Civil & Environmental Engineering

FACTORS AFFECTING ANOMALY FORMATION IN DRILLED SHAFTS - FINAL REPORT

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CONVERSION FACTORS, US CUSTOMARY TO METRIC UNITS

Multiply	***by***	***to obtain***
inch	25.4	mm
foot	0.3048	meter
square inches	645	square mm
cubic yard	0.765	cubic meter
pound (lb)	4.448	Newtons
kip (1000 lb)	4.448	kiloNewton (kN)
Newton	0.2248	pound
kip/ft	14.59	kN/meter
pound/in^2	0.0069	MPa
kip/in^2	6.895	MPa
MPa	0.145	ksi
kip-ft	1.356	kN-m
kip-in	0.113	kN-m
kN-m	.7375	kip-ft

PREFACE

This research project was funded as a supplemental contract awarded to the University of South Florida, Tampa by the Florida Department of Transportation. Dr. David Horhota was the Project Manager. It is a pleasure to acknowledge his contribution to this study.

This project was carried out in part with the cooperation and collaboration of Auburn University. The contributions provided by this institution are greatly appreciated with particular acknowledgment to Dr. Dan Brown.

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Special thanks are extended to the College of Engineering's Machine Shop staff of Mr. Bob Smith, Mr. James Christopher, and Mr. Tom Gage for their unending support. The assistance of graduate researchers (*not in any order*) Mr. Byron Anderson, Mr. Kadir Uslu, Mr. Greg Deese, Dr. Ed Garbin, Mr. Kevin Johnson, Mrs. Sonia Lowry, Mr. Michael Stokes, Mr. Van Wagner and Mr. Danny Winters as well as undergraduate assistant Mr. Ryan Florence is duly recognized.

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EXECUTIVE SUMMARY

In instances where excavation around drilled shafts has been required (e.g. footings, foundation caps, etc.) imperfections or anomalous conditions have frequently been observed. In many instances this was thought to have been caused by the presence of the water table. This seemingly alarming trend prompted the Florida Department of Transportation to sponsor a research program aimed at revealing the mystery of this phenomenon.

Upon reviewing the preliminary research findings this study defined factors that likely affected the occurrence of these conditions. Primary focus was directed at borehole cleanliness, concrete placement techniques, slump, clear spacing of rebar, aggregate size, and placement of concrete under a fluid head (such as drilling slurry).

Laboratory testing in the Lateral Pressure Cell was conducted to investigate the relationship between lateral pressure development and slump during pour, coarse aggregate size, clear spacing of rebar, and fluid head in the borehole. The most interesting finding of this series of tests was that the rebar clear spacing to aggregate diameter ratio of 3 to 5, which is most often specified, leads to substantial build-up of material inside the cage before enough pressure is developed to push the shaft mix through the cage to the annular volume outside the cage.

Field testing on full scale drilled shafts on numerous sites and over 40 data sets was then conducted in order to corroborate the findings in the lab. Using a down-hole camera and/or weighted tape measurements, head differentials between inner and outer cage material were found to be excessively large even when using common mixes and rebar spacing. This build-up was found to be closely related to the clear spacing of the rebar, aggregate size, and rate of concreting.

A second series of laboratory testing in the Frustum Confining Vessel evaluated construction factors affecting finish shaft performance (e.g. casing extraction rate, slump, and slump loss). These tests found that when using the temporary casing method of construction, the unit skin friction developed by the shaft was drastically reduced when the slump of the concrete was allowed to drop below 5 inches prior to pulling the casing. At slumps of 3.5 inches or less, most shafts were damaged during casing extraction, and those that appeared fine developed nearly zero unit skin friction.

Finally, a large scale concrete pour simulator was designed and fabricated to investigate the effect of slurry properties and sand content on accumulation (settling soil particles) at the bottom of the excavation and/or a rising concrete-slurry interface. Results showed that almost half of the suspended sand would fall out of suspension within 2 hours. If left undisturbed, the remaining sand stayed in suspension up to 12 hours (the longest test run).

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1. INTRODUCTION

Drilled shafts are large diameter cast-in-place concrete foundations that extend deep into the ground up to hundreds of feet. As the form-work (typically soil) is rarely removed, the actual shape and quality of the concrete goes largely unverified. In some instances where the shafts have been exhumed or at least partially exposed, aberrant conditions have been found. These anomalies were observed in the form of soil inclusions, concrete segregation, or cross-section reductions. This project investigates many of the mechanisms that lead to compromised shaft integrity.

The original problem statement for this project focused on the effects of the water table elevation on the integrity of drilled shaft foundations. This was in response to many anomalies found upon excavation around test shafts or for footings and their location appeared to coincide with the location of the water table. Augercast piles, although quite different in construction, were also known to exhibit this problem (Figure 1-1).

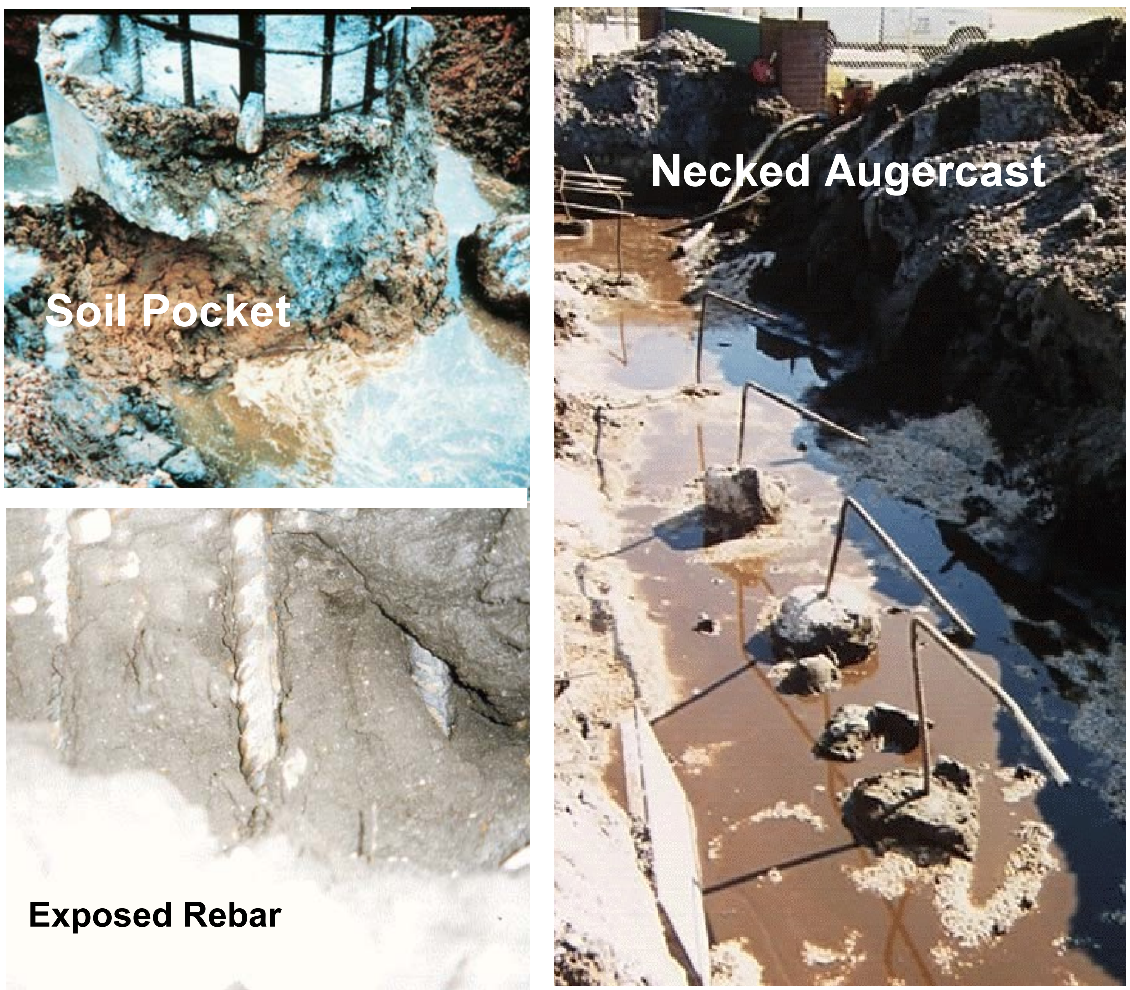

Figure 1-1. Soil Inclusions, Exposed Rebar, and Necking Found Near Water Table.

A review of construction sites where anomalies could be exposed, showed that anomalies could appear in all locations along the shaft length and were apparently caused by numerous construction related factors (Figure 1-2). Further, initial lab tests concluded that the location of the water table did not contribute to the formation of anomalies in any of the lab scale specimens. In fact, when imperfections in the finished shafts were observed, almost always the location of these was markedly different from the location of the water table during casting. More critical, it seemed, were factors such as borehole cleanliness, construction (concrete placement) techniques, slump loss during the pour, slurry properties, and borehole open time.

Figure 1-2. Anomalies in cased construction caused by concrete quality (top) and in slurry supported construction caused by bottom of excavation soil accumulation (bottom).

After discussion of these early findings with FDOT personnel, these observations ultimately led to a broadened scope of research which included the factors listed above, as well as additional items such as clear spacing of rebar and placement of concrete under a fluid head (such as drill slurry) versus a dry hole. This report will present the findings from both laboratory and field testing aimed at more accurately describing the effects of the aforementioned parameters on finished shaft integrity and axial capacity. The organization of this report is presented below.

Chapter 2 will introduce the original problem as outlined in the USF proposal submitted to the FDOT. Following this, the findings of a comprehensive review of literature on topics such as drilled shaft history, techniques of drilled shaft construction, borehole stabilization methods, and quality assurance testing will be presented.

A new laboratory device is presented in Chapter 3. This device, designated the *Lateral Pressure Cell*, was designed and constructed specifically for this study in an effort to better understand the effects of such parameters as slump loss and clear spacing of rebar on the flow of fresh concrete. The results of the lateral pressure cell tests are presented and discussed. These tests focused on shaft integrity and quality, as affected by various construction related parameters.

In Chapter 4, the *Frustum Confining Vessel* is introduced with a standard testing procedure for pressurization, casing installation, excavation, and specimen casting. The results of the lab scale study are presented. The focus is on the results of three separate series of tests in the frustum confining vessel. These tests targeted the effects of various construction parameters on the axial capacity of a shaft.

Chapter 5 presents a series of tests designed to address drill slurry properties as well as the effect of sand content on settling time. A discussion of the design, fabrication, and testing using a large scale concrete pour simulator are presented that was used to simulate an open excavation filled with mineral slurry. A summary of the results is included.

Chapter 6 discusses the full scale testing program carried out in conjunction with Auburn University. A total of five shafts were constructed, and video footage of the rising concrete in the hole was obtained through the use of an experimental down-hole camera known as the *borescope*. Several non-destructive tests were also carried out to show the effectiveness of detecting known anomalies. The findings from the Auburn full scale test program are presented.

In addition to the full scale work addressed in Chapter 6, additional monitoring of drilled shafts during the concreting process was undertaken. The primary focus of this work was to capture the head differential between the inside and outside of the reinforcing cage when concrete was placed. Chapter 7 discusses the specifics of each construction site visited as well as the results of those efforts.

Finally, Chapter 8 concludes this report with a summary of the important findings of the study. Additionally, some general recommendations for construction procedure changes are presented, and suggestions for possible avenues of future research are given.

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2. BACKGROUND

The original proposal written by the University of South Florida and submitted to the Florida Department of Transportation, entitled "The Influence of Water Table in Drilled Shaft Construction," was prepared in response to what seemed to be a recurring problem in drilled shaft construction. Although drilled shaft capacity is closely linked to soil type, it has been long understood that construction practices can drastically affect anticipated capacity in addition to integrity and durability. A scenario that has plagued numerous construction projects to date involves anomalies occurring at the water table elevation. Although many types of anomalies can occur during normal shaft construction, this specific type of flaw is manifested in the form of necking or a reduced section. As a result the structural integrity and corrosion durability is compromised. The full extent of this problem is often unknown and/or unrealized. This condition is not limited to drilled shaft construction as there is evidence of this scenario occurring in other types of bored piles such as auger cast-in-situ (augercast) piles.

In order to properly present the findings of the research project, a thorough review of available literature was first conducted. Topics of concern include a brief history on the evolution and use of drilled shaft foundations, varieties, construction and borehole stabilization methods, excavation clean out techniques and equipment, and quality assurance (shaft integrity) methods. Additionally, any previous research related to this report will be summarized.

2.1 Historical Background of Drilled Shafts

The primary reason for using deep foundations is to transfer structural loads and moments through the relatively weak upper strata of many sites to deeper, stronger geomaterials having sufficient bearing capacity for the anticipated loading. These structural loads are often the result of very heavy buildings, tall buildings having considerable weight and wind loads, and bridges spanning long distances which require foundations capable of resisting substantial dynamic loads in addition to large dead loads. As such, the use of deep foundations, and in particular drilled shafts, can be traced back to the early 1900's and the beginning of the growth of major cities such as Chicago and Detroit (Bowles, 1996).

As populations grew and the industrial revolution progressed, buildings started reaching higher and higher. Shallow foundations were simply inadequate to support these structures since the soil beneath these foundations did not possess the needed strength. Early civil engineering pioneers began considering ways to transfer the enormous loads of these superstructures directly to bedrock or other high strength subsurface strata.

As city buildings grew larger and higher during the early 20th century, hand-excavated caissons became popular. There were primarily two methods of construction: the Chicago method and the Gow method (O'Neill and Reese, 1999). In the Chicago method, workers dug to a depth equal to the length of stave boards used to shore up the walls of the excavation. These boards were placed against the internal walls of the borehole, and were held in place using compression rings (similar to the construction of a wooden barrel but opposite in force application). Excavation then proceeded, in increments equivalent to the length of the staves, until the desired depth was reached. The Gow method was similar to this, except that a telescoping wall liner (casing) was used in place of the stave boards. This method resulted in a reduction in cross section of the shaft as the excavation was made deeper.

As excavations needed to be constructed to greater depths and required larger diameters, man-power was replaced with machines powered by teams of horses (horse-power). Early model auger machines could bore a 12 inch diameter hole to a depth of up to 30 feet. When rotary auger machines incorporated horse-power, greater depths became possible. Motorized, truck-mounted boring machines began to appear in the early 1930's. A.H. Beck of San Antonio, TX and Hugh B. Williams of Dallas, Texas were among the first to develop these devices, first for digging shallow holes and later for drilled shaft excavation.

2.2 Varieties of Drilled Shafts

Bowles refers to the general case of a rotary drilled, cylindrical earthen hole filled with concrete as a *drilled pier*, and lists the commonly used types as follows:

(1) Drilled shaft (used herein)
(2) Drilled caisson, or simply a caisson
(3) Bored pile, when the diameter is less than about 30 inches (1996).

When the base or tip of the shaft is drilled to a larger diameter than that of the shaft itself (a process known as *underreaming*), two additional nomenclatures are possible:

(1) Belled pier or belled caisson
(2) Underreamed foundation.

The term "caisson" is used, in addition to the above, as a classification of early excavation systems employing pre-made box structures. These weighted boxes, which were typically made of wood, were placed on the site to be excavated (normally under water) and filled with pressurized air. Laborers inside the box hand-dug the soil beneath them causing the caisson box to progressively sink deeper into the ground. Once the desired depth was reached, the workers were removed and the structure was filled with concrete and used as a base in the foundation system. This type of caisson is markedly different from the auger bored piers of today, and is not considered a type of drilled shaft.

There are other types of deep foundation systems that utilize technologies such as percussion digging (for excavation in rock), clamshell grab buckets, and auger-grouted excavations. These systems are not considered as drilled shafts. However, some of the construction issues associated with hole stability of grab bucket excavation are also similar to those encountered in conventional drilled shaft construction.

2.3 Methods of Construction

Most literature agrees that there are three main types of drilled shaft construction. These are the dry method, the casing method, and the slurry or wet method. On some sites it may be necessary to employ more than one of these methods, and in some instances a combination of techniques may be utilized on one borehole location. An overview of each of these methods is presented in this section.

2.3.1 Dry Method

In the dry method of construction, soil is excavated using a rotary auger tool and the corresponding borehole is left unsupported. This method is used in non-caving, cohesive soils, which are generally located above the local ground water table (GWT). As the auger tool advances, it must be removed from the hole periodically to place the spoil material aside for later removal. Once the desired depth is reached, a clean-out bucket can be used to remove any remaining loose debris from the bottom of the borehole, and if desired, underreaming can be completed. Once the excavation is completed, concrete is normally placed by means of a tremie, although in some locals it is allowable to place the concrete by free fall from the top of the excavation. This can have undesirable consequences, however, as segregation of the concrete may occur, and partial or full caving of the borehole may be induced by concrete striking the wall of the excavation as it falls. However, some research reports that this is not the case (STS, 1994).

When temporary casings are to be used, care must be taken to ensure that the steel is free of any old concrete, oils, or other contaminants that may prove detrimental to the integrity of the finished shaft. Although it is common place in the U.S. to use recycled pipe for temporary casing, if the walls of the pipe are not clean and relatively smooth, problems may be encountered during extraction. Increased adhesion forces between the walls of the casing and the fluid concrete can, in effect, drag the freshly placed concrete upward while the casing is being extracted. This drag can ultimately lead to necking in the shaft. In addition, any old concrete left on the outside of the casing can chip away and drop into the fluid concrete as the casing is extracted, causing imperfections and possible weak points in the finished shaft (FHWA, 1997).

If the drilled shaft is designed to resist tensile stresses (as would occur from the application of a bending moment) then a steel rebar cage will have to be placed into the hole. This can be done prior to pouring concrete, or some concrete can first be placed, then the cage set at the required depth before completing the pour. In any case, when using a rebar cage in a drilled shaft, care must be taken to ensure the rebar does not come in contact with the soil, especially at the base of the shaft. This could lead to a corrosion problem which will undermine the effectiveness of the foundation system.

The dry method of construction can sometimes be completed to depths greater than the location of the GWT if the geomaterial being penetrated has a low permeability, or if the drilling and concreting operations are completed rapidly enough that no significant amount of water enters the borehole.

2.3.2 Casing Method

The casing method of construction employs some type of borehole liner, usually in the form of a simple steel pipe. Construction of a drilled shaft using the casing method is used on sites where caving or excessive lateral deformation of the excavation is probable, or when the geomaterial on the site is stable until cut. Additionally, it can be used when it is desired to seal a borehole from the GWT.

The cased method can use wet or dry drilling techniques. When using this method of construction, a temporary structural reinforcing sleeve (casing) is installed in the excavation to provide the lateral support necessary for maintaining the integrity of the hole. The temporary casings are generally installed such that they extend into an impervious formation, such as rock, and are left in place until the concrete is placed. These casings are installed using any one of a variety of procedures. Two popular casing installation techniques are vibrated or driven and twisted (oscillated). Vibrated casings are usually continuous pipe sections driven to the required depth of a competent formation. This is generally a fast and efficient method of installing a temporary casing, but if competent rock or the target "good layer" is at a variable depth, cutting and welding of the casing becomes necessary. Additionally, vibration of existing structures in the vicinity of the excavation is a concern. Twisted, sectional casings are a good alternative when vibrated casings become problematic. Because they are installed segmentally, no cutting or welding of the steel is required, and the vibration of adjacent structures is not an issue. However, the installation time is greater than that for vibrated casing, and specialized equipment is also necessary.

The casing can be installed prior to, during, or immediately after drilling. It is often necessary to install cutting teeth on the bottom of the casing to allow it to core into rock or other strong material. The casing can also be installed after the borehole has been drilled, though it may be necessary to fill the hole with slurry to stabilize it until the casing is placed. Then, the slurry must be completely bailed from the inside of the casing prior to placing a rebar cage and pouring concreted (Bowles, 1996).

When using a casing to construct a drilled shaft, it can either be removed after the placement of concrete is complete (temporary casing) or left behind to become an integral part of the foundation (permanent casing).

2.3.3 Wet (Slurry) Method

In any situation where the casing method is applicable, the wet method (a.k.a. slurry method) can optionally be used. The premise behind the wet method is that by maintaining a fluid pressure within the borehole that is at a higher level than the piezometric surface on the site, inward flow into the borehole is prevented and the likelihood of caving is reduced substantially. The wet method utilizes one of the three classifications of drilling fluid: natural (freshwater or saltwater), mineral (bentonite, attapulgite, or sepiolite), or polymer.

When excavating a drilled shaft using the wet method, two methods of removing the spoil from the borehole are typically employed. The first, and most common according to O'Neill and Reese, is the static method (1999). In this method the cuttings are transported to the surface by means of the drilling tool, and the fluid is left in the borehole while drilling advances. The other is the reverse circulation method of drilling. Here, the drilling fluid is continuously pumped out of the borehole using a vacuum pump which is hose-connected to the hollow stem of the auger tool. As drilling progresses, cuttings are forced to the center of the borehole under the drill bit, and are removed using the vacuum pump along with any drilling fluid in the vicinity. The removed material is then passed through a series of screens which removes the spoil from the drilling fluid. This "conditioned" drilling fluid is then returned to the borehole while drilling continues to the desired depth. The entire system forms a closed loop, and the fluid head within the borehole is always maintained at a level above the local GWT.

2.4 Drilling Slurries

Drilling fluids commonly used today are slurries of either naturally occurring minerals or synthetic polymers and water. In the first type, mineral slurries, bentonite is mixed with the water to create a slurry with a unit weight that is moderately higher than water alone. Bentonite, which is a processed, powdered sodium montmorillonite clay, consists of microscopic plate-like particles that remain suspended in the mixing water. When this slurry is placed into an excavation and maintained at a higher head than the natural piezometric surface, the suspended particles permeate the walls and form a *mud-cake* layer that helps to stabilize the hole.

Synthetic polymer slurries are a relatively new alternate to mineral slurries. However, they are only currently accepted in 12 out of the 21 states that have drilled shaft specifications. Table 2-1 lists each of the 50 states and notes permissible slurry types. In polymer slurries, the active mechanism comes from very long chains of hydrocarbons. These chains lead to the characteristic strings that are visible during drilling. Similar to the permeation of the excavation walls by bentonite slurries, the hair-shaped hydrocarbon chains of polymer slurries serve to stabilize the excavation through continuous cohesion and drag forces as filtration of the chains into the walls occurs. Unlike bentonite slurries, synthetics do not leave a mud cake or have the gel capacity to suspend and transport drill cuttings for any appreciable time. Also, since the unit weights of most polymer slurries is just slightly higher than or equal to that of water, the slurry level must be kept significantly higher than the piezometric surface of the formation. FHWA recommends at least a +2 meter head differential (O'Neill and Reese, 1999).

Table 2-1. Summary of Slurry Acceptance From Individual State Specifications

	SLURRY ALLOWED	BENTONITE	ATTAPULGITE	OTHER MINERAL	POLYMER	YEAR
ALABAMA (AL)	Y	Y	Y	W/A	W/A	2002
	AL506.pdf					
ALASKA (AK)	Y	N	N	N	N	2002
	special provisions for water (natural) slurry only					
ARIZONA (AZ)	W/A	Y	Y	N	N	2002
	AZ609.pdf					
ARKANSAS (AR)	no formal drilled shaft specs					1996
CALIFORNIA (CA)	Y	Y	Y	Y	Y	1999
COLORADO (CO)	W/A*	Y	Y	Y	Y	1999
	*follow specs from FHWA 1999 construction procedures & design					
CONNECTICUT (CT)	no formal drilled shaft specs					1999
DELAWARE (DE)	No Drilled Shaft Construction Allowed					2004
FLORIDA (FL)	Y	Y	Y	Y	N	2004
	FLD455.pdf & FLD455S1.pdf					
GEORGIA (GA)	W/A	Y	N	N	N	2001
	special provisions, not in pdf format					
HAWAII (HI)	Y	Y	Y	Y	N	1994
	HI511.pdf					
IDAHO (ID)	no formal drilled shaft specs					2004
ILLINOIS (IL)	W/A	W/A	W/A	W/A	W/A	2004
	special provisions, IDOTspecprov.pdf					
INDIANA (IN)	no formal drilled shaft specs					1999
IOWA (IA)	Y	Y	Y	Y	Y	2004
	IADS-01038.pdf					
KANSAS (KS)	Y	Y	N	N	Y	2004
	KA5.pdf					
KENTUCKY (KY)	no formal drilled shaft specs					2004
LOUISIANA (LA)	Y	Y	Y	N	Y	2000
	LA814.pdf					
MAINE (ME)	no drilled shaft specs					2002
MARYLAND (MD)	Y	N	N	N	N	2001
	MD400-412changes2001.doc					
MASSACHUSETTS (MA)	no formal drilled shaft specs					1995
MICHIGAN (MI)	no formal drilled shaft specs					2003
MINNESOTA (MN)	no formal drilled shaft specs					2000
MISSISSIPPI (MS)	no formal drilled shaft specs					2003
MISSOURI (MO)	no formal drilled shaft specs					1999

Table 2-1. (Continued)

	SLURRY ALLOWED	BENTONITE	ATTAPULGITE	OTHER MINERAL	POLYMER	YEAR (as of)
MONTANA (MT)	no formal drilled shaft specs					1995
NEBRASKA (NE)	no formal drilled shaft specs					2002
NEVADA (NV)	Y	Y	Y	Y	Y	1997
	NV509.pdf					
NEW HAMPSHIRE (NH)	no formal drilled shaft specs					2002
NEW JERSEY (NJ)	no formal drilled shaft specs					2001
NEW MEXICO (NM)	Y	Y	Y	Y	Y	2000
	NM502.pdf					
NEW YORK (NY)	no formal drilled shaft specs					2002
NORTH CAROLINA (NC)	no formal drilled shaft specs					2002
NORTH DAKOTA (ND)	no formal drilled shaft specs					
OHIO (OH)	no formal drilled shaft specs					2000
OKLAHOMA (OK)	Y	Y	Y	Y	Y	1999
	OK516.pdf					
OREGON (OR)	Y	Y	Y	Y	Y	2002
	OR02-00500.pdf					
PENNSYLVANIA (PA)	Y	Y	N	N	N	2000
	PASection1006.pdf					
RHODE ISLAND (RI)	no formal drilled shaft specs					1997
SOUTH CAROLINA (SC)	Y	Y	Y	N	N	2000
	SC712.pdf					
SOUTH DAKOTA (SD)	N	N	N	N	N	2002
	SD465dual.pdf					
TENNESSEE (TN)	no formal drilled shaft specs					1995
TEXAS (TX)	Y	Y	N	N	N	1993
	TX416.pdf					
UTAH (UT)	N	N	N	N	N	2002
	UT0466.pdf					
VERMONT (VT)	no formal drilled shaft specs					2001
VIRGINIA (VA)	no formal drilled shaft specs					2002
WASHINGTON (WA)	no formal drilled shaft specs					2002
WASHINGTON DC	no formal drilled shaft specs					1996
WEST VIRGINIA (WV)	Y	W/A	W/A	W/A	W/A	2000
	WV_SUP_Y2K[1].pdf					
WISCONSIN (WI)	no formal drilled shaft specs					2004
WYOMING (WY)	W/A	W/A	W/A	W/A	W/A	1995
	WY506.pdf					

2.4.1 Effect of Slurry on Drilled Shaft Capacity

It is important to note that the use of drill slurries, either mineral or polymer, may have significant effects on the axial capacity of drilled shafts. Extensive research has been done to verify load carrying capacities suggested by current design practices. Most of this research is based on dry, cased, or mineral (bentonite) slurry construction methods in clayey or sandy subsurface environments. More recent research shows the effect of polymer slurry on perimeter load transfer and end-bearing capacity. However, little research has been conducted that investigates the effect of polymer slurry on borehole stability and design parameters for drilled shafts.

In permeable subsurface conditions, filter cake formation of bentonite slurry is essential in preventing the loss of slurry from the borehole. Additionally, the filter cake adds stability to the excavation by minimizing the required slurry head differential from the ground water elevation. However, the formation of a filter cake may also have a downside. As concrete is placed into the borehole through a tremie pipe, it displaces the bentonite slurry, but does not generate enough friction to remove the filter cake. Constant, adequate mixing of the slurry can help decrease the thickness of the filter cake, but it will not prevent it from forming all together. Studies have shown that the filter cake interferes with the bond between the concrete and the borehole wall, thus decreasing unit side shear. According to Brown (2002), shafts excavated and inspected several months after construction have a distinct *mud-cake* layer of approximately 1 to 3 mm separating the concrete from the surrounding soil. However, Thasnanipan (1998) states that if the following bentonite slurry parameters are maintained within the given ranges and construction time does not exceed 24 hours, there is no significant effect on the axial capacity: marsh cone viscosity of 30 - 60, pH between 7 and 11, and sand content of less than 4%. Drilled shaft specifications adopted by the Florida DOT closely resemble these ranges (Table 2-2), with the exception that prior to concrete placement, slurry is allowed to remain in the borehole up to 36 hours before the sidewalls must be overreamed. Though the upper limit on sand content is 4%, advances in drilled shaft construction procedures and equipment warrant stricter tolerances, and are in fact being implemented by other states (e.g. Louisiana and North Carolina).

Table 2-2. State of Florida Specifications for Road and Bridge Construction: 455-15.8.1

Item to be Measured	Range of Results at 20°C	Test Method
Density	1030 to 1170 kg/m^3 (in freshwater) 1060 to 1200 kg/m^3 (in saltwater)	Mud density balance: FM 8-RP13B-1
Viscosity	28 to 40 seconds	Marsh Cone Method: FM 8-RP13B-2
pH	8 to 11	Electric pH meter or pH FM 8-RP13B-4
Sand Content	4% or less	FM 8-RP13B-3

Most side shear degradation occurs during the first 24 hours of construction, but shaft capacity will still exceed the estimated design value. After 24 hours, there is less time dependance for side shear degradation, but side shear will be ultimately reduced to less than the estimated design capacity (Thasnanipan, 1998).

Polymer slurries do not form a filter cake. Fluid loss prevention depends on the formation of long chemical chains. Because there is no filter cake or boundary layer formed between the slurry and the sidewall, a nearly indistinct bond will form between the concrete and surrounding soil. Frizzi (2004) showed that shafts constructed and load tested in a sandy South Florida location exhibited 25% to 50% less side shear in the upper half of shafts constructed with bentonite slurry when compared to those constructed with polymer slurry. In a study conducted by Brown, et al. in 2002, two identical shafts were constructed using the wet method. One borehole was drilled using a bentonite slurry, while the other utilized a synthetic polymer slurry. Load tests conducted on the shafts after adequate curing time showed that the shaft constructed using the bentonite slurry had a much lower axial capacity than did the other shaft. In particular, a 300% increase in side friction resistance was observed with the shaft constructed using polymer. A study conducted with 11 drilled shafts constructed in Bangkok Subsoil (clay and dense sand layers) using polymer slurry showed that overall capacity was 1.5 times greater than calculated values determined for bentonite slurry method. Most of this capacity is believed to result from greater skin friction in sandy layers (Thasnanipan, 2002).

For shafts constructed in impermeable formations, the measured side shear was very similar (Ata, 1998). This may be because no filter cake forms in the borehole containing bentonite slurry and the concrete/soil bond is preserved (Ata, 1998 and Camp, 2002).

2.4.2 Borehole Stability and Particle Suspension

Maintaining hole stability at all times is paramount for all drilled shaft construction and is not slurry type specific. When the hole becomes unstable, the soil structure relaxes and changes the soil parameters used for design. The consequence is that the resultant shaft capacity in no way reflects the anticipated/designed capacity. Bentonite slurry is known to maintain hole stability as long as it is kept higher than the GWT. Polymer slurries have proven to maintain borehole stability, but maybe more sensitive to surrounding ground vibrations. If a borehole is filled with polymer slurry after sidewall sloughing has already begun, the hole will continue to collapse. However, if polymer slurry is added during excavation and prior to reaching the piezometric surface, the borehole stability will be maintained for more than 18 hours (Ata, 1998). Thasnanipan (2002) used cross hole sonic logging to show that borehole integrity was maintained for over 24 hours using polymer slurry in Bangkok subsoil.

Bentonite and other mineral slurries have an excellent ability to suspend solids due to the gel-like structure formed between clay molecules. Under this construction method, bottom cleanliness is usually maintained once it has been achieved within a reasonable time frame (1-2 hrs). Polymer slurries do not tend to suspend solids, especially sands. Sands and larger particles will settle very quickly in excavations filled with polymer slurry. Polymer constructed shafts have less end-bearing capacity possibly due to greater settlement of solids (Frizzi, 2004). However, no significant difference in end-bearing was observed for shafts drilled with polymer and mineral slurries through both sand and clay layers above the Cooper Marl formation which is a very stiff clay (Camp, 2002). This was surmised to have been because there was not a significant amount of sand particles that settled to the borehole bottom.

2.4.3 Slurry Economics

Construction costs can be greatly impacted by choosing mineral or polymer slurry. Costs can increase based on product yield and preparation or clean-up time required. Bentonite slurries require more clean up for reuse and must be treated prior to disposal. They must be desanded and then readjusted to the required rheological properties prior to use in a new excavation. This requires additional time and equipment, resulting in higher mobilization costs (Ballard, 2000). Bentonite is also known to be harmful to aquatic life and can be detrimental to normal groundwater flows because it creates an impermeable layer where it is disposed. Thus, it must be treated extensively before it can be disposed of at a regulated location.

Polymer (according to manufacturers) can be mixed directly in the borehole, or shortly before excavation in an onsite tank, and requires minimal clean up for reuse, and no clean up for disposal. It can also be disposed of anywhere as it is biodegradable and has no adverse environmental effects. Also, a larger quantity of bentonite is required for slurry mixing (10 to 40 tons of bentonite per 1 ton of most polymers). This can be more costly and requires more space for storage at the construction site (Beresford, 1989).

2.5 Excavation Clean out Techniques and Equipment

Regardless of the method of excavation, the finished quality of a drilled shaft depends largely on the cleanliness of the borehole. Current FDOT specifications (455-15.11.4) limit the amount of loose material remaining on the bottom of a borehole to no more than 1/2 inch across 50% of the base area of the shaft, and no more than 1.5 inches in any one location for most structures (FDOT, 2000). Visual inspection is accomplished using a Shaft Inspection Device or some derivation thereof, or by use of a diver. Other methods, such as sounding using a weighted tape are used at the discretion of the supervising engineer (Crapps, 1992). Removal of loose debris from a borehole can be completed using any one of several methods, the most common of which are:

(1) Cleaning bucket (or, clean out bucket)
(2) Air lift
(3) Sweeper air lift
(4) Three-in-one bucket
(5) Submersible pumps
(6) Over-reaming

A cleaning bucket is similar to a core bit except that it has the capability of being closed at the bottom so as to prevent material from falling out while the tool is lifted from the borehole. This tool, which is also known as a bail bucket or clean-out bucket, is nearly watertight when closed, and thus it can be used to remove drilling slurry from the excavation (Crapps, 1992). Ideally these buckets have one or more vent tubes located in them to prevent vacuum from developing beneath the tool as it is extracted. Because this phenomena could induce failure of the borehole through suction action on the walls, it is important to keep these passages clear.

Air lift systems are another means of removing loose debris from the base of a drilled shaft when the wet method of construction is used. In these systems, compressed air is introduced into the bottom of the borehole by means of a special pipe that extends to the surface. As this air is forced into the excavation, it creates bubbles that rise in the column of fluid therein. As these air bubbles ascend toward the surface, loose debris and slurry are carried upward through the pipe and are removed at the surface. The procedure is analogous to a vacuum cleaner.

Designed in Australia, the sweeper air lift system is similar to the standard air lift, except that it incorporates a brush that pushes loose material toward the center of the excavation. There, the debris is removed using a vacuum pump system as described previously. The three-in-one bucket is a combination tool that incorporates a sweeper air lift system into a clean-out bucket. Larger materials are collected in the bucket while finer materials are removed using the air lift system.

Specially modified submersible pumps are sometimes used at the bottom of a borehole. These pumps are placed onto risers so that they do not damage the bottom of the excavation. The pumps force slurry and suspended material out of the shaft through hoses that are connected to some type of spoil removal system (i.e. de-sanding unit), and at the same time fresh fluid is returned to the borehole from the top. This method of circulation continuously replaces the fluid column in the borehole until all suspended materials are removed (Huerstel, et. al., 1989).

Overreaming is a technique that is used to scrape the walls of a drilled shaft excavation in order to remove any filter cake left by the slurry as well as soft soils or clays left behind by the drilling process (Crapps, 1992). This is done to increase the side skin resistence of the finished shaft (Passe, 1993).

2.6 The State of Drilled Shaft Integrity Testing

The design of structural elements, including mass foundations, assures that the resistance of a specific structural element exceeds the Ultimate, Serviceability, and Specific limit states with an acceptable factor of safety. The magnitude of the factor of safety applied to a structural system has a direct correlation to the ability of the designer to verify the resistances from structural components. It is then verified that these resistances exceed the direct load effects. The factor of safety also has a direct effect on the overall size and cost of any structural system.

In the case of super structure members such as bridge girders, the inspection process can be straight forward, since the structural elements are easily inspected during manufacturing, shipping, and installation, yielding an efficient system that accomplishes the required task with little waste. However, substructure elements, such as drilled concrete shaft foundations, do not have the same liberties of inspection as the aforementioned counterparts. Therefore, several methods have been developed in an effort to determine the integrity of a completed drilled shaft. The most widely accepted methods are described herein along with a synopsis of the underlying physical principals or science that these tests are founded upon, and a discussion of each test's pros and cons.

