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**Research Report**

# EVALUATION OF HIGH-PERFORMANCE DRILLED SHAFT CONCRETE IN THE B.B. COMER BRIDGE

*Submitted to*

The Alabama Department of Transportation

*Prepared by*

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Recently developed techniques have given engineers the ability to assess the in-place integrity and quality of drilled shaft foundations. These techniques have provided insight to problems that are associated with materials and construction practices that in some instances have led to poor quality foundations. Self-consolidating concrete (SCC) with its increased flowability and higher passing ability, will be more likely to exhibit a uniform upwards flow throughout the length of the shaft, which is a critical requirement for tremie-placed concrete. SCC offers great potential for drilled shafts as it may (1) offer high flowability throughout the cross section of large shafts, (2) offer high passing ability through reinforcement cage, and (3) reduce the amount of bleed water. SCC may thus minimize some of the problems encountered in the drilled shaft industry in the past.

This report covers the findings from two field projects that were conducted to examine the feasibility of using SCC in drilled shafts in Alabama. The first field project compared ordinary drilled shaft concrete to two types of SCC. Using a different concrete mixture in each, three 6 ft diameter experimental shafts were constructed, exhumed, and cored. From the experimental shafts, it is concluded that SCC provides a drilled shaft with a much higher quality cover concrete when compared to using ordinary drilled shaft concrete. The second project was conducted by evaluating the first use of SCC in full-scale production drilled shafts in the state of Alabama. Problems that occurred on site, the variability of the concrete delivered to site, and flow of the concrete through the reinforcement cage are presented. Data show that SCC provides a drilled shaft with homogenous in-place properties and high-quality cover concrete. It is concluded that high-quality drilled shafts can be constructed by using SCC for challenging drilled shaft applications.

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# Evaluation of High-Performance Drilled Shaft Concrete in the B.B. Comer Bridge

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JANUARY 2015

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# ABSTRACT

Recently developed techniques have given engineers the ability to assess the in-place integrity and quality of drilled shaft foundations. These techniques have provided insight to problems that are associated with materials and construction practices that in some instances have led to poor quality foundations. Self-consolidating concrete (SCC) with its increased flowability and higher passing ability, will be more likely to exhibit a uniform upwards flow throughout the length of the shaft, which is a critical requirement for tremie-placed concrete. SCC offers great potential for drilled shafts as it may (1) offer high flowability throughout the cross section of large shafts, (2) offer high passing ability through reinforcement cage, and (3) reduce the amount of bleed water. SCC may thus minimize some of the problems encountered in the drilled shaft industry in the past.

This report covers the findings from two field projects that were conducted to examine the feasibility of using SCC in drilled shafts. The first field project compared ordinary drilled shaft concrete to two types of SCC in Alabama. Using a different concrete mixture in each, three 6 ft diameter experimental shafts were constructed, exhumed, and cored. From the experimental shafts, it is concluded that SCC provides a drilled shaft with a much higher quality cover concrete when compared to using ordinary drilled shaft concrete. The second project was conducted by evaluating the first use of SCC in full-scale production drilled shafts in the state of Alabama. Problems that occurred on site, the variability of the concrete delivered to site, and flow of the concrete through the reinforcement cage are presented. Data show that SCC provides a drilled shaft with homogenous in-place properties and high-quality cover concrete. It is concluded that high-quality drilled shafts can be constructed by using SCC for challenging drilled shaft applications.

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# 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). Drilled shafts, a popular deep foundation type, have been increasing in sized 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. Tremie placement is schematically shown in Figure 1.1. 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).

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,
- Good durability,
- Appropriate strength and stiffness, and
- Low heat of hydration.

Large drilled shafts often also have congested reinforcement cages to enable the shaft to resist lateral loads due to wind, seismic, and impact load effects. 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 high level of congestion prevented the concrete from encapsulating the reinforcement. An example of poor consolidation in the cover region is shown in Figure 1.4. In this figure, the concrete was capable of flowing through the reinforcement, but lacked sufficient workability to consolidate under its own weight, creating a concrete with poor quality in the cover region.

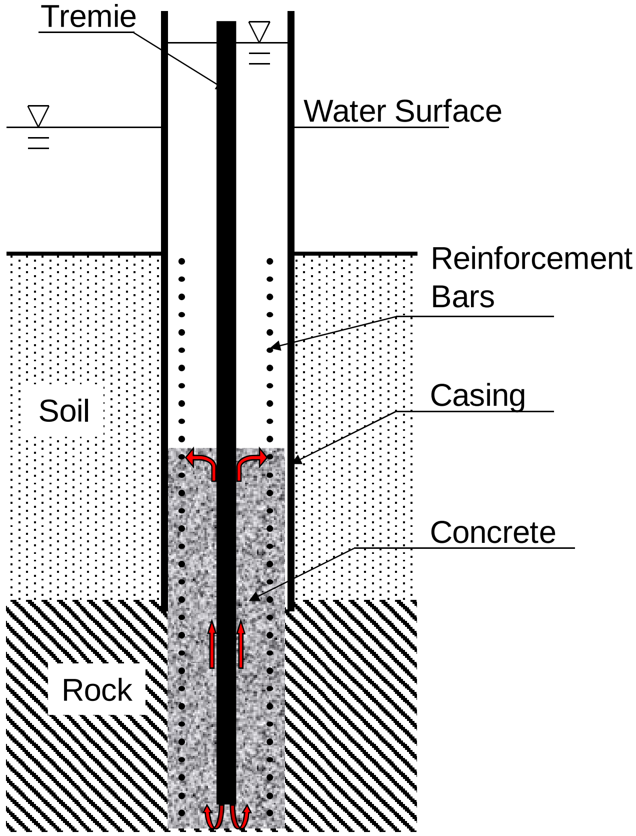



**Figure 1.1:** Schematic of tremie placed concrete

**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)

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 concrete placement. If the concrete lacks this ability, it will be difficult to maintain tremie flow during placement. Due to these difficulties, the tremie is often lifted to improve the concrete flow; however, the tremie must not be lifted high enough to risk breaching the concrete surface. A picture of shafts that had this problem during construction is shown in Figure 1.5.

**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, research has been conducted on using 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 been shown, in previous research and in 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.

Camp et al. (2007) analyzed the integrity results from more than 400 shafts from over 42 projects completed for the South Carolina Department of Transportation (SCDOT). 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 report, 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 not well understood. 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 bleed water is prevented from escaping, such as in cased shafts or shafts socketed into solid rock. A conventional bleed test is conducted at atmospheric pressure conditions on the concrete mixture to determine the amount of bleed water it could potentially release. SCC is known for having a reduced potential for bleeding (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

The main objectives of this report are as follows:

- Determine the effectiveness of using self-consolidating concrete (SCC) in drilled shafts placed under full-scale production conditions,
- Determine the flow characteristics and the in-place properties of *experimental* castings made with conventional-slump and self-consolidating concrete,
- Evaluate the in-place properties of shafts constructed with SCC with state-of-the-art non-destructive testing techniques, and
- Document the performance of using SCC in large-scale production drilled shafts of an ALDOT project.

## 1.3 RESEARCH METHODOLOGY

To compare SCC to ordinary drilled shaft concrete, three six-foot diameter experimental shafts were created with a different drilled shaft concrete mixture in each shaft:

- One with an ordinary drilled shaft concrete (ODSC) mixture used in a completed 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 introduced 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 reported.

The second part of this research was to document the first-use 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 regions 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 this 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 practices, 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, experimental 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 review of a pressurized bleed test that could be used to test drilled shaft concrete.

## 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).

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 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

There are many ways of constructing drilled shafts. 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.

#### 2.2.1.1 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.

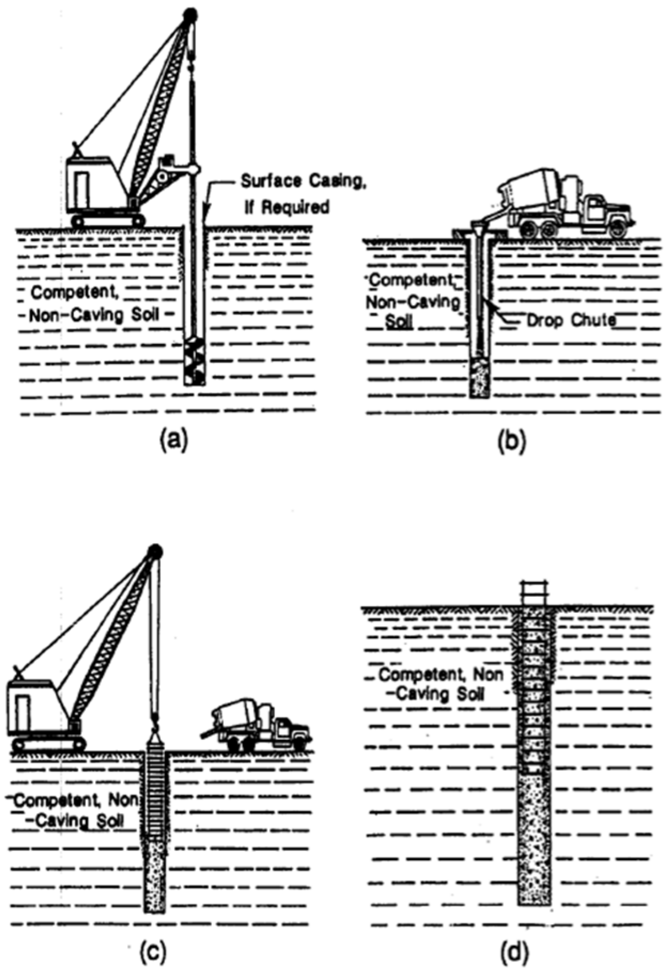



**Figure 2.1:** Dry method of construction: (a) initiating drilling, (b) starting concrete placement, (c) placing rebar cage, (d) completed shaft (O'Neil and Reese 1999)

#### 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 can be installed 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. 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 has a higher viscosity and unit weight than water. Therefore, it 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 schematically shown 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).

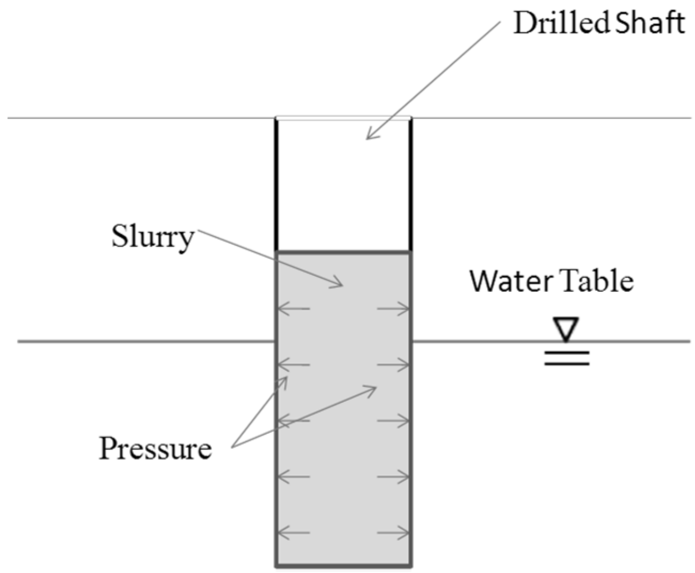



**Figure 2.2:** Pressure on outside of drilled shaft due to drilling slurry

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

Another way to construct a cased holed is to vibrate the casing into place, which seals 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 of 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 shaft using either gravity or a pump truck.

#### 2.2.1.3.1 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 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. The tremie is placed on or near the bottom of the shaft. It is popular to slice the tremie tip so 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 or more for the entire concrete placement operation (ACI 336 2001).

**Figure 2.3:** Cut groove at the bottom of the tremie

#### 2.2.1.3.2 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, known 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 the mixture can be fed through a tremie to the bottom of the excavated hole. Then the concrete must flow to the cover region 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 through the reinforcement cage to the outside of the shaft, 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 adequately in this environment, a special type of concrete must be designed. O'Neil and Reese (1999) state that 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 attack 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.

Concrete is made by mixing water with portland cement, fine aggregate, coarse aggregate. Chemical admixtures are often added to concrete to enhance its properties or to change its performance. Portland cement is a manmade 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 supplementary cementing materials (SCMs), such as fly ash, slag cement, and silica fume, are used to replace a portion of the portland cement. The combination of water. portland cement, and SCMs form 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).

In general, the strength of the concrete is dependent on the water-to-cementitious materials (w/cm) ratio. The lower this ratio, the stronger and denser the concrete will be. Conventional concrete has w/cm of 0.35 to 0.45. High-strength concrete can have w/cm below 0.35 with the use of fly ash, silica fume and water-reducing admixtures (Mindess et al. 2003). To achieve the workability required, O'Neil and Reese (1999) recommend a high w/cm 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, which results in 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 approximately 5 ft in diameter have characteristics of mass concrete in which the heat of hydration can feed on itself and generate high temperatures within the shaft. Recent measurements in Florida have shown temperatures as high as 180 °F (Mullins 2006). 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. In some instances, microcracking 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 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 to detect anomalies within the completed drilled shaft. These methods can be broken up into two different categories: destructive testing and non-destructive testing.

#### 2.2.3.1 Assessment of Completed Shaft Integrity

##### 2.2.3.1.1 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).

#### 2.2.3.1.2 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).

#### 2.2.3.1.3 Driving Completed Shaft

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

### 2.2.3.2 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.

#### 2.2.3.2.1 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 report 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, thus creating a pulse wave that will travel down and back through the shaft. The impulse wave will reflect off of any anomalies or irregularities within the concrete. The return wave is then recorded using a geophone or accelerometer. The time it takes 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.

An advantage of this test is that access tubes do not need to be previously installed, thus saving time and preventing increased congestion in the reinforcement cage. Also, it is a relatively fast test to conduct. However, a disadvantage of this test is that the results can be very difficult to interpret.

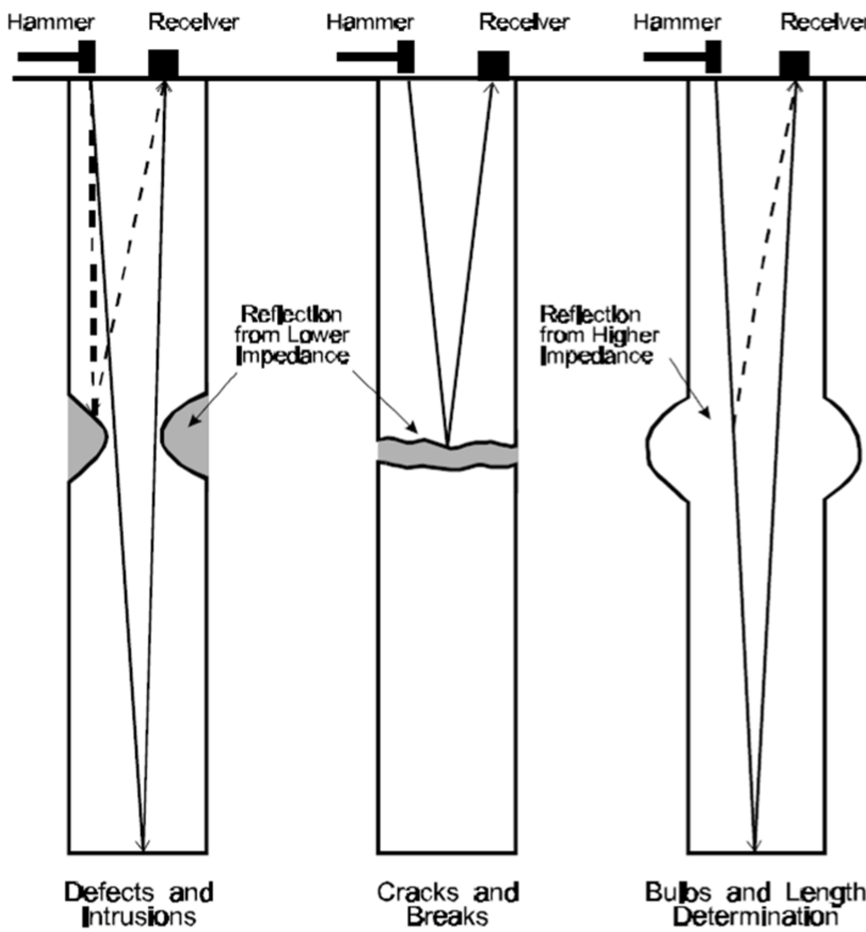



**Figure 2.4:** Diagram of sonic echo test (Olson, Aouad and Sack 1998)

**2.2.3.2.2 Crosshole Sonic Logging (CSL)**

The CSL test is a popular drilled shaft integrity testing method. A diagram of this test is shown in Figure 2.5. To conduct this test, the hollow access tubes, known as CSL tubes, are filled with water. An 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).

Some advantages of this test are 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 assesses 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).

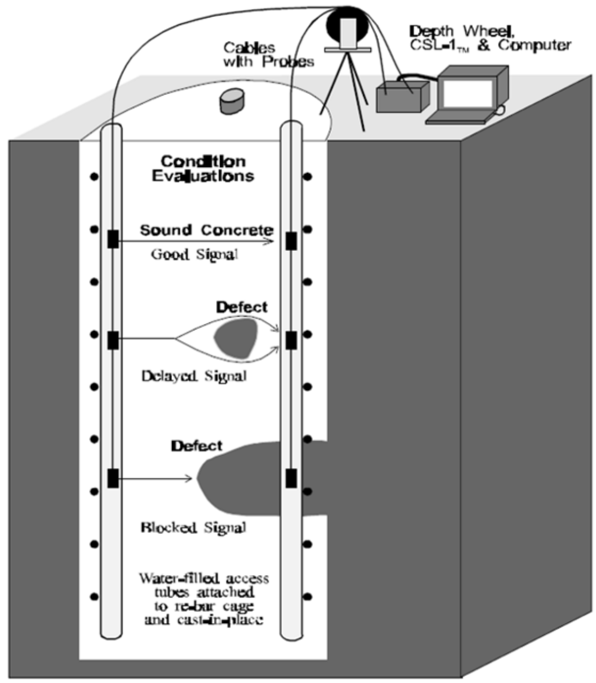



**Figure 2.5:** Diagram of CSL test (Olsen, Aouad, and Sack 1998)

#### 2.2.3.2.3 Crosshole Tomography

Crosshole tomography is very similar to the 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. A diagram of this is presented in Figure 2.6. 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 three-dimensional image can be created of the unknown anomaly.

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).

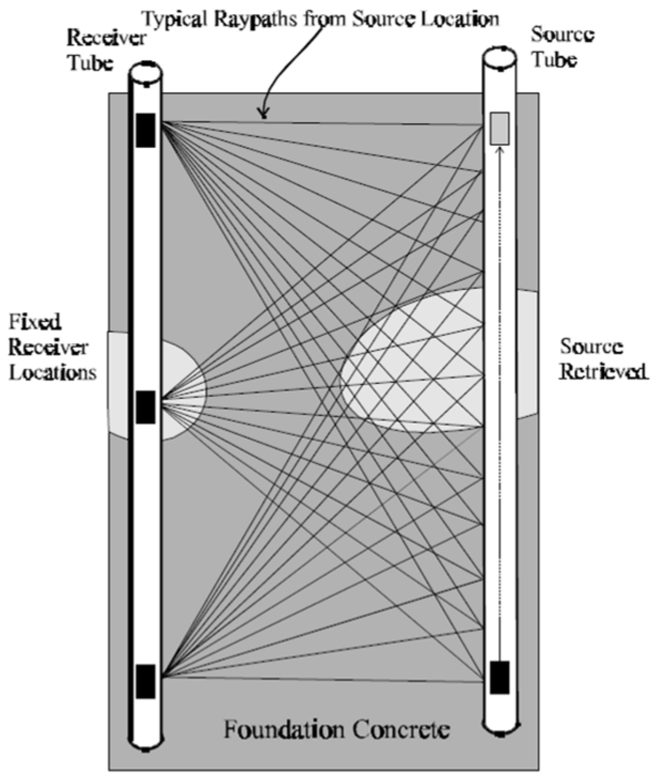



**Figure 2.6:** Diagram of crosshole tomography (Olsen, Aouad, and Sack 1998)

**2.2.3.2.4 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 receiver 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 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; 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.

#### 2.2.3.2.5 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 one and three days after the concrete was placed. In shafts greater than 8 feet in diameter, this test may be conducted no later than six days after the concrete placement (Mullins and Kranc 2007).

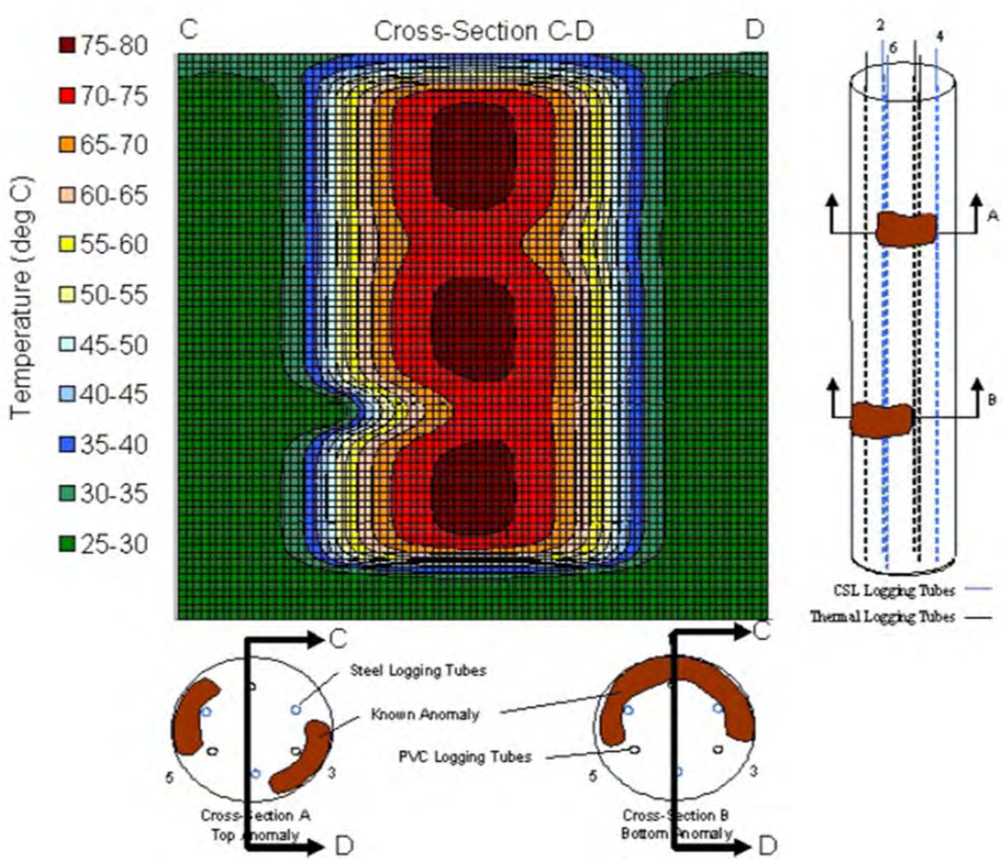



**Figure 2.7:** Model from thermal integrity testing showing anomalies intentionally placed in a test shaft (Mullins and Kranc 2007)

### 2.2.4 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.

#### 2.2.4.1 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. This occurs when either free-falling concrete into a shaft that has not been totally pumped of all water, or when tremie placing 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 form laitance (weak mortar formed by the mixing of concrete and water). During tremie placement, this problem can occur anywhere in the shaft if the tremie breaches the concrete surface. This defect is a very difficult 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).

#### 2.2.4.2 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. In the most severe cases, it 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).

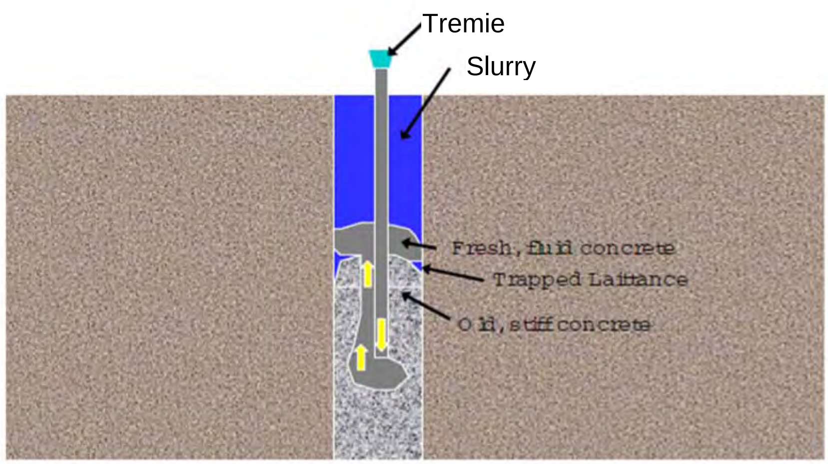



**Figure 2.8:** Effects of loss of workability during concrete placement (Brown and Schindler 2007)

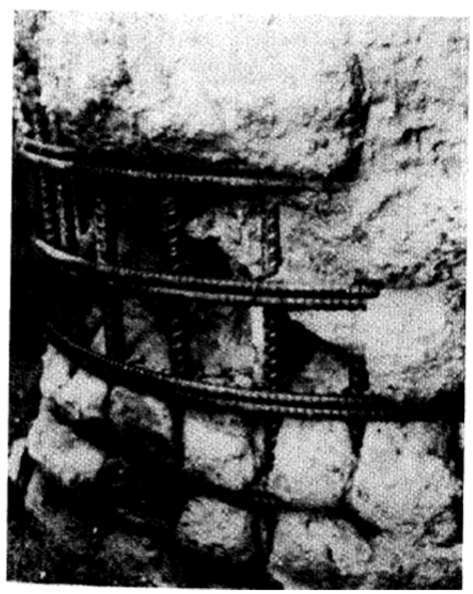


**Figure 2.9:** Defect in a drilled shaft caused by interruption in concrete supply during pumping (Photograph courtesy of Caltrans) (O'Neil and Reese 1999)

#### 2.2.4.3 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, thus weakening the bearing tip of the completed shaft. The particles can also 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).

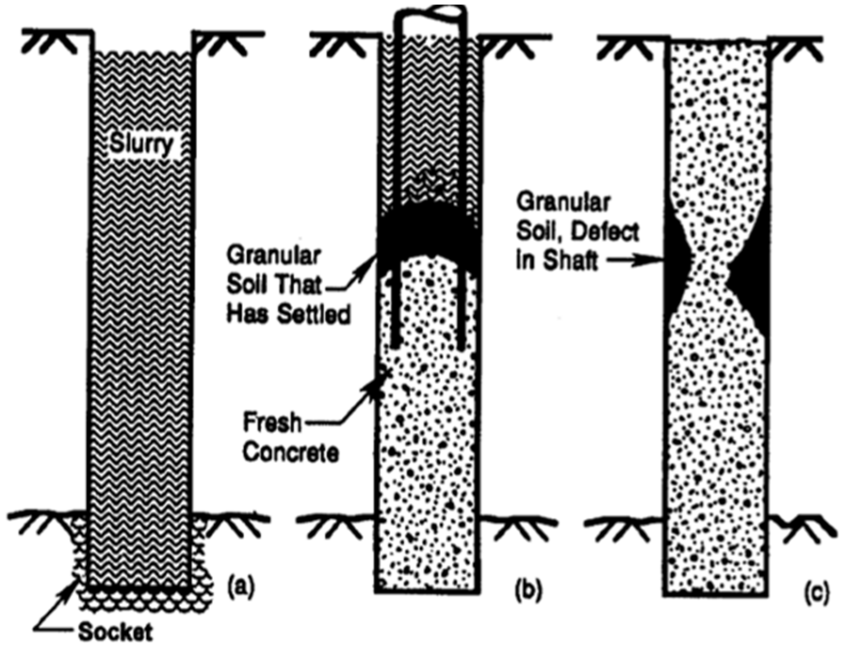
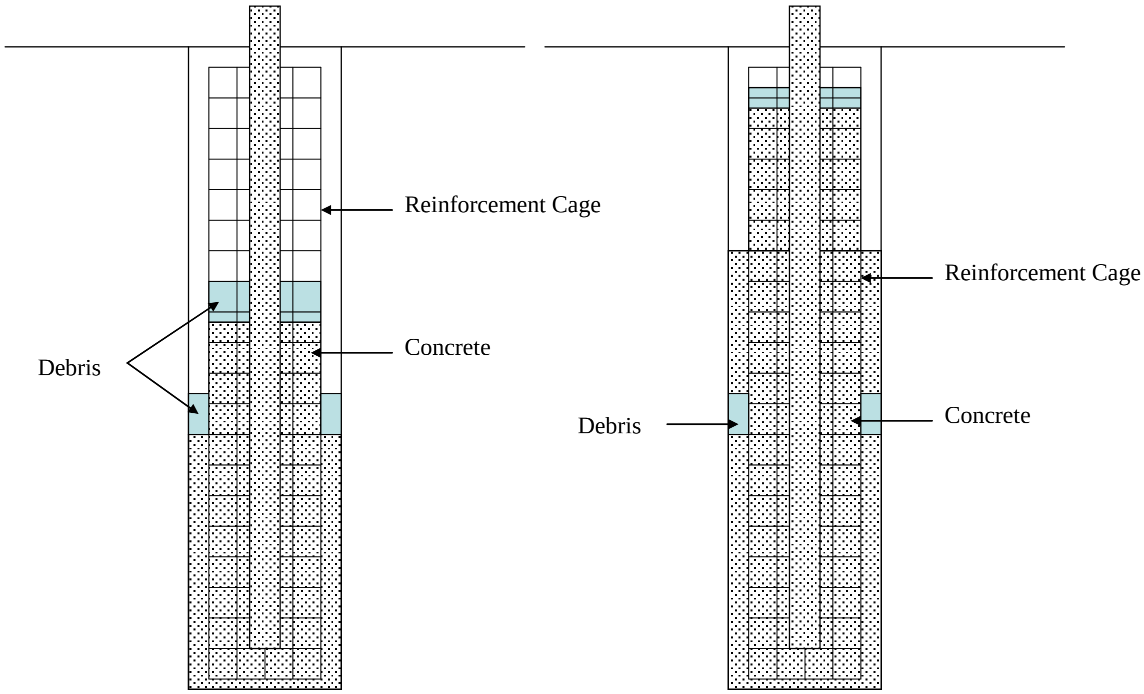



**Figure 2.10:** Placing concrete through heavily contaminated slurry (O'Neil and Reese 1999)

#### 2.2.4.4 Congested Reinforcement Cages

Brown (2004) states 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:** Diagram congested reinforcement cage causing concrete to trap debris (adapted from Bailey 2009)

#### 2.2.4.5 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 mentioned previously in this report.

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).

#### 2.2.4.6 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. 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 report 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 as SCC in this report.

### 2.3.1 Self-Consolidating Concrete (SCC)

Self-consolidating concrete (SCC) is a type of high performance concrete (HPC). McCraven (2002) wrote an article about 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
- Durability (based on exposure) and toughness
- Low heat of hydration
- Volume stability (minimal shrinkage or thermal expansion)
- Impermeability and high density
- Long service life (≥75 years)
- 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 was 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, it is 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 liters/$m^3$ 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 viscosity 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 high-range water reducing admixture (superplasticizer) controls the yield point of the mixture. As the names implies, the VMA type of SCC uses a VMA to control the plastic viscosity, while the yield point is still controlled by a water reducer.

A third classification was described by the European Federation for Specialist Construction Chemicals and Concrete Systems in its 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 Chemical Admixtures

Most SCC uses high-range water reducing (HRWR) admixture to give the concrete its high flow ability. This is achieved by the HRWR admixture giving a negative charge to all the cement particles, which causes the particles to repel one another and disperse within the mixture as presented in Figure 2.12 (Bury and Christianson 2003).

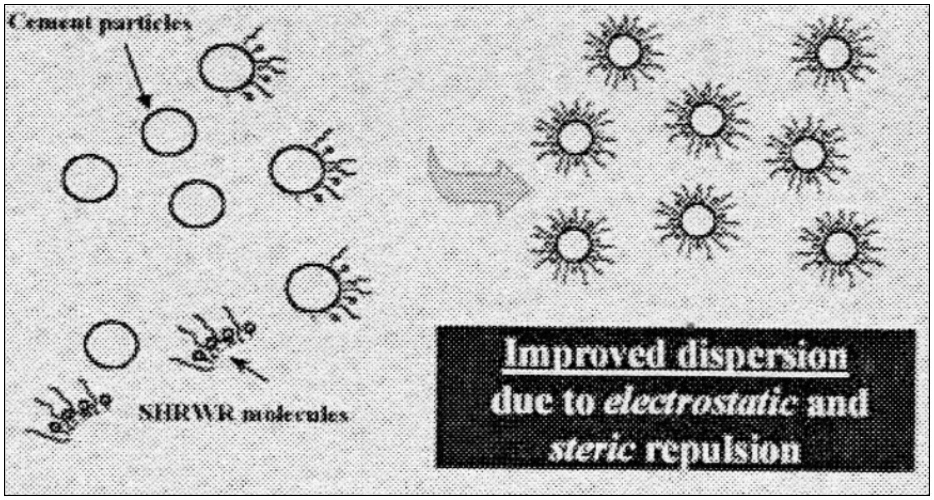



**Figure 2.12:** Dispersion of cement particles (Bury and Christensen 2003)

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 a VMA Thickening-Type. This VMA type controls the stability of the concrete mixture by thickening the mixture, 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. This VMA type 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 admixture 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 drilled-shaft 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), the retarders slow the initial concrete reaction by slowing down the growth of crystals within the mixture. 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).

