) MIXING AND STORAGE.

The mineral slurry shall be premixed thoroughly with clean fresh water and adequate time allotted for hydration prior to introduction into the shaft excavation. Slurry tanks of adequate capacity will be required for slurry circulation, storage, and treatment. Excavated slurry pits will not be allowed in lieu of slurry tanks without the written permission of the Engineer.

(c) DESANDING.

Desanding equipment shall be provided by the Contractor as necessary to control slurry sand content at less than 4 percent by volume at any point in the borehole. Desanding will not be required for setting temporary casing, sign post, or lighting mast foundations unless required by the plans or special provisions.

(d) REQUIRED FLUID LEVEL.

1. GENERAL.

During construction, the level of the slurry shall be maintained at a height sufficient to prevent caving of the hole. In the event of a sudden significant loss of slurry in the hole, the

construction of that foundation shall be stopped until methods to stop slurry loss or an alternate construction procedure have been approved by the Engineer.

2. REQUIRED HEAD.

Mineral slurry in a shaft excavation shall be maintained at a level not less than 4 feet {1.2 m} above the highest expected static water surface along the depth of the shaft. If at any time the Engineer determines the slurry construction method fails to produce the desired final results, the Contractor shall discontinue this method and propose an alternate method for approval of the Engineer.

(e) CONTROL OF SLURRY.

1. SETUP PREVENTION.

The Contractor shall take all steps necessary to prevent the slurry from "setting up" in the shaft. Such methods may include but are not limited to: agitation, circulation and/or adjusting the properties of the slurry.

2. CONTROL TESTING.

Control tests using suitable apparatus shall be carried out on the mineral slurry by the Contractor to determine density, viscosity and pH. An acceptable range of values for these physical properties is shown in the following table:

MINERAL SLURRY (Sodium Bentonite or Attapulgite in Fresh Water) Acceptable Range of Values			
Property (Units)	At Time of Slurry Introduction	In Hole at Time of Concreting	Test Method
Density (pounds per cubic foot) {kg/m^3}	64.3** - 69.1** {1030** - 1110**}	64.3** - 75.0** {1030** - 1200**}	Density Balance
Viscosity (seconds / quart) {seconds / liter}	28 - 45 {30 - 48}	28 - 45 {30 - 48}	Marsh Cone
pH	8-11	8-11	pH paper, pH meter
** Increase by 2 pounds per cubic foot {32 kg/m^3} in salt water			
a. Tests should be performed when the slurry temperature is above 39 °F.			
b. If desanding is required, sand content shall not exceed 4 percent (by volume) at any point in the bore hole as determined by the American Petroleum Institute sand content test.			

(f) TESTING OF SLURRY.

1. FREQUENCY.

Tests to determine density, viscosity and pH value shall be done during the shaft excavation to establish a consistent working pattern. A minimum of four sets of tests shall be made during the first 8 hours of slurry use. When the results show consistent behavior the testing frequency may be decreased to one set every four hours of slurry use.

2. TEST REPORTS.

Reports of all tests required above, signed by an authorized representative of the Contractor, shall be furnished to the Engineer on completion of each drilled shaft.

(g) DISPOSAL.

Disposal of all slurry shall be done off site by the Contractor.

506.06 Excavation Measurement and Cleaning.

(a) GENERAL.

The Contractor shall provide equipment and personnel for checking the dimensions and alignment of each permanent shaft excavation. The dimensions, depth and alignment shall be determined under the direction and to the satisfaction of the Engineer after final cleaning.

(b) CLEANING.

Unless otherwise stated in the contract, a minimum of 50 percent of the base of each shaft will have less than 1/2 inch {13 mm} of sediment at the time of concrete placement. The maximum depth of sediment or any debris at any place on the base of the shaft shall not exceed 1.5 inches {40 mm}. Shaft cleanliness will be determined by visual inspection for dry shafts. For wet shafts the bottom of the shaft shall be sounded with an airlift pipe, a tape with a heavy weight {mass} attached to the end of the tape or other means acceptable to the Engineer. In addition, for dry excavations the maximum depth of water covering the bottom of the excavation shall not exceed 3 inches {75 mm} prior to concrete pour.

506.07 Reinforcing Steel Construction and Placement.

(a) GENERAL.

The reinforcing steel cage, consisting of longitudinal and transverse bars, ties, cage stiffeners, spacers, centralizers, and other necessary appurtenances, shall be completely assembled and placed as a unit immediately after the shaft excavation is inspected and accepted, and prior to concrete placement. The reinforcing steel in the shaft shall be securely tied and supported so that the reinforcing steel will remain within allowable tolerances given in Subarticle 506.11(c) of this Specification.

(b) SPACERS.

1.   Concrete spacers or other approved noncorrosive spacing devices shall be used at sufficient intervals near the bottom, and at intervals not exceeding 10 feet up the shaft, to insure concentric spacing for the entire cage length.

2.   Spacers shall be constructed of approved material equal in quality and durability to the concrete specified for the shaft. The spacers shall be of adequate dimension to insure the proper annular space between the outside of the reinforcing cage and the side of the excavated hole and/or permanent casing as detailed on the plans or proposed in the installation plan. If not detailed on the plans, a minimum 4 inch {100 mm} annular space will be required.

(c) CAGE SUPPORTS.

Cylindrical concrete feet (bottom supports) shall be provided to insure that the bottom of the cage is maintained at the proper distance above the base as specified by the project plans.

(d) CAGE EXTENSION.

If the drilled shaft excavation is extended to an elevation lower than the plan bottom elevation, reinforcing cage length shall also be extended by the same amount. Cages may be extended at the plan bottom elevation by lap splicing additional longitudinal bars, per planned cage requirements, of sufficient length to provide a compression splice, 4.17 feet {1270 mm} in length, plus the required extension. Hoops for the extension shall be spaced the same as shown for other hoops. Any additional splices of the cage above the plan bottom elevation and not shown on the plans, must have prior approval of the Bridge Engineer. Stiffeners, spacers and other appurtenances shall also be extended as required.

506.08 Concrete Placement Requirements.

(a) GENERAL.

Concrete used for drilled shaft construction shall meet the requirements of Subarticle 506.02(b).

After the reinforcing steel has been placed and before the concrete is ordered, the bottom of the drilled shaft must be resounded to verify cleanliness.

(b) CONCRETE PLACEMENT TIME LIMITATIONS.

1. GENERAL.

Concrete shall be placed as soon as possible after the reinforcing steel has been placed and the bottom of the shaft has been resounded. The concrete placement shall be continuous from the bottom to the top elevation of the shaft.

The elapsed time from the beginning of concrete placement in the shaft to the completion of placement shall not exceed 2 hours except as allowed by the Engineer. The Engineer may allow the concrete placement time to exceed 2 hours if the Contractor adequately demonstrates that the slump of the concrete will not be less than 4 inches {100 mm} during the entire time of concrete placement.

2. SLUMP LOSS/TIME RELATIONSHIP.

a. General.

The Contractor may choose either a laboratory test or a field test to demonstrate the slump loss/time relationship. Adjustments to chemical admixture dosages will be allowed for the sole purpose of extending the time of concrete placement provided that the admixtures are included in the approved concrete mix design. A new slump loss test will be required if changes are made to the concrete mix, including adjustments to chemical admixtures.

b. Laboratory Test.

The Contractor shall demonstrate by trial mix and slump loss tests that the slump of the concrete will not be less than 4 inches {100 mm} during the longer placement time. These tests shall be conducted by an independent testing laboratory, approved by the Department as per ALDOT-405, and in the presence of a Department representative. The slump loss tests shall be performed at intervals not to exceed 30 minutes and shall be made from a trial mix proportioned from the approved concrete mix design. The temperature of the trial mix shall be kept at a level representative of construction site conditions.

c. Field Test.

The Contractor shall demonstrate by construction site slump loss tests that the slump of the concrete will not be less than 4 inches {100 mm} during the longer placement time. The slump loss tests shall be performed at intervals not to exceed 30 minutes and shall be made from the first batch of concrete that is placed in a trial drilled shaft. The concrete used for these slump loss tests shall be sampled at the trial drilled shaft site and shall be kept covered during testing. If a trial shaft is not required then a field test may be performed at the construction site prior to the beginning of the work. The slump test shall be performed by the contractor's Concrete Technician, certified by the Department as per ALDOT-405, in the presence of a Department representative.

(c) PLACEMENT THROUGH SLURRY AND/OR ENCASED EXCAVATIONS.

1. GENERAL.

The Contractor shall insure that a heavily contaminated slurry suspension, which could impair the free flow of concrete, has not accumulated in the bottom of the shaft.

2. REQUIRED SLURRY SAMPLING.

Prior to placing concrete in a slurry filled shaft excavation, the Contractor shall take slurry samples using a sampling tool. Slurry samples shall be extracted from the base of the shaft and at intervals not exceeding 10 feet {3 m} up the shaft, until two consecutive samples produce acceptable values for density, viscosity, pH, and sand content as noted in Subarticle 506.05(c) and Item 506.05(e)2, respectively.

3. UNACCEPTABLE SAMPLING RESULTS.

When any slurry samples are found to be unacceptable, the Contractor shall take whatever action is necessary to bring the mineral slurry within specification requirements. Concrete shall not be poured until resampling and testing results produce acceptable values.

4. REQUIRED CONCRETE LEVEL DURING PLACEMENT.

The level of fresh concrete placed into a casing shall be a minimum of 5 feet {1.5 m} above either the hydrostatic water level or the level of drilling fluid whichever is higher. As a temporary casing is withdrawn, care shall be exercised to maintain an adequate level of concrete within the casing so that fluid trapped behind the casing is displaced upward and discharged at the ground surface without contaminating or displacing the shaft concrete.

506.09 Concrete Placement Methods.

(a) GENERAL.

If a method of concrete placement has not been specifically identified in the contract documents, the Contractor may use any of the placement methods described hereafter. If a concrete pump is used to move the concrete to the drilled shaft, a standby pump shall be immediately available to pump the concrete if there is a pump failure. Details pertaining to compliance with this specification shall be presented as part of the Contractors "Installation Plan" as outlined in Item 506.03(a)2.

Concrete placement shall continue after the shaft excavation is full until good quality concrete is evident at the top of the shaft. Any overflow of concrete at the top of the shaft shall be removed to maintain a uniform appearance and the proper dimensions of the shaft.

(b) FREE FALL PLACEMENT.

1. GENERAL.

The free fall placement of concrete shall only be permitted in dry vertical shafts where the clear opening (inside the reinforcing cage) is not less than 24 inches {610 mm} in diameter. The height of free fall placement shall not exceed 75 feet {22 m}. Concrete placed by free fall shall fall directly to the placement location without contacting either the reinforcing cage or the shaft sidewall.

The Engineer will observe the falling of the concrete within the shaft. If the concrete strikes the reinforcing cage or sidewall, or if there is excessive spatter from the impact of the falling concrete, the Contractor shall reduce the rate of concrete placement, reduce the height of free fall or provide a drop chute for concrete placement as directed by the Engineer.

2. DROP CHUTE REQUIREMENTS.

a. General.

Drop chutes shall consist of a smooth tube of either one piece construction or sections which can be added and removed. Concrete may be placed through either a hopper at the top of the tube or side openings as the drop chute is retrieved during concrete placement.

b. Chute Support.

The drop chute shall be supported so that the free fall of the concrete measured from the bottom of the chute to the point of deposition is less than 75 feet {22 m}. If concrete placement causes the shaft excavation to cave or slough, or if the concrete strikes the rebar cage or sidewall, the Contractor shall reduce the height of free fall and/or reduce the rate of concrete flow into the excavation.

3. DISQUALIFICATION OF FREE FALL METHOD.

If in the opinion of the Engineer, placement cannot be satisfactorily accomplished by the free fall and drop chute method, the Contractor shall change to either tremie or pumping methods to accomplish the pour.

(c) TREMIE CONCRETE PLACEMENT.

Tremies may be used for concrete placement in either wet or dry holes.

1. TREMIE REQUIREMENTS.

a. General.

Tremies shall consist of a tube of sufficient length, weight {mass}, and diameter to discharge concrete at the shaft base elevation. The tremie shall not contain aluminum parts which will have contact with the concrete. The tremies inside diameter shall be at least 6 times the maximum size of aggregate used in the concrete mix but shall not be less than 10 inches {250 mm}.

b. Tremie Tube Wall.

Inside and outside surfaces of the tremie shall be clean and smooth to permit both flow of concrete and unimpeded withdrawal during concreting. The wall thickness of the tremie shall be adequate to prevent crimping or sharp bends which restrict concrete placement.

c. Concrete Placement.

The tremie used for wet concrete placement shall be watertight. Underwater placement shall not begin until the tremie is placed to the shaft base elevation. Valves, bottom plates or plugs may be used to insure concrete discharge begins within one tremie diameter of the base. Plugs

shall either be removed from the excavation or be made of a material which will not cause a defect in the shaft if not removed. The discharge end of the tremie shall be constructed to permit the free radial flow of concrete during placement operations.

 2. PLACEMENT REQUIREMENTS.
  a. General.
   The tremie discharge end shall be immersed at least 5 feet {1.5 m} in concrete at all times after starting the flow of concrete. The flow of the concrete shall be continuous. The concrete in the tremie shall be maintained at a positive pressure differential at all times to prevent water or slurry intrusion into the shaft concrete.
  b. Defective Shafts.
   If at any time during the concrete pour, the tremie line orifice is removed from the fluid concrete column and discharges concrete above the rising concrete level, the shaft shall be considered defective. In such case, the Contractor shall either:
   - remove the reinforcing cage and concrete, complete any necessary sidewall removal directed by the Engineer, and repour the shaft or,
   - the tremie shall be replugged, recharged with concrete and inserted a minimum of 5 feet {1.5 m} below the existing top level of concrete prior to continuing the pour. The contractor shall be responsible for correcting any defect caused by this procedure without additional compensation.
   All costs for replacement of defective shaft concrete shall be the responsibility of the Contractor.

(d) PUMPED CONCRETE PLACEMENT.

 Concrete pumps and lines may be used for concrete placement in either wet or dry excavations.

 1. EQUIPMENT REQUIREMENTS.
  Pump lines shall have a minimum diameter of 4 inches {100 mm} and shall be constructed with watertight joints. Except as modified herein, requirements pertaining to tremie lines as stated in Item 506.09(c)1, also apply to pump lines and their use. The concrete pump unit shall have sufficient power to insure continuous placement of concrete under all foreseeable placement conditions.

 2. PLACEMENT REQUIREMENTS.
  a. Discharge Orifice Location and Pressure.
   The discharge orifice shall remain at least 5 feet {1.5 m} below the surface of the fluid concrete. When lifting the pump line during concreting, the Contractor shall temporarily reduce the line pressure until the orifice has been repositioned at a higher level in the excavation.
  b. Defective Shafts.
   If at any time during the concrete pour the pump line orifice is removed from the fluid concrete column and discharges concrete above the rising concrete level, the shaft shall be considered defective. In such case, the Contractor shall remove the reinforcing cage and concrete, complete any necessary sidewall removal directed by the Engineer, and repour the shaft. All costs for replacement of defective shaft concrete shall be the responsibility of the Contractor.

506.10 Testing Requirements For Drilled Shafts.

(a) CROSSHOLE SONIC LOGGING OF DRILLED SHAFTS.

 1. GENERAL REQUIREMENTS.
  The nondestructive testing method called Crosshole Sonic Logging (CSL) shall be used on all production and trial drilled shafts (a) when constructed with the placement of concrete underwater or through slurry, (b) when required by special note on the plans, (c) when a full length temporary casing is used to prevent water from entering the shaft, or (d) when determined to be necessary by the Engineer. The testing shall not be conducted until forty-eight hours after the placement of all concrete in a shaft and must be completed within 20 calendar days after placement.
  The CSL tests shall be conducted by an experienced independent testing consultant approved by the Engineer prior to testing.

The CSL tests measure the time it takes for an ultrasonic pulse to travel from a signal source in one access tube to a receiver in another access tube. In uniform, good quality concrete, the travel time between equi-distant tubes will be relatively constant and correspond to a reasonable concrete pulse velocity from the bottom to the top of the foundation. In uniform, good quality concrete, the CSL test will also produce records with good signal amplitude and energy. Longer travel times and lower amplitude/energy signals indicate the presence of irregularities such as poor quality concrete, void, honeycomb and soil intrusions. The signal will be completely lost by the receiver and CSL recording system for the more severe defects such as voids and soil intrusions.

2. PREPARATION FOR TESTING.

A number of tubes shall be installed in each shaft to permit access for CSL. The number of tubes installed will depend on the diameter of the shaft as specified below:

Shaft Diameter D	Minimum Number of Tubes
D ≤ 4.5 feet {1372 mm}	4
4.5 feet {1372 mm} < D ≤ 5.5 feet {1676 mm}	5
5.5 feet {1676 mm} < D ≤ 6.5 feet {1981 mm}	6
6.5 feet {1981 mm} < D ≤ 7.5 feet {2286 mm}	7
7.5 feet {2286 mm} < D ≤ 8.5 feet {2591 mm}	8
8.5 feet {2591 mm} < D ≤ 9.0 feet {2743 mm}	9
9.0 feet {2743 mm} < D ≤ 10.0 feet {3048 mm}	10
10.0 feet {3048 mm} < D ≤ 11.0 feet {3353 mm}	11
11.0 feet {3353 mm} < D ≤ 12.0 feet {3658 mm}	12

The tubes shall have a 1.5 inch {40 mm} inside diameter and shall be schedule 40 steel pipe. The pipes shall have a round, regular internal diameter free of defects or obstructions, including any at pipe joints, in order to permit the free, unobstructed passage of a 1.3 inch {30 mm} diameter source and receiver probes. The tubes shall be watertight and free from corrosion with clean internal and external faces to ensure passage of the probes and a good bond between the concrete and the tubes.

The pipes shall each be fitted with a water tight shoe on the bottom and a removable cap on the top. The pipes shall be securely attached to the interior of the reinforcement cage with a minimum cover of 4 inches {100 mm}. The tubes shall be installed in each shaft in a regular, symmetric pattern such that each tube is equally spaced from the others around the perimeter of the cage. The Contractor shall submit to the testing organization his selection of tube size, along with his proposed method to install the tubes, prior to construction. The tubes shall be as near to parallel as possible. The tubes shall extend from 6 inches {150 mm} above the shaft bottoms to at least 3 feet {1 m} above the shaft tops. If the shaft top is sub-surface, the tubes shall extend at least 2 feet {600 mm} above the ground surface. Any joints required to achieve full length tubes shall be made watertight. Care shall be taken during reinforcement installation operations in the drilled shaft hole so as not to damage the tubes. As the cage is being lowered into the shaft, the tubes shall be checked to assure that they are vertical and parallel and that all connections are water tight. After placement of the reinforcement cage, the tubes shall be filled with clean water as soon as possible. After the tubes are filled with water, the tube tops shall be capped or sealed to keep debris out of the tubes prior to concrete placement.

The pipe caps or plugs shall not be removed until the concrete in the shaft has set. Care shall be exercised in the removal of caps or plugs from the pipes after installation so as not to apply excess torque, hammering, or other stresses which could break the bond between the tubes and the concrete.

3. TYPICAL CSL TEST EQUIPMENT.

Typical CSL test equipment consists of the following components:

- A microprocessor based CSL system for display of individual CSL records, analog-digital conversion and recording of CSL data, analysis of receiver responses and printing of CSL logs.

- Ultrasonic source and receiver probes for 1.5 or 2 inch {40 or 50 mm} ID pipe, as appropriate.

- An ultrasonic voltage pulser to excite the source with a synchronized triggering system to start the recording system.

- A depth measurement device to determine and record depths.

- Appropriate filter/amplification and cable systems for CSL testing.

4. CSL LOGGING PROCEDURES.

Before the placement of concrete, a minimum of one tube per shaft shall be plumbed and the tube length recorded, including a notation of the stickup of the tubes above the shaft tops. Information on the shaft bottom and top elevations and/or length, along with construction dates shall be provided to the Engineer and the approved testing organization before the CSL tests are performed. CSL tests shall be conducted between pairs of tubes. The approved testing organization shall test two principle diagonals through the center and between each tube pair around the perimeter of all tested shafts. Additional logs shall be conducted at no additional cost in the event anomalies are detected.

The CSL tests shall be carried out with the source and receiver probes in the same horizontal plane unless test results indicate potential defects in which case the questionable zone may be further evaluated with angled tests (source and receiver vertically offset in the tubes). CSL measurements shall be made at depth intervals of 0.2 feet {60 mm} or less, and shall be done from the bottom of the tubes to the top of each shaft. The probes shall be pulled simultaneously, starting from the bottoms of the tubes, over a depth measuring device. Any slack shall be removed from the cables prior to pulling to provide for accurate depth measurements of the CSL records. Any defects indicated by longer pulse arrival times and significantly lower amplitude/energy signals shall be reported to the Engineer and further tests shall be conducted as required to evaluate the extent of such defects. Additional NDT methods which may be used to evaluate possible defects include Singlehole Sonic Logging, Gamma-Gamma Nuclear Density Logging, and/or Surface Sonic Echo and Impulse Response tests.

5. CSL TESTING RESULTS.

The CSL results shall be presented to the Engineer in a report. This report shall include recommendations as to the acceptability, unacceptability, soundness, etc., of the drilled shaft. The report shall be checked, stamped approved, and signed by a Professional Engineer licensed by the Alabama Board of Licensure for Professional Engineers. This Professional Engineer shall not be an employee of the ALDOT. The report shall be submitted directly to the Materials and Tests Engineer with a copy to the Project Engineer. The test results shall include CSL logs with analyses of:

- Initial pulse arrival time versus depth
- Pulse energy/amplitude versus depth

A CSL log shall be presented for each tube pair tested with any defect zones indicated on the logs and discussed in the test report as appropriate.

6. EVALUATION OF CSL TEST RESULTS.

The Engineer will evaluate the CSL test results and determine whether or not the drilled shaft construction is acceptable. This evaluation will be completed within 14 calendar days of the date of receipt of the report by the Materials & Tests Engineer.

If the Engineer determines that the drilled shaft is acceptable, the CSL tubes shall be dewatered and grouted. The grout shall be of the same strength or higher than the strength of the concrete used in the original drilled shaft. The contractor may use any of the grout mixes listed in Table 1 of Item 453.03(b)2. with the exception that calcium chloride will not be allowed. The contractor may submit another design mix for approval.

If the Engineer determines that the drilled shaft is unacceptable, the shaft shall be cored in accordance with the requirements given in Subarticle 506.10(b) to allow further evaluation of the shaft. Cores shall be taken without additional compensation unless the testing of the cores indicates that the concrete in the shaft meets all specification requirements. If the testing of the cores indicates that the concrete meets specification requirements, the cost of the coring will be paid for as Extra Work.

(b) CORE DRILLING OF DRILLED SHAFT CONCRETE.

Production or trial drilled shafts that are determined to be unacceptable by the CSL tests may be cored to determine the quality of the shaft. The required number and depth of cores will be determined by the Engineer.

Because it is necessary to obtain a high percentage of core recovery for visual inspection and compressive strength testing, the core bit used for core drilling shall be warranted by the manufacturer as being capable of coring the concrete as strong as could possibly be present in the shaft. A new bit or new core barrel will be required at any time the Engineer determines that the equipment may not be capable of obtaining good quality cores. The minimum diameter of the cores shall be 3.0 inches {76 mm}.

An accurate log of cores shall be kept and the cores shall be placed in a crate and properly marked showing the shaft depth at each interval of core recovery. The cores along with three copies of the coring log shall be transported to the ALDOT Bureau of Materials and Tests, Montgomery, Alabama, for inspection.

Construction shall not proceed above a drilled shaft until the quality of the shaft, as represented by the core samples, is determined to be acceptable and notification to continue construction is given by the ALDOT Construction Engineer.

If the Engineer determines that the drilled shaft is acceptable, the core holes and the CSL tubes shall be dewatered and grouted. The grout shall be of the same strength or higher than the strength of the concrete used in the original drilled shaft. The contractor may use any of the grout mixes listed in Table 1 of Item 453.03(b)2. with the exception that calcium chloride will not be allowed. The contractor may submit another grout design mix for approval.