The following subsections will investigate the current state of integrity testing. These methods can be categorized as either destructive or nondestructive and include: concrete coring, seismic echo, impulse response, cross-hole sonic logging, and density testing by downhole gamma-gamma logging.

2.6.1 Cross Hole Sonic Logging

Cross Hole Sonic Logging (CSL) is arguably the most widely accepted and used integrity testing method. CSL evaluates the uniformity and continuity of concrete by recording the velocity of signals from an emitter to a receiver, each inserted into the pile in preset tubes or pipes (Lewet al., el, 2002). In fact, Alabama Department of Transportation's *Specifications for Drilled Shaft Construction*, Section 506 states in 506.10(a)1, states that "the nondestructive testing method called Crosshole Sonic Logging (CSL) shall be used on all production and trial drilled shafts (a) when constructed with the placement of concrete under water or through slurry, (b) when required by special note on the plans, (c) when full length temporary casing is used to prevent water from entering the shaft, or (d) when determined to be necessary by the Engineer (ADOT, 2001)." In short, whenever there is a high probability of the existence of drilled shaft inclusions or a problem, Alabama requires CSL testing be performed. Alabama does not recognize or accept any other testing methods in their state specifications.

State of California Department of Transportation Engineering Service Center Division of Structures, *California Foundation Manual* and New York State Department of Transportation's *Drilled Shaft Inspector's Guidelines,* have similar requirements as Alabama but also allow the use of several different testing methods included in this chapter.

The primary reason that CSL is so widely accepted is because it is an accurate, cost-effective, and nondestructive means of investigating the integrity of concrete in drilled shaft foundations (Branagan & Associates, Inc, 2002). Furthermore, CSL determines the integrity and homogeneity of concrete in a deep foundation and identifies voids or soil intrusions within the structure.

The CSL test includes placing a signal generator in one access tube and a signal receiver in another access tube. The basic theory of the CSL test is that the arrival time of the compression wave signal from the generator to the receiver has a direct correlation to the density of the concrete. The ultrasonic compression-wave (or p-wave) arrival time from a signal source in one tube is measured to a receiver in another tube. The test is normally run from the bottom to the top with both the receiver and the generator at the same vertical elevation. Knowing the tube separation distance, the p-wave velocity is calculated for each depth. The results are plotted as velocity with respect to depth (2002).

P-wave velocities for sound concrete free of defects are typically around 3,700 m/sec (12,000 ft/sec). Decreases in sonic-velocity from the local velocity average, accompanied by decreases in signal energy, indicate a departure from uniform concrete quality. Soil intrusion, poor concrete mix quality, voids, or other non-cemented intrusive materials can cause a decrease in p-wave velocity (2002). Table 2-3 indicates the p-wave velocity in different media.

Table 2-3. P-Wave Velocity in Different Media

Material	Velocity (ft/sec)	Velocity (km/sec)
Sound Concrete	12,000	3.7
Water	4,800	1.5
Air	1,100	0.3

The equipment set up for the CSL test includes placing either steel or PVC access tubes around the perimeter of the reinforcement cage. The number of tubes to be installed will depend on the diameter of the shaft and requirements of the state drilled shaft construction specifications where the shaft is being installed. The general guide, however, is to install one access tube per foot of shaft diameter (or one tube per 0.25 to 0.30 meters of shaft diameter) (2002). Alabama requires that 1.5 inch to 2.0 inch (40 mm to 50 mm) inside diameter schedule 40 steel pipe be used in quantities specified in Table 2-4 below.

Table 2-4. Alabama's Minimum Number of CSL Tubes per Shaft (ADOT, 2001)

Shaft Diameter, D	Minimum # Of Tubes
D≤ 4.5 feet {1372 mm}	4
4.5 feet {1372 mm} < D ≤ 5.5 feet {1676 mm}	5
5.5 feet {1676 mm} < D ≤ 6.5 feet {1981 mm}	6
6.5 feet {1981 mm} < D ≤ 7.5 feet {2286 mm}	7
7.5 feet {2286 mm} < D ≤ 8.5 feet {2591 mm}	8
8.5 feet {2591 mm} < D ≤ 9.0 feet {2743 mm}	9
9.0 feet {2743 mm} < D ≤ 10.0 feet {3048 mm}	10
10.0 feet {3048 mm} < D ≤ 11.0 feet {3353 mm}	11
11.0 feet {3353 mm} < D ≤ 12.0 feet {3658 mm}	12

Whether steel or PVC access tubes are used, the tubes must have end caps and couplers that are watertight. The tube inside diameter must allow for the top-to-bottom free and unobstructed passage probes having dimensions of 1.41 inches diameter and 4 inches in length. Prior to CSL testing, tubes should have removable caps at the surface to prevent foreign material which could obstruct the tube (Branagan & Associates, Inc, 2002). The access tubes are filled with potable water prior to testing. The water provides a medium through which the p-wave signal can be transmitted and received inside the access tube.

Another consideration for assuring the accuracy and reliability of the test is de-bonding. De-bonding occurs when the concrete loses bond with the surface of the access tube. This loss of bonding means that an air space has developed between the surface of the pipe and the concrete. This space will produce false and attenuated signals. To help curb this problem, the tube surface shall be clean and free from contamination and the test should be completed with the specified time after concrete placement. Shrinkage of concrete increases with time and is a major cause of de-bonding. PVC tubes cost less than steel but tend to de-bond more rapidly from concrete than do steel tubes. Therefore, when PVC tubes are used they shall be roughened by abrasion prior to installation (2002). Table 2-5 indicates the approximate time window for acquiring optimal CSL data.

Table 2-5. Approximate Time Window for Acquiring Optimal CSL Data

Tube Composition	Tube ID (inches)	Time Window
Schedule 40 Black Steel	1.5 to 2.0	24 hours up to 45 days
Schedule 40 PVC	1.5 to 2.0	24 hours up to 10 days

The Installation of access tubes shall in general terms follow the following procedures (Branagan & Associates, Inc, 2002 and ALDOT, 2001):

(1) Consult with the project engineer and project specifications to verify: the type of tubes to be used; the quantity to be installed per shaft; required tube dimensions; and the method of tube installation. For example, Alabama's specifications require that "the tubes shall be installed in each shaft in a regular, symmetric pattern such that each tube is equally spaced from the others around the perimeter of the cage. The Contractor shall submit to the testing organization his selection of tube size, along with his proposed method to install the tubes, prior to construction."

(2) Watertight caps shall be placed on the bottom and the top of the tubes. In addition, any couples used to make full-length tubes shall be watertight. Butt welding of steel tube couplings and the use of tape to wrap pipes is not permitted. In addition, if PVC pipe is used, couplers shall be threaded or glued.

(3) Access tubes shall be attached to the interior of the reinforcing cage with wire ties at regular intervals along the length of the shaft, for example every three feet. Tubes shall be secured as to maintain vertical and parallel alignment during cage lifting, lowering and concrete placement. Tubes that are not vertical and parallel can adversely affect the outcome of a CSL test. In addition, care shall be taken during reinforcement installation operations in the drilled shaft hole not to damage the tubes.

(4) In order to include the toe of the shaft in the testing, the access tubes shall be installed such their bottom is close to the bottom of drilled shaft. Generally, tubes are placed within six inches of the toe of the shaft. Tubes are also extended two to three feet above what will be the top of the concrete shaft.

(5) The tubes must be filled with clean water as soon as possible after the reinforcement cage is set; in no case shall the tubes remain dry for more than one hour after concrete placement. The addition of the water to the tubes helps prevent the tubes from de-bonding. The tubes shall be capped or plugged immediately after being filled with water to prevent debris from entering into the tubes.

(6) Alabama's Specifications require, "The pipe caps or plugs shall not be removed until the concrete in the shaft has set. Care shall be exercised in the removal of caps or plugs from the pipes after installation so as not to apply excess torque, hammering, or other stresses which could break the bond between the tubes and the concrete."

(7) Normally, after the CSL test is completed, the access holes are evacuated and filled with an approved grout mix.

When determining how soon after concrete placement a CSL test can be completed, consideration must be given to how much the concrete has cured. In no case shall a test be considered accurate if completed within twenty-four hours after concrete placement. This time may need to be extended for larger diameter shafts or if mix designs that retard concrete setting are used (2002). In the interest of correcting any problems found in a timely manner and before the concrete is fully cured, the shaft should be tested as soon as possible after it has adequately cured.

The industry leader in manufacturing CSL equipment is Olson Engineering of Wheat Ridge, Colorado. The instrumentation provided for a CSL test from Olson Engineering typically consists of the following components (2002):

(1) A depth wheel/counter cabling system that is used to measure the vertical elevation of the CSL probes during the CSL Test.

(2) 35-kHz hydrophone source and receiver probes with a diameter of 1.41 inches and a length of 4 inches.

(3) A synchronized triggering system that starts recording data at the same time that the source is excited.

(4) A microprocessor-based computer system that has the capability to display individual records, perform analog to digital conversion, record data, data manipulation, and data output.

(5) As an option, a 12-volt DC battery power source may be incorporated into the system to allow for remote use of the system.

The procedure for completing a CSL test includes the following (2002):

(1) The top and bottom elevations for the shaft being tested, or the shaft length, are recorded. The concrete placement date is recorded along with any other pertinent data regarding unusual observations or events that occurred during construction of the concrete shaft.

(2) A sketch of the shaft under consideration is made and the access tubes are assigned a reference number. This reference number is recorded on the sketch along with a precise distance measurement between all of the tube pairs and the tube stick-ups above the concrete surface. Since the theory behind the CSL test is based on the p-wave arrival time, exact determination of the distance between tube pairs is necessary to accurately analyze a test.

(3) A tripod with the depth wheel is set up over top of the foundation to be tested and cabling is spooled from the computer, over the depth wheel, to each of the source and receiver probes. The cabling along with a cable from the depth wheel runs back to the microprocessor based data acquisition unit. These cables provide both excitation and signal return for the hydrophone, receiver, and depth encoder.

(4) Standard CSL tests are run with both the source and receiver probes in the same horizontal plane. Therefore, the test is started by lowering the probes to the full depth of the tubes being tested. After the slack is removed from the cables attached to the probes, the cables are simultaneously hand pulled over the depth wheel to steadily bring the probes to the surface. CSL acoustic travel time measurements are made at depth intervals of 0.2 feet or less from the bottom to the top.

(5) Well trained personnel are able to field analyze the measured data as well as the derived sonic velocities in terms of completeness and accuracy to determine the validity of the test. Final determination of whether a suspect

shaft contains an anomaly or not should only be made after additional data reduction and evaluation is completed. This additional work is not ordinarily completed in a field environment.

The data from a CSL test is typically presented to the client in the form of a written report which in addition to general descriptive information, includes the following specific information. Results are based on transit times and signal strength between each tube pair tested and should include:

(1) Interpretation of velocity profile logs with regard to the integrity of the concrete.

(2) Identification of the depth interval and tube pair that includes a sonic anomaly.

(3) Profiles of the initial acoustic pulse arrival time versus depth and pulse energy / amplitude versus depth (2002).

If it is determined, from CSL testing that a drilled shaft contains an anomaly, several methods may be used to try and isolate the location of the anomaly. Ultimately, however, the drilled shaft will likely need to be cored to determine whether or not the shaft is acceptable.

2.6.2 Crosshole Tomography

Several variations of the CSL test have been developed using the same instrumentation as the CSL. These tests are typically employed after CSL has determined the high probability of an anomaly in a given area and are applied to improve location accuracy and to further characterize the feature. The additional information is utilized to reduce the uncertainty in coring and remediating the defective area (2002). One of the most popular of these variations is Crosshole Tomography.

A Crosshole Tomography test is performed by leaving the receiver in a fixed position and raising the hydrophone while the hydrophone is producing sonic pulses. As in the CSL test, the arrival times from the hydrophone to the receiver are recorded. This procedure produces ray-paths that allows for three dimensional modeling of the suspect shaft. Figure 2-1 shows a velocity tomogram on a drilled shaft of a highway bridge. The anomalous zone is the slow-velocity area which lies in between 31 and 33 feet below the top of the concrete near tube one. Note that the center of the shaft is sound (Olson, 2003).

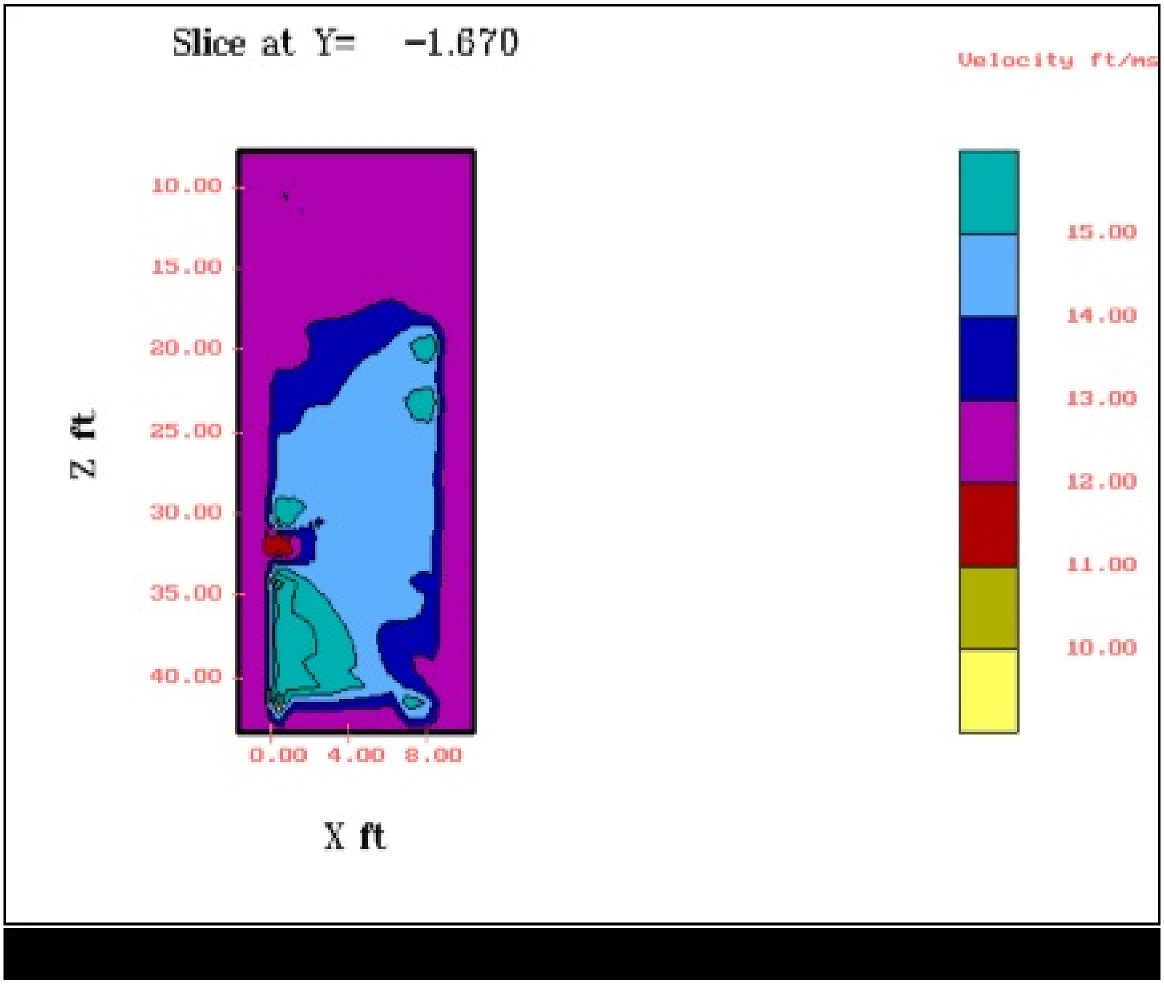

Figure 2-1. A Velocity Tomogram of a Drilled Shaft on a Highway Bridge showing an anomalous zone between 31 and 33 feet

2.6.3 Sonic Echo Test

Sonic Echo Tests (SET), or Sonic Integrity Tests (SIT), are probably the most un-intrusive and economical of any of the integrity tests and do not require any access holes in the drilled shaft. The SET is based on stress wave theory and is part of a family of tests referred to as surface reflection methods.

In surface reflection tests, a stress wave is introduced into the structure by a hammer impact. The hammer generates stress waves which causes physical distortion to the media in which the impact occurred. Four types of waves of concern are generated in the medium due to the impact (Finno & Prommer, 1994):

(1) Compression waves which are also referred to as primary, “bar”, or longitudinal waves,

(2) Shear waves which are also referred to as secondary or transverse waves,

(3) Surface waves or Rayleigh waves, and

(4) Stonely waves or tube wave.

The basis behind integrity tests that are developed from stress wave theory is: the time that it takes for a compressive stress wave to be generated at the surface, reflected off of the toe, and return to the surface is based on the velocity of the wave and the length of the shaft. Any variation from the expected arrival time may be indicative of a problem.

The compression wave is the primary concern for SET and is the fastest wave traveling 13,100 ft/s (4,000 m/s) in solid high quality concrete. The compression wave causes the material being tested to alternate compressive and tensile stresses by the wave (Blitz, 1971). At the point of the hammer impact, a compression zone is produced which results in the formation of a compressive stress wave. The compression wave travels down the shaft at a velocity (v_c) with a force (F). Shaft impedance (Z) is a ratio of v_c and F. It is also a function of the elastic modules (E), cross-sectional area (A), and compression wave velocity (v_c) of the shaft (Finno & Prommer, 1994).

A change in the cross-sectional area of the drilled shaft and/or a change in the density of the medium (concrete) will cause a change in impedance. Part of the stress wave reflects back up the shaft whenever the wave encounters a change in impedance. The remainder of the stress wave continues down the length of the shaft. A compression or negative wave is reflected back up the shaft whenever an increase in cross-section or density occurs. A tensile or positive wave is reflected back up the shaft with a reduction in cross-sectional area or concrete density. In the absence of changes in impendence, the stress wave travels until it is reflected off of the pile toe (1994).

Figure 2-2 shows a typical response curve obtained from an SET (SIT). The initial impact at the top of the pile is visible and the distance to the toe is observed by noting the return reflection of this wave. Any early reflections would indicate some type of increase in the impedance of the material to stress wave propagation. This can be a break in the shaft, a change in soil stiffness, or a significant change in cross section of the shaft. Additionally, the polarity of the reflected wave with respect to the original impact can often be an indicator of the type of anomaly encountered. In general, when the reflected wave is of the same polarity (algebraic sign) as the impact wave, this indicates an increase in cross section or a decrease in impedance. If the polarity of the reflection is opposite that of the original impact, a higher impedance is being encountered, such as would be caused by a necked region of the shaft.

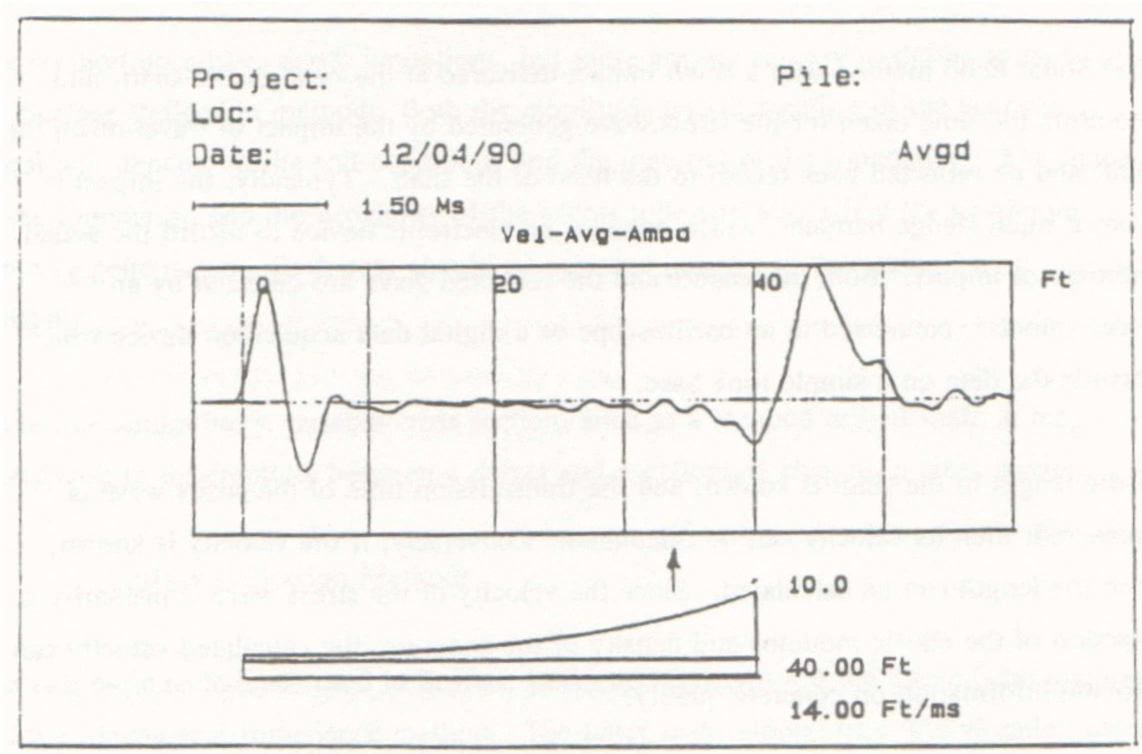

Figure 2-2. Typical Response Curve for SIT *(after Baker, 1993)*

Part of the impact energy is radiated into the soil (Paquet, 1968). Attenuation of the stress waves are continually occurring as they travel down the shaft as a function of the concrete quality, soil conditions surrounding the shaft, and the shaft cross-sectional area. Stress wave attenuation will be greater for shafts placed in stiff soil than for those placed in loose soil. In addition, wave attenuation will be less for shafts with larger cross-sectional areas than for those with smaller cross-sectional areas. "In general, scattering of the signal occurs due to changes in the material the wave is propagating through, especially at boundaries where there are abrupt changes in the impedance between two materials" (Finno & Prommer, 1994).

The Sonic Echo Test was first used in Holland in the 1970's as a means to provide quality for control driven precast concrete piles. As stated above, if the shaft contains irregularities in the concrete, such as cracks, a change in cross sectional area, or poor concrete quality, reflections will occur which indicate the presence of an irregularity. Therein, discontinuities can be severe enough to prevent the stress wave from reaching the toe altogether (1994).

Test equipment for the SET includes a hammer with a triggering device combined with a vertical geophone attached to a laptop or portable computer. Geophones are low frequency transducers that can measure frequencies below 2 kHz. As an alternate, accelerometers may be used in lieu of the geophone. However, since the accelerometer measures acceleration, additional signal processing must be performed to obtain velocity. The laptop typically contains a data acquisition card in combination with a signal conditioning unit.

In order to perform a SET, two areas on the top of the concrete shaft being tested are cleaned and ground to a smooth surface. One area is cleaned close to the center of the shaft. This location is the impact point for the hammer. The second location shall be near the perimeter, but within the rebar cage. The geophone is fixed to the concrete surface with a coupling agent. Next, the shaft is struck with the hammer at the area close to the center. Once the hammer impacts the top of the shaft, the portable computer is triggered and records the response of the top of the shaft via the geophone or the accelerometer. Background noise can be eliminated by performing the test several times and comparing and averaging the results (1994).

The results of the SET indicating a sound shaft show compression wave reflection planes at the concrete-soil interface at the toe and the concrete-air interface at the top. Defects in the shaft will also cause the waves to reflect. The waves will continue to reflect back and forth between the reflection points until full attenuation has occurred. The depth of the reflector is determined by the following equation:

$$Z = \frac{V_b \, \Delta t}{2}$$

where, z is the depth to the reflector (either a defect or the toe of the shaft), V_b is the longitudinal wave velocity in concrete, and Δt is the travel time of the reflected wave. Since the velocity of the compression wave varies with concrete quality, it is preferable to determine V_b based on concrete samples that are representative of the in-situ concrete in the drilled shaft. As a less accurate method it is reasonable to use the values indicated in Table 2-6, though these values differ from source to source. Therefore, the values indicated are based on ultrasonic pulse velocity measurements represented by an infinite concrete medium as reported by Hearneet al., al. (1981). These values are reduced 5%, assuming a Poisson's ratio of 0.2 for concrete since Hearne's experiments were based on an infinite concrete medium.

Table 2-6. Compression Wave Velocity in Concrete

Compression wave velocity, feet per second (meters per second)	General concrete quality
Above 14,200 (4,300)	Excellent
11,400 – 14,200 (3,500 – 4,300)	Good
9,500 – 11,400 (2,900 – 3,500)	Questionable
6,700 – 9,500 (2,000 – 2,900)	Poor
Below 6,700 (2,000)	Very Poor

According to a report published by the FHWA (Baker et al., 1993), concrete with a compressive strength of approximately 4,300 to 5,100 psi (30 to 35 MPa) has a V_c between 12,500 and 13,100 ft/s (Finno & Prommer, 1994).

In order to interpret a SET, both the concrete quality and the shaft length must be known or assumed. Given these parameters, it is easy to determine if a reflection in the signal is either the toe or an anomaly present in the shaft (1994).

Exponential amplification is used to progressively increase the amplitude of the reflected signal in a similar manner to its attenuation. This amplification process is required because the impact at the top of the shaft produces small strains relative to those required to mobilize the shaft capacity. However, it is important to verify that the reflection is being amplified and not just background noise (1994).

There is a limiting length / diameter (L/D) ratio beyond which all wave energy is dissipated and no toe response can be detected (Baker et al., 1993) The limiting L/D ratio varies depending on the damping and signal loss into adjacent soils. In soft soil deposits, such as silts, good results can be obtained for L/D ratios of 50:1 (Davis and Robertson, 1976). For stiff clays, this ratio is reduced to 30:1 (Hearne et al., 1981). In cases where the L/D ratio is exceeded, the only useful information that can be obtained from a SET test is the presence or absence of anomalies in the upper portion of the shaft (Finno & Prommer, 1994).

As with the CSL test, several variations of the SET have been developed in which a receiver is embedded along the shaft or at the shaft toe. The additional points allow direct measurement of the compression wave arrival time and permit more accurate velocity calculations. This method also allows for shafts that exceed the L/D ratio to be tested with a higher level of confidence. However, this method drastically increases the cost of the SET since care must be given to the installation of the embedded receivers and the receivers are not able to be retrieved and used again; they become a permanent part of the shaft (1994).

The SET is only useful for determining the linear continuity of a shaft. Its limitations include but are not limited to (1994):

(1) SET provides no quantifiable information about the shafts cross-sectional area or behavior of the shaft under load.

(2) Unless the test utilizes embedded receivers, only the uppermost defect can be readily detected. Any defects occurring below the initial defect may not receive adequate signal to return a reflection that will not be masked out by noise.

(3) The hammer impact generates Rayleigh waves which propagate along the shaft surface and cause a noisy environment. This problem is especially troublesome in the top 10 feet of a shaft.

(4) Defects located near the toe of the shaft can be difficult to identify since reflections from anomalies close to the toe may easily be misinterpreted as the toe and not an anomaly.

(5) Since a reduction in concrete shaft cross-sectional area, or necking, and poor concrete quality both produce reductions in impedance, it is not possible to distinguish them. In addition, a gradual decrease in cross-sectional area may not generate a reflection at all.

(6) A layer of stiff soil may cause reflections, increasing the uncertainty of the shaft integrity.

(7) An increase in cross-sectional area will actually increase the shaft capacity and is generally not viewed as a problem. However, the reflection from the bulb will be similar to that of a defective shaft.

(8) Utilizing higher frequency waves would improve the accuracy of the test. However the wavelength used for the SET cannot decrease much less that the diameter of the shaft. If shorter wavelengths are used, the shaft will behave as an elastic medium where compression waves will occur from all shaft boundaries and not behave like a rod-type structure.

(9) In stiff soils, wave attenuation can be a problem. The more similarity between the toe bearing material and the concrete, the lower the amplitude toe reflection. Therefore, shafts with end bearing in rock have very poor toe reflections.

It has been summarized in Finno & Prommer (1994), that “the method (SET) is best suited for checking precast and permanently cased piles due to the straight-sided shafts these structures provide. It is not as suitable for drilled shafts due to variations in cross-section that often exist causing multiple reflections.”

Several case studies of SET discussed in Finno & Prommer (1994) are summarized below as supporting documentation to the information contained herein:

(1) SET tests were performed on 1.6 feet (0.5 m) diameter auger cast friction piles with lengths ranging from 20 to 59 feet (6 to 18m) and founded in stiff and very stiff clay layers. As expected, the piles with a L/D ratio of 32:1 and 36:1 did not produce a toe reflection. Since the length of the piles were known, allowing for the determination of the compression wave velocity, it was also possible to determine that one pile had poor or questionable concrete quality. Specifically, the compression wave velocity in this pile was V_c = 11,200 ft/s (3,400 m/s). The remaining piles had V_c ranging from 11,500 to 13,100 ft/s.

(2) As part of a FHWA test program for evaluating drilled shafts for bridge foundations, a test section was constructed at Texas A & M University with nine shafts. These shafts varied in length from 34 to 79 feet with 3 feet of stick-up above the ground level. All of the shafts were constructed with a 36 inch diameter (L/D ratio from 11:1 to 26:1). The integrity tests performed on the piles included the Impulse Response, Sonic Logging, and Sonic Echo. In this study, several shafts were constructed with

both planned and unplanned defects. In conclusion of this test, SET was able to determine some areas where defects occurred. However, the results also showed the limits of the SET and in one case, "…one could have been deceived into thinking that the reflection at 31 feet (9.4m) was the toe, if it was unknown that the shaft was 79 feet (24.1 m) long."

Clearly the SET method of integrity testing can return some questionable results that may lead an engineer to falsely accept or deny the integrity of a test.

2.6.4 Impulse Response Test

Another method that is based on the measurement of compression wave reflections is the Impulse Response Test (IRT). This test is an extension of the vibration test, which was found to provide more information than the SET, particularly with the irregular profiles of drilled shafts. In the IRT the head of the shaft is impacted with a hammer that induces transient vibrations with frequencies as high as 2 kHz. The response of the shaft to these vibrations is measured in the time domain, and the signal is digitally converted to the frequency domain for analysis (1994).

The equipment set up for IRT is similar to SET except the impact hammer has a load cell that measures the impact force with time. A vertical geophone is triggered upon hammer impact and records the vibrations at the shaft head. Both the hammer and the geophone are connected to a portable PC which is used for acquiring, analyzing and storing the data.

The testing procedure for IRT is identical to the SET. However, it is critical that the hammer strike the shaft head squarely to ensure proper force measurement.

Unlike SET, IRT provides a determination of the homogeneity of concrete in the shaft and measure of the shafts performance (Higgs and Robertson, 1979). IRT provides a stiffness value of the shaft which has a direct correlation to shaft performance. In addition, the shafts length may be determined from the IRT (Finno & Prommer, 1994).

Analysis of the results obtained from an IRT includes performing Fast Fourier Transform (FFT) on the force and velocity signals to convert them from the time to frequency domain. Next, a plot of mobility versus frequency is obtained by dividing the velocity spectrum by the force spectrum. The length of the shaft is calculated by measuring the frequency change between resonant peaks (1994).

IRT data when presented on a mobility plot can be subdivided into two distinct regions. First, at low frequencies there is a lack of significant inertial effects and the response of the system is linear, like a spring. However, at higher frequencies the system goes into resonance. The frequencies associated with this resonance depend on the length of the shaft, and their relative amplitude depends on the damping characteristics of the soil (Baker et al., 1993). If the results of an IRT test indicate a smaller shaft length than expected, this is interpreted as an anomaly. Then, information from the linear portion of the mobility curve (Figure 2-3) is used to obtain the dynamic stiffness of the shaft. This aids in the determination of whether the measured anomaly is a neck or a bulge. The following equation illustrates this calculation:

$$E' = \frac{2\pi f_m}{(V_o/F_o)_m}$$

where, E' is the Dynamic Stiffness, f_m is the frequency, and the ratio $(V_o/F_o)_m$ is known as the Mobility.

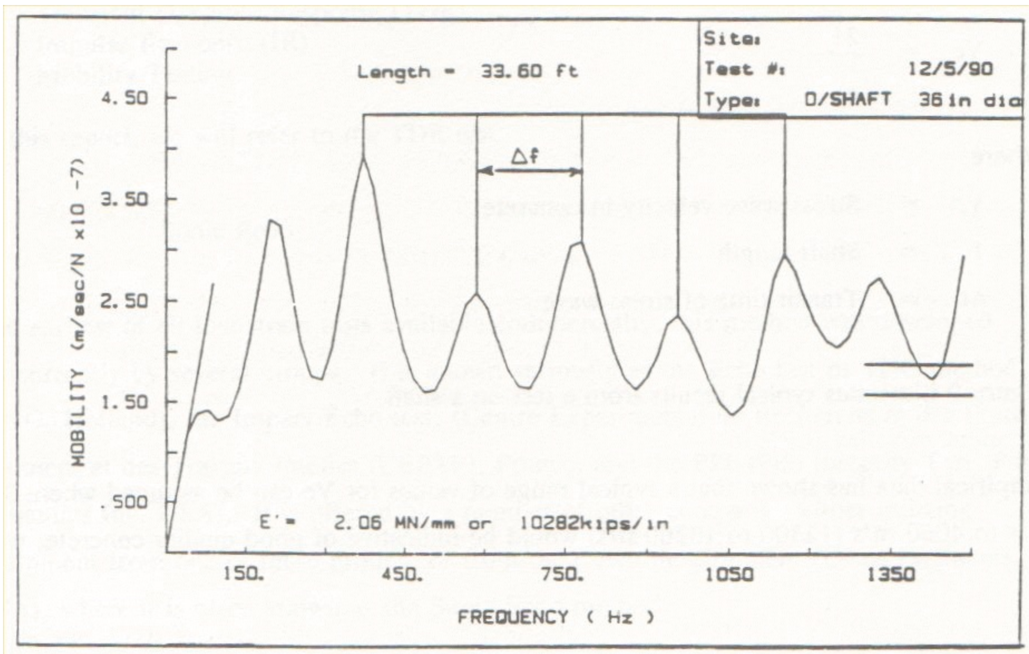

Figure 2-3. Typical Mobility Plot for IRT *(after Baker, 1993)*

The low-strain dynamic stiffness (K') can be calculated and correlated to the static stiffness. At low frequencies, the lack of internal effects causes the shaft/soil system to behave as a spring. This behavior appears as a linear increase in amplitude of mobility from zero to the onset of resonance (Baker et al., 1993). The commonly accepted dynamic stiffness value (K) provides a good indication of the low-strain, soil-foundation interaction. For a rigid base a high stiffness value will be calculated; for a compressible base a low stiffness value will be calculated. By comparing stiffness values for similar sized shafts, it can be determined which ones should be considered questionable. Shafts founded in loose soil and shafts that have soil inclusions, necks, and breaks will have lower stiffness values than sound shafts founded in solid soils (Finno & Prommer, 1994).

An evaluation of the shaft performance is made by calculating the K_{min} and K_{max} or the theoretical limiting values for dynamic stiffness and comparing them with the actual stiffness value. In addition, the IRT provides a calculation of the theoretical mass of the shaft which can be cross-checked against the amount of concrete used in the shaft (1994).

It is easier to determine if a bulb or a neck has occurred with IRT than with SET. Stiffness values are lower than normal for necked shafts and higher than normal for shafts with bulbs. In all, this method proves to be better for testing the integrity of drilled shafts than SET with some of the same limitations, such as:

(1) This is a surface reflection method that relies on measuring reflected responses.

(2) There is a limiting L/D ratio based on the soil conditions. As this L/D ratio is approached, the curve will flatten out to where resonant peaks are not discernable.

(3) This method is subject to the problems associated with surface waves.

(4) There is a limit on the size of defects that can be detected.

(5) Only the top defect in a shaft can be detected, if multiple defects exist.

(6) In order to interpret the results from a test, either the concrete compression wave velocity or the shaft length must be known.

2.6.5 Gamma-Gamma Testing

Gamma-Gamma Testing (GGT) is an integrity testing method that is widely used and accepted by Caltrans. In GGT testing, a source of ionizing radiation is lowered down an access tube similar to CSL. The probe that emits the radiation also contains a gamma-ray detector. The basic theory behind GGT is that the number of gamma-ray photons per unit of time that are reflected from the nuclei of the modules of the material surrounding the tube and return at a given energy level to the detector is related to the density of the material surrounding the tube. In short, GGT can detect significant reductions in localized density of the shaft concrete which would be indicative of a void or imperfection in the shaft (O'Neill & Reese, 1999). GGT has an advantage over CSL in that it can detect reductions in concrete density outside of the reinforcement cage.

The access tubes for GGT must be made of a material such as PCV that will allow the photons to pass through and reflect back through the wall of the tube. In general GGT will not detect changes in density of the concrete outside of about a 4 inch (100 mm) radius from the access tube. In order to accurately determine the density of the entire shaft, access tubes would need to be placed at 8 inches (200 mm) on center. Since this is not possible, an engineer must be content with having intermittent sampling of the concrete density around the perimeter of the cage (1999). Caltrans recommends that one access tube be used per foot of pile diameter (Lew et al., 2002). This leaves a tube spacing of about 2.75 feet around the circumference of the cage.