### 2.3.3 Test Methods for Fresh SCC

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

#### 2.3.3.1 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 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 of the bottom of the cone and spread into a concrete patty. Once the concrete has stopped moving, two perpendicular 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.

#### 2.3.3.2 J-Ring

This test characterizes the passing ability of the fresh concrete, meaning 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, spreading through the tightly spaced vertical bars. The diameter of the impeded flow is compared to the diameter of the unimpeded flow (slump flow test) in order 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

#### 2.3.3.3 Column Segregation

This test characterizes the stability of the fresh concrete. The specification for this test can be found in ASTM C 1610 (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. This is done separately through a No. 4 sieve. 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 large drilled shafts placed by tremie with congested reinforcement steel. When the shafts are placed underwater, drilling slurry must be used. The concrete is tremie placed through the slurry to the bottom of the shaft. The concrete must also be able to flow through the congested reinforcement cage 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, 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 wall 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 standardized at the time of this research. Therefore, it cannot be verified if the current ASTM standards were used. Thus, 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. × 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):

- “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,”
- Differing mixture designs show significant CSL and PIT testing results,
- “Flow around the tube, and thus flow through tightly spaced rebar cage was improved with increasing slump flow,” and
- “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 Experimental 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 experimental 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):

- 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.
- Rapid mixing of mixtures with HRWR results in excessive air contents within the concrete mixture.
- 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.
- 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.
- 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.
- 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.
- Concrete mixtures with the No. 7 river gravel appeared to have a better aggregate distribution than the concrete mixtures with crushed limestone.
- Each shaft with the river gravel seemed to have an even amount of aggregate throughout the length of the shaft.
- 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 admixture.

#### 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-reducing admixtures 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 an increased 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. The 28-day compressive strengths of both concretes 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:

- Both SCC and the SC Coastal mix performed well in difficult construction conditions.
- “Good performance can be obtained with relatively modest attention to quality control and inspection.”
- Conventional CSL test results may exaggerate the magnitude of potential defects.
- The use of greater amounts of cementitious material in the SCC mixture did not cause an increase in the in-place concrete temperatures.
- 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.
- 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 was 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 report, 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 and built in only 11 months. Not only is this bridge well-known because of the original bridge’s catastrophic collapse, but because of the speed at which this bridge was reconstructed. The bridge construction challenge was to rapidly build a bridge with a minimum service life of 100 years. 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, “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 acceptable.  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.

#### 2.4.7 New Jersey DOT Project

The New Jersey Department of Transportation has sponsored a research project to evaluate SCC.  This project was made up of two phases.  Phase 1 was to develop SCC mixture designs to use in pre-cast structures and evaluated the use of supplementary cementing materials (SCMs).  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 flow values greater than 24 inches, “indicate good flowability, as well as good ability to self consolidate without segregation” (Nassif et al. 2008).

Phase Two consisted of the construction of five drilled shafts and 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. × 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. × 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:

- It was recommended that the L-box or J-ring test supplement the slump flow test to ensure adequate resistance to segregation,
- 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,
- “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”,
- 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
- 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 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. 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).

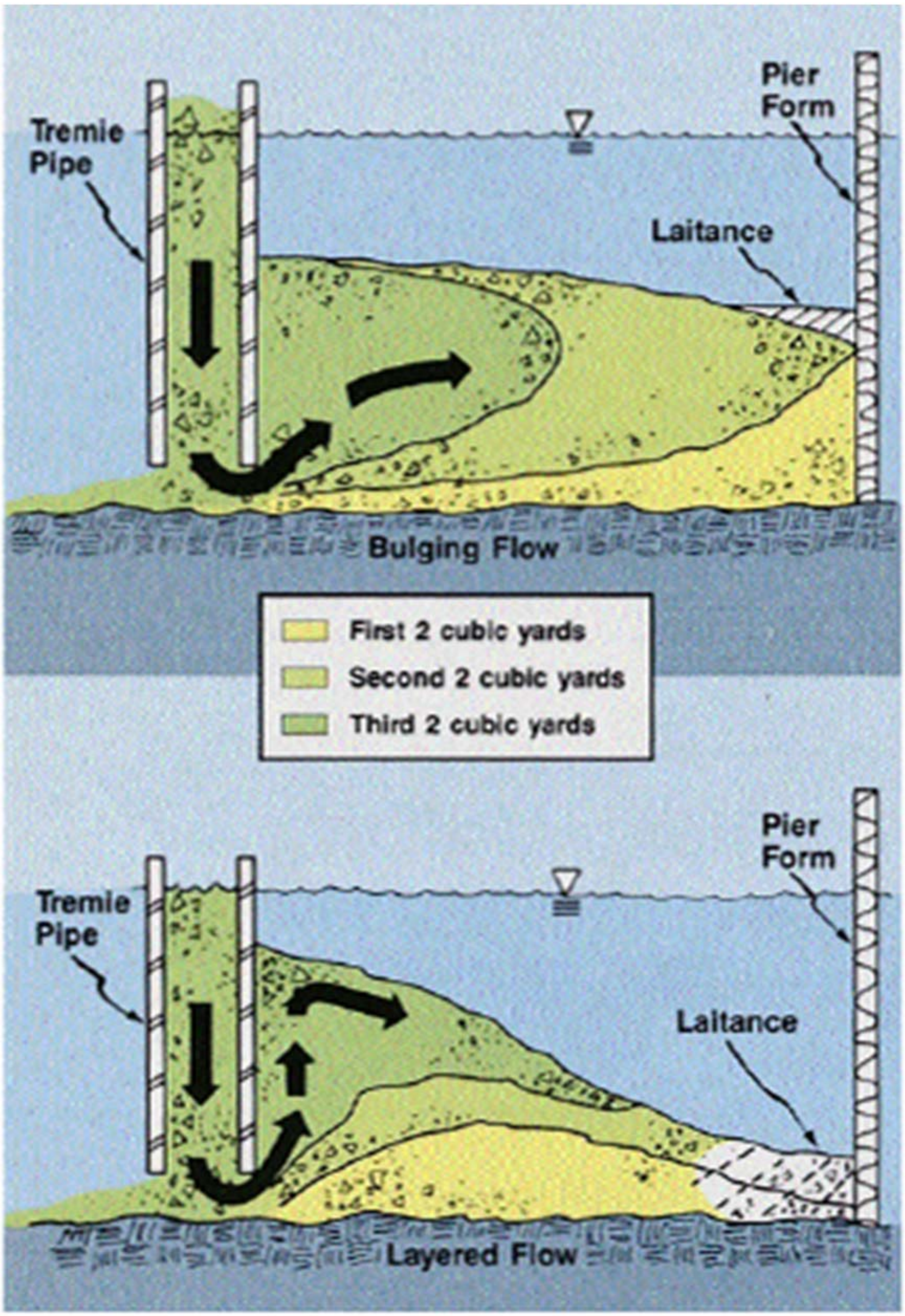



**Figure 2.15:** Bulge flow versus layered flow (Gerwick and Holland 1986)


**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 HRWRAs 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 bleed ability 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).

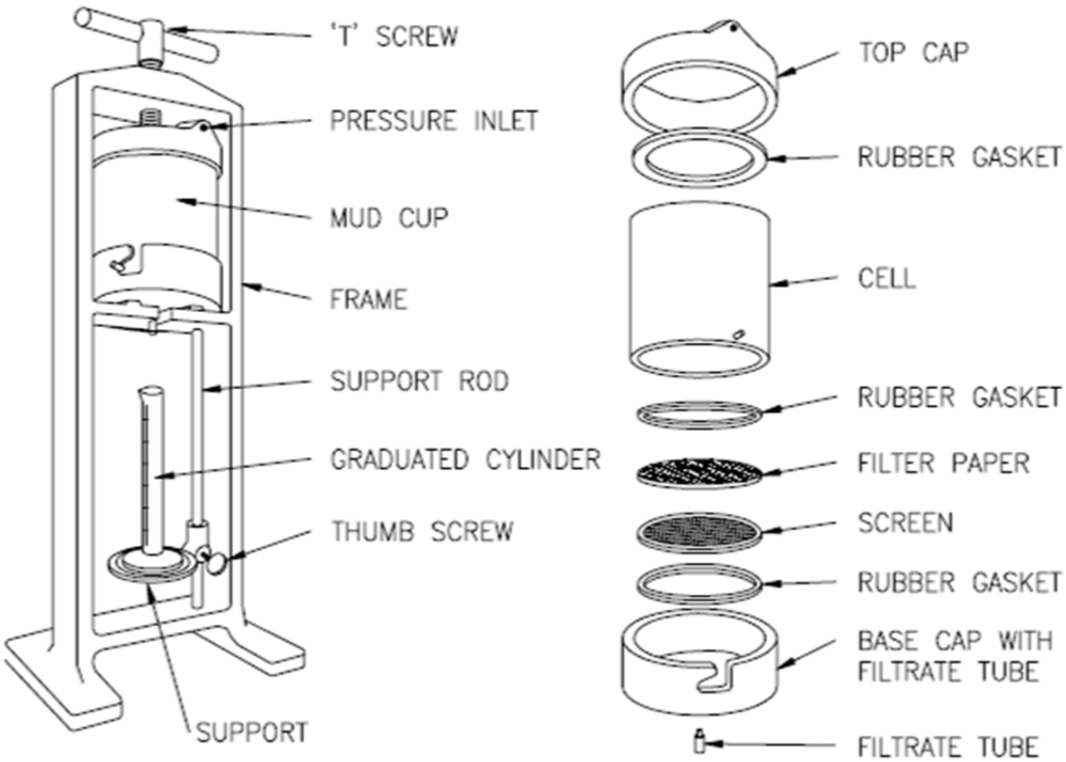



**Figure 2.19:** Standard filter press (Ball et al. 2006)

# Chapter 3

# EXPERIMENTAL 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 experimental shafts,
- Fresh concrete property testing,
- Hardened concrete property testing,
- Placement monitoring,
- Temperature measurement,
- Crosshole sonic logging testing,
- Exhuming of shafts,
- Testing of cores from exhumed shafts, and
- Analysis of exhumed shafts.

### 3.1.2. Project Location

The field study was located on AL-35 in Scottsboro, Alabama. As presented in Figure 3.1, the field study was located on the north bank of the future southbound lane of the new B.B. Comer Bridge. Three, 6 ft diameter, 25 ft long, experimental 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."

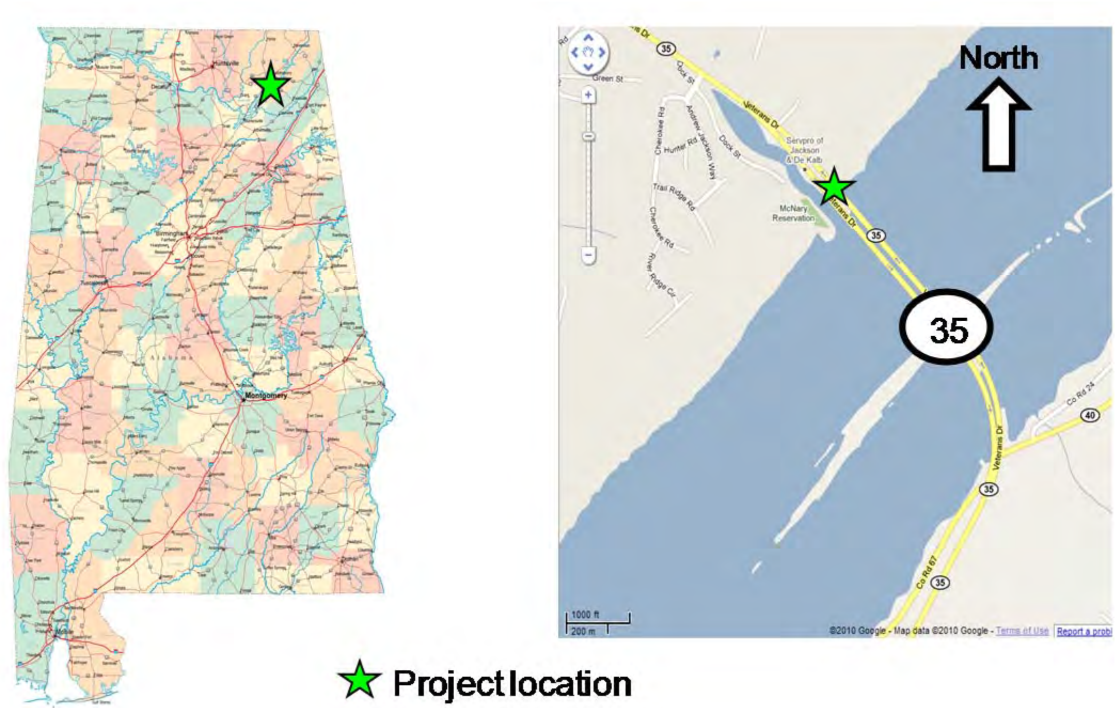



**Figure 3.1:** Project location (Google Maps 2010)

**Figure 3.2:** Location of experimental shafts (river shown in background) (Photo by D. Brown)

### 3.2.1 Experimental Shafts

Three experimental shafts made of three different concrete mixtures were constructed, tested, and exhumed. These shafts were exhumed at 28 days, or later, after placement for visual inspection and testing. Each shaft was constructed using a sono-tube casing with loose sand backfill around the outside of this casing. Loose sand was used to allow for easy removal of the cast shaft. The casing was filled with polymer slurry, and the concrete was pumped through the slurry-filled shaft. A polymer slurry was 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 the 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 shafts, each with their own unique concrete mixtures were made:

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

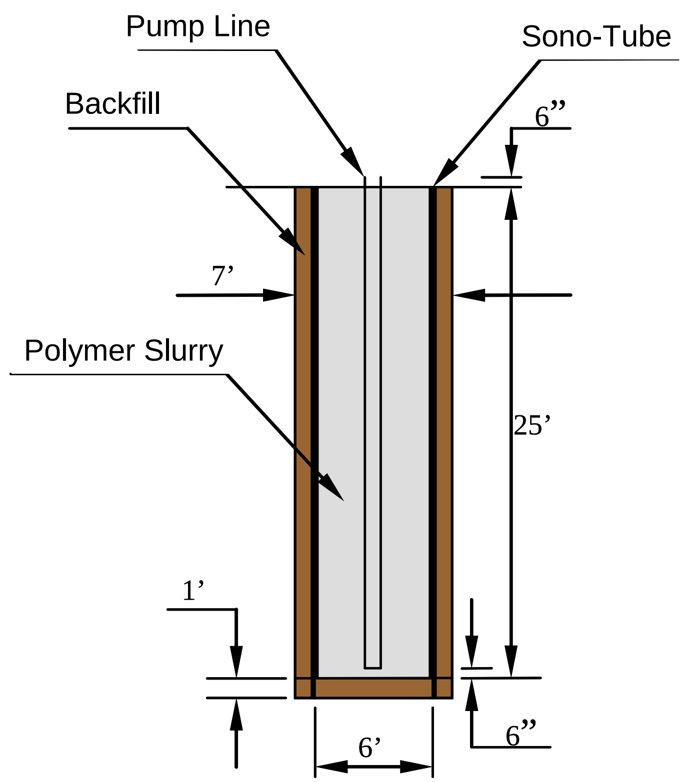



**Figure 3.3:** Longitudinal section of shaft (Dachelet 2008)

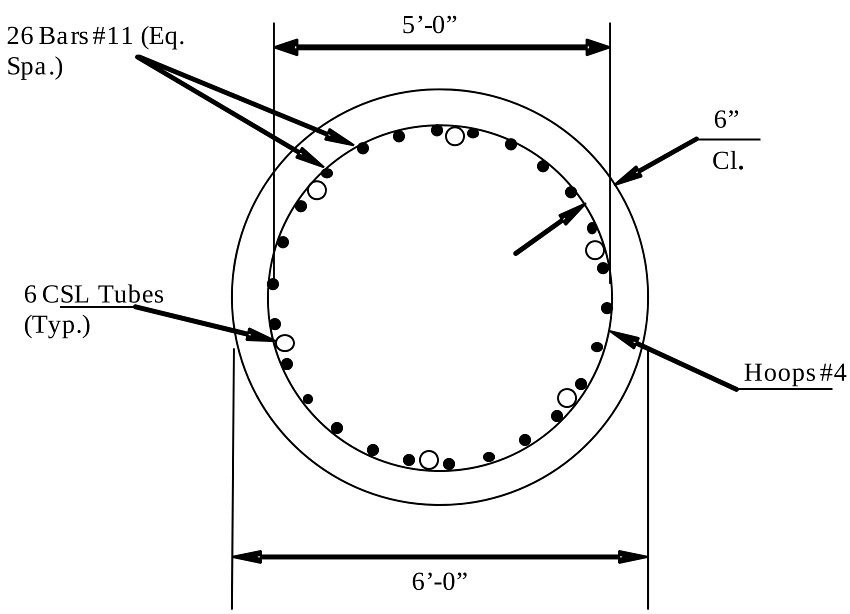



**Figure 3.4:** Shaft **c**ross section (Dachelet 2008)

### 3.2.2 Assessment of Concrete Flow During Placement

To assess the flow of the concrete within the drilled shaft, colored mortar cubes were made and added to the concrete during placement. 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 were made for use during construction of the experimental castings. These cubes were ½ × ½ × ½ in. square. Approximately 4,000 red cubes and 2,000 cubes were made for each of the following colors: blue, green, yellow and orange. A picture of a sample of these cubes is presented in Figure 3.5. These cubes were added to the tremie-placed concrete at selected locations. After the exhumed shafts were cut, the location of these colored cubes was evaluated to determine the flow characteristics of the concretes during placement.

Enough cubes were made for use in all three experimental shafts. The cubes were tested and had 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 were added to the concrete while the concrete was placed into the back of the pump truck. These cubes were placed into the concrete at a time when the concrete depth is known. Each color was introduced at a different depth.

**Figure 3.5:** Colored mortar cubes for use in experimental shafts

The first two buckets of mortar cubes placed into the shaft was 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 targeted placement of cubes of a specific 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, each shaft was removed and cut longitudinally down its center. The exposed surface was cleaned and shellacked to allow visual examination of the cube locations.

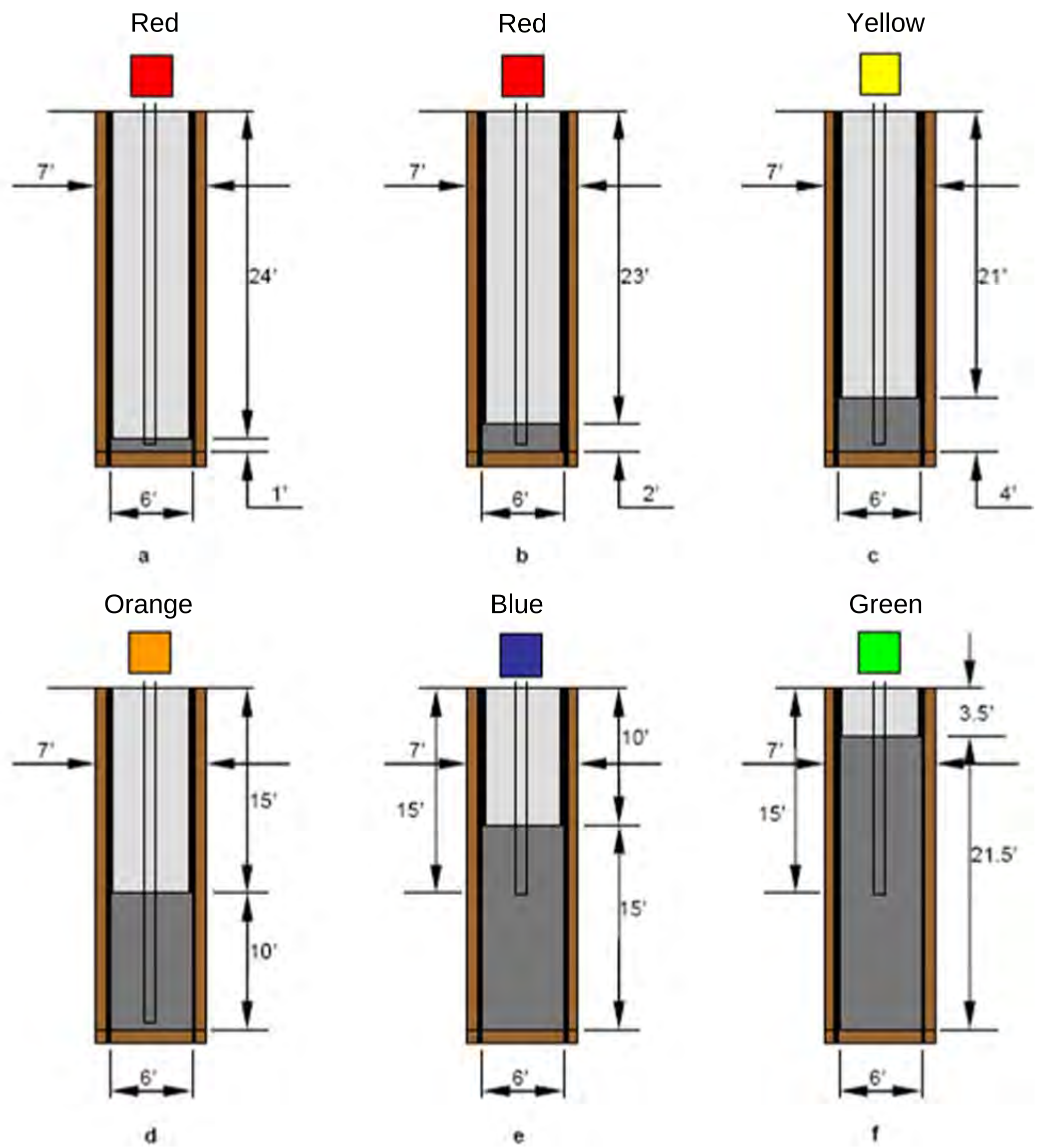



**Figure 3.6:** Placement order for each mortar cube (Dachelet 2008)

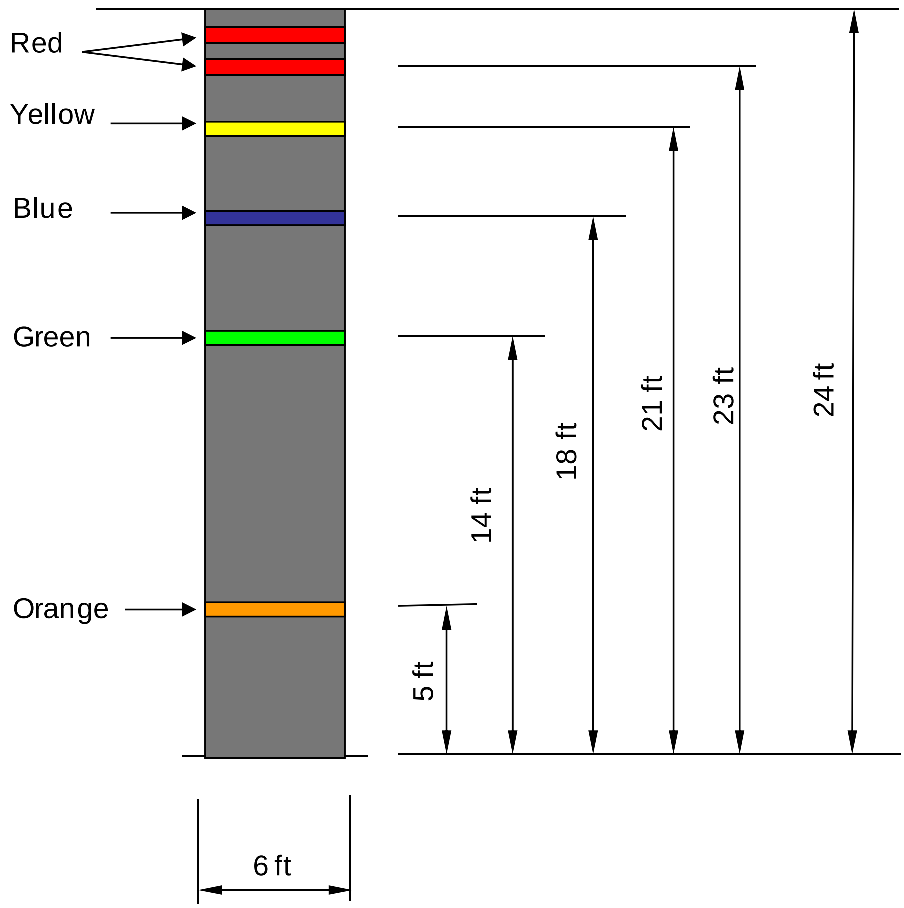



**Figure 3.7:** Hypothetical cube locations based on laminar flow (adapted from Dachelet 2008)

### 3.2.3 Assessment of Fresh Concrete Behavior

In order to test a representative sample of concrete from the concrete trucks, concrete samples were taken from the trucks after they discharged approximately half of their load. The truck was sent to the testing area at this time in order to fill wheelbarrows with concrete for property testing. The tests outlined in the remainder of this section were conducted.

#### 3.2.3.1 Slump Test

To measure the consistency of the ODSC, a slump test was performed on all concrete batches at the time of placement. This test was conducted as specified in ASTM C 143 (2005). The slump of the ODSC, at the time of placement, must be between 6 to 9 inches to meet the proposed specification (Appendix A). These samples were taken from the middle of every truck. Also, to measure the consistency of the concrete over time, this test was performed every 30 min., for a 6-hour 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.

#### 3.2.3.2 Slump Flow Test

To measure the consistency of the SCC and SCC-LP shafts, slump flow tests were performed. This test was 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 were taken from the middle of every truck. Also, to measure the fluidity of the concrete over time, this test was performed every 30 min. for a 6-hour 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.

#### 3.2.3.3 Total Air Content and Unit Weight

To determine the total air content and unit weight of the concrete, a pressure meter was used. These tests were conducted as specified in ASTM C 138 (2001). These tests were performed on samples from all concrete batches. To meet the project specifications, the air content must be 4 % ± 2 % (Appendix A). The concrete for this test was sampled from the middle of a single truck.

#### 3.2.3.4 Modified J-Ring Test

To test the concrete's ability to flow through the reinforcement cage, modified J-Ring tests were performed on the SCC and SCC-LP batches at the time of placement. This test was conducted as specified by ASTM C 1621 (2006); however, the standard J-Ring was considered too congested for drilled shaft applications. The modified J-Ring used for this project 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). The concrete for this test was sampled from the middle of a single truck.

#### 3.2.3.5 Segregation Column

To assess the static segregation of the concrete, segregation column tests were performed on the SCC and SCC-LP batches at the time of placement. This test was conducted as specified by ASTM C 1610 (2006), but the wait time will be extended from 10 min. to 1 hour. This additional wait time was used because of the extended placement times typically used for large shafts. The concrete for this test was sampled from the middle of a single truck.

#### 3.2.3.6 Bleed Test

To assess concrete bleeding, a bleed test was performed on a sample of SCC and SCC-LP at the time of placement. This test was conducted in accordance with ASTM C 232 (2005). With this method, the bleeding of a concrete sample is determined at standard atmospheric pressure. The concrete for this test was sampled from the middle of a single truck.

#### 3.2.3.7 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 was specifically developed to assess the concrete's ability to bleed under this pressure. The data collected during this project were to assist with the development of this new test method. This test was performed on a sample of SCC and SCC-LP at the time of placement.

This test is based on a forced bleed test, developed by Khayat and Yahia (1997), to test the ability of grout to bleed in prestressed applications. 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 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 approximately 75 psi for 30 min., and readings are recorded every 10 minutes. 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 approximately one third the pressure experienced in the chamber.

#### 3.2.3.8 Concrete Set Time

To test the time for the concrete to reach its initial and final set times, the penetration resistance test was used on a concrete sample from the first truck of each shaft. This test was 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:** Assembled pressurized bleed test apparatus

### 3.2.4 Assessment of Hardened Concrete Behavior

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

#### 3.2.4.1 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. high molded specimens were cast and tested per testing age for each mixture. The compressive strength was tested in accordance with ASTM C 39 (2005). The modulus of elasticity shall was tested in accordance with ASTM C 469 (2002). All specimens were cured in accordance with ASTM C 31 (2003). Because of the extended set time of some of the concrete, the specimens were removed from their molds no earlier than two times the initial set time. The specimens were tested at ages of 7, 28, 56, and 91 days.

After being cast, the samples were placed into a temperature controlled, water-filled, curing tank located in a trailer at the jobsite. These cylinders were moved to the Auburn University curing room at the conclusion of the field project. The tests were conducted in the Auburn University concrete testing laboratory.

#### 3.2.4.2 Drying Shrinkage

To assess the shrinkage of the concrete mixtures, three, 3 × 3 × 12 in. molded specimens were cast for each mixture. These specimens were tested in accordance with ASTM C 157 (2006). The shrinkage prisms were removed from the molds no earlier than two times the initial set time and placed in a lime-saturated bath for the first seven days. This bath was located at the jobsite, but was moved to Auburn University at the completion of the field project. At an age of 7 days, the specimens were removed from the lime bath and placed in air storage at ASTM specified temperature and humidity conditions. The specimen length change was measured at 1, 2, 3, 7, 14, 28, 56, 91, 180, and 365 days after removal from lime-saturated water bath.

#### 3.2.4.3 Resistance to Chloride Ion Penetration

To assess the concrete's ability to resist chloride ion penetration, three, 4 in. diameter x 8 in. tall molded specimens were cast per testing age for each mixture. These samples were tested in accordance with ASTM C 1202 (2005). The specimens were removed from the molds no earlier than two times the initial set time. After casting the specimens were placed in water-filled curing tanks, located at the jobsite. At the conclusion of the field project, these cylinders were transported to the Auburn University laboratory where the molds were removed, and the specimens placed in the curing room. The cylinders were cut to 2 in. thick slices one week before testing. The 2 in. thick specimens were cut using a water-cooled, diamond saw, and a sanding block was used to smooth blemishes around the circumference on the sample. The specimens were tested 91 and 365 days after casting.

### 3.2.5 Placement Monitoring

To assess the ability of the concrete mixture to flow through the reinforcement cage, the elevation difference between the inside and outside of the steel reinforcement cage was periodically measured during concrete placement. This monitoring was done by using plumb-bobs attached to a nylon measuring tape.

### 3.2.6 Assessment of Shaft Integrity

To assess the quality of the in-place concrete, crosshole sonic logging (CSL) was performed when the concrete exceeded an age of seven days. Six metal tubes, with inside diameters of approximately 1.75 in., were attached to the transverse reinforcement to provide access for CSL and gamma-gamma testing.

### 3.2.7 Assessment of In-Place Concrete Properties

To assess the in-place concrete properties, the shafts were removed, cut, cored, and visually examined. All shafts were exhumed at an age no earlier than 28 days after placement. After being exhumed, the shafts were laid on their sides and cut longitudinally down the center of each shaft. After cutting, the longitudinal cut surface was pressure washed, allowed to dry, and shellacked. Shellacking was applied to assist with visual examination of the in-place concrete and analyze the location of the colored mortar cubes. One-half of the longitudinal slice was then cut transversely to expose half of the shaft cross section, at locations of 7 ft and 20 ft from the top of the shaft. Visual assessment of aggregate distribution, colored mortar cube locations, and defects was conducted for each cross section. A diagram of the planned cut planes and cube locations is presented in Figure 3.11.

To assess the properties of the in-place concrete, cores were taken from the cross-sectional cuts. The cores were 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 were tested from each elevation to determine the in-place concrete's compressive strength, modulus of elasticity, and permeability. A diagram of the core locations for each cross section is presented in Figure 3.12.