If the quality of the drilled shaft is determined to be unacceptable then the Contractor shall construct another foundation to carry the load that will be placed on the shaft or perform corrective work as required by the Department. This foundation or the corrective work shall be constructed without compensation from the Department. The details of the replacement foundation shall be submitted in accordance with the requirements given in Article 105.02 for Working Drawings.

506.11 Drilled Shaft Construction Tolerances.

The following construction tolerances apply to drilled shafts unless otherwise stated in the contract documents. Drilled shaft excavations and completed shafts not constructed within the required tolerances are unacceptable. The Contractor shall correct all unacceptable shaft excavations and completed shafts to the satisfaction of the Engineer. Materials and work necessary to complete corrections for out of tolerance drilled shaft excavations and/or completed shafts, including engineering analysis and redesign, shall be furnished without either cost to the State or an extension of the contract time of the project.

(a) GENERAL LOCATION.

The drilled shaft shall be within 3 inches {75 mm} of plan position in the horizontal plane at the elevation of the top of the shaft.

(b) VERTICAL ALIGNMENT.

The vertical alignment of a shaft excavation shall not vary from the plan alignment by more than 1/4 inch per foot {20 mm/m} of depth. The alignment of a battered shaft excavation shall not vary by more than 1/2 inch per foot {40 mm/m} of depth from the prescribed batter.

(c) REINFORCING STEEL CAGE.

The spacers for the reinforcing cage shall have a tolerance of minus 1 inch {25 mm} from the required spacing shown on the plans.

The reinforcing steel cage shall be within 1 inch {25 mm} of plan position in the horizontal plane at the elevation of the top of the shaft.

After all the concrete is placed, the top of the reinforcing steel cage shall be no more than 6 inches {150 mm} above and no more than 3 inches {75 mm} below plan position.

(d) CASINGS.

All casing diameters shown on the plans refer to OD (outside diameter) dimensions. Casing shall be clean, round, straight and free of weld breaks and/or holes that would permit passage of water or wet concrete. When approved by the Engineer, the Contractor may elect to provide a casing larger in diameter than shown in the plans. No payment will be made for additional construction materials used in accommodating the Contractor's request for a larger casing diameter.

(e) SHAFT SOCKET.

The diameter of an excavated socket shall have a tolerance of minus 2 inches {50 mm} from the plan diameter.

(f) TOP ELEVATION OF SHAFTS.

The top elevation of the shaft shall have a tolerance of plus 1 inch {25 mm} or minus 3 inches {75 mm} from the plan top of shaft elevation.

(g) EXCAVATION EQUIPMENT AND METHODS.

Excavation equipment and methods shall be designed so that the completed shaft excavation will have a planar bottom. The cutting edges of excavation equipment shall be normal to the vertical axis of the equipment within a tolerance of +/- 3 % of the diameter.

506.12 Drilled Shafts Constructed and Exhumed for Research Purposes.

(a) DETAILS OF THE DRILLED SHAFTS FOR RESEARCH PURPOSES.

The drilled shafts for research purposes shall be constructed in accordance with the details shown on the plans. The approximate required location of the shafts is shown on the plans. The Engineer will designate the final required location of these shafts.

(b) PROTECTION FROM DISTURBANCE OF SHAFTS CONSTRUCTED FOR RESEARCH.

At least 16 hours after the concrete in each drilled shaft has been placed, adjacent shafts shall not be excavated, adjacent piles shall not be driven, and equipment wheel loads and vibration from equipment shall not be allowed to occur at any point within a 25 foot radius of the drilled shaft.

(c) INSTRUMENTATION OF SHAFTS CONSTRUCTED FOR RESEARCH.

The Contractor shall inform the Engineer at least 72 hours and no more than 7 days before the installation of each reinforcing cage. This notification is required to allow the attachment of instrumentation to the cages. Instrumentation will be furnished and installed by the Engineer. The Contractor shall allow a delay of up to 6 hours per shaft for the installation and calibration of the instrumentation.

(d) CROSSHOLE SONIC LOGGING OF SHAFTS CONSTRUCTED FOR RESEARCH.

The shafts shall be tested by Crosshole Sonic Logging (CSL).

[handwritten note: Do we review or Send them to AU]

(e) EXUMATION OF SHAFTS CONSTRUCTED FOR RESEARCH.

The drilled shafts constructed for research shall be exhumed by the Contractor no earlier than 28 days after the completion of the placement of the concrete. Exhumation of the drilled shafts may require the use of temporary shoring and casing. The Contractor shall protect and maintain the integrity of the drilled shaft castings during the exhumation process. The Contractor shall construct and exhume a replacement drilled shaft if a shaft is damaged during exhumation.

(f) BACKFILL AT LOCATION OF EXHUMED SHAFTS.

The void left after the exhumation of the drilled shafts shall be backfilled with coarse aggregate as directed by the Engineer. The coarse aggregate material shall be crushed or uncrushed gravel, crushed stone or crushed slag. The coarse aggregate shall not be contaminated with adherent material. The maximum size of any component of the coarse aggregate shall be 6 inches. Adherence to a gradation will not be required but sufficient small size aggregate shall be present to choke the larger size aggregate. Moisture and density controls will also not be required. The aggregate shall be mixed to provide a uniform distribution of the aggregate sizes in the backfill. The aggregate backfill shall be compacted and graded during placement to provide a uniform stable backfill.

There will be no measurement or payment for the placement of more than 80 tons of aggregate backfill per experimental shaft. This limitation shall apply regardless of the allowance for overruns in quantities noted elsewhere in these specifications. The Contractor shall control the excavation of the shafts to minimize the amount of aggregate backfill that must be placed to fill the void. The Contractor shall place as much aggregate beyond the measurement and payment limit as is required to completely fill the void.

The coarse aggregate shall come from an aggregate producer that is participating in and meeting the requirements given in ALDOT-249, "Quality Control Program for Acceptance of Fine and Coarse Aggregates". The producer's name shall be listed in the Department's "Materials, Sources and Devices with Special Acceptance Requirements" manual, List I-1.

(g) LAYDOWN AREA FOR EXUMED DRILLED SHAFTS.

The Contractor shall prepare and maintain a laydown area at the site for the exhumed drilled shaft castings to be placed horizontally. The Contractor shall submit the proposed location and configuration of the laydown area to the Engineer for approval prior to exhumation of the shafts. The laydown area shall be large enough to accommodate the exhumed experimental drilled shafts and the equipment used for wire sawing, coring and other physical tests. The laydown area shall be maintained to allow access at all times by to the Engineer.

(h) PREPARATION AND TESTING OF THE EXHUMED DRILLED SHAFTS.

1. CLEANING THE EXUMED SHAFTS.

The Contractor shall pressure wash the exterior surface of all exhumed shafts immediately after placement of the shafts in the laydown area. All debris and soil shall be removed from the exterior of all shafts so that the surface of the shafts is visible without obstruction.

2. CUTTING THE SHAFTS.

The shafts will be cut by research personnel with a diamond wire saw. At least four cuts will be made through the cross section of each shaft. The Contractor shall move each cut piece of the shaft as directed by the Engineer to allow the coring of the exposed faces of the cuts by research personnel.

3. DISPOSAL OF THE SHAFTS.

The Contractor shall allow research personnel approximately one month to perform the research testing on the shafts in the laydown area. The Contractor shall remove and dispose all parts of the exhumed drilled shaft castings after the testing is complete.

506.13 Method of Measurement.

(a) DRILLED SHAFT EXCAVATION.

Drilled shaft excavation will be measured by the linear foot {meter} of excavated shaft.

(b) SPECIAL DRILLED SHAFT EXCAVATION.

Special drilled shaft excavation will be measured by the linear foot {meter} of excavated shaft.

(c) DRILLED SHAFT CONSTRUCTION.

Drilled shaft construction will be measured by the linear foot {meter} of shaft.

(d) EXPLORATION BELOW DRILLED SHAFT.

The exploratory drilling below the bottom of a drilled shaft will be measured by the linear foot {meter} of core hole.

(e) PERMANENT DRILLED SHAFT CASING.

Permanent drilled shaft casings will be measured by the linear foot {meter} of casing left in place.

(f) CROSSHOLE SONIC LOGGING (CSL).

Testing by the CSL method will be measured per each shaft tested.

(g) DRILLED SHAFT EXHUMATION.

Exhumation of a drilled shaft will be measured by the linear foot of the shaft.

(h) AGGREGATE FOR DRILLED SHAFT BACKFILL.

The coarse aggregate for backfilling the void at an exhumed drilled shaft will be measured in units of tons {metric tons}. There will be no measurement or payment for the placement of more than 80 tons of aggregate backfill per experimental shaft. This limitation shall apply regardless of the allowance for overruns in quantities noted elsewhere in these specifications. The Contractor shall control the excavation of the shafts to minimize the amount of aggregate backfill that must be placed to fill the void. The Contractor shall place as much aggregate beyond the measurement and payment limit as is required to completely fill the void.

506.14 Basis of Payment.

(a) DRILLED SHAFT EXCAVATION.

The linear foot {per meter} bid price shall be full compensation for all labor, materials and equipment required to complete and support the excavation. This shall also be full compensation for the utilization of slurry and temporary casings, for the disposal of all surplus excavated materials and for incidentals necessary to complete the work. No additional payment will be made for larger diameter or deeper excavations that are made by the choice of the Contractor.

(b) SPECIAL DRILLED SHAFT EXCAVATION.

The linear foot {per meter} bid price shall be full compensation for all labor, materials and special equipment required to complete and support the excavation. This shall also be full compensation for the removal of obstructions, the utilization of slurry and temporary casings, for the disposal of all surplus excavated materials and for incidentals necessary to complete the work. No additional payment will be made for larger diameter or deeper excavations that are made by the choice of the Contractor.

(c) DRILLED SHAFT CONSTRUCTION.

The linear foot {per meter} bid price shall be full compensation for all labor, materials, equipment and incidentals required for the construction of a shaft except for reinforcing steel which will be paid for under Item 502-A. No additional compensation will be made for larger diameter or deeper shafts that are constructed by the choice of the Contractor.

(d) EXPLORATION BELOW DRILLED SHAFT.

The linear foot {per meter} bid price shall be full compensation for all labor, materials, equipment and incidentals required for coring and sample retrieval.

(e) TRIAL DRILLED SHAFT.

Payment for a trial drilled shaft will be made under the appropriate production drilled shaft items of 506-A, B, C, F or G as they may apply. No separate payment will be made for cutting off the trial shaft or site restoration.

(f) PERMANENT DRILLED SHAFT CASING.

The linear foot {per meter} bid price shall be full compensation for all labor, materials, equipment and incidentals required for furnishing, painting and installing the casing. No payment will be made for cutoffs.

If there is no pay item in the contract for permanent casing then the casing will be paid for as extra work as outlined in Article 104.03, Extra Work.

(g) CROSSHOLE SONIC LOGGING.

The price bid for each shaft tested shall be full compensation for all labor, materials, equipment and incidentals necessary to perform the required test and furnish the Engineer with the test results. The bid price shall also include dewatering the tubes and filling the tubes with grout.

Where a drilled shaft consists of different shaft diameters, the price bid shall be full compensation for the sonic logging of the complete depth of the drilled shaft, regardless of differences in the diameter of the shaft. The shaft diameter shown in the pay item for sonic logging is for identification purposes and will be the smallest diameter portion of a drilled shaft.

(h) DRILLED SHAFT EXHUMATION.

The linear foot {per meter} bid price for the exhumation of a drilled shaft shall be full compensation for all labor, materials, equipment and incidentals required for exhuming and cleaning the shaft, moving and positioning the shaft at a laydown area.

(i) AGGREGATE FOR DRILLED SHAFT BACKFILL.

The per ton {metric ton} bid price for the coarse aggregate for backfilling the void at an exhumed drilled shaft shall be full compensation for all labor, materials, equipment and incidentals required for furnishing and placing the aggregate.

(j) PAYMENT WILL BE MADE UNDER ITEM NO.:

506-A   Drilled Shaft Excavation, _*_ Diameter - per linear foot {meter}

506-B   Special Drilled Shaft Excavation, _*_ Diameter - per linear foot {meter}

506-C   Drilled Shaft Construction, _*_ Diameter, Class _**_ Concrete - per linear foot {meter}

506-D   Exploration Below Drilled Shaft - per linear foot {meter}

506-F   Permanent Drilled Shaft Casing, _*_ Diameter - per linear foot {meter}

506-G   Crosshole Sonic Logging, _*_ Diameter - per each

506-N   Drilled Shaft Exhumation, _*_ Diameter - per linear foot {meter}

506-O   Aggregate for Drilled Shaft Backfill - per ton {metric ton}[SEE NOTE BELOW]

*   Specify diameter of shaft in feet and inches {millimeters}.

**   Specify either "DS1", "DS2" or "DS3".

NOTE: The maximum quantity for which payment will be made for Pay Item 506-O "Aggregate for Drilled Shaft Backfill" will be 80 tons per each of the three required experimental shafts.

Appendix B: Production Shaft Specifications

ALABAMA DEPARTMENT OF TRANSPORTATION

DATE: February 2, 2009   Special Provision No. 08-0332

SUBJECT: Drilled Shaft Construction, Project Number BRF-0035(503), Jackson County.

Alabama Standard Specifications, 2008 Edition, shall be amended by the replacing SECTION 506 with the following:

SECTION 506
DRILLED SHAFT CONSTRUCTION

506.01 Description.
This work shall consist of all labor, materials, equipment and services necessary to perform all operations to complete a drilled shaft installation in accordance with these Specifications and the details and dimensions shown on the plans.

506.02 Materials.

(a) GENERAL.

All materials shall conform to requirements set forth in Division 800, Materials. The requirements provided for Structural Portland Cement Concrete, Section 501, shall apply in all respects to drilled shafts, except where otherwise indicated by specific requirements given in this Section or noted by plan details.

(b) CONCRETE.

Portland cement concrete used in construction of drilled shafts shall designated as either "Class DS1", "Class DS2", "Class DS3" or "Class HPDS" concrete. The specific class of concrete that is required will be shown in the Pay Item Description for Drilled Shaft Construction.

The concrete producer shall establish the proportion of materials for each class of drilled shaft concrete following the guidelines described in ALDOT-170, *"Method of Controlling Concrete Operations for Structural Portland Cement Concrete"*, except that, instead of the reference to the Master Proportion Table, the concrete producer shall use the criteria outlined hereinafter in this Subarticle. The concrete supplier shall submit for approval the proposed concrete mix design to the State Materials and Test Engineer following the requirements in ALDOT-170. The distribution of the approved concrete mix design and re-approval of concrete mix designs will be as per ALDOT-170 respectively. Any changes of the materials and/or proportions of the mix design will require a concrete mix resubmittal.

1. Criteria applicable to Class DS1, Class DS2, Class DS3, and Class HPDS concrete:
   Minimum Compressive Strength at 28 days shall be 4000 psi {30 MPa}.
   The amount of cementitious material shall be a minimum of 600 pounds {360 kg} and a maximum of 800 pounds per cubic yard {475 kg per cubic meter} of concrete.
   The total air content shall not exceed 6.0 % by volume.
   The maximum water to total cementitious material ratio shall be 0.40.
   The temperature of the concrete, at the time of placement in the shaft, shall not be less than 50 °F {10 °C} nor more than 95 °F {35 °C}.

2. Criteria applicable to Class DS1, Class DS2 and Class DS3 concrete:
   The allowable range of consistency slump during concrete placement shall be from 6 inches to 9 inches {150 mm to 230 mm}. The minimum consistency slump for all of the concrete placed in an individual shaft shall be no less than 4 inches {100 mm} at the end of the concrete placement in that shaft.

Gradation of the coarse aggregate shall meet the requirements for either ALDOT Size No. 57, No. 67 or No. 7.

2. Additional criteria applicable to Class DS1 concrete:

Either Type I or Type II cement shall be used.

The cementitious content may be composed of up to 30% by weight {mass} substitution of either Class C or Class F fly ash additive. In lieu of fly ash, ground granulated blast furnace slag may be substituted for cement up to a minimum substitution rate of 25% and a maximum substitution rate of 50% by weight {mass}.

3. Additional criteria applicable to Class DS2 concrete:

Type II cement shall be used.

The cementitious content shall be composed of no less than 20% nor more than 30% by weight {mass} of Class F fly ash additive. In lieu of fly ash, ground granulated blast furnace slag may be substituted for cement up to a minimum substitution rate of 35% and a maximum substitution rate of 50% by weight {mass}.

4. Additional criteria applicable to Class DS3 concrete:

Type II cement shall be used.

The cementitious content shall be composed of 20% by weight {mass} of Class F fly ash and 10% by weight {mass} of microsilica additives. In lieu of the percentages of fly ash and microsilica, the cementitious content may be composed of 50% by weight {mass} substitution of ground granulated blast furnace slag and 5% by weight {mass} addition of microsilica additives.

5. Additional criteria applicable to Class HPDS concrete:

Type I cement shall be used.

The cementitious content may be composed of up to 30 % by weight {mass} substitution of either Class C or Class F fly ash additive. In lieu of fly ash, ground granulated blast furnace slag may be substituted for cement up to a minimum substitution rate of 25 % and a maximum substitution rate of 50 % by weight {mass}.

Slump Flow of 21 inches ± 3 inches as per ASTM C 1611, Standard Test Method for Slump Flow of Self-Consolidating Concrete.

Visual Stability Index (VSI) of 1.5 or less during concrete placement as per ASTM C 1611.

Fine aggregate shall be No. 100 natural sand. The fine sand to total aggregate by volume shall be 0.45 % to 5.0 %.

Coarse aggregate shall be No. 67 or 78.

A Type "D", Water Reducing and Retarding Admixture, shall be used for hydration stabilization to control the set time and temperature of the concrete.

A Type "F", polycarboxylate based High Range Water Reducing Admixture, shall be used to increase flowability of the concrete. The dosage shall be determined to meet the slump flow requirements at the time of placement. A Type "F" admixture may only be added at the batch plant.

A Viscosity-Modifying Admixture (VMA) shall be used to control the stability of the concrete mix.

All chemical admixtures including VMA shall be from the same manufacturer

(b) SLURRY.

When use of slurry is either shown to be required in the contract documents or selected by the Contractor, mineral slurries shall be used unless another type of slurry is proposed for use by the Contractor and approved by the Engineer. The following minimum requirements apply to material components used in slurries:

1. APPROVED MINERALS.

Sodium Bentonite or Attapulgite shall be used as the principal mineral constituents of slurry. The Engineer may approve use of other minerals upon receipt of demonstrated proof that the requested alternate mineral insures shaft stability at the applicable shaft construction site.

2. MIXING WATER.

Mixing water shall be capable of meeting drinking water standards as outlined in Section 807.

3. SAND.

Clean, locally available sand meeting the requirements of Section 802 (not to exceed four (4) percent by volume) may be mixed in drilling slurries.

4. ADDITIVES.

At the Contractors discretion, additives may be used to control the consistency and/or yield of slurries subject to the limitation that the type and amount of additives used shall not exceed the recommendation(s) of the principal mineral manufacturer.

(c) CASING.

When use of casing is either specified by the contract documents or selected by the Contractor, casings shall be smooth, non-corrugated, clean, watertight steel of ample strength to withstand both handling and driving stresses and the pressures of concrete and the surrounding earth materials. Where permanent casing is required, serviceable used casing may be installed with the approval of the Engineer.

The Contractor is responsible for insuring that all casing, new or used, is capable of withstanding the aforementioned stress and pressure requirements.

(d) STEEL REINFORCEMENT.

Unless otherwise noted on the contract documents, all steel reinforcement shall be Grade 60 {420} billet steel meeting the requirements of Section 502, sized and installed in accordance with the contract plans as applicable. Welding of the reinforcing steel will not be permitted without the written approval of the Bridge Engineer. Welding to the main vertical reinforcing steel will not be permitted.

506.03 Construction Methods and Equipment.

The Contractor shall perform excavations required for shafts through whatever materials are encountered, to the dimensions and elevations shown in the plans or otherwise required by the specifications and special provisions. The Contractor's methods and equipment shall be suitable for the site conditions and materials encountered. The permanent casing method shall be used only at locations shown on the plans or authorized by the Bridge Engineer.

Actual cores recovered from the test borings are available for inspection at the Bureau of Materials and Tests.

(a) GENERAL REQUIREMENTS.

1. CONTRACTOR QUALIFICATIONS.

The Contractor shall submit descriptions of the drilled shaft construction projects completed in the last three years to serve as evidence of the capability to construct drilled shafts. The descriptions of the drilled shaft projects shall contain names and telephone numbers of owners' representatives who can verify the Contractor's participation on those projects. These descriptions shall be submitted with the Installation Plan and will be evaluated by the Engineer.

The evaluation of the Contractor's capability for constructing drilled shafts will have a bearing on the decision by the Engineer to require the construction of a Trial Drilled Shaft.

2. INSTALLATION PLAN.

a. Installation Plan Requirements.

No later than 30 days after the date of the Notice to Proceed, the Contractor shall submit an installation plan for review by the Engineer. This plan shall provide information on the following items as applicable:

- Name and experience record of the drilled shaft superintendent in charge of drilled shaft operations for this project;
- List of proposed equipment to be used including cranes, drills, augers, bailing buckets, final cleaning equipment, desanding equipment, slurry pumps, core sampling equipment, tremies, concrete pumps, casing, etc.;
- Details of the overall anticipated construction operation sequence and the proposed sequence of shaft construction;
- Details of planned shaft excavation methods;

- Details of the methods to be used to insure shaft stability (i.e. prevention of caving, bottom heave, etc., using temporary casing, slurry or other means) during excavation and concrete placement. This shall include a review of method suitability to the anticipated site and subsurface conditions. If casings are proposed or required, casing dimensions and detailed procedures for permanent casing installation, and temporary casing installation and removal shall be provided.

- When use of slurry is required or proposed, details of the methods for mixing, circulating and desanding slurry;

- Details of methods to clean the shaft excavation;

- Details of reinforcement placement including support and centralization methods;

- Details of concrete placement method required or proposed including operational procedures for free fall, tremie or pumping as appropriate; and

- The method used to fill or eliminate all voids between the plan shaft diameter and excavated shaft diameter, or between the shaft casing and surrounding soil, if permanent casing is specified.

- Details of the material, equipment, and procedures proposed to accomplish the required load testing.

b. Evaluation of Installation Plan.

The Engineer will evaluate the drilled shaft installation plan for conformance with the plans and specifications. Within 15 days following receipt of the installation plan, the ALDOT Construction Engineer will return the plan for corrections, distribute the plan for construction inspection, or contact the Contractor to establish a mutually agreeable date and time for a meeting to discuss the installation plan. If a meeting is held to discuss the installation plan the Contractor and his drilled shaft project superintendent shall be in attendance. The Contractor will be notified of changes in the submitted installation plan deemed necessary by the ALDOT Construction Engineer within seven days after the aforementioned meeting. Shaft construction shall not begin until the installation plan has been distributed by the ALDOT Construction Engineer for construction inspection. Distribution of the installation plan for construction inspection shall not relieve the Contractor of the responsibility to satisfactorily complete the work as detailed on the plans and in the specifications.

c. Modification of Installation Plan.

Any proposed modification of the installation plan during construction shall be submitted to the Construction Engineer for review and distribution.

3. PROTECTION OF EXISTING STRUCTURES AND UTILITIES.

The Contractor shall control his operations to prevent damage to existing structures and utilities as outlined in Article 107.12. Preventive measures shall include, but are not limited to, selecting construction methods and procedures that will prevent caving of the shaft excavation, monitoring and controlling the vibrations from construction activities such as the driving of casing or sheeting, drilling of the shaft, or from blasting, if permitted.

4. CONSTRUCTION SEQUENCE.

a. Excavation to the bottom of shaft elevation shall be completed before shaft construction begins unless otherwise noted in the contract documents or approved by the Engineer. Any disturbance caused by shaft installation to a planned drilled shaft area shall be repaired by the Contractor prior to the shaft construction.

b. When drilled shafts are to be installed in conjunction with embankment placement, the Contractor shall construct drilled shafts after the placement of fills unless shown otherwise in the contract documents or approved by the Engineer.

c. Substructure concrete shall not be placed on a drilled shaft until the concrete in the shaft reaches a minimum of 80 % of the required 28-day compressive strength and until all CSL test results (when required) are accepted and the CSL tubes have been dewatered and grouted.

(b) METHODS OF CONSTRUCTION.