It is further recommended that the access tubes be placed at least 3 inches (76 mm) away from any vertical reinforcement. Since the vertical steel reinforcement is about 3 times more dense than concrete, having the reinforcement too close to the access tubes or varying the distance between the tubes and the reinforcement will cause false readings in the bulk density readings. Furthermore, if the access tubes around the perimeter of the cage are not placed so that they have the same influence from the reinforcement, different tubes within the shaft cannot be compared (Speer, 1997).

Due to the length of the gamma-gamma probe, vertical alignment of the access tubes must be maintained so that a 2 foot (0.6 m) long by 1.9 inch (48 mm) diameter rigid cylinder can pass from the top to bottom of the tube (1997). Since radiation sources are subject to the Nuclear Regulatory Commission (NRC), regulations that require special training and licensing is required for the handling and transporting of the device. Abandoning a lodged device and grouting it in place is not an option. During one well documented GGT, the ion transmitting probe was wedged in the inspection tube. It took two weeks to remove the probe using special drilling and finishing tools (Lew et al., 2002). Caltrans specifications require access tubes that do not allow the probe to pass be drilled out to within tolerances for the probe to pass (Speer, 1997). In addition, the process can be time consuming compared to other testing methods. A typical 7 foot (2.1 m) diameter shaft with seven inspections tubes 60 feet (18 m) long will take eight hours to test.

The standard GGT has a 1.87 inch (47 mm) diameter gamma-gamma probe with a 10 millicurie Cs 137 source. The probe is lowered down the access tube with a cable. The probes are configured for the determination of density by backscatter. This method allows for the use of a less powerful radioactive source. Radiation is emitted from the source at the bottom of the probe. The radiation is simultaneously absorbed and scattered by the concrete and reinforcement surrounding the inspection tube. The receiver at the top of the tube counts the reflected gamma rays over a time interval. More gamma rays are counted in less dense material than in dense material (1997).

GGT data is processed by first plotting the bulk densities versus time. Next, this data is compared to all of the data for shafts in the same vicinity, using the same probe. The data sets are reviewed and data that is insignificant and redundant is discarded. The mean, mean minus two standard deviations, and the mean minus three standard deviations are plotted on the same graph. Figure 2-4 show the results of GGT that clearly shows the presence of a necking defect.

Interpretation of the test results rely equally on experience, engineering judgment, and statistical analysis (1997). In any shaft, there is normally some variation in the density of normal concrete from different points in the shaft. In general, anomalies can be interpreted if the bulk density of the concrete appears to drop below the mean minus three standard deviations line (O'Neill & Reese, 1999). As with most, if not all of the integrity testing methods, it is difficult, if not impossible, to determine the exact nature and full extent of potential anomalies without extracting the shaft.

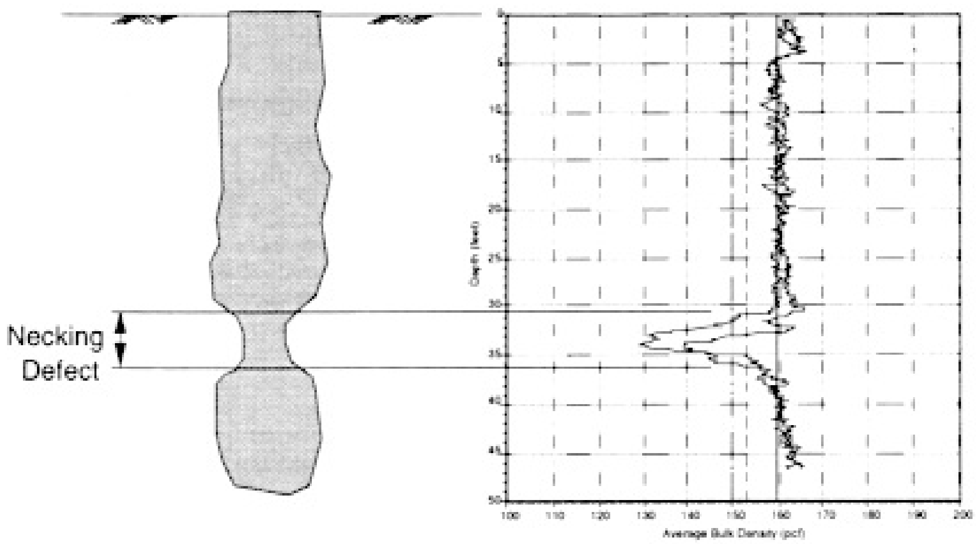

Figure 2-4. Results of a Gamma Gamma Test Indicates Necked Shaft

2.6.6 Concreteoscopy

Concreteoscopy is a relatively new testing method that allows the integrity of the shaft to be checked in real time. In this testing method, ½ inch diameter (12.7 mm) clear plastic tubes are attached to the rebar cage. As the concrete is placed, a miniature television camera on a fiber-optic cable is used to view the concrete from within the tubes. This test is similar to the GGT except that the results provide visual conformation as to the soundness of the concrete instead of relying on density monitoring. As with GGT, Concreteoscopy, is able to view only the material directly adjacent to the access tubes (1999).

2.6.7 Parallel Seismic Integrity Testing

Parallel Seismic Integrity Testing (PSIT) was developed in France in the 1970's to evaluate the conditions of piles and drilled shafts under existing structures (Davis and Hertlein, 1993). The general procedure for the (PSIT) is that after the shaft construction is complete, a borehole is drilled adjacent to and slightly deeper than the shaft. Stress wave energy is generated on the surface of the structure by impacting it with a hammer and the arrival time of the compression wave is monitored in the bore hole by means of a hydrophone. The tests are typically completed in increments of approximately 20 inches (50 cm) (Stain, 1987). A profile of signals is built up for the entire length of the shaft. Under good conditions, transmission distances up to 130 feet (40 m) are possible (Davis and Hertlein, 1993).

As in the CSL test, the access tubes are filled with water. As discussed in earlier testing methods, the stress waves generated on the surface due to the impact have a velocity that is directly proportional to the density of the medium in which they are propagating. Therefore, the arrival times of the stress waves are proportional to the depth of the hydrophone. A difference in the rate of the arrival time velocity with respect to depth indicates a change in the medium or the base of the shaft (Finno & Prommer, 1994).

The test equipment required for a PSIT includes an impulse hammer, a hydrophone receiver, and a data acquisition system that allows high speed sampling of the hydrophone. In a typical arrangement, the data acquisition system is triggered when the impulse hammer strikes the pile or the shaft. The hydrophone then records the arrival of the generated stress waves. The hydrophone receiver must be capable of withstanding the hydrostatic pressure generated within the access tubes or boeholes, which can be in excess of 43 psi. The data is collected by the acquisition system, which typically consist of a data acquisition card in a portable computer, and may be analyzed on site.

The borehole should be made in the ground adjacent to the foundation and slightly deeper. To assure the correctness of the test, the boreholes must be parallel to and within 3 feet of the shaft wall. If the distance between the shaft and the borehole is not relatively constant, variations in arrival time will occur leading to false readings. In addition, if the borehole is greater than three feet from the shaft wall, the signal will be affected due to attenuation and the non-homogeneity of the different soil layers. Since compression waves are able to travel through fluids, water is added to the borehole to act as a coupling medium for the hydrophone receiver (1994).

This method of testing can also be accomplished by coring a hole in the suspect structure, filling with water, and performing the test as outlined above. This test method is referred to as the downhole seismic method. The arrival of the compression wave is critically refracted along the core hole wall at the propagation wave velocity of concrete (Davis and Hertlein, 1993).

The test results and integrity are dependent on the ability to measure the direct arrival time of stress waves. An anomaly in a drilled shaft will appear as an increase in the arrival time of the wave at the depth in which the anomaly occurs. A test result for a continuous shaft without defects should contain a linear increase in arrival time with depth. The toe of the shaft will also cause an increase in the wave arrival time, giving a clear indication of the location of the shaft toe.

The downhole seismic test records a second wave in addition to the compression wave. This second wave is known as the tube, hydo, or Stonely wave. The tube wave velocity is a function of tube diameter, roughness, and the permeability of the material surrounding the hole. The velocity of the tube wave is approximately 5,000 ft/s (1,500 m/s) in cored concrete and, its amplitude is much greater than that of the concrete compression wave (1993).

Tube waves are reflected from the ends of the core hole. They are refracted at locations where the bulk elastic modulus of concrete and the diameter of the core hole change. The shape and the spectral content of the tube waves have a higher frequency and a shorter duration than the first direct waves. Vertical receivers are generally used for recording tube waves (Galperin, 1985). Little has been studied about obtaining information from the tube wave, which theoretically should be able to yield information on the concrete quality (Finno & Prommer, 1994).

All that can be determined from reviewing the results from either a parallel seismic or downhole test is that arrival time of the compression wave or tube wave has increased as a result of lower wave propagation velocity. These changes can be caused by several factors including changes in concrete quality, cracks, soil inclusions, and the toe of the shaft. Therefore, it is difficult to determine the type of defect, if any exists, that has caused the change in the slope of the arrival time line. This may lead to inconclusive results. Another major disadvantage of the tests is the cost of coring and installing the access hole (1994).

In review of a case study as reported by Finno & Prommer (1994), two test piles were constructed at a research site in France by the CEBTP (Centre Expérimental d'Études du Bâtiment et des Traveux Publics). They were both 46 feet (14 m) long and 26 inches (65 cm) in diameter. One was continuous and the other had a break at 24.6 feet (7.5 m). Access tubes were installed adjacent to the shafts to a depth of 66 feet (20 m) to facilitate parallel seismic testing. The sound shaft had a signal transmission time that increased linearly with depth down to the toe, proving continuity over the full length. The broken pile was linear down to 24.6 feet (7.5 m), but the arrival time increased significantly below this point. For this shaft, it was known that the pile extended to 46 feet and had a break at 24.6 feet, but the test was unable to differentiate weather the break was in fact a defect or whether it was the pile toe.

2.6.8 Thermal Integrity Testing

Recently, a new method for evaluating the integrity of drilled shafts has been developed that uses the natural temperature rise of curing concrete. This concept involves precisely measuring the temperature within the shaft due to the heat of hydration generated within the first 48 hours. Subsequent signal matching with a computer model is used to discern the location and size of inclusions that produce no heat. Aside from the merits of thorough integrity evaluation, this method provides integrity feedback while the concrete is still "green" allowing easy coring, flushing, and grouting of affected areas.

In the late 1980's, University of South Florida researchers idealized a method of evaluating the integrity of drilled shafts on the basis of the measured soil temperature around the curing concrete foundation elements. At that time, it was conceived that the then-evolving quasi-static cone penetrometer could be equipped with thermocouples capable of registering subtle variations in soil temperature caused by the generation of heat from cement hydration reactions. These variations in temperature would be indicative of a compromised shaft structure. In concept, a homogeneous concrete cylinder would produce a uniform temperature profile with depth (with some variations attributed to soil stratigraphy). However, as many drilled shafts were cast in soils that were not amenable to cone penetration testing (e.g. rock or gravelly soils), the applications for this approach were severely limited.

The general use of access tubes in drilled shaft reinforcement cages refocused the thermal integrity concept to consider capturing and modeling temperature data from the pile interior. Recent developments in the remote monitoring of industrial processes, using windowed or focused infrared systems, has allowed the workers at USF to design a preliminary system for recording continuous thermal traces on the interior walls of access tubes. The infrared transducers used are robust and can register surface temperature using reflected wave technology.

The first probe capable of making these measurements was outfitted with a single infrared thermocouple that was inserted into a logging tube four times to ascertain the temperature variation in both the radial and circumferential directions. Subsequently, this device was outfitted with four orthogonally-oriented infrared sensors that would simultaneously register the wall temperature in four directions. Due to space considerations, each sensor was separated vertically by 3 inches. This improvement (of four sensors) led to more intelligible data, as the introduction of the probe into the access tube disrupts the wall temperature (by inducing cooling), and thus reducing the temperature for the subsequent three soundings.

Although the temperature-voltage response is not linear, a roughly linear response can be found in a useful range of temperatures, if the thermocouple junctions are selected properly. As these recently developed transducers are sensitive (with high repeatability and very small time constant) signal conditioning and careful calibration of each transducer is important.

The transducer is based on thermocouple technology (junctions of dissimilar metal conductors) using radiative thermal energy focused on a thermocouple. Each infrared transducer generates a voltage signal output, on the order of 5-10mV over the range of temperatures in outdoor conditions and hydrating concrete. The slope of the response curve in the useful range of temperatures is roughly 0.3 mV per degree Celsius. Maintaining the integrity of the signal from the sonde (probe) to the data recording device (e.g., laptop computer adjacent to the pile test) through lengthy copper cable requires high gain amplification of the differential signal, and conversion to a current signal. Therefore a high input impedance amplifier stage with gain and a voltage-to-current amplifier stage (4-20mA "current loop" AD697 chip) is included in the sonde. Also included in the updated design is a digitizing vertical displacement counter/encoder, similar to that used in computer "mice", for registering the depth of the sonde within the shaft.

The data output of the sonde consists of the depth within the access tube of the encoder, and four current signals from each of the infrared sensors. The four current signals are converted to voltage signals directly by simply reading a voltage drop across a resistor placed in each sensor circuit near the recording equipment. These voltages, as well as the encoder signal, are read directly into a laptop through a National Instruments DAQ700, 12-bit multi-channel data acquisition card. As the depth encoder/counter represents one value of depth within the shaft, a depth "shift" is added for each sensor, which represents the vertical separation of each temperature sensor and the observed depth. Also, the voltage readings are applied to an *a priori* linear calibration model of each sensor. After the voltage-temperature conversions and depth corrections are made, individual temperature readings from each channel will consist of an ordered pair of depth and temperature. Figure 2-5 shows the general setup of the testing equipment.

Temperature anomalies that can be expected in a drilled shaft may derive from various phenomena, and are listed in Table 2-7. In general, three overall categories exist based upon the anomaly type (heat source changes , boundary condition changes, and the scalar product grad T • grad diffusivity). All three categories of thermal signal anomalies are observed in the Auburn field data (see Chapter 6).

Table 2-7. List of thermal anomaly types and descriptions.

Phenomena	Anomaly Type	Anomaly Source
GWT	boundary condition	higher specific heat of water
aggregate	heat source (none)	lack of heat production
rebar	diffusivity gradient	very high conductivity
shaft toe	boundary condition	vertical flow condition
lift	heat source	mix age or mix proportions
soil slump	heat source boundary condition	lack of heat production change of heat flow geometry

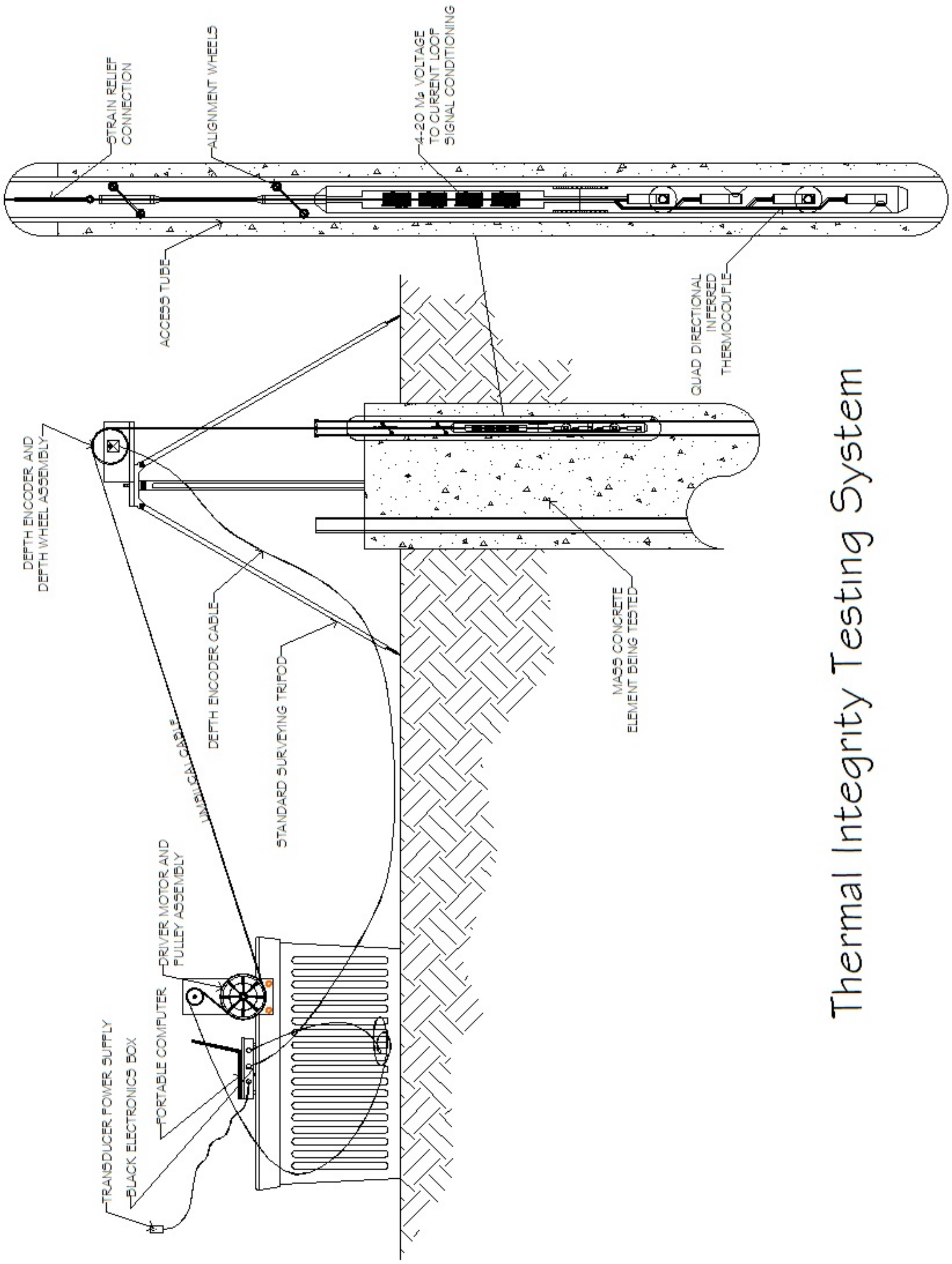

Figure 2-5. Thermal Integrity Test Equipment Setup

2.7 Prior Research into Drilled Shaft Anomalies

As it happens, there is not an abundance of available information on drilled shaft foundations that have had severe anomalies or other unacceptable quality. This is perhaps due to the reluctance of drilled shaft contractors and engineers to publish information which highlights how their particular project failed. However, when looking at the issue with a broadened perspective, some information is available. Several items of interest that were studied in this report have been addressed in some way in the past. These include borehole stabilization, behavior of fresh concrete, and lateral pressure distribution of fresh, cast-in-place concrete.

2.7.1 Borehole Stability and Stabilization

When drilled shafts are to be installed in cohesive soils, a major factor affecting the stability of the borehole is the moisture content of the soil and the location of the ground water table. Past research has proposed that when a borehole is drilled into this type of material, excess pore water suction forces will dissipate very slowly (Carter, 1986). This will allow the void ratio of the soil around the hole to increase, and as a result swelling will occur. If this swelling is allowed to proceed unabated, collapse of the borehole is probable.

The problem of maintaining a stable borehole is not limited to cohesive soils, however, as can be seen in case histories published, mainly, in the oil drilling industry (Maury, 1987). Unexpected conditions encountered while drilling in rock and sand often result in partial or total loss of a borehole, and the related financial loss. Research into various drill slurries aimed at improving the stability of boreholes in questionable material has been ongoing for years (Gray, 1974). Concerted efforts at instrumenting boreholes so as to better understand the three dimensional displacement of strata have been undertaken as well (Smart, 1978). Data collected from such research has led to the development of many calibrated numerical models that allow at least a qualitative prediction of a site's response to drilling (Bandis, 1986).

2.7.2 Behavior of Fresh Concrete and Lateral Pressure Distribution

In order to remain within the scope of this report, past research into the behavior of freshly placed concrete in a drilled shaft borehole was explored. In particular, tremie placement and free fall techniques were studied both above and below the water table. It has been argued the method of drilling plays an important role in the strength and quality of the finished drilled shaft (Chadeisson, 1971). This is due mainly to issues already discussed above. However, the rate of placement of concrete in a drilled shaft is a parameter which some say merits more investigation (Bernal, 1983). In this report, rate of concrete placement as well as rate of casing extraction is addressed. Also, Bernal suggested that the extraction of the tremie from the newly placed shaft results in an unpredictable increase in the lateral pressure exerted by the concrete (1983). Data presented later in this report will suggest that the opposite it true, at least for laboratory scale tests.

Rodin's experimental work (1952) on the lateral pressure of concrete on formwork shows that the lateral stress distribution in a freshly poured concrete column is hydrostatic up to some critical depth. The actual value of this depth depends in large part on the geometry of the shaft; in particular the length to diameter ratio. Beyond this critical value, there is a reduction in lateral pressure due mostly to arching of the concrete. This phenomenon was not reproducible in the lab scale testing (discussed later, Chapter 3).

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3. LATERAL PRESSURE CELL TESTING

3.1 Concept and Development

Early research into the causes of voids, necks, and bulges in finished shafts prompted researchers at USF to explore the kinematics of flowing concrete inside a borehole containing reinforcing steel. It was desired to at least qualitatively study the rheology of concrete flow as it emerged from the bottom of the tremie and rose inside the borehole. The *Lateral Pressure Cell* (LPC) was developed to meet this research need.

Initially the LPC concept was to measure the lateral stress distribution as a function of depth in a borehole and to compare this to design and construction parameters such as aggregate size and gradation, cement type, water/cementitious materials ratio, reinforcement size and placement, and borehole stabilization method. To accomplish this, a lab-scale cell was constructed out of Lucite (Figure 3-1) and instrumented using resistive strain gages (Figure 3-2) aligned to measure hoop strains in the cell as it was filled with fluid (water, slurry, or concrete).

The LPC was additionally outfitted with 6-psi pressure transducers flush mounted to the inside of the Lucite walls (Figure 3-3). A JP-2000 load cell was mounted to the base of the device (Figure 3-4). This allowed calibrations to be performed as needed by simply filling the device with a known volume water and measuring the weight. The pressure transducers proved to be more stable and precise than the strain gages. Accordingly, the data analysis was far simpler.

A recommended modification to the LPC, should a future generation be constructed, would be to place additional pressure transducers at each level on opposing sides of the cell. This could be done at half-points or quarter-points on the circumference. These added sensors would allow more complete lateral pressure data measurement, and could be used to quantify stress localizations and mortar arching. This additional data may even be used to record (roughly) the shape of the mortar column as it flows into the cell, although to do this would require much more testing and calibration.

3.2 LPC Test Program Overview

The first testing performed in the LPC was designed to study the effect of concrete slump on lateral pressure of the finished shaft. A series of mortar mixes were designed, each having the same cement content but different w/c ratios. Early selection of the w/c ratio required to obtain a desired slump was largely a trial and error process, but after a significant number of test mixes had been measured, it was found that varying the w/c ratio between 0.46 and 0.5 produced a range of slumps that could be poured through the laboratory tremie. This was adopted as the standard range acceptable for use in the lab study, with very few mixes deviating from this range at either the upper or lower bound.

This first series of tests in the LPC did not include a model rebar cage and was constructed in dry conditions. The only parameter being investigated in these trials was slump. The hypothesis was that a stiffer material will not flow radially as fully as a looser material, and therefore the pressure exerted by the shaft on the wall of the borehole would be lower for stiffer mixes.

The second series of tests in the LPC was designed to compare the difference in sidewall pressures when rebar cages having various bar spacings were used. The hypothesis was that tighter steel spacing would impede the flow of concrete and cause the material to segregate. This segregation would leave a thicker, more dense material on the inside of the cage while the material in contact with the wall of the borehole would be thinner and less dense. The result would be a lower axial capacity for shafts having tighter steel spacing than those with spacings nearer the upper range allowable by current design specifications.

Within this series of tests, the effects of drill fluid head in the borehole on the flow of concrete out of the tremie was investigated. The hypothesis was that increased pressure head in the borehole, caused by the use of any type of soil stabilizing fluid, would impede the flow of concrete exiting the tremie. This too could lead to segregation of the mix and result in a reduction in axial strength.

Allowable slurries have unit weights of 64-73 pcf for freshwater and 66-75 pcf for saltwater (Crapps, 1992). Since freshwater having a density of 62.4 pcf is slightly less dense than the minimum allowable during construction, proving this hypothesis using freshwater in the borehole would also prove it for any fluid having a higher unit weight. Therefore, a series of test shafts were poured in the LPC where the slump and rebar spacing were kept constant for each pair of shafts. Additionally, each shaft was constructed and poured under dry and wet hole conditions.

The final series of tests in the LPC were designed to compare the head differential inside and outside of the reinforcement cage. The hypothesis was that tighter steel spacing would impede the flow of concrete and cause head differentials from inside to outside of the rebar cage. This head differential has the potential of causing voids or trapping loose materials (i.e. sand) in the drilled shaft.

3.2.1 LPC Mix Designs

A total of seven trials were completed for this series of tests, and of those seven, five were unique mix designs. The w/c ratio was adjusted for the different mixes in order to obtain the desired range of slump values. The first mix designs used in this series were specified as a low to medium slump, in the range of 6 to 8 inches. Water to cement ratios of 0.47 to 0.50 were used, and batch volumes of 1.0 to 1.5 ft^3 were specified depending on how many additional slump tests would be performed. Speedy moisture testing was completed on the fine aggregate material prior to each mix, and proportions were adjusted accordingly. As an example, for a w/c ratio of 0.47, a 1.0 ft^3 batch volume using the standard 1100 lb/yd^3

cement content, and fine aggregate moisture content of 1.3%, the following proportions were used: 40.74 lbs cement, 80.85 lbs sand, and 18.09 lbs water.

A second round of testing under the same conditions was completed using higher slump material, because during the first set of tests, the lower slump material proved very difficult to pour through the 2 inch I.D. hopper valve and tremie pipe. Water cement ratios of 0.51, 0.52, and 0.53 were specified for these trials.

3.2.2 LPC Casting Procedure

The LPC was flushed prior to each test with water. Data during the filling and draining of the cell with water was collected in order to verify acceptable performance of the base load cell and lateral pressure transducers (Figures 3-5 and 3-6). Mixing of the mortar commenced during the water-draining operation of the LPC verification stage, and pouring was completed shortly thereafter. Video footage and still photography was used to collect qualitative information on the characteristics of flow within the cell.

Just after mixing each batch, but prior to pouring, the slump was measured. It was desired to keep all pours within the FDOT 7 inch to 9 inch slump guidelines, although some deviation above or below these values was acceptable. Also, any batch having a slump of about 6 inches or below would be difficult if not impossible to pour with the 2 inch tremie in the lab. The required slump for pouring was 8 in.

3.3 LPC Test Series I - Slump vs. Lateral Pressure

The first series of tests completed in the LPC focused on the effect of slump on the lateral pressure exerted by the concrete column on the walls of a borehole. The hypothesis of LPC test series one was that as the slump of the concrete (as placed) decreased, there would be a notable decrease in skin friction capacity of the finished shaft due to a smaller lateral pressure within the concrete column. This lower pressure during the pour would cause the fluid concrete to penetrate a smaller distance beyond the rebar cage than would be seen with a high slump mix. Ultimately, the contact between concrete and soil would be reduced and skin friction capacity would suffer.

In order to test this hypothesis the lateral pressure cell was used, but for this first series of tests no rebar cages were installed. The only variable between tests was the mini-slump (slump) of the mortar mixes. The LPC was calibrated using clean tap water prior to testing with mortars, which for the first series included seven mixes.

Upon completing each pour, the LPC was allowed to sit undisturbed for 2 minutes so that pressure transducer and load cell readings would stabilize. The resulting lateral pressure distribution over the 33.25 inches of depth in the LPC was then recorded, and is plotted in

Figure 3-7 for several representative slumps. Notice that regardless of the slump, the lateral pressure was relatively the same. As a reference, the dashed line is the lateral pressure of water.

Any excess fill was recorded and used to normalize all pressures to a standard fill of 33.25 inches. The measured vertical pressure distribution was also recorded for this same fluid depth, and a plot of lateral pressure versus vertical pressure, as shown in Figure 3-8, leads to a *coefficient of lateral concrete pressure* (κ), which is the slope of this graph as defined in the following equation. Note that the Greek letter kappa (κ) is used here in lieu of using K, which is normally reserved for lateral pressures in soils.

$$\kappa = \frac{(\sigma_{lat2} - \sigma_{lat1})}{(\sigma_{v2} - \sigma_{v1})}$$

Shown in Figure 3-8 is the lateral concrete pressure as a function of vertical pressure. The relationship between the two is linear, and the slope of the line for each equals the value of κ for that mix. As a reference, water pressure is also plotted and is seen to be the lower bound of these curves (dashed line). Again, very high slump materials show a tendency towards having lateral pressure coefficients closer to that of water, with a corresponding decrease in vertical fluid pressure. Overall, however, the higher unit weight of the mortars (and concrete) leads to a considerably higher vertical pressure as compared to water, so even with a slight reduction in κ the lateral pressures developed are still well above those of water.

3.4 LPC Test Series I Results

For the mixes studied, κ varied from 0.818 to 0.982, with an average value of 0.882. As shown in Figure 3-9, approximately 71% of the collected data falls within +/- one standard deviation of the computed κ value. This average value of 0.882 is slightly lower than that of water, which theoretically has a "κ" equal to 1.0, and was measured in the LPC to have an experimental value of 0.99.

The results of this preliminary series of tests suggest that the lateral pressure of the completed concrete shaft does not vary significantly with slump within the specified range of values (from 7 to 11 inches) (Figure 3-10). The coefficient of lateral concrete pressure remains nearly constant at 0.88 throughout the range of slumps tested. Pressures measured at the upper and lower transducers for all tests were 1.89 psi and 1.18 psi, respectively.

As expected, the radial flow of the mortar leaving the tremie was greatly affected by the slump of the material. It was apparent during all pours that the higher slump material flowed more easily outward and also tended to rise higher in the LPC without any necessary withdraw of the tremie during placement.

3.5 LPC Test Series II - Effects of Cage on Lateral Pressure

The hypothesis of LPC test series two was that as the minimum clear spacing of rebar in the reinforcement cages was decreased, there would be a corresponding decrease in skin friction capacity of the finished shaft due to a reduction in lateral pressure (because of impeded concrete flow) and also because of segregation of the concrete constituents. Aggravating this situation is the use of a drilling fluid for borehole stabilization. The increased vertical pressure (fluid head) in the hole would further retard the flow of concrete out of the tremie and through the rebar cage. This would ultimately result in a shaft having a slightly reduced diameter and potentially lower skin friction capacity.

For each group of tests, mortar mixes having approximately the same slump value were used while steel cages having different clear spacing between bars were included in the cell. Three different steel cages were constructed from standard hardware cloth. The hardware cloth had grid sizes of one quarter inch, one half inch, and one inch (Figure 3-11). Also, tests were either performed using an initially empty cell (dry hole) or with 29 inches of water head added prior to placement of the concrete (wet hole). The rate of concrete placement was kept low in order to avoid dynamic fluid effects, and to allow an adequate number of measurements to be taken during each pour.

3.6 LPC Test Series II Results

Multiple mortar pours were completed for this series of tests, and the results were added to those of Series I testing to expand the database. The results of this second series of LPC tests suggest that the lateral pressure of the completed shaft does not vary significantly with minimum clear spacing of rebar, for materials having a slump within a range of values from 7 inches to 11 inches. Apparent in Figure 3-12, all values of lateral pressure are similar regardless of the clear spacing of the rebar. The median pressures observed in Figures 3-10 and 3-13 (Series I and Series II, respectively) are nearly identical. This is due to zero compliance once the mortar made contact with the cell walls. Therefore, pressure did not vary considerably with clear spacing because the coefficient of lateral concrete pressure (κ) remains nearly constant.

For the mixes studied in this test series, κ varied from 0.818 to 0.982, with an average value of 0.886. As shown in Figure 3-14, approximately 80% of the collected data falls within +/- one standard deviation of the computed κ value. This increase in the certainty of κ lends credence to the conclusion that lateral pressure is not significantly affected by clear rebar spacing. For the mortar mixes used, and with the 1/4" grid spacing (the tightest cage), a *clear spacing to diameter ratio* (CSD) of approximately 18 is calculated as determined by the following equation.

$$CSD = \frac{Min.\ Clear\ Spacing\ (inch)}{Max.\ Coarse\ Aggregate\ Diameter\ (inch)}$$

The combination graph shown in Figure 3-15 also illustrates clearly that for the range of slumps tested, the lateral pressure developed in the shaft remains confined to a small band, with a mean value of about 2 psi. Additionally shown in this figure is the measured wet unit weight (density) of the mixes as-poured, again as a function of slump. These values appear to be even less a function of slump than do the lateral pressure values, with a mean unit weight of 130 pcf obtained very consistently for the different mixes.

3.7 LPC Test Series III - Head Differentials

The final series of tests in the lateral pressure cell were derived out of necessity. During the earlier test series, it was evident that during tremie placement of the mortar into the LPC a measurable head differential existed between the material inside the rebar and that on the outside. Contrary to what is expected in the field, a uniform rising column of concrete was not observed in any of the earlier LPC tests. It appeared that the material inside the rebar cage rose to a much higher level than that on the outside, and only after a critical head differential was reached did the material finally move to the outside of the cage. It was decided to perform a series of experiments in which common field practice during tremie placement of a shaft was simulated, and the depth-to-concrete on the inside and outside of the cage during each pour was measured. This was performed using a weighted-tape system, but an obvious advantage of the lab study was that at all times the material was visible as it flowed in the LPC. Therefore, still photography and video footage was recorded for each of these experiments. Using the measured head differentials and interpreting the video footage, a quantitative analysis of this phenomenon was performed and is documented herein. As with Series II, the rate of concrete placement was kept very low. This allowed near-static head differentials to be measured, and eliminated the need to consider dynamic effects in the analysis of the data.

The variables in this test series were construction method (dry hole versus wet hole) and shear steel spacing. Three different steel cages were again used, having a grid pattern identical to those of Test Series II. Each of these cages were used once for dry hole construction and once for wet hole construction, for each mix tested. The goal was to monitor the aforementioned head differential as shear steel spacing and external pressures were varied, and also as the slump of the material was varied from stiff to loose.

3.8 LPC Test Series III Results

Test Series III provided new and useful data that strongly refuted the idea that concrete rises in a uniform manner, scouring the side walls of the borehole as its level increases. To the contrary, it was observed that as the previously defined clear spacing ratio (CSD) decreases, and as slump decreases, the concrete level differential between inner-cage and outer-cage material increases. This differential is responsible for radial flow of concrete through the cage as observed in the flow characteristics illustrated in Figures 3-16 through 3-22. These photographs were taken during LPC testing using the three different cage configurations previously discussed, and the formation of large head differentials as the clear spacing of the cage decreases was apparent. Also, as can be seen in Figures 3-23 and 3-24, the head differential varies considerably with slump. Figures 3-25 and 3-26 show the head differentials that developed in the LPC using the 1/2 inch and 1/4 inch cage spacings and mixes of similar slump varying from 6 inches to 10 inches. These head differentials developed quite readily even in the absence of coarse aggregate.

In and of itself, this differential is not generally a problem. However, when one considers exactly how the material migrates from the higher levels inside the cage to the lower levels outside the cage, it becomes apparent that the material does not flow as anticipated. Instead, material builds up along the tremie until some critical inner-to-outer differential is reached. At this point, the material begins to slide from the higher potential to the lower potential, and as it interacts with the rebar cage rotation of the material is introduced in addition to vertical and radial translation. This rotation during translation can be describe in laymen's terms as a "rolling" of the concrete surface as it rises in the borehole.

Obviously this rolling motion has serious implications when considering the final structural integrity of the shaft. Any loose debris remaining in the borehole after drilling is likely to be left in place as the concrete rolls and folds over itself, creating potential void pockets in the finished shaft. Also, any material that falls into the concrete from either the surface or the sidewalls of the excavation during pouring will likely become collected and confined by the rolling concrete surface. Once this material becomes stationary, the concrete will simply roll over and around it, and again the potential to form significant voids is present. One other possible consequence of this rolling action is that it prevents the rising concrete from scouring the sidewall of a borehole during placement. Should a filter-cake forming drill fluid be left in place long enough that the cake thickness becomes significant, the rising column of concrete will not scour it clean. Therefore, the skin friction of the shaft will be much less that anticipated during design.

Comparing the flow characteristics observed in the LPC in a quantitative manner proved challenging because most of the data collected was qualitative in nature (i.e. video, photographs, etc.). However, there was a way to numerically describe the flow through the cage as a function of the aggregate diameter, clear spacing, and construction method. Using the video records collected for each test, radial flow was determined based on the time it took for the mortar column to rise a fixed distance. Knowing the annular volume between the cage and the LPC wall, this rate of rise was converted to radial flow.

With this information, several comparisons become possible. First, shown in Figure 3-27, the radial flow rate of the mortar is given as a function of the nominal grid opening for each of the three cages used in the LPC tests. As expected, the flow rate increased as the clear spacing increased, but appeared to approach an asymptote with increasing grid openings.