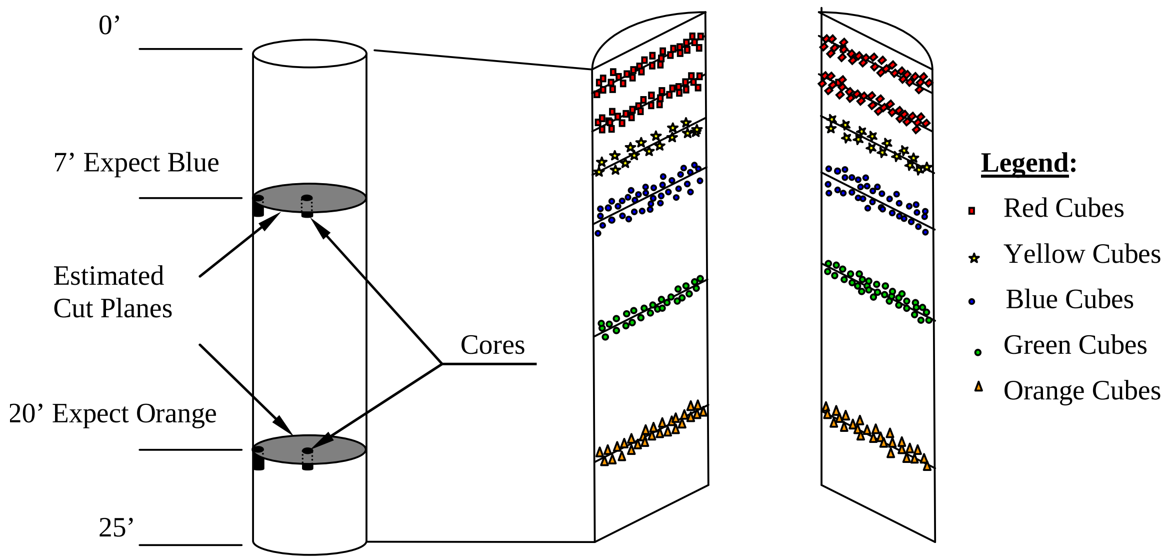



**Figure 3.11:** Cutting and coring locations of exhumed shafts (Dachelet 2008)

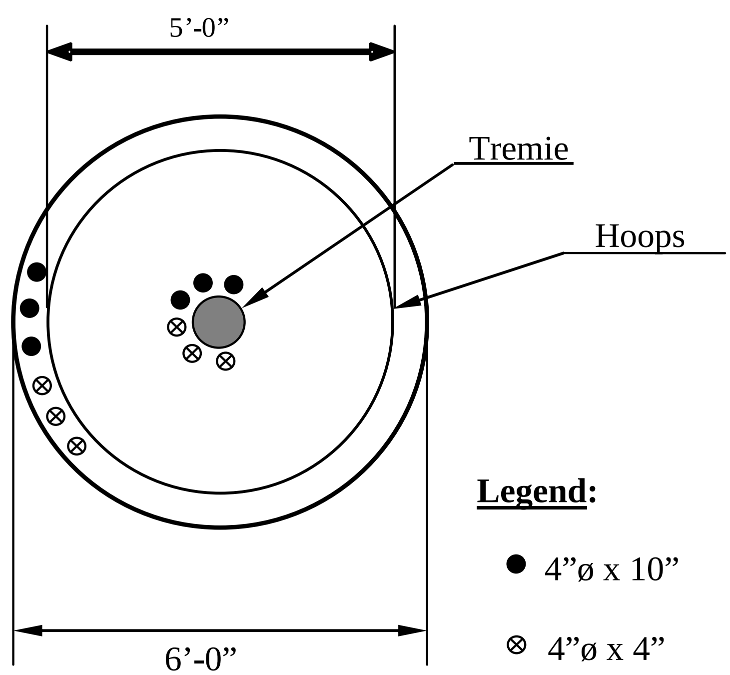



**Figure 3.12:** Coring detail of exhumed shafts (Dachelet 2008)

#### 3.2.7.1 Compressive Strength and Elastic Modulus

To determine the compressive strength and elastic modulus of the in-place concrete, three core specimens, 4 in. diameter by 10 in. high, were acquired. The compressive strength was tested in accordance with ASTM C39 (2005), and the modulus of elasticity was tested in accordance with ASTM C469 (2002). These cores were taken from the bottom side of each cross-sectional cut. After exhuming, the concrete samples were placed into sealed plastic bags. Each bag was sealed in another bag to ensure an airtight seal in retained. The samples were removed from the bags no earlier than two days before testing. The samples were trimmed to a length of 8 in. using a water-cooled, diamond saw. These cut samples were placed back into plastic bags and afterwards capped with sulfur mortar in accordance with ASTM C 617 (2003). The cores were tested at a concrete age of 56 days.

#### 3.2.7.2 Permeability

To assess the permeability of the in-place concrete, three core specimens, 4 in. diameter × 4 in. tall, were recovered for testing in accordance to ASTM C 1202 (2005). These cores were also taken from the bottom side of each cross-sectional cut. After exhuming, the concrete samples were placed into two sealed plastic bags. The samples were removed from the bags no earlier than two days before testing. The samples were trimmed to a length of 2 in. using a water-cooled, diamond saw. A sanding block was used to smooth blemishes around the circumference on the sample. These samples were tested at a concrete age of 91 days.

## 3.3 Materials and Mixture Properties for Experimental Shafts

The Ordinary Drilled Shaft Concrete (ODSC) is the standard mixture ALDOT used in the drilled shafts of the previous phase of the Scottsboro bridge project. The research team had not input on the selection of the proportions for the ODSC. Two SCC mixtures for evaluation were designed by Auburn University. 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. The following raw materials were used in the concretes:

- 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.
- 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 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 stability and passing ability of the concrete.

**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 |

**Table 3.2:** Cement chemical composition

| 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 |

- Fine aggregate: The fine aggregate used for this project was supplied by Madison Materials, in Summit, Alabama. This aggregate met ALDOT and ASTM gradation requirements for concrete sand.
- 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) specially obtained from Canada for this project. The use of this material was also expected to increase the mixtures resistance to segregation (Khayat et al. 2006) and lower the heat of hydration of the concrete.
- 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.
- High-range water reducing (HRWR) admixture: The SCC and the SCC-LP mixtures used ADVA® 380 as the HRWR admixture.
- Hydration-stabilizing admixture: Recover® was used as the Hydration Stabilizing admixture in both the SCC and the SCC-LP mixtures. This type of admixture is often use in drilled shaft construction as it extends the setting time much more than convention retarding admixtures.
- Air-entraining admixture: Daravair® 1000 was used as the air-entraining admixture in the ODSC. No air-entraining admixture was use in the SCC mixtures, because the entrapped air content of these concrete were such that their total air content met ALDOT's drilled shaft specification requirement by being between 2.5 to 6.0 percent.

## 3.4 OVERVIEW OF CONSTRUCTION

An overview of the construction process used to construct and test the experimental shafts are provided in this section.

### 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 experimental shafts. A different concrete mixture was placed into a different shaft on each day. To assist with shaft removal, a corrugated steel casing was placed into each hole, instead of the proposed sono-tube. Approximately one foot of concrete was placed within each test shaft as presented in Figure 3.13.

The experimental 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 constructed first. The middle shaft was designated the SCC shaft and was constructed second. The final shaft filled was the westernmost shaft, and this shaft was designated the SCC-LP shaft.

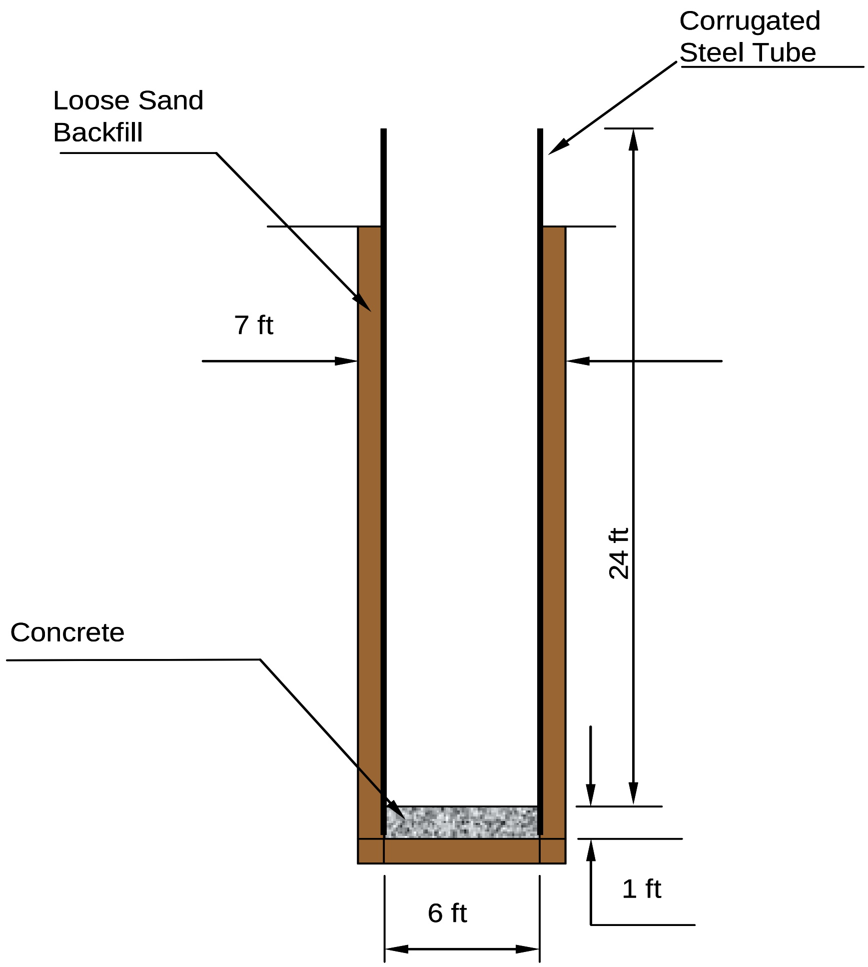


**Figure 3.13:** Schematic of open shaft before test concrete placement

### 3.4.2 Steel Reinforcement

The steel reinforcement used was the same in each shaft and was designed to be representative of Alabama drilled shaft construction. Each cage consisted of 26, No. 11 bars running longitudinally, 6 CSL tubes equally spaced, and No. 4 hoops at 4 in. center-to-center spacing. The CSL tubes had an inside diameter of approximately 1.75 in. Four additional threaded No. 11 bars were added to extend a few feet from the shaft top to assist with lifting each shaft after hardening. A diagram of the cross section and reinforcement layout, is presented in Figure 3.14.

### 3.4.3 Slurry Mixing

The experimental shafts were neither below the water table, nor capable of collapsing; however, drilling slurry was used to simulate the placement methods used in the production drilled shafts. For this project, Poly-Bore™ 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 1 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.

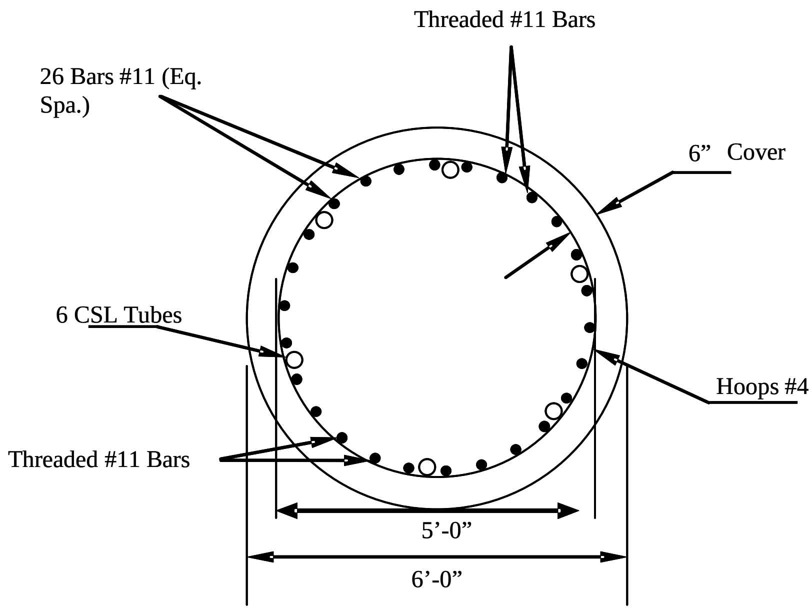



**Figure 3.14:** Shaft cross section

### 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 added to 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 Ordinary Drilled Shaft Concrete (ODSC) Placement

This shaft was poured on August 11, 2008. The average ambient 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, which served as the tremie for these experimental shafts. 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 resting 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 down the tremie.


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

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.

An intentional one-hour long delay was introduced after the second truck to extend placement, to get conditions more representative of construction times in large shafts. 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.

**Figure 3.16:** Bottom of tremie pipe

**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 shaft was constructed 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)

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 used for the ODSC shaft was used for this shaft.

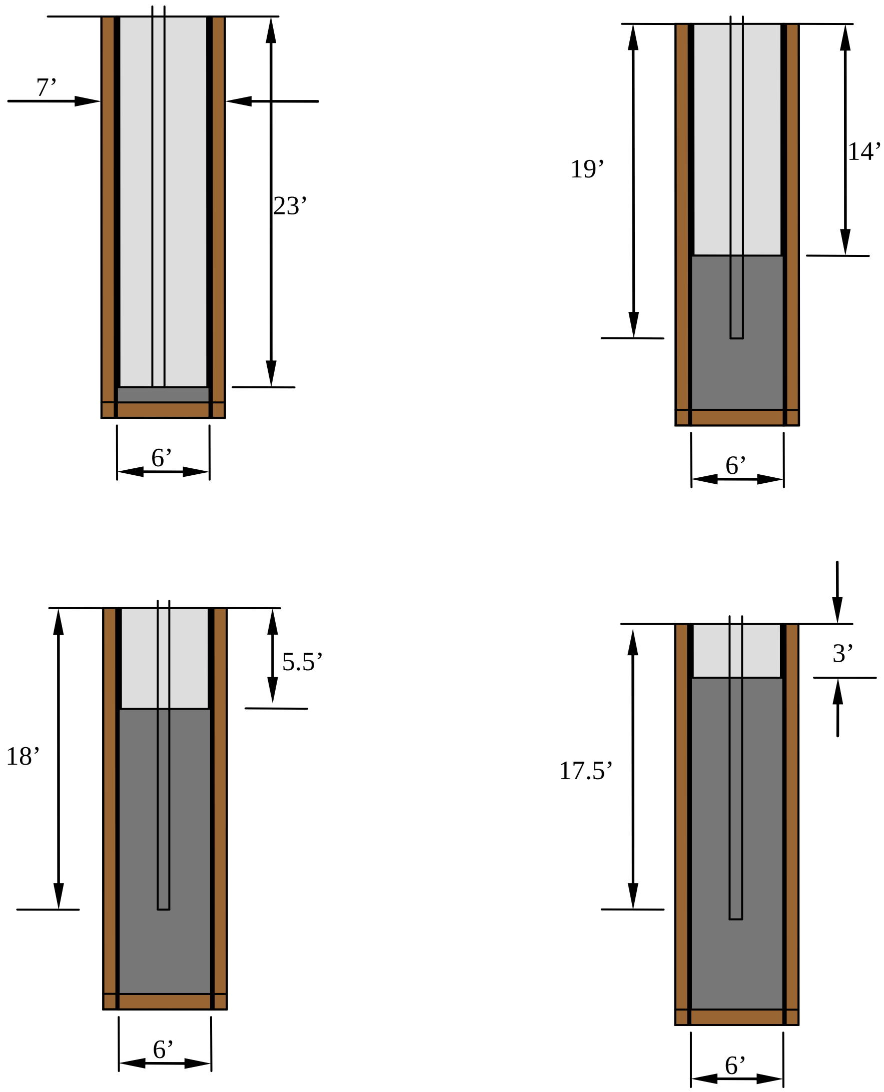



**Figure 3.17:** ODSC tremie movement

**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 (2005).

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 vertical rise of the tremie was recorded and is presented in Figure 3.18.

### 3.4.7 Overview of the SCC with Limestone Powder Placement

The shaft made with SCC and 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).

It should be noted that only a limited amount of limestone powder was acquired and because of this the ready-mixed concrete producer could not perform any trial batches with this mixture. The limestone powder was also manually added to the concrete truck once it came out of the batch plant. Both these issues significantly influenced the ability of the concrete producer to produce concrete with limestone powder.

The first truck was batched at 9:35 a.m. with only 3.5 cubic yards of concrete. However, placement of this truck was delayed until it could be confirmed that the second truck was available and on its way. The second truck batched was rejected because the slump flow was too low. The third truck to arrive on site, now designated as Truck No. 2, required four attempts to get the concrete mixture's slump flow within the specified range. The fourth truck to arrive on site, now designated as Truck No. 3, also required additional HRWR admixture before it had sufficient slump flow. Arrival and pour times are summarized in Table 3.5.

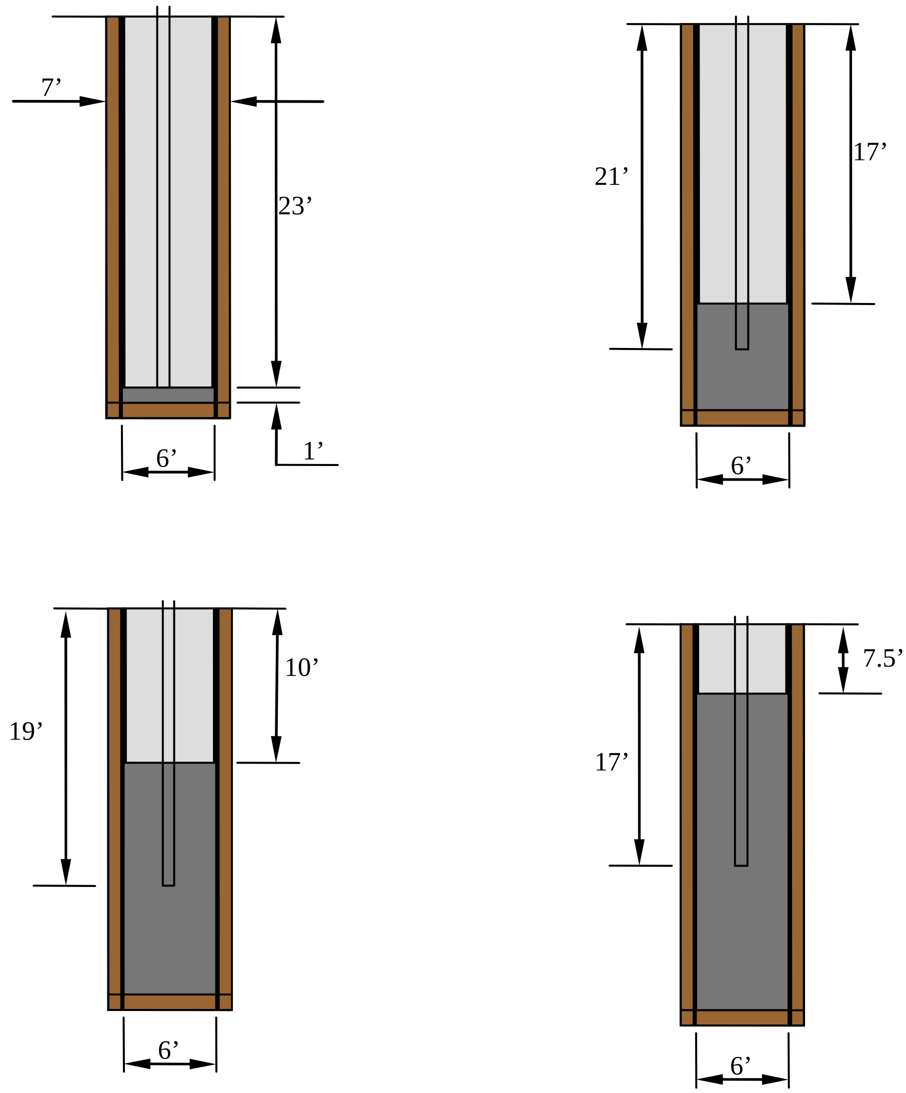



**Figure 3.18:** SCC tremie movement

**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 concrete sampled from this truck:

- Bleed Test,
- Pressurized Bleed Test,
- Segregation Column,
- Modified J-Ring, and
- Slump Flow Retention.

During concrete placement, the tremie was purposely not moved, but it rose throughout the pour due to concrete being discharged from its bottom. The tremie rise was recorded and this is presented in Figure 3.19.

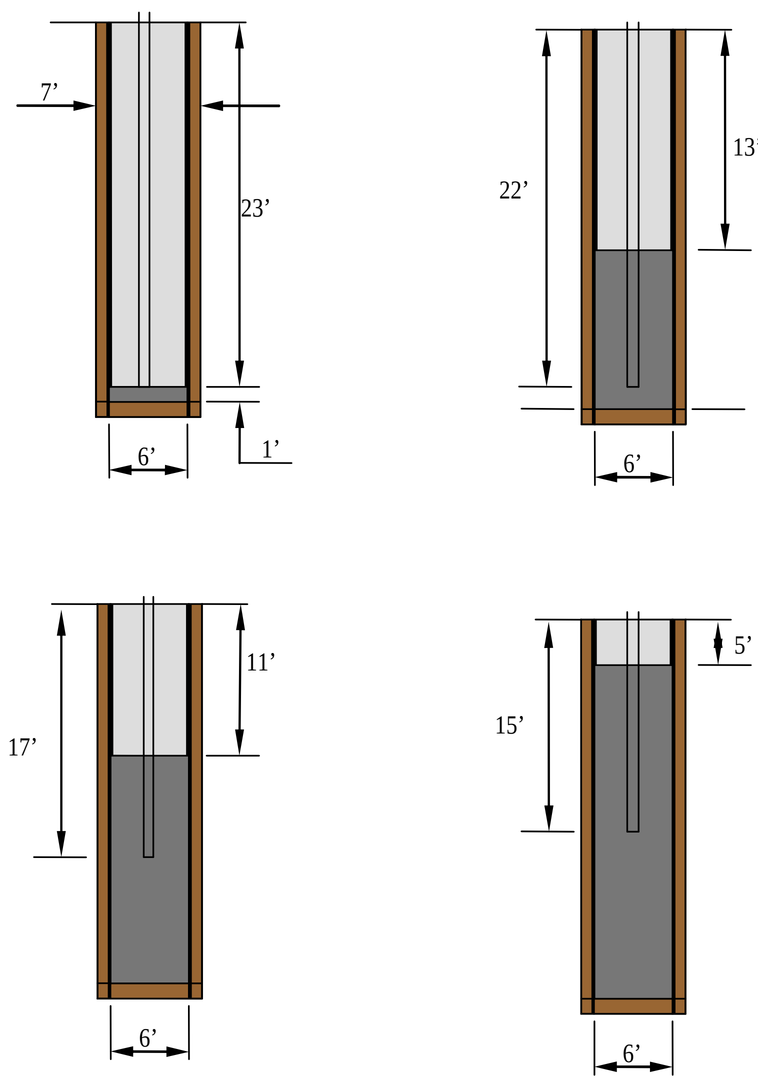



**Figure 3.19:** SCC-LP tremie movement

### 3.4.8 Addition of Colored Mortar Cubes

The time at which the colored mortar cubes were added 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.

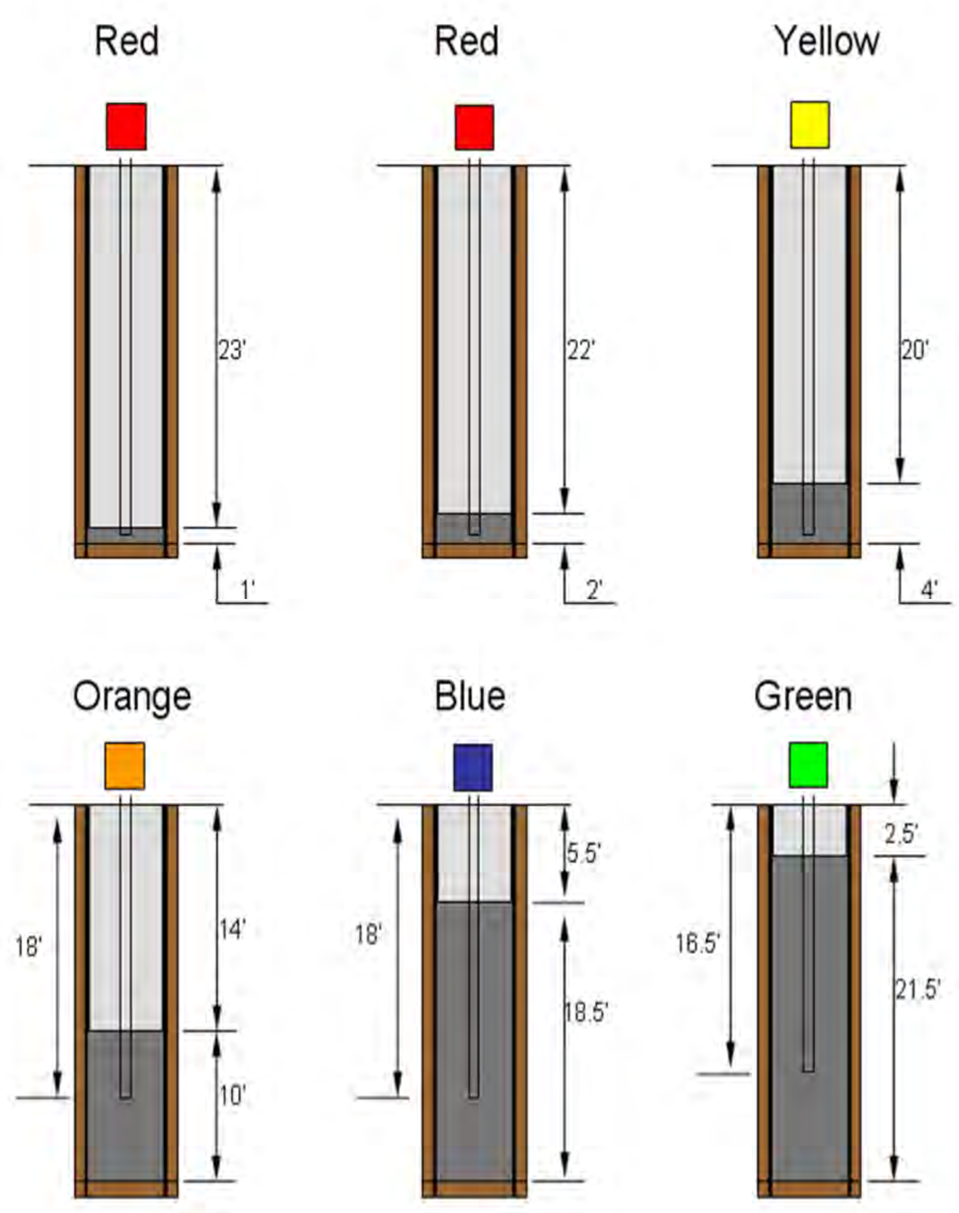



**Figure 3.20:** Addition of cubes during ODSC shaft placement

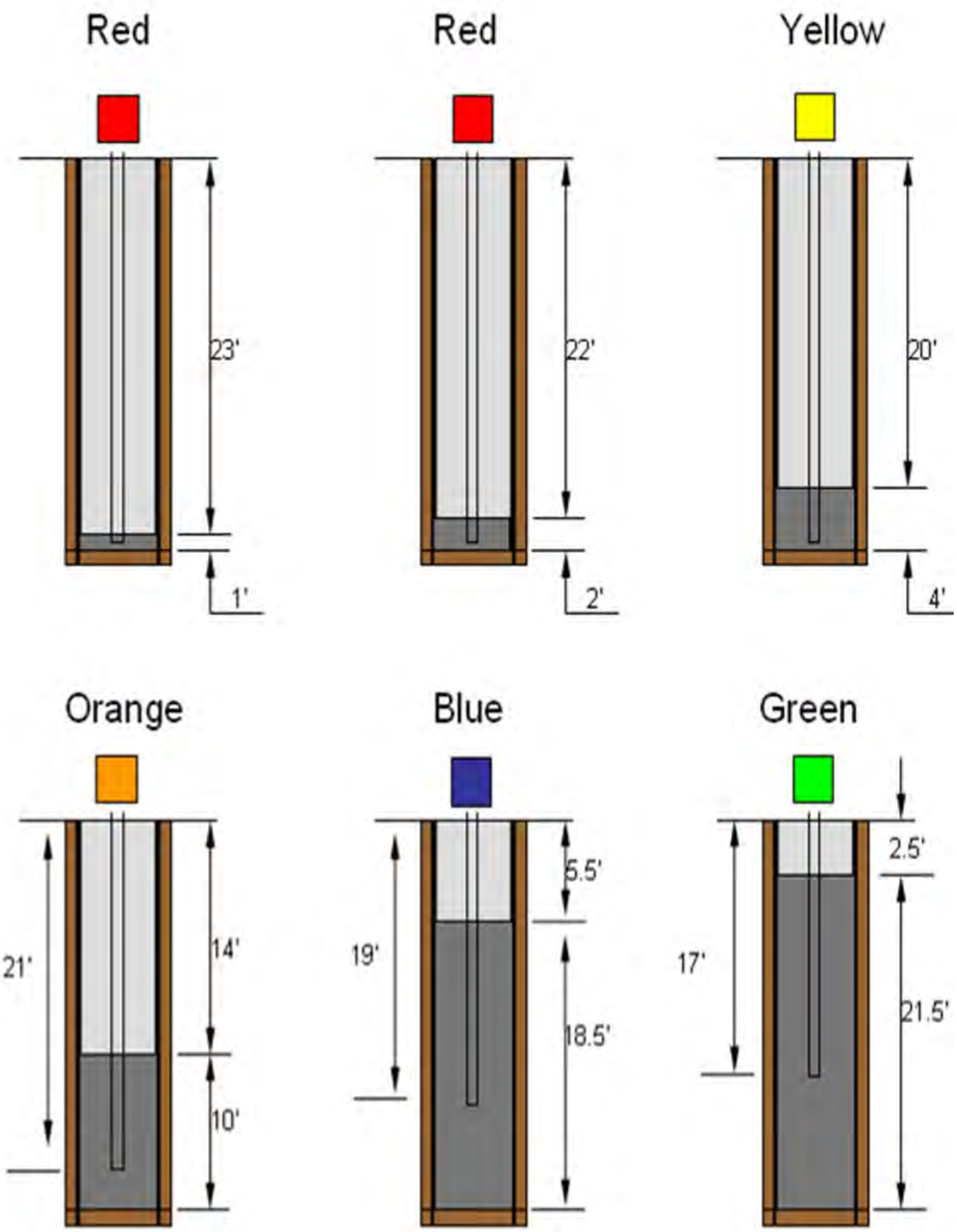



**Figure 3.21:** Addition of cubes during SCC shaft placement

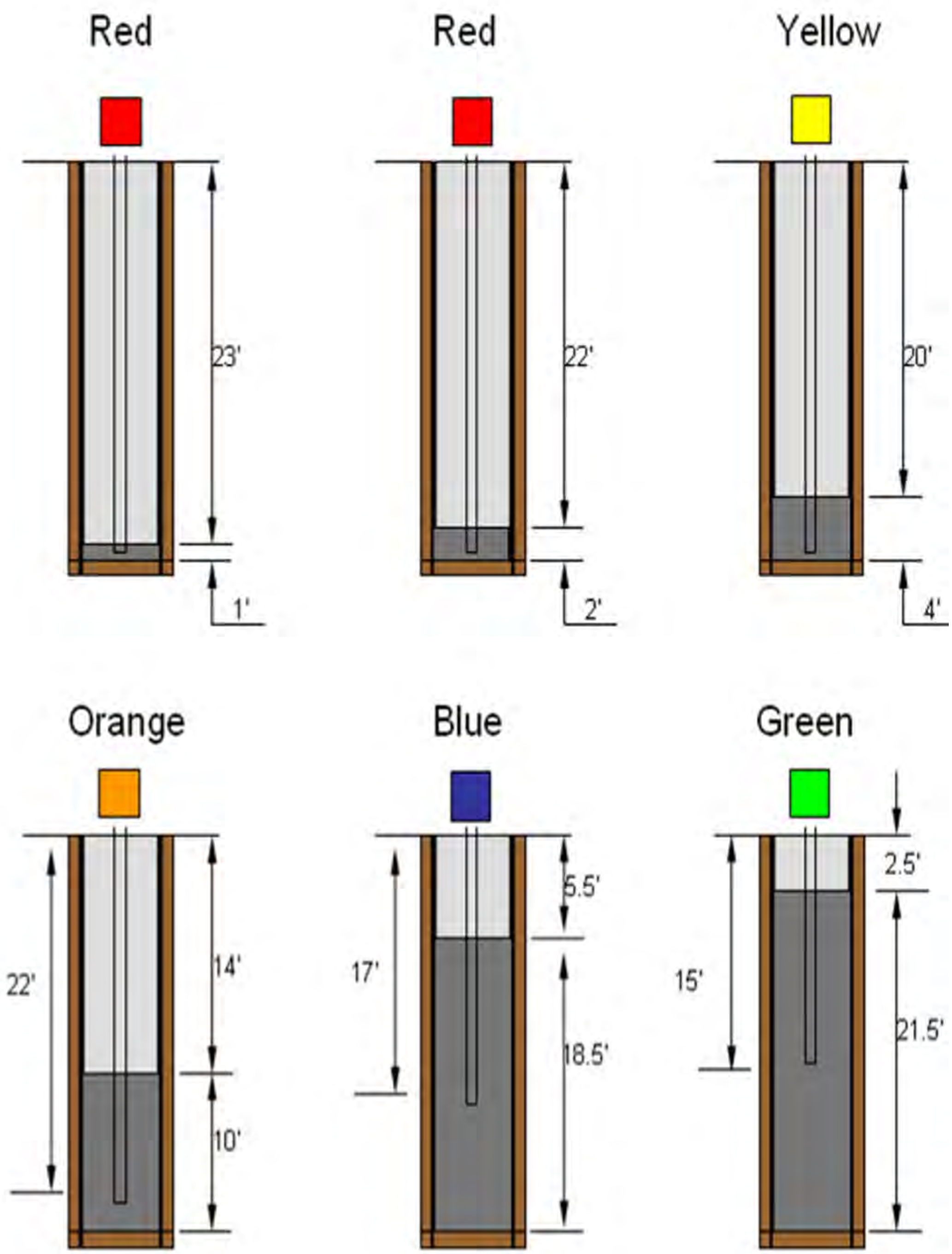



**Figure 3.22:** Addition of cubes during SCC-LP shaft placement

### 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 recorded. The recorded elevations plotted against shaft depth for all three shafts are summarized in Figure 3.23. The measured differences between the cover elevation and interior elevation are summarized in Figure 3.24.