1. DRY METHOD.

The dry construction method shall be used only at sites where the groundwater level and soil conditions are suitable to permit construction of the shaft in a relatively dry excavation, and where the sides and bottom of the shaft may be visually inspected by the Engineer prior to placing

reinforcement and concrete. The dry method consists of drilling the shaft excavation, removing accumulated water and loose material from the excavation, placing the reinforcing cage, and concreting the shaft in less than 3 inches of water.

2. WET METHOD.

The wet construction method may be used at sites where a dry excavation can not be maintained for placement of the shaft concrete. This method consists of using water or mineral slurry to maintain stability of the hole perimeter while advancing the excavation to final depth, placing the reinforcing cage, and concreting the shaft. Where drilled shafts are located in open water areas, exterior casings shall be extended from above the water elevation into the ground to protect the shaft concrete from water action during placement and curing of the concrete. The casing shall be installed in a manner that will produce a positive seal at the bottom of the casing so that no seepage of water or other materials occurs into or from the shaft excavation.

3. CASING FOR DRY OR WET CONSTRUCTION METHODS.

Permanent or temporary casing may be used when shown on the plans or at sites where the dry or wet construction methods are inadequate to prevent caving or excessive deformation of the hole. In this method the casing may be either placed in a predrilled hole or advanced through the ground by twisting, driving or vibration before being cleaned out. Casing which is going to be installed by predrilling and permanently left in rock for the purpose of shielding voids, shall be installed in not more than a 2 inch {50 mm} oversized drill hole. When downsizing of permanent casing is required, no more than six feet of overlap of casing will be allowed.

When the casing method is required but not shown on the plans, the Contractor shall submit details of the proposed casing method (including casing lengths and diameters) and the proposed procedures of casing installation to the Bridge Engineer for review in the installation plan. If the need is determined after work on the shafts has begun, a revised plan proposing this method must be submitted for review.

(c) EXCAVATION PROCEDURES.

1. EXCAVATION LOCATION, COORDINATION AND TIME CONSTRAINTS.

Shaft excavations shall be made at locations and to the elevations, geometry and dimensions shown in the contract documents or as directed by the Engineer.

A shaft shall not be excavated as long as an adjacent shaft in the same substructure unit is open unless authorized in writing by the Construction Engineer. Blasting and vibrating casings in place will not be allowed until the concrete in adjacent shafts has reached 80 % of the required 28-day compressive strength.

Once the excavation of a shaft has been started, the excavation shall be conducted in a continuous operation until the excavation is completed.

When an excavation is performed with any type of drilling fluid (i.e. slurry, water, etc.) used to stabilize the excavation, the placement of concrete shall begin within 36 hours from the start of excavation and within 12 hours from the start of the excavation of the bottom 5 feet {1.5 m} of the shaft. If the Contractor exceeds these time limits, additional work may be required to insure that the condition of the excavation is adequate to result in an acceptable load carrying capacity in the completed drilled shaft. The Contractor may be required to over ream the entire depth of excavation (or the bottom 5 feet {1.5 m} if the 12 hour time limit is exceeded), increase the depth of the excavation or perform other work that may be required by the Engineer to provide an acceptable excavation. There will be no compensation for this additional work.

The minimum width of over reaming shall be 1/2 inch {13 mm} and the maximum width shall be 3 inched {75 mm}.

2. EXCAVATION LOG.

The Contractor shall maintain an excavation log during shaft excavation. The log shall contain information such as: the description and approximate top and bottom elevation of each soil or rock material encountered during shaft excavation, elevations at which seepage or groundwater flow are encountered, and remarks. The type of tools used for the excavation shall be shown on the log. All changes in the type of tools used for excavation shall be shown on the log. The Engineer will monitor

these operations and the logs will be used as a basis of measurement for payment. The Contractor shall resolve all discrepancies on the log noted by the Engineer at the end of each work day. Two copies of the legible, final log shall be furnished to the Engineer within 24 hours after a shaft excavation is completed and accepted.

      3. HANDLING EXCAVATED MATERIAL.

           Excavated materials which are removed from shaft excavations shall be disposed of by the Contractor in accordance with Subarticle 215.03(g).

      4. EXCAVATION SAFETY.

           The Contractor shall not permit workers to enter the shaft excavation for any reason unless: suitable casing has been installed, the water level has been lowered and stabilized below the level to be occupied, and adequate safety equipment and procedures have been provided to protect workers entering the excavation. The Contractor is responsible for compliance with applicable State and Federal safety regulations.

  (d) TYPES OF DRILLED SHAFT EXCAVATION.

      1. DRILLED SHAFT EXCAVATION.

           The excavation of the shaft using conventional earth drilled shaft excavation tools will be designated as "drilled shaft excavation".

      2. SPECIAL DRILLED SHAFT EXCAVATION.

           The excavation of the shaft requiring rock tools and/or procedures to accomplish hole advancement will be designated as "special drilled shaft excavation". This excavation will be for the removal of rock or other hard material within the planned shaft.

  (e) EXCAVATING AND DRILLING EQUIPMENT.

      1. GENERAL.

           Excavation and drilling equipment shall have adequate capacity including power, torque and down thrust to excavate a hole of both the maximum specified diameter and to a depth of twenty (20) percent beyond the depths shown on the plans when operated at rated capacity.

      2. ROCK TOOLS AND EQUIPMENT.

           When the material encountered cannot be drilled using conventional earth drilling tools and equipment, the Contractor shall provide rock drilling equipment including air tools, approved blasting materials, and other equipment as necessary to construct the shaft excavation to the size and depth required. Concurrence of the Engineer shall be obtained prior to switching from earth to rock drilling tools and equipment. Approval of the Engineer is required before excavation by blasting is permitted.

      3. OVERREAMING.

           a. Sidewall overreaming shall be required when the sidewall of the hole is determined to have either softened due to excavation methods, swollen due to delays in concreting, or degraded because of slurry cake buildup. Overreaming thickness shall be a minimum of 1/2 inch {13 mm} and a maximum of 3 inches {75 mm}.

           b. Overreaming may be accomplished with a grooving tool, overreaming bucket or other approved equipment. The thickness and extent of sidewall overreaming shall be as directed by the Engineer. The Contractor shall bear all costs associated with both sidewall overreaming and additional shaft concrete placement.

      4. LOST TOOLS.

           Drilling tools which are lost in the excavation shall not be considered obstructions and shall be promptly removed by the Contractor without compensation. All costs due to lost tool removal shall be borne by the Contractor including costs associated with correcting hole degradation due to removal operations and time delays.

  (f) EXPLORATORY SHAFT EXCAVATION.

      1. GENERAL.

           The Contractor will be required to perform some type of exploratory shaft excavation (soil samples, rock cores or drilling or probing) below the bottom elevations shown on the plans unless

this requirement is noted on the plans as being deleted. The Contractor shall extend drilled shaft tip elevations when the Engineer determines that the material encountered during this exploratory excavation is unsuitable and/or differs from that anticipated in the design of the drilled shaft.

2. ROCK CORES AND SOIL SAMPLES.

The Contractor shall take 2.0 inch {51 mm} minimum diameter rock cores and/or soil samples at locations as designated on the plans or as directed by the Engineer to determine the character of the material directly below the completed shaft excavation. The soil samples shall be extracted with a split spoon sampler or undisturbed sample tube in accordance with AASHTO T 206 and T 207. The methods and equipment used for the rock coring shall be those given in Subarticle 506.10(b) for the core drilling of drilled shaft concrete. The cores and/or soil samples shall be taken to a minimum of 10 feet {3 m} below the bottom of the drilled shaft excavation unless otherwise noted on the plans or directed by the Geotechnical Engineer. The Engineer may require this depth to be extended up to a total depth of 20 feet {6 m} below the bottom of the shaft. The Contractor may choose to take these cores and/or soil samples prior to excavating for the drilled shafts, however, payment will only be considered for that portion of the cores taken below the bottom elevation of the shafts shown on the plans.

Rock core and soil test samples shall be measured, visually identified and described on the Contractor's log. The samples shall be placed in suitable containers, identified by shaft location, elevation and project number and delivered to the Central Laboratory in Montgomery with the Contractor's field log within 24-hours after the exploration is completed. The Engineer will inspect the samples/cores and determine the final depth of required excavation based on his evaluation of the sampled materials suitability.

3. DRILLING OR PROBING.

At all drilled shaft locations where rock cores and/or soil samples are not designated, the Contractor will be required to drill or probe an exploratory hole below the bottom elevation of the shaft to determine if any voids or crevices are present. The exploratory hole shall be taken to a depth of 10 feet {3 m}, unless noted otherwise on the plans. Exploratory drilling or probing will not be required if it is noted on the plans that this requirement is not necessary. No direct payment will be made for this operation.

(g) OBSTRUCTION REMOVAL.

Surface and subsurface obstructions at drilled shaft locations shall be removed by the Contractor. Such obstructions may include man-made materials such as old concrete foundations and natural materials such as boulders. Special procedures and/or tools shall be employed by the Contractor in the event the hole cannot be advanced using conventional augers fitted with soil or rock teeth, drilling buckets and/or underreaming tools. Special procedures/tools may include but are not limited to: chisels, boulder breakers, core barrels, air tools, hand excavation, temporary casing, and increasing the hole diameter. Blasting shall not be permitted unless specifically approved in writing by the Engineer. Removal of obstructions will be classified as "special drilled shaft excavation".

(h) TRIAL DRILLED SHAFT INSTALLATION.

1. GENERAL.

The Engineer will require the construction of a trial shaft if the submittal of descriptions of previous drilled shaft construction projects does not, in the opinion of the Engineer, substantiate the Contractor's capability for constructing the drilled shafts on this project. The Engineer may also require the construction of a trial shaft to verify the adequacy of unusual construction methods and/or equipment proposed for use in the construction of the production shafts.

The trial drilled shaft shall be constructed if required by special note on the plans.

2. LOCATION AND DEPTH.

The trial shaft(s) shall be positioned as indicated on the plans or as directed by the Engineer. Unless otherwise indicated, shafts shall be drilled to the maximum depth of any production shaft shown on the plans.

3. FAILURE TO DEMONSTRATE ABILITY.

Failure of the Contractor to demonstrate the adequacy of his equipment, methods and/or expertise shall be reason for the Engineer to require alterations necessary to eliminate unsatisfactory results. Additional trial shafts required to demonstrate correction of deficiencies shall be at the Contractor's expense.

4. TRIAL SHAFT APPROVAL.

Once approval has been given to construct production shafts, no changes will be permitted in the personnel, methods or equipment that were used to construct the satisfactory trial shaft without written approval of the Engineer.

5. SITE RESTORATION.

Unless otherwise shown in the contract documents, the trial shaft holes will be filled with non-reinforced concrete in the same manner that production shafts will be constructed. The concreted trial shafts shall be cutoff 2 feet {600 mm} below finished grade or at the mudline if in water. The disturbed areas at trial shaft holes shall be restored as nearly as practical to their original condition. No direct payment will be made for cutting off the top of the trial shaft or for the site restoration.

506.04 Encased Excavations.

(a) GENERAL.

The outside diameter of casings shall not be less than the specified shaft size. No extra compensation will be allowed for concrete required to fill an oversized casing or excavation. All casings, except permanent casing, shall be removed from shaft excavations.

(b) TEMPORARY CASING.

1. GENERAL.

All casing shall be considered temporary unless specifically shown as permanent casing in the contract documents. The Contractor will be required to remove temporary casing before completion of concreting the drilled shaft. Telescoping, predrilling with slurry, and/or overreaming to beyond the outside diameter of the casing may be required to install casing.

2. SIZE SUBSTITUTION.

If the Contractor elects to remove a specified diameter or length of casing and substitute a longer or larger diameter casing through caving soils, the excavation shall be either stabilized with slurry or backfilled before the new casing is installed. Other methods, as approved by the Engineer, may be used to control the stability of the excavation and protect the integrity of the foundation soils.

3. BOUND OR FOULED CASINGS.

Temporary casings which become bound or fouled during shaft construction and cannot be practically removed shall constitute a defect in the drilled shaft. The Contractor shall be responsible for correcting such defective shafts to the satisfaction of the Engineer. Correction may consist of, but is not limited to: removing the shaft concrete and extending the shaft deeper to compensate for loss of frictional capacity in the cased zone, providing straddle shafts to compensate for capacity loss, or providing a replacement shaft. All corrective measures including redesign of shafts caused by defective shafts shall be done to the satisfaction of the Engineer without compensation or an extension of the completion date of the project. In addition, no compensation will be paid for casing remaining in place.

4. REMOVABLE CASING.

When the shaft extends above ground or through a body of water, the portion exposed above ground or through a body of water may be formed with suitable, removable casing except when permanent casing is specified. Removable casing shall be stripped from the shaft in a manner that will not damage the concrete. Casings can be removed when the concrete has attained a compressive strength of not less than 2500 psi {20 MPa} as determined from concrete cylinder breaks provided: curing of the concrete is continued for the full period in accordance with specifications and the shaft concrete is not exposed to salt water or moving water for seven days.

(c) PERMANENT CASINGS.

1. GENERAL.

Permanent casing shall be used when shown in the contract documents. The casing shall be continuous between top and bottom elevations prescribed in the plans. After installation is complete, the permanent casing shall be cut off at the prescribed elevation and the shaft completed by installing necessary reinforcing steel and concrete in the casing.

Exterior surfaces of permanent casings shall be cleaned and coated with the prime coat only of a System 1A Coating in accordance with the requirements given in Section 521 and as shown on the plans. The exterior surfaces shall be coated prior to the installation of the casings. After the installation of the casings, all damage to the coated surfaces of the casings exposed to the air shall be repaired by a repeated application of the same prime coat. When not shown in the contract documents, permanent casing may be used if determined to be necessary by the Engineer and if approved by the Bridge Engineer.

2. MULTIPLE CASINGS.

In cases where special temporary casings are shown on the plans or authorized in writing by the Engineer, the Contractor shall maintain alignment of both the temporary outer and permanent inner casing, and a positive, watertight seal between the two casings during excavation and concreting operations.

506.05 Use of Slurry.

(a) GENERAL.

Slurries shall have a mineral grain size that will remain in suspension and sufficient viscosity and gel characteristics to transport excavated material to a suitable screening system. The percentage and specific gravity of the material used to make the suspension shall be sufficient to maintain stability of the excavation and allow proper concrete placement.

(b) MIXING AND STORAGE.

The mineral slurry shall be premixed thoroughly with clean fresh water and adequate time allotted for hydration prior to introduction into the shaft excavation. Slurry tanks of adequate capacity will be required for slurry circulation, storage, and treatment. Excavated slurry pits will not be allowed in lieu of slurry tanks without the written permission of the Engineer.

(c) DESANDING.

Desanding equipment shall be provided by the Contractor as necessary to control slurry sand content at less than 4 percent by volume at any point in the borehole. Desanding will not be required for setting temporary casing, sign post, or lighting mast foundations unless required by the plans or special provisions.

(d) REQUIRED FLUID LEVEL.

1. GENERAL.

During construction, the level of the slurry shall be maintained at a height sufficient to prevent caving of the hole. In the event of a sudden significant loss of slurry in the hole, the construction of that foundation shall be stopped until methods to stop slurry loss or an alternate construction procedure have been approved by the Engineer.

2. REQUIRED HEAD.

Mineral slurry in a shaft excavation shall be maintained at a level not less than 4 feet {1.2 m} above the highest expected static water surface along the depth of the shaft. If at any time the Engineer determines the slurry construction method fails to produce the desired final results, the Contractor shall discontinue this method and propose an alternate method for approval of the Engineer.

(e) CONTROL OF SLURRY.

    1. SETUP PREVENTION.

        The Contractor shall take all steps necessary to prevent the slurry from "setting up" in the shaft. Such methods may include but are not limited to: agitation, circulation and/or adjusting the properties of the slurry.

    2. CONTROL TESTING.

        Control tests using suitable apparatus shall be carried out on the mineral slurry by the Contractor to determine density, viscosity and pH. An acceptable range of values for these physical properties is shown in the following table:

MINERAL SLURRY (Sodium Bentonite or Attapulgite in Fresh Water) Acceptable Range of Values			
Property (Units)	At Time of Slurry Introduction	In Hole at Time of Concreting	Test Method
Density (pounds per cubic foot) {kg/m^3}	64.3** - 69.1** {1030** - 1110**}	64.3** - 75.0** {1030** - 1200**}	Density Balance
Viscosity (seconds / quart) {seconds / liter}	28 - 45 {30 - 48}	28 - 45 {30 - 48}	Marsh Cone
pH	8-11	8-11	pH paper, pH meter
** Increase by 2 pounds per cubic foot {32 kg/m^3} in salt water			
a. Tests should be performed when the slurry temperature is above 39 °F.			
b. If desanding is required, sand content shall not exceed 4 percent (by volume) at any point in the bore hole as determined by the American Petroleum Institute sand content test.			

(f) TESTING OF SLURRY.

    1. FREQUENCY.

        Tests to determine density, viscosity and pH value shall be done during the shaft excavation to establish a consistent working pattern. A minimum of four sets of tests shall be made during the first 8 hours of slurry use. When the results show consistent behavior the testing frequency may be decreased to one set every four hours of slurry use.

    2. TEST REPORTS.

        Reports of all tests required above, signed by an authorized representative of the Contractor, shall be furnished to the Engineer on completion of each drilled shaft.

(g) DISPOSAL.

    Disposal of all slurry shall be done off site by the Contractor.

506.06 Excavation Measurement and Cleaning.

    (a) GENERAL.

        The Contractor shall provide equipment and personnel for checking the dimensions and alignment of each permanent shaft excavation. The dimensions, depth and alignment shall be determined under the direction and to the satisfaction of the Engineer after final cleaning.

    (b) CLEANING.

        Unless otherwise stated in the contract, a minimum of 50 percent of the base of each shaft will have less than 1/2 inch {13 mm} of sediment at the time of concrete placement. The maximum depth of sediment or any debris at any place on the base of the shaft shall not exceed 1.5 inches {40 mm}. Shaft cleanliness will be determined by visual inspection for dry shafts. For wet shafts the bottom of the shaft shall be sounded with an airlift pipe, a tape with a heavy weight {mass} attached to the end of the tape or other means acceptable to the Engineer. In addition, for dry excavations the maximum depth of water covering the bottom of the excavation shall not exceed 3 inches {75 mm} prior to concrete pour.

506.07 Reinforcing Steel Construction and Placement.

(a) GENERAL.

The reinforcing steel cage, consisting of longitudinal and transverse bars, ties, cage stiffeners, spacers, centralizers, and other necessary appurtenances, shall be completely assembled and placed as a unit immediately after the shaft excavation is inspected and accepted, and prior to concrete placement. The reinforcing steel in the shaft shall be securely tied and supported so that the reinforcing steel will remain within allowable tolerances given in Subarticle 506.11(c) of this Specification.

(b) SPACERS.

1.   Concrete spacers or other approved noncorrosive spacing devices shall be used at sufficient intervals near the bottom, and at intervals not exceeding 10 feet up the shaft, to insure concentric spacing for the entire cage length.

2.   Spacers shall be constructed of approved material equal in quality and durability to the concrete specified for the shaft. The spacers shall be of adequate dimension to insure the proper annular space between the outside of the reinforcing cage and the side of the excavated hole and/or permanent casing as detailed on the plans or proposed in the installation plan. If not detailed on the plans, a minimum 4 inch {100 mm} annular space will be required.

(c) CAGE SUPPORTS.

Cylindrical concrete feet (bottom supports) shall be provided to insure that the bottom of the cage is maintained at the proper distance above the base as specified by the project plans.

(d) CAGE EXTENSION.

If the drilled shaft excavation is extended to an elevation lower than the plan bottom elevation, reinforcing cage length shall also be extended by the same amount. Cages may be extended at the plan bottom elevation by lap splicing additional longitudinal bars, per planned cage requirements, of sufficient length to provide a compression splice, 4.17 feet {1270 mm} in length, plus the required extension. Hoops for the extension shall be spaced the same as shown for other hoops. Any additional splices of the cage above the plan bottom elevation and not shown on the plans, must have prior approval of the Bridge Engineer. Stiffeners, spacers and other appurtenances shall also be extended as required.

506.08 Concrete Placement Requirements.

(a) GENERAL.

Concrete used for drilled shaft construction shall meet the requirements of Subarticle 506.02(b).

After the reinforcing steel has been placed and before the concrete is ordered, the bottom of the drilled shaft must be resounded to verify cleanliness.

(b) CONCRETE PLACEMENT TIME LIMITATIONS.

1. GENERAL.

Concrete shall be placed as soon as possible after the reinforcing steel has been placed and the bottom of the shaft has been resounded. The concrete placement shall be continuous from the bottom to the top elevation of the shaft.

The elapsed time from the beginning of concrete placement in the shaft to the completion of placement shall not exceed 2 hours except as allowed by the Engineer. The Engineer may allow the concrete placement time to exceed 2 hours if the Contractor adequately demonstrates that the slump of the concrete will not be less than 4 inches {100 mm} during the entire time of concrete placement.

2. SLUMP LOSS/TIME RELATIONSHIP FOR CLASS DS1, DS2 AND DS3 CONCRETE.

a. General.

The Contractor may choose either a laboratory test or a field test to demonstrate the slump loss/time relationship. Adjustments to chemical admixture dosages will be allowed for the sole purpose of extending the time of concrete placement provided that the admixtures are included in

the approved concrete mix design. A new slump loss test will be required if changes are made to the concrete mix, including adjustments to chemical admixtures.

b. Laboratory Test.

The Contractor shall demonstrate by trial mix and slump loss tests that the slump of the concrete will not be less than 4 inches {100 mm} during the longer placement time. These tests shall be conducted by an independent testing laboratory, approved by the Department as per ALDOT-405, and in the presence of a Department representative. The slump loss tests shall be performed at intervals not to exceed 30 minutes and shall be made from a trial mix proportioned from the approved concrete mix design. The temperature of the trial mix shall be kept at a level representative of construction site conditions.

c. Field Test.

The Contractor shall demonstrate by construction site slump loss tests that the slump of the concrete will not be less than 4 inches {100 mm} during the longer placement time. The slump loss tests shall be performed at intervals not to exceed 30 minutes and shall be made from the first batch of concrete that is placed in a trial drilled shaft. The concrete used for these slump loss tests shall be sampled at the trial drilled shaft site and shall be kept covered during testing. If a trial shaft is not required then a field test may be performed at the construction site prior to the beginning of the work. The slump test shall be performed by the contractor's Concrete Technician, certified by the Department as per ALDOT-405, in the presence of a Department representative.

(c) PLACEMENT THROUGH SLURRY AND/OR ENCASED EXCAVATIONS.

1. GENERAL.

The Contractor shall insure that a heavily contaminated slurry suspension, which could impair the free flow of concrete, has not accumulated in the bottom of the shaft.

2. REQUIRED SLURRY SAMPLING.

Prior to placing concrete in a slurry filled shaft excavation, the Contractor shall take slurry samples using a sampling tool. Slurry samples shall be extracted from the base of the shaft and at intervals not exceeding 10 feet {3 m} up the shaft, until two consecutive samples produce acceptable values for density, viscosity, pH, and sand content as noted in Subarticle 506.05(c) and Item 506.05(e)2, respectively.

3. UNACCEPTABLE SAMPLING RESULTS.

When any slurry samples are found to be unacceptable, the Contractor shall take whatever action is necessary to bring the mineral slurry within specification requirements. Concrete shall not be poured until resampling and testing results produce acceptable values.

4. REQUIRED CONCRETE LEVEL DURING PLACEMENT.

The level of fresh concrete placed into a casing shall be a minimum of 5 feet {1.5 m} above either the hydrostatic water level or the level of drilling fluid whichever is higher. As a temporary casing is withdrawn, care shall be exercised to maintain an adequate level of concrete within the casing so that fluid trapped behind the casing is displaced upward and discharged at the ground surface without contaminating or displacing the shaft concrete.

506.09 Concrete Placement Methods.

(a) GENERAL.

If a method of concrete placement has not been specifically identified in the contract documents, the Contractor may use any of the placement methods described hereafter. If a concrete pump is used to move the concrete to the drilled shaft, a standby pump shall be immediately available to pump the concrete if there is a pump failure. Details pertaining to compliance with this specification shall be presented as part of the Contractors "Installation Plan" as outlined in Item 506.03(a)2.