Taking the radial flow rate and dividing by the clear area penetrated yields the quantity of flux. Shown in Figure 3-28 is the relationship of both flow and flux to grid opening for the three cages tested.

It should be noted here that the above relationships were the result of construction using the dry-hole method. If data from wet-hole construction is introduced, the trends between tests remain similar, but the overall flow through the cage is further reduced for each configuration. This is due to the fluid pressure in the borehole that has to be overcome by the outward bound concrete.

When comparing the lateral pressure gradient within the LPC for dry versus wet construction, in all cases the gradient attained in the wet-hole tests was smaller. This can be interpreted as the reduction of lateral flow of material through the cage under these conditions. However, there is perhaps a much more useful (and practical) relationship that can be derived from the LPC testing.

Collection of data from several of the dry-hole tests in the LPC led to the development of a possible relationship between CSD and the maximum head differential observed during a pour. For concrete within a range of slump values from 6 inches to 10 inches, the potential head differential as a function of CSD is shown in Figure 3-29. This relationship was derived from measurements of 13 individual LPC tests. When deciding on how to present this relationship, several more complex equations were studied. These included second and third order logarithms, first through third order inverse logarithms, and higher order polynomial functions. However, this relationship as presented was merely a starting point for an obvious avenue of additional research (addressed in Chapter 7).

As presented, the head differential is analogous to a pressure differential between the inner and outer cage material, and this differential stems from head loss through the cage. Further, the head difference should also be proportional to the square of the radial flow rate. In all cases herein, flow rates were maintained slow enough so that numerous static head measurements could be taken during the pour.

It is interesting to note that the commonly used CSD minimum of 3 to 5 suggested by O'Neill and Reese (1999) has the potential of leading to flow differentials in excess of 14 inches under dry hole conditions, which is a best-case scenario. As suggested earlier, the addition of water or drill fluid in the borehole increases the magnitude of this differential, and so amplifies the possibility of anomaly formation. If this 14 inch differential is doubled to 28 for wet hole conditions, the pressure associated with this amount of concrete is (28in/12in/ft)*130pcf = 303 psf. Looking at this information from the opposite perspective, it would take a concrete pressure of roughly 300 psf to penetrate a rebar cage under wet hole conditions when the CSD was in the range of 3 to 5.

Clearly further research into this relationship is desirable. With an increased number of data points collected for a variety of construction techniques and slump values, a family of curves could be developed that would guide engineers in the proper design combinations of cages and aggregates. Improving the choices made while selecting clear spacing and maximum aggregate size will surely reduce the likelihood of constructing faulty drilled shafts.

3.9 Arching of Mortar During Tremie Placement

During the execution of the experiments composing the three LPC test programs, the phenomena referred to herein as “arching” of the advancing mortar surface as the column rises in the cell was observed. This can be seen by studying the shape of the top of the mortar column shown in Figure 3-30. As previously discussed, during tremie placement a head differential between the inside and outside of the reinforcing cage is created. Material closest to the tremie on the inside of the cage tends to rise higher than the level measured outside of the cage. In addition to this differential, the material within the confines of the cage has a slight head differential which manifests itself in a arched surface at the leading edge of the rising column.

As material leaves the tremie and moves outward, it must displace material already in place in the cell. The existing material is forced outward and also upward. At the interface between the tremie pipe and the mortar, frictional forces develop and tend to cause down drag on that part of the material. Along the inner surface of the reinforcing cage, frictional forces develop which again cause a down drag on the mortar. Between the tremie and the cage there is nothing for the advancing mortar column to interact with and so the upper surface tends to be at a higher elevation than the material directly adjacent to the cage and the tremie. The resulting arched shape is interesting to study as it rises in the cell.

As stated earlier, fresh material leaving the tremie causes the mortar already in the LPC to move radially outward and longitudinally upward at the same time. This motion is visible in the arched section of material between the tremie and the reinforcement cage. The rising material tends to “roll over” on itself as it moves up the cell. Material from the upper portion of the arch rolls downward toward the reinforcing steel as well as toward the tremie pipe. At both locations, down drag pulls this material back under the surface.

This arching must occur during the full scale construction procedure as well. Potentially this leads to a serious problem with drilled shaft construction, particularly when slurries are used to stabilize the borehole. Design and construction specifications are based on the assumption that “all of the slurry is displaced from the borehole by the rising column of fresh concrete” and that “the slurry does not weaken the bond between the concrete and the natural geomaterial” (O’Neill and Reese, 1999). However, the behavior observed in the LPC testing does not support this assumption, and in fact refutes it. The concrete does not rise in a uniform column flushing material out of the borehole, and more importantly scouring the walls of the borehole as it advances. Instead, this rolling action just presented serves to press material in the borehole into the walls, potentially weakening the bond between concrete and soil.

Figure 3-1. Lucite Tube Used to Construct the Lateral Pressure Cell

Figure 3-2. Resistive Strain Gages Mounted

Figure 3-3. Lateral Pressure Transducers Mounted to LPC Side Wall

Figure 3-4. Load Cell (inset) is Mounted Between the Base Plates

Figure 3-5. Water Calibration of the Lateral Pressure Cell Prior to Mortar Testing

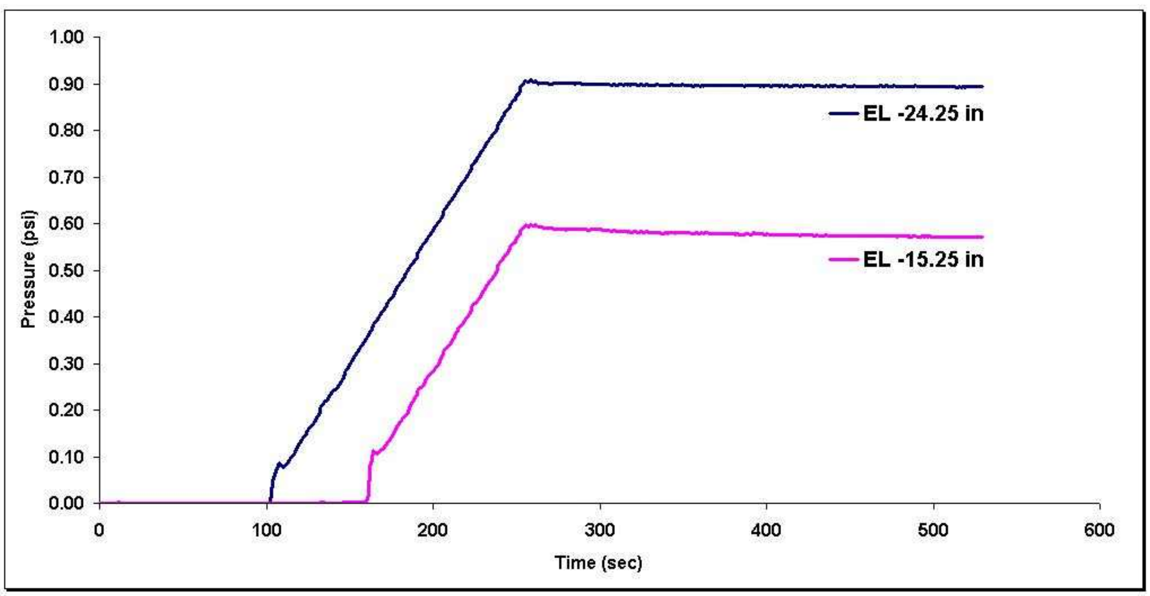

Figure 3-6. Water Calibration Data Collected Before Each Mortar Test in the LPC

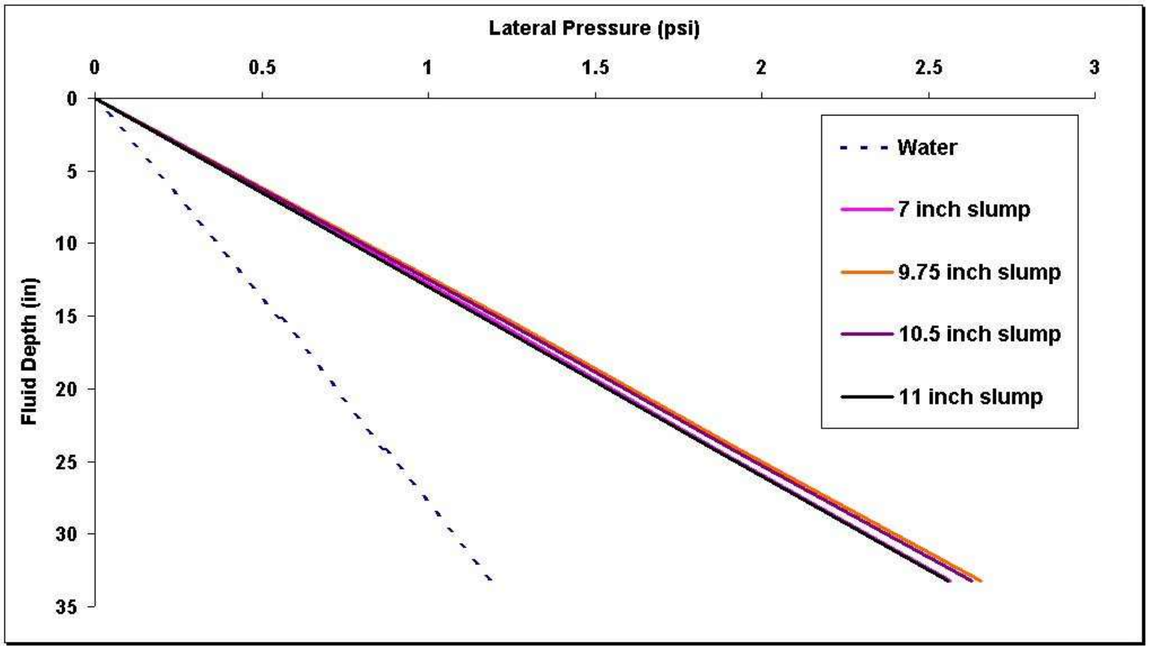

Figure 3-7. Lateral Pressure Developed for Various Values of Slump

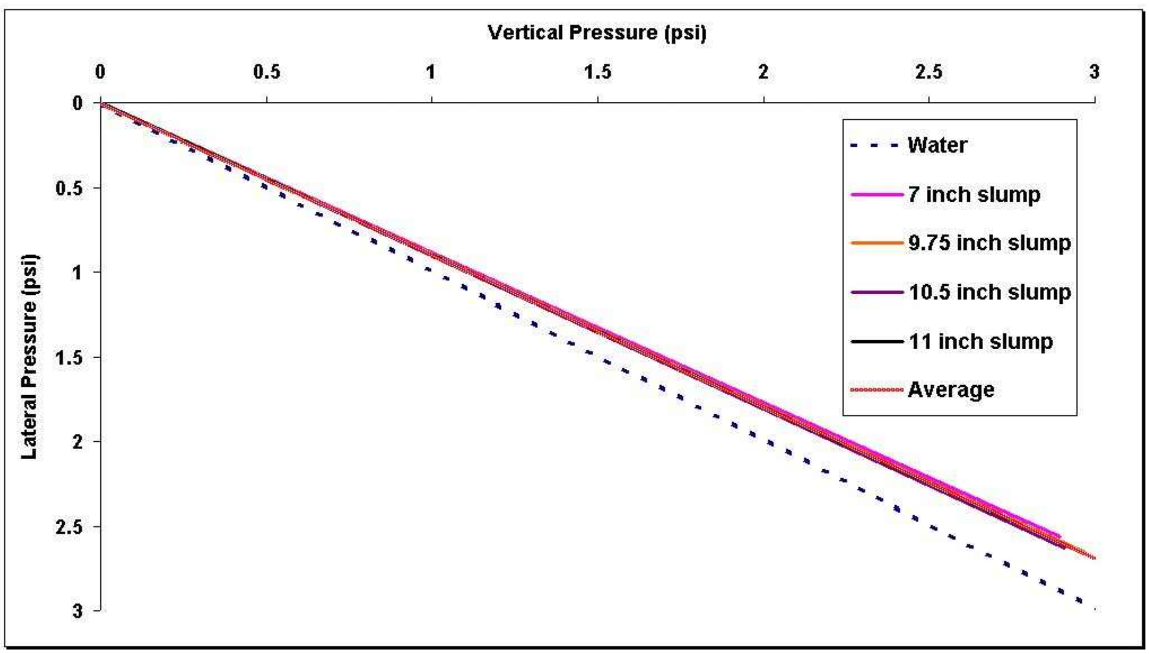

Figure 3-8. Lateral Pressure as a Function of Vertical Pressure in the LPC

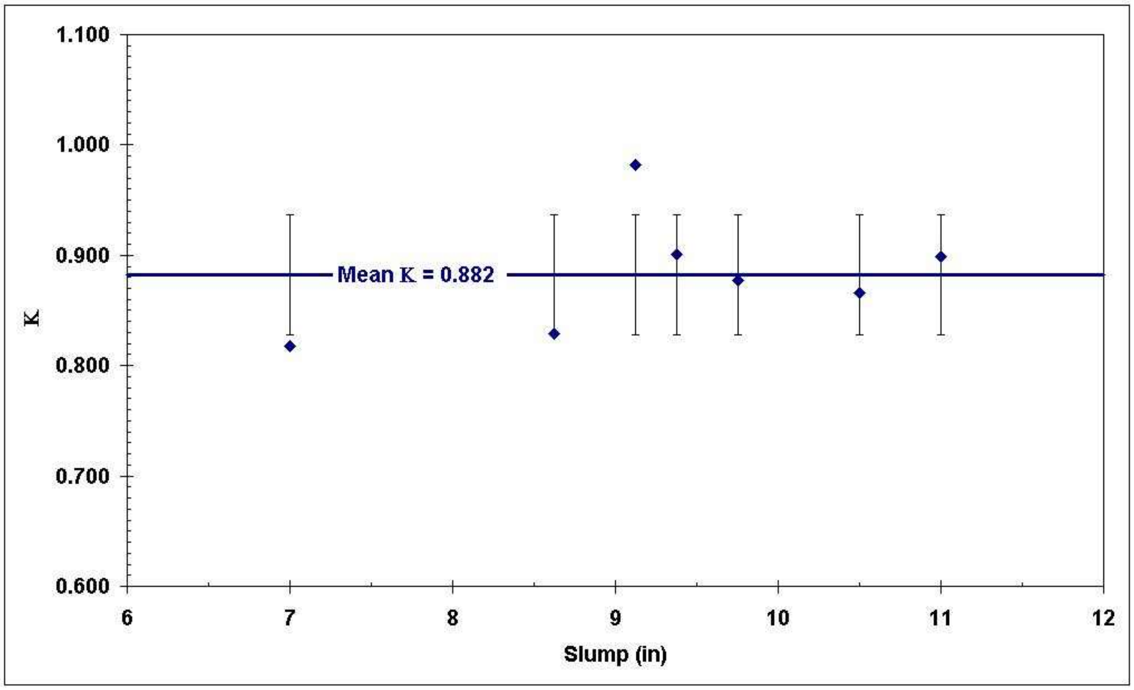

Figure 3-9. Kappa as a Function of Slump for LPC Series I

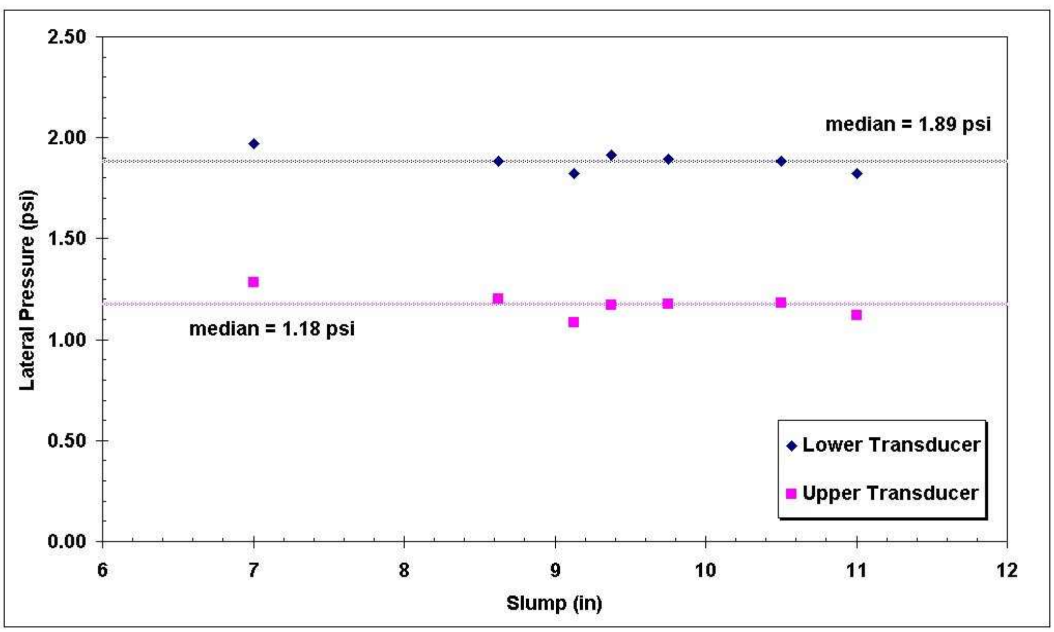

Figure 3-10. LPC Series I Lateral Pressure as a Function of Slump

Figure 3-11. Model Cages Used in LPC from Left to Right; 1", 1/2", 1/4" Grid

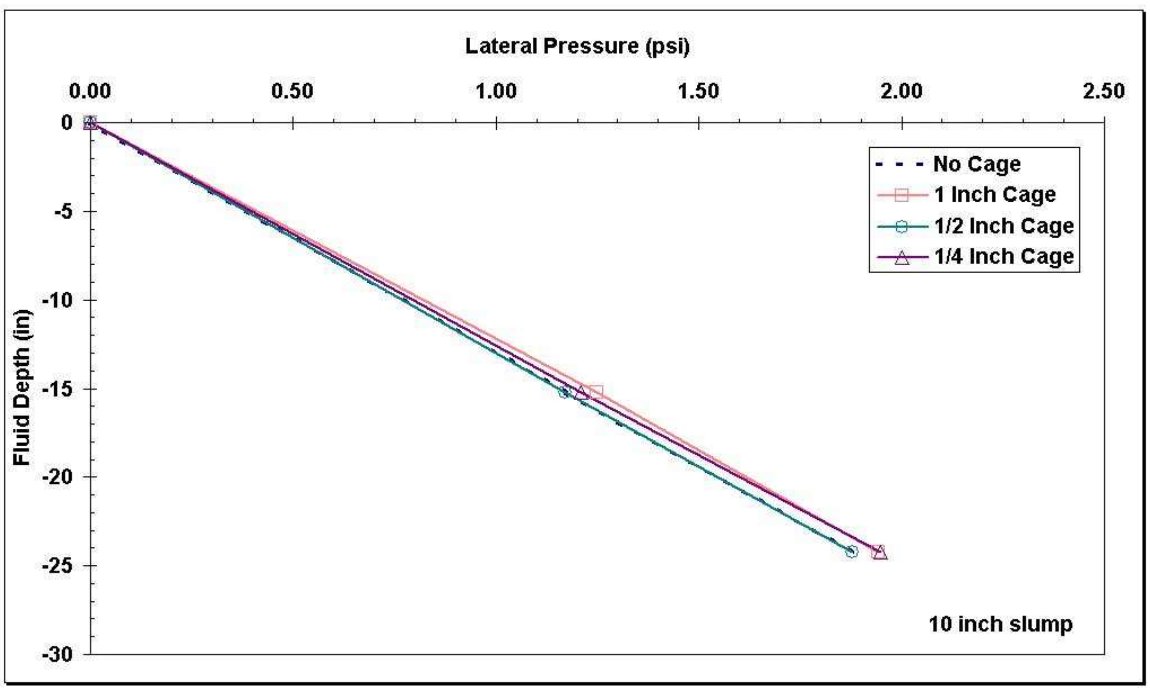

Figure 3-12. Lateral Pressure Versus Depth for Various Rebar Clear Spacing

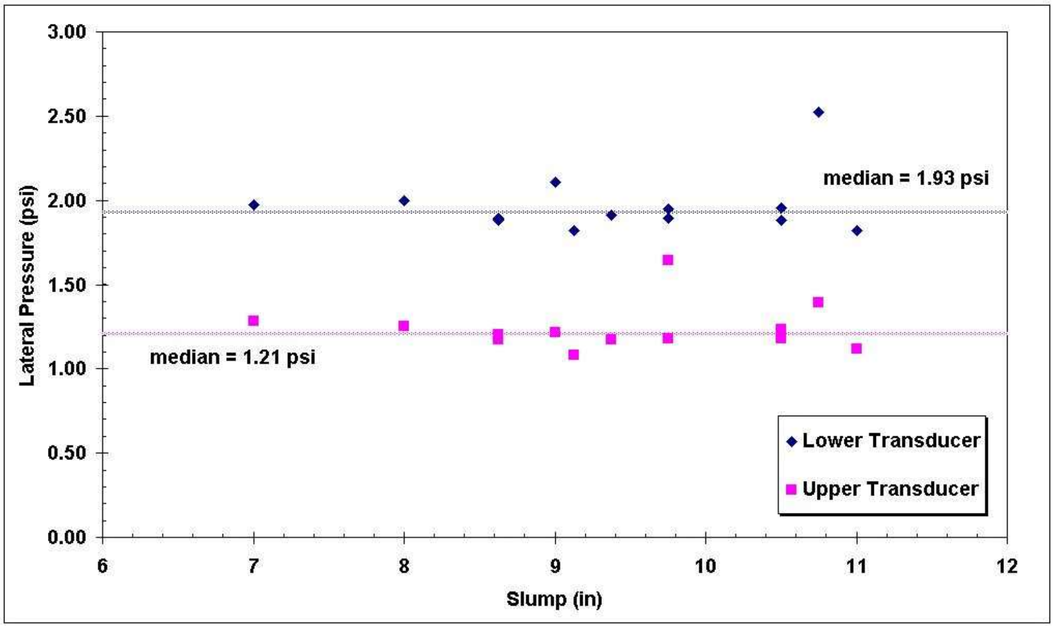

Figure 3-13. LPC Series II Lateral Pressure as a Function of Slump

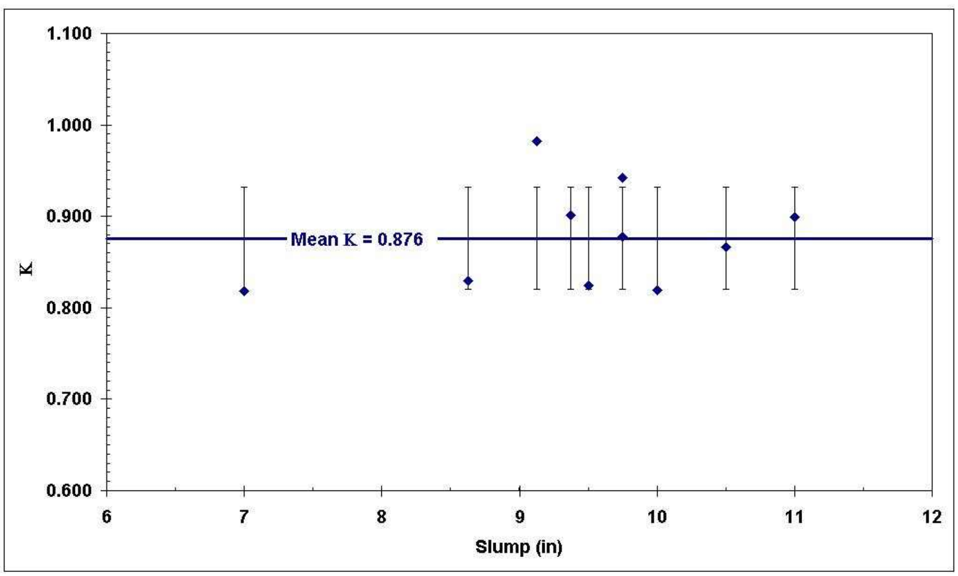

Figure 3-14. Kappa as a Function of Slump for LPC Series I & II

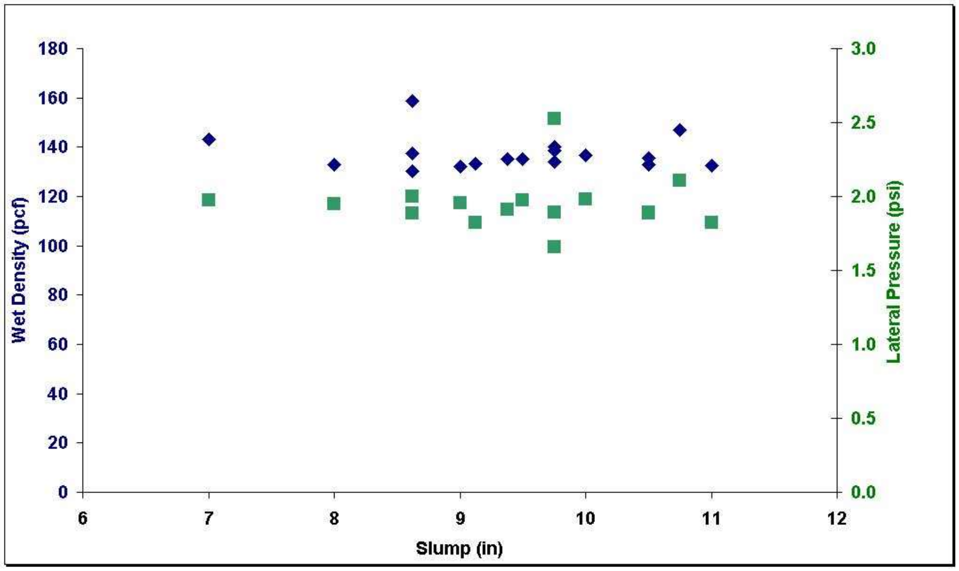

Figure 3-15. Wet Density and Lateral Pressure Versus Slump for Various Mortar Mixes

Figure 3-16. Head Differential Using 1/4" Spacing, Dry Hole Construction

Figure 3-17. Alternate View of Head Differential Using 1/4" Spacing, Dry Hole

Figure 3-18. Flow past Lateral Pressure Transducer with Obvious Head Differential