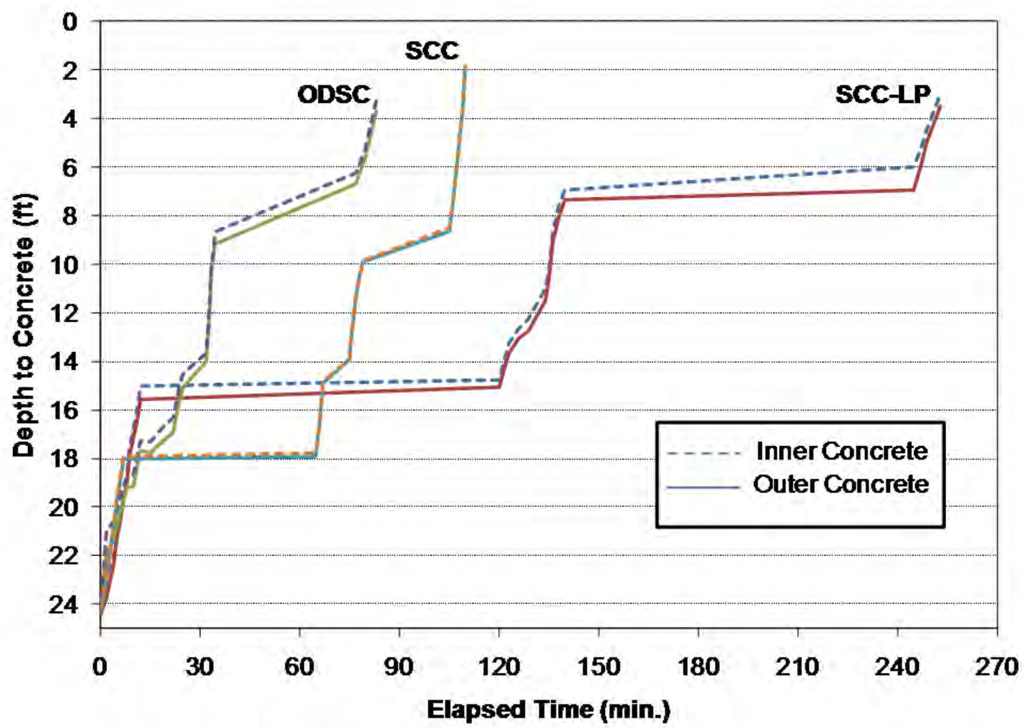



**Figure 3.23:** Concrete height measured throughout the pour

The placement of the ODSC shaft 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 that lasted longer than one hour and occurred after the first truck. The SCC-LP shaft placement occurred over the longest 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, due to a lack of trucks dedicated to this placement.

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: 3.24 and are displayed by the parts in the graph where there is a slight increase in elevation over a time of 30 minutes or more.

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 than 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. Hodgson et al. (2003) concluded that the uniform upward flow of the SCC should prevent debris from being entrapped against the side of the shaft.

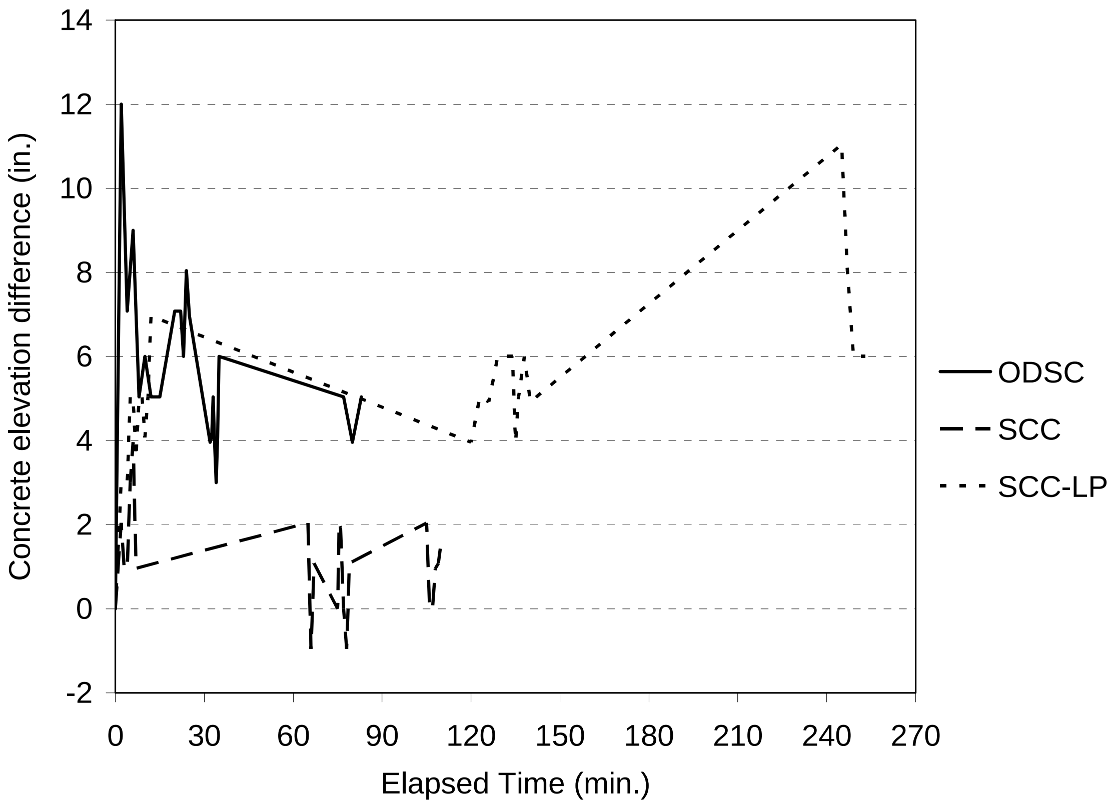



**Figure 3.24:** Elevation differences during concrete placement

### 3.4.10 Shaft Integrity Testing

Crosshole sonic logging (CSL) of the experimental 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 Experimental Shafts

On September 10, 2008, the experimental 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 used to remove the loose sand from 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 shown 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 Experimental 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. The following 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 7 ft and 20 ft 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 7 ft from the top of the shaft, is presented in Figure 3.31.

**Figure 3.29:** Longitudinal cutting of shafts

**Figure 3.30:** Diamond wire ready for cross-sectional cut

**Figure 3.31:** Coring SCC 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 SCC shaft section (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.

#### 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 6.0 percent). 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. The 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.

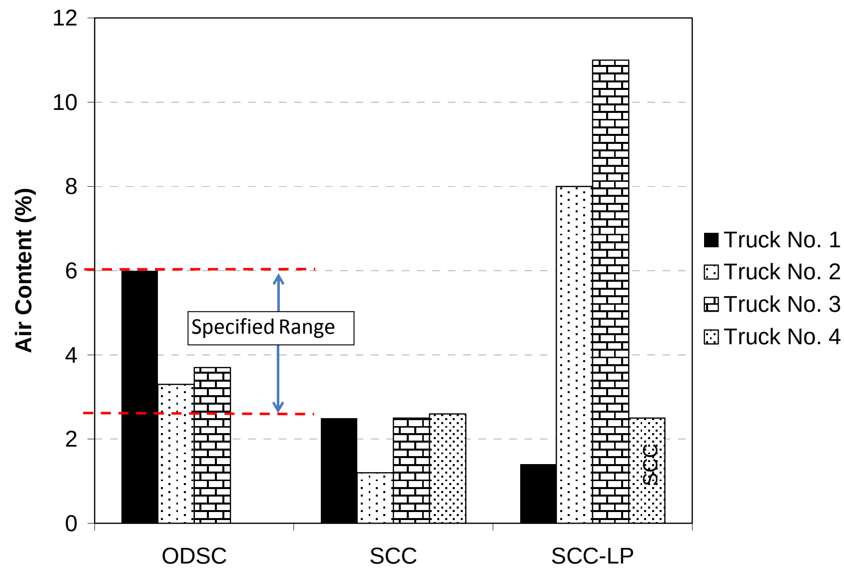



**Figure 3.33:** Air content test results

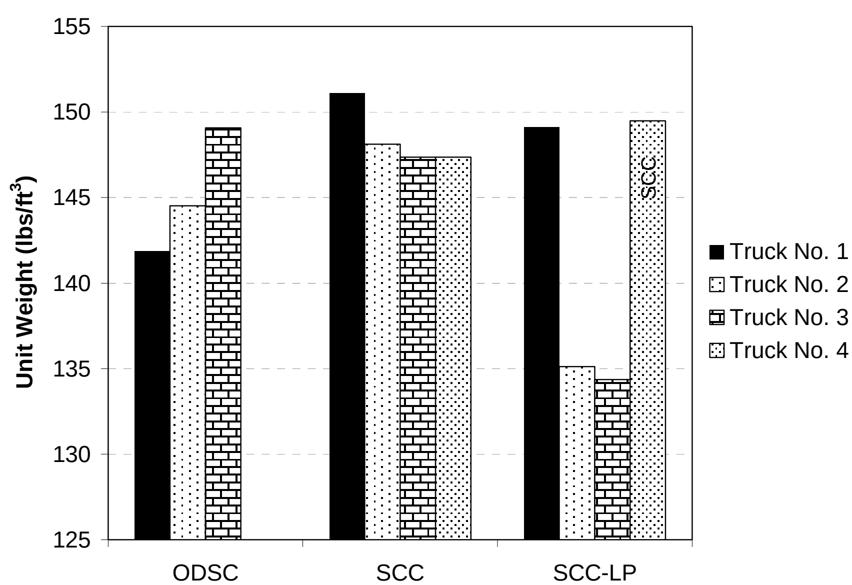



**Figure 3.34:** Unit weight test results

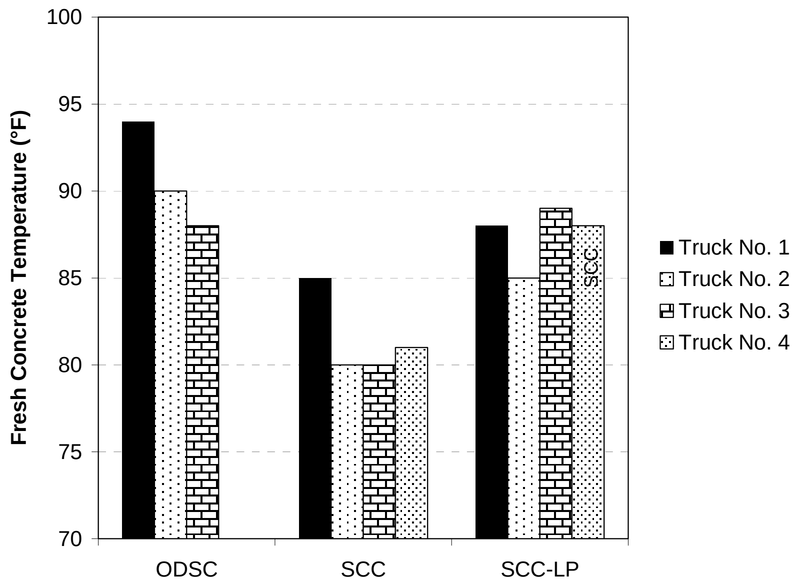



**Figure 3.35:** Fresh concrete temperature results

#### 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 ready-mixed concrete truck. The SCC mixture's slump flow varied throughout the concrete placement. Two of the loads from this mixture exceeded the project slump flow specification; however, the first load tested past the specification and this would normally be the only load tested by ALDOT technicians. The SCC-LP concrete slump flow met the project specification for all the loads placed.

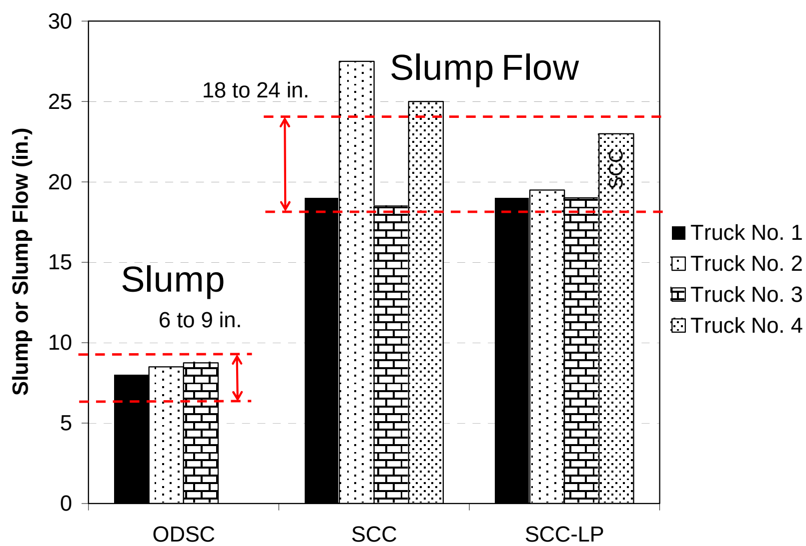



**Figure 3.36:** Slump and slump flow results

#### 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 truckload of each SCC and SCC-LP mixtures, respectively. Results from this test are presented in Table 3.6.

**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 |

The passing ability of the samples was calculated by subtracting the Modified J-Ring results from the slump flow results (ASTM C 1621 2006). 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 1621 (2006) specification. The blocking assessment table from this ASTM specification is presented in Table 3.7. From this test, the SCC-LP mixture was determined to have minimal to noticeable blocking, whereas, the SCC mixture had noticeable to extreme blocking.

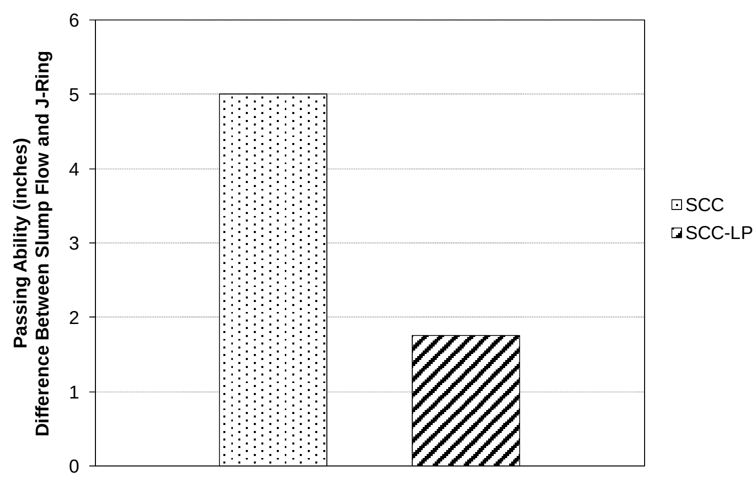



**Figure 3.37:** Passing ability of SCC and SCC-LP mixtures

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 3.23, 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 from the J-Ring test that is applicable to drilled shaft applications.

**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 truckload 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 percent, whereas the SCC-LP batch had a static segregation index of 3.1 percent. The SCC mixture's segregation is not considered acceptable by the comments offered by ACI Committee 237 (2007), which states that the percent static segregation should be below 10 percent. 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 values of 6 percent and 7 percent for the SCC and SCC-LP mixtures, respectively (Dachelet 2008).

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 required that the SCC and SCC-LP mixtures must have VSI ratings less than or equal to 1.5 (Appendix A).

**Figure 3.38:** Segregation column

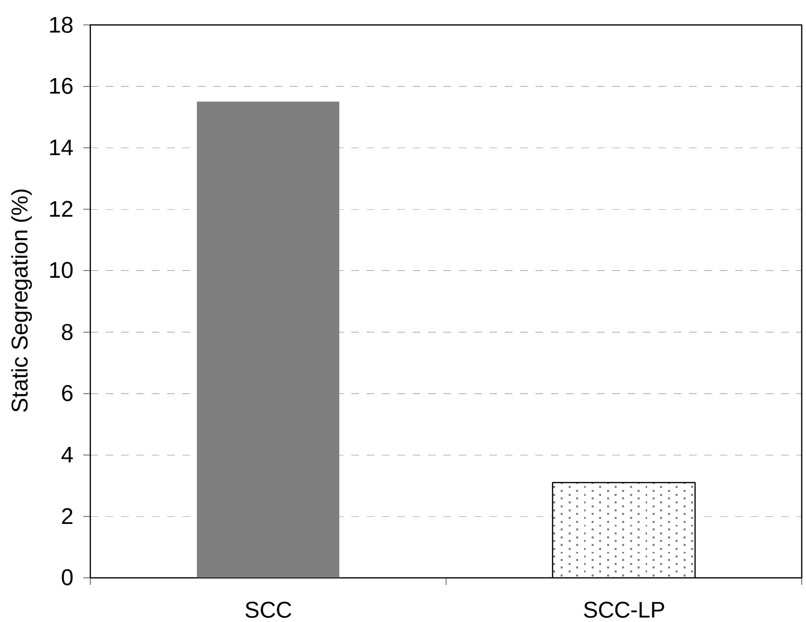



**Figure 3.39:** Segregation column results

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 with VSI ratings were 0, 1, or 1.5 which are all considered to be stable, meeting the project specification.

**Table 3.8:** Visual Stability Index values (ASTM C 1611 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 ≤ 0.5 in.(≤ 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, the time required for the concrete to flow to a 20 in. diameter (50 cm), refer to as the $T_{50}$ time, was recorded. 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. The SCC mixture had one batch with a relatively long $T_{50}$ time, which corresponds with the lowest slump flow tested for this mixture.

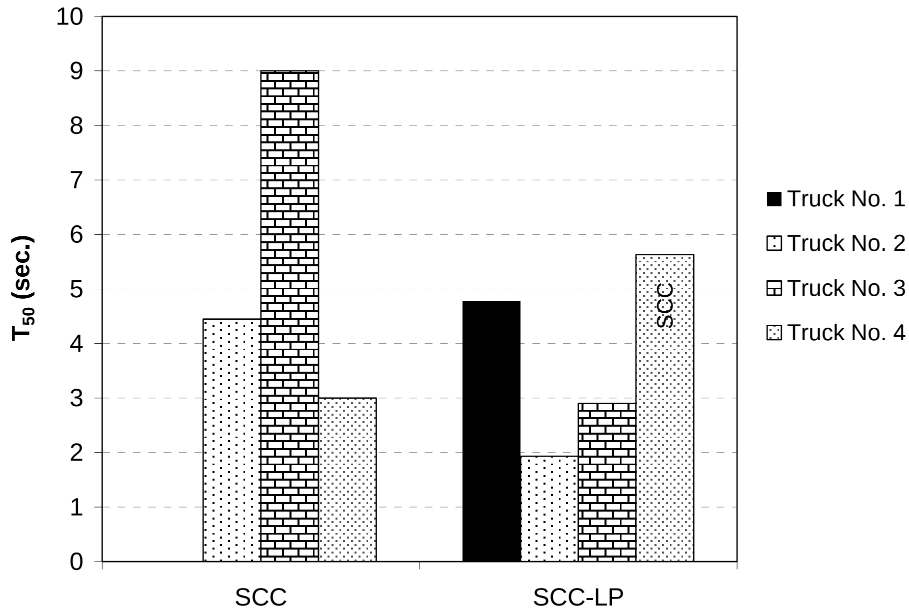



**Figure 3.40:** Recorded $T_{50}$ times

#### 3.5.1.5 Assessment of the Concrete's Workability Retention

Setting tests by the penetration resistance method were performed on a sample of concrete obtained from the second truck of the ODSC mixture and the third truck of the SCC and SCC-LP mixtures. These results are summarized 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 10 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.

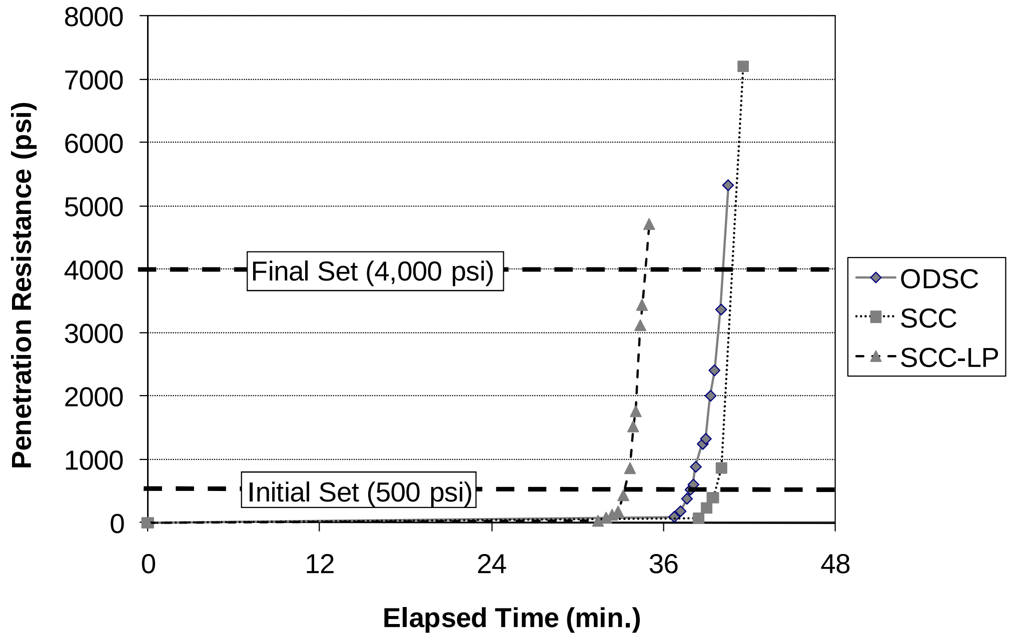



**Figure 3.41:** Setting by penetration resistance results

**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 4 in. slump for the duration of the placement. The SCC and SCC-LP mixtures had slump flows of 17 in. and 17.5 in. at the conclusion of the concrete placement, respectively.

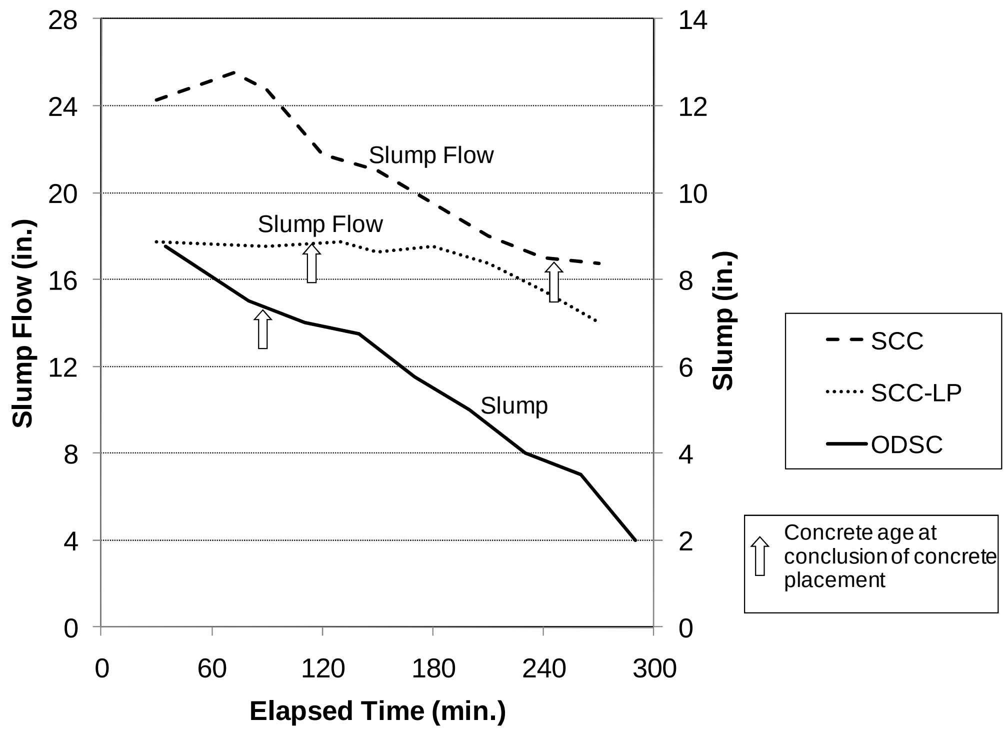



**Figure 3.42:** Slump/slump flow retention results

#### 3.5.1.6 Concrete Bleeding

##### 3.5.1.6.1 Conventional Bleed Test

The bleed test was conducted on a sample of concrete from the second truck of the ODSC and SCC mixtures and the third truck of the SCC-LP mixture. 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 steadily bled water to reach its maximum bleed water of 118 milliliter. 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 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 laboratory testing demonstrated less bleeding in the SCC mixture than that of a similar ODSC mixture (Dachelet 2008). Note that 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, which explains the discrepancy in bleeding results obtained from the laboratory and field tests.

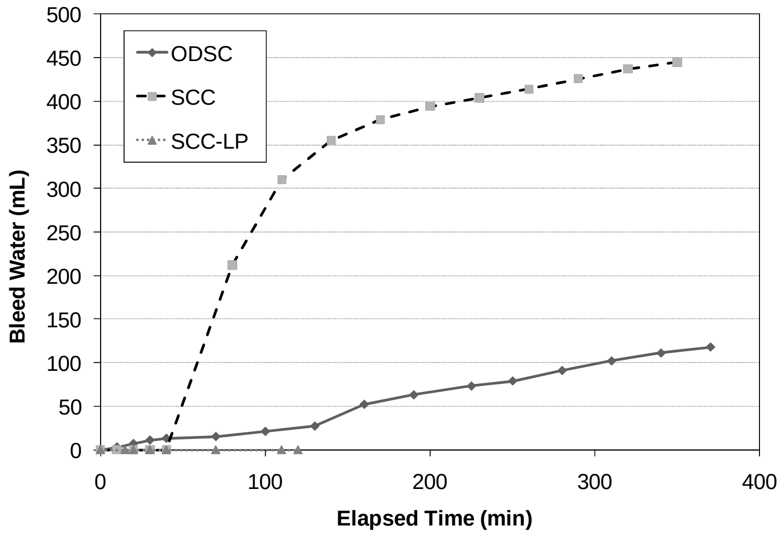



**Figure 3.43:** Conventional bleed test results

**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 |

**Figure 3.44:** SCC bleed water after 80 minutes

#### 3.5.1.6.2 Pressurized Bleed Test

**This test was developed during this project and is not standardized in any specification. This 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 Figures 3.46 and 3.47Figure 3.47:** SCC pressure bleed test results

. The amount of bleed shown on the right hand side of the graph is in percent of free water available the concrete.

**Figure 3.45:** Pressurized bleed test being conducted

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 and controlled enough, due to the imprecision of the adjuster knob on the air compressor. When the target pressure was exceeded, the gauge pressure was recorded, and the chamber was kept at this recorded pressure for its duration.

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. Considering the precision of this test, the amount of bleed water obtained from both these concretes are considered very similar. The results for the SCC-LP mixture are not shown, because the test apparatus was faulty at the time of testing.

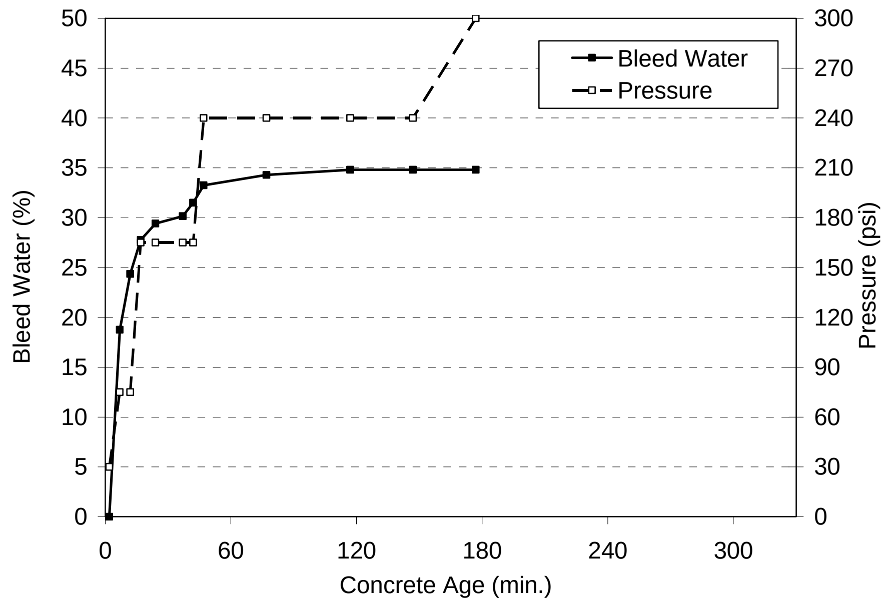



**Figure 3.46:** ODSC pressure bleed test results

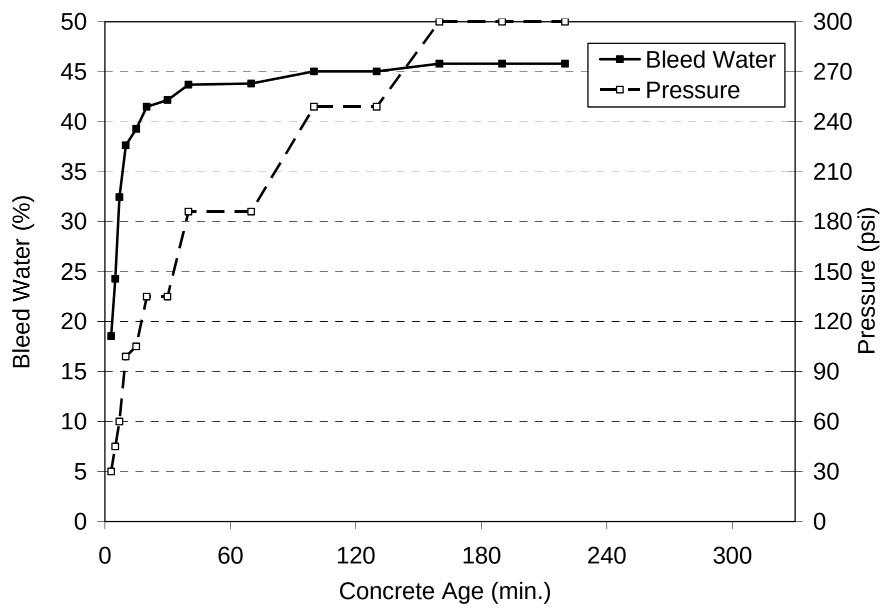



**Figure 3.47:** SCC pressure bleed test results

### 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 Figures 3.48 and 3.49, respectively, are averages of three cylinders per testing age.

The strength of all three the concretes exceeded 4,000 psi at 28 days. The SCC mixture had the highest compressive strength for each testing age, whereas the SCC-LP had the lowest compressive strength at each testing age. The low strengths of the SCC-LP mixture are attributed to the high air content of the concrete batch from which these cylinders were made. 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).

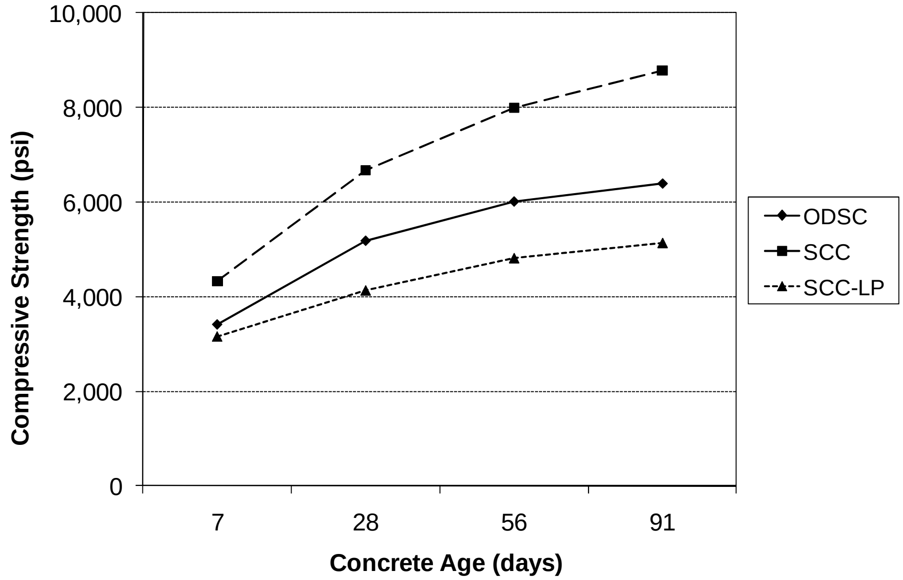



**Figure 3.48:** Molded cylinder compressive strength results

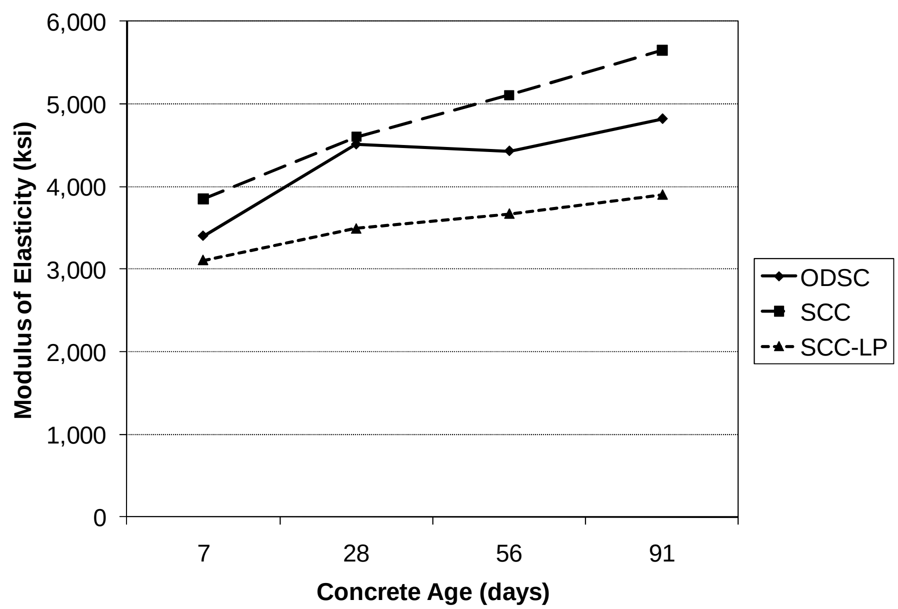



**Figure 3.49:** Molded cylinder modulus of elasticity results

#### 3.5.2.2 Resistance to Chloride Ion Penetration

The chloride ion penetration resistance test results 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 all the molded samples were very low.