Concrete placement shall continue after the shaft excavation is full until good quality concrete is evident at the top of the shaft. Any overflow of concrete at the top of the shaft shall be removed to maintain a uniform appearance and the proper dimensions of the shaft.

(b) FREE FALL PLACEMENT.

1. GENERAL.

The free fall placement of concrete shall only be permitted in dry vertical shafts where the clear opening (inside the reinforcing cage) is not less than 24 inches {610 mm} in diameter. The height of free fall placement shall not exceed 75 feet {22 m}. Concrete placed by free fall shall fall directly to the placement location without contacting either the reinforcing cage or the shaft sidewall.

The Engineer will observe the falling of the concrete within the shaft. If the concrete strikes the reinforcing cage or sidewall, or if there is excessive spatter from the impact of the falling concrete, the Contractor shall reduce the rate of concrete placement, reduce the height of free fall or provide a drop chute for concrete placement as directed by the Engineer.

2. DROP CHUTE REQUIREMENTS.

a. General.

Drop chutes shall consist of a smooth tube of either one piece construction or sections which can be added and removed. Concrete may be placed through either a hopper at the top of the tube or side openings as the drop chute is retrieved during concrete placement.

b. Chute Support.

The drop chute shall be supported so that the free fall of the concrete measured from the bottom of the chute to the point of deposition is less than 75 feet {22 m}. If concrete placement causes the shaft excavation to cave or slough, or if the concrete strikes the rebar cage or sidewall, the Contractor shall reduce the height of free fall and/or reduce the rate of concrete flow into the excavation.

3. DISQUALIFICATION OF FREE FALL METHOD.

If in the opinion of the Engineer, placement cannot be satisfactorily accomplished by the free fall and drop chute method, the Contractor shall change to either tremie or pumping methods to accomplish the pour.

(c) TREMIE CONCRETE PLACEMENT.

Tremies may be used for concrete placement in either wet or dry holes.

1. TREMIE REQUIREMENTS.

a. General.

Tremies shall consist of a tube of sufficient length, weight {mass}, and diameter to discharge concrete at the shaft base elevation. The tremie shall not contain aluminum parts which will have contact with the concrete. The tremies inside diameter shall be at least 6 times the maximum size of aggregate used in the concrete mix but shall not be less than 10 inches {250 mm}.

b. Tremie Tube Wall.

Inside and outside surfaces of the tremie shall be clean and smooth to permit both flow of concrete and unimpeded withdrawal during concreting. The wall thickness of the tremie shall be adequate to prevent crimping or sharp bends which restrict concrete placement.

c. Concrete Placement.

The tremie used for wet concrete placement shall be watertight. Underwater placement shall not begin until the tremie is placed to the shaft base elevation. Valves, bottom plates or plugs may be used to insure concrete discharge begins within one tremie diameter of the base. Plugs shall either be removed from the excavation or be made of a material which will not cause a defect in the shaft if not removed. The discharge end of the tremie shall be constructed to permit the free radial flow of concrete during placement operations.

2. PLACEMENT REQUIREMENTS.

a. General.

The tremie discharge end shall be immersed at least 5 feet {1.5 m} in concrete at all times after starting the flow of concrete. The flow of the concrete shall be continuous. The concrete in the tremie shall be maintained at a positive pressure differential at all times to prevent water or slurry intrusion into the shaft concrete.

b. Defective Shafts.

If at any time during the concrete pour, the tremie line orifice is removed from the fluid concrete column and discharges concrete above the rising concrete level, the shaft shall be considered defective. In such case, the Contractor shall either:

- remove the reinforcing cage and concrete, complete any necessary sidewall removal directed by the Engineer, and repour the shaft or,

- the tremie shall be replugged, recharged with concrete and inserted a minimum of 5 feet {1.5 m} below the existing top level of concrete prior to continuing the pour. The contractor shall be responsible for correcting any defect caused by this procedure without additional compensation.

All costs for replacement of defective shaft concrete shall be the responsibility of the Contractor.

(d) PUMPED CONCRETE PLACEMENT.

Concrete pumps and lines may be used for concrete placement in either wet or dry excavations.

1. EQUIPMENT REQUIREMENTS.

Pump lines shall have a minimum diameter of 4 inches {100 mm} and shall be constructed with watertight joints. Except as modified herein, requirements pertaining to tremie lines as stated in Item 506.09(c)1, also apply to pump lines and their use. The concrete pump unit shall have sufficient power to insure continuous placement of concrete under all foreseeable placement conditions.

2. PLACEMENT REQUIREMENTS.

a. Discharge Orifice Location and Pressure.

The discharge orifice shall remain at least 5 feet {1.5 m} below the surface of the fluid concrete. When lifting the pump line during concreting, the Contractor shall temporarily reduce the line pressure until the orifice has been repositioned at a higher level in the excavation.

b. Defective Shafts.

If at any time during the concrete pour the pump line orifice is removed from the fluid concrete column and discharges concrete above the rising concrete level, the shaft shall be considered defective. In such case, the Contractor shall remove the reinforcing cage and concrete, complete any necessary sidewall removal directed by the Engineer, and repour the shaft. All costs for replacement of defective shaft concrete shall be the responsibility of the Contractor.

506.10 Testing Requirements For Drilled Shafts.

(a) CROSSHOLE SONIC LOGGING OF DRILLED SHAFTS.

1. GENERAL REQUIREMENTS.

The nondestructive testing method called Crosshole Sonic Logging (CSL) shall be used on all production and trial drilled shafts (a) when constructed with the placement of concrete underwater or through slurry, (b) when required by special note on the plans, (c) when a full length temporary casing is used to prevent water from entering the shaft, or (d) when determined to be necessary by the Engineer. The testing shall not be conducted until forty-eight hours after the placement of all concrete in a shaft and must be completed within 20 calendar days after placement.

The CSL tests shall be conducted by an experienced independent testing consultant approved by the Engineer prior to testing.

The CSL tests measure the time it takes for an ultrasonic pulse to travel from a signal source in one access tube to a receiver in another access tube. In uniform, good quality concrete, the travel time between equi-distant tubes will be relatively constant and correspond to a reasonable concrete pulse velocity from the bottom to the top of the foundation. In uniform, good quality concrete, the CSL test will also produce records with good signal amplitude and energy. Longer travel times and lower amplitude/energy signals indicate the presence of irregularities such as poor quality concrete, void, honeycomb and soil intrusions. The signal will be completely lost by the receiver and CSL recording system for the more severe defects such as voids and soil intrusions.

2. PREPARATION FOR TESTING.

A number of tubes shall be installed in each shaft to permit access for CSL. The number of tubes installed will depend on the diameter of the shaft as specified below:

Shaft Diameter D	Minimum Number of Tubes
D ≤ 4.5 feet {1372 mm}	4
4.5 feet {1372 mm} < D ≤ 5.5 feet {1676 mm}	5
5.5 feet {1676 mm} < D ≤ 6.5 feet {1981 mm}	6
6.5 feet {1981 mm} < D ≤ 7.5 feet {2286 mm}	7
7.5 feet {2286 mm} < D ≤ 8.5 feet {2591 mm}	8
8.5 feet {2591 mm} < D ≤ 9.0 feet {2743 mm}	9
9.0 feet {2743 mm} < D < 10.0 feet {3048 mm}	10
10.0 feet {3048 mm} < D < 11.0 feet {3353 mm}	11
11.0 feet {3353 mm} < D < 12.0 feet {3658 mm}	12

The tubes shall have a 1.5 inch {40 mm} inside diameter and shall be schedule 40 steel pipe. The pipes shall have a round, regular internal diameter free of defects or obstructions, including any at pipe joints, in order to permit the free, unobstructed passage of a 1.3 inch {30 mm} diameter source and receiver probes. The tubes shall be watertight and free from corrosion with clean internal and external faces to ensure passage of the probes and a good bond between the concrete and the tubes.

The pipes shall each be fitted with a water tight shoe on the bottom and a removable cap on the top. The pipes shall be securely attached to the interior of the reinforcement cage with a minimum cover of 4 inches {100 mm}. The tubes shall be installed in each shaft in a regular, symmetric pattern such that each tube is equally spaced from the others around the perimeter of the cage. The Contractor shall submit to the testing organization his selection of tube size, along with his proposed method to install the tubes, prior to construction. The tubes shall be as near to parallel as possible. The tubes shall extend from 6 inches {150 mm} above the shaft bottoms to at least 3 feet {1 m} above the shaft tops. If the shaft top is sub-surface, the tubes shall extend at least 2 feet {600 mm} above the ground surface. Any joints required to achieve full length tubes shall be made watertight. Care shall be taken during reinforcement installation operations in the drilled shaft hole so as not to damage the tubes. As the cage is being lowered into the shaft, the tubes shall be checked to assure that they are vertical and parallel and that all connections are water tight. After placement of the reinforcement cage, the tubes shall be filled with clean water as soon as possible. After the tubes are filled with water, the tube tops shall be capped or sealed to keep debris out of the tubes prior to concrete placement.

The pipe caps or plugs shall not be removed until the concrete in the shaft has set. Care shall be exercised in the removal of caps or plugs from the pipes after installation so as not to apply excess torque, hammering, or other stresses which could break the bond between the tubes and the concrete.

3. TYPICAL CSL TEST EQUIPMENT.

Typical CSL test equipment consists of the following components:

- A microprocessor based CSL system for display of individual CSL records, analog-digital conversion and recording of CSL data, analysis of receiver responses and printing of CSL logs.

- Ultrasonic source and receiver probes for 1.5 or 2 inch {40 or 50 mm} ID pipe, as appropriate.

- An ultrasonic voltage pulser to excite the source with a synchronized triggering system to start the recording system.

- A depth measurement device to determine and record depths.

- Appropriate filter/amplification and cable systems for CSL testing.

4. CSL LOGGING PROCEDURES.

Before the placement of concrete, a minimum of one tube per shaft shall be plumbed and the tube length recorded, including a notation of the stickup of the tubes above the shaft tops. Information on the shaft bottom and top elevations and/or length, along with construction dates shall

be provided to the Engineer and the approved testing organization before the CSL tests are performed. CSL tests shall be conducted between pairs of tubes. The approved testing organization shall test two principle diagonals through the center and between each tube pair around the perimeter of all tested shafts. Additional logs shall be conducted at no additional cost in the event anomalies are detected.

The CSL tests shall be carried out with the source and receiver probes in the same horizontal plane unless test results indicate potential defects in which case the questionable zone may be further evaluated with angled tests (source and receiver vertically offset in the tubes). CSL measurements shall be made at depth intervals of 0.2 feet {60 mm} or less, and shall be done from the bottom of the tubes to the top of each shaft. The probes shall be pulled simultaneously, starting from the bottoms of the tubes, over a depth measuring device. Any slack shall be removed from the cables prior to pulling to provide for accurate depth measurements of the CSL records. Any defects indicated by longer pulse arrival times and significantly lower amplitude/energy signals shall be reported to the Engineer and further tests shall be conducted as required to evaluate the extent of such defects. Additional NDT methods which may be used to evaluate possible defects include Singlehole Sonic Logging, Gamma-Gamma Nuclear Density Logging, and/or Surface Sonic Echo and Impulse Response tests.

5. CSL TESTING RESULTS.

The CSL results shall be presented to the Engineer in a report. This report shall include recommendations as to the acceptability, unacceptability, soundness, etc., of the drilled shaft. The report shall be checked, stamped approved, and signed by a Professional Engineer licensed by the Alabama Board of Licensure for Professional Engineers. This Professional Engineer shall not be an employee of the ALDOT. The report shall be submitted directly to the Materials and Tests Engineer with a copy to the Project Engineer. The test results shall include CSL logs with analyses of:
- Initial pulse arrival time versus depth
- Pulse energy/amplitude versus depth

A CSL log shall be presented for each tube pair tested with any defect zones indicated on the logs and discussed in the test report as appropriate.

6. EVALUATION OF CSL TEST RESULTS.

The Engineer will evaluate the CSL test results and determine whether or not the drilled shaft construction is acceptable. This evaluation will be completed within 14 calendar days of the date of receipt of the report by the Materials & Tests Engineer.

If the Engineer determines that the drilled shaft is acceptable, the CSL tubes shall be dewatered and grouted. The grout shall be of the same strength or higher than the strength of the concrete used in the original drilled shaft. The contractor may use any of the grout mixes listed in Table 1 of Item 453.03(b)2. with the exception that calcium chloride will not be allowed. The contractor may submit another design mix for approval.

If the Engineer determines that the drilled shaft is unacceptable, the shaft shall be cored in accordance with the requirements given in Subarticle 506.10(b) to allow further evaluation of the shaft. Cores shall be taken without additional compensation unless the testing of the cores indicates that the concrete in the shaft meets all specification requirements. If the testing of the cores indicates that the concrete meets specification requirements, the cost of the coring will be paid for as Extra Work.

(b) CORE DRILLING OF DRILLED SHAFT CONCRETE.

Production or trial drilled shafts that are determined to be unacceptable by the CSL tests may be cored to determine the quality of the shaft. The required number and depth of cores will be determined by the Engineer.

Because it is necessary to obtain a high percentage of core recovery for visual inspection and compressive strength testing, the core bit used for core drilling shall be warranted by the manufacturer as being capable of coring the concrete as strong as could possibly be present in the shaft. A new bit or new core barrel will be required at any time the Engineer determines that the equipment may not be capable of obtaining good quality cores. The minimum diameter of the cores shall be 3.0 inches {76 mm}.

An accurate log of cores shall be kept and the cores shall be placed in a crate and properly marked showing the shaft depth at each interval of core recovery. The cores along with three copies of the coring log shall be transported to the ALDOT Bureau of Materials and Tests, Montgomery, Alabama, for inspection.

Construction shall not proceed above a drilled shaft until the quality of the shaft, as represented by the core samples, is determined to be acceptable and notification to continue construction is given by the ALDOT Construction Engineer.

If the Engineer determines that the drilled shaft is acceptable, the core holes and the CSL tubes shall be dewatered and grouted. The grout shall be of the same strength or higher than the strength of the concrete used in the original drilled shaft. The contractor may use any of the grout mixes listed in Table 1 of Item 453.03(b)2. with the exception that calcium chloride will not be allowed. The contractor may submit another grout design mix for approval.

If the quality of the drilled shaft is determined to be unacceptable then the Contractor shall construct another foundation to carry the load that will be placed on the shaft or perform corrective work as required by the Department. This foundation or the corrective work shall be constructed without compensation from the Department. The details of the replacement foundation shall be submitted in accordance with the requirements given in Article 105.02 for Working Drawings.

506.11 Drilled Shaft Construction Tolerances.

The following construction tolerances apply to drilled shafts unless otherwise stated in the contract documents. Drilled shaft excavations and completed shafts not constructed within the required tolerances are unacceptable. The Contractor shall correct all unacceptable shaft excavations and completed shafts to the satisfaction of the Engineer. Materials and work necessary to complete corrections for out of tolerance drilled shaft excavations and/or completed shafts, including engineering analysis and redesign, shall be furnished without either cost to the State or an extension of the contract time of the project.

(a) GENERAL LOCATION.

The drilled shaft shall be within 3 inches {75 mm} of plan position in the horizontal plane at the elevation of the top of the shaft.

(b) VERTICAL ALIGNMENT.

The vertical alignment of a shaft excavation shall not vary from the plan alignment by more than 1/4 inch per foot {20 mm/m} of depth. The alignment of a battered shaft excavation shall not vary by more than 1/2 inch per foot {40 mm/m} of depth from the prescribed batter.

(c) REINFORCING STEEL CAGE.

The spacers for the reinforcing cage shall have a tolerance of minus 1 inch {25 mm} from the required spacing shown on the plans.

The reinforcing steel cage shall be within 1 inch {25 mm} of plan position in the horizontal plane at the elevation of the top of the shaft.

After all the concrete is placed, the top of the reinforcing steel cage shall be no more than 6 inches {150 mm} above and no more than 3 inches {75 mm} below plan position.

(d) CASINGS.

All casing diameters shown on the plans refer to OD (outside diameter) dimensions. Casing shall be clean, round, straight and free of weld breaks and/or holes that would permit passage of water or wet concrete. When approved by the Engineer, the Contractor may elect to provide a casing larger in diameter than shown in the plans. No payment will be made for additional construction materials used in accommodating the Contractor's request for a larger casing diameter.

(e) SHAFT SOCKET.

The diameter of an excavated socket shall have a tolerance of minus 2 inches {50 mm} from the plan diameter.

(f) TOP ELEVATION OF SHAFTS.

The top elevation of the shaft shall have a tolerance of plus 1 inch {25 mm} or minus 3 inches {75 mm} from the plan top of shaft elevation.

(g) EXCAVATION EQUIPMENT AND METHODS.

Excavation equipment and methods shall be designed so that the completed shaft excavation will have a planar bottom. The cutting edges of excavation equipment shall be normal to the vertical axis of the equipment within a tolerance of +/- 3 % of the diameter.

506.12 Method of Measurement.

(a) DRILLED SHAFT EXCAVATION.

Drilled shaft excavation will be measured by the linear foot {meter} of excavated shaft.

(b) SPECIAL DRILLED SHAFT EXCAVATION.

Special drilled shaft excavation will be measured by the linear foot {meter} of excavated shaft.

(c) DRILLED SHAFT CONSTRUCTION.

Drilled shaft construction will be measured by the linear foot {meter} of shaft.

(d) EXPLORATION BELOW DRILLED SHAFT.

The exploratory drilling below the bottom of a drilled shaft will be measured by the linear foot {meter} of core hole.

(e) PERMANENT DRILLED SHAFT CASING.

Permanent drilled shaft casings will be measured by the linear foot {meter} of casing left in place.

(f) CROSSHOLE SONIC LOGGING (CSL).

Testing by the CSL method will be measured per each shaft tested.

506.13 Basis of Payment.

(a) DRILLED SHAFT EXCAVATION.

The linear foot {per meter} bid price shall be full compensation for all labor, materials and equipment required to complete and support the excavation. This shall also be full compensation for the utilization of slurry and temporary casings, for the disposal of all surplus excavated materials and for incidentals necessary to complete the work. No additional payment will be made for larger diameter or deeper excavations that are made by the choice of the Contractor.

(b) SPECIAL DRILLED SHAFT EXCAVATION.

The linear foot {per meter} bid price shall be full compensation for all labor, materials and special equipment required to complete and support the excavation. This shall also be full compensation for the removal of obstructions, the utilization of slurry and temporary casings, for the disposal of all surplus excavated materials and for incidentals necessary to complete the work. No additional payment will be made for larger diameter or deeper excavations that are made by the choice of the Contractor.

(c) DRILLED SHAFT CONSTRUCTION.

The linear foot {per meter} bid price shall be full compensation for all labor, materials, equipment and incidentals required for the construction of a shaft except for reinforcing steel which will be paid for under Item 502-A. No additional compensation will be made for larger diameter or deeper shafts that are constructed by the choice of the Contractor.

(d) EXPLORATION BELOW DRILLED SHAFT.

The linear foot {per meter} bid price shall be full compensation for all labor, materials, equipment and incidentals required for coring and sample retrieval.

(e) TRIAL DRILLED SHAFT.

Payment for a trial drilled shaft will be made under the appropriate production drilled shaft items of 506-A, B, C, F or G as they may apply. No separate payment will be made for cutting off the trial shaft or site restoration.

(f) PERMANENT DRILLED SHAFT CASING.

The linear foot {per meter} bid price shall be full compensation for all labor, materials, equipment and incidentals required for furnishing, painting and installing the casing. No payment will be made for cutoffs.

If there is no pay item in the contract for permanent casing then the casing will be paid for as extra work as outlined in Article 104.03, Extra Work.

(g) CROSSHOLE SONIC LOGGING.

The price bid for each shaft tested shall be full compensation for all labor, materials, equipment and incidentals necessary to perform the required test and furnish the Engineer with the test results. The bid price shall also include dewatering the tubes and filling the tubes with grout.

Where a drilled shaft consists of different shaft diameters, the price bid shall be full compensation for the sonic logging of the complete depth of the drilled shaft, regardless of differences in the diameter of the shaft. The shaft diameter shown in the pay item for sonic logging is for identification purposes and will be the smallest diameter portion of a drilled shaft.

(h) PAYMENT WILL BE MADE UNDER ITEM NO.:

506-A Drilled Shaft Excavation, _*_ Diameter - per linear foot {meter}
506-B Special Drilled Shaft Excavation, _*_ Diameter - per linear foot {meter}
506-C Drilled Shaft Construction, _*_ Diameter, Class _**_ Concrete - per linear foot {meter}
506-D Exploration Below Drilled Shaft - per linear foot {meter}
506-F Permanent Drilled Shaft Casing, _*_ Diameter - per linear foot {meter}
506-G Crosshole Sonic Logging, _*_ Diameter - per each
   *   Specify diameter of shaft in feet and inches {millimeters}.
   **   Specify either "DS1", "DS2", "DS3" or "HPDS".