Figure 3-19. Head Differential Using 1/2" Cage, Dry Hole Construction

Figure 3-20. Alternate View of Head Differential Using 1/2" Cage, Dry Hole

Figure 3-21. Head Differential Using 1" Cage, Dry Hole Construction

Figure 3-22. Alternate View of Head Differential Using 1" Cage, Dry Hole

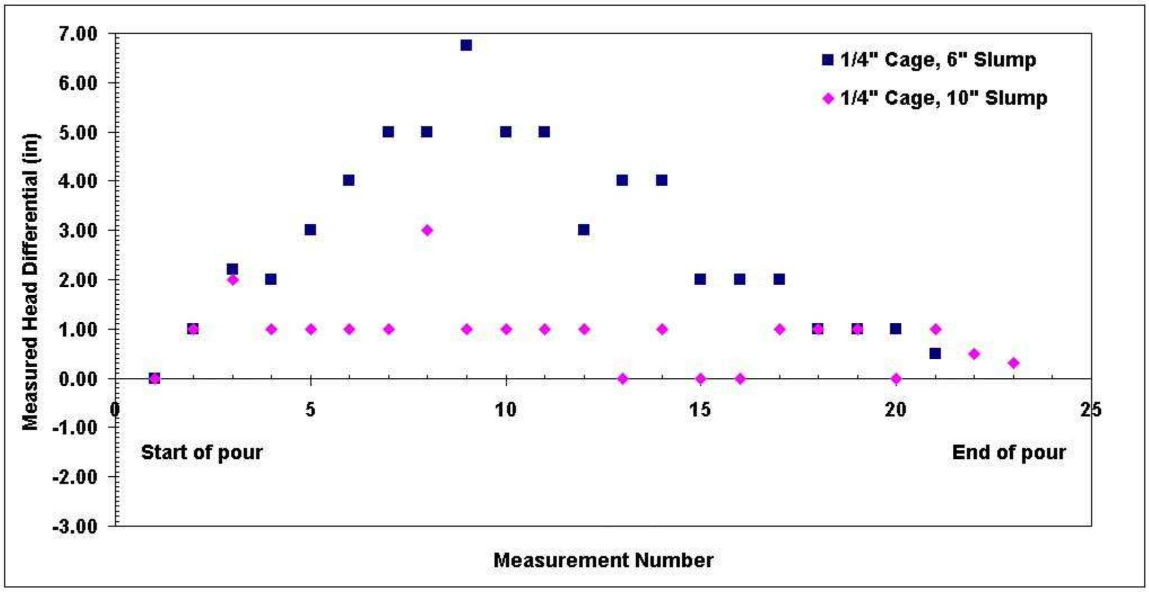

Figure 3-23. Head Differential Comparison for 1/4" Cage, 6 and 10 Inch Slump

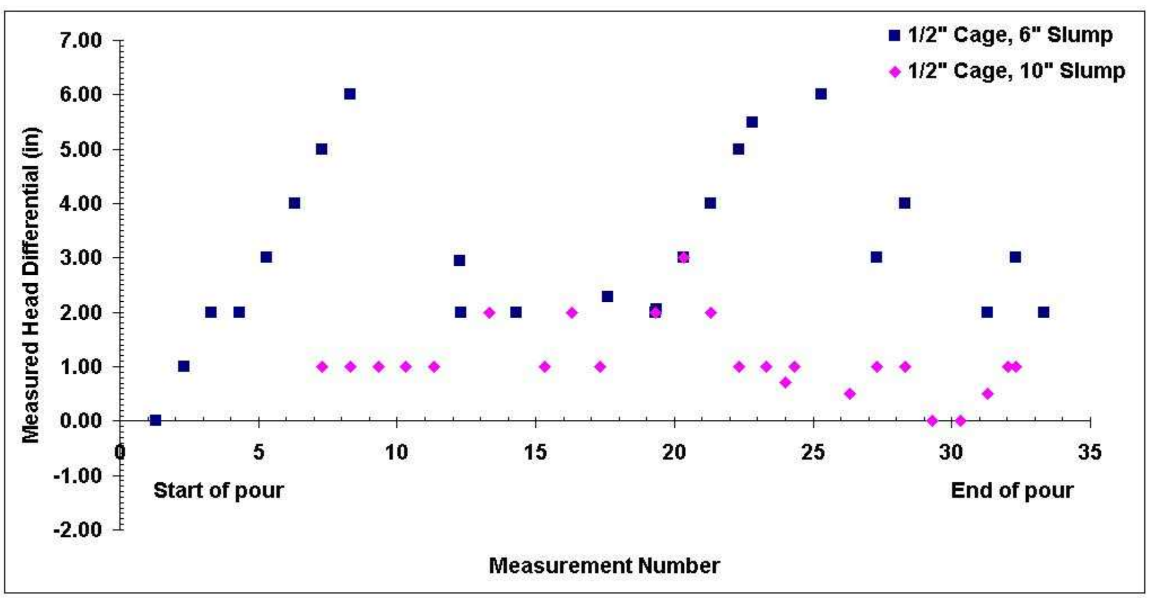

Figure 3-24. Head Differential Comparison for 1/2" Cage, 6 and 10 Inch Slump

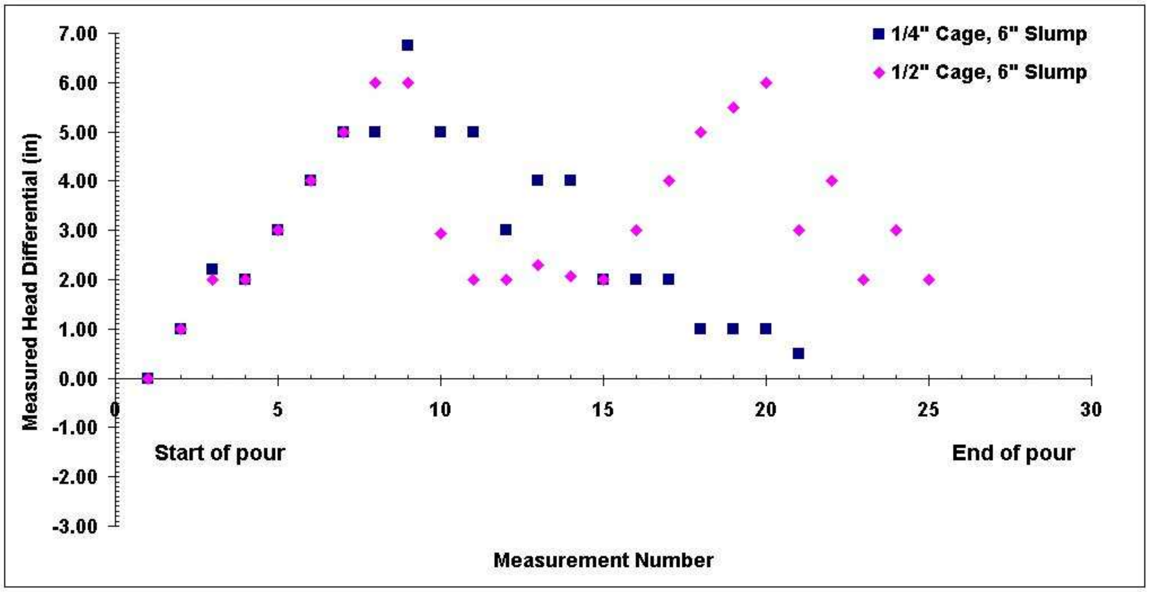

Figure 3-25. Head Differential Comparison for 6" Slump, 1/4" and 1/2" Cages

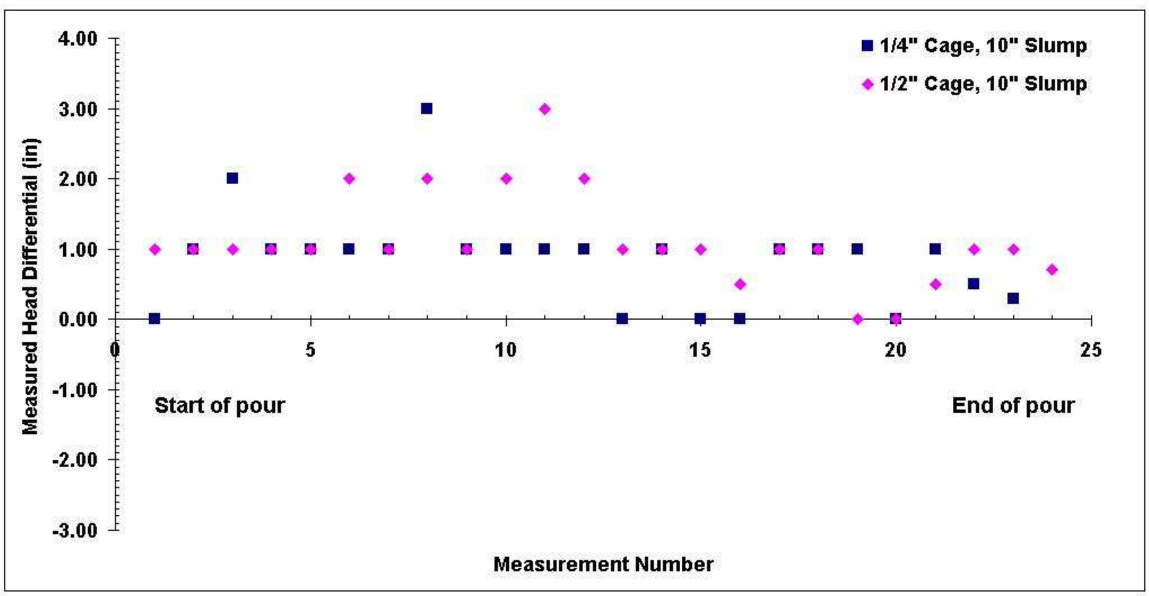

Figure 3-26. Head Differential Comparison for 10" Slump, 1/4" and 1/2" Cages

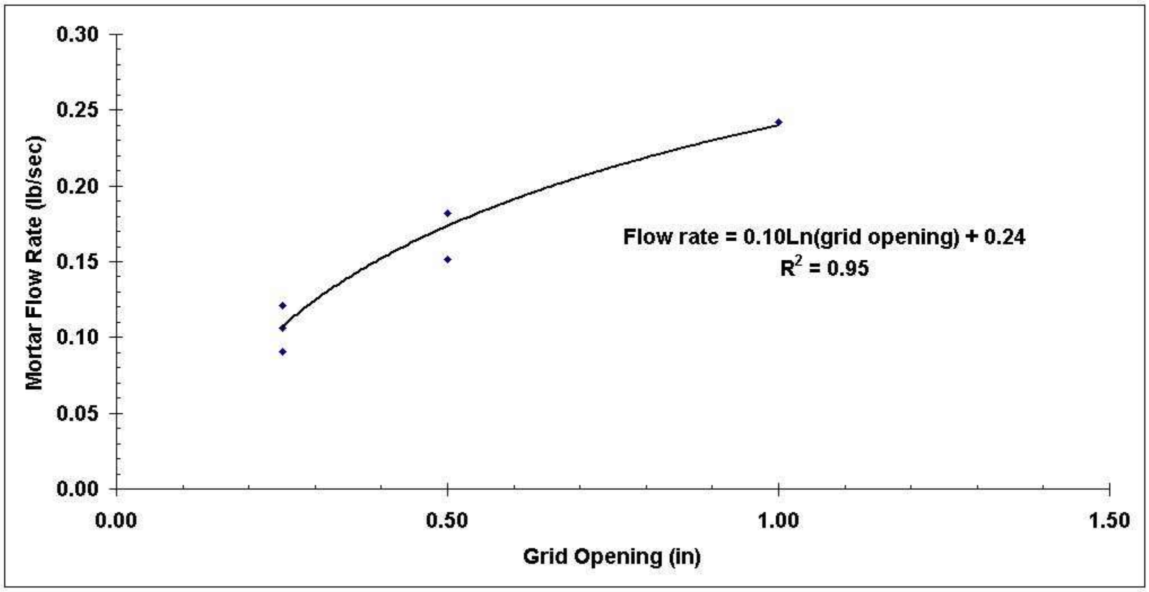

Figure 3-27. Radial Flow Rate as a Function of Grid Opening in the LPC

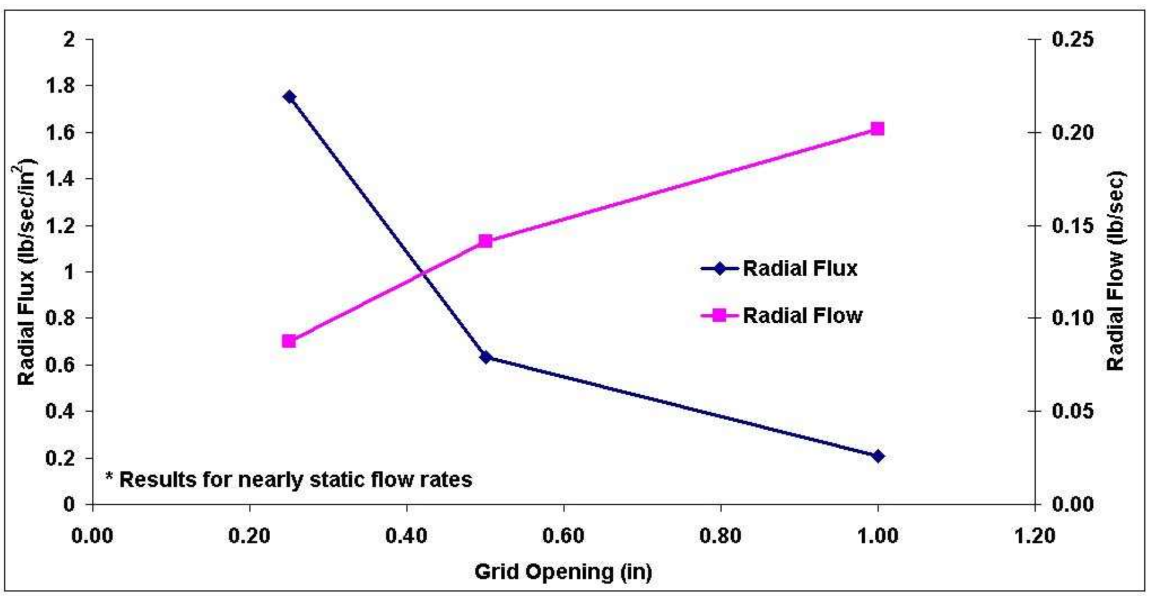

Figure 3-28. Radial Flow and Flux as a Function of Grid Opening

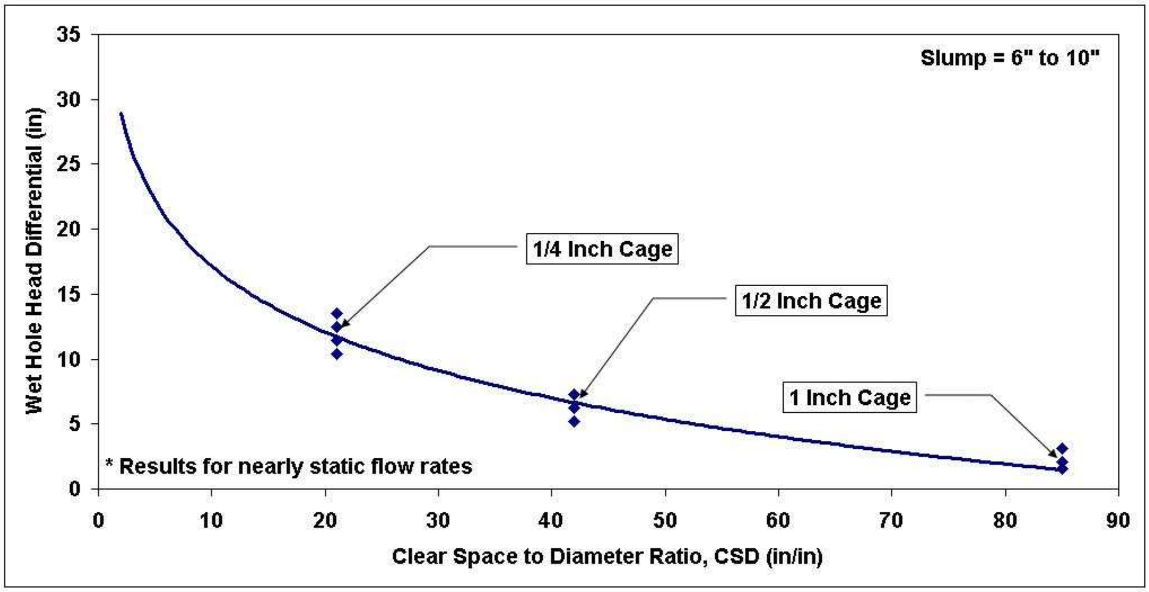

Figure 3-29. Mortar Head Differential as a Function of CSD


Figure 3-30. One Inch Head Differential in an Early LPC Pour

4. FRUSTUM CONFINING VESSEL TESTING

4.1 Background

The Frustum Confining Vessel (FCV) is a laboratory device that allows scale model testing to be accomplished without the use of an expensive centrifuge. It is not new, in fact several studies have already employed it as the basis of small scale testing (Horvath, 1996; Frederick, 2001; and Dapp, 2002). The concept of the device is that an inverted conically-shaped steel chamber, open to the atmosphere at the top, is filled with a geomaterial and stressed from the bottom via pressurized rubber bladder. The soil stress gradient is magnified due to this arrangement, varying from atmospheric pressure at the top to the bladder pressure near the base. For a bladder pressure of 30 psi, which is used throughout this research, the gradient was achieved over a depth of approximately 47 inches. If the unit weight of the sand in the frustum was 105 pcf, then without the bladder pressure the vertical soil stress at a depth of 47 inches would be about 2.9 psi. Therefore, the soil stress gradient was scaled by a factor of about 10. Figure 4-1 shows the vertical soil stress distribution in the FCV before and after bladder pressure is applied. However, fluid pressures, such as those of a mortar column or a simulated ground water table, are not dependent on the bladder pressure and therefore cannot be scaled in the FCV. In this project shafts were constructed in the FCV in the stressed state to better simulated the construction related issues.

4.2 FCV Test Program Overview

At the start of the project, 40 to 50 lab scale drilled shaft specimens were envisioned in order to study the parameters thought to be affecting shaft quality. A construction test matrix was established that targeted the three main parameters: ground water table location, casing extraction rate, and slump of the shaft mix at the time of extraction.

(1) The location of the simulated ground water table (GWT) in the FCV was varied for different drilled shaft specimens during placement of the concrete as well as during casing extraction and curing. As many of the observed anomalies in the field are located in the general vicinity of the GWT, it had been thought that the sudden change in unit weight (from saturated to submerged) of the soil in this region may have an impact on the final integrity of the shaft (taught in the drilled shaft inspectors course). Therefore, during construction of the first lab specimens the only major variable was GWT elevation. Referring to the top of the FCV as elevation zero, the GWT was placed at elevations ranging from -3 inches to -12 inches.

(2) The extraction rate of the temporary casing used in the FCV was another parameter studied. The overhead lift available in the test area had a standard upward advance rate of 8 feet/minute. This was therefore the "standard" rate used for specimens that were cast with other parameters varying. However, a number of shafts were constructed where this extraction rate was increased to 16 feet/minute and 24

feet/minute. The focus was to determine if the rate of casing extraction had any effect on the diameter of the finished shaft.

(3) Slump loss during the time it takes to pour a drilled shaft is also a concern, and FDOT currently mandates that the slump of the concrete during the time of construction never should fall below 4 inches (FDOT, 2000). In the lab, construction times are generally short because of the small volume of material needed to build a lab scale specimen. In order to study the effect of slump loss on the quality of the finished shaft, the temporary casing was left in place after the pour was completed, and excess material was stored in a plastic tub kept dry and undisturbed. Every 15 to 20 minutes the material in the tub was mixed by hand, and slump tests (full slump and mini-slump) were performed. In this way, the slump loss was reported as a function of time, and only after a target slump was reached was the temporary casing extracted from the FCV. Normally the target slump was chosen at or below the FDOT 4 inch guideline. Once axial load tests were performed on the specimens, the shaft strength was compared to the slump loss before casing extraction. Also, the diameter of the finished shaft was compared to this same slump loss information.

4.2.1 FCV Mix Designs

The mix design for the FCV testing was similar to that used in the Lateral Pressure Cell (LPC) testing program. The mortar mix consisted of 1100 lb/yd^3 cement content, water to cement ratios of 0.30 to 0.56, and batch volumes of 0.9 to 1.5 ft^3. The moisture content of the fine aggregate was tested with a Speedy moisture content testing device just prior to each mix to maintain the w/c ratio specified for each pour. The w/c ratio was adjusted for each test in order to obtain the desired slump ranges.

4.2.2 FCV Casting Procedure

For the purpose of this study, the FCV was filled with masonry sand and had an adjustable water table. Of the 53 lab scale drilled shafts constructed, 50 were completed below the water table. Only the first three specimens cast were constructed in the dry for purposes of obtaining baseline frustum performance information. The following casting procedure was used throughout the test program.

(1) Alluviation of the FCV (Figure 4-2) was used to randomly redistribute the soil inside the FCV and to allow it to fall out of suspension under the influence of gravity in a manner similar to natural alluvial processes. This ensured there was no residual state of stress remaining from the installation and testing of previous samples, and approximated virgin material without having to completely replace the soil in the FCV after each test.

(2) Once the soil in the FCV had time to sufficiently settle out of suspension (procedural 4 hr minimum), pressure was slowly applied to the bladder in 5 psi increments every

15 minutes up to the target pressure of 30 psi. The FCV was then left undisturbed for twelve hours prior to excavation.

(3) Using a duplex excavation / casing installation approach, the casing was installed to a depth of 33 inches (Figure 4-3). A common hand auger was used for most the bulk of the excavation and a small-scale clean-out bucket was used to finish excavation, as shown in Figures 4-4 and 4-5. Upon completion of the excavation, a permanent steel casing, 6 inches in length, was placed around the temporary casing to a depth of 3 inches (Figure 4-6). The additional 3 inches of stickup provided the necessary height for load testing.

(4) Prior to pouring the shaft, the water table was lowered to the desired elevation and a final pass with the clean-out bucket was performed. Pouring typically commenced within 15 minutes of the completion of excavation. A mini-slump test was used to verify that the mix can be poured through the tremie pipe.

(5) The FCV casting was designed to replicate full-scaled tremie-placed drilled shaft construction. Once the workability of the mix was verified, the entire batch was transferred to the hopper/tremie assembly and positioned directly over the center of the excavation. The pour started and continued until fresh concrete flowed from the top of the excavation. The bottom of the tremie was kept several inches below the top of the rising concrete at all times in order to maintain a seal. Once fresh concrete reached the top of the excavation, the hopper valve was closed and the hopper/tremie assembly was removed. Figures 4-7 and 4-8 show the tremie pipe and the concrete placement with the hopper/tremie assembly.

(6) The temporary casing was then removed using an overhead lift, as shown in Figure 4-9. The rate of casing extraction was 8 ft/min, 16 ft/min, or 24 ft/min depending on the goal of the particular test series.

4.2.3 Static Load Test Apparatus and Procedure

Each drilled shaft specimen constructed in the FCV was subjected to multiple cycles of static loading prior to being removed. This test was conducted axially in general accordance with ASTM 1143 and was conducted at a time between 48 and 72 hours after casting (ASTM, 1996). The apparatus used to complete the static load tests (SLT) consisted of a reaction frame, bearing plates, a hydraulic jack, and a manual pump. Instrumentation and data acquisition consisted of a resistive load cell and two linear variable differential transformers (LVDT) used to measure top of shaft load and displacement, respectively, and a StarLogger Pro (Figure 4-10) data collection unit used to condition and record the signals. The SLT test equipment and instrumentation is shown in Figures 4-11 through 4-15.

Once the SLT equipment was properly set up and calibrated, each load test was comprised of multiple load-reload cycles. The first cycle ended at a displacement of 0.25 inch, at which time the load was held for several minutes prior to unloading. A second load cycle was then performed, and was carried out until a total top-of-shaft displacement of 0.5 inches was achieved. Final load and displacement readings were taking for an additional 30 to 60 seconds following completion of the second load cycle.

4.2.4 Shaft Extraction and Investigation

Each test shaft, following load testing, was extracted and investigated for any anomalies. Once the shaft was removed from the FCV and cleaned of any loose sand, diameter and length measurements were recorded. The diameter measurements were taken at 6 inch intervals along the length of the shaft as well as just below the permanent casing. The length measurements were taken at third points around the shaft. The summary of the three testing series is presented below. Details can be found elsewhere (Garbin, 2003).

4.3 Control Shaft Tests

The first shafts constructed in the FCV were performed in dry conditions. Out of test shafts A1, A2, and A3, only A3 was load tested. Figure 4-16 shows the load-displacement curve for test shaft A3. Each of the two loading cycles of A3 were performed in 0.5 inch increments. The maximum load achieved was 9000 lbs at a displacement of 1 inch. The side shear was fully mobilized within 0.1 inches of displacement at a load of 1800 lbs.

4.4 FCV Test Series I - Water Table Elevation

At the start of the research program there were many potential parameters that were considered as the focus for lab scale testing. Obviously at this early stage of the research, the elevation of the simulated GWT in the FCV was considered an important variable. The first series of test shafts constructed would therefore focus on this parameter.

To summarize, a total of twelve model shafts were constructed for the first series of FCV testing. The focus of this test series was on the influence of the water table elevation on the integrity of the shaft. Various water table elevations were selected, ranging in value from -3.0 inches to -12.0 inches, below the top of the FCV. Shafts were constructed using the cased method, with the casing being extracted immediately following placement of the concrete. The extraction rate was kept constant at 8 feet/minute, and for the most part the mix designs used for this group of shafts were similar. For most specimens cast, mini-slump testing was performed prior to pouring. After load testing, each shaft was removed from the FCV then measured for diameter as previously described. Additionally, photographic records of any anomalies were taken, and any deviation from standard procedure was noted in the construction logs. Summarized in Table 4-1 are the relevant statistics for each of the shafts included in Test Series I. These include Shaft ID, length measurements, median diameter, mini-slump, anomalies that may have formed, water table elevation, and probable cause of the anomalies.

4.5 FCV Test Series I - Results

Studying the results of these preliminary tests in the FCV, it is apparent that the location of the water table had little to no effect on the formation of anomalies within a shaft. In every model specimen constructed, any irregularities in the finished shaft were attributed, most often, to some abnormality with the construction methodologies employed. Based on the findings of FCV Test Series I, it seems that the quality of a finished shaft depended more heavily on quality and consistency of construction rather than on location of the water table. Figures 4-17 through 4-22 show typical anomaly formations in the test shafts for Test Series I.

Table 4-1. Shaft Summary for FCV Test Series I

Shaft Information		Shaft Physical Statistics			Shaft Strength Statistics				Anomaly Type		Comments
Shaft ID	Water Table EL (in)	Nominal Length (in)	Median Diameter (in)	Mini-slump at pour (in^2)	Shear Capacity (lbs)	Shear Disp (in)	Max Load (lbs)	Max Disp (in)	Anomaly Location (in)	Anomaly Type	
S1	-3.0	23.00	4.45	14.02	-	-	1000	0.75	-23.0	Short shaft	Improper placement of mortar
S2	-3.0	34.00	4.58	14.35	1100	0.10	2200	0.60	-3.0	Neck	Improper handling of permanent casing
									-40.5	Bulge	Over-excavation
S3	-3.0	36.00	4.58	19.60	4900	0.15	6000	0.30	N/A	N/A	Good shaft
S4	-12.0	37.00	4.58	21.57	1250	0.08	2300	0.30	N/A	N/A	Good shaft despite irregular casing extraction
S5	-5.0	36.00	4.53	9.84	2150	0.06	4000	0.24	-36.0	Taper	Poor excavation and irregular casing extraction
S6	-7.0	36.00	4.57	28.32	2350	0.10	3900	0.50	-26.0	Neck	Irregular casing extraction (wobble during pull)
									-37.0	Bulge	Over-excavation
S7	-8.0	36.00	4.55	9.26	1000	0.05	3500	0.33	-3.0	Bulge	Improper handling of permanent casing
S8	-8.0	36.00	4.58	9.68	500	0.08	2500	0.63	-35.0	Odd shape	Dirty toe - poor excavation
									-3.0	Bulge	Permanent casing handling
S13	-8.0	34.50	4.59	7.69	470	0.05	2500	0.63	Entire Length	Necks	Liquefaction during casing installation
									-34.5	Short shaft	Dirty toe - poor excavation
S14	-8.0	35.75	4.57	-	1100	0.10	2900	0.50	-3.0	Bulge	Permanent casing handling
S15	-8.0	37.25	4.54	14.73	-	-	-	-	-3.0	Neck	Permanent casing handling
									-37.25	Long shaft	Over-excavation
S16	-8.0	36.50	4.54	-	1000	0.10	3300	0.78	-36.0	Neck	Irregular casing extraction

4.6 FCV Test Series II - Temporary Casing Extraction Rate

As this research study commenced, it started to become apparent that, at least in the FCV, the location of the ground water table had little if any influence on the quality of the finished shaft. Construction procedure was much more critical, however, as most of the anomalies recorded in the first test series could be attributed to some deviation from the norm during borehole excavation, casting, or finishing the shaft. Accordingly, the second series of test shafts were constructed using the same methodologies as before, but with the rate of extraction of the temporary casing being varied. Three individual extraction rates were studied, starting with the 8 feet/minute standard that was used previously, and including additional shafts for which the casings were removed at either 16 feet/minute or 24 feet/minute. The following sections present and discuss the strength testing and quality inspection of each group of shafts, starting with the 8 feet/minute specimens and concluding with the 24 feet/minute specimens.

4.7 FCV Test Series II - Results

A total of 28 shafts were constructed and studied for this second series of FCV tests. The main parameter varied was extraction rate of the temporary casing. Shafts were grouped by extraction rate into three categories: 8 feet/minute, 16 feet/minute, and 24 feet/minute. The water table elevation was kept constant for all tests within this series at -8.0 inches, and although mortar mixes of various w/c ratios and slumps were used, all fell within the FDOT range of 7 to 9 inches at pour. The casing was extracted immediately after each pour, with no appreciable time allowed for slump loss to occur.

Table 4-2 summarizes the physical properties of each of these shafts as well as the strength values obtained from axial SLT testing. Figures 4-23 through 4-27 show some of the test shafts from Test Series II with anomaly formations. When comparing the median diameter of each shaft with its corresponding extraction rate, it can be seen that from 8 feet/minute to 16 feet/minute there was a small reduction in median shaft diameter for the faster extraction rate. However, increasing the rate again to 24 feet/minute (3 times the reference rate) resulted in a slight increase in the median shaft diameter.

The first trend is expected. Doubling the extraction rate of the temporary casing increases the dynamic friction forces between the inner casing wall and the concrete shaft. Because the shaft itself is in no way anchored to the bottom of the FCV, as the casing is pulled it causes the shaft to be stretched. Due to the Poisson effect, the cross sectional diameter decreases as this stretching takes place. What is initially puzzling is the second trend. It seems as though once the extraction rate becomes high enough, there is a tendency for the casing to move so quickly that the inertia of the semi-fluid shaft mix overcame the previous scenario of stretching or net suction at the base of the shaft. Therefore, although some reduction in cross sectional diameter is observed, it is not severe while the shaft mix is still fluid (i.e. no significant slump loss).

Table 4-2. Shaft Summary for FCV Test Series II

Shaft Information		Shaft Physical Statistics			Shaft Strength Statistics				Anomaly Type		Comments
Shaft ID	Extraction Rate (ft/min)	Nominal Length (in)	Median Diameter (in)	Mini-slump at pour (in^2)	Shear Capacity (lbs)	Shear Disp (in)	Max Load (lbs)	Max Disp (in)	Anomaly Location (in)	Anomaly Type	
S9	8.0	34.75	4.56	7.79	800	0.10	2000	0.50	-3.0	bulge	Improper handling of permanent casing
S10	8.0	33.00	4.52	7.69	600	0.10	1100	0.22	full length	irregular cross section	Very low slump at pour made placement difficult
S11	8.0	36.00	4.55	9.33	1450	0.10	4120	0.46	-3.0	neck	Improper handling of casing
									-36.0	slight bulge	Slight over-excavation
S12	8.0	35.00	4.54	11.75	1000	0.06	3000	0.36	-36.0	tapered toe	Poor excavation (clean-out)
S17	16	36.00	4.50	15.98	1500	0.10	4100	0.50	-36.0	tapered toe	Poor excavation (clean-out)
S18	16	35.00	4.55	39.87	1600	0.08	2600	0.50	-3.0	neck	Improper handling of casing
									-35.0	slight taper	Poor excavation (clean-out)
S19	16	36.50	4.50	10.01	-	-	5000	0.50	-3.0	neck	Improper handling of casing
									-36.5	taper	Poor excavation (clean-out)
S20	16	36.00	4.52	-	1700	0.10	5000	0.50	-3.0	neck	Improper handling of casing
S21	16	36.00	4.52	-	1100	0.10	2400	0.50	full length	reduced diameter	Casing extraction rate
S22	16	36.00	4.48	-	2700	0.10	8400	0.50	full length	reduced diameter	Casing extraction rate
S23	16	36.00	4.48	10.93	1100	0.10	3500	0.50	full length	reduced diameter	Casing extraction rate

Table 4-2. (Continued)

Shaft Information		Shaft Physical Statistics			Shaft Strength Statistics				Anomaly Type		Comments
Shaft ID	Extraction Rate (ft/min)	Nominal Length (in)	Median Diameter (in)	Mini-slump at pour (in^2)	Shear Capacity (lbs)	Shear Disp (in)	Max Load (lbs)	Max Disp (in)	Anomaly Location (in)	Anomaly Type	
S24	16	36.00	4.46	10.69	1150	0.10	3500	0.50	full length	reduced diameter	Casing extraction rate
S25	16	36.00	4.40	20.29	1800	0.10	4500	0.50	entire length	reduced diameter	Casing extraction rate
S26	16	36.00	4.47	16.56	2040	0.10	5000	0.50	-3.0	cracking	Improper casing handling
									entire length	reduced diameter	Casing extraction rate
S27	16	35.50	4.55	11.34	950	0.10	2500	0.50	-3.0	neck	Improper casing handling
									-35.5	tapered toe	Poor excavation (clean-out)
S28	16	37.00	4.55	14.97	2630	0.10	7000	0.50	-37.0	bulge	Over-excavation
S29	16	36.00	4.61	10.02	2190	0.10	4500	0.50	n/a	n/a	Excellent shaft quality
S30	16	36.00	4.56	10.97	2340	0.10	3500	0.50	-36.0	tapered toe	Poor excavation (clean-out)
S31	16	36.00	4.53	7.72	1650	0.10	3300	0.50	-32.0	tapered toe	Poor excavation (clean-out)
S33	16	36.00	4.52	8.39	1990	0.05	4700	0.50	-3.0	neck	Improper casing handling
S40	16	36.00	4.55	10.9	1950	0.10	6000	0.50	-9.0	taper	Casing extraction rate
S41	16	36.00	4.53	8.72	2700	0.10	8100	0.50	-36.0	irregular toe	Poor excavation (clean-out)
									entire length	reduced diameter	Casing extraction rate

Table 4-2. (Continued)

Shaft Information		Shaft Physical Statistics			Shaft Strength Statistics				Anomaly Type		Comments
Shaft ID	Extraction Rate (ft/min)	Nominal Length (in)	Median Diameter (in)	Mini-slump at pour (in^2)	Shear Capacity (lbs)	Shear Disp (in)	Max Load (lbs)	Max Disp (in)	Anomaly Location (in)	Anomaly Type	
S32	24	35.50	4.48	8.48	1800	0.10	5000	0.50	-3.0	slight neck	Improper casing handling
									entire length	grooves	High casing extraction rate
S34	24	35.75	4.56	9.13	900	0.07	3000	0.50	-24.0	grooves	High casing extraction rate
									-35.75	irregular toe	Improper casing handling
S35	24	36.25	4.50	8.39	500	0.10	2000	0.50	entire length	reduced diameter	High casing extraction rate
S39	24	36.25	4.58	8.69	900	0.10	2250	0.50	entire length	porous texture	High casing extraction rate
S42	24	36.00	4.52	9.36	2150	0.10	9000	0.50	entire length	grooves	High casing extraction rate
S43	24	36.25	4.51	7.91	-	-	-	-	entire length	grooves	High casing extraction rate

4.8 FCV Test Series III - Slump Loss During Pour

The final series of tests in the FCV targeted slump loss during construction as the main parameter. FDOT specifies that at no time during the construction of a drilled shaft should the slump of the concrete drop below 4 inches (FDOT, 2000). This includes all time that elapses during transportation of the material to the site, delays on the site, setup at the tremie, and pouring of the shaft. Any material that cannot maintain a slump of 4 inches or greater during this time must be rejected. This series of tests was sculpted to better define this specified construction limit on the basis of performance (i.e. function of shaft capacity).

In order to study the effects, if any, of excessive slump loss on the quality and capacity of drilled shafts in the FCV, 10 specimens were cast at an initial slump between 7 to 9 inches. Extraction of the temporary casing was completed at an extraction rate of 8 feet/minute, after the slump of the material had dropped to 4 inches or lower. The water table elevation was kept constant for all tests within this series at -8.0 inches, and load testing was conducted in accordance with the standard operating procedure.

4.9 FCV Test Series III - Results

As with the previous two test series, a summary of the physical and strength characteristics and anomaly locations for each shaft in Test Series III can be found in Table 4-3. Although it was often difficult to pull the temporary casing once the slump had fallen below about 4.5 inches, doing so in this series of tests has shed new light on the effect of slump loss on the frictional capacity of a shaft. Figure 4-28 shows the results of the tests conducted in this series as compared with those of series two, where little to no slump loss was allowed to occur before casing extraction. It is immediately apparent that slump loss has a strong effect on skin friction capacity. For the shafts where little to no slump loss was allowed, unit skin friction approached about 5 psi. However, as slump loss increased, unit skin friction values decreased dramatically. At the FDOT allowable slump loss limit of 4 inches, unit skin friction values were only about 1 psi. Below a slump of 3.5 inches, it became impossible to construct a shaft in the FCV, as the casing extraction would tear out most, if not all, of the shaft (Figure 4-29). In this instance, the slump was allowed to drop to 3.5 inches over a period of about 2 hours. When the casing was extracted, the entire shaft was removed.

It is clear that an allowable slump loss resulting in a minimum 4 inch slump is much too risky, especially when using temporary casing. Slip forming of the concrete occurs as the casing is pulled, and unit skin friction values drop off substantially as an annular space is formed around the shaft where no shaft to soil contact is achieved. Also, as this 4 inch lower bound is approached, the likelihood that a portion of the shaft will be lost during casing extraction increases. Therefore, it will be the recommendation of this report to increase this lower limit on slump loss. Further research into this phenomenon is warranted, and perhaps this slump loss criteria can be refined with more data.

Table 4-3. Shaft Summary for FCV Test Series III

Shaft Information		Shaft Physical Statistics			Shaft Strength Statistics				Anomaly Type		Comments
Shaft ID	Slump at Casing Pull (in)	Nominal Length (in)	Median Diameter (in)	Mini-slump at pour (in^2)	Shear Capacity (lbs)	Shear Disp (in)	Max Load (lbs)	Max Disp (in)	Anomaly Location (in)	Anomaly Type	
S36	4.875	36.00	4.58	8.55	680	0.07	1200	0.50	n/a	n/a	Good shaft
S37	4.75	36.00	4.60	9.04	525	0.07	2100	0.50	entire length	reduced diameter	Porous surface along entire shaft length
S38	4.875	36.25	4.56	9.62	750	0.07	2700	0.50	-36.25	tapered toe	Porous surface along entire shaft length
S44	3.625	36.00	4.54	9.52	-	-	3000	0.50	-36.00	tapered toe	Porous surface along entire shaft length
S45	4.25	36.00	4.44	8.38	390	0.07	1100	0.50	-3.0 to -8.0	minor necking	Porous surface along entire shaft length
S46	4.50	36.00	4.48	7.99	240	0.07	1300	0.50	-3.0 to -10.0	green	Porous surface and not well cured along entire length
S47	3.625	-	-	8.36	-	-	-	-	n/a	n/a	Lost shaft during casing extraction
S48	6.875	36.00	4.51	9.79	700	0.07	2500	0.50	-3.0	minor necking	Reduced diameter and rounded toe
S49	5.0	36.25	4.54	8.14	880	0.07	2500	0.50	n/a	n/a	Good shaft
S50	3.75	36.00	4.47	8.08	200	0.05	1800	0.50	entire length	reduced diameter	Porous surface along entire shaft length

4.10 Summary of Results

Of the three main parameters studied during FCV testing, only two were found to have an effect on shaft quality: casing extraction rate and slump loss. Though variations in the elevation of the water table had little effect on shaft quality, results of the construction mishaps noted during this series of tests directed attention away from geotechnical site issues and refocused on the impact of construction techniques. Figure 4-30 shows the effects of water table elevation on side shear within the FCV. Typically in design, submerged side shear values are lower than dry values. This suggests that other factors may have controlled side shear within the FCV. Though minor, the casing extraction rate proved to have an effect on the diameter of the cured shaft. Table 4-4 shows the statistical data for the extraction rate versus diameter for all the test shafts with water table elevation of -8 inches. Figure 4-31 shows the diameter of the test shafts at various slumps.

Table 4-4. Extraction Rate versus Shaft Diameter

Extraction Rate (ft/min)	Median Diameter (in)	Average Diameter (in)	Standard Deviation (in)	Number of Test Shafts
8	4.54	4.52	0.0915	20
16	4.52	4.52	0.0463	18
24	4.52	4.53	0.0362	6

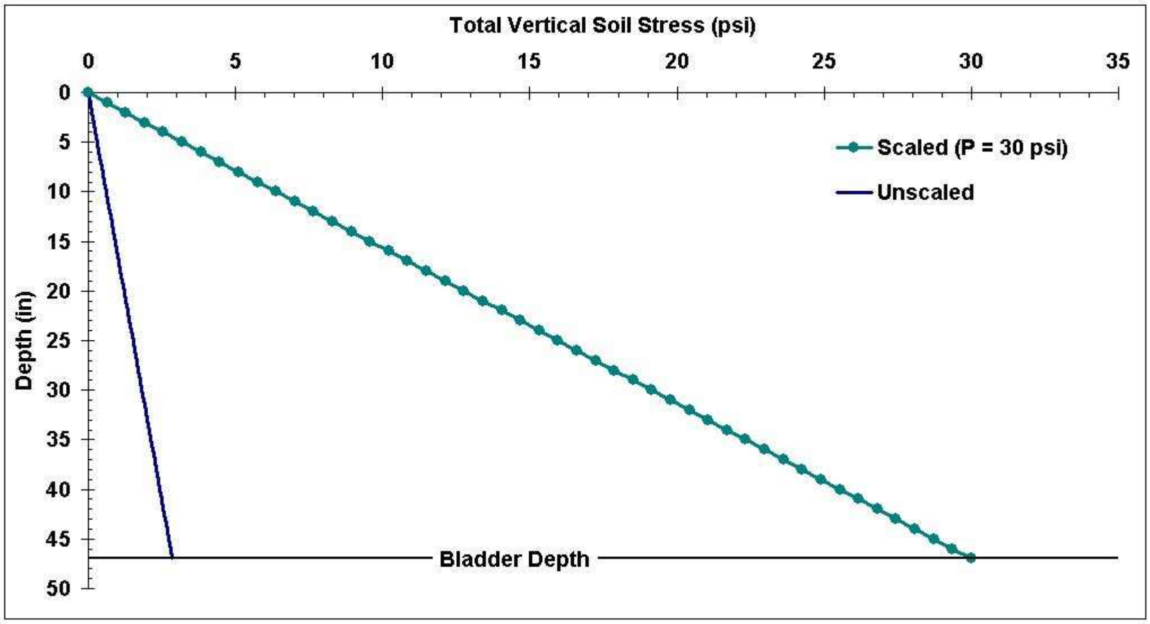

Figure 4-1. Total Vertical Soil Stress in the FCV


Figure 4-2. Alluviation of the FCV Using the Water Jet Tool

Figure 4-3. Casing Driven into Specified Depth

Figure 4-4. Primary Auger Tool Used to Excavate Most of the Shaft

Figure 4-5. Clean-out Bucket Used to Finish Excavation

Figure 4-6. Permanent Casing Installed Around Temporary Casing

Figure 4-7. Tremie Plugged to Prevent Segregation

Figure 4-8. Connecting Tremie Pipe to Hopper in Preparation for Pour

Figure 4-9. Casing Extraction Using the Overhead Lift

Figure 4-10. Starlogger Pro Data Acquisition System

Figure 4-11. Reaction Frame Used for FCV Static Load Testing

Figure 4-12. Load Cell and Bearing Plates used for FCV Static Load Testing

Figure 4-13. Load Cell Assembly Installed on Shaft

Figure 4-14. Hydraulic Jack and LVDTs Installed on Shaft

Figure 4-15. Manual Pump Used to Control Hydraulic Jack

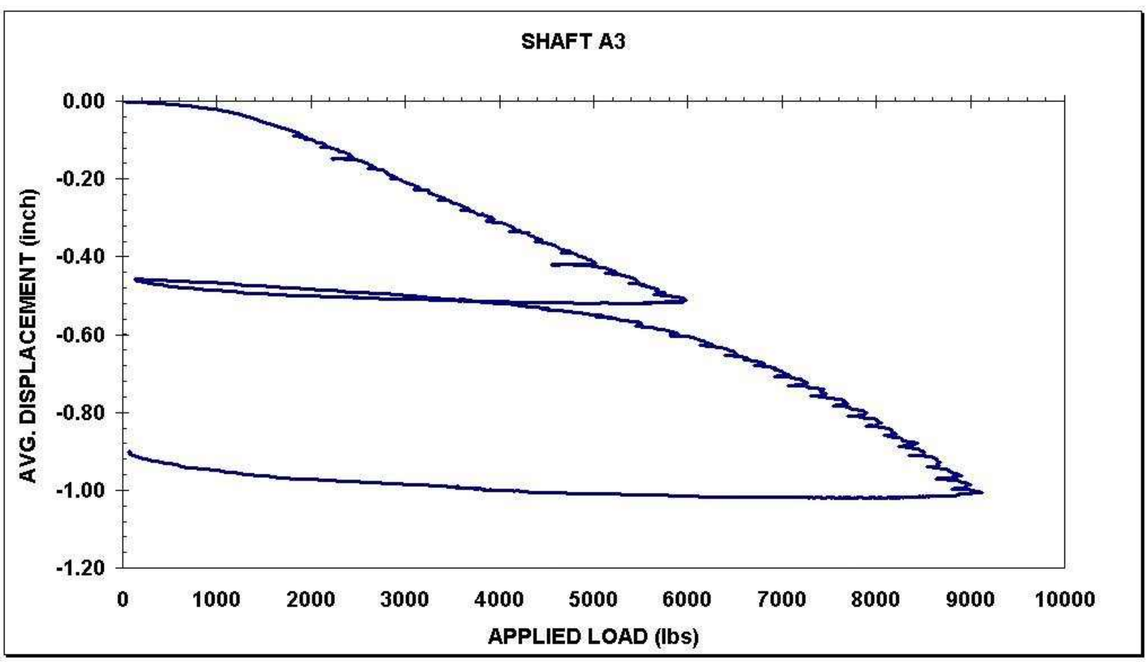

Figure 4-16. SLT Results for Shaft A3

Figure 4-17. Shaft S1 Removed from the Frustum - Undersized

Figure 4-18. Toe Bulb on Shaft S2 due to Over-excavation

Figure 4-19. Slight Neck below Casing on Shaft S5

Figure 4-20. Toe Anomaly on Shaft S5

Figure 4-21. Shaft S7 Showing Bulge Just below Permanent Casing

Figure 4-22. Uneven Toe Formation on Shaft S8

Figure 4-23. Reduced cross section at toe Shaft S10

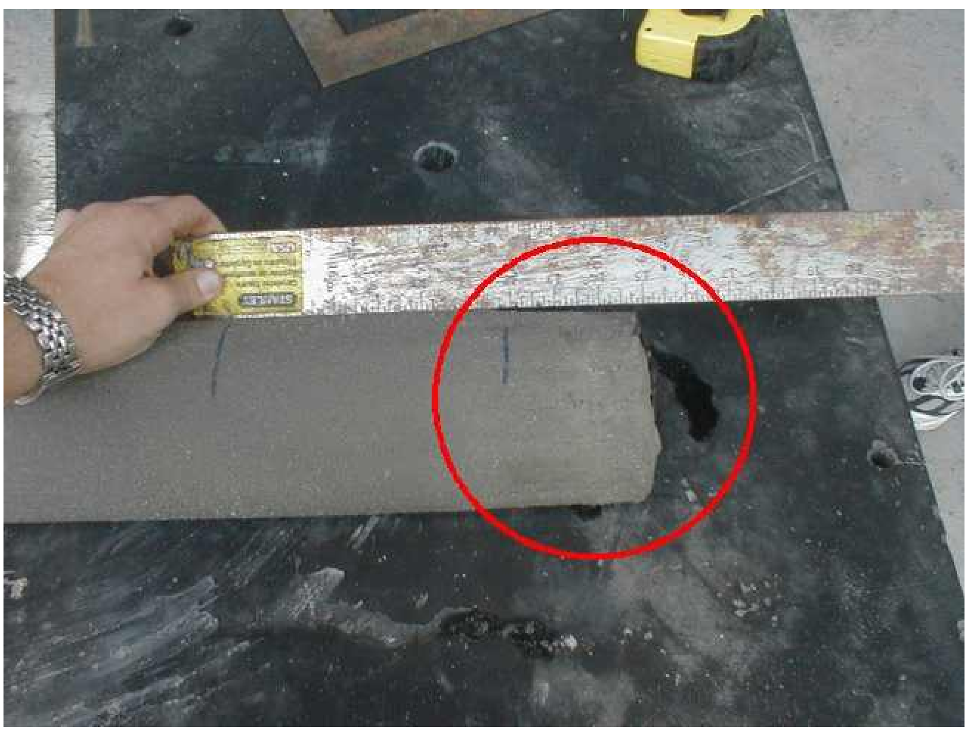

Figure 4-24. Reduced toe cross-section due to poor excavation on Shaft S40

Figure 4-25. Surface Cracking in Upper Portion of Shaft Specimen

Figure 4-26. Porous surface noted during rapid casing extraction on Shaft S41

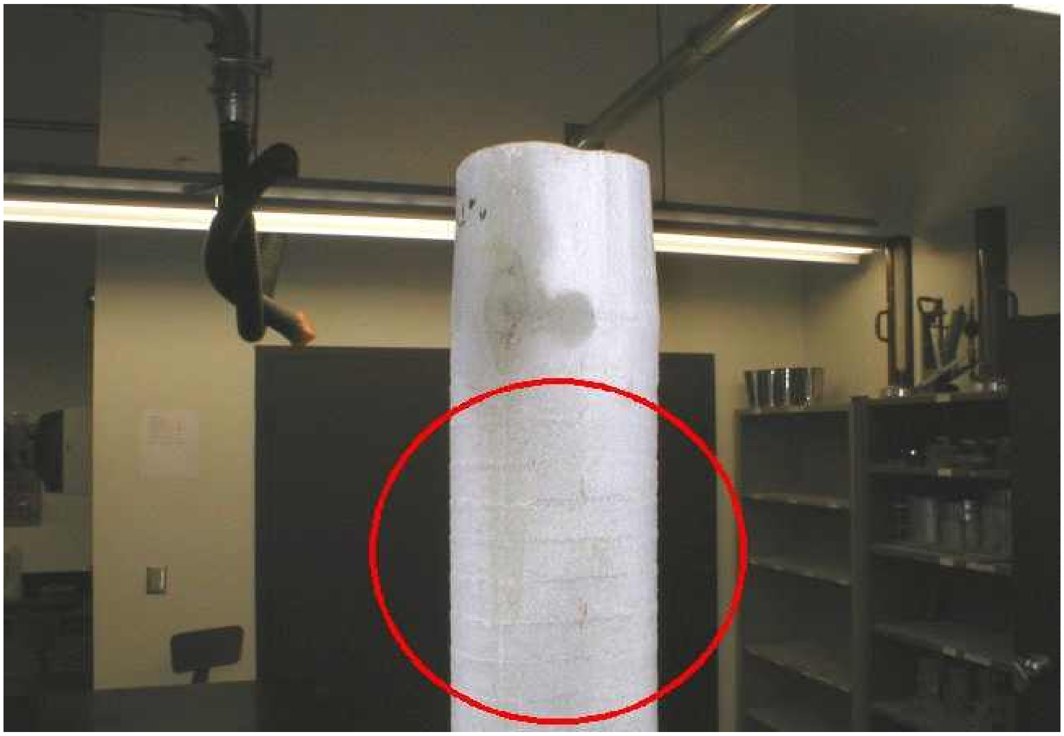

Figure 4-27. Surface markings due to casing wobble during extraction on Shaft S32