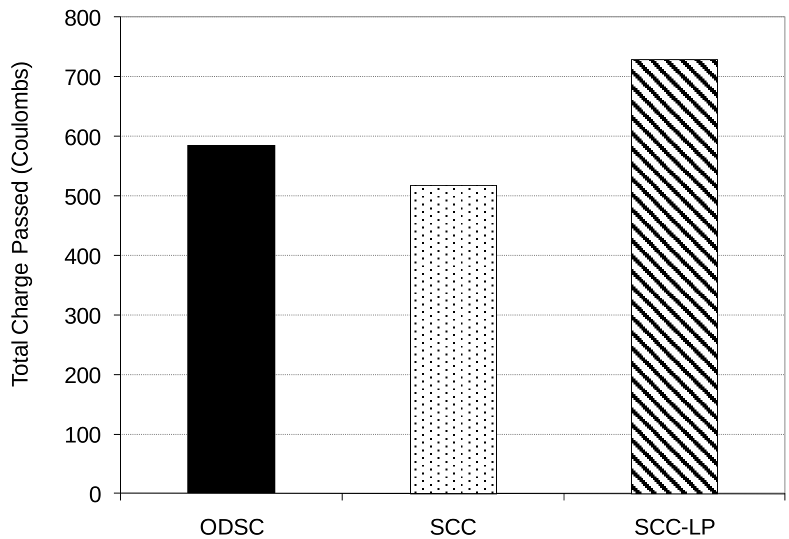



**Figure 3.50:** Molded cylinder 180-day chloride ion penetration results

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

| 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. Considering the precision of this test, the measured drying shrinkage appears similar for all the mixtures tested.

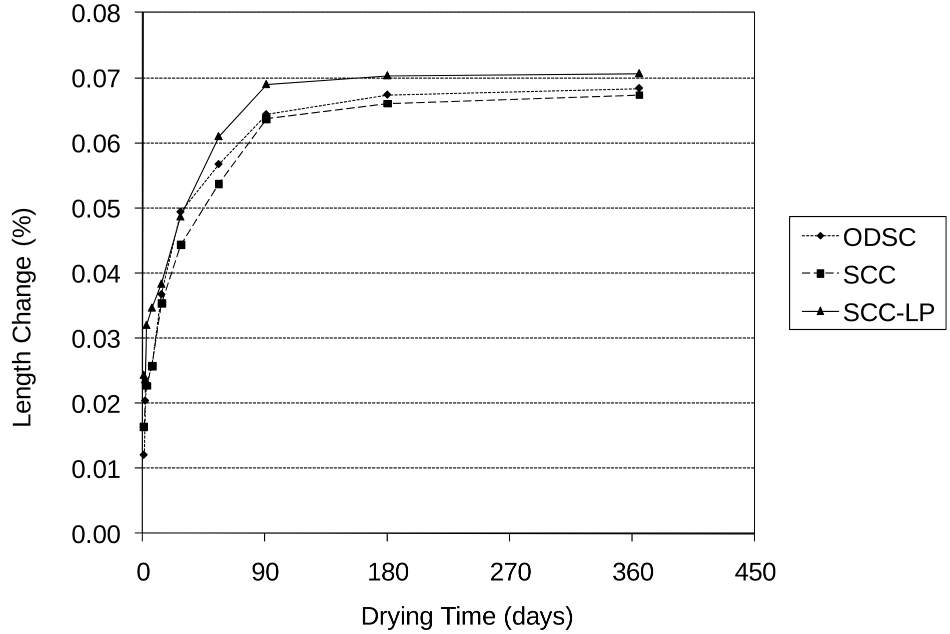



**Figure 3.51:** Drying shrinkage results

### 3.5.3 In-Place Shaft Integrity

Crosshole sonic logging (CSL) was conducted on the experimental 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 CSL test 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 provided by stating the tube pair and the depth from the surface.

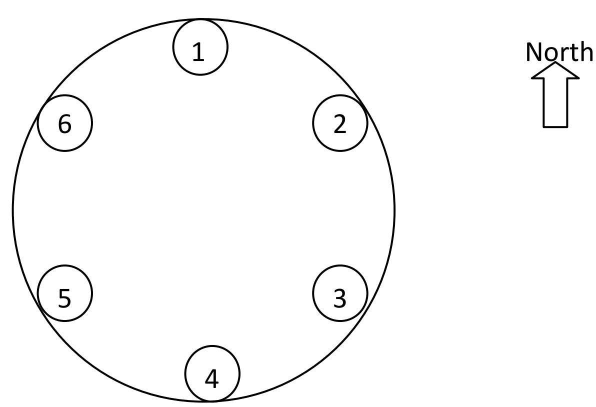



**Figure 3.52:** CSL tube numbering

#### 3.5.3.1 ODSC Shaft Results

No severe imperfections were detected in the ODSC shaft; however, four slight imperfections were observed. These imperfections were not large enough to cause any alarm, but for research purposes, they 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 detected in the SCC shaft; however, a slight imperfection was discovered in the shaft at tube pair 2-5, 20.4 feet from 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 were not large enough to cause any alarm, but for research purposes, they 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 crosshole sonic logging did not identify any major imperfections in the experimental 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).

The shale pieces purposely added to the shafts, were visible in the cut sections of the SCC shaft, but did not show up in the CSL results. 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 are in the slurry, within a shaft that is cased or bearing on a rock socket. 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 was assumed to flow with the slurry into the next shafts. Therefore, it would be expected that the sand accumulation would be less for each subsequent 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.

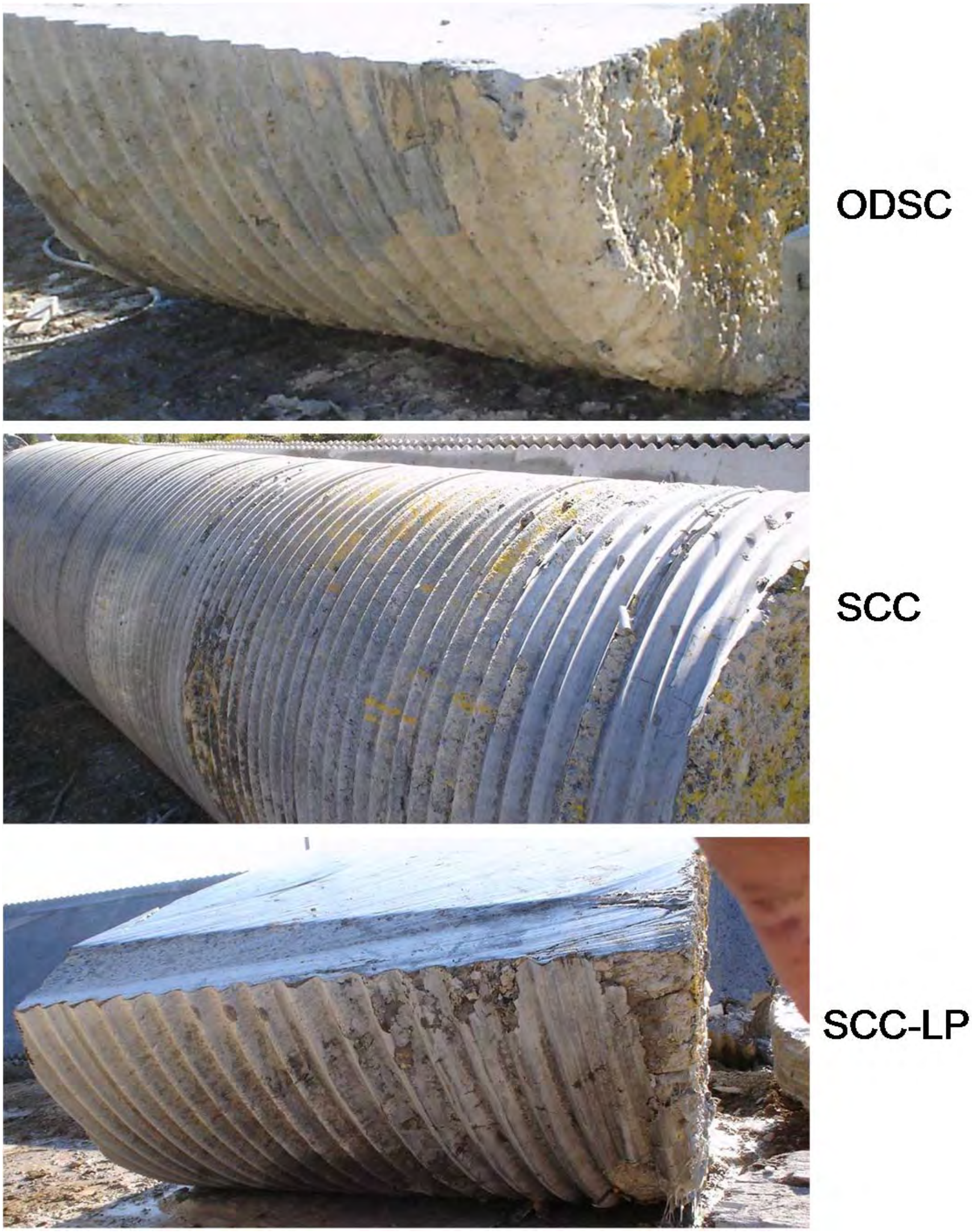



**Figure 3.53:** Outside surface of bottom of the shafts

#### 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. 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 a 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. The possible cause of this anomaly can be explained by looking at the inability of ODSC to heal back after encountering the vertical obstruction bars in a 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 present in the concrete.

(a) (b)

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

**Figure 3.55:** Longitudinal void located within the ODSC shaft

**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 referring to 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 as shown in 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 during moving the shaft around after excavation. A few bleed channels were also visible near the corners of the SCC-LP shaft as shown in Figure 3.60.

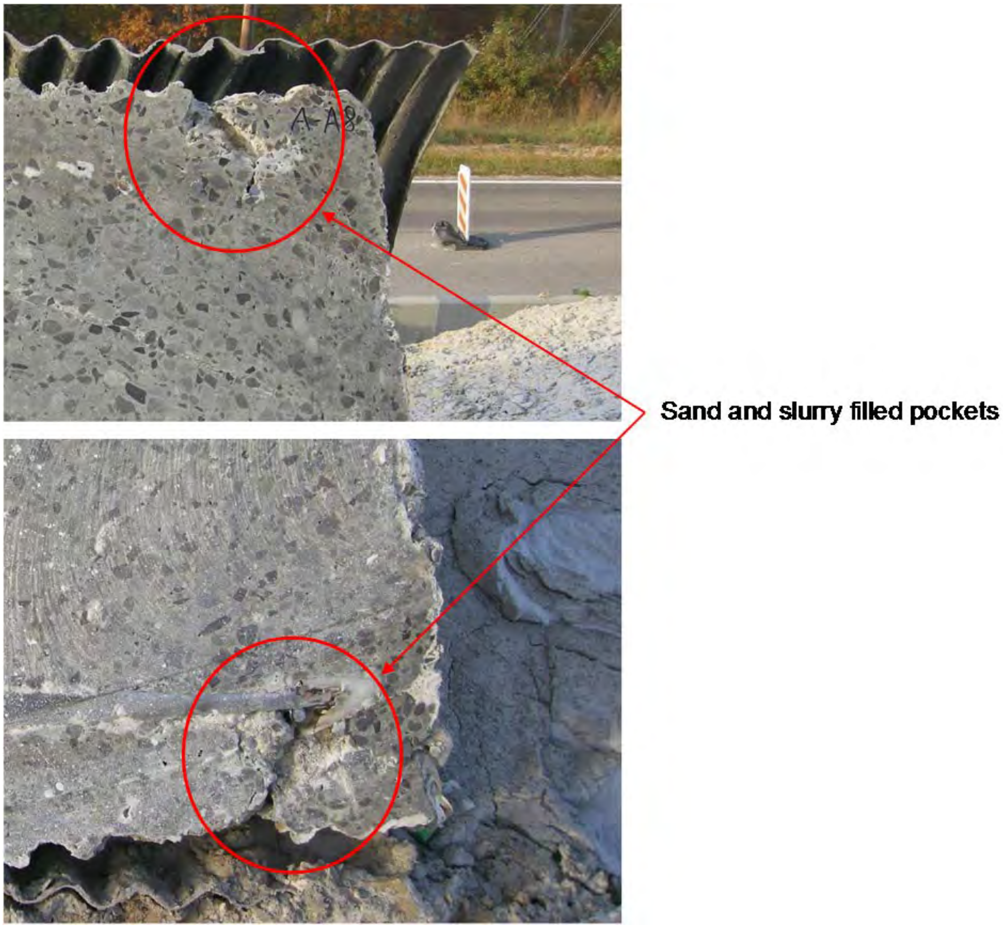



**Figure 3.58:** Bottom of ODSC shaft with sand and slurry filled voids

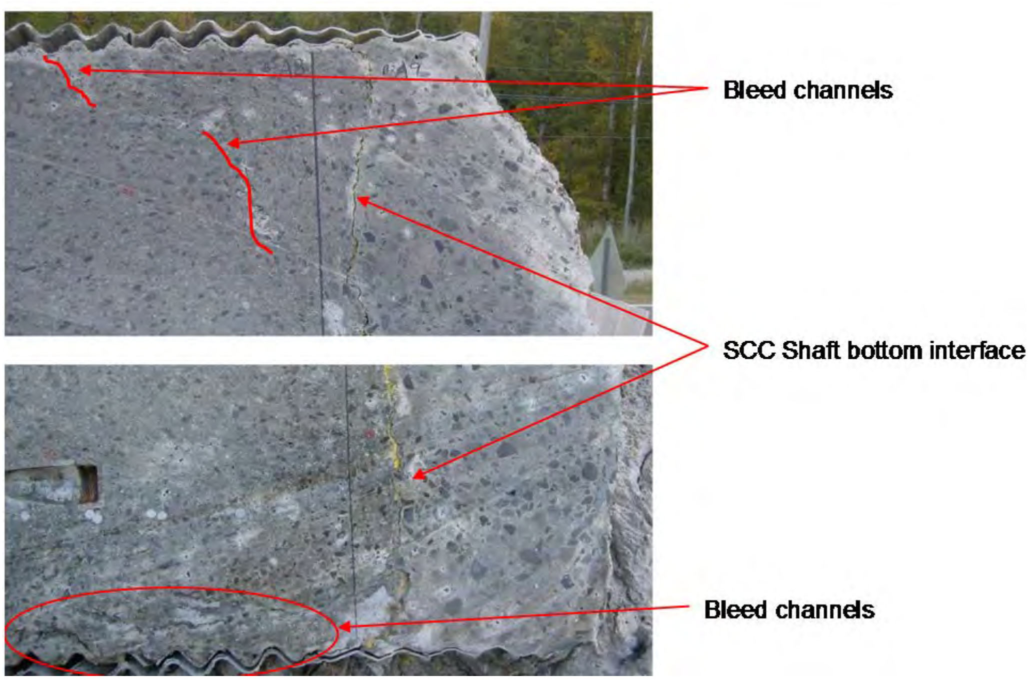



**Figure 3.59:** SCC bottom surface

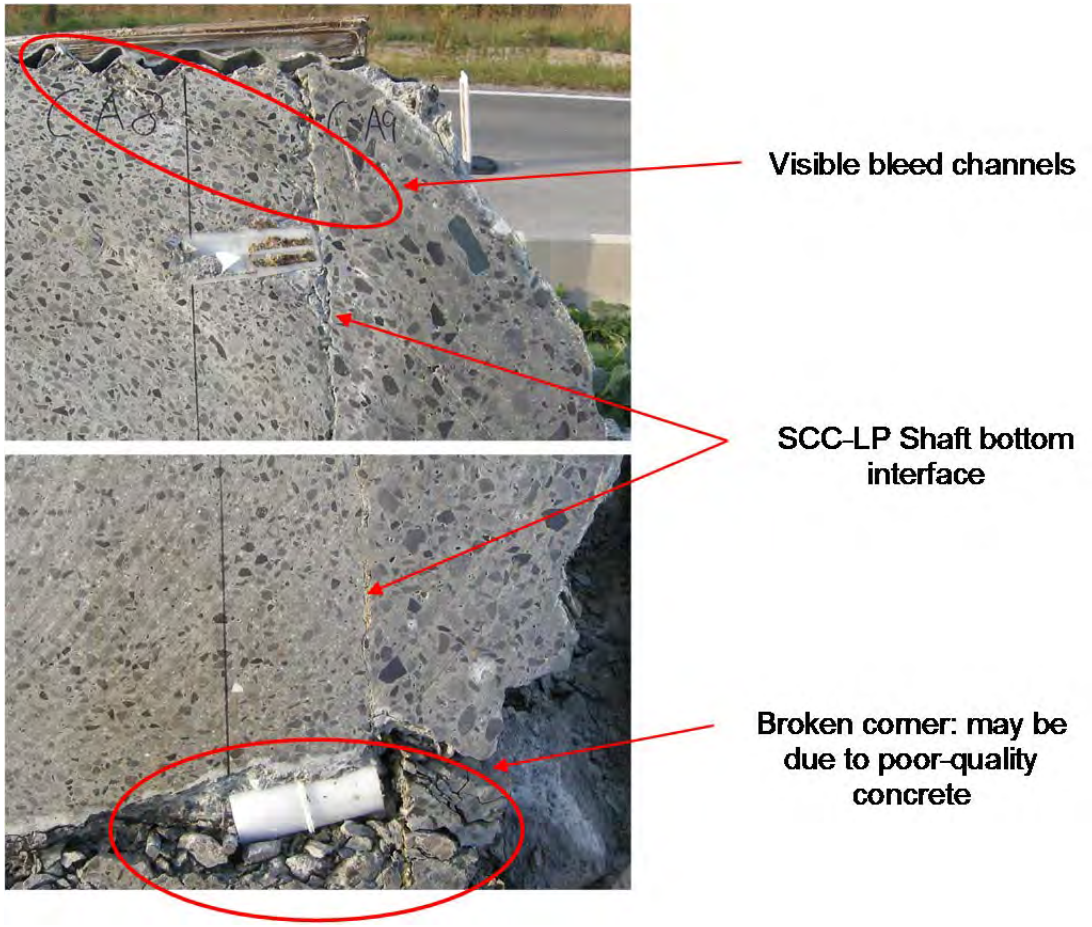



**Figure 3.60:** SCC-LP bottom surface

### 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, the number of cubes counted at each elevation is presented.

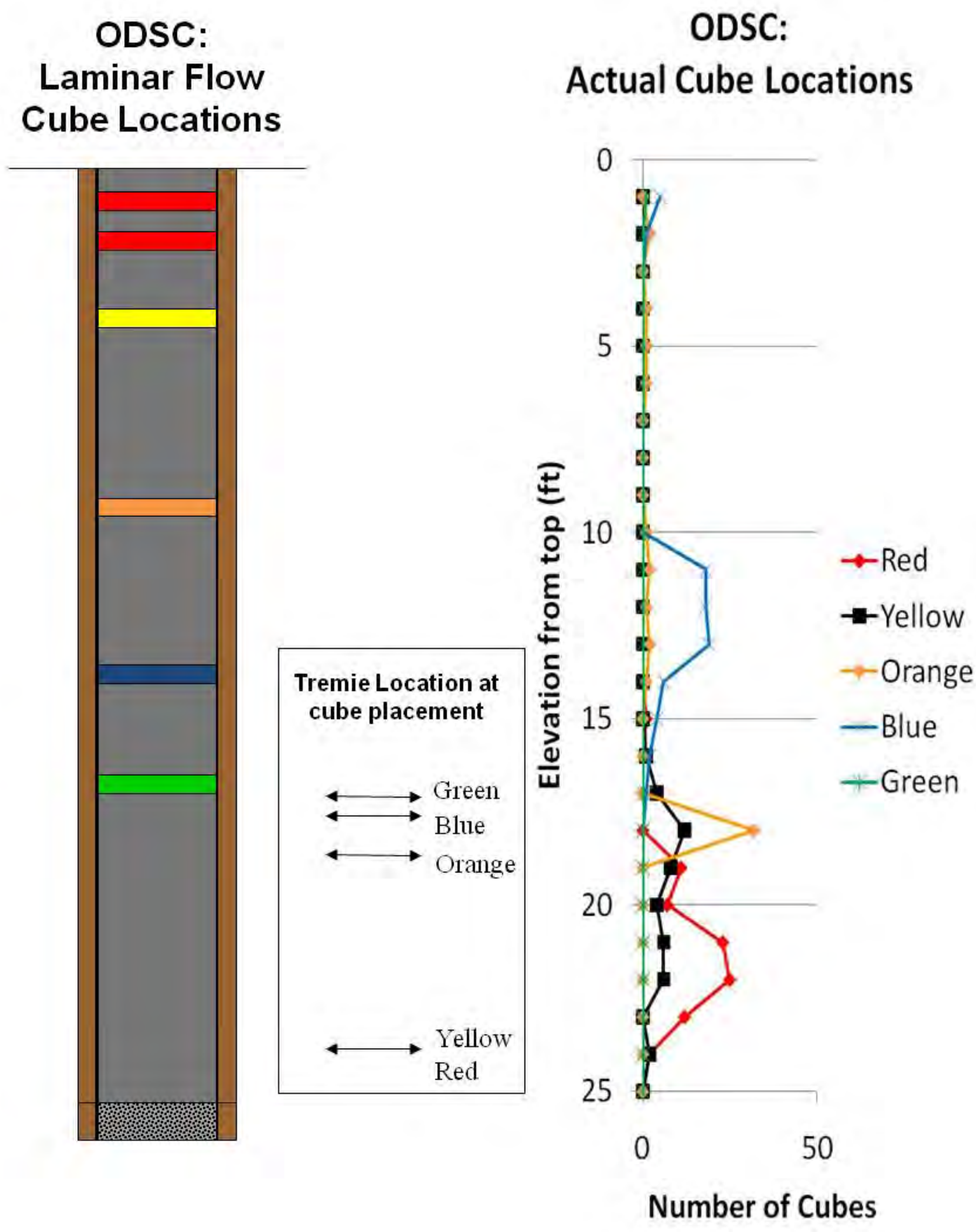



**Figure 3.61:** ODSC actual versus laminar cube location

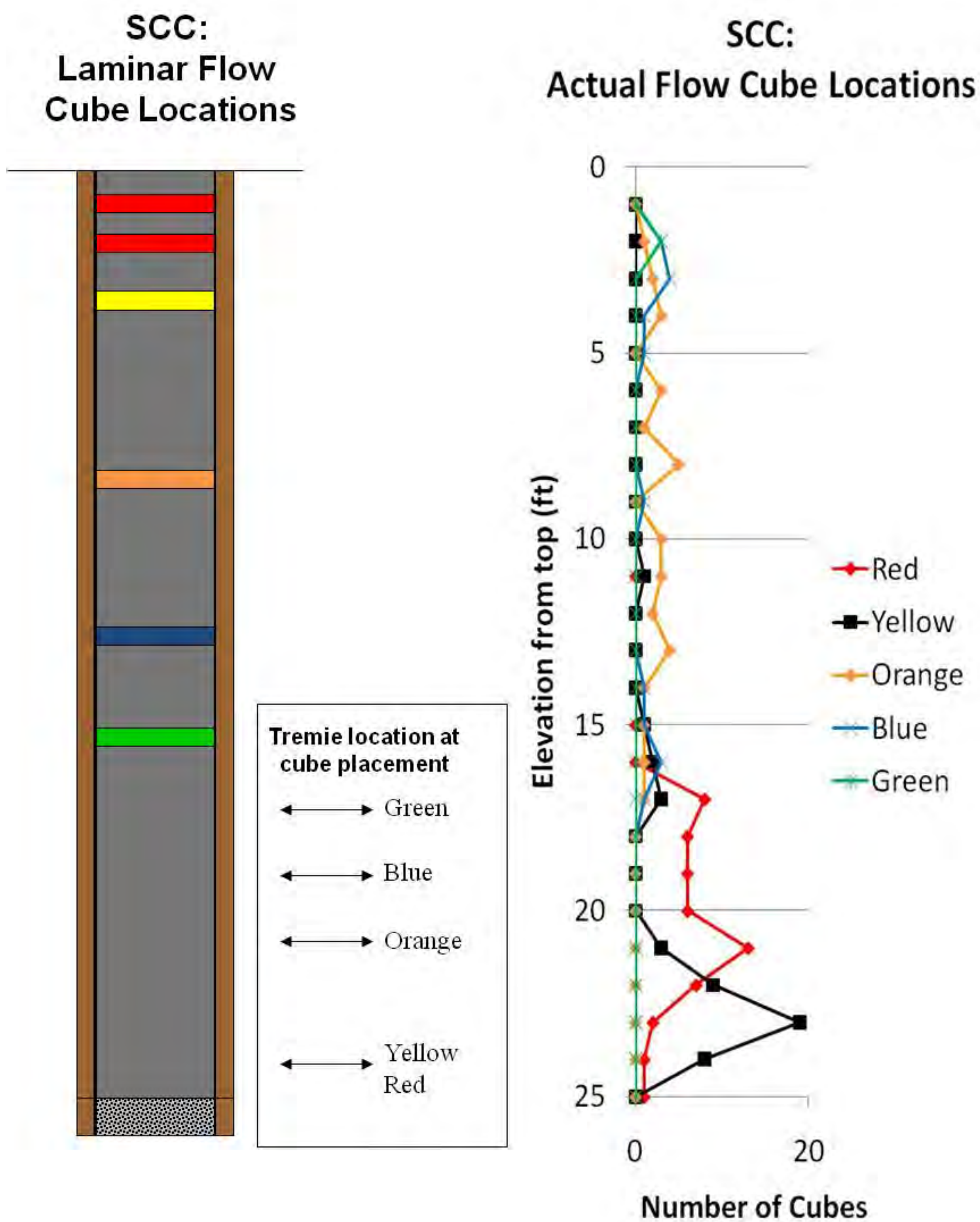



**Figure 3.62:** SCC actual versus laminar cube location

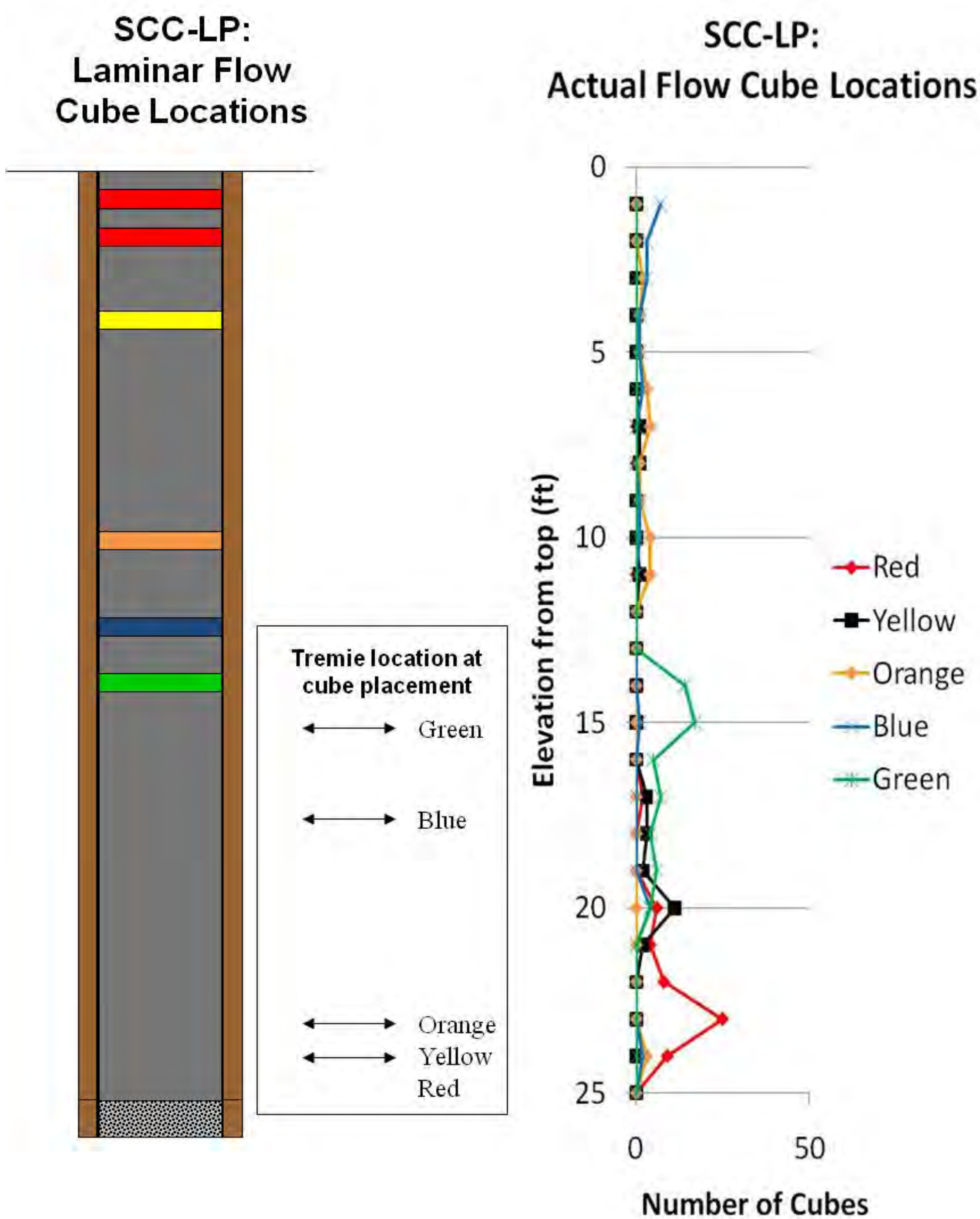



**Figure 3.63:** SCC-LP actual versus laminar cube location

The FHWA drilled shaft manual (O'Neil and Reese 1999) states, "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 flow in a laminar manner. 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. From the figures, it can be concluded that the concrete did not primarily flow in a laminar manner.

Gerwick and Holland (1986) performed tests on concrete tremie flow underwater and determined that the concrete they tested also did not flow in a laminar state, but it rather flowed in either a bulging or layered manner. Bulging flow and layered flow as schematically shown by Gerwick and Holland (1986) is presented in Figure 3.64.