Appendix C: Visual Stability Index (VSI) Manual

AUBURN ENGINEERING
SAMUEL GINN COLLEGE OF ENGINEERING

Guidelines for
Visual Stability Index (VSI)

A Self-Consolidating Concrete (SCC) Test
Based on ASTM C 1611, ASTM C 1611 Appendix and 2002 PCI Interim Guidelines

Created by *Auburn University* for use on Project: ALDOT 930-688S

Terminology:

Halo: an observed cement paste or mortar ring that has clearly separated from the coarse aggregate, around the outside circumference of concrete after flowing from the slump cone (a wet surface will cause a halo that can falsely identify segregation)

Spread: the distance of lateral flow of concrete during the slump-flow test

Stability: the ability of a concrete mixture to resist segregation of the paste from the aggregates

Viscosity: resistance of a material to flow under it's own weight

Materials:
- Slump-cone: same cone used for a standard slump test
- Base Plate: non-absorbent, smooth, rigid plate with a minimum diameter of 36 inches
- Strike-off rod: straight, flat, rigid, non-absorbent rod
- Level
- 5-gallon bucket

Procedure:

1. Perform the slump flow test (ASTM C 1611 - 07).
   - Clean base plate with a wet sponge
   - Ring out the sponge then wipe off the excess water on the base plate (it is very important that no standing water in on the base plate)
   - Wet the sponge again and dampen the slump-cone (if water is dripping off it is too wet and needs to dry)
   - Lay the slump-cone inverted in the center of the moistened surface of the base plate
   - Acquire your SCC concrete sample in a 5 gallon bucket and carefully pour the sample into the inverted cone at a constant rate until the cone is full
   - Use the strike-off rod to strike off the excess concrete from the inverted cone
   - Ring out the sponge again and wipe away any concrete that spilled on the base-plate
   - Using the handles on the slump-cone take 2 to 4 seconds to lift the cone in one steady upward motion
   - Once the concrete stops flowing the concrete should form a uniform concrete "patty" on the base-plate, measure the widest part of the patty and then measure perpendicular to the widest part, the average of these two measurements is the slump-flow of the concrete. **DO NOT CLEAN OFF CONCRETE YET!**

2. Observe the stability of the concrete patty
   - Look at the outside of the concrete patty. Is there a definite halo? If so, how much past the diameter of the patty?
   - Look at the surface of the patty. Is there shininess to the concrete (excess water)? Bleed water will cause a sheen on the concrete and or puddle on top of the patty
   - Is there any uncoated aggregate? Did the aggregate spread with the mortar or is there a pile of rocks in the center of the patty?
   - The following table is based on the ASTM C 1611 Appendix and shows the criteria for assigning a VSI to the concrete

| VSI Value	Criteria
0 = Highly Stable	No evidence of segregation or bleeding
1 = Stable	No evidence of segregation and slight bleeding observed as a sheen on the concrete patty
1.5 = Sufficiently Stable	Slight evidence of segregation, mortar halo less than 0.25" and/or small puddles on patty, but concrete has a consistent look over the entire patty
2 = Unstable	A slight mortar halo less than 0.5" and/or aggregate pile in the center of the concrete patty and/or water puddles on concrete patty
3 = Highly Unstable	Clearly segregating by evidence of a large mortar halo greater than 0.5" and/or a large aggregate pile in the center of the concrete patty

3. Assign a VSI value to the concrete patty (use the following pictures for reference)

- No mortar halo
- No puddles of water

VSI = 0
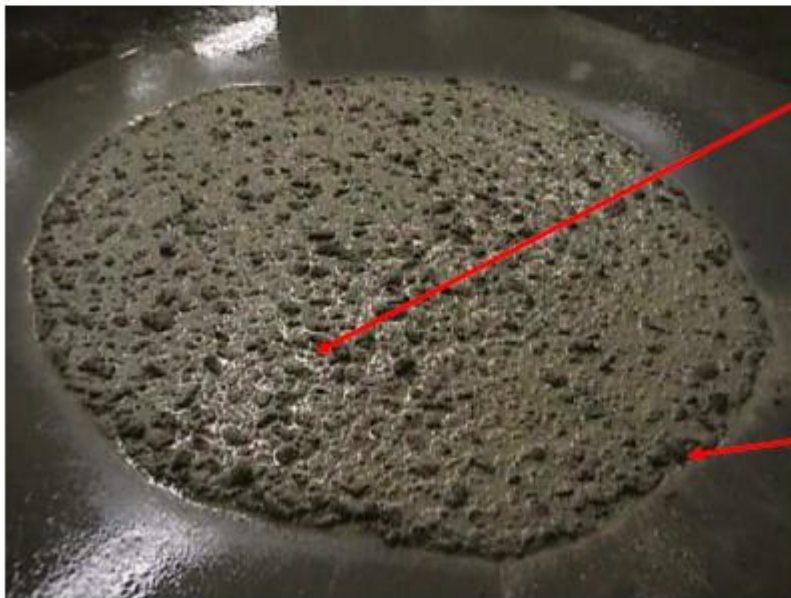
Sheen on the concrete

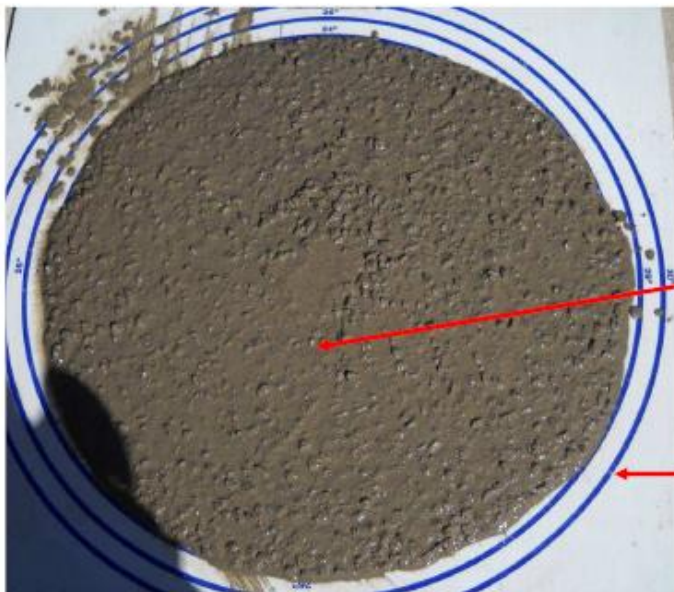Slight mortar halo that may have been caused by excess water on base plate

VSI = 1.0

255

Small mortar halo but very uniform concrete patty

VSI = 1.5

Sheen on the concrete

Small mortar halo

VSI = 1.5

Small water puddle in concrete patty

Small mortar halo

VSI = 1.5

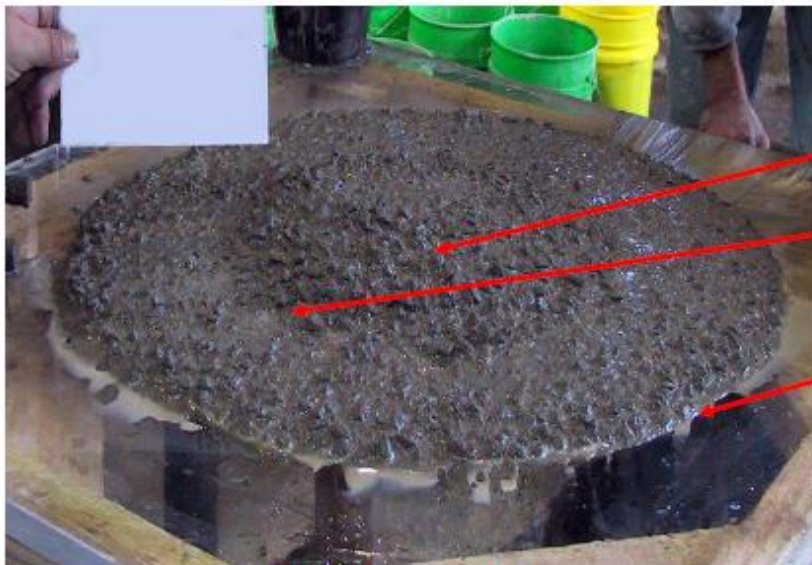- Very glossy finish

Greater than 0.25" mortar halo

VSI = 2.0

257

Water puddle in concrete patty

Greater than 0.25" mortar halo

VSI = 2.0

Pile of aggregate in the center of patty

Small puddles of water

Mortar halo at approximately 0.25"

VSI = 2.0

Water puddles in concrete patty

Pile of aggregate in the center of patty

Greater than 0.25" mortar halo

VSI = 2.0
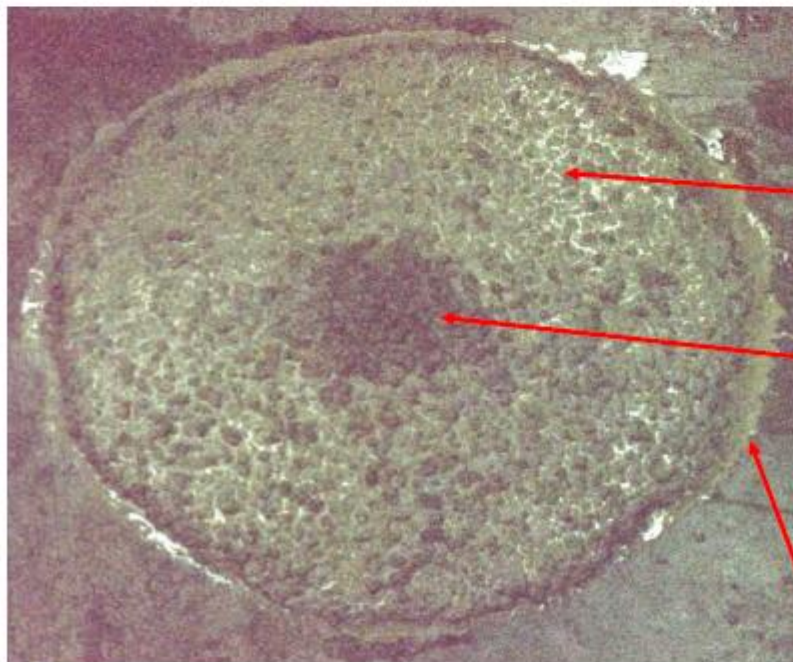
Water puddles in concrete patty

Pile of aggregate in the center of patty

Greater than 0.5" mortar halo

VSI = 3.0

References:

2002. *Interim Guidelines for the Use of Self-Compacting Concrete in Precast/Prestressed Concrete Institute Member Plants*. Precast/Prestressed Concrete Institute. PCI Publication Number D03016.

ASTM C 1611. 2007. Standard Test Method for Slump Flow of Self-Consolidating Conrete. *Annual Book of ASTM Standards* 2007. Section 04, V04.02: Concrete and Aggregates

Appendix D: Production Shaft Daily Logs

Construction Notes
By Phillip Gallet, Graduate Research Assistant, Auburn University
Last Updated on January 4, 2010

Project: BRF-0035(502)
Location: SR-35 at the Tennessee River in Scottsboro, Jackson County, AL
Construction Date: 8/28/09
Weather: Rainy
Shaft ID: Pier 7, Shaft 4
Depth: 42.5 ft
Concrete Amount: Approximately 113 yd^3
Time Started: 1:35pm
Time Finished: 7:24pm

Existing B.B. Comer Bridge

#4

North

Existing N. Bound Bridge

Remarks:
Concrete placement was scheduled to start between 8am and 12pm on August 29, 2009. Upon arrival at 8:30am Russo personnel were moving the steel reinforcement cage to the shaft location from the shore as shown in Figure 1.

Figure 1: Moving the steel reinforcement cage to the shaft location

Kirkpatrick received the call for concrete at approximately 12:30pm. The ready mix truck was batched and fresh concrete properties tested at 1:07pm. At the plant this truck had a slump flow of 21 in., a VSI of 0 [1], and a total air content of 4.6%. This truck arrived at the site for testing at 1:30pm with a slump flow of 21 in., a VSI of 0.5[1], a total air content of 4.4%, and a fresh concrete temperature of 81 °F. An example of the slump flow patty is shown in Figure 2.

Figure 2: Slump flow of concrete

The second truck left the plant with a slump flow of 21 in., but no other tests were conducted on this truck. Tests were conducted once for every 50 yd^3 of concrete.

---

[1] VSI stated here was conducted by Phillip Gallet.

The ready mix concrete trucks discharged the concrete into a pump truck that was located in the closed left-lane of the existing AL-35 northbound bridge. The pump line went from the pump truck down to a barge where it laid on the barge deck then curved up over the reinforcement cage and to a 90° bend into the shaft as shown in Figure 3.

Figure 3: Configuration of the pump line near the top of the tremie

With approximately 1 yd^3 left in Truck No. 3, 2 gallons of water were added to the truck to help break up the concrete clumps. Ricky Swancey (with BASF) informed the author that the concrete clumps were caused by mixing sequence issues at the plant. The plant was going to change the mixing sequence and try to fix the problem.

The tremie pipe was clogged for the first time at the beginning of Truck No. 4. This occurred when two other trucks were waiting on the bridge. To fix the clog, the tremie was pulled completely out of the water and beat with a hammer to dislodge the clogged concrete as shown in Figure 4. Before placing the tremie back into the shaft a foam plug was placed into its end.

Figure 4: A worker hammering the tremie to unclog it

263

The distance from the top of the concrete to the top of the steel casing was measured during construction as shown in Figure 5.

Figure 5: A worker measuring the distance from the top of the concrete to the top of the steel casing

At 3:45pm the tremie clogged for the second time. To fix the clog, the tremie was pulled completely out of the water and beat with a hammer to dislodge the clogged concrete. Before placing the tremie back into the shaft a foam plug was placed into the end of the tremie.

Clumps were witnessed in every truck poured. Most of the trucks were sent away with between 0.5 and 1 yd^3 left in the truck because of the clumps. Examples of some clumps are shown in Figure 6. The larger clumps were retained on the grate of the pump truck as shown in Figure 7.

Figure 6: Example of clumps in concrete

Figure 7: Example of a clump on the pump truck grate

Tremie clogged for the third time at 4:35pm. To fix the clog, the tremie was pulled completely out of the shaft and beat with a hammer to dislodge the clogged concrete. Before placing the tremie back into the shaft a foam plug was placed into the end of the tremie.

Truck No. 9 and No. 10 were sent away because they had a 30 in. slump flow, which exceeded the specification limit of 24 in.

The initial concrete order was for 81 yd^3, this was changed to 113 yd^3.

The fourth and final clog occurred at 6:10pm. After this clog the tremie was taken out and approximately half of it was removed. The clogging apparently occurred at the bottom of the tremie each time.

Placement was completed at 7:24 pm.

Construction Notes

By Phillip Gallet, Graduate Research Assistant, Auburn University
Last Updated on January 4, 2010

Project: BRF-0035(502)
Location: SR-35 at the Tennessee River in Scottsboro, Jackson County, AL
Construction Date: 8/28/09
Weather: Rainy
Shaft ID: Pier 7, Shaft 4
Depth: 42.5 ft
Concrete Amount: Approximately 113 yd^3
Time Started: 1:35pm
Time Finished: 7:24pm

Existing B.B. Comer Bridge

#4

North

Existing N. Bound Bridge

Remarks:

Concrete placement was scheduled to start between 8am and 12pm on August 29, 2009. Upon arrival at 8:30am Russo personnel were moving the steel reinforcement cage to the shaft location from the shore as shown in Figure 1.

Figure 1: Moving the steel reinforcement cage to the shaft location

Kirkpatrick received the call for concrete at approximately 12:30pm. The ready mix truck was batched and fresh concrete properties tested at 1:07pm. At the plant this truck had a slump flow of 21 in., a VSI of 0 [2], and a total air content of 4.6%. This truck arrived at the site for testing at 1:30pm with a slump flow of 21 in., a VSI of 0.5[1], a total air content of 4.4%, and a fresh concrete temperature of 81 °F. An example of the slump flow patty is shown in Figure 2.

Figure 2: Slump flow of concrete

The second truck left the plant with a slump flow of 21 in., but no other tests were conducted on this truck. Tests were conducted once for every 50 yd^3 of concrete.

---

[2] VSI stated here was conducted by Phillip Gallet.

The ready mix concrete trucks discharged the concrete into a pump truck that was located in the closed left-lane of the existing AL-35 northbound bridge. The pump line went from the pump truck down to a barge where it laid on the barge deck then curved up over the reinforcement cage and to a 90° bend into the shaft as shown in Figure 3.

Figure 3: Configuration of the pump line near the top of the tremie

With approximately 1 yd^3 left in Truck No. 3, 2 gallons of water were added to the truck to help break up the concrete clumps. Ricky Swancey (with BASF) informed the author that the concrete clumps were caused by mixing sequence issues at the plant. The plant was going to change the mixing sequence and try to fix the problem.

The tremie pipe was clogged for the first time at the beginning of Truck No. 4. This occurred when two other trucks were waiting on the bridge. To fix the clog, the tremie was pulled completely out of the water and beat with a hammer to dislodge the clogged concrete as shown in Figure 4. Before placing the tremie back into the shaft a foam plug was placed into its end.

Figure 4: A worker hammering the tremie to unclog it

268

The distance from the top of the concrete to the top of the steel casing was measured during construction as shown in Figure 5.

Figure 5: A worker measuring the distance from the top of the concrete to the top of the steel casing

At 3:45pm the tremie clogged for the second time. To fix the clog, the tremie was pulled completely out of the water and beat with a hammer to dislodge the clogged concrete. Before placing the tremie back into the shaft a foam plug was placed into the end of the tremie.

Clumps were witnessed in every truck poured. Most of the trucks were sent away with between 0.5 and 1 yd^3 left in the truck because of the clumps. Examples of some clumps are shown in Figure 6. The larger clumps were retained on the grate of the pump truck as shown in Figure 7.

Figure 6: Example of clumps in concrete

Figure 7: Example of a clump on the pump truck grate

Tremie clogged for the third time at 4:35pm. To fix the clog, the tremie was pulled completely out of the shaft and beat with a hammer to dislodge the clogged concrete. Before placing the tremie back into the shaft a foam plug was placed into the end of the tremie.

Truck No. 9 and No. 10 were sent away because they had a 30 in. slump flow, which exceeded the specification limit of 24 in.

The initial concrete order was for 81 yd^3, this was changed to 113 yd^3.

The fourth and final clog occurred at 6:10pm. After this clog the tremie was taken out and approximately half of it was removed. The clogging apparently occurred at the bottom of the tremie each time.

Placement was completed at 7:24 pm.

Construction Notes

By Phillip Gallet, Graduate Research Assistant, Auburn University
Last Updated on January 6, 2010

Project: BRF-0035(502)
Location: SR-35 at the Tennessee River in Scottsboro, Jackson County, AL
Construction Date: 9/28/09
Weather: Partly cloudy with the temperature in the high 70's
Shaft ID: Pier 7, Shaft 5
Shaft Depth: 41 ft
Concrete Amount: N/A
Time Started: 12:55pm
Time Cancelled: 1:45pm
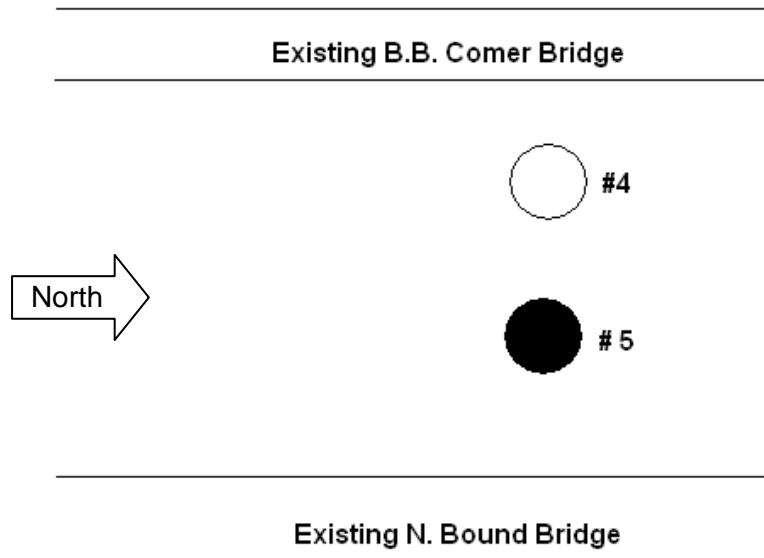
Remarks:

Due to the cancellation of the previous pour and the troubles that have occurred with both of the previous pours, much of the people in charge of this project arrived onsite. This included, but was not limited to the following:

- Harris Wilson and Paul Wilson; Russo
- Scott Overby and Lindy Blackburn; ALDOT
- Bo Canning, Ricky Meade and Tony Cornelius; Kirkpatrick Concrete
- Dr. Anton Schindler and Dr. Dan Brown; Auburn University

Concrete placement was scheduled to start between 9am and 12pm on September 28, 2009. Kilpatrick begins batching concrete at approximately 10:55am.

As conducted for the previous pour, the concrete was mixed into the truck without the superplasticizer, then after 30 revolutions the superplasticizer was added. Also, each truck was loaded with only 6 cu yds of concrete.

271

The ready mix truck was batched and fresh concrete properties tested. At the plant this truck had a slump flow of 27 in., a VSI of 2.0 [3], and a total air content of 3.8%. This slump flow exceeded specification. Therefore, this truck was held at the plant and retested after mixing for an extended time period. This truck was then retested and had a slump flow of 20 in., a VSI of 0.0, and a total air content of 5.5%. The patty from the first slump flow test is shown in Figure 1. The patty of the concrete after the waiting time is shown in Figure 2.

Figure 1: Failing slump flow patty at the plant
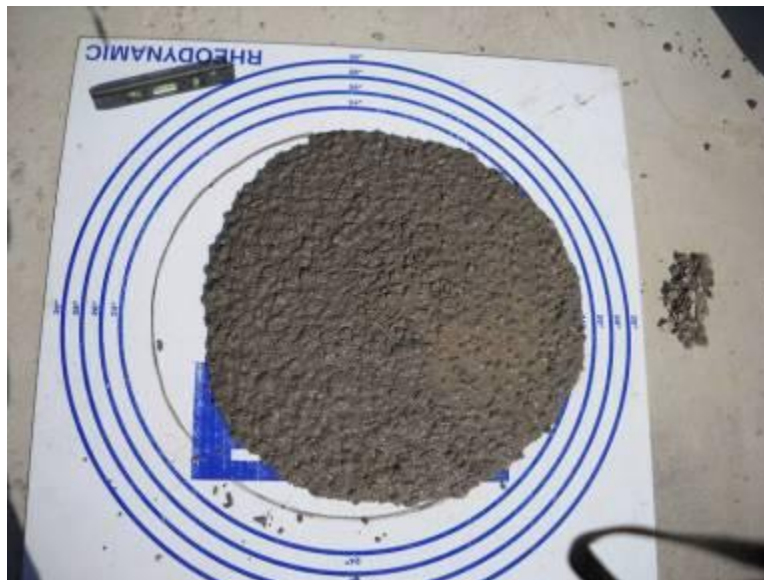
Figure 2: Slump flow of concrete after waiting period

This truck arrived at the site for testing shortly afterward with a slump flow of 22.5 in., a VSI of 0.5 [1], and a total air content of 6.0%. The slump flow patty is shown in Figure 3.

---

[3] VSI stated here was conducted by Phillip Gallet.

Figure 3: Slump flow of concrete at the site

Harris Wilson requested that concrete from this truck was also dispensed into a wheel barrel for a non-ASTM segregation test. To conduct this test, the wheel barrel sample was walked around the testing area to simulate vibration in the concrete. After the wheel barrel was walked around the concrete was poured onto the ground. This poured out concrete is shown in Figure 4.

Figure 4: Slump flow of concrete at the site

The ready mix concrete trucks discharged the concrete into 3 yd³ buckets that were moved using one of two different cranes located on barges on the water. A total of four buckets were utilized to help speed up the concrete pour. The buckets were filled and carried down to the barges until all four buckets were filled. The cranes then took turns picking up the buckets from their location on the barge and discharged them into a hopper located on the top of a 10 in. tremie pipe. Once all four buckets were used one crane was used to fill one bucket and continue the concrete pour.

The first truck began discharging concrete into the bucket at approximately 12:55pm. The tremie appears to have clogged at the conclusion of the first bucket. To fix this tremie clog, the second

bucket was attached to the hopper and both the bucket and hopper were raised to unclog the tremie. The hopper and tremie being raised is shown in Figure 5. As soon as the tremie was unclogged the bucket was discharged. This process was repeated for each bucket up to bucket #5.

Figure 5: Raising the hopper to unclog the tremie

At approximately 1:20pm, bucket #5 was discharged into the hopper. The tremie clogs again and there is a pause in the pour. At this point approximately 15 yd^3 of concrete have been poured into the shaft and no more filled buckets are lying on the barges.

At this point a meeting was held on the barge around the hopper to decide why the concrete is not flowing and what actions needed to occur. A picture of this meeting is shown in Figure 6.

Figure 6: Meeting to figure out why concrete is not flowing

Dr. Dan Brown determined that water was infiltrating the tremie. This was concluded by dropping pebbles into the hopper and listening for slashes in water. Once it was established that water was penetrating the tremie, it was decided to weld a steel plate to the bottom of the tremie so that the tremie may be filled with water and the leak location be localized.

While the tremie was removed for welding, a mudbucket was used to clean the concrete out of the hole. The mudbucket is shown in Figure 7.

Figure 7: Mudbucket used to clean the concrete out of the hole

After the hole was sufficiently cleaned, the tremie was lowered back into the hole and left there for approximately 8 minutes. After this time the tremie was removed and water was seen discharging from one of the tremie's gaskets. A picture of the water discharging is shown in Figure 8 and Figure 9

Figure 8: Water discharging from tremie

Figure 9: Close up of water discharging from gasket

The leaking tremie was to be fixed and another try at the concrete pour was to be held later in the week.

Construction Notes

By Phillip Gallet, Graduate Research Assistant, Auburn University
Last Updated on April 6, 2010

Project: BRF-0035(502)
Location: SR-35 at the Tennessee River in Scottsboro, Jackson County, AL
Construction Date: 9/29/09
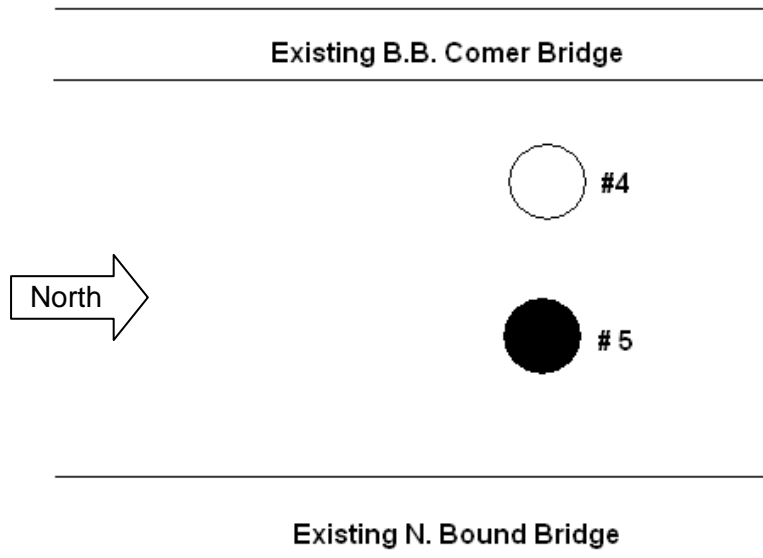**Weather:** No clouds, little humidity, temperatures in the 70's
Shaft ID: Pier 7, Shaft 5
Shaft Depth: 41 ft
Concrete Amount: 88 yds^3
Time Started: 11:45am
Time Finished: 2:30pm

Existing B.B. Comer Bridge

North

#4

5

Existing N. Bound Bridge

Remarks:

Concrete placement was scheduled to start between 9am and 12pm on September 29, 2009. Kilpatrick begins batching concrete at approximately 10:15am.