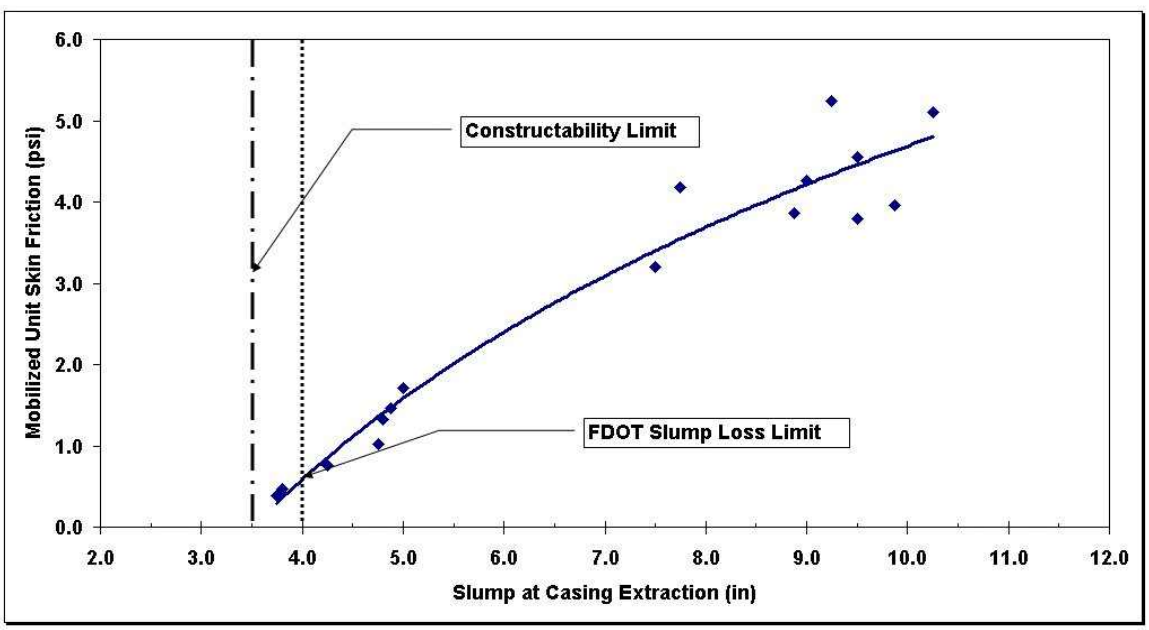

Figure 4-28. Mobilized Unit Skin Friction as a Function of Slump at Casing Extraction

Figure 4-29. Entire Shaft Lodged in Casing Due to Excessive Slump Loss on Shaft S47

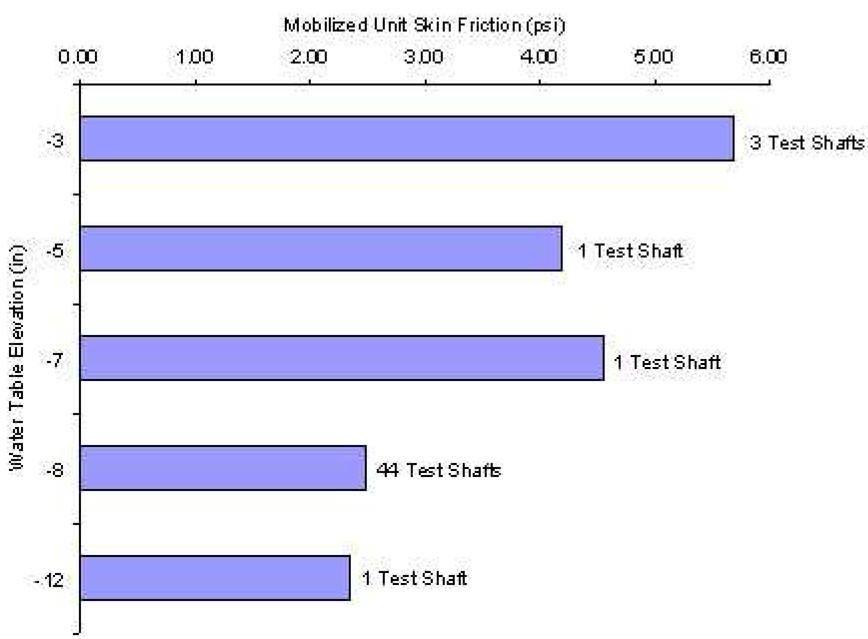

Figure 4-30. Side Shear versus Water Table Elevation

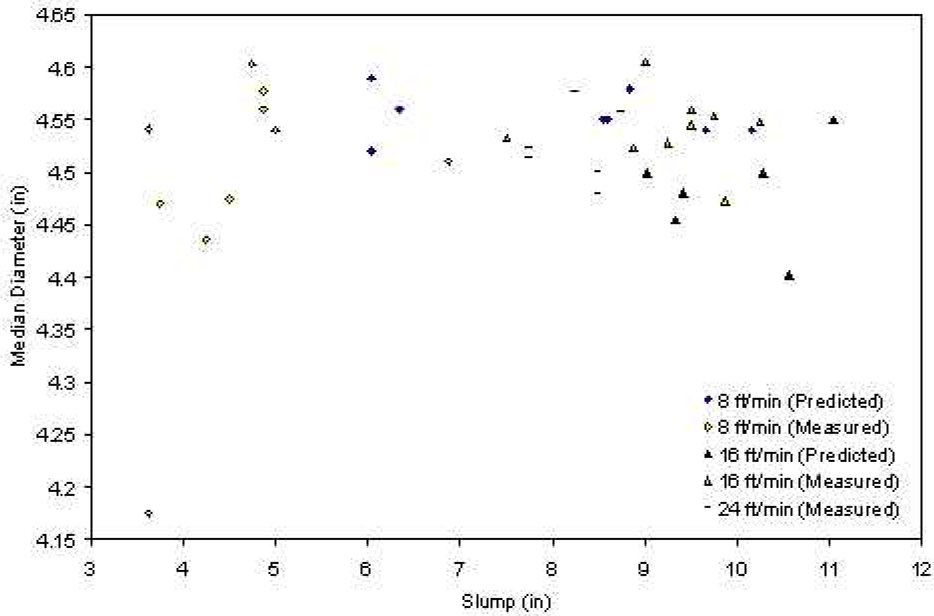

Figure 4-31. Shaft Diameter versus Slump (Measured and Predicted from Mini-slump)

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5. SLURRY / SAND FALLOUT TESTING

5.1 Sand Fallout Background

The permissible standard sand content widely used in state specifications for slurry mixes is 4% at the time of concrete placement. For example, a typical 4 foot diameter, 60 foot long drilled shaft filled with a 4% sand content slurry, contains approximately 30 ft^3 of sand in suspension. If the slurry is incapable of suspending the sand, fallout will occur. Sand fallout during a large drilled shaft pour which takes several hours will accumulate on top of the advancing concrete which has the potential of creating anomalies within the final shaft.

In order to test sand fallout in a drilled shaft borehole supported by slurry, a large scale apparatus was conceived and developed. The "concrete pour simulator" device was designed to simulate tremie placed concrete in drilled shafts (Figure 5-1). As such, variable concrete placement rates and slurry standing times could be addressed.

5.2 Concrete Pour Simulator

It was critical that the device have enough height to adequately simulate drilled shafts, and also a diameter that would not limit the applicability of the experiment. This unique device was envisioned as a 20 foot tall circular shaft with the ability to hold slurry by means of a sealed bottom plug. Further, it needed to have the ability to raise this bottom plug at different velocities to simulate the rise of concrete in a commercially constructed drilled shaft (similar to a vertical syringe pushing upwards). Also necessary was a tank to store the slurry while it was not in use, a pump to fill the shaft and circulate the slurry, a drain pipe for slurry to escape from the top of the shaft, and easy accessibility to the top and bottom of the shaft. Each component will be discussed in the following sections.

5.2.1 Simulated Shaft

To simulate the shaft, a 20 foot long section of 12 inch diameter PVC pipe was used (Figure 5-2). The pipe assembly was mounted to the side of the USF 4MN statnamic hydraulic catching frame. The plumbing fittings were installed at the top and bottom of the shaft by tapping into the side of the pipe. The intake fitting (Figure 5-3) consisted of a cam-lock adapter for easy connection with an inline valve so that the pump could be disconnected after filling the shaft. This plumbing was placed approximately one foot from the bottom of the pipe so the fully inserted plug would not block slurry from being pumped into the shaft. Approximately one foot from the top of the shaft, the drainage plumbing was installed (Figure 5-4). This consisted of a 2 inch PVC pipe draining directly back into the storage tank for slurry re-circulation. This allowed for approximately 18 feet of effective shaft length for the experimental tests.

5.2.2 Simulated Rising Concrete Head

It was necessary to develop a plug that would be relatively easy to pull up and yet have the capability of sealing the pipe such that no slurry could leak out even with the imperfections of the inner diameter of the PVC. It would also need the robustness to sustain the weight of slurry for up to twelve hours at a time during the extended length tests. This sealing device would simulate the rising concrete in a drilled shaft.

The plug was built from 3/4 inch plywood discs turned down to a diameter of 11.5 inches, sandwiching 1/8 inch thick rubber discs of 12.5 inches diameter. Three rubber discs created an adequate seal and did not provide excessive friction for pulling the plug through the pipe. These discs were center drilled and a 3/4 inch threaded rod was used to bolt them together. A shackle was adapted to the end of the rod such that the plug could be pulled through the pipe. When inserted, the rubber would engage the inner walls of the PVC pipe, creating a seal so no slurry could pass through. A consistent way of collecting settled sand was also required. Therefore, a plastic lip was attached to the top of the plug, which allowed for the fallout to be collected and removed easily for measurements. It also ensured that no settled sand would escape through the drainage tube at the end of the pull. Figure 5-5 shows the assembled plug.

5.2.3 Pulling Device

A pulling device was constructed for the plug from a W4x13 section (Figure 5-6) to provide reaction against the weight of the slurry. This device incorporated a hand winch to control the upward velocity and a pulley to center the steel cable connecting the plug. This device was mounted to a cross-member of the statnamic frame. The winch was calibrated by turns per second to control velocities similar to those found in drilled shaft construction conditions (approximately 1 to 4 ft/min). This calibration was found to be very accurate and easy to maintain with the assistance of a stopwatch. Table 5-1 shows the calibrated velocity times per revolution for the hand winch.

Table 5-1. Calibrated Velocity Times

Desired Velocity	Turns/Second	Time of Pull
1 ft/min	1 turn/6 seconds	18 min.
2 ft/min	1 turn/3 seconds	9 min.
4 ft/min	2 turns/3 seconds	4.5 min.

5.2.4 Storage/Mixing System

Instead of drill auger mixing or a slurry gun, an innovative device called a Hootanany (Figure 5-7) was used to introduce dry bentonite into the slurry. The Hootanany uses venturi induced vacuum to draw dry bentonite through a tube and into the top of the device. The suction is created by water or previously mixed slurry, under pressure of approximately 20 psi, that is pumped into the body of the device. The pressure causes a violent mixing of bentonite particles with the water. This device ensures complete mixing of the bentonite powder. After extended use, there was no evidence of agglomeration of dry bentonite in the slurry tank.

The slurry was stored directly below the pipe in a 200 gallon plastic tank (Figure 5-8). This tank was covered between test runs for protection from environmental elements (i.e. rain). This helped ensure that the slurry mix stayed consistent from test to test. Before introduction into the shaft, the slurry was mixed and various tests were run to ensure proper slurry properties. A powerful mixing device was required to reintroduce sand that had settled during storage into the slurry mix. Such a device was designed and fabricated consisting of a ½ horsepower electric motor driving a shaft connected to a 5 blade fan placed deep into the slurry tank (Figure 5-9). This system was very effective in re-circulating the settled sand and suspending the sand in the slurry (Figure 5-10). This system proved to be the most efficient mixing system and allowed for the most consistent slurry properties to be obtained. The pumping of the slurry was accomplished using a helical pump mounted on a FDOT drill rig (Figure 5-11).

5.3 Slurry/Sand Fallout Test Program Overview

The apparent variables that influence sand settlement were the reasons the concrete pour simulator was built. The testing matrix was tailored to simulate the wide variety of conditions that can be encountered in drilled shaft construction. Its aim was to seek how changing each condition would affect the slurry and sand suspension. A construction test matrix was established that targeted the three main parameters: velocity, wait time, and slurry properties.

(1) Upward velocity of rising concrete was a concern for both the field and lab aspects of this study. Since a head differential was observed, it was desired to discover if sand accumulation could be minimized with faster concrete pours (less wait time). Upward concrete surface velocities, based upon collected field data, ranged between 1 and 4 ft/min. The velocity is dictated by the diameter of the drilled shaft and the concrete flow rate. Concrete flow rate was in turn affected by method of pour (e.g. bucket or pump).

(2) The wait time of a slurry supported borehole is defined as the time taken between the completion of drilling and concreting of the shaft. It is obviously ideal to pour the concrete directly after the drilling and de-sanding is complete, however, it is entirely

possible for the concrete to arrive later for a variety of logistical reasons. It is also common for the contractor to separate drilling and concreting to different days. If this is the case, it is possible to have slurry supported excavations left open overnight.

The most obvious effect of wait time is the settlement of suspended sand in the slurry mixture. An aim of this study was to survey a wide array of wait times in order to examine the settlement behavior. Wait times up to twelve hours were tested.

(3) Also, it was desired to see how different slurry properties would affect the sand accumulation phenomenon. It was originally assumed that altering the volumetric sand content independently of slurry viscosity and density would be adequate from a research standpoint. However, after initial tests and the literature review, it was found that these three primary characteristics of slurry (density, viscosity, and sand content) are quite dependent on each other.

If correlation between slurry properties and sand settlement could be found, it would be easy to establish criteria for contractors to use that dictates if the need to clean out the borehole before concreting exists.

Figure 5-12 gives a representation of the original testing matrix. Sand contents tested were 1%, 2%, 4% and 8%. This range would give researchers adequate information outside the parameters of the current FDOT specifications.

5.4 Sand Fallout Testing Procedure

In order to ensure the consistency of test results, a standard procedure was established to use the concrete pour simulator. The first step in the process was to ensure the quality of the slurry. Standard density, Marsh cone viscosity, and sand content tests were run to ensure desired slurry properties. The plug was then inserted to the bottom of the shaft and the shaft was filled with slurry shortly after final mixing. Once the shaft was filled, the pump continued to circulate slurry for approximately 5 minutes. The wait time began as soon as this mixing ceased and the slurry in the shaft was isolated. After the wait period, the plug was pulled up by turning the winch at the appropriate velocity (Table 5-1) and sand accumulation on the surface of the plug was collected at the top of the column. The accumulation often contains some amount of bentonite, so the collected material was passed through a No. 200 sieve and placed in an oven overnight to dry. The dried material was weighed and counted as sand accumulation.

5.5 Fallout Testing Results

The sand fallout testing was run in geometrically progressive steps from 1% to 8% sand content. The following sections discuss the results from each sand content testing.

5.5.1 1% Sand Content

The first full battery of tests were run with 1% sand content slurry with properties approximately in the mid-range of the FDOT specifications. Figure 5-13 shows the accumulation of sand for the array of velocities and wait times. As can be seen, the scatter data does not suggest any logical trends. This series of data prompted researchers to revise the data collection technique. It was assumed that some accumulated sand was exiting through the drainage pipe at the end of the pull or collecting in the rubber seals. A plastic lip was devised to ensure sand fallout would be caught more reproducibly. Unfortunately, further testing with the new plug design did not display any trends.

5.5.2 2% Sand Content

The first series of tests used slurry properties similar to those in the 1% sand content tests. Once again, the results of these tests were unintelligible. The bentonite content of the slurry was raised, which subsequently led to the increase of density and viscosity to the upper end of the FDOT specifications. This caused an apparent difference in the sand accumulation. Various 2% tests are documented in Figure 5-14. As can be seen, the higher viscosity, approximately 40 second Marsh Cone, creates a slurry mix where much less fallout was obtained as opposed to the tests run with 34 second Marsh Cone viscosities. This prompted a continuance with high viscosity slurry. Figure 5-15 shows the accumulation results of the new testing. The data with thoroughly mixed slurry indicates accumulation increases with wait time.

5.5.3 4% Sand Content

The accumulation testing was continued on slurry with similar properties from the final 2% testing. The density of this slurry was slightly increased due to the extra weight of the sand. A similar pattern of accumulation with respect to wait time was expected. However, Figure 5-16 suggests that wait time had no substantial effects on accumulation. This trend can be explained by the gelling behavior of bentonite slurry. After a certain time period (1-2 hours), slurry particles bond due to opposing charges which is able to suspend sand indefinitely (Reese and Tucker, 1985).

5.5.4 8% Sand Content

In an effort to examine beyond the FDOT specifications, sand content was increased to 8%. The slurry properties were maintained with the exception of density which was again slightly increased due to additional weight of sand. In an effort to catch all of the accumulated sand, a deeper lip was constructed and mounted to the top of the plug. This deemed useful as the amount of sand would have overflown the original lip. Figure 5-17 shows accumulation that is roughly double of the 4% sand content tests. It is also apparent that accumulation increases slightly with wait times, however it is fairly insignificant compared to immediate accumulation.

5.6 Sieve Analysis of Fallout Sand

A sieve analysis was conducted on the dried accumulated material to determine the pattern of grain sizes which settled out. It was assumed that coarse grains would fall out quickly, followed by finer grains over the period of the higher wait time tests. Figure 5-18 displays the results of the sieve analyses for the 4% sand content tests compared against the material added to the slurry. It is apparent that increasing wait time facilitates increased fallout of almost all grain sizes. For No. 16 (1.18 mm) and larger, virtually all grains settled out after 4 hours. For grains as small as No. 100 (0.150 mm), increasing wait time increases fallout up to about 4 hours. It can be seen that for materials as fine as No. 200 (0.075 mm), minimal increase occurs which suggests fines may remain suspended indefinitely.

Figure 5-19 shows the weight retained of each grain size versus sieve opening for the 4% sand content tests. Also included is the grain size analysis of the total amount of sand in the column. This graph gives an indication of the fallout particles relative to the total sand in the slurry column. It is easy to see that the coarser grained particles falling out in a higher proportion to the finer material.

Figure 5-20 shows all the 1, 2, 4, and 8% tests combined. A clear trend of increased fallout (in lbs) is observed with increased sand content.

5.7 Effect of Sand Accumulation

The graphs of the accumulated material mentioned earlier in this chapter use oven-dried weights of accumulation to graph against wait time. This was done to maintain consistency of results, but this data requires some regression to estimate fallout in commercially constructed drilled shafts. Since sand is suspended in a slurry solution, the resulting relative density would be very low. Hence, a bulk density was estimated using the weight of dry accumulation and the average depth of accumulation in the plug.

Using the calibrated bulk density, Figure 5-21 shows the sand fallout as a percentage of total sand in the slurry column. It is apparent that as sand content increased in the slurry, the percentage of fallout did as well. It can also be noted that for sand contents of 4% or higher, wait time becames a smaller factor. This suggests that slurry mixed within FDOT specifications will settle out most material within 2 hours.

Figures 5-22 through 5-24 show predicted volume and height of sand fallout for drilled shafts with various diameters and depths at the FDOT maximum limit of 4% sand content. Sand accumulation can clearly be seen to have a greater effect on deeper shafts. Paired with the effects of tremie-placed concrete desegregating immediately after charging, this accumulation can have a substantial effect on the formation of toe inclusions.


Figure 5-1. Concrete Pour Simulator


Figure 5-2. Simulated Shaft


Figure 5-3. Intake Fitting


Figure 5-4. Drainage Plumbing

Figure 5-5. Sealing Device (Simulated top of Concrete)