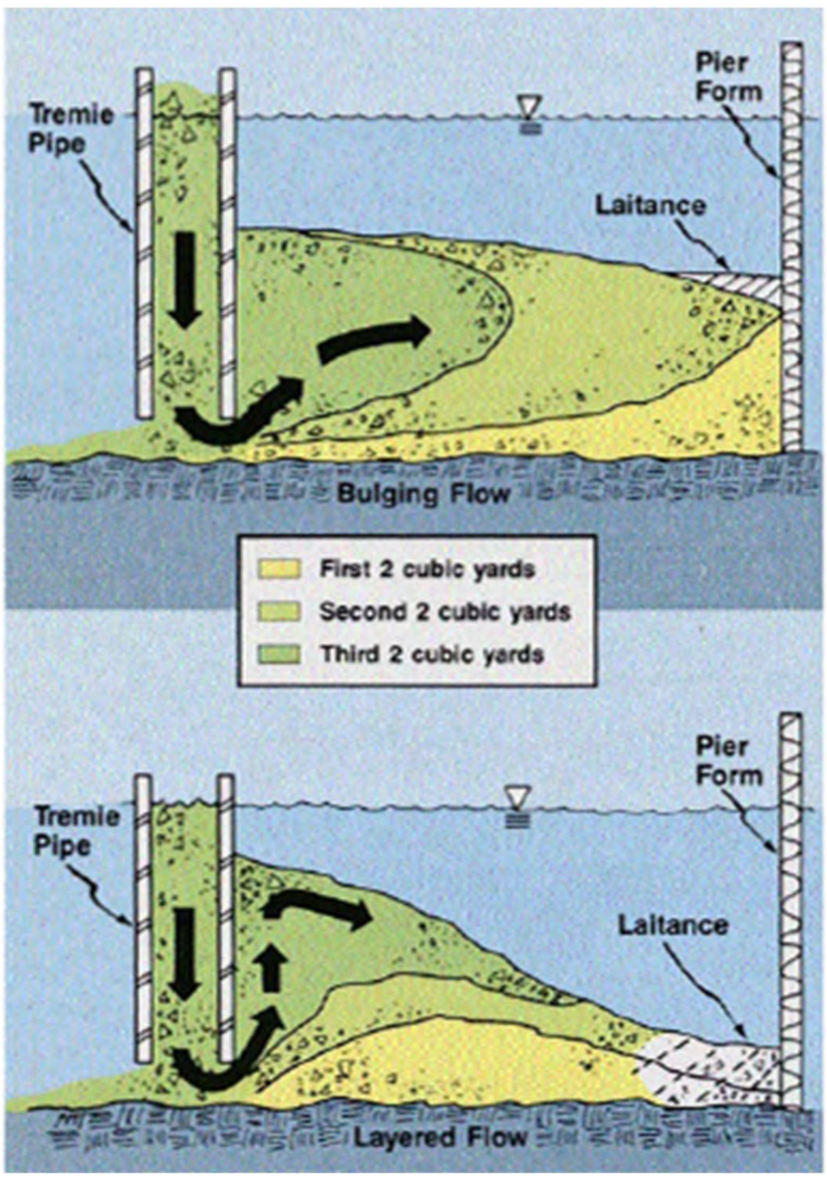



**Figure 3.64:** Bulging flow versus layered flow (Gerwick and Holland 1986)

It was concluded by Gerwick and Holland (1986) that bulging flow was the most desirable to limit the amount of laitance. However, this research was conducted on underwater concrete, where the concrete is allowed to laterally flow and not in a shaft where the concrete is laterally confined.

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 (and last) load of concrete was dyed red. The second and third loads were not dyed. Assuming lamellar flow, it was hypothesized 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 3.65:** 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 budget constraints. 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. Holley et al. (2005) 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"


**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 experimental drilled shafts, and to compare the 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 appear to flow in a layered manner near the outside of the shaft, since the majority of the yellow cubes are 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 seem 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 to 14 ft, 5 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 5 to 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 side of tremie, much like in the South Carolina project. Very few green cubes were observed on the cut of this shaft.

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 displacing 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 very few 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.

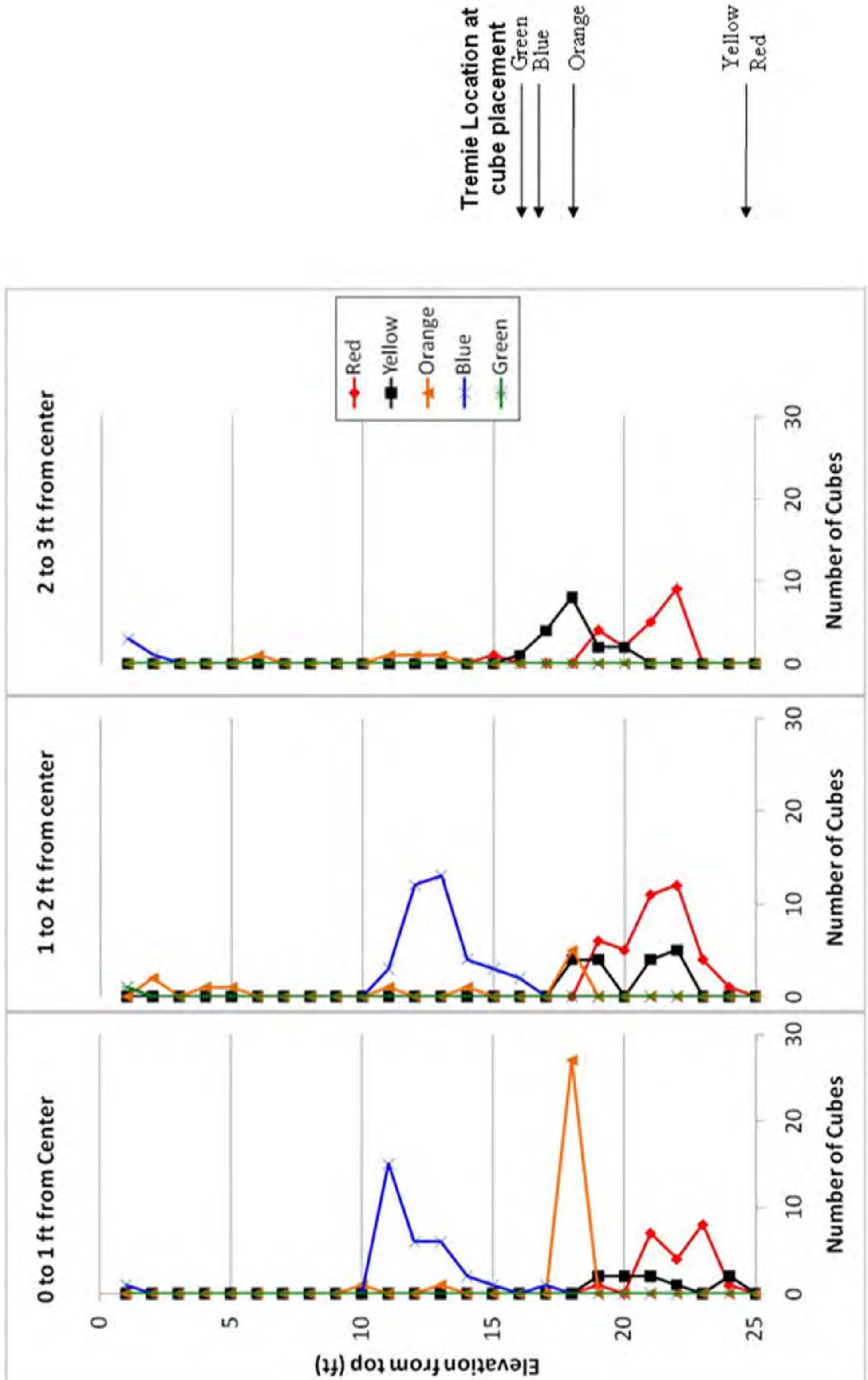



**Figure 3.68:** ODSC shaft cube locations from center

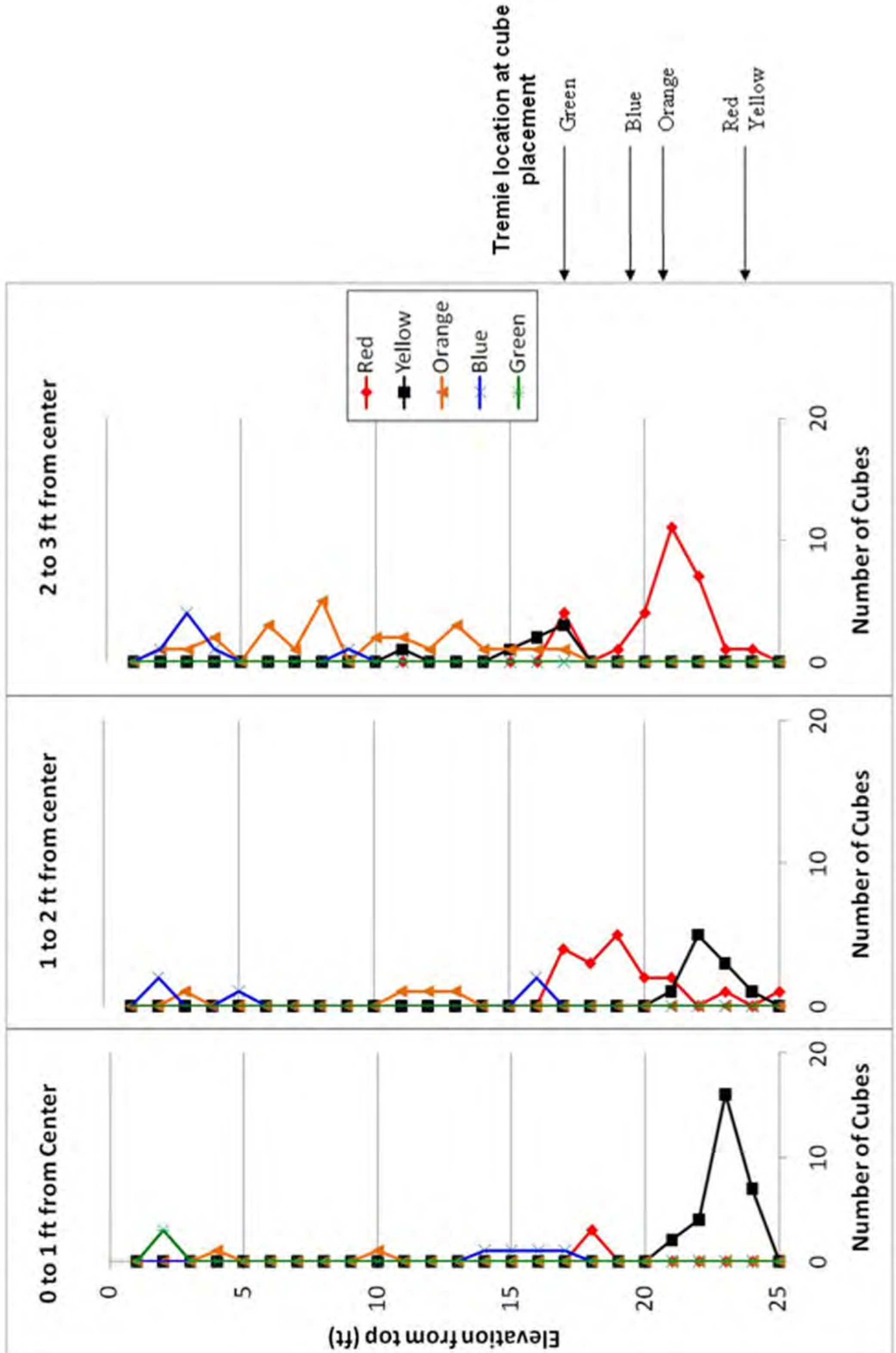



**Figure 3.69:** SCC Shaft cube locations from center

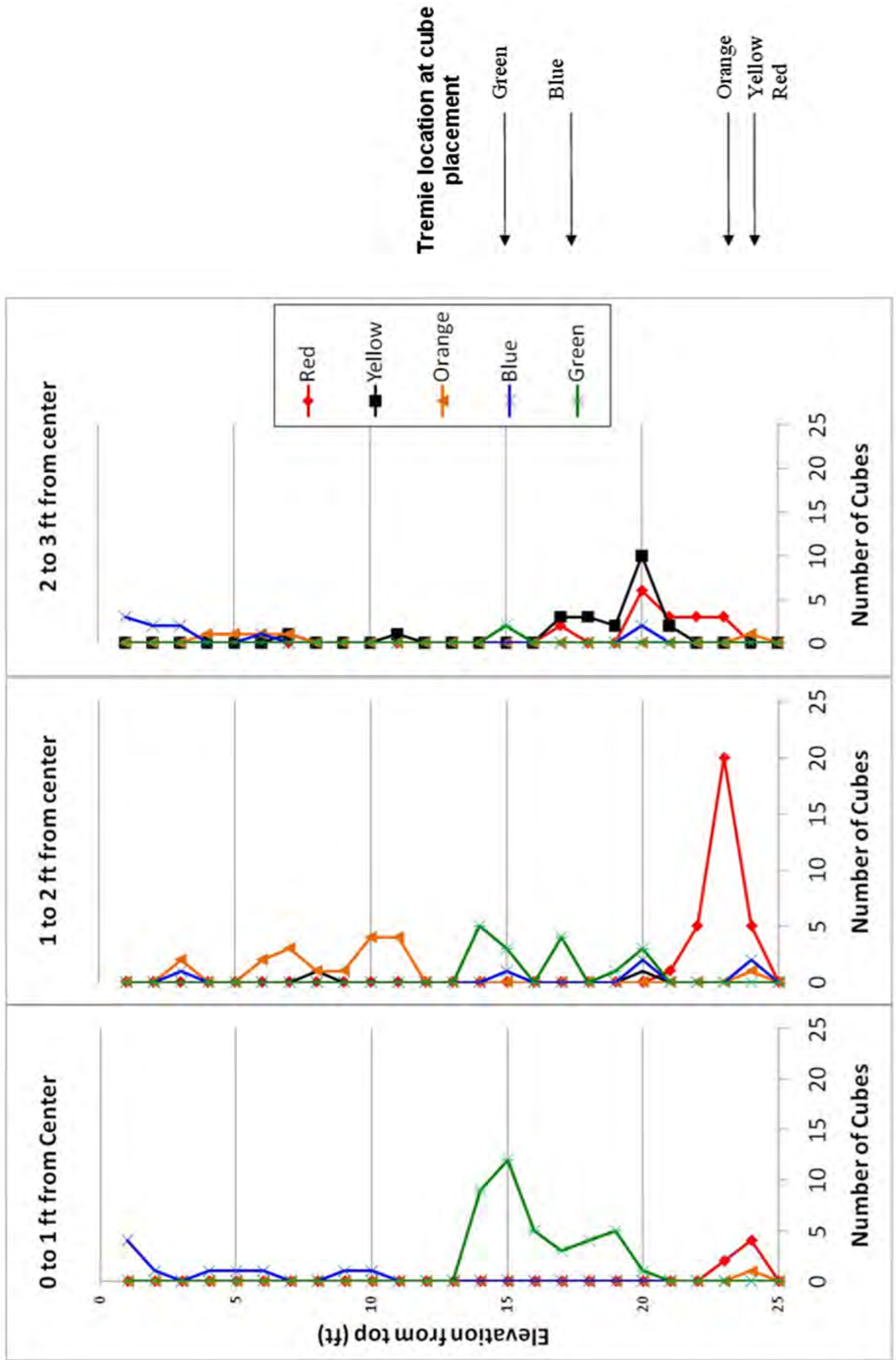



**Figure 3.70:** SCC-LP Shaft cube locations from center

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, first fill the bottom of the shaft 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 side due to the confining pressure. This confining pressure will cause some of the concrete to travel upwards around the tremie side, but also cause the concrete to displace the previously placed concrete upward and then outward near the top concrete surface. 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.

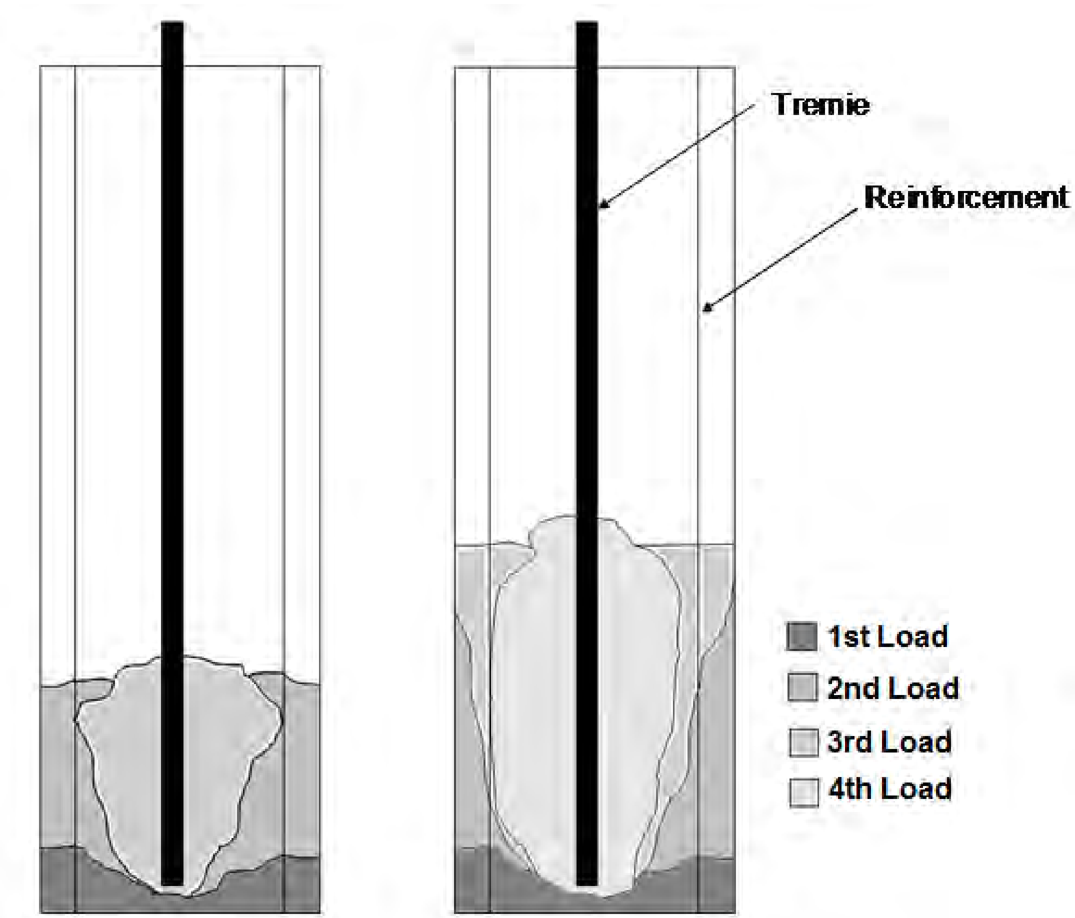



**Figure 3.71:** Hypothetical movement of high-viscosity concrete, such as ODSC

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 side causing more of the previously placed concrete to spread out and rise up the shaft. At the tremie tip elevation, the concrete bulges into the previous concrete; then, when the next concrete exits the tremie, 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.

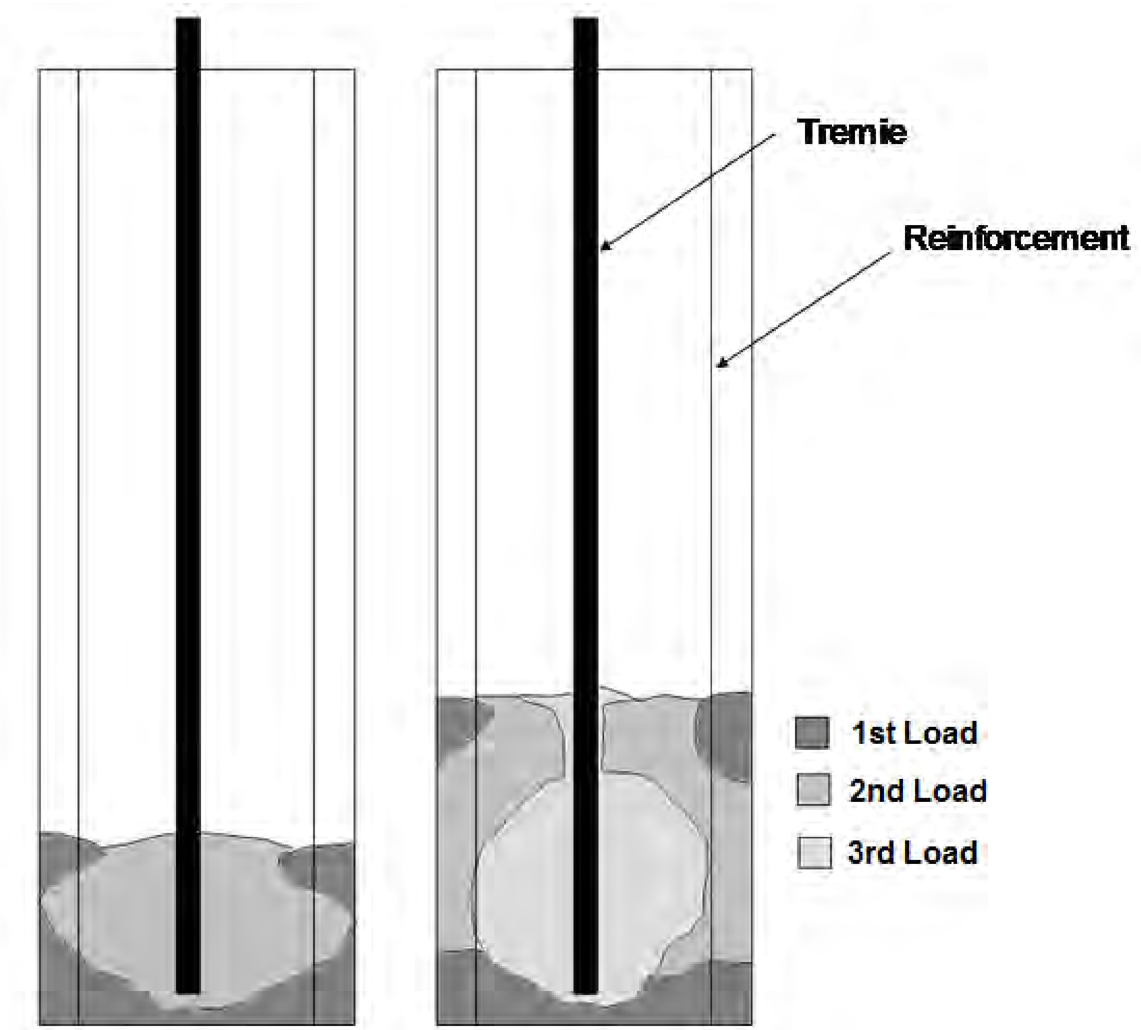



**Figure 3.72:** Hypothetical movement of low-viscosity concrete; such as SCC

#### 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 of 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.

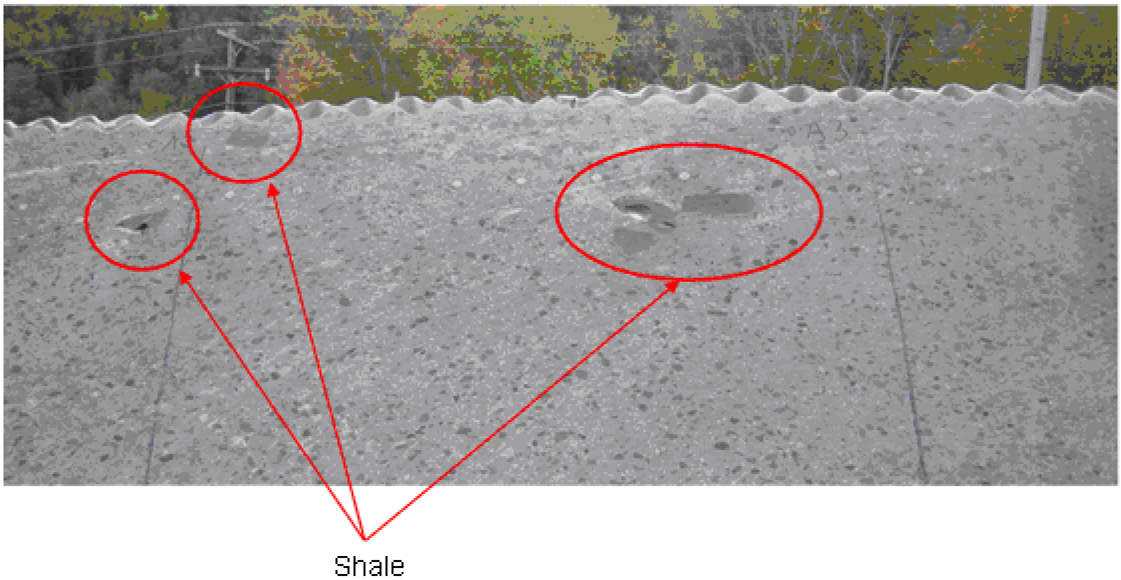



**Figure 3.73:** Shale pieces within the SCC shaft six to nine feet below the top

### 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 cut cross sections. The results from the laboratory tests performed on these cores are presented below.

#### 3.5.6.1 Compressive Strength Modulus of Elasticity of Cores

The cores were acquired at elevations of 7 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 findings 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.

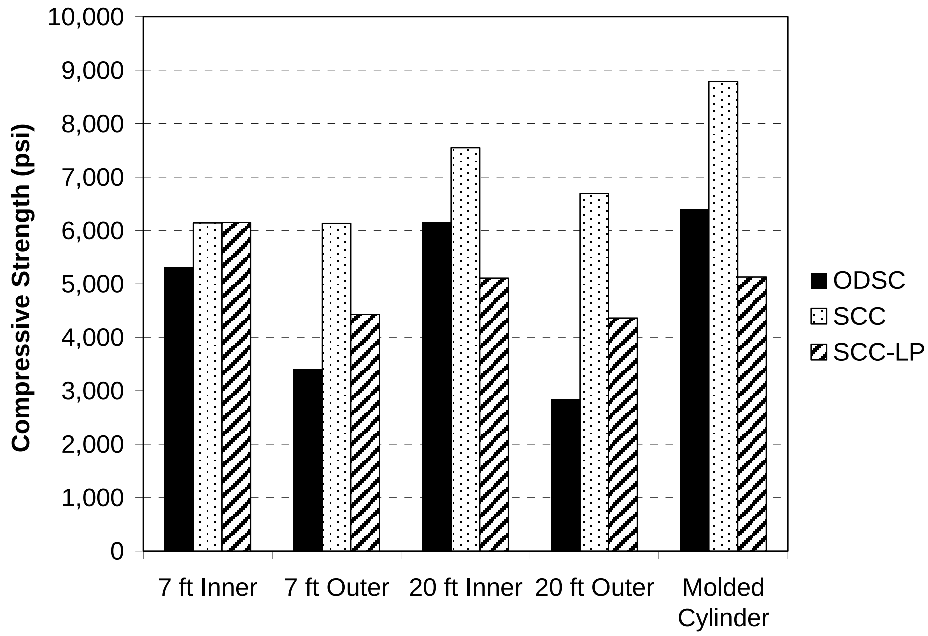



**Figure 3.74:** Compressive strengths of cores versus the molded cylinders

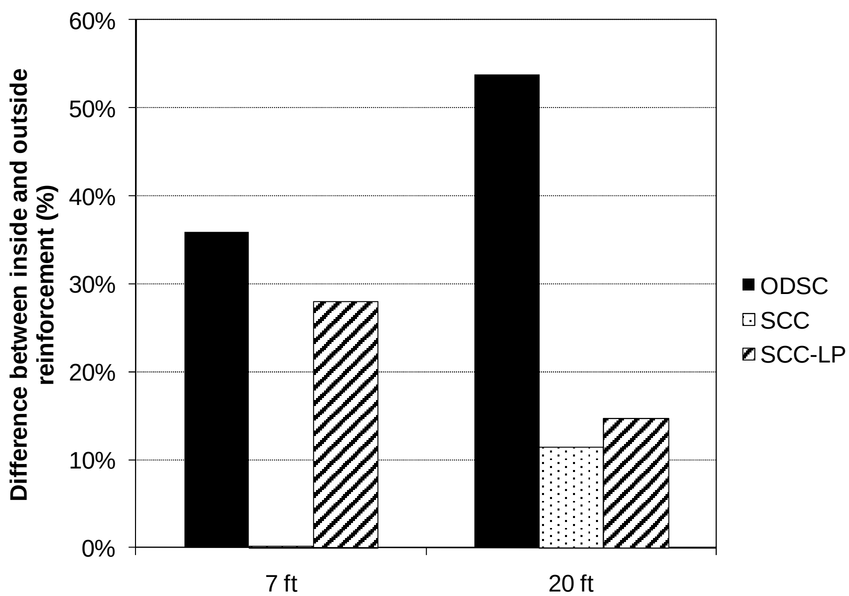



**Figure 3.75:** Difference in concrete compressive strength between the inside and the outside of the reinforcement cage

The SCC-LP cores results are slightly 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. From the SCC-LP test results it was concluded that the reinforcement cage would have a minimal effect on the flow of this concrete. From the J-Ring test results of the SCC mixture it was 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 near similar within and outside of the reinforcement cage. This difference in behavior could be attributed to the additional consolidation energy due to overhead concrete pressures available in the shaft that is not present when performing the J-Ring test.

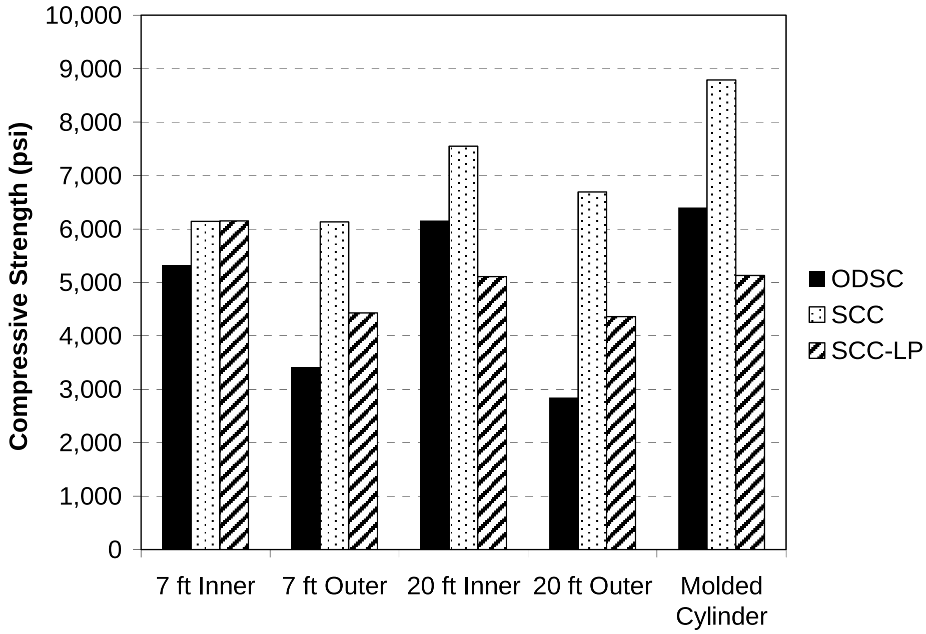



**Figure 3.76:** Modulus of elasticity of the cores compared to the molded cylinder

#### 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 AASHTO T 277 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. The highest chloride ion penetration result was recorded on a sample taken from the cover region of 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.

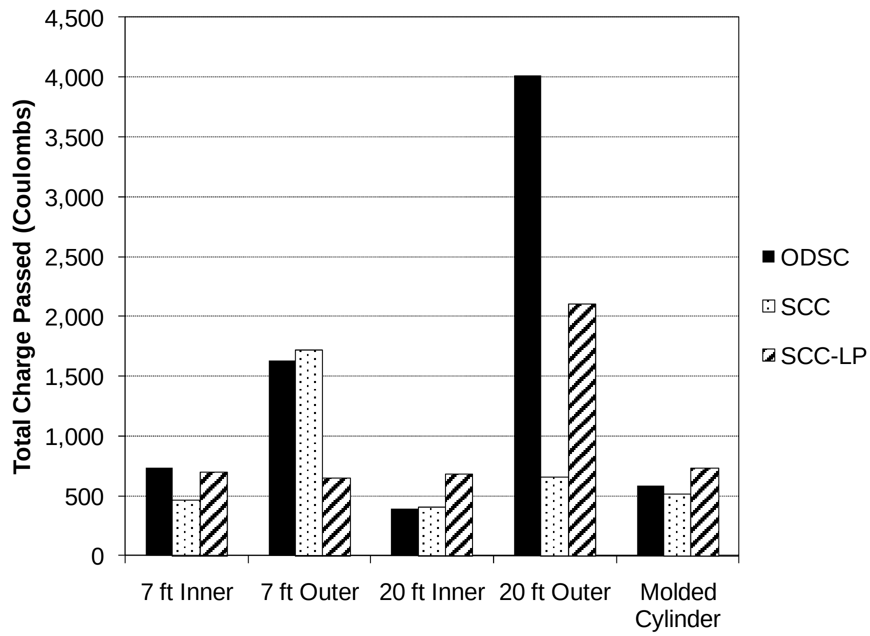



**Figure 3.77:** Chloride ion penetration test results of the cores compared to molded cylinders

**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 shafts by all measures. The cores taken from this shaft had the most consistent compressive strength and modulus of elasticity values throughout all tested 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 segregation and bleed testing had a slump flow value greater than the maximum allowed in the project specification, which could explain some of these test results. This concrete passed the VSI test, and therefore, was allowed to be placed to note any issues 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 in-place properties. Large amounts of sand were built up on the outside of the shaft. Large voids were observed in the bottom corners of the shaft, thus 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. The chloride ion penetration results of the cover region was also very high, indicating high permeability 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. However, this is attributed to the limited limestone powder available and lack of concrete producer experience to produce concrete with this material.

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 report that the higher viscosity concrete, such as ODSC, flow in a layered upwards around the tremie and then outward near the top concrete surface as shown in Figure 3.71. Lower viscous concrete, such as SCC, flows in a bulging manner with less of the concrete flowing directly upwards around the tremie, and instead it bulges laterally when existing the tremie, which displaces the previously placed layers upwards to fill the shaft, as shown in Figure 3.72. More research is required to further evaluate this hypothesis and the actual flow patterns within drilled shafts. This phase of this research project determined that self-consolidating concrete is a viable 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 experimental shafts, ALDOT decided to require SCC for the drilled shafts on all the shafts of Phase II of the B.B. Comer Bridge located on AL-35 near Scottsboro, Alabama.