As conducted for the previous pour, the concrete was mixed into the truck without the superplasticizer, then after 30 revolutions the superplasticizer was added. Also, each truck was loaded with only 6 cu yds of concrete.

Cement balls were observed in the first truck batched requiring the first truck to be mixed more at a high revolution to break up the cement.

The first three trucks batched (including the one described above) had slump flows outside of the specified amount.
- Truck #1: Slump Flow: 16", Air content: 5.1%, VSI: 0.0[4]
- Truck #2: Slump Flow : 28", Air content: N/A, VSI: 2.0[1]
- Truck #3: Slump Flow: 16, Air content: N/A, VSI: 0.0[1]

---

[4] VSI stated here was conducted by Phillip Gallet.

Truck #2 was set aside to be tested after a short waiting time. At this time the author left the mixing plant to go to the job site to interview a Russo or Scott Bridge employee in order to see what testing methods were conducted to make sure the tremie leak was fixed. A picture of the slump flow patty from Truck #2 is shown in Figure 1.
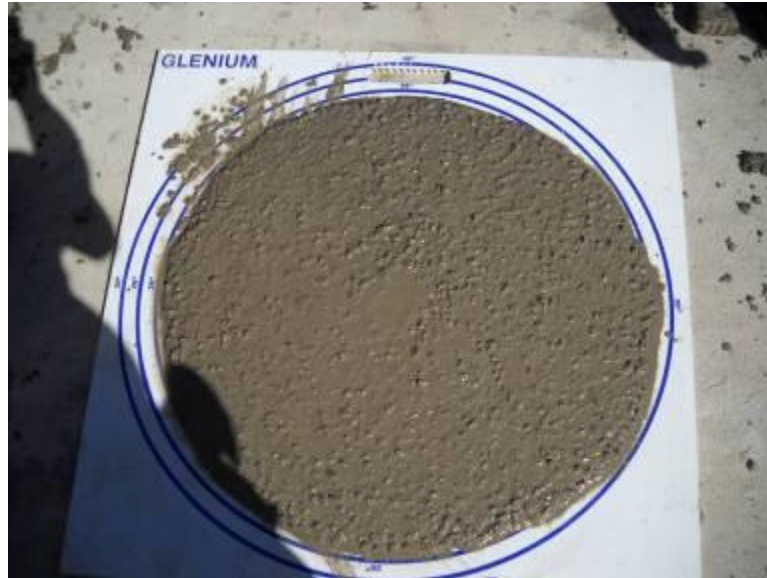
Figure 1: Truck #2 Slump Flow Patty

At the job site the author was informed that the tremie leak was fixed on the previous afternoon and was checked earlier in the day by welding a steel plate to the bottom of the tremie and submerging the empty tremie into the hole. The informant stated that when the tremie was taken out of the water no leaks were present.

The first truck arrived onsite at 11:00AM. Five gallon bucket samples were taken from the back of this truck upon arrival. The slump flow from this first truck was 15" and 17" taken from two separate buckets.

Five gallons of water was then added to the truck. After mixing the water the next slump flows were 17" and 17" from two separate buckets. More water was added to the concrete mixture. The resulting slump flow was 21". Being within the specifications this truck was sent to the bridge to begin filling the 3 yd^3 loading buckets.

To place the concrete the ready mix concrete trucks discharged the concrete into 3 yd^3 buckets that were moved using one of two different cranes located on barges on the water. A total of four buckets were utilized to help speed up the concrete pour. The buckets were filled and carried down to the barges until all four buckets were filled. The cranes then took turns picking up the buckets from their location on the barge and discharged them into a hopper located on the top of a 10 in. tremie pipe. Once all four buckets were used one crane was used to fill one bucket and continue the concrete pour.

The concrete placement from the filled buckets began at 11:45AM. At this time two full buckets were lowered above the open tremie hopper. The first bucket was discharged into the hopper. The concrete took time to push the foam plug through the tremie, but once the plug was pushed out the concrete began to flow freely down the tremie. Once the hopper was emptied the second bucket was moved into place and began filling the hopper as soon as possible.

At 12:10PM 6 buckets had been used to fill the shaft with no delays. The shaft was filled complete with 88 yds^3 at 2:20PM.

278

At no point during the concrete placement was the tremie lifted. The tremie was lifted at the conclusion of the concrete placement at 2:25PM. No problems were noticed during the removal of the tremie. The tremie being lifted out of the completed shaft is shown in Figure 2.

Figure 2: Tremie being lifted out of completed shaft

A slump test was conducted at the conclusion of the concrete pour. This test measured a concrete slump of 10"; much higher than the ALDOT specified 4" minimum.

The author spoke to Russo Corp. President, Harris Wilson, after the tremie was removed. Mr. Wilson made the following comments about the SCC concrete:
- There is no different on the surface between this mix and the standard Alabama drilled shaft mix
- This mix is better at not having to move the tremie
- After 40' on the standard drilled shaft mix, the tremie may have to be lifted whereas with this mix the tremie did not have to move
- A bad slump [flow] on the low end is not as scary with this mix
- There is a lot more "play" in the specified slump [flow] for this mix

Project: BRF-0035(502)
Location: SR-35 at the Tennessee River in Scottsboro, Jackson County, AL
Construction Date: 10/13/09
Weather: Overcast skies, temperatures in the 70's
Shaft ID: Pier 7, Shaft 3
Shaft Depth: 43 ft
Concrete Amount: 96 yds^3
Time Started: 1:15pm
Time Finished: 3:50pm
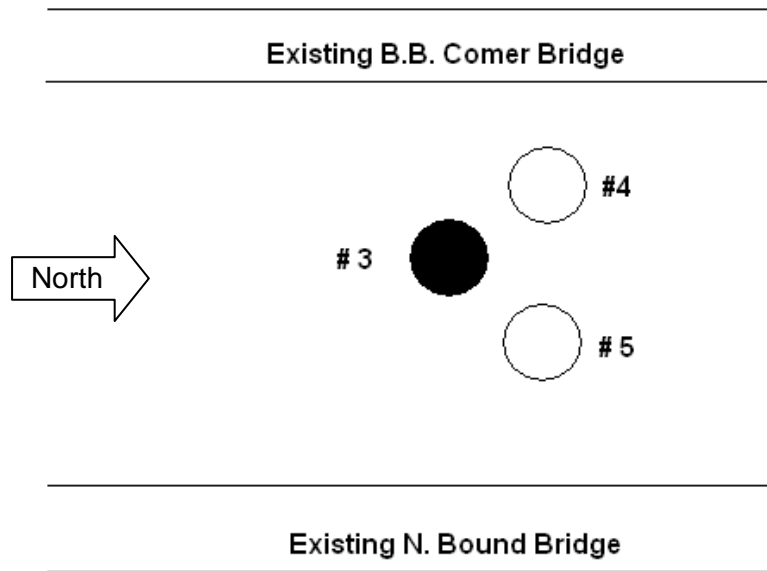
Remarks:

Kirkpatrick concrete received the call to place concrete at 11:57AM on October 13, 2009. As conducted for the previous pour, the concrete was mixed into the truck without the superplasticizer, then after 30 revolutions the superplasticizer was added. Also, each truck was loaded with only 6 yds^3 of concrete.

At 10:21AM the first truck filled at the batch plant was tested. This concrete had slump flow of 19.5". This concrete met specifications, but 1.75 hrs had passed since batching therefore this concrete could not be poured in the shafts and would be used somewhere else.

The second truck batched had a slump flow of 24". This being on the high end of the specifications, this truck was held for a short period of time. After this short time period this concrete had a slump flow of 18.5". Three gallons were added to the concrete and this truck was sent to the job site.

The third truck batched had a slump flow of 16". Five gallons were added, the truck was remixed and tested again. After adding the water the slump flow was 17.5". Five gallons was again added to the concrete to end up with a slump flow of 17.5". This truck was sent to the jobsite with this slump flow value.

At the jobsite the first truck arrived to the testing area and filled 5-gallon buckets for fresh concrete property testing and cylinder making. This concrete was taken directly off the back of the truck. The concrete properties were: slump flow = 23" and air content = 3.1%.

To place the concrete the ready mix concrete trucks discharged the concrete into 3 yd^3 buckets that were moved using one of two different cranes located on barges on the water. A total of four buckets were utilized to help speed up the concrete pour. The buckets were filled and carried down to the barges until all four buckets were filled. The cranes then took turns picking up the buckets from their location on the barge and discharged them into a hopper located on the top of a 10 in. tremie pipe. Once all four buckets were used one crane was used to fill one bucket and continue the concrete pour.

The concrete placement from the filled buckets began at 1:30PM. At this time two full buckets were lowered above the open tremie hopper. The first bucket was discharged into the hopper. The concrete took time to push the foam plug through the tremie, but once the plug was pushed out the concrete began to flow freely down the tremie. Once the hopper was emptied the second bucket was moved into place and began filling the hopper as soon as possible.

The entire concrete placement took 2 hours and 35 minutes from start to finish. No problems were observed during placement. The shaft was filled complete with 96 yds^3 at 3:45PM.

At no point during the concrete placement was the tremie lifted. The tremie was lifted at the conclusion of the concrete placement at 3:50PM. No problems were noticed during the removal of the tremie.

Construction Notes

By Phillip Gallet, Graduate Research Assistant, Auburn University
Last Updated on April 6, 2010

Project: BRF-0035(502)
Location: SR-35 at the Tennessee River in Scottsboro, Jackson County, AL
Construction Date: 10/21/09
Weather: Sunny, temperatures in the 60's
Shaft ID: Pier 7, Shaft 1
Shaft Depth: 33 ft
Concrete Amount: 69 yds^3
Time Started: 2:40pm
Time Finished: 4:20pm
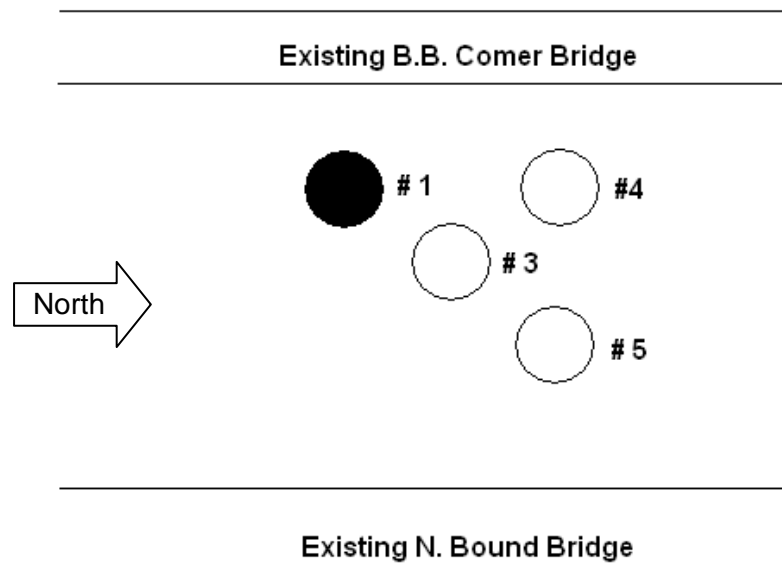
Remarks:

On this date, the author observed the installation of the reinforcement cage and the concrete placement operation from the barge. Upon arrival to the jobsite, the author got a boat ride to the barge to watch the steel reinforcement cage preparation. On the barge, the reinforcement cage was being spliced. A picture of the steel cage being spliced is shown in Figure 1.

Figure 1: Splicing the reinforcement cage

The steel cage was seven feet in diameter with 48 #11 reinforcement bars located around the cage and hoops located at approximately seven inch spacing. Figure 2 shows a close up of the cage confinement.

Figure 2: Close-up of reinforcement cage

The concrete trucks arrived and began filling up the three cubic yard buckets at 2:20PM. The trucks were again loaded with 6 yds^3 each. From ALDOT's report, the slump flow of the concrete within first truck was 21 inches. The only other slump flow taken from ALDOT was on the ninth truck and this concrete had a flow of 24 inches.

At 2:40PM, after the four buckets were filled, a single bucket was moved over the tremie hopper to begin placing the concrete. Just before the concrete placement a foam plug was pushed into the top of the hopper to prevent the concrete from mixing with the water on its way down the tremie pipe. Figure 3 shows a picture of the foam plug ("pig"). Figure 4 shows a bucket being dispensed into the hopper.

Figure 3: Foam plug to separate the initial concrete from the water within the tremie

Figure 4: Bucket being dispensed into tremie hopper

During the pour the author measured the depth to the top of the concrete on the outside of the reinforcement cage at the same time as the depth to the top of the concrete at the center was measured. Comparing the difference in elevation gives a direct measurement of how well the concrete is flowing through the reinforcement cage. Figure 5 shows a graph of the measurements.
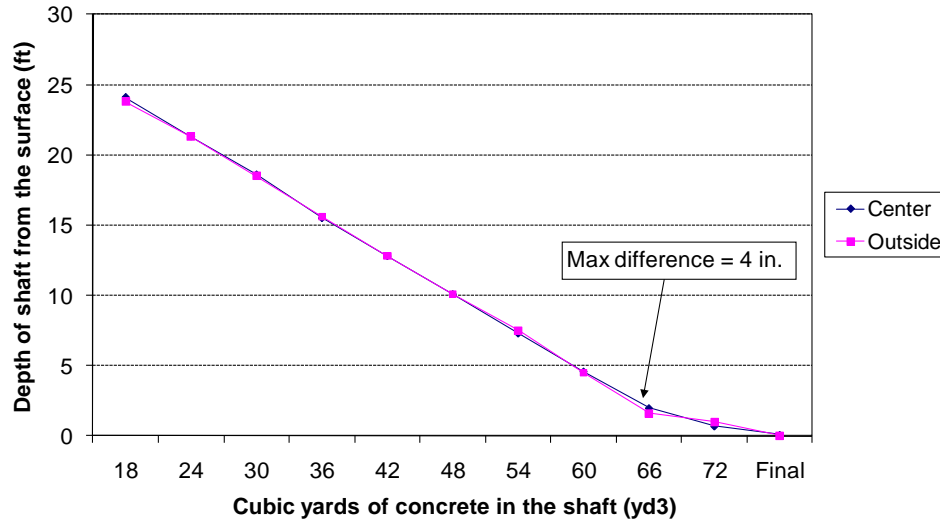
Figure 5: Concrete elevations at the center of the shaft compared to elevation outside the reinforcement cage

Figure 5 shows that the concrete does not seem to have trouble flowing through the reinforcement cage. The maximum elevation difference was four inches, which given that the concrete is traveling four feet to the outside of the cage, is not a substantial difference.

The entire concrete placement took 1 hour and 40 minutes from start to finish. No problems were observed during placement. The shaft was filled complete with 69 yds[3] at 4:20PM.

At no point during the concrete placement was the tremie lifted. The tremie was lifted at the conclusion of the concrete placement at 4:25PM. No problems were noticed during the removal of the tremie.

Construction Notes

By Phillip Gallet, Graduate Research Assistant, Auburn University
Last Updated on April 6, 2010

Project: BRF-0035(502)
Location: SR-35 at the Tennessee River in Scottsboro, Jackson County, AL
Construction Date: 11/16/09
Weather: Sunny, temperatures in the 60's
Shaft ID: Pier 7, Shaft 1
Shaft Depth: 43 ft
Concrete Amount: 93 yds^3
Time Started: 10:55am
Time Finished: 1:50pm

Existing B.B. Comer Bridge

North   # 1   #4   # 3   # 2   # 5

Existing N. Bound Bridge
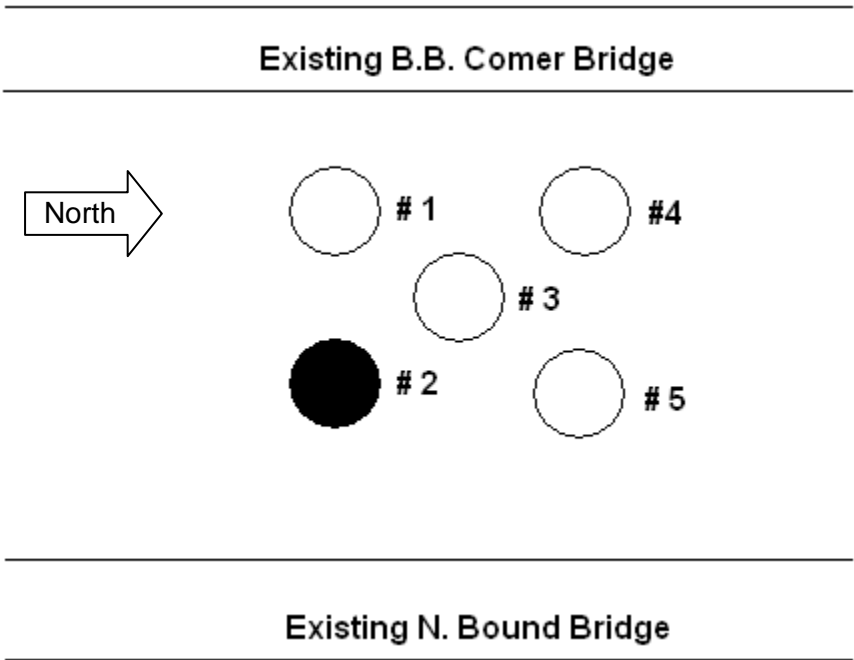
Remarks:

Kirkpatrick concrete received the call to place concrete before 10:00AM on October 13, 2009. As conducted for the previous pour, the concrete was mixed into the truck without the superplasticizer, then after 30 revolutions the superplasticizer was added. Also, each truck was loaded with only 6 yds^3 of concrete.

At 10:14AM the first truck filled at the batch plant was tested. This concrete had slump flow of 19.75" with a VSI of 0.0. The second truck batched had a slump flow of 19", air content of 4.2% and a VSI of 0.0. A few cement balls were observed in both of these trucks. The second truck had one basket ball sized clump that was removed at the plant. Figure 1 shows the slump flow patty of the first truck.

Figure 1: Slump flow patty of first truck (Slump flow =19.75")

At the jobsite the first truck arrived to the testing area and filled 5-gallon buckets for fresh concrete property testing and cylinder making. This concrete was taken directly off the back of the truck. The concrete properties were: slump flow = 19" and air content = 4.2%.

To place the concrete the ready mix concrete trucks discharged the concrete into 3 yd^3 buckets that were moved using one of two different cranes located on barges on the water. A total of four buckets were utilized to help speed up the concrete pour. The buckets were filled and carried down to the barges until all four buckets were filled. The cranes then took turns picking up the buckets from their location on the barge and discharged them into a hopper located on the top of a 10 in. tremie pipe. Once all four buckets were used one crane was used to fill one bucket and continue the concrete pour.

The concrete placement from the filled buckets began at 10:55AM. The first bucket was discharged into the hopper. The concrete took time to push the foam plug through the tremie, but once the plug was pushed out the concrete began to flow freely down the tremie. Once the hopper was emptied the second bucket was moved into place and began filling the hopper as soon as possible.

The entire concrete placement took 2 hours and 55 minutes from start to finish. A good amount of time delays occurred during the pour due to a lack of concrete trucks from the batch plant or traffic on the way to and from the site. The delays were usually 5 to 15 minutes in length. Despite these time delays, no problems were observed during placement. The shaft was filled complete with 93 yds^3 at 1:50PM.

At no point during the concrete placement was the tremie lifted. The tremie was lifted at the conclusion of the concrete placement at 2:00PM. No problems were noticed during the removal of the tremie.

Construction Notes

By Phillip Gallet, Graduate Research Assistant, Auburn University
Last Updated on May 12, 2010

Project: BRF-0035(502)
Location: SR-35 at the Tennessee River in Scottsboro, Jackson County, AL
Construction Date: 4/28/09
Weather: Sunny, high in the upper 60 °F
Shaft ID: Pier 7, Shaft 4
Depth: 38.5 ft
Concrete Amount: 78 yd^3
Time Started: 11:40 a.m.
Time Finished: 2:10 p.m.

Existing B.B. Comer Bridge

#4

North

Existing N. Bound Bridge
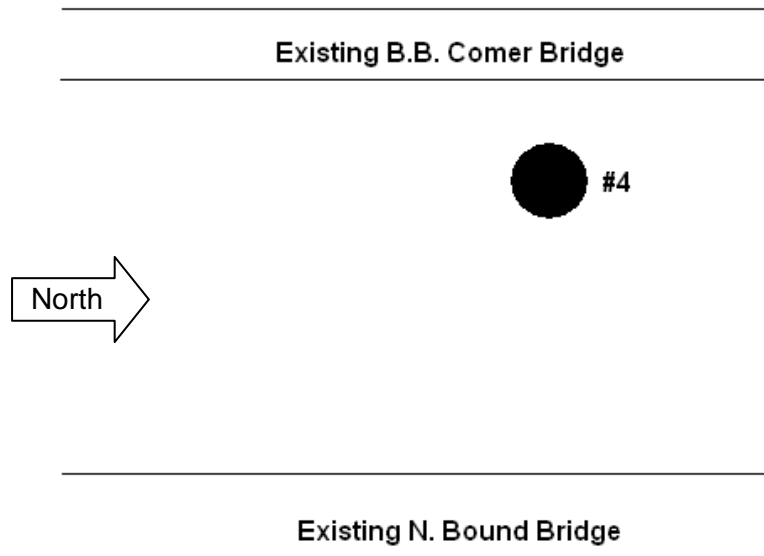
Remarks:

Concrete placement was scheduled to start between 8am and 12pm on April 28, 2010. Upon arrival, at 9:00 a.m., Russo personnel were lowering the steel reinforcement cage into the shaft as shown in Figure 1.

Figure 1: Lowering the steel reinforcement cage into the shaft

Unlike all the shafts constructed for Pier 7, this shaft was constructed without a full depth permanent casing. Some concerns were expressed by Scott Bridge Co. personnel that the concrete may escape into fissures in the rock due to the lack of this full depth permanent casing.

Kirkpatrick received the call for concrete at approximately 10:45 a.m. As conducted for the previous placements, the concrete was mixed into the truck without the superplasticizer, then after 30 revolutions the superplasticizer was added. Each truck was loaded with only 6 yds^3 of concrete.

At the jobsite the quality control testing was conducted by ALDOT technicians in the designated concrete testing area on the North side of the Tennessee River. The ALDOT selected truck must stop by this area before heading to the bridge. In this area the truck places the concrete from the very back of the truck into 5-gallon buckets. These buckets are then loaded into the back of a pick-up truck and driven a short distance to the area where the quality control testing is conducted. In this area the air content, unit weight, fresh concrete temperature, and slump flow are determined. If this concrete meets the required specifications it is allowed to proceed to the bridge.

To place the concrete into the shaft the ready mix concrete trucks discharged the concrete into 3 yd^3 buckets. These buckets were moved using one of two different cranes located on barges. A total of four buckets were utilized to accelerate the concrete placement. The buckets were filled off the existing AL-35 North bound bridge and moved to lay on one of the barges until all four buckets were filled. The cranes then took turns picking up the full buckets from their location on the barge and discharged them into the tremie hopper located on the top of a 10 in. tremie pipe. Once all four buckets were used, one crane was used to control one bucket and continue the concrete placement.

During the concrete placement operation on this date, the author performed slump flow and VSI tests on a concrete sample from each truck. Each truck was capable of filling the 3 yd^3 bucket twice. Therefore, the samples were taken in 5-gallon buckets after the concrete truck had filled the first 3-yd^3 bucket. The 5-gallon buckets were transported by cart to the testing area located on the bridge approximately 100 ft away.