Figure 5-6. Pulling Device


Figure 5-7. Hootonany Mixer


Figure 5-8. Slurry Tank


Figure 5-9. Slurry Mixing Device


Figure 5-10. Mixing Slurry with Device

Figure 5-11. FDOT Drill Rig Pump

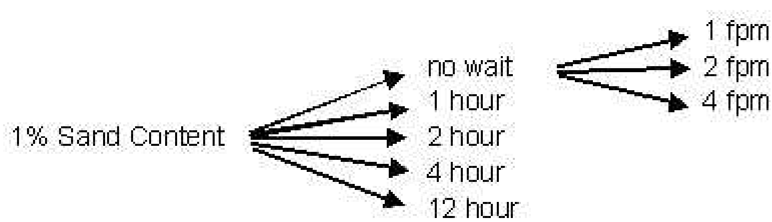

Figure 5-12. Text Matrix Flowchart

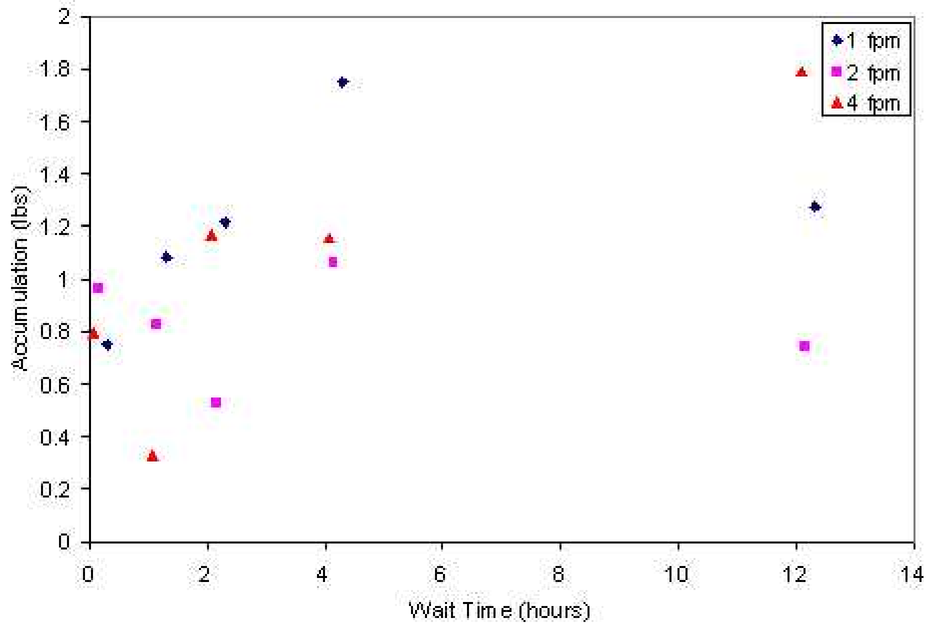

Figure 5-13. Accumulation for 1% Sand Content Tests

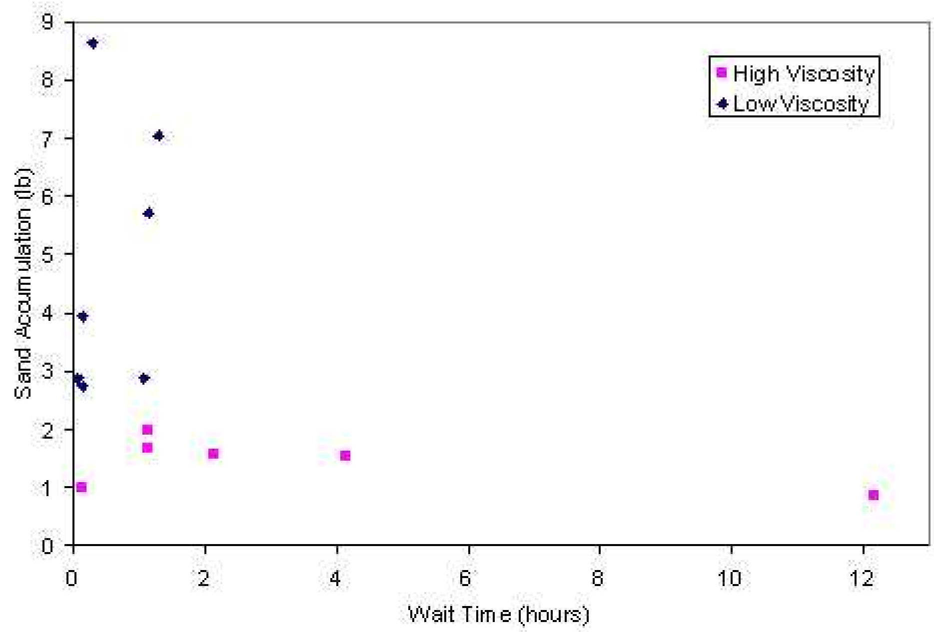

Figure 5-14. Accumulation for Initial 2% Sand Content Tests

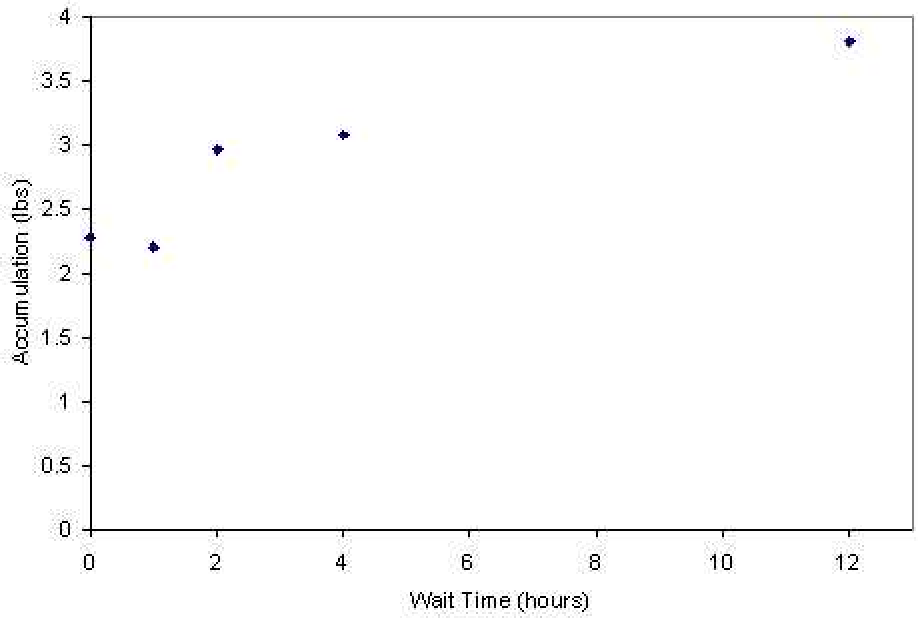

Figure 5-15. Accumulation for Refined 2% Sand Content Tests

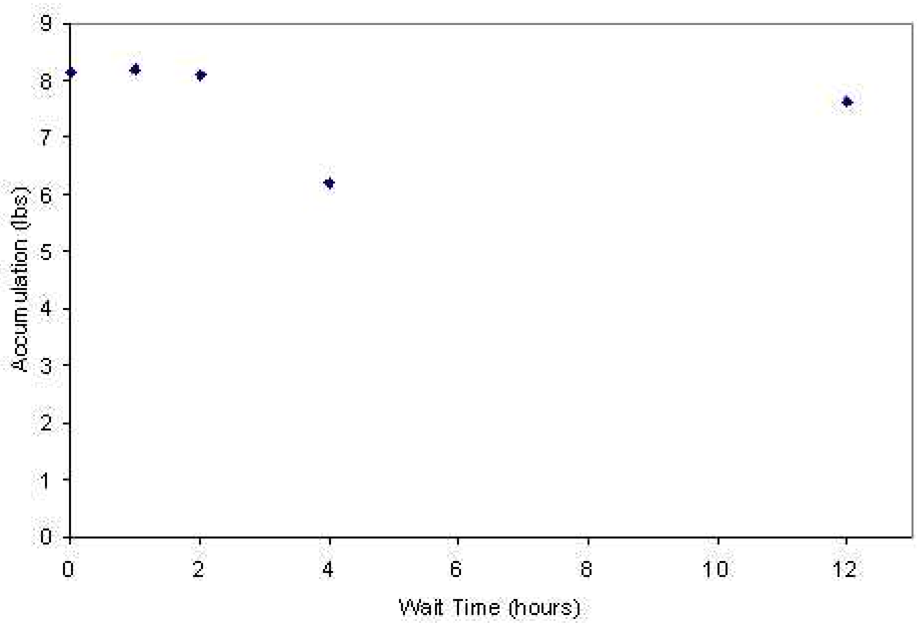

Figure 5-16. Accumulation for 4% Sand Content Tests

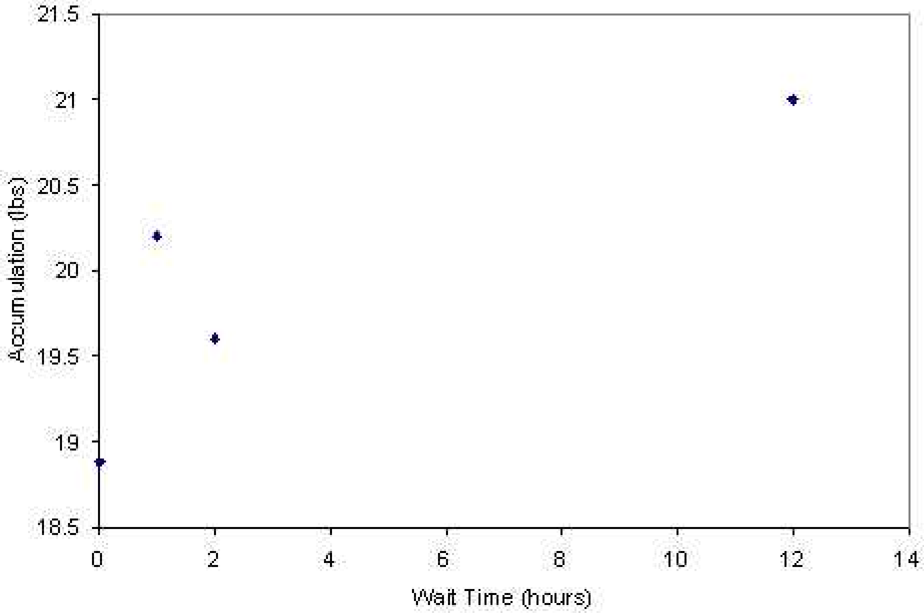

Figure 5-17. Accumulation for 8% Sand Content Tests

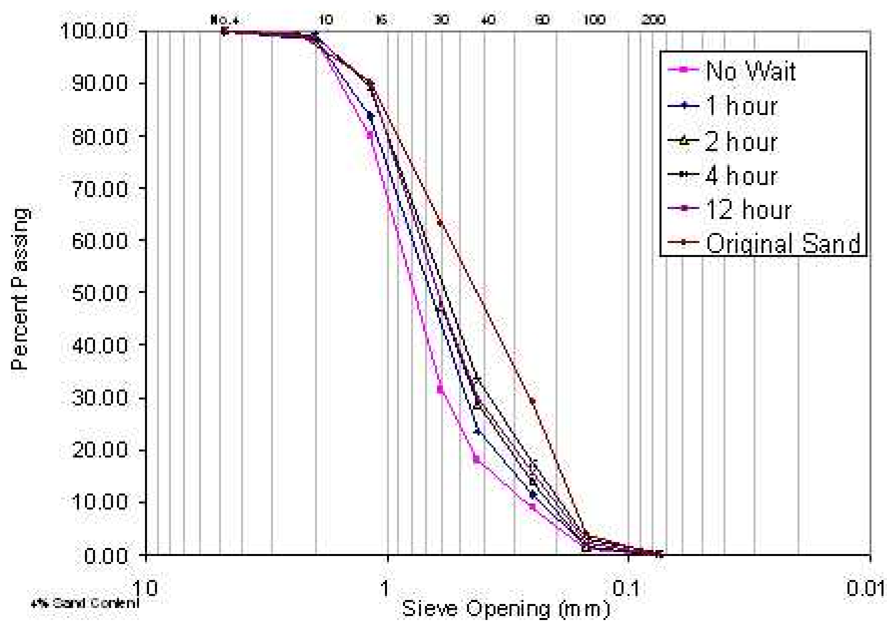

Figure 5-18. Sieve Analysis for 4% Sand Content Accumulation and Pit Sand

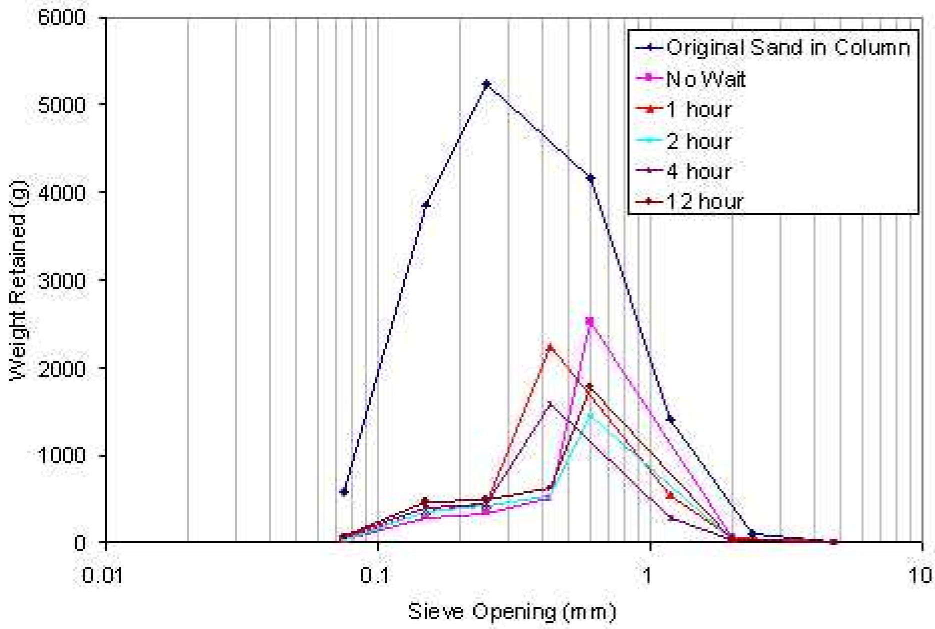

Figure 5-19. Weight Retained versus Sieve Opening for 4% Sand Content

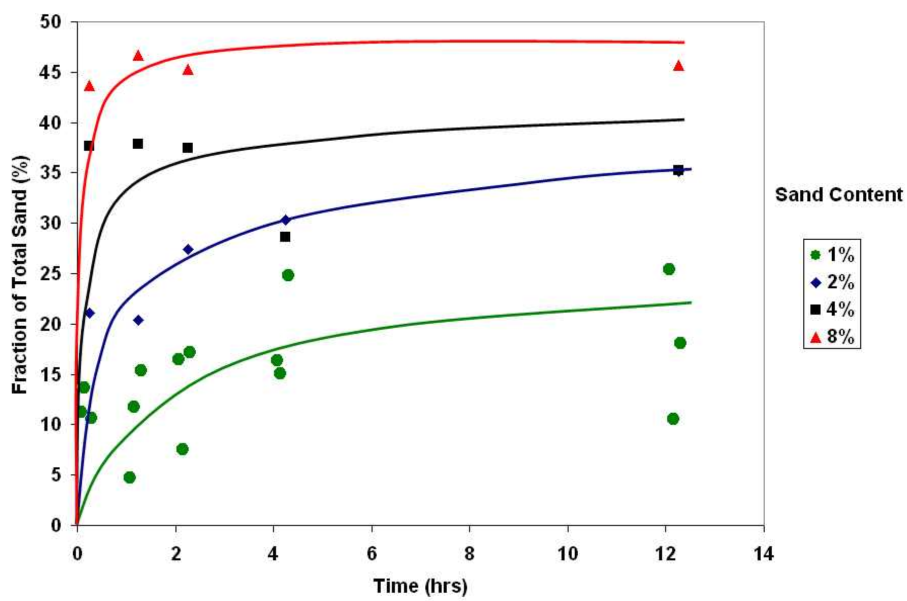

Figure 5-20. Sand Fallout as Percentage of Total Sand in Column

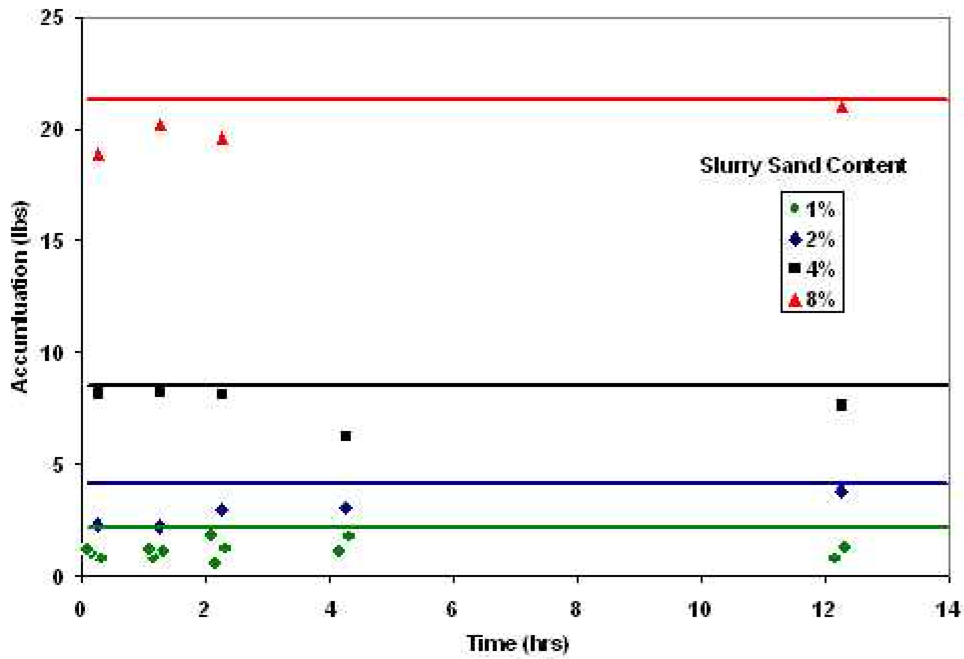

Figure 5-21. Summary of Sand Accumulation of All Sand Contents

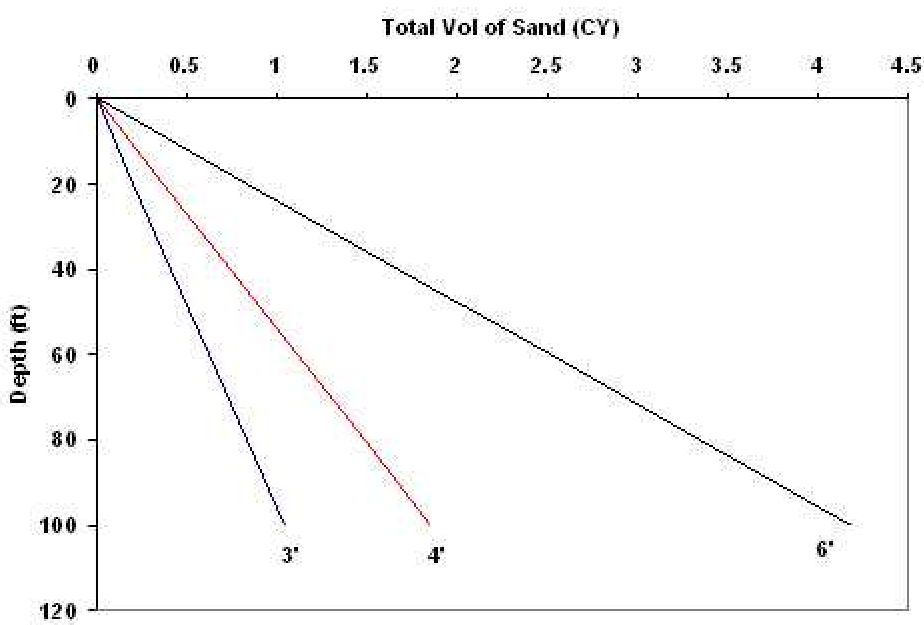

Figure 5-22. Total Volume of Sand for Various Diameter Drilled Shafts at 4% Sand Content

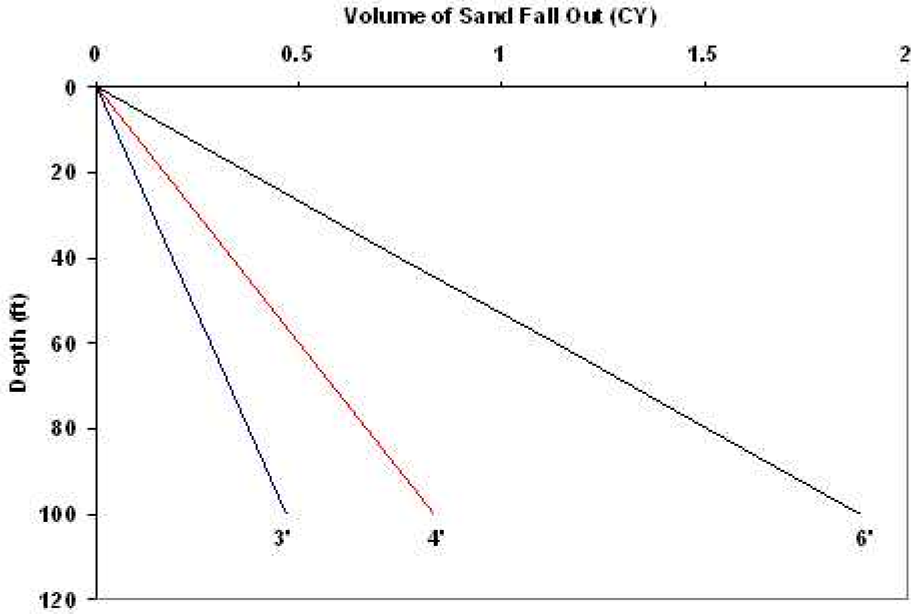

Figure 5-23. Volume of Fallout for Various Diameter Drilled Shafts at 4% Sand Content

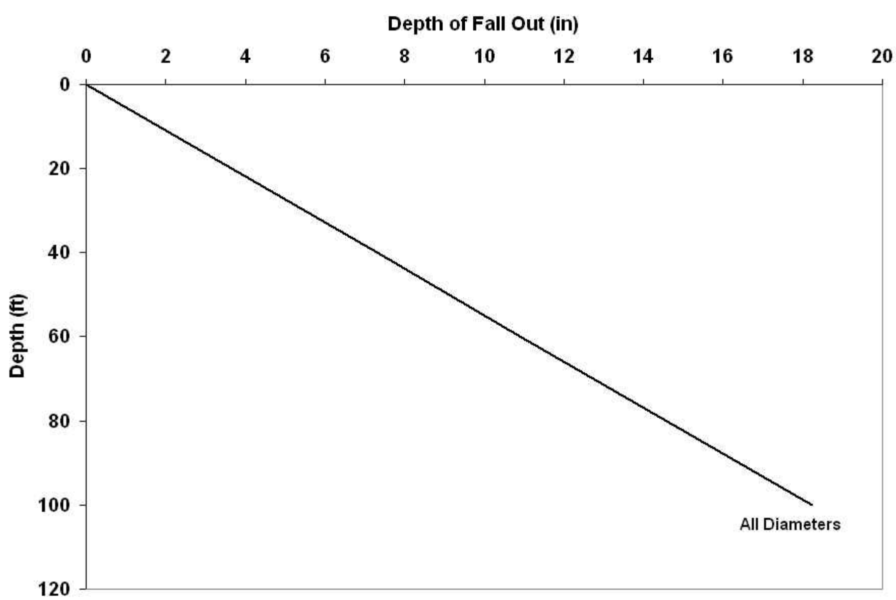

Figure 5-24. Depth of Fallout versus Depth of Drilled Shaft for 4% Sand Content

6. NGES FULL SCALE TESTING

6.1 NGES Test Program Overview

In order to verify and add credibility to the findings from the lab study, a full scale study was conducted at Auburn University's National Geotechnical Experimental Site (NGES) in October of 2002 (Figure 6-1). This program called for the construction of five different drilled shaft foundations. Parameters that were studied included construction method (dry hole versus wet hole), rebar spacing, aggregate size, and slump. Borehole quality assurance was accomplished using an experimental device known as a borescope (see below).

Four of the shafts had rebar cages outfitted with post-grout cells as can be seen in Figure 6-4 (Dapp, 2002). Additionally, these shafts each have one specific type of anomaly integrated into their design. These anomalies were constructed using sand bags attached directly to the cages, such as shown in Figures 6-5 and 6-6. The remaining shaft (TS-4) had a standard rebar cage and was used as a control shaft. All shafts included CSL access tubes and four toe-mounted axial strain gages (sister bars). The four shafts containing the post-grout cells also included the PVC piping necessary for grouting. These details are highlighted in Figure 6-7. For the completed shafts, quality assurance methods to be compared included shaft integrity testing (SIT), cross-hole sonic logging (CSL), thermal integrity testing, and post grouting.

6.2 NGES Site Details and Layout

The site on which the five test shafts were constructed is located in Opelika, Alabama, and was made available by Dr. Dan Brown of Auburn University. The site had been used for previous geotechnical studies and remnants of said research could be found readily on the property. Using the USF miniature cone penetrometer, a subsurface investigation was carried out prior to any site layout (Figures 6-8 and 6-9). Virgin material was located in the northwest corner of the property, and the area was large enough to contain all five proposed shafts. Cone penetration tests (CPT) were performed at the centerline locations of each test shaft. Figure 6-10 thru Figure 6-14 shows the CPT soundings for TS-1 thru TS-5, respectively. The soil in this area was mainly a reddish-brown clayey silt with some gravel, mostly near the surface. Full documentation of the various geotechnical investigations can be found in Brown and Drew (2000), Mayne, et al. (2000), and Brown and Vinson (1998).

Each shaft had a outer diameter of 42 inches, and the total length was 24 feet. The reinforcement cages for each had an outer diameter of 31 inches and length of 25 feet. This allowed for 12 inches of rebar stick-up in each of the finished shafts, which would make future extraction of the shafts easier. The shafts were laid out as shown in Figure 6-15. Two rows of shafts were excavated, with the inside-to-inside spacing equal to 17.5 feet, which is approximately 5 shaft diameters. This same spacing was also used between the two rows of shafts. This was done to ensure that each shaft's radial zone of influence did not interfere with that of adjacent shafts. Five shaft diameters is equal to twice the value recommended in recent literature to minimize group effects in drilled shafts (O'Neill, 1981; Poulos, 1980).

6.3 Rebar Cage Preparation

The contractor delivered the five rebar cages to the site already assembled according to specifications by Dr. Brown. These cages had a outer diameter of 31 inches and a length of 25 feet. Longitudinal steel consisted of two DYWIDAG bars, 16 #9 bars, and 75 #4 shear hoops spaced at 4 inches on edge (clear spacing). One of the cages (TS-5) was observed to have a tighter shear steel spacing, measured to be 3.2 inches on edge. This cage had 93 hoops.

Sister bars were installed at the toe on each cage (four per cage). These were configured as full bridge devices, and were placed at quarter points around the circumference of each cage. The wiring for the gages was collected in a single harness that was secured to the shear stirrups along the length of each cage, and a service loop was left at the top for future access.

Every cage was fitted with steel CSL access tubes installed at third points along the circumference. These tubes were sealed at the bottom to prevent concrete from entering them, which would render them unusable, however they were not sealed well enough to prevent grout from entering them during the post grouting procedure.

Four of the five cages had post grout cells installed at the toe. The type of cell used on these shafts is known as a "flat jack" system (Dapp, 2002). PVC grout tubes are installed along the length of the cage and mate with ports installed on the steel plate of the cell. Also for four of the five cages, prefabricated anomalies were incorporated. This was done by attaching sand bags full of local material to predetermined locations on each cage. Location TS-1 contained a cluster of bags at the toe, resting atop the post grout cell within the rebar cage. Location TS-2 had an annular neck centered on a CSL tube and located approximately 3/4 the shaft length down from the top. This neck encompassed an area on the cage measuring 18 inches in depth by 34 inches along the circumference. Location TS-3 had an annular neck installed at mid-height on the cage. This anomaly measured 23 inches in depth by 56 inches along the circumference. Location TS-5 contained an anomaly similar to that of TS-2, however this one was placed between CSL access tubes and approximately 1/4 the shaft length down from the top of shaft. The area it encompassed measured 18 inches in depth by 31 inches along the circumference. Only the cage placed in location TS-4 was left without a post grout cell. This cage also contained no prefabricated anomaly.

6.4 Drilled Shaft Excavation

The excavation of the five shafts was completed in one afternoon and each shaft was poured on that day as well. A truck-mounted, diesel drill rig equipped with a double-flight rotary auger with stinger was supplied by the contractor as shown in Figure 6-16.

Each of the five shafts was excavated in dry conditions with no casing. This was appropriate because the soil was non-caving and the water table was below the proposed bottom elevation of the borehole (Figure 6-17). Because of this, drilling proceeded quickly and each excavation was finished in approximately 30 minutes. Also, the clean-out bucket was not used because there was little loose material left at the bottom of the excavation once auger drilling was complete. This was verified using the borescope.

As noted, the water table was below the bottom elevation of each excavation, so the holes remained relatively dry. It was desired to construct the control shaft using tremie placed concrete poured under water. Therefore, the excavation denoted TS-4 was filled with water prior to placing the rebar cage. This was performed using water stored in a tank onsite. After the hole was filled with water, the borescope was again used to verify the final quality of the excavation before placement of the rebar cage.

6.5 Mix Specifications

Several different concrete mix designs were used in this study. The first two shafts poured used a standard portland cement concrete with #57 angular stone (maximum aggregate size of 1 inch). These two mixes were identical except for the slump. One of the mixes was required to have a slump of 8 to 9 inches, while the other needed a slump of 5 to 7 inches. FDOT recommends a slump of 7 to 9 inches for all drilled shafts constructed using the tremie placed method, so this governed the selection of slumps for the test mixes.

One shaft was constructed using a standard Portland cement concrete and #7 river rock. This particular aggregate is rounded and has a maximum aggregate size of ½ inch. Number 7 stone is not typically used on FDOT drilled shaft projects. However, it was anticipated that this mix design should perform much better than the #57 stone mix in terms of tremie placement and flowability.

The remaining two shafts, one of which was the control shaft constructed under water, used a mix known as a "Self Compacting Concrete" (SCC). This mix design is a very fluid, high slump (>10 inches) material that is designed to exhibit very favorable flow characteristics even in the presence of tight rebar spacing. Because of this, a different assessment of workability is normally used with a SCC mix. The apparatus, pictured in Figures 6-18 and 6-19, measures the flow of the material through a series of rebar sections. Detailed information regarding this method of testing can be found in literature by Hodgsen (2003) at Auburn University.

SCC mixes generally have higher cementitious contents and higher fines, with lower water to cement ratios and high dosages of high range water reducers (W.R. Grace, 2003). The first SCC mix was modified at the site by the addition of Glenium 3000NS to reach the required slump. This delayed addition of the high range water reducer could be responsible for different characteristics between the mixes for TS-4 and TS-5. Of other significance is the high water to cementitious ratio. Generally SCC mix have lower water to cementitious ratios, 0.32 to 0.38, to avoid excess bleed water and segregation which can result from the high dosages of high range water reducers. Additional information regarding SCC mix design can be found in literature from the RILEM Technical Committee (Petersson et al., 2000). Table 6-1 details the 5 mixes used for the Auburn test shaft series.

Table 6-1. Detailed Summary of Concrete Mixes

Material	TS-1 & 3	TS-2	TS-4 & 5
Cement Type I	560 lbs/cy	560 lbs/cy	420 lbs/cy
Fly Ash Class F	140 lbs/cy	140 lbs/cy	-
Fly Ash Class C	-	-	227 lbs/cy
GGBF Slag	-	-	97 lbs/cy
Sand	1075 lbs/cy	1075 lbs/cy	1595 lbs/cy
#57 Limestone	1900 lbs/cy	-	-
#7 Pea Gravel	-	1900 lbs/cy	1231 lbs/cy
Water	34 gals/cy	34 gals/cy	40 gals/cy
Micro Air (Air Ent.)	4.5 oz/cwt	4.5 oz/cwt	-
Delvo (HRWR)	40 oz	40 oz	-
Glenium 3000NS	-	-	4.0 oz/cwt
Water / Cementitious Ratio	0.4	0.4	0.45

6.6 Borescope Video Interpretation

The borescope is a new type of downhole camera and video recording system made available by FDOT for this project. It consists of a camera bell, shown in Figure 6-2, and a video monitor and recorder, shown in Figure 6-3. The device allows the flow of concrete deep within the borehole and the integrity of the borehole itself to be monitored visually, and the video recorded for archiving and future reference.

Extensive video footage was collected during pours for all shafts except the control shaft, which was constructed under water where murky conditions hampered visibility. Additionally, weighted-tape measurements during the pour were taken both inside and outside the cage, allowing for the monitoring of any head differential that existed during the placement of the concrete. The head differential readings were taken 180 degrees apart to account for the off-centered location of the tremie due to the inclusion of the borescope.

Shaft TS-1. As with all the following shafts, borescope video of the bottom of the excavation just prior to pouring shows some minor inflow of water and a muddy bottom. Overall the quality of the excavation was acceptable, as can be seen in Figures 6-20 and 6-21. The borehole was stable and the toe was reasonably clean. The toe-mounted sandbags just above the post grout cell are visible in Figure 6-22.

The borescope video quality for this first shaft was poor, but still useful. It is apparent that the large aggregate size (up to 1 inch diameter) does have an adverse effect on the flowability of the material in the hole. Figures 6-23 through 6-25 show some of the detail of the flow that occurred during this pour. The material leaving the tremie tends to build up on the inside of the cage. Along the tremie, the movement of the concrete is slowed by the friction that develops. Along the inside of the rebar cage, concrete flow is again impeded by friction, and radial flow is impeded due to the low CSD ratio of 4. These conditions manifest themselves in the arched surface that is visible in the video. Meanwhile, the material on the outside of the cage is maintained at a much lower level. Evident in Figures 6-24 and 6-25, the concrete inside the cage simply builds up until some critical differential is reached, and then it sloughs to the outside of the cage. Larger particles remain on the inside of the cage particularly concentrated around rebar, and more of the water and fines flow through the reinforcement. This action is observed even though the material is poured at the high end of the FDOT slump guidelines, again reinforcing the notion that CSD has much more of an impact on flow in the borehole than does slump. The measured head differential reached values of 6 to 10 inches during this pour.

Shaft TS-2. For this cage and mix configuration the CSD increased to 8. The clear spacing did not change, but the smaller maximum aggregate size increased the CSD. According to what was observed in the LPC testing program, this alone should result in a much more uniform flow with lower head differentials and higher consistency of the material on the inside and outside of the cage.

From the early stages of this pour, as shown in Figure 6-26, the material flowed in a more consistent manner with little visible segregation of the material within the inner cage area. Further observation of the video footage shows that the flow of material from the tremie into the borehole is markedly different than that observed with the #57 stone mixes. Instead of the material arching up between the tremie wall and the inside of the cage, and then sloughing to the outer cage area, the material inside the cage rises in a much more uniform manner across its surface. This can be seen in Figures 6-27 through 6-29. This was characteristic of the entire pour for this shaft, and as expected, much lower differentials of only 3 to 5 inches were recorded.

Shaft TS-3. For this cage and mix configuration, the CSD was again about 4. Overall, the flow characteristics evident in the borescope video are similar to those seen in the TS-1 pour and are consistent with expectations derived from LPC testing in the lab. The decrease in slump, however, adds to the uncertainty already present in the flow.

Shown in Figure 6-30 is footage from the early stages of this pour. In the circle on the lower right, the material is apparently very dry and the coarse aggregate concentration in this region is large. However, on the other side of the tremie (top left circle) there is more water and fines in roughly the same area indicated. This demonstrates that at low slump, this material displays differential flow even within the inner cage area, in addition to that which exists from inner to outer cage.

In Figure 6-31 the material highlighted with the circle is more liquid and is about 1 to 2 inches lower than the material on the other side of the tremie (as estimated from the visible shear stirrups). Further, no material is visible on the outside of the cage, indicating a significant head differential. In fact, the measured head differential reached 12 to 16 inches during the early stages of construction, and by the end of pour, a differential of 6 inches was still observed. Figure 6-32 shows again, with greater detail, the binding of coarser material between the tremie and the inner cage area. The material appears to be very dry, as most of the water has segregated and flowed to the outer cage area.

Shaft TS-5. The increased number of stirrups and maximum aggregate size produced a CSD of 6.5. The flow behavior of this material, however, could not necessarily be predicted from LPC test results since the properties of the mix were different than what was used in the lab. If a projection of the flow had to be made based solely on CSD and slump (>10 inches for SCC), then again a much more uniform flow with lower head differentials and higher consistency of the material on the inside and outside of the cage should be expected.

Beginning with Figure 6-33, it can be seen that the SCC flow characteristics are similar to what was observed with the #7 stone mix, although it appears this mix has a higher water content. The surface of the flowing concrete, as shown in Figures 6-34 and 6-35, does maintain a rather uniform consistency with no obvious particle segregation. There does not appear to be significant binding of the aggregate on the rebar cage, and although some excess water is visible on the surface of the mix, it was not observed bleeding from the material during the pour.

Perhaps even more impressive was the very low head differential that was maintained during the pour. Although difficult to see in the still photograph of Figure 6-36, the level of the concrete on the inside and on the outside of the cage is nearly identical. There is no obvious arching of the inner cage material, and sloughing to the outside of the cage was not recorded. Instead, the flow was very much radial in direction, with only a 2 inch differential being measured at the early stages of the pour. This differential disappeared completely by the end of the pour.

It would have been interesting to visually record the second SCC shaft poured (TS-4), because the measured head differential for this shaft was 8 to 10 inches at the start of pour, and 6 inches toward the end. Comparing these results to what was observed in TS-5, the addition of a fluid pressure in the borehole has obvious negative implications on the flow characteristics of the system. It should be noted that water in the borehole, with a unit weight of only 62.4 pcf, has the lowest unit weight of the possible drilling fluids that could be present. Therefore, as slurries of increasing unit weight are introduced, and as CSD decreases, much larger head differentials are possible, and have been observed. The possible magnitude of these differentials is on the order of several feet (Brown and Camp, 2002) as opposed to the 2 to 16 inches observed in the Auburn testing (Table 6-2).

Table 6-2. Summary of Differing Shaft Parameters and Associated Head Differentials

Shaft I.D.	Excavation	Reinforcement	Anomaly Location (24 ft shaft)	Concrete Mix	Head Differential (in)	
					Start of Pour	End of Pour
TS-1	Dry Hole	4" stirrup spacing with CSD of 4	Toe	#57 stone with 8.5" slump	8 to 12	4 to 6
TS-2	Dry Hole	4" stirrup spacing with CSD of 8	3/4 point from top	#7 stone with 8.75" slump	3 to 5	0
TS-3	Dry Hole	4" stirrup spacing with CSD of 4	Midpoint	#57 stone with 5.5" slump	12 to 16	6
TS-4	Wet Hole	4" stirrup spacing with CSD of 8	None	SCC with 24.5" slump flow	8 to10	4 to 6
TS-5	Dry Hole	3.25" stirrup spacing with CSD of 6.5	1/4 point from top	SCC with 24" slump flow	2	0

6.7 Quality Assurance

The deliberate placement of anomalous regions within the shafts offered an opportunity to compare different non-destructive quality assurance methods. Cross-hole Sonic Logging and Thermal Integrity Testing were performed on each shaft shortly following construction.

Sonic Integrity Testing was also performed on each shaft during the curing of the shaft. Once fully cured, the four shafts containing grout cells were tip-grouted while monitoring both the grout pressure and upward displacement.

6.7.1 Cross-hole Sonic Logging

Cross-hole Sonic Logging readings for each shaft were taken at 12 hour intervals for two days following shaft construction. The wave speed as a function of depth from the initial reading to the final cured reading of TS-3 (tubes A-B) can be seen in Figure 6-37. The signal does not show significant change from these readings. The results from TS-1 (Figure 6-38) show a decreasing velocity from 23 to 24 feet. This indicates a less dense material within this region (i.e. sand bags). Results from TS-2, TS-3, and TS-5 (Figures 6-39, 6-40, and 6-41, respectively) do not indicate the presence of anomalous regions in the test shafts. This is due to the fact that the included anomalies for these shafts are not arranged inside of the triangulated path between the 3 CSL tubes, but rather attached to the outside of the rebar cage. TS-4 (Figure 6-42) shows no significant change in velocity for all readings, although when excavated, TS-4 had a bullet shaped toe with CSL tubes exposed (Figure 6-43).

6.7.2 Thermal Integrity Testing

Thermal temperature readings were taken in the same fashion as CSL; however, the results proved more promising. Temperature soundings were taken in each of the three access tubes for all 5 shafts roughly every 4-6 hours over a 2 day period. Figure 6-44 shows the layout of TS-2 anomaly placement around the outside reinforcement cage at the third quarter point, and the recorded thermal data for access tube B, at about 24 hours after placement. Despite the very high thermal conductivity of the steel CSL tubes, the thermal signal was repeatable and sensitive on the order of 0.1° F (see below).

Shaft TS-2B. Thermal data taken from shaft TS-2B show the effects that boundary conditions and diffusivity changes of various materials have on the heat flow regime of a drilled shaft. The data show sharply increasing temperature curves at the top of the shaft, with peak temperatures occurring at a depth of 7 to 8 feet. Below this depth temperatures are decreasing somewhat steeply, with a flattening of the curves occurring around the shaft midpoint. At 13 feet deep, a 6 inch pipe coupling induces a temperature increase on all the traces, below which is the sandbag anomaly. Pipe coupling anomalies are identifiable in all shaft traces where a coupling exists. Below this point, the thermal signal decreases steeply as some of the heat flow is diminished due to changing boundary conditions: increasing free water in pore spaces in the capillary fringe (previously noted GWT below tip of shafts >24 feet) and the end condition, requiring outward heat flow with both radial and vertical components.

The anomaly at the third quarter point can be seen in all four traces at approximately 17 to 22 feet, with the coolest temperature trace (outward-facing) showing the greatest anomaly. The base of the anomalous signal is clearly defined by the sharply increasing temperature curve beginning at 20 feet, progressing toward the non-anomalous temperature. This upward temperature change occurs over approximately a 1 foot vertical span in which the purely outward radial heat flow of a cylinder is disrupted, adding a vertical component of heat flow. The placement of the sandbag adjacent to the coupling enhances the heat flow (vertically and radially) through the coupling and around the sandbag. This "heat channeling" effect may raise the temperature above that of the neighboring material, which appears to contradict Fourier's Law $Q = -k\, dT/dx$ (where k is the thermal conductivity), but which is only valid in a constant-k domain.

Sensitivity and Noise. The manufacturer's specification of the infrared thermocouple gives a repeatability error for any device without temperature control (*viz.*, this design) of 1 to 2% of reading value, and for complete ambient temperature control an error of 0.01% of value. This is a large range of values, and an analysis of sampled signal and noise may more accurately place the expected values of error and signal/noise for this design.

Figure 6-45 shows the magnitude spectrum of the outward-facing sensor data from access tube A of shaft TS-3, one sampling cycle after the data in Figure 6-46. Note that this spectral peak at 0 Hz resides in the decade 100-1000 degrees, so that magnitude percentages can be read approximately from the log plot. The plot was generated using 441 data from the trace TS-3A (outward), and was zero-padded to 512 points, and Fourier transformed using the "boxcar" window only. Without any filter windows the boxcar will yield spectra that are factorable into sinc functions, where sinc(x) = sin(x)/x, which produce the lobes seen throughout the data. This effect itself introduces noise to the transformed data.

The data exhibit a 0.5dB amplitude falloff per wavenumber and a total drop of ~15dB for wavenumbers < 30. This spatial frequency (wavenumber = 30) represents a wavelength of 8.8 inches, and it suggests that important information in the data can be restricted statistically to length scales larger than 8.8 inches. Also at this wavenumber, spectral magnitude fall below approximately 1% of the maximum value, the nominal signal error. The data "noise" appears to follow an exponential falloff (shown by the curve, Figure 6-45). The exception to this trend occurs at wavenumber equal to 62 to 65, which was in part masked by the lobular character of the data, but which clearly stands above the curve. These wavenumbers have equivalent wavelengths of 4.06 to 4.26 inches, indicating rebar signal in the data, which is concordant with the placement of circular hoop rebars at nominal 4 inch separations (although some cages had smaller clear spacings).

Figure 6-46 shows four "rebar-filtered" traces of shaft TS-3A and the unfiltered trace. The unfiltered trace shows a strong heat transfer coupling between the circular rebar hoops and the access tube nearby which the data was taken. Although the rebar "signals" are readily apparent in the trace, their magnitude is <0.1% , higher than that of a temperature controlled device, but much less than expected in this device. This signal is sufficiently greater than the combination of all sources of noise, including uncontrolled ambient temperature and those introduced by poor data transformations (i.e., the un-windowed FFT). It should be concluded that the sensitivity and signal/noise ratio have surpassed the predicted values for this design.

6.7.3 Shaft Integrity Testing

The goal of this non-destructive testing was to detect the anomalous regions placed within the test shafts. SIT soundings were taken prior to post grouting, after load testing, and after extraction. These readings were taken to show the additional affects of post grouting, load testing, and the influence of the surrounding soil. Figure 6-47 through 6-51 show the results from the SIT soundings prior to post grouting and after load testing for TS-1 through TS-5, respectively. The results show no significant change in the signal from the control shaft (TS-4) to the shafts with known anomalies. TS-1, TS-2, and TS-3 show a stronger toe reflection after grouting and load testing. Additional information on the effects of post grouting can be found in Mullins and Winters (2004).

6.7.4 Post Grout

Post grouting was performed on four of the five test shafts (TS-1, TS-2, TS-3, and TS-5). Portland type I cement with a w/c ratio of 0.55 was pumped to the tip of each shaft. Figures 6-52 through 6-54 show the grouting process and test setup. The specified upward displacement was set at 2.5% the diameter (approximately 1 inch for the 42 inch diameter shafts). The displacement and grout pressure were monitored during the grouting process with LVDTs and a pressure transducer. The strains at the toe of the test shafts were also measured by four resistive strain gages. The grout pressure versus displacement graphs for TS-1, TS-2, TS-3, and TS-5 are shown in Figures 6-55 through 6-58, respectively. The post grouting results are summarized as follows:

- TS-1 maximum grout pressure of 82 psi and uplift of 0.105 inches.
- TS-2 maximum grout pressure of 88 psi and uplift of 0.074 inches.
- TS-3 maximum grout pressure of 100 psi and uplift of 0.132 inches.
- TS-5 maximum grout pressure of 109 psi and uplift of 0.075 inches.

Figures 6-59 through 6-62 show the side shear results from grouting and downward load testing for each test shaft. In the case for TS-5, only grouting side shear is shown. Post grouting in silty/clayey soils cannot fully verify the side shear capacity, but rather may only be able to proof test the shaft up to the ultimate capacity of the end bearing strata (Mullins and Winters, 2004).

6.8 Statnamic Load Testing

Statnamic load testing was performed on all shafts using a 4 MN statnamic device, equipped with the hydraulic catching mechanism (Figure 6-63). Two load cycles were performed on each shaft; the target displacement was set at 2.5% the shaft diameter. The strain gages located at the toe of each shaft made it possible to separate the applied load into side friction and end bearing components. Side shear results from the first load cycles can be seen in Figure 6-64. The figure also shows the respective slumps for each test shaft. The results show that within this test program the slump does not affect the side shear component. This is also noted in the LPC and FCV testing.

6.9 Core Sample Tests

After all load tests were completed, each shaft was exhumed by excavating the surrounding soil (Figures 6-65) with the intent of a thorough dissection. A 4 inch and 2-2 inch diameter core samples were taken at 1/4, 1/2 and 3/4 points along each shaft (Figure 6-66) to provide specimens for permeability tests. It was ensured that each core was of sufficient depth to provide specimen inside and outside the rebar cage. Also, each shaft was cross-cut using a diamond impregnated steel band (Figure 6-67). This allowed for visual inspection of aggregate distribution.

6.9.1 Permeability

FDOT personnel conducted permeability tests on cores without reinforcements from all of the shafts using the Rapid Chloride Ion Penetration in accordance with ASTM C1202. Results from the permeability testing can be seen in Figure 6-68. The findings suggest that there is a negligible difference in the hydraulic conductivity between the inside and outside of the rebar cage for shafts TS-1 through TS-3. However, a large disparity exists between the inside and outside of the rebar cage for both SCC test shafts, TS-4 and TS-5.

6.9.2 Aggregate Distribution

The difference in coarse aggregate shape is easily identified when studying Figures 6-69 through 6-74. The angular #57 stone used in shafts TS-1 and TS-3 contrasts with the rounded, river rock (#7 stone) used in shafts TS-2, TS-4, and TS-5. This difference in shape influenced the flow characteristics of each concrete mix. Figures 6-75 through 6-77 show saw cuts of the test shafts. From these cross-sections and the cores, no aggregate segregation between the inside and outside of the rebar cage is discernable.

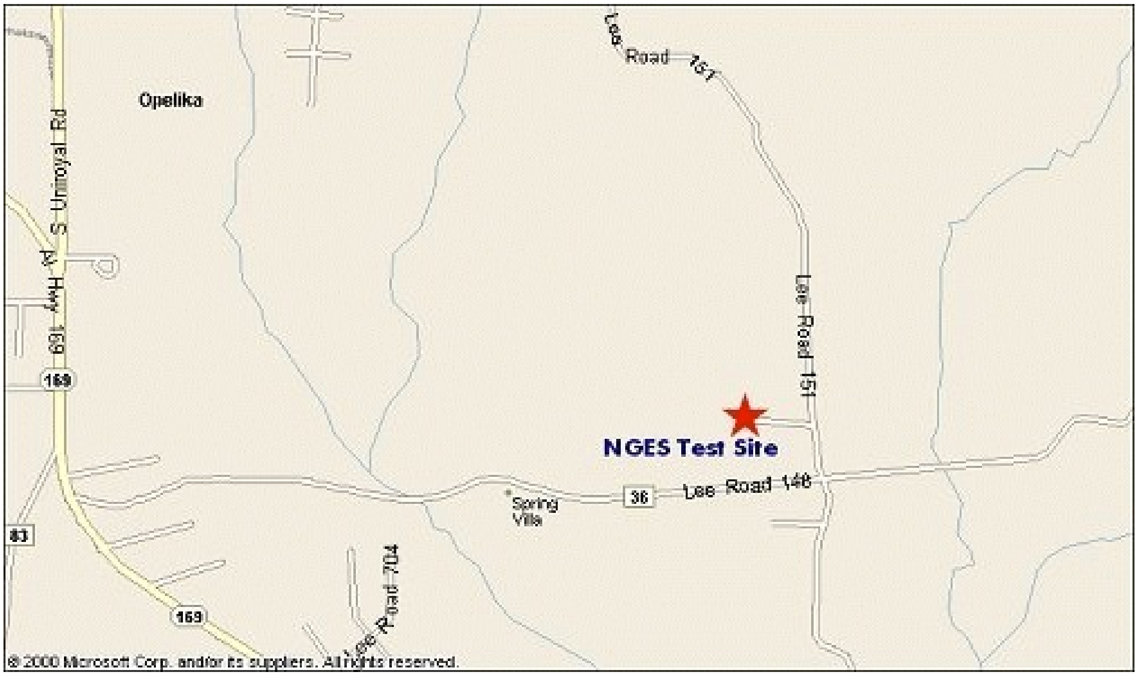

Figure 6-1. Location of Auburn University's NGES Test Site in Opelika, Alabama

Figure 6-2. Borescope Video Camera Bell

Figure 6-3. Borescope Video Monitor and Recorder Unit

Figure 6-4. Post Grout Cell as Installed in Auburn Test Shaft

Figure 6-5. Toe Anomaly Created by Installing Sand Bags Inside the Cage

Figure 6-6. View of Toe Sand Bags from Inside the Cage

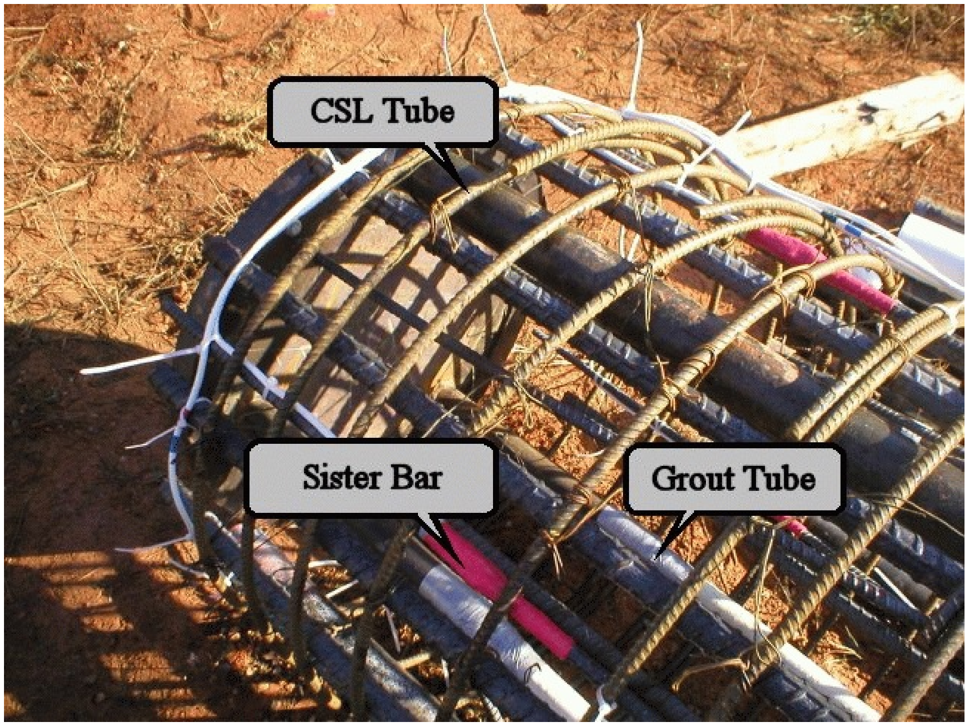

Figure 6-7. Detail View of Sister Bars, CSL Tubes, and Grout Tubes


Figure 6-8. Auburn Site CPT Soundings in Test Area

Figure 6-9. Subsurface Exploration Using the USF Mini-CPT Rig

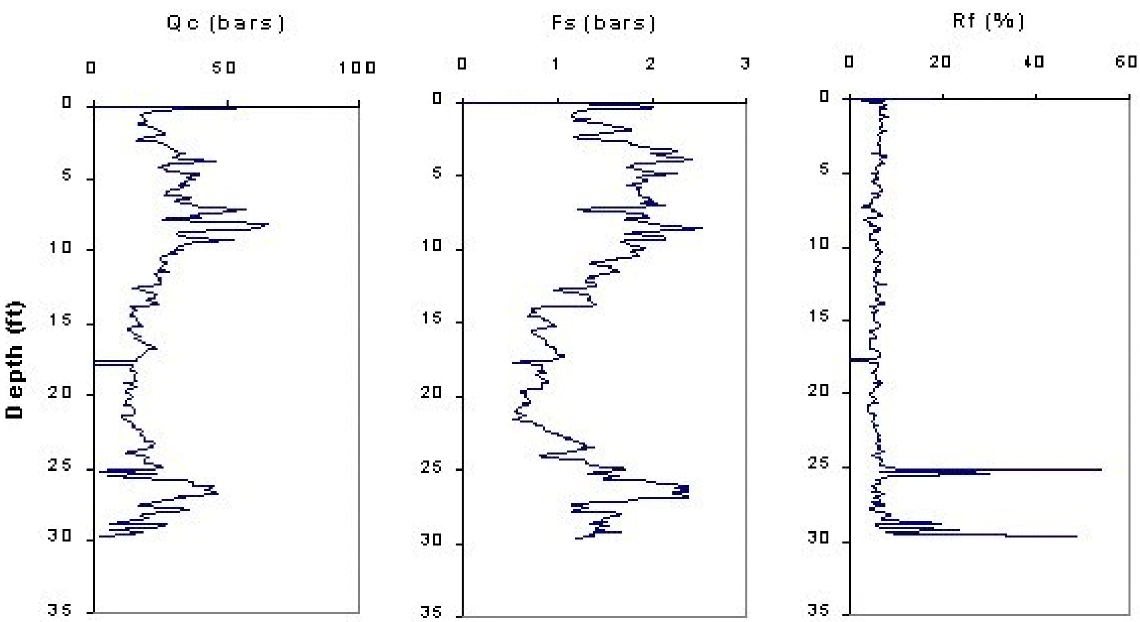

Figure 6-10. TS-1 Mini-CPT Sounding

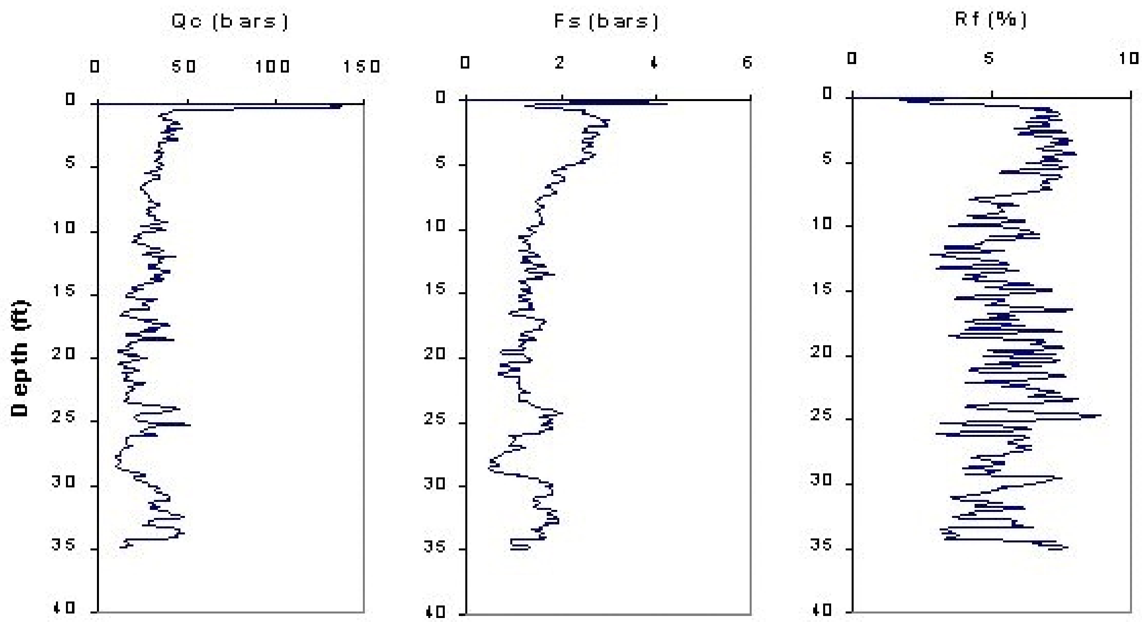

Figure 6-11. TS-2 Mini-CPT Sounding

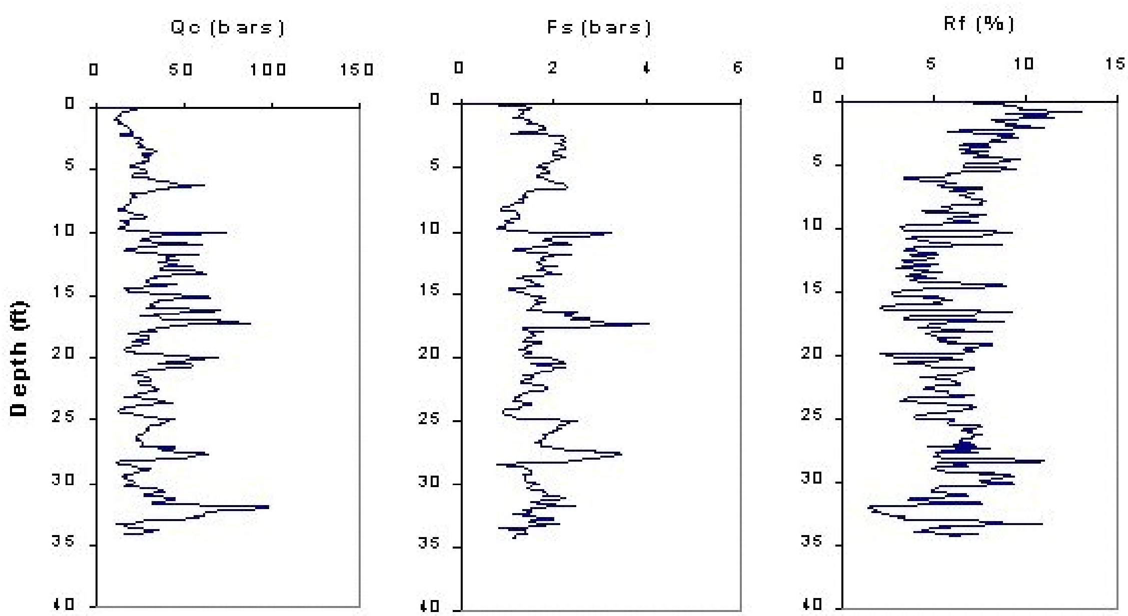

Figure 6-12. TS-3 Mini-CPT Sounding

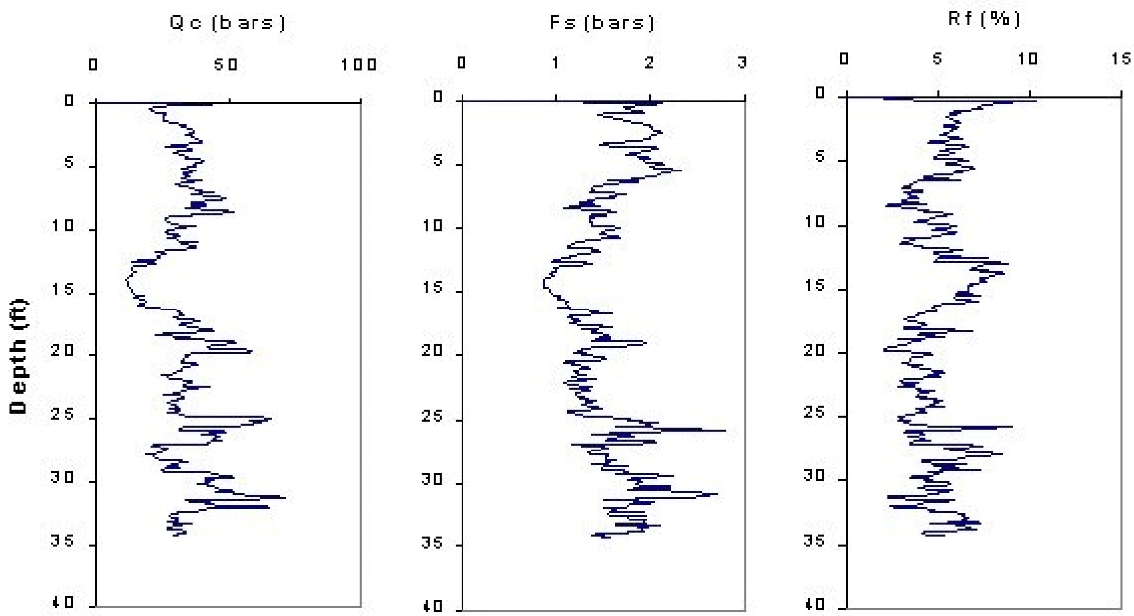

Figure 6-13. TS-4 Mini-CPT Sounding

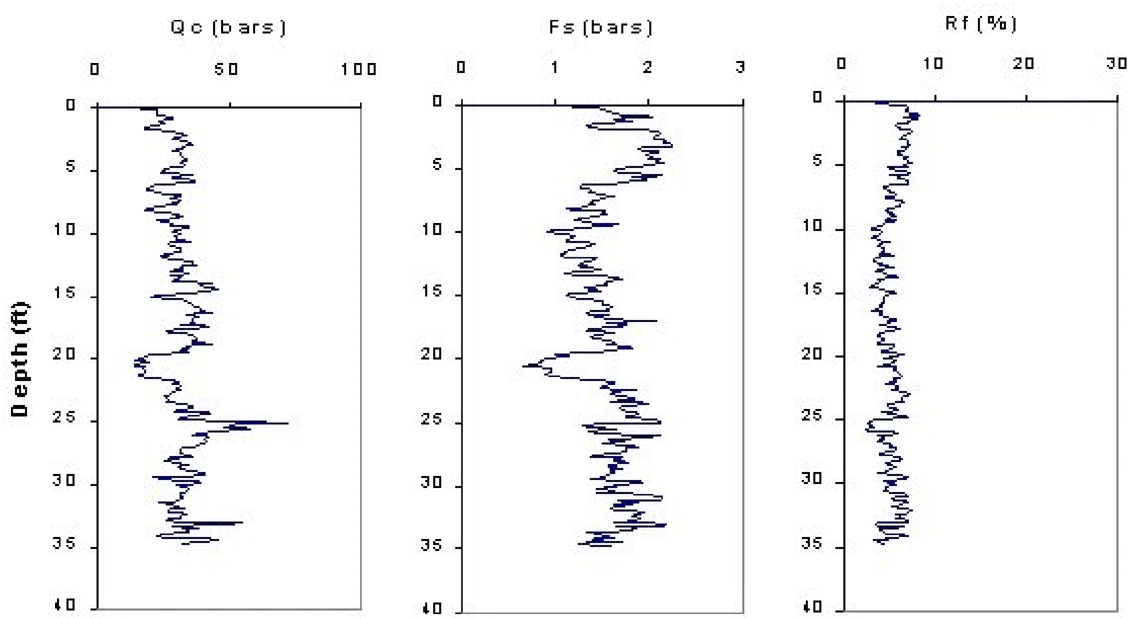

Figure 6-14. TS-5 Mini-CPT Sounding

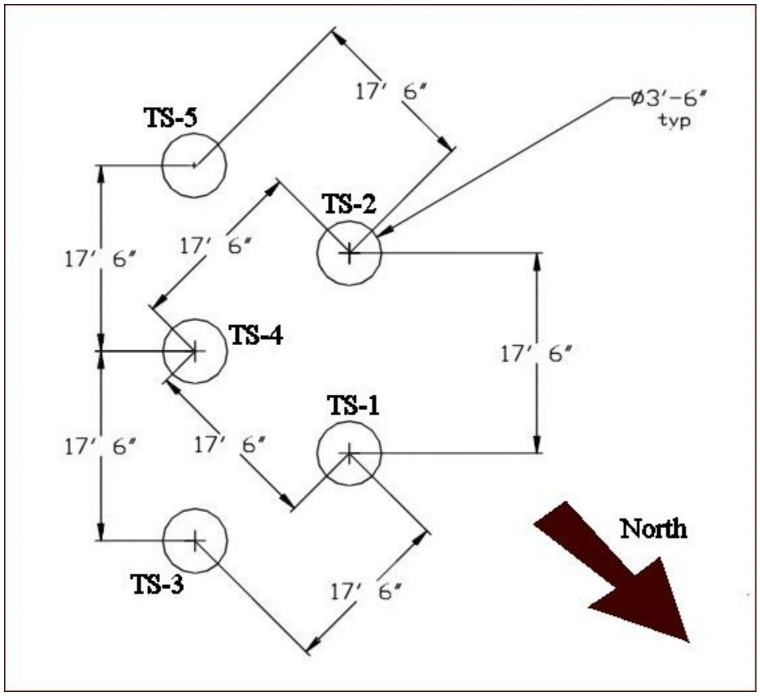

Figure 6-15. Auburn Test Site Layout

Figure 6-16. Rotary Drill Rig

Figure 6-17. Dry Hole Drilling Was Used Because of Soil Conditions on Site

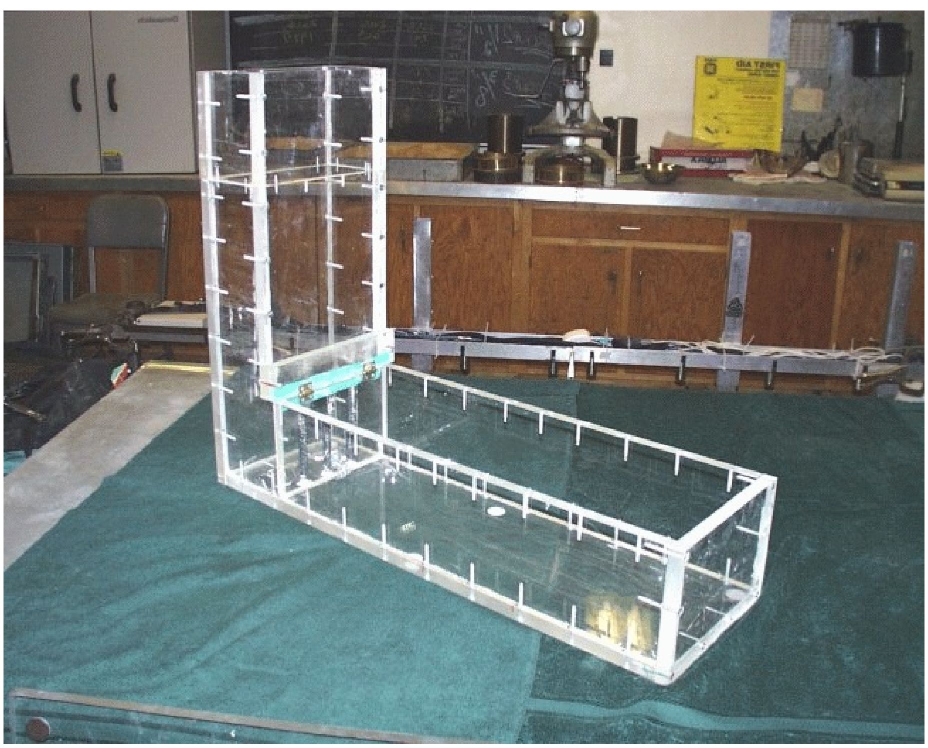

Figure 6-18. L-box Flow Test Device Used for SCC Mixes

Figure 6-19. Material Flows Through Lower Gate and Drop Height Is Recorded

Figure 6-20. TS-1: Borehole Cleanliness as Observed with the Borescope

Figure 6-21. TS-1: Sidewall of Borehole Viewed with Borescope

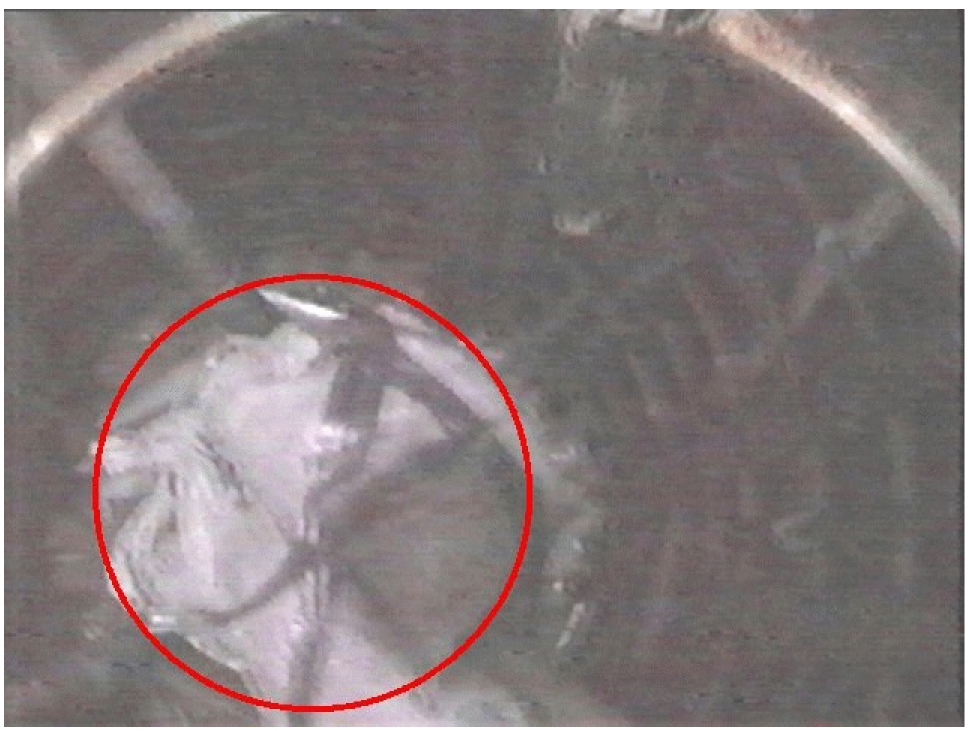

Figure 6-22. TS-1: Toe Anomaly Formed Using Sand Bags above Grout Cell

Figure 6-23. TS-1: #57 Stone Mix, High Slump, Showing Some Binding near Rebar

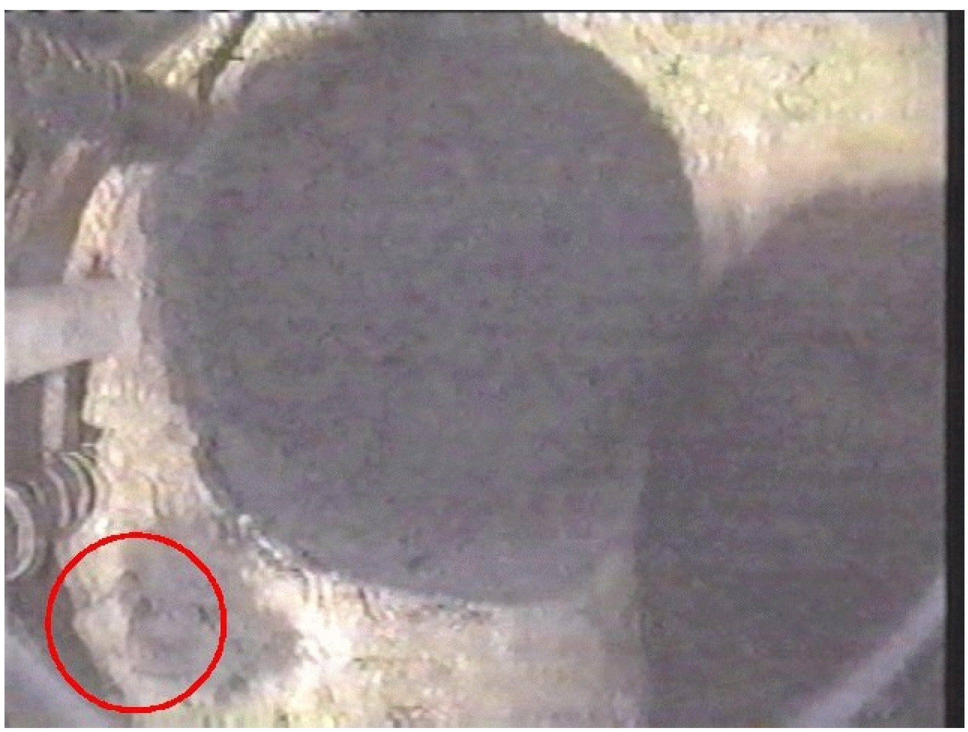

Figure 6-24. TS-1: Binding of Coarse Aggregate near Rebar

Figure 6-25. TS-1: Longitudinal Bars Create an Even Larger Area of Binding

Figure 6-26. TS-2: #7 Stone Mix, High Slump, Good Flow Characteristics

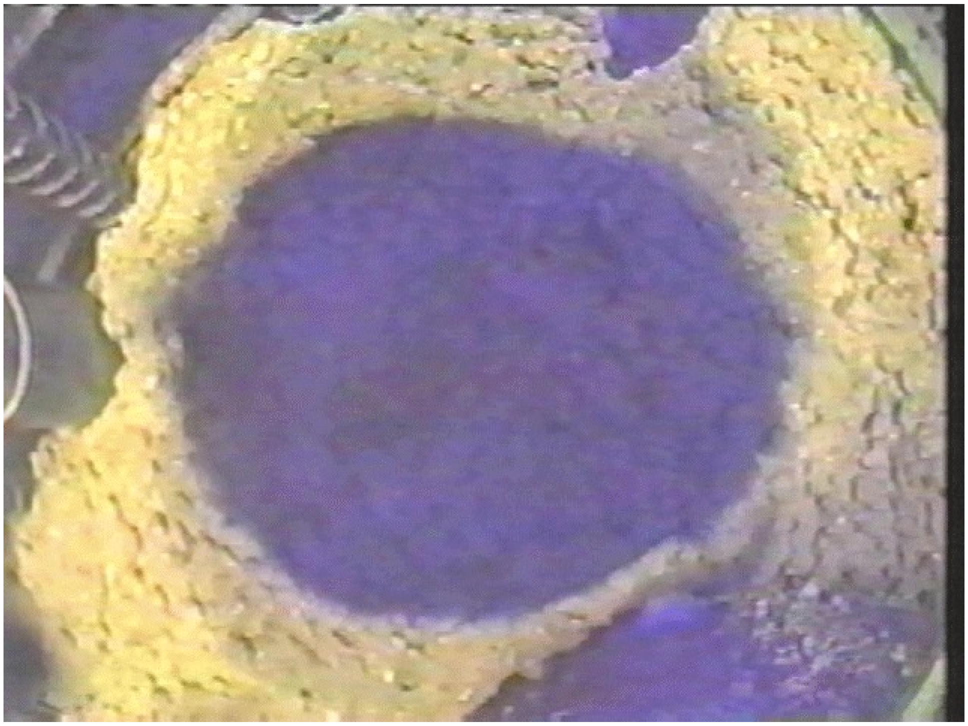
Figure 6-27. TS-2: Overall Flow for #7 Stone Mix Is More Uniform

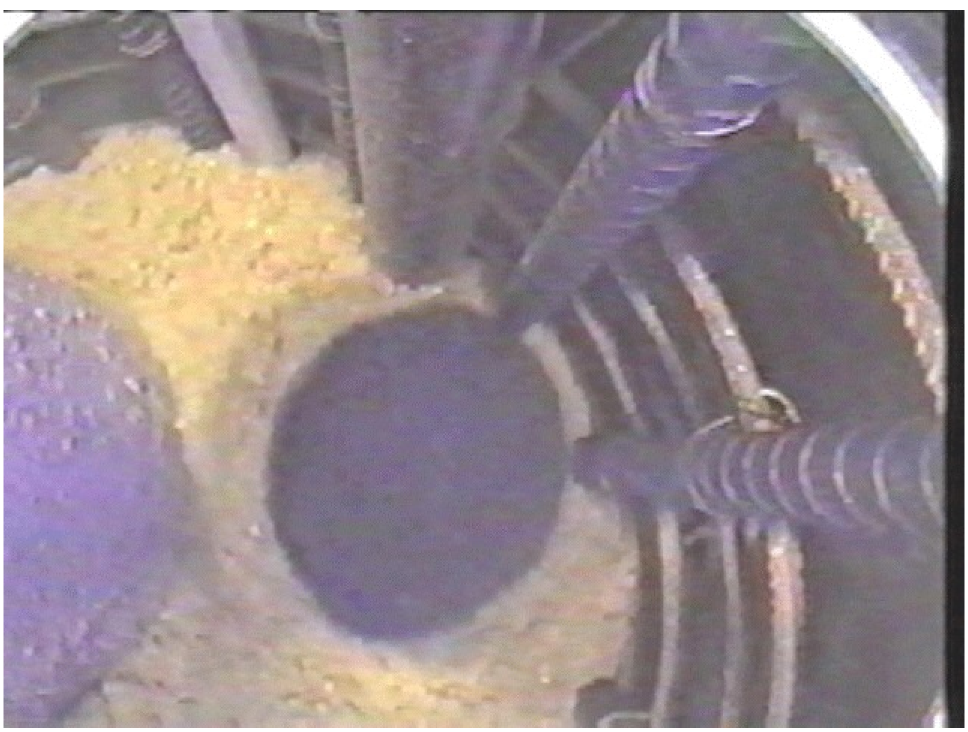
Figure 6-28. TS-2: Tight Longitudinal Spacing but No Obvious Binding

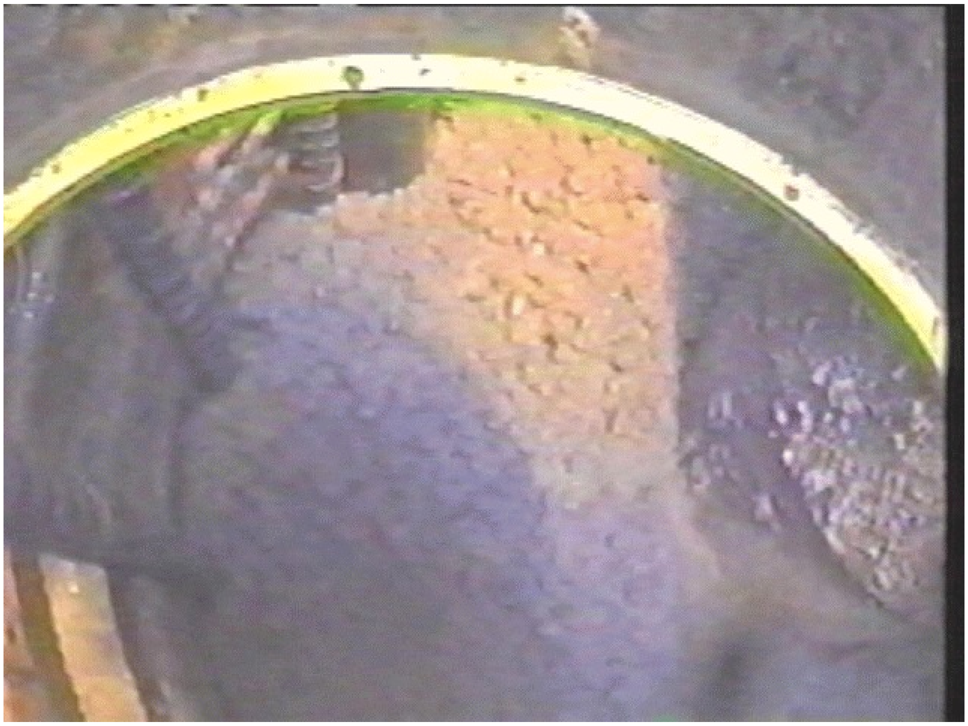

Figure 6-29. TS-2: #7 Stone Mix Pours with Minimal Head Differential

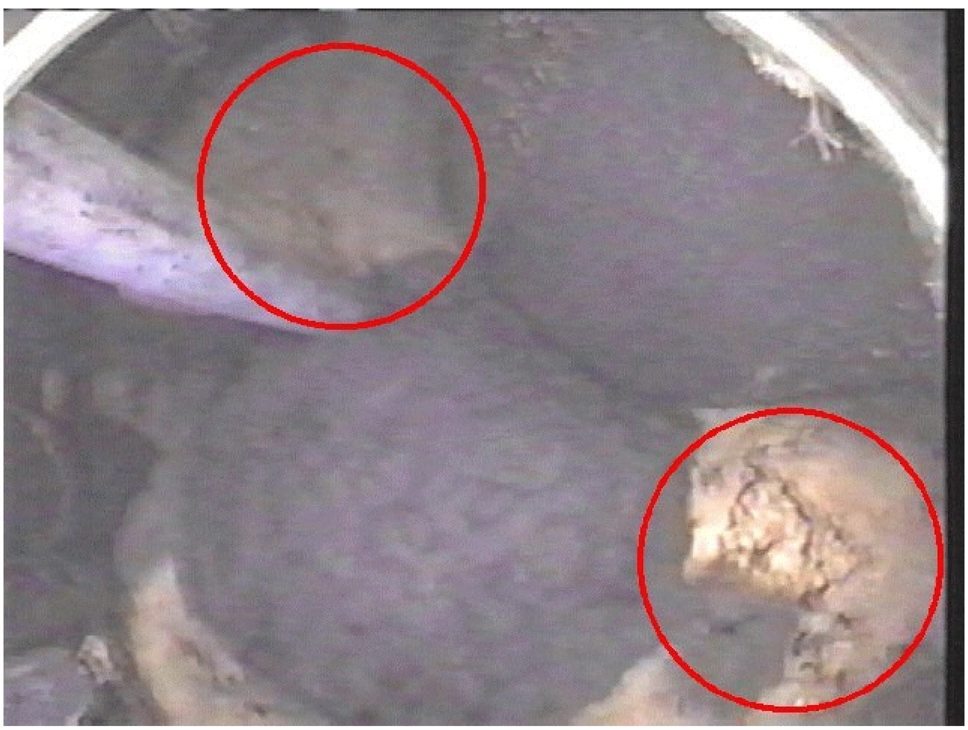

Figure 6-30. TS-3: #57 Stone, Low Slump, Showing Segregation and Bleeding

Figure 6-31. TS-3: Obvious Segregation and Bleed off of Mix Water During Pour

Figure 6-32. TS-3: Low Slump Mix Also Exhibits High Degree of Clumping


Figure 6-33. TS-5: SCC Mix Flows Well Around Tight Obstructions

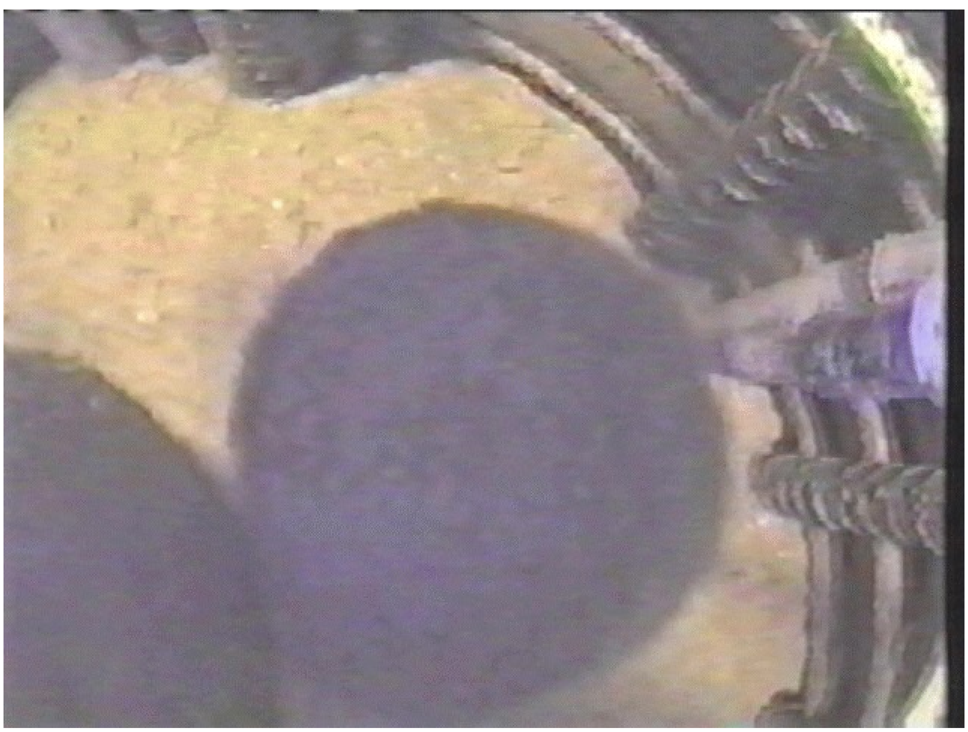
Figure 6-34. TS-5: No Visible Binding Near Rebar for SCC Mixes