The purpose of this phase of this project was to evaluate the use of SCC in large-scale production drilled shafts. All ALDOT quality assurance tests results are presented in this chapter. The research team also documented the use of SCC for many of the placements and placement summaries are provided in Appendix D. The research personnel conducted additional tests on some placements to evaluate the following:

- Variability of the concrete (slump flow and VSI) arriving to the jobsite,
- Flowability of the concrete throughout the placement,
- Bleeding of the concrete under pressurized and non-pressurized conditions, and
- Temperature development due to hydration of the concrete.

By evaluating the fresh and hardened concrete properties, the progress of concrete placement in these shafts, and in-place integrity test results, it is believed that a better understanding of using SCC in drilled shaft applications is acquired.

## 4.2 CONSTRUCTION PLAN

### 4.2.1 Contributing Companies

Kirkpatrick Concrete, Russo Corporation, and Scott Bridge Co. made up the team that produced and constructed the drilled shafts for this project. Kirkpatrick Concrete was the concrete contractor and developed the SCC mixture specifically for this project while meeting ALDOT's special provision. Russo Corporation was the drilled shaft contractor and performed the drilling and installation of the drilled shafts. Scott Bridge Co. was the bridge contractor, performing surveying and moving barges for the drilled shaft contractor. ALDOT technicians performed the quality assurance testing. GMS Testing was hired by the drilled shaft contractor to conduct the CSL tests to verify the integrity of the completed shaft. For research purposes, Applied

Foundation Testing (AFT) was hired by Auburn University to perform specialized, non-destructive tests (gamma-gamma tests) on selected drilled shafts.

### 4.2.2 Description of Production Shafts

This phase of the bridge project consisted of three piers installed over water (Piers No. 7, No. 8, and No. 9). Each of these piers consists of five, 8 ft diameter, drilled shafts. A schematic of the drilled shaft locations for each pier is presented in Figure 4.1. Shaft lengths ranged from 21 to 46 ft that were 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). The first shaft (Shaft No. 4 of Pier No. 7) was constructed on 08/28/2009 and the last shaft (Shaft No. 5 of Pier No. 9) was constructed on 12/01/2010.

Each shaft was reinforced with a 7 ft diameter reinforcement 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. The drilled shafts were excavated and installed using methods determined by the drilled shaft contractor.

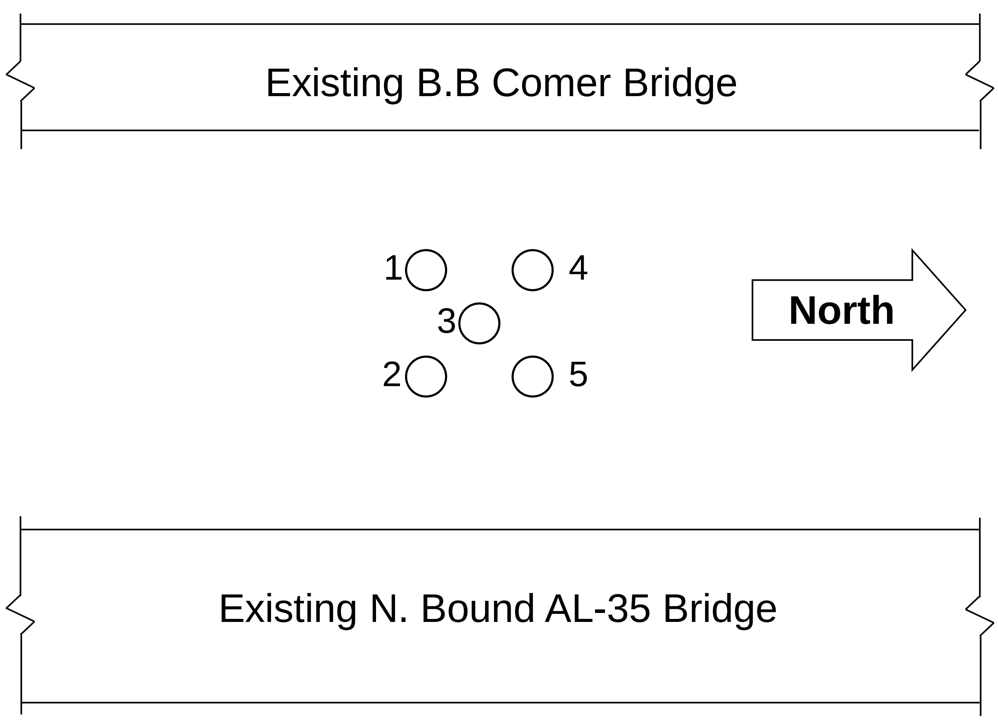



**Figure 4.1:** Location of drilled shafts for each pier (not to scale)

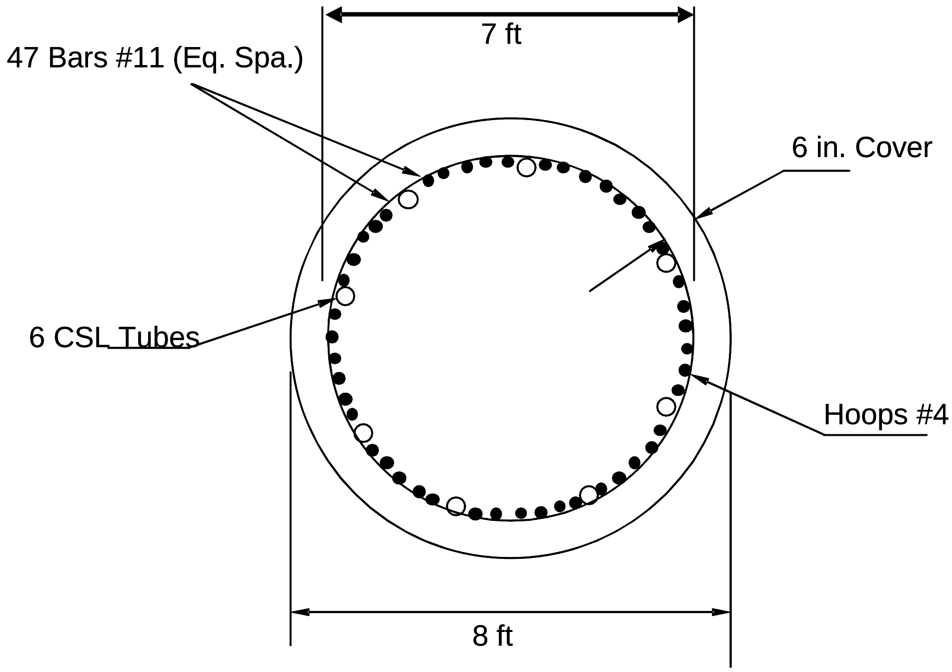



**Figure 4.2:** Reinforcement cage arrangement

**Figure 4.3:** Drilled shaft reinforcement cage

### 4.2.3 Testing Fresh Concrete Properties

To ensure concrete with the desired properties is placed in 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 concrete for selected drilled shafts:

- Slump flow and VSI on every truck during shaft construction,
- Measurement of concrete elevations inside and outside the rebar cage,
- 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, and
- Gamma-gamma integrity testing.

## 4.3 ALDOT FRESH CONCRETE TESTING

### 4.3.1 SCC Fresh Concrete Test Training

Since the ALDOT technicians on this project never had experience with SCC, two training days were set up to train them how to perform the slump flow (ASTM C 1611) and VSI (ASTM C 1611 Appendix) tests. In addition to these training days, a VSI test manual was created to help explain the VSI test; this manual is presented in Appendix C. This manual was laminated and handed to all that attended the training event. The training days were conducted at the Kirkpatrick batch plant, near the project site. 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 make 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 jobsite trailer with temperature-regulated curing tanks.

## 4.4 ADDITIONAL FRESH CONCRETE TESTS PERFORMED FOR RESEARCH PURPOSES

### 4.4.1 Batch-to-Batch Variability of Slump Flow and VSI Test Results

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 as per the project specification, ALDOT only tested one sample for every 50 $yd^3$ of concrete delivered to the site. Each sample was taken after the ready-mixed concrete truck had dispensed approximately half of its load into a 3 $yd^3$ bucket. The samples were acquired directly from the concrete chute into a 5-gallon 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 ready-mixed concrete trucks discharged their load. A picture of this testing area is shown in Figure 4.5.

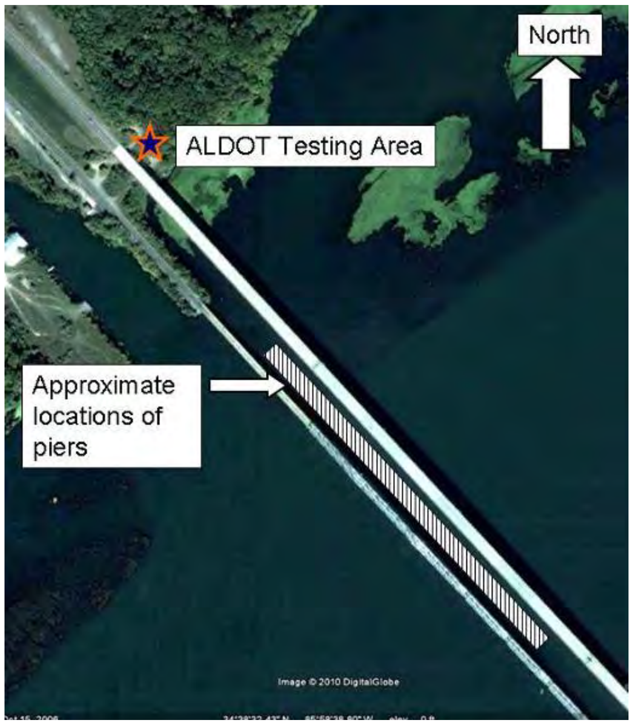



**Figure 4.4:** Location of ALDOT testing area in relation to pier locations (Adapted from Google Earth 2010)

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

### 4.4.2 Concrete Elevation Inside and Outside the Rebar Cage

A weighted measuring tape was lowered through the water to the concrete surface on the outside of the reinforcement cage. In addition, the concrete depth near the tremie was also measured by a representative from the drilled shaft contractor after every other bucket was placed into the shaft. The difference between the concrete depth near the tremie and the outside of the reinforcement cage provides a measure of the concrete's ability to flow through the reinforcement cage. These data were only conducted for a few placements during this project.

### 4.4.3 Bleed Test

To assess the concrete's bleeding, a bleed test was performed on concrete specimens collected during the placement of selected shafts. This test was conducted in accordance with ASTM C 232 (2004). The samples were acquired from the ready-mixed concrete 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, known as a 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 tap the outside of the bucket 20 times (five times for each direction, North, South, East, and West). Then, a plastic lid was placed 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 2 min. 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 covered 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 over a 90 min. period, 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, the chamber pressures were changed. These pressures were decreased for this new series of tests. The new applied pressure profile consisted of gradually increasing to 60 psi over the first 10 min., maintaining 60 psi for 50 minutes, gradually increasing to 90 psi between 50 and 60 minutes, and then maintaining 90 psi for the remainder of the test. These pressures were selected to be representative of concrete placed underwater in a 150 ft long drilled shaft.

After starting the project, the testing apparatus was once more modified once 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 water pressure in the shaft as the concrete leaves the bottom of the tremie. The apparatus was set up in the same manner as previously described, 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 (included in Appendix B) required that CSL tests be performed on each drilled shaft. For research purposes, some of the shafts had temperature probes installed and gamma-gamma tests were only performed to assess the concrete quality.

### 4.5.1 Installation of Temperature Sensors

To assess the in-place impacts of heat of hydration of the cementitious materials contained in these large-diameter shafts, temperature sensors were installed in selected drilled shafts. Three sensors were attached at the following three 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 on the reinforcement cage,
- One sensor near the center of the cage on an added reinforcement bar, and
- One sensor approximately 3 in. from the edge of the shaft in the cover region.

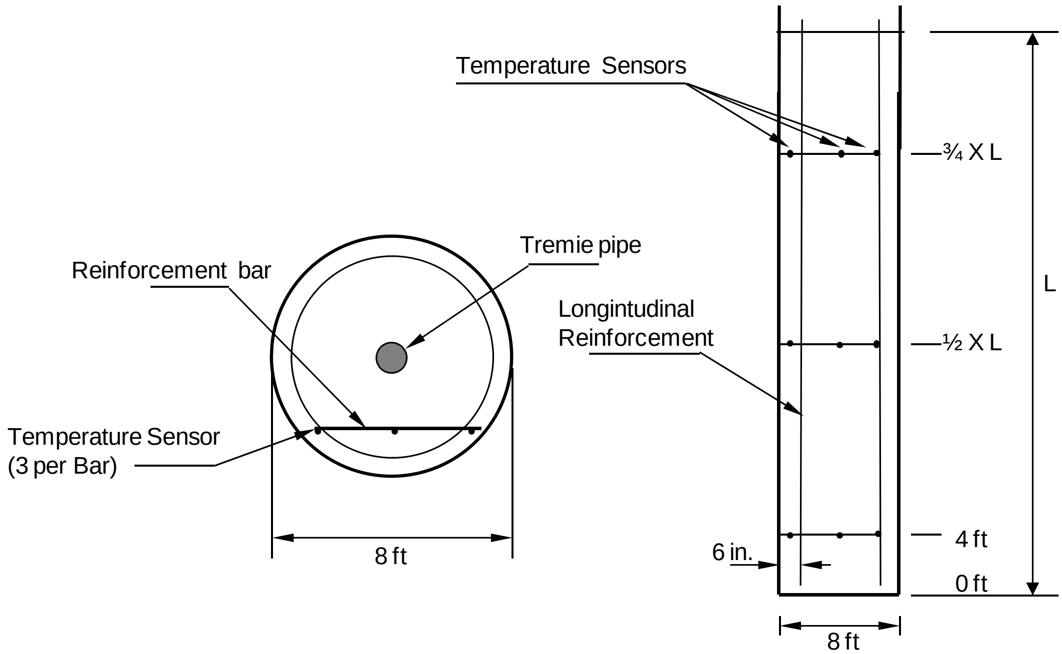
To install the probes located near the center of the cage and 3 in. from the edge, it was necessary to install additional reinforcement bars at three different cage elevations. A schematic of the temperature probe locations is presented in Figure 4.8.

### 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 hours and no later than 20 days after concrete placement. The results from these CSL tests were thus covered as part of the construction project and they were used by ALDOT for shaft quality acceptance. These CSL tests were performed by GMS Testing, Inc., Muscle Shoals, Alabama.

Gamma-gamma tests were performed by COLOG, a division of Layne Christensen Company, Lakewood, Colorado. Gamma-gamma tests were only performed on select shafts, and these tests were not used for shaft acceptance. The shafts tested with gamma-gamma tests were:

- Pier 8: Shafts 1 and 2
- Pier 9: Shafts 2, 3, and 4




**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. The concrete producer selected the following raw materials for this project:

- Portland cement: Type I/II manufactured by National Cement Co. in Ragland, Alabama.
- Class F fly ash: Supplied by SEFA, Inc. from Cumberland, Tennessee.
- Coarse aggregate: A dolomitic limestone with a No. 78 gradation was obtained from Vulcan Materials Co. in Scottsboro, Alabama.
- Fine aggregate: A natural sand was supplied by Madison Materials from Summit, Alabama.
- Hydration stabilizing admixture: DELVO® STABILIZER supplied by BASF Construction Chemicals. This admixture retards setting time by controlling the hydration of the cementitious materials.
- High-range water reducing (HRWR) admixture: Glenium® 7000 supplied by BASF Construction Chemicals.
- Viscosity modifier admixture: RHEOMAC® VMA 362 supplied by BASF Construction Chemicals.
- Air-entraining admixture: MB-AETM 90 supplied by BASF Construction Chemicals.

**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 |

## 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 ready-mixed concrete truck route 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 producer. 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 the sensor readings were verified, an empty ready-mixed concrete truck was positioned under the batch plant and filled with the aggregates, cement, fly ash, and water. The load was then mixed in the truck, at a high rate, for 60 revolutions. After initial mixing was completed, chemical admixtures were dispensed into 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. With this approach, each truck could fill two 3 yd$^3$ concrete placement 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 many of the loads.

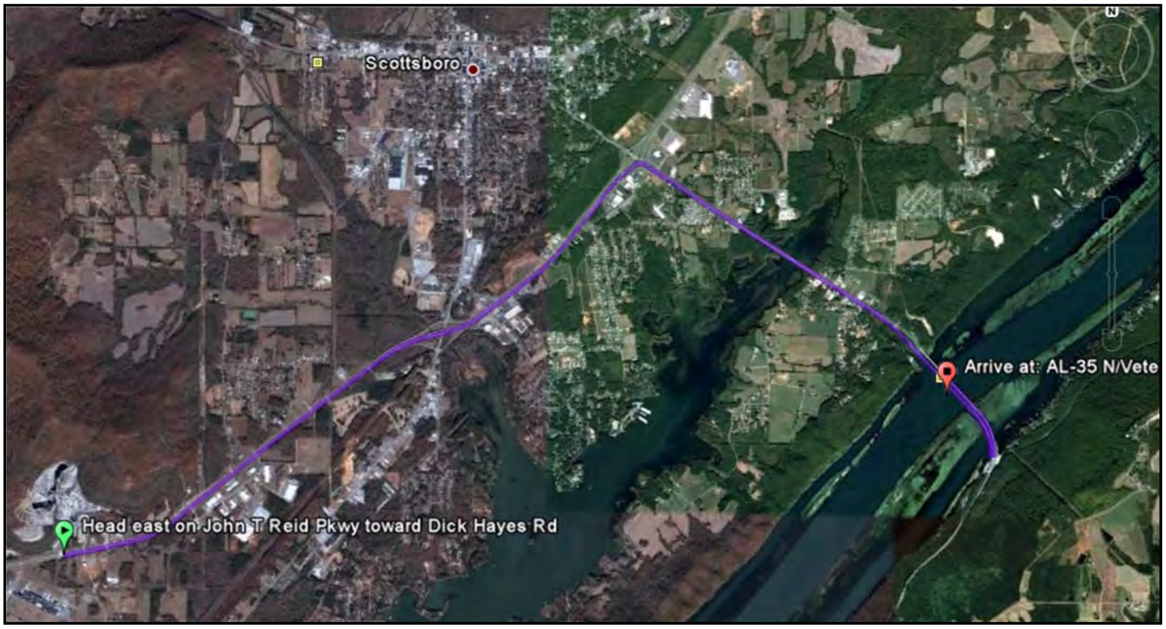



**Figure 4.9:** Route from batch plan to project location (adapted from Google Earth 2010)

### 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 close to 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 quality control testing was conducted.

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). A schematic of the importance of the concrete curve relative to what may occur in the shaft is presented in Figure 4.10.

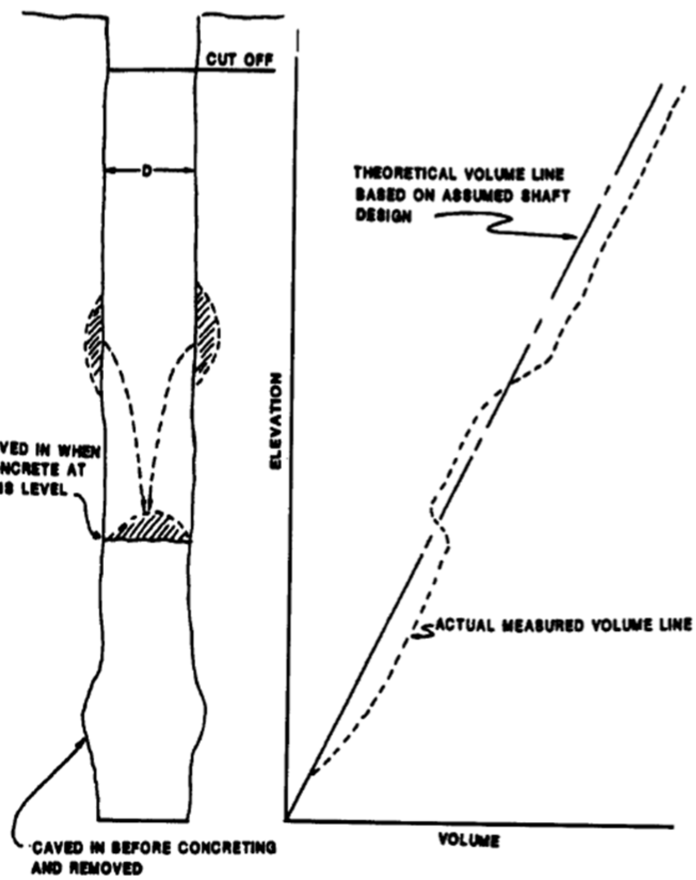



**Figure 4.10:** Comparison of actual amount of concrete to theoretical amount of concrete (ADSC/DFI 1989)

## 4.8 PRODUCTION PROCESS

The drilled shafts were constructed under water, 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 (23 to 31 ft long) that did not have full-length casing, in order to increase the skin friction of the shaft.

Before the concrete was batched, the drilled hole was prepared. To prepare the hole, the drilled shaft contractor generally used the following process for this project:

- After the driller reached the required depth, the debris was cleared from the bottom of the shaft,
- Above the concrete level, the assembled reinforcement cage was spliced to a less congested cage to provide continued support for the CSL tubes to keep them straight for their entire length, as shown in Figure 4.11,
- 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 keep the cage in the center of the hole during the concrete placement,
- A 10 in. diameter 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,
- The crane then lifted the hopper slightly and steel bars were slid under the hopper to make sure the hopper was secure and the tremie was just slightly above the bottom of the shaft, and
- Just before discharging concrete into the hopper, a wet foam plug, known as a pig (shown in Figure 4.13), was pushed into the top of the tremie.

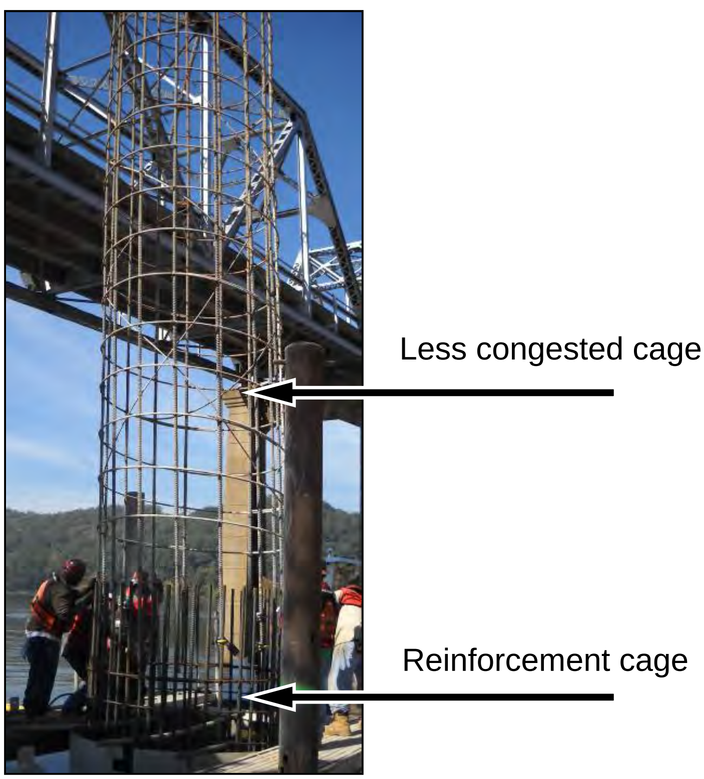



**Figure 4.11:** Less congested cage spliced to the reinforcement cage

**Figure 4.12:** Tremie and hopper being lowered into the shaft

**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-mixed trucks to discharge concrete into the 3 yd$^3$ concrete placement buckets. Once prepared, the batch plant was contacted to send the concrete trucks to the jobsite.

Once the concrete was tested and deemed suitable at the batch plant, the ready-mixed 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 concreting 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 barges. Each crane would take turns moving an empty bucket from the barge, to the bridge for filling, 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.

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).

**Figure 4.15:** Three cubic yard buckets being discharged into the tremie hopper of Shaft No. 2 of Pier No. 7

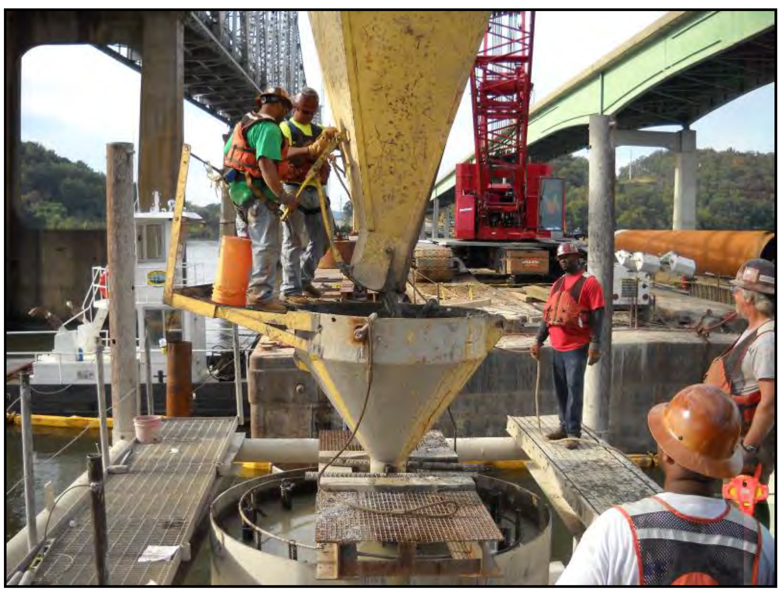


**Figure 4.16:** Three cubic yard bucket being discharged into the tremie hopper of shaft no. 1 of pier no. 7

### 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. Note that this placement method was only used for this shaft. The pump truck was positioned on the left-hand lane, on the existing Northbound bridge, on U.S. Hwy 35. Concrete placement of this shaft occur with difficulty. The following problems occurred:

- the tremie pipe clogged multiple times,
- the pump lines on the barge became disconnected multiple times, due to pressure in the line, and
- numerous cement balls were visible in the concrete delivered to site.

Examples of the cement balls that were found on the pump truck grate are shown in Figure 4.17. A picture of the pump line configuration is shown in Figure 4.18**Error! Reference source not found.**. A more detailed account of the concrete placement for this shaft is given in the daily construction notes included 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, four micropiles were installed to provide additional support to this shaft.

The cause of all the problems that occurred on this date are unknown. The experimental shafts described in the Chapter 3 were installed using a pump truck without the problems described on this shaft. However, since the construction of the experimental shafts, the concrete contractor has changed the SCC proportions from those used for the experimental shafts. Also, a different drilled shaft crew was used to place the concrete for the production shafts. Additionally, for the experimental shafts, the pump truck 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. The problems experienced on this shaft shows that a test shaft should be constructed for all projects to test the concrete 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 was 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 $yd^3$ was 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 easily seen discharging from a gasket on the tremie pipe, as shown in Figure 4.19.

With the location of the leak isolated, 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 observed. Any subsequent placement of the shaft using the fixed tremie pipe did not have any delays or problems.

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

### 4.8.3 Discussion of Concrete Placement

Other than the problems discussed in Sections 4.8.1 and 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.
- Although the tremie did not need to be moved during placement of a shaft, no problems were experienced when extracting the tremie from the shaft.
- 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 minor cement balls were still visible within some batch trucks, but did not cause any problems.
- As it typical practice, each shaft was over-filled a few feet with concrete due to the possible presence of laitance.

## 4.9 Fresh Concrete Test Results and Discussion

The results of the tests conducted by the Auburn University research staff for this project are presented and discussed in this section.

### 4.9.1 Slump Flow and VSI

#### 4.9.1.1 Shaft No. 4 of Pier No. 8

To test the variability of the concrete delivered to the jobsite, slump flow and VSI tests were performed on each truckload delivered to site. The results of these tests, taken during the installation of Shaft No. 4 of Pier No. 8, are presented in Figure 4.20. The summary 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 and in both instances, the concrete in both these trucks met all project specification requirements. However, four of the trucks only tested by Auburn University staff had slump flow values below the project specification of 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.

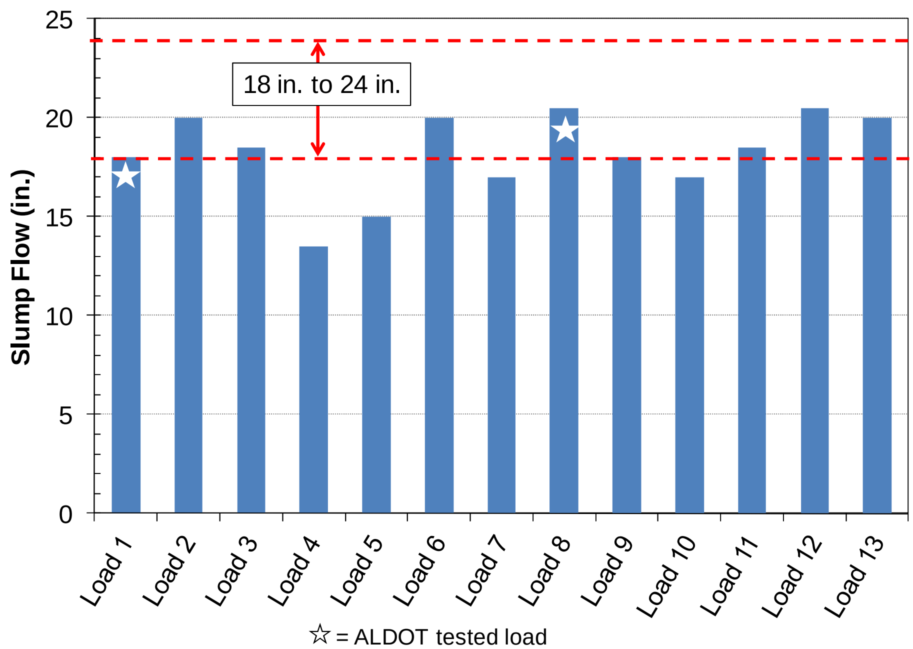



**Figure 4.20:** Slump flow results from Shaft No. 4 of Pier No. 8

**Table 4.2:** Slump flow statistics for 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 |

#### 4.9.1.2 Shaft No. 5 of Pier No. 8

The flump slow results of all the tests of Shaft No. 5 of Pier No. 8, are presented in Figure 4.21. The summary statistics calculated from these data are presented in Table 4.3**Error! Reference source not found.**. As with the previous shaft, only two trucks were sampled by ALDOT to test the fresh concrete properties. The concrete in both the trucks tested by ALDOT met all project specification requirements. However, as shown in Figure 4.21 six of the trucks tested by Auburn University staff had slump flow values below the project specification. The first truck tested and passed by ALDOT, was retested on the bridge and it had a slump flow of 17.5 in., which was slightly below the minimum of 18 in. The concrete was very stable, and all the loads tested on this date had a VSI value of 0.

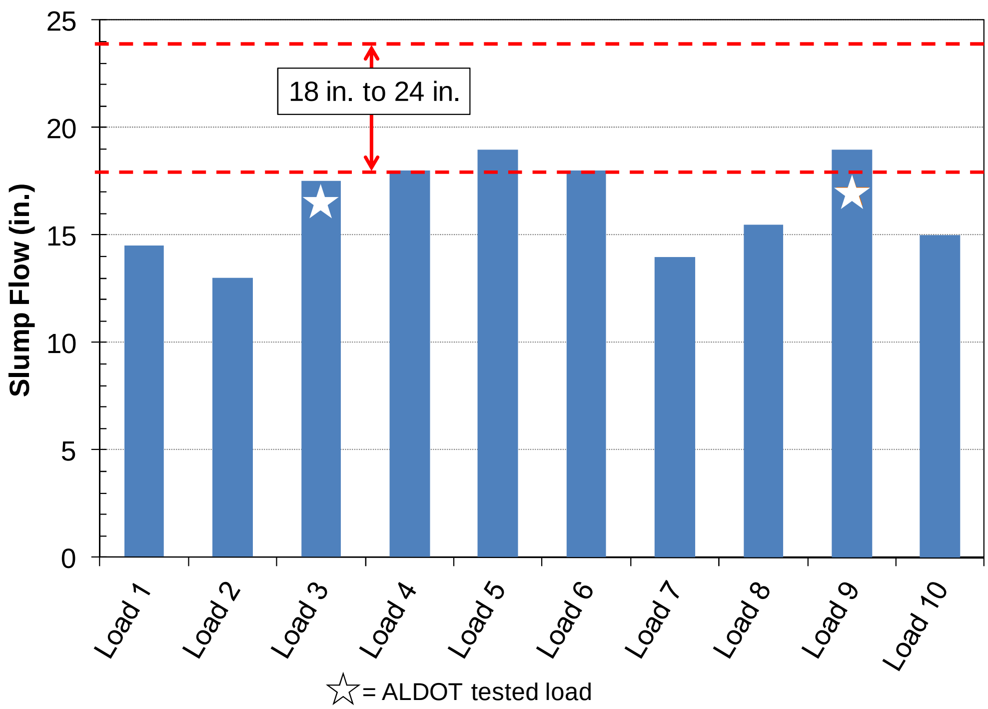



**Figure 4.21:** Slump flow results from Shaft No. 5 of Pier No. 8

**Table 4.3:** Slump flow statistics for 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 |

#### 4.9.1.3 Shaft No. 3 of Pier No. 8

The slump flow tests results for Shaft No. 3 of Pier No. 8, are presented in Figure 4.22. The summary statistics calculated from these data are presented in Table 4.4. Load 5 was tested by ALDOT and was rejected because its slump flow exceeded 24 in. This truck was sent back to the batch plant and this concrete was not placed in the shaft.