Most of the concrete samples tested on this date had VSI values of 0. Load 8 and Load 13 had a VSI value of 0.5. The slump flow test results are shown in Figure 2. The average slump flow for the tests taken on this date was 18.0 in., the minimum allowed slump flow determined by the project specification (21 in. ± 3 in.). The minimum slump flow was 13.5 in. and the maximum was 20.5 in.
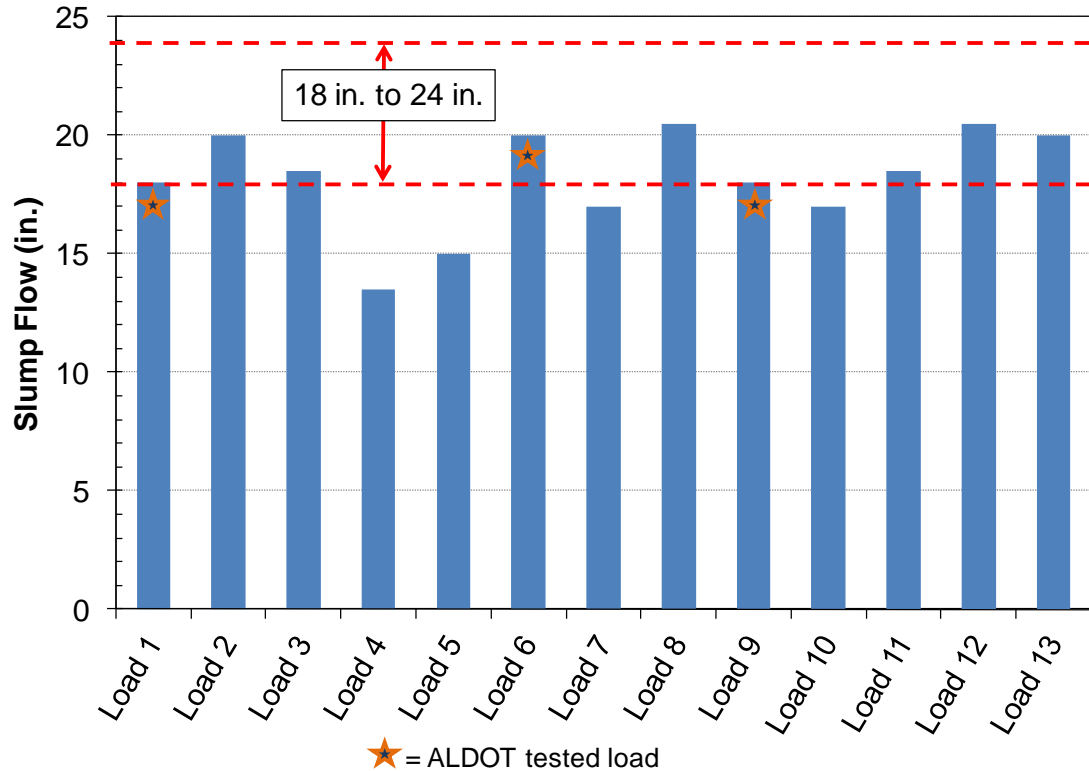

Figure 2: Slump flow results

In addition to the slump flow and VSI testing, a bleed test and pressurized bleed test were performed on a sample of concrete from Load 12.

To approximate the amount of bleed water under pressure a pressurized bleed test was conducted. The pressurized bleed test is based on a forced bleed test, developed by Khayat, to test the ability of grout to bleed in pre-stressed applications (Khayat and Yahia 1997). This test was performed placing a concrete sample into a 6-in. Ø by 12-in. tall piston chamber. The concrete was placed into the chamber using one steady motion. A rubber mallet was used to tamp the sides of the chamber (4 tamps per side: North, South, East, and West). The top of the chamber was then screened off to remove any excess concrete. Next, the cap was placed on the chamber. The chamber cap has a metal screen filter, filter paper, and a steel plate with holes to prevent the concrete and paste from leaving the chamber. A picture of the piston cap is shown in Figure 3. The bottom of the chamber is a piston that is actuated by a rubber tire tube. This tire tube is pressurized from an adjustable air-compressor. A picture of the pressurized bleed test chamber is shown in Figure 4. The assembled pressurized bleed test is shown in Figure 5.

Figure 3: Piston cap with metal filter, filter paper, and a metal plate with holes to prevent aggregate and paste from leaving the piston chamber

Figure 4: Pressurized bleed test chamber and air compressor

Figure 5: Constructed pressurized bleed test apparatus

Before the air compressor was connected to the apparatus, water was added to the beaker located on the top of the cap to fill the air voids located in the cap. Water was added until water began to come out of the bleed valve (located next to the beaker). Once air stopped exiting the bleed valve, the valve was shut and the amount of water in the beaker was recorded. The air compressor was then attached to the apparatus and an attempt was made to slowly turned up the air pressure to 10 psi. Over approximately one minute the chamber pressure went from 0 psi to 20 psi. The chamber was kept at this pressure for 30 min. taking readings every 5 min. After 30 min. the pressure was increased to 30 psi for 30-min. taking readings every 10-min. for the next 30 min. and every 30 min. after that until the bleeding had concluded. This conclusion was determined by two consecutive equal beaker readings.

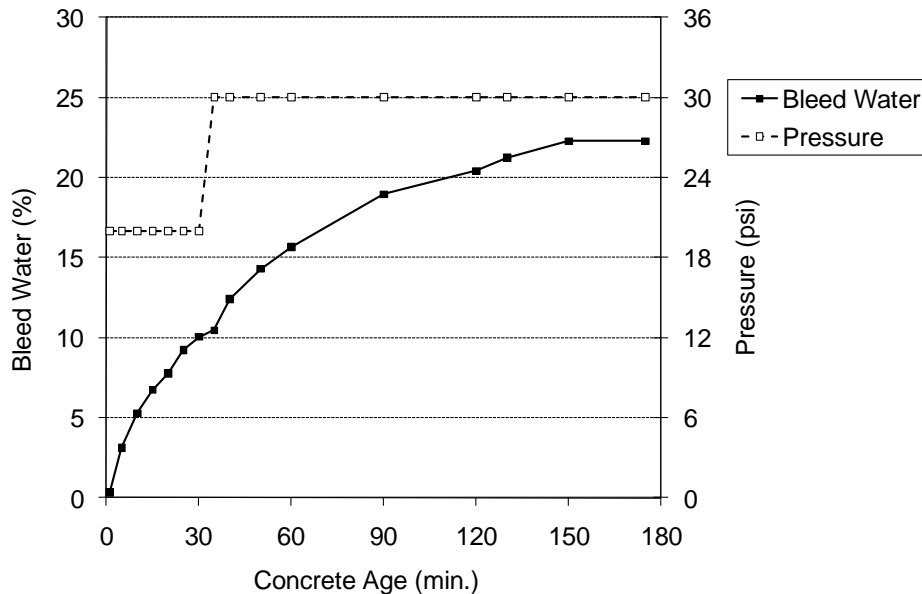The results from the pressurized bleed test are shown in Figure 6.

Figure 6: Pressurized bleed test results

292

The amount of bleed water that occurs at atmospheric pressure was determined by a conventional bleed test. This test was conducted in accordance to ASTM C232 (2004). The concrete sample did not produce any bleed water. A table comparing the total pressurized bleed water to the conventional bleed water is shown in Table 1.

Table 1: Total bleed water from conventional bleed test and pressurized bleed test

	Total Bleed Water (mL)
Conventional Bleed Test	0
Pressurized Bleed Test	215

The author was not present at the extraction of the tremie, but was not informed of any issues. The calculated amount of concrete required to fill this shaft was 72 yds^3. The total amount of concrete placed in the shaft was 78 yds^3. Because the calculated volume is only 6 yds^3 less than the actual volume, the loss of concrete did not seem to be an issue for this shaft. The author left the jobsite at 6:00 p.m.

Construction Notes

By Phillip Gallet, Graduate Research Assistant, Auburn University
Last Updated on May 18, 2010

Project: BRF-0035(502)
Location: SR-35 at the Tennessee River in Scottsboro, Jackson County, AL
Construction Date: 5/14/10
Weather: Sunny, high in the upper 70 °F
Shaft ID: Pier 8, Shaft 5
Depth: 23.5 ft
Concrete Amount: 57 yd^3
**Time Started:** 11:15 a.m.
Time Finished: 1:15 p.m.

Existing B.B. Comer Bridge

North   ◯ #4

⬤ # 5

Existing N. Bound Bridge
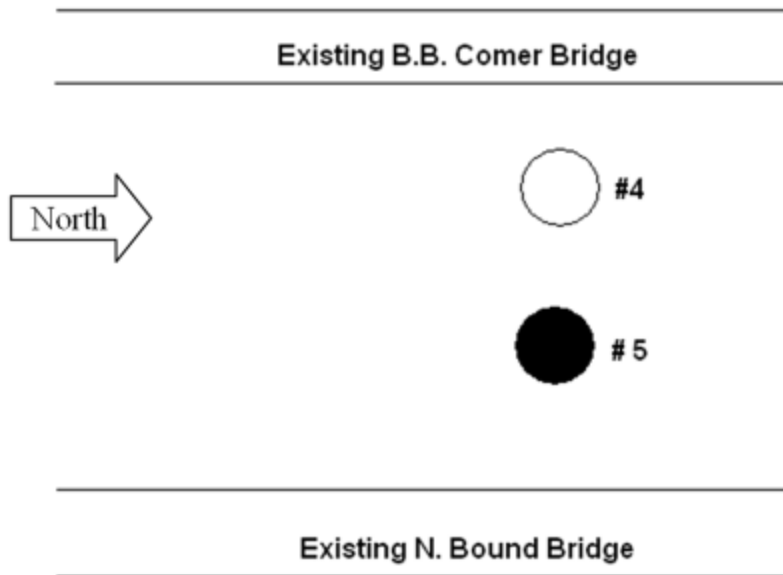
Remarks:

On the day before the concrete placement, temperature sensors were installed onto the steel reinforcement cage. The location of these sensors was 4 ft, 12 ft, and 18 ft from the bottom of the shaft. At each location, 3 temperature sensors were installed:

- One sensor approximately 3-in. outside the reinforcement,
- One sensor on the reinforcement cage, and
- One sensor approximately 17-in. into the center of the reinforcement
  A diagram of the temperature sensor locations is shown in Figure 1.
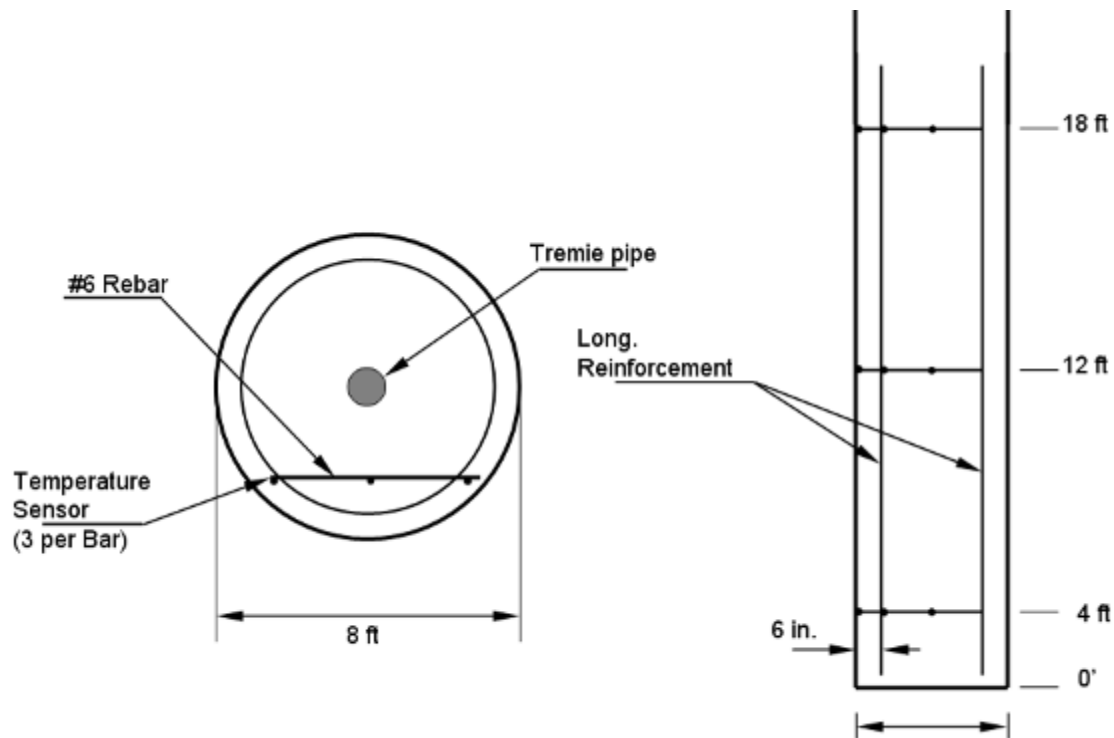
Note: 3 sensors per bar, 9 sensors per shaft.

Figure 1: Temperature sensor location

Concrete placement was scheduled to start at 9am on May 14, 2010. Upon arrival to the batch plant, at 7:30 a.m., the placement time had been extended to 10:00 a.m.

Unlike all the shafts constructed for Pier 7, this shaft was constructed without a full depth permanent casing.

Kirkpatrick received the call for concrete at approximately 10:25 a.m. As conducted for the previous placements, the concrete was mixed into the truck without the superplasticizer, then after 30 revolutions the superplasticizer was added. Each truck was loaded with only 6 yds^3 of concrete.

At the jobsite the quality control testing was conducted by ALDOT technicians in the designated concrete testing area on the North side of the Tennessee River. The ALDOT selected truck must stop by this area before heading to the bridge. In this area the truck barrel was rotated at a high rate before the concrete sample was taken. To take the sample, concrete from the very back of the truck is placed into 5-gallon buckets. These buckets are then loaded into the back of a pick-up truck and driven a short distance to the area where the quality control testing is conducted. In this area the air content, unit weight, fresh concrete temperature, and slump flow are determined. If this concrete meets the required specifications it is allowed to proceed to the bridge.

To place the concrete into the shaft the ready mix concrete trucks discharged the concrete into 3 yd^3 buckets. These buckets were moved using one of two different cranes located on barges. A total of four buckets were utilized to accelerate the concrete placement. The buckets were filled off the existing AL-35 North bound bridge and moved to lay on one of the barges until all four buckets were filled. The cranes then took turns picking up the full buckets from their location on the barge and discharged them into the tremie hopper located on the top of a 10 in. tremie pipe.

Once all four buckets were used, one crane was used to control one bucket and continue the concrete placement.

During the concrete placement operation on this date, the author performed slump flow and VSI tests on a concrete sample from each truck. Each truck was capable of filling the 3 yd^3 bucket twice. Therefore, the samples were taken in 5-gallon buckets after the concrete truck had filled the first 3-yd^3 bucket. The 5-gallon buckets were transported by cart to the testing area located on the bridge approximately 100 ft away.

All of the concrete samples tested on this date had VSI values of 0. The slump flow test results are shown in Figure 2. The average slump flow for the tests taken on this date was 16.4 in., the minimum allowed slump flow determined by the project specification (21 in. ± 3 in.). The minimum slump flow was 13.0 in. and the maximum was 19.0 in.
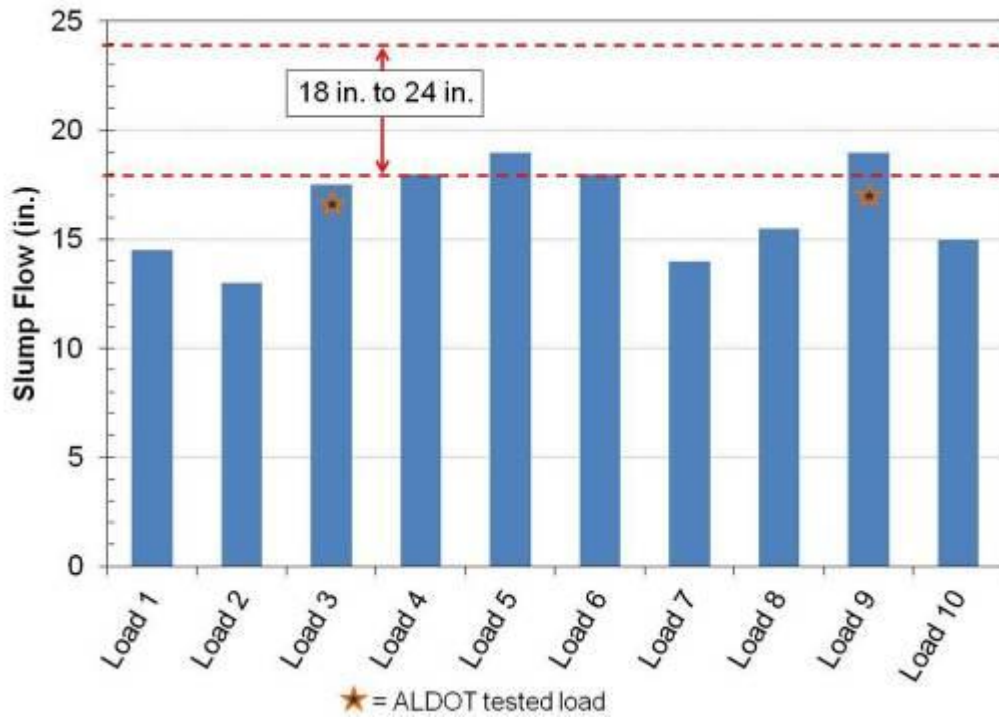
Figure 2: Slump flow results

In addition to the slump flow and VSI testing, a bleed test and pressurized bleed test were performed on a sample of concrete from Load 12.

To approximate the amount of bleed water under pressure a pressurized bleed test was conducted is the same manner as the previous shaft. The results from the pressurized bleed test are shown in Figure 6.
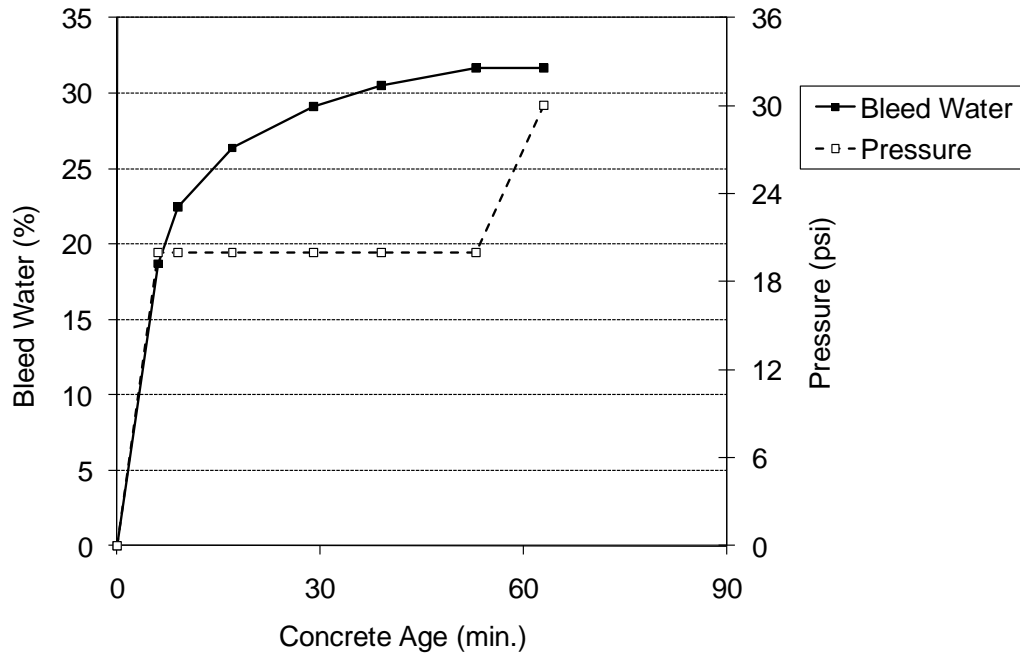
Figure 6: Pressurized bleed test results

The amount of bleed water that occurs at atmospheric pressure was determined by a conventional bleed test. This test was conducted in accordance to ASTM C232 (2004). The concrete sample did not produce any bleed water. A table comparing the total pressurized bleed water to the conventional bleed water is shown in Table 1.

Table 1: Total bleed water from conventional bleed test and pressurized bleed test

	Total Bleed Water (mL)
Conventional Bleed Test	0
Pressurized Bleed Test	322

The author was not present at the extraction of the tremie, but was not informed of any issues. The calculated amount of concrete required to fill this shaft was 60 yds^3. The total amount of concrete placed in the shaft was 57 yds^3. The actual concrete volume is 3 yds^3 less than the calculated volume. The author left the jobsite at 3:15 p.m.

Project: BRF-0035(502)
Location: SR-35 at the Tennessee River in Scottsboro, Jackson County, AL
Construction Date: 6/8/10
Weather: Sunny, high in the upper 80 °F
Shaft ID: Pier 8, Shaft 3
Depth: 23 ft
Concrete Amount: 54 yd^3
**Time Started:** 10:30 a.m.
Time Finished: 12:15 p.m.

Existing B.B. Comer Bridge

North

3   #4

5

Existing N. Bound Bridge
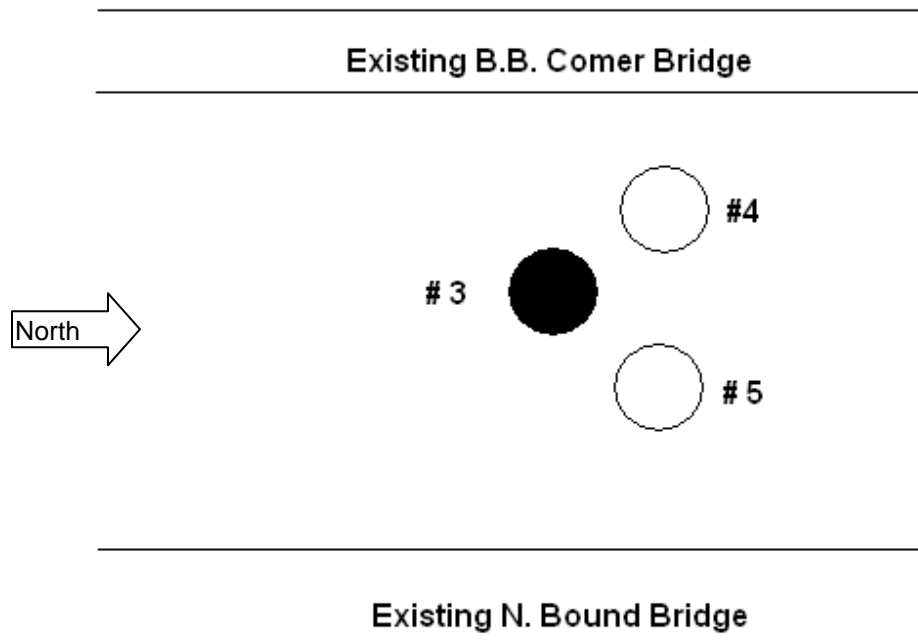
Remarks:
Before the concrete placement, temperature sensors were installed onto the steel reinforcement cage. The location of these sensors was 4 ft, 12 ft, and 17.5 ft from the bottom of the shaft. At each location, 3 temperature sensors were installed:

- One sensor approximately 3-in. outside the reinforcement,
- One sensor on the reinforcement cage, and
- One sensor approximately 19-in. into the center of the reinforcement
  A diagram of the temperature sensor locations is shown in Figure 1.
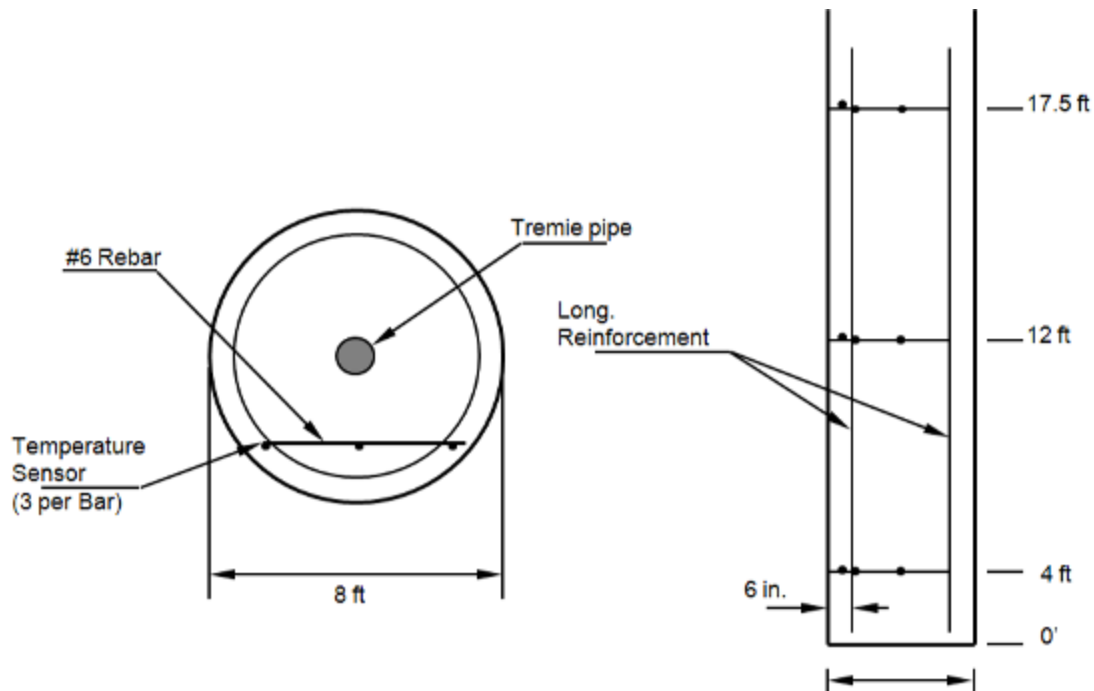
Note: 3 sensors per bar, 9 sensors per shaft.

Figure1: Temperature sensor location

In the same way as the rest of the shafts in the pier, this shaft was constructed without a full depth permanent casing.

As conducted for the previous placements, each truck was loaded with only 6 yds^3 of concrete.

At the jobsite the quality control testing was conducted by ALDOT technicians in the designated concrete testing area on the North side of the Tennessee River. The ALDOT selected truck must stop by this area before heading to the bridge. In this area the truck barrel was rotated at a high rate before the concrete sample was taken. To take the sample, concrete from the very back of the truck is placed into 5-gallon buckets. These buckets are then loaded into the back of a pick-up truck and driven a short distance to the area where the quality control testing is conducted. In this area the air content, unit weight, fresh concrete temperature, and slump flow are determined. If this concrete meets the required specifications it is allowed to proceed to the bridge.

To place the concrete into the shaft the ready mix concrete trucks discharged the concrete into 3 yd^3 buckets. These buckets were moved using one of two different cranes located on barges. A total of four buckets were utilized to accelerate the concrete placement. The buckets were filled off the existing AL-35 North bound bridge and moved to lay on one of the barges until all four buckets were filled. The cranes then took turns picking up the full buckets from their location on the barge and discharged them into the tremie hopper located on the top of a 10 in. tremie pipe. Once all four buckets were used, one crane was used to control one bucket and continue the concrete placement.

To directly measure the ability of the concrete to flow through the reinforcement cage, measurements were taken from the water surface to the top of the concrete at the center of the shaft and outside of the reinforcement cage, the cover of the shaft. The results of this test show a

299

maximum elevation difference of less than 2 ft between the center of the shaft and the cover region, located 4 ft away horizontally. The results are shown in Figure 2.
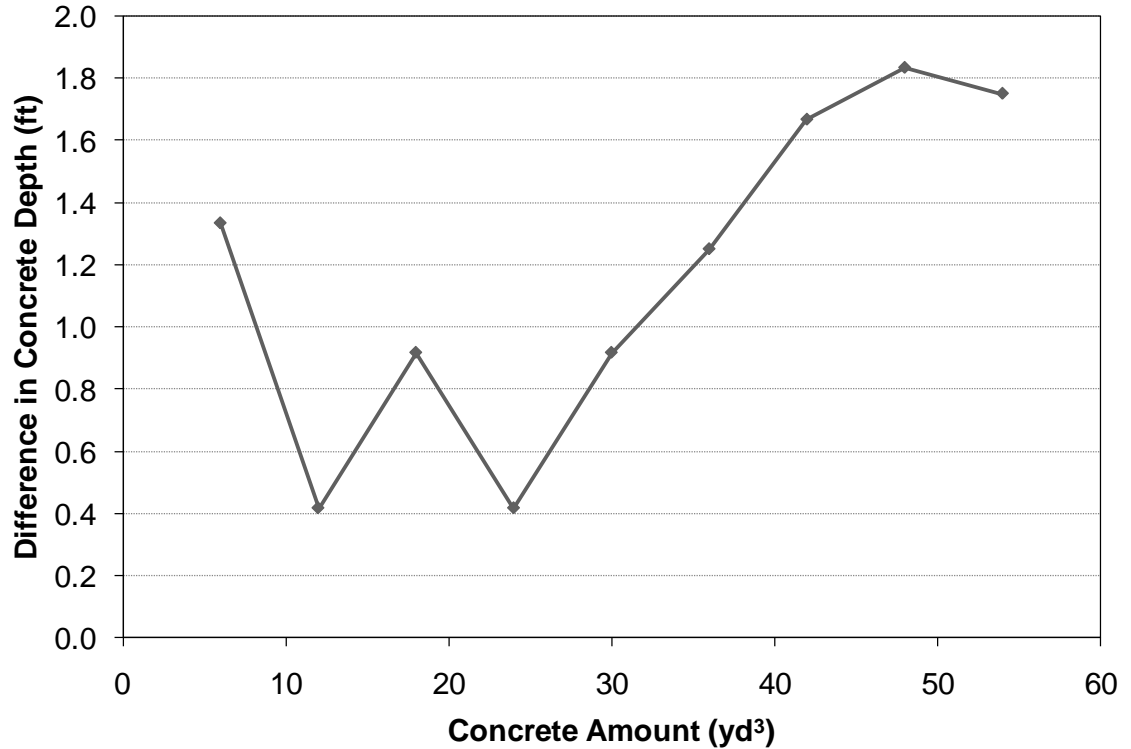

Figure 2: Elevation difference between the central and cover region of the drilled shaft during placement

During the concrete placement operation on this date, the author performed slump flow and VSI tests on a concrete sample from each truck. Each truck was capable of filling the 3 yd^3 bucket twice. Therefore, the samples were taken in 5-gallon buckets after the concrete truck had filled the first 3-yd^3 bucket. The 5-gallon buckets were transported by cart to the testing area located on the bridge approximately 100 ft away.

All of the concrete samples had VSI values within the project specification (VSI < 1.5). The first 3 loads had VSI's of 0.5, and the fifth load had a VSI of 1, all the other loads had values of 0.0. The slump flow test results are shown in Figure 3. The average slump flow for the tests taken on this date was 19.5 in., within project specification (21 in. ± 3 in.). The minimum slump flow was 17.0 in. and the maximum was 23.5 in.
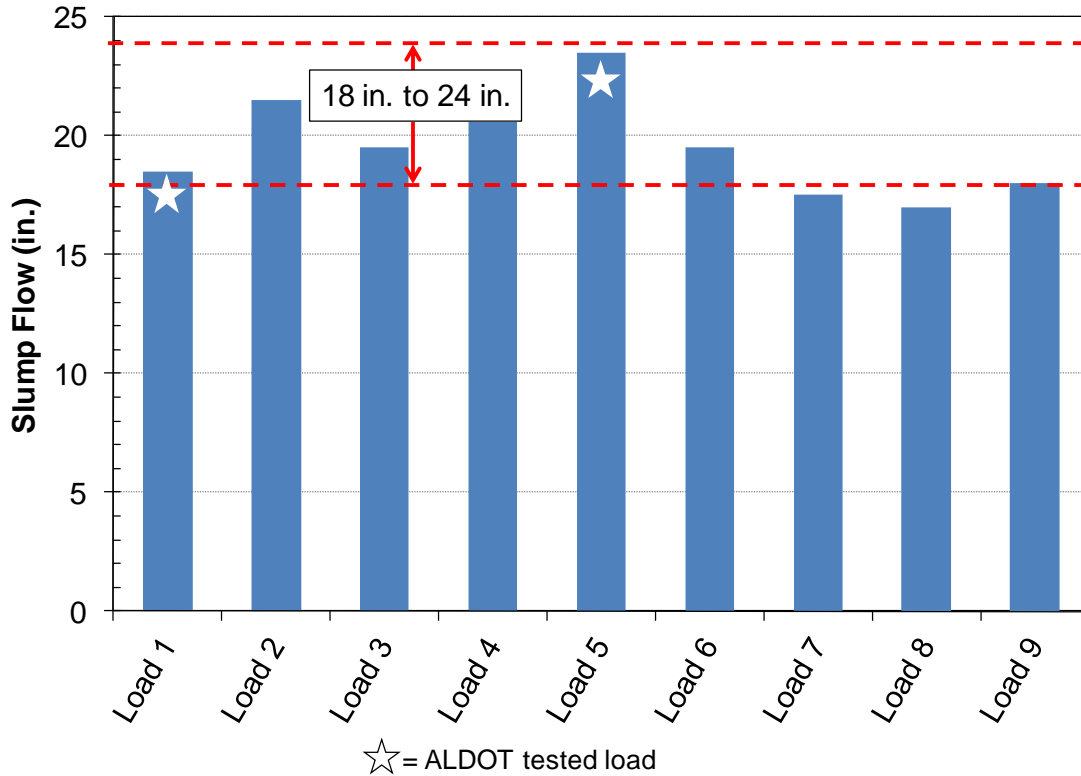
Figure 3: Slump flow results

In addition to the slump flow and VSI testing, a bleed test and pressurized bleed test were performed on a sample of concrete from a load of concrete that was not placed into the hole. The expected concrete amount was 66 yd^3, but only 54 yd^3 were placed. Samples for these tests were taken from the truck that would have been load 10.

To approximate the amount of bleed water under pressure a pressurized bleed test was conducted. The pressurized bleed test is based on a forced bleed test, developed by Khayat, to test the ability of cement grout to bleed in pre-stressed applications (Khayat and Yahia 1997). This test was performed placing a concrete sample into a 6-in. Ø by 12-in. tall piston chamber. The concrete was placed into the chamber using one steady motion. A rubber mallet was used to tamp the sides of the chamber (4 tamps per side: North, South, East, and West). The top of the chamber is then screened off to remove any excess concrete. Next, the cap was placed on the chamber.

The chamber cap has a metal screen filter, filter paper, and a steel plate with holes to prevent the concrete and paste from leaving the chamber. A picture of the piston cap is shown in Figure 4. The bottom of the chamber is a piston that is actuated by a rubber tire tube. This tire tube is pressurized from an adjustable air-compressor. A picture of the pressurized bleed test chamber is shown in Figure 5.

To simulate a shaft that is placed below the water table, the beaker on the top of the piston is pressurized to 20 psi, or approximately 50 ft of water. The assembled pressurized bleed test is shown in Figure 6.

Figure 4: Piston cap with metal filter, filter paper, and a metal plate with holes to prevent aggregate and paste from leaving the piston chamber

Figure 5: Pressurized bleed test chamber and air compressor

Figure 6: Constructed pressurized bleed test apparatus

Before the air compressor was connected to the apparatus, water was added to the bleed valve to make the water level in the beaker read 1 cm. The air compressor was then attached to top of the beaker and the pressure was slowly increased to 20 psi. This increase caused the piston to lower slightly and push the water out of the beaker. This back pressure was kept constant throughout the entire test. At 10 min. increments the piston pressure was increased to 15, 30, 45, and 60 psi.

The bleed values, in mL, were compared to the difference in pressure, taken at the piston pressure minus the back pressure. The results from the pressurized bleed test are shown in Figure 6.
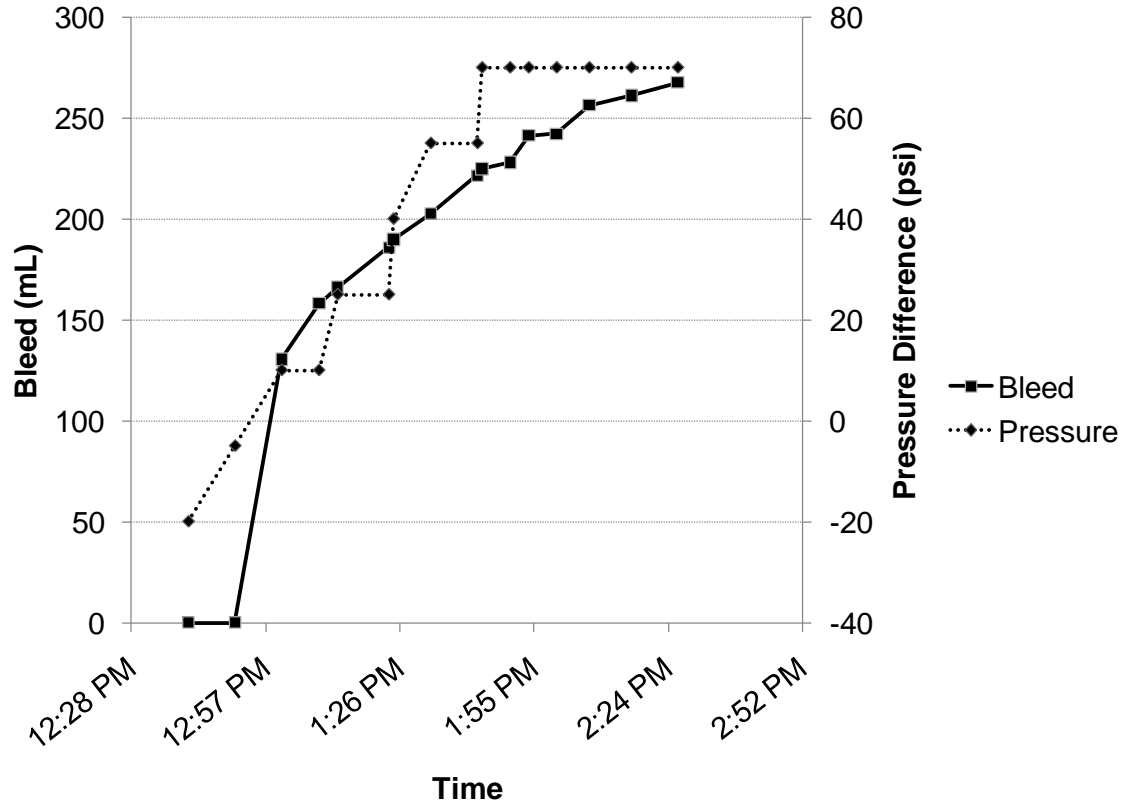
Figure 6: Pressurized bleed test results

The amount of bleed water that occurs at atmospheric pressure was determined by a conventional bleed test. This test was conducted in accordance to ASTM C232 (2004). The concrete sample did not produce any bleed water. A table comparing the total pressurized bleed water to the conventional bleed water is shown in Table 1.

Table 1: Total bleed water from conventional bleed test and pressurized bleed test

	Total Bleed Water (mL)
Conventional Bleed Test	0
Pressurized Bleed Test	268

On the day after the pour, the author went back to the jobsite to check the temperature sensors. Three of the sensors were checked and appeared to be working properly. A print out of the data from each of these sensors is shown in Figure 7, Figure 8, and Figure 9.
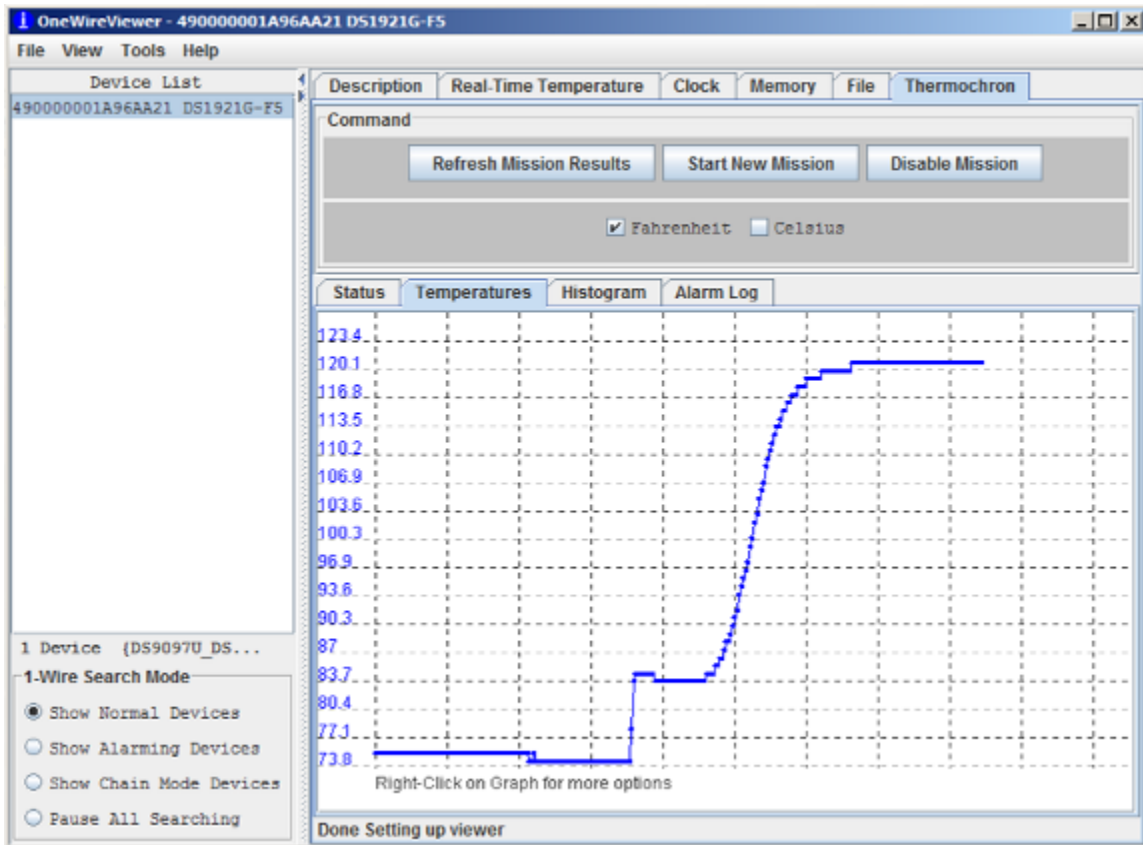
Figure 7: Data from temperature sensor located on the cage, 4 ft from the bottom of the shaft
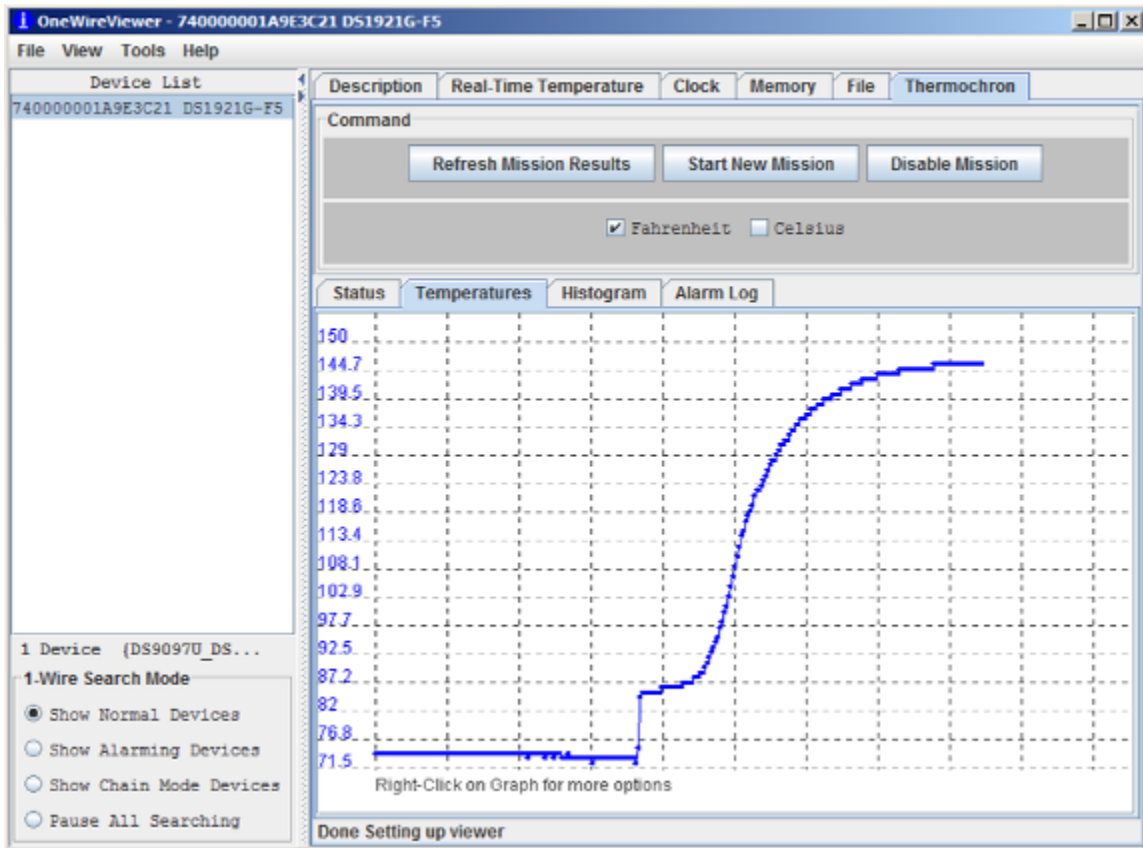
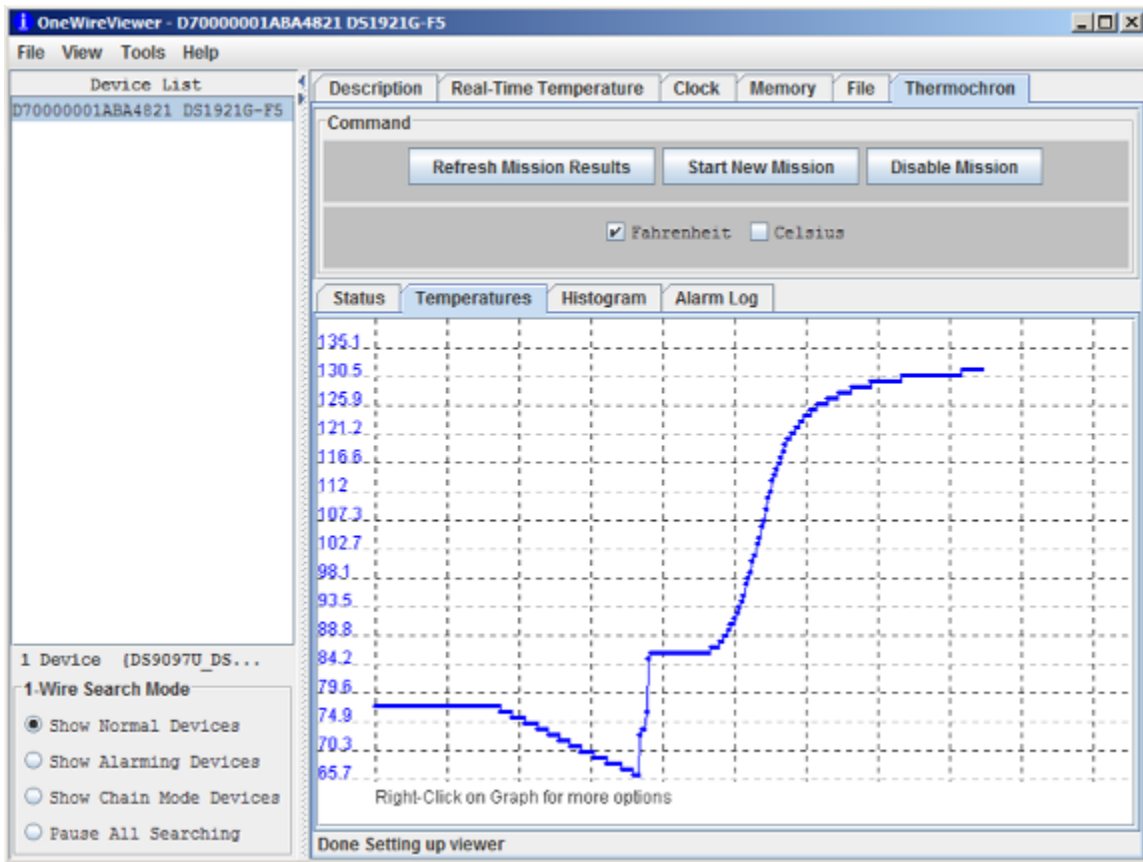Figure 8: Data from temperature sensor located on the cage, 12 ft from the bottom of the shaft

Figure7: Data from temperature sensor located on the cage, 17.5 ft from the bottom of the shaft