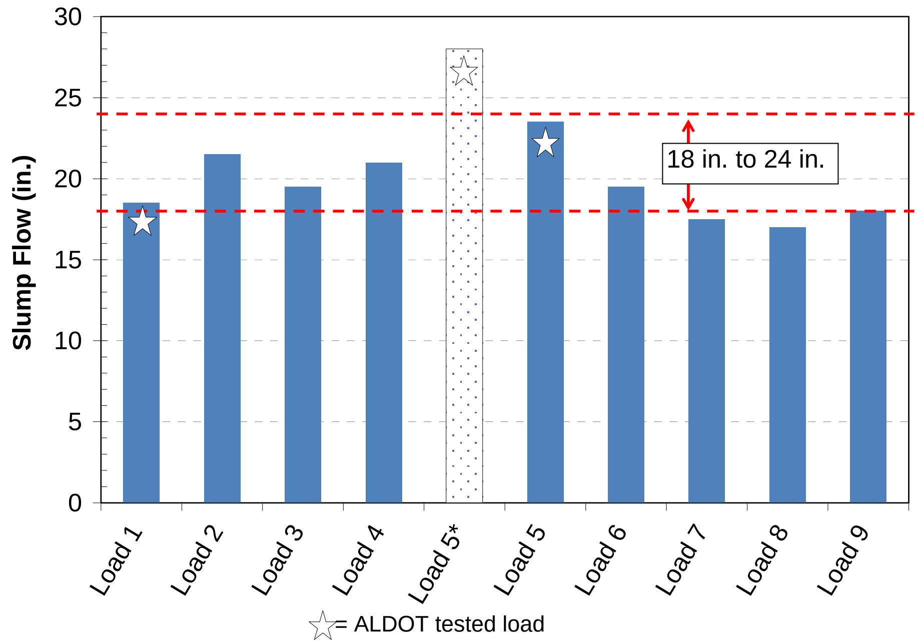



**Figure 4.22:** Slump flow results from Shaft No. 3 of Pier No. 8

**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 |

#### 4.9.1.4 Discussion of Concrete Slump Flow Results

It is typical practice for SCC specifications, 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-mixed 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. range specified. It is recommended that producers of ready-mixed 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 average slump flow for each day tended to be at, or below the lower limit of 18 in. used in 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. The use of a slump flow that is less than the lower limit specified in the project is not recommended, because concrete with a conventional slump between 6 and 9 in. may not be able to consolidate in the cover region as demonstrated in Chapter 3.

### 4.9.2 Concrete Elevation Inside and Outside the Rebar Cage

These measurements were collected for two shafts constructed during this project. The measurements acquired from Shaft No. 1 of 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 measurements varied between 3.5 and 5 in. during the concrete placement. On the contrary, the concrete in Shaft No. 3 of Pier 8, did seem to be obstructed to some extent by the reinforcement cage, with a maximum elevation difference of 22 in. Note that 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. It is also unknown how this elevation difference compares to what would have been obtained should conventional-slump concrete have been used on this project. It is worth noting here that the CSL test results of Shaft No. 3 of Pier 8 provided no indication of any significantly anomaly or inclusion.

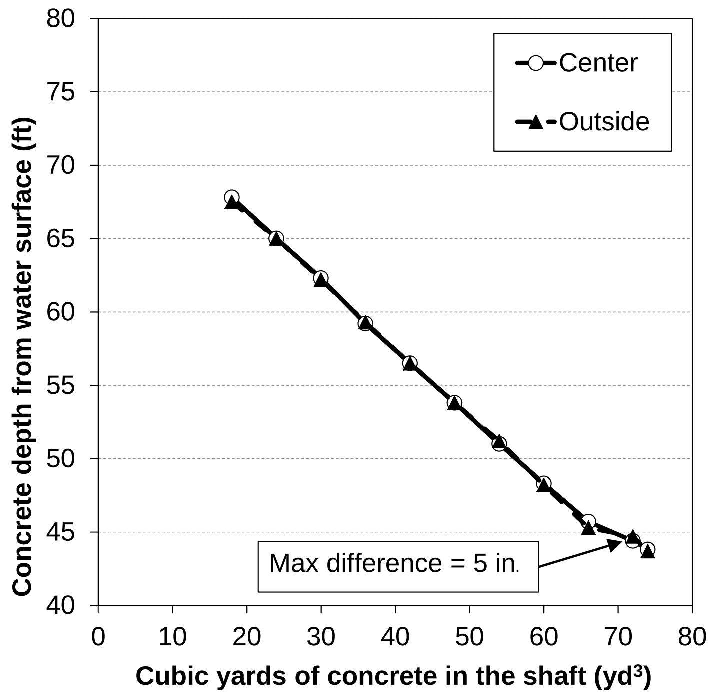



**Figure 4.23:** Concrete depth outside the reinforcement cage compared to the concrete depth taken near the center of the shaft from Shaft No. 1 of Pier No. 7

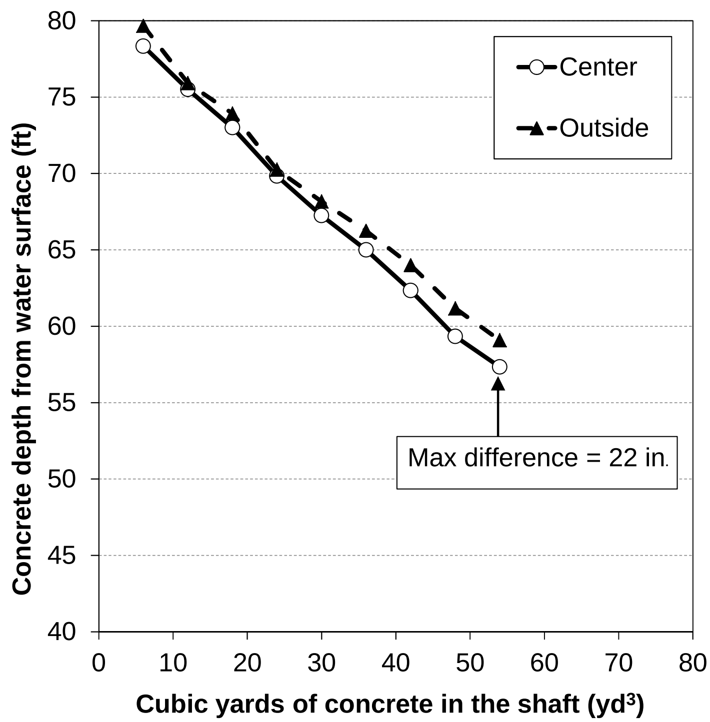



**Figure 4.24:** Depth of concrete outside the reinforcement cage compared to the depth of concrete taken near the center of the shaft from Shaft No. 3 of Pier No. 8

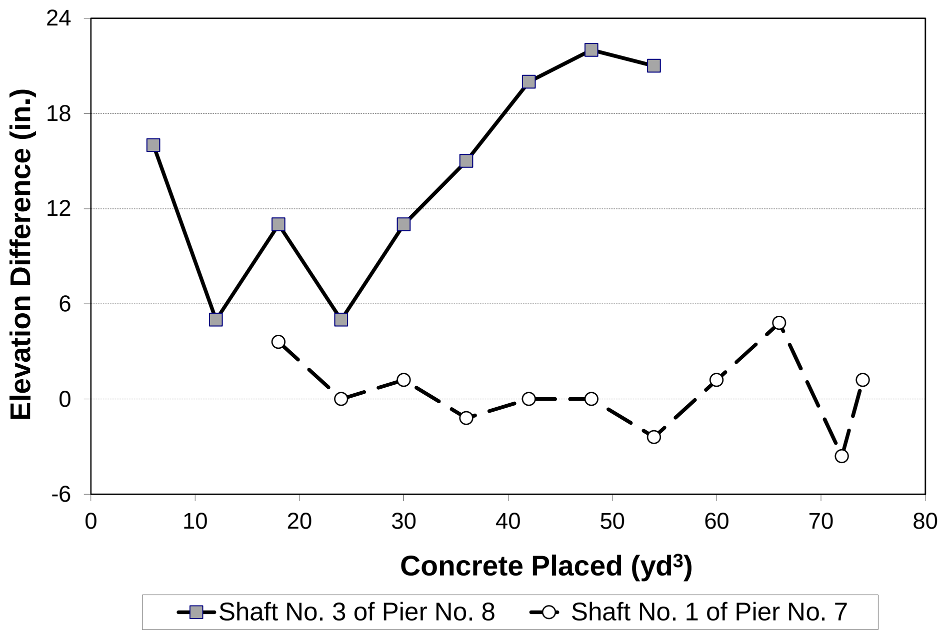



**Figure 4.25:** Elevation measurements compared

### 4.9.3 Conventional Bleed Test and Pressurized Bleed Test

Bleed tests were conducted on concrete sampled during the construction of Shaft No. 3, 4, and 5 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 in each case the SCC had 0.0 percent bleeding.

**Table 4.5:** Results from conventional and pressurized bleed tests

| Pier No. | Shaft No. | Conventional Bleed Result (%) | Pressurized Bleed Result (%) |
|---|---|---|---|
| 8 | 3 | 0.0 | 26.4 |
| 8 | 4 | 0.0 | Not Available |
| 8 | 5 | 0.0 | 31.7 |

The pressurized bleed test was performed on concrete sampled during the construction of Shaft No. 3 and 5 of Pier No. 8. The results from this test are presented in Figures 4.26 and 4.27, respectively. The pressurized bleed test 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. The pressurized bleed test conducted on the concrete from Shaft No. 5 did not include any backpressure.

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, only two tests were performed with this new configuration. Therefore, additional research is recommended to further develop and evaluate this bleed test apparatus specifically for drilled shaft applications, under controlled-laboratory conditions, with various concrete mixtures.

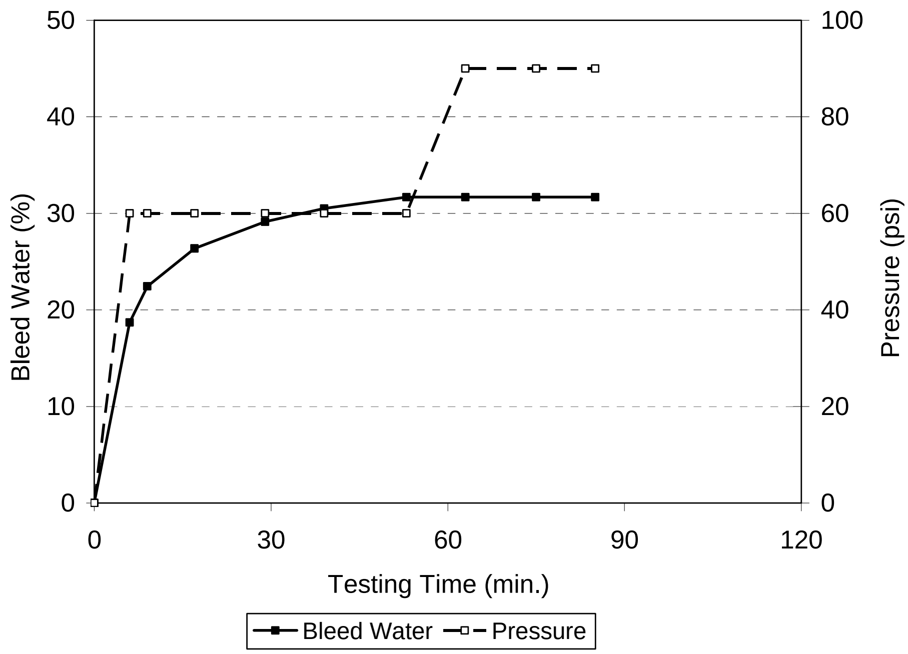



**Figure 4.26:** Results from the pressurized bleed from Shaft No. 5 of Pier No. 8

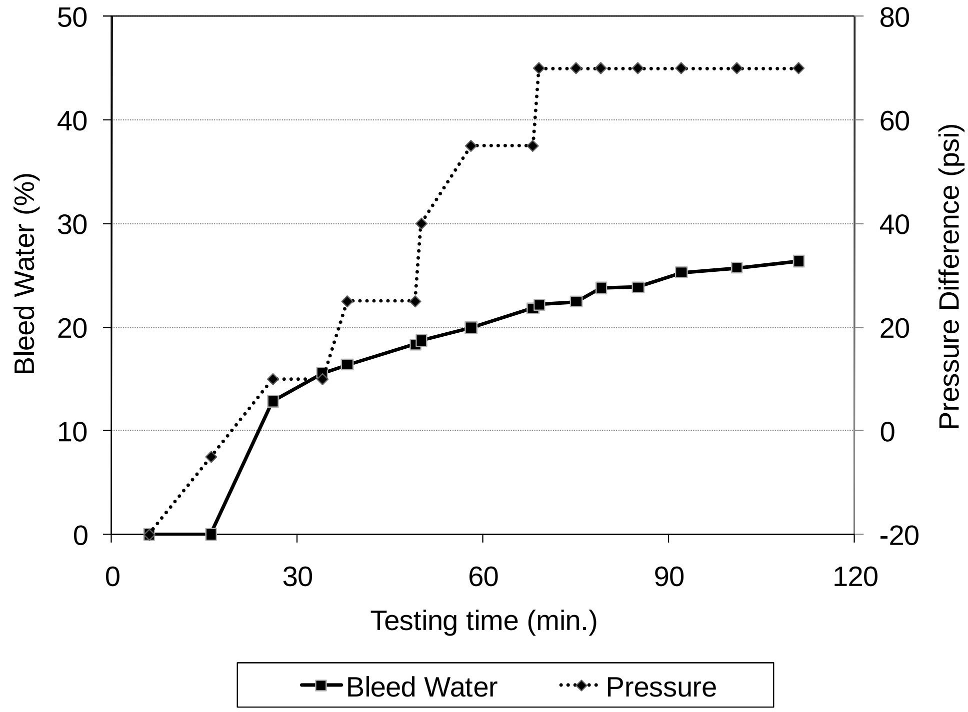



**Figure 4.27:** Results from the pressurized bleed test with backpressure from Shaft No. 3 of Pier No. 8

### 4.9.4 In-Place Drilled Shaft Temperatures

High temperatures due to 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).

Concrete temperatures were measured in Shaft No. 1 and 3 of Pier No. 8, and Shaft No. 1 and 5 of Pier No. 9. Key concrete temperatures recorded from the temperature probes installed in all the instrumented shafts are summarized in Table 4.6. Sample temperature histories for placement under warm and cooler placement conditions are shown in Figures 4.28 and 4.29. Note that the peak concrete temperature and the difference between the center and the edge are both significantly impacted by the concrete placement temperature.

**Table 4.6:** Results from conventional and pressurized bleed tests

| Pier No. | Shaft No. | Concrete Placement Temperature (°F) | Maximum Concrete Temperature (°F) | Maximum Concrete Temp. Difference (°F) |
|---|---|---|---|---|
| 8 | 1 | 91 | 166 | 48 |
| 8 | 3 | 80 | 170 | 49 |
| 9 | 1 | 93 | 172 | 33 |
| 9 | 5 | 56 | 126 | 35 |

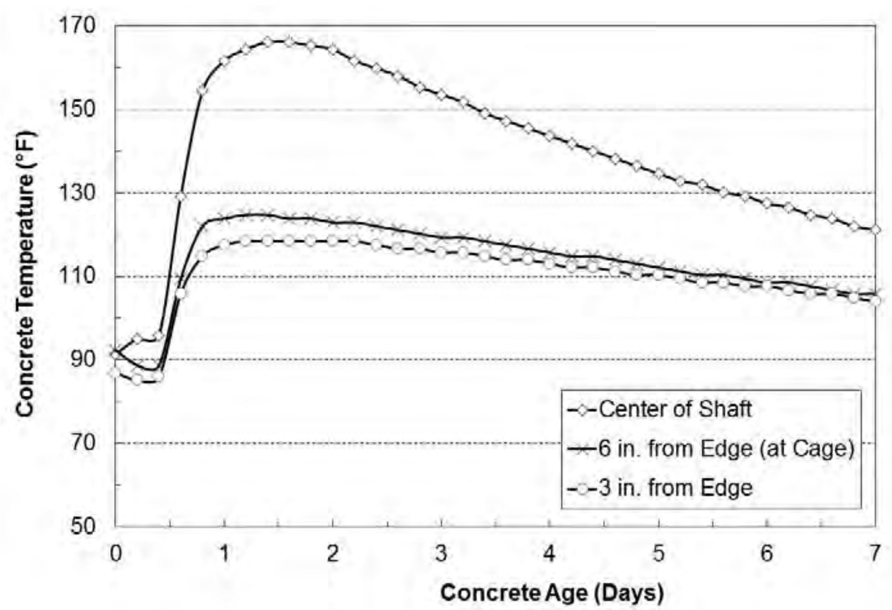



**Figure 4.28:** Temperature measurements from Shaft No. 8 of Pier No. 1 at shaft mid-height

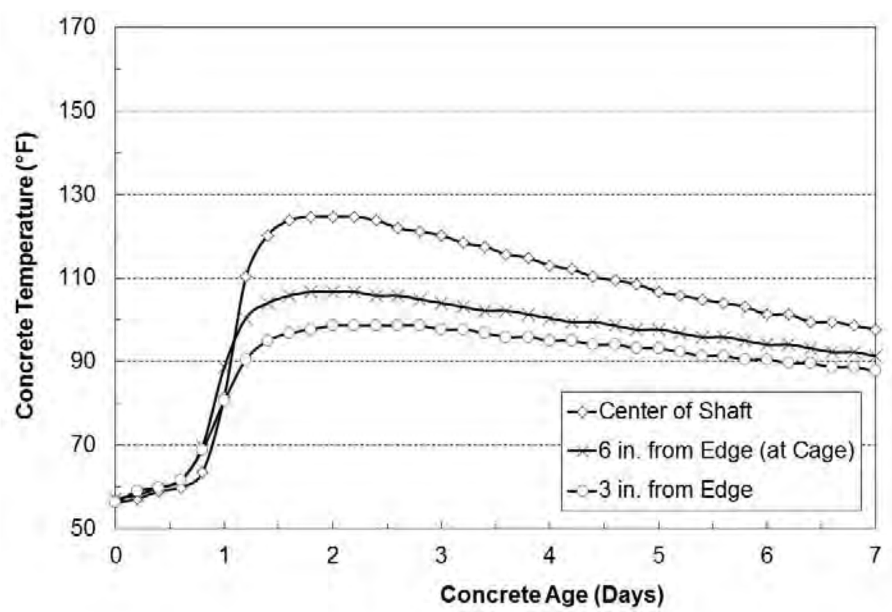



**Figure 4.29:** Temperature measurements from Shaft No. 5 of Pier No. 9 near shaft top

Delayed-ettringite formation (DEF), may develop in plain portland cement concrete structures exposed to humid environments when the early-age concrete temperatures exceed 158 °F (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. Note from Table 4.6 that the maximum concrete temperature exceeded 158 °F in all the shafts placed under warm weather conditions. When using at least 25 percent Class F fly concrete temperatures up to about 170°F can be tolerated without significant concerns of DEF (Folliard et al. 2006). Therefore, since this SCC mixture contained 30 percent Class F fly ash and the maximum concrete temperature recorded was 172 °F, the risk of DEF is unknown. It is recommended to evaluate the use of mass concrete specifications for drilled shaft construction to consider limiting the maximum in-place concrete temperatures reached in large-diameter shafts.

Large differences between the concrete core and surface temperatures may also lead to the development of excessive internal stresses and eventually cracking, which may reduce the structure's service life (ACI 207.2R 2007). In an effort to reduce the potential risk of thermal cracking in mass concrete structures, most mass concrete specifications limit the maximum allowable temperature difference between the core and the surface of the concrete to 35 °F (Gajda and Alsamsam 2006). It can be seen from Table 4.6 that in two of the instrumented shafts the measured temperature difference exceeded the 35 °F threshold. It is recommended to evaluate the use of mass concrete specifications for drilled shaft construction to consider limiting the maximum concrete temperature difference reached in large-diameter shafts.

## 4.10 CONCRETE INTEGRITY TEST RESULTS AND DISCUSSION

### 4.10.1 CSL Test Results

The CSL results performed by GMS Testing, Inc., Muscle Shoals, Alabama and obtained from ALDOT are summarized in this section. The results were as follows:

- **Pier No. 7**
  - Shaft No. 1—The shaft had less than 10% velocity deviation from average throughout its tested length. There was no indication of any significantly anomaly or inclusion. Debonding between the concrete and the CSL tube was noted 8 ft below the top of the shaft. ALDOT deemed this shaft acceptable as built.
  - Shaft No. 2—CSL testing on this shaft was done when the concrete was 94 days old, and this affected the results. ALDOT specifications require that CSL testing be performed within 20 calendar days after concrete placement. The results indicate that the shaft was satisfactory throughout the tested length and ALDOT deemed this shaft acceptable as built.
  - Shaft No. 3—The shaft had less than 10% velocity deviation from average throughout its tested length. There was no indication of any significantly anomaly or inclusion. ALDOT deemed this shaft acceptable as built.
  - Shaft No. 4—This was the first shaft constructed and problems were experienced during its construction as previously discussed in Section 4.8.1. The shaft had less than 10% velocity deviation from average throughout its tested length. However, multiple anomalies were detected and were attributed to tremie blockage and cement

balling issues.  ALDOT deemed this shaft unacceptable as built.  This shaft was retrofitted by installing four micropiles to restore the design capacity of the shaft.

- Shaft No. 5—This shaft was 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 occurred in only the top 2.5 ft.  Debonding at the top often occurs when tested late.  The shaft had less than 10% velocity deviation from average throughout its tested length. There was no indication of any significantly anomaly or inclusion.  ALDOT deemed this shaft acceptable as built.

- **Pier No. 8**
  - Shaft No. 1—The shaft had less than 10% velocity deviation from average throughout its tested length.  There was no indication of any significantly anomaly or inclusion. Debonding between the concrete and the CSL tube was noted 3 ft below the top of the shaft.  ALDOT deemed this shaft acceptable as built.
  - Shaft No. 2—The following CSL testing problems were experienced: 1) the tubes extended 45 ft above the concrete surface, which could have caused debonding, and 2) tube no. 2 was blocked 4 ft from the bottom by silt or trash.  A significant variation in transit time was noted between tubes 2 and 3, which ALDOT attributed to the presence of an obstruction at the bottom of Tube 2.  The rest of the shaft had less than 10% velocity deviation from average.  There was no indication of any significantly anomaly or inclusion.  However, since signal loss occurred at the bottom of Tube 2, this shaft was accepted as built with a six percent price reduction.
  - Shaft No. 3—The shaft had less than 10% velocity deviation from average throughout its tested length.  There was no indication of any significantly anomaly or inclusion. Minor debonding was noted between the concrete and the CSL tube around the top of the shaft.  ALDOT deemed this shaft acceptable as built.
  - Shaft No. 4—The shaft had less than 10% velocity deviation from average throughout its tested length.  There was no indication of any significantly anomaly or inclusion. Minor debonding was noted between the concrete and the CSL tube around the top of the shaft.  ALDOT deemed this shaft acceptable as built.
  - Shaft No. 5— The shaft had less than 10% velocity deviation from average throughout its tested length.  There was no indication of any significantly anomaly or inclusion.  Minor debonding was noted between the concrete and the CSL tube around the top of the shaft.  ALDOT deemed this shaft acceptable as built.
- **Pier No. 9**
  - Shaft No. 1—The shaft had less than 10% velocity deviation from average throughout its tested length.  There was no indication of any significantly anomaly or inclusion.

Minor debonding was noted between the concrete and the CSL tube around the top of the shaft. ALDOT deemed this shaft acceptable as built.

- Shaft No. 2—The shaft had less than 10% velocity deviation from average throughout its tested length. There was no indication of any significantly anomaly or inclusion. Minor debonding was noted between the concrete and the CSL tube around the top of the shaft. ALDOT deemed this shaft acceptable as built.
- Shaft No. 3—The shaft had less than 10% velocity deviation from average throughout its tested length. There was no indication of any significantly anomaly or inclusion. ALDOT deemed this shaft acceptable as built.
- Shaft No. 4— The shaft had less than 10% velocity deviation from average throughout its tested length. There was no indication of any significantly anomaly or inclusion. ALDOT deemed this shaft acceptable as built.
- Shaft No. 5—The shaft had less than 10% velocity deviation from average throughout its tested length. There was no indication of any significantly anomaly or inclusion. ALDOT deemed this shaft acceptable as built.

### 4.10.2 Gamma-Gamma Test Results

Gamma-gamma testing primarily detects anomalies within a 5 to 6 in. radius from the inspection tubes both inside and outside of the rebar cage. The gamma-gamma results performed by COLOG, a division of Layne Christensen Company, Lakewood, Colorado are summarized in this section. In this section, the gamma-gamma testing results indicate a "questionable" concrete condition when the reduction in density is between 2-3 standard deviations from mean, and "poor" concrete condition when the reduction in density is greater than 3 standard deviations from mean. The results were as follows:

- **Pier No. 8**
  - Shaft No. 1—No questionable or poor anomalies were detected around the inspection tubes.
  - Shaft No. 2—No poor anomalies were detected around the inspection tubes. One questionable anomaly was detected in the last 1 ft of Tube 2. This location coincides with the issues described with CSL testing where tube no. 2 was blocked from the bottom by silt or trash.
- **Pier No. 9**
  - Shaft No. 2—No questionable or poor anomalies were detected around the inspection tubes.
  - Shaft No. 3—No questionable or poor anomalies were detected around the inspection tubes.

- Shaft No. 4—No questionable or poor anomalies were detected around the inspection tubes.

#### 4.10.3 Discussion of Concrete Integrity Test Results

Both crosshole sonic logging (CSL) testing and gamma-gamma testing were performed. Only in the first shaft constructed (Shaft No. 4 of Pier No. 7), were anomalies detected that were attributed to poor concrete quality within the shaft. These anomalies were attributed to tremie blockage and cement balling issues discussed in Section 4.8.1. In the remainder of the shafts, both means of integrity testing indicated that the use of self-consolidating concrete produced good quality in-place concrete.

### 4.11 CHAPTER CONCLUSIONS

Based on the work documented in this chapter, the following can be concluded:

- A test shaft should be constructed for all projects to check the concrete and construction methods used.
- Technician training is required to ensure that all SCC test methods can be performed in accordance with specifications.
- Since the slump flow of the SCC arriving at the jobsite can vary, it is recommended that testing of fresh SCC properties be conducted more frequently than once every 50 $yd^3$ to make sure the concrete placed meet the project specifications. A testing frequency of once every 25 $yd^3$ is recommended to assess the fresh properties of SCC used for drilled shafts.
- 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.
- Based on the CSL and gamma-gamma test 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 OVERVIEW

The primary objective of this research was to determine the effectiveness of using self-consolidating concrete (SCC) in drilled shafts placed under full-scale production conditions. Two large-scale, field projects were conducted to examine the feasibility of using SCC in drilled shafts. The first project compared ordinary drilled shaft concrete to two types of SCC in three, 6 ft diameter, *experimental* shafts. Based on the results from the experimental shafts, ALDOT approved the first-use of SCC in production shafts in the state of Alabama. The second project thus pertains to *production* shafts installed for a bridge across the Tennessee River in Alabama.

The experimental shafts were also constructed near AL-35 close to Scottsboro, Alabama. The following drilled shaft concrete mixtures were used in these shafts:

- One with an ordinary drilled shaft concrete (ODSC) mixture used in a completed ALDOT project in North Alabama,
- One with SCC specifically proportioned for drilled shaft applications (SCC), and
- One with an experimental SCC that used limestone powder (SCC-LP).

The concrete for the experimental shafts was placed using a pump line, 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 measured for the ODSC mixture. Fresh concrete properties such as slump flow, total air content, unit weight, bleeding, temperature, and segregation were measured for both SCC mixtures. Thousands of colored mortar cubes were also added to each test shaft at different intervals to allow the research team to assess how the concrete flowed from the tremie pipe into each shaft. These shafts were exhumed 30 days after concrete placement. After exhuming, the shafts were cut to visually inspect concrete uniformity and to survey the final location of the colored mortar cubes. Cores were also recovered to assess the hardened properties of the in-place concrete, both inside and outside of the reinforcement cage.

Based on the results from the experimental shafts, 15, 8 ft diameter, production drilled shafts were constructed. The *production* shafts were installed for Phase II of the AL-35 Southbound bridge (called the B.B. Comer Bridge) across the Tennessee River near Scottsboro, Alabama. The research performed during this phase of this project included the following:

- Documenting the placement and properties of the drilled shaft concrete,

- 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 regions of the shaft during concrete placement, and
- Measuring the temperature development due to hydration of the concrete.

In addition to the experimental and production shafts, a prototype pressurized bleed test was developed to assess the concrete's bleeding potential under pressure. Various pressure bleed tests were conducted during this research to determine the best process and methods for performing this test.

## 5.2 CONCLUSIONS

The results of this study support the following conclusions:

### 5.2.1 Conclusions from Experimental Shafts

- Both self-consolidating concrete shafts showed significantly better consolidation than the ODSC in the cover region of the drilled shafts.
- 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 when compared to shafts constructed with SCC type mixtures (slump flow 18 to 24 in.). The compressive strengths of the cores from the ODSC shaft differed the most between the inner and outer locations, indicating poor quality concrete was present in the cover region.
- The SCC shaft had the best in-place properties of the three experimental shafts. The cores taken from this shaft had the most consistent compressive strength, modulus of elasticity, and permeability values throughout the cross sections, the visual inspection of the cut concrete showed the fewest imperfections, and its CSL results revealed the fewest minor defects in this shaft.
- The surface of the SCC flowed upwards, in a horizontal manner, unobstructed by the congestion provided by the reinforcement cage used in this project.
- The addition of limestone powder to the drilled shaft concrete, lowers the bleed potential of the concrete. However, it is recommended to perform more full-scale tests where limestone powder is used to improve the properties of drilled shafts.

### 5.2.2 Conclusions from Evaluating the Concrete Flow in the Experimental Shafts

- Concrete forced out of the bottom of the tremie does not flow in a perfectly laminar manner, 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 occurs between different layers of concrete in the drilled shaft.
- Higher viscosity concrete, such as ordinary drilled shaft concrete, flows in a layered manner upwards around the tremie and then outward near the top concrete surface as shown in Figure 3.71. Lower viscous concrete, such as SCC, flows in a bulging manner with less of the concrete flowing directly upwards around the tremie, and instead it bulges laterally when existing the tremie, which displaces the previously placed layers upwards to fill the shaft, as shown in Figure 3.72.

### 5.2.3 Production Shaft Findings

- Concrete placement problems (cement balling and leaking tremie) were experienced during the first two production drilled shafts. Due to these problems, it is recommended that 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.
- Technician training is required to ensure that all SCC test methods can be performed in accordance with specifications.
- A knowledgeable and well-qualified concrete producer is required to produce SCC with acceptable properties for drilled shaft applications. Producer attention to controlling the water content of the mixture and fresh properties of the SCC is needed for a successful project.
- Concretes with high powder contents and low water-to-cementitious materials ratios, like SCC, may form cement balls and this phenomenon can be avoided by using a suitable batching sequence.
- After the initial start-up problems were resolved, the SCC placed much easier than the conventional drilled shaft concrete. The SCC easily flowed through the tremie and the tremie did not need to be moved to establish or maintain concrete flow. Although the tremie did not need to be moved during placement of a shaft, no problems were experienced when extracting the tremie from the shaft.
- Since the SCC fresh properties can vary at the jobsite, it is recommended that testing of fresh SCC properties be conducted more frequently than once every 50 $yd^3$ to make sure the concrete placed meets the project specifications. A testing frequency of once every 25 $yd^3$ is recommended to assess the fresh properties of SCC used for drilled shafts.
- SCC placed with slump flow values below the minimum slump flow value (18 inches) 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).

- Based on the CSL and gamma-gamma test results, SCC produces shafts with acceptable in-place integrity throughout the shaft.
- Properly designed and installed SCC can produce high-quality drilled shafts with little to no 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

The following additional research is recommended to evaluate the use of SCC in drilled shafts:

- More research should be conducted to develop drilled shaft concrete with low bleeding potential. The addition of limestone powder lowers concrete bleeding. However, in this project the SCC made with limestone powder had variable fresh concrete properties. Since the addition of limestone powder may also be beneficial to lower heat of hydration in large shafts, more research is needed on the use of limestone powder in concrete mixtures used for drilled shaft construction.
- Future research is recommended to evaluate the pressure 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 was increased from the standard ASTM spacing, 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 is recommended to assess the flow of high- and low-viscosity concretes during tremie placement of drilled shafts.
- Research is needed to develop a less subjective fresh concrete test than the Visual Stability Index (VSI) to rapidly assess the SCC stability in the field.
- In-place concrete temperatures in excess of the DEF threshold of 158 °F was measured in the production shafts. However, since this SCC mixture contained 30 percent Class F fly ash and the maximum concrete temperature recorded was 172 °F, the risk of concrete specifications for drilled shaft construction to guard against the possible formation of delayed-ettringite formation (DEF) is uncertain. It is recommended to develop mass concrete specifications for large-diameter drilled shafts to guard against the possible formation of DEF.

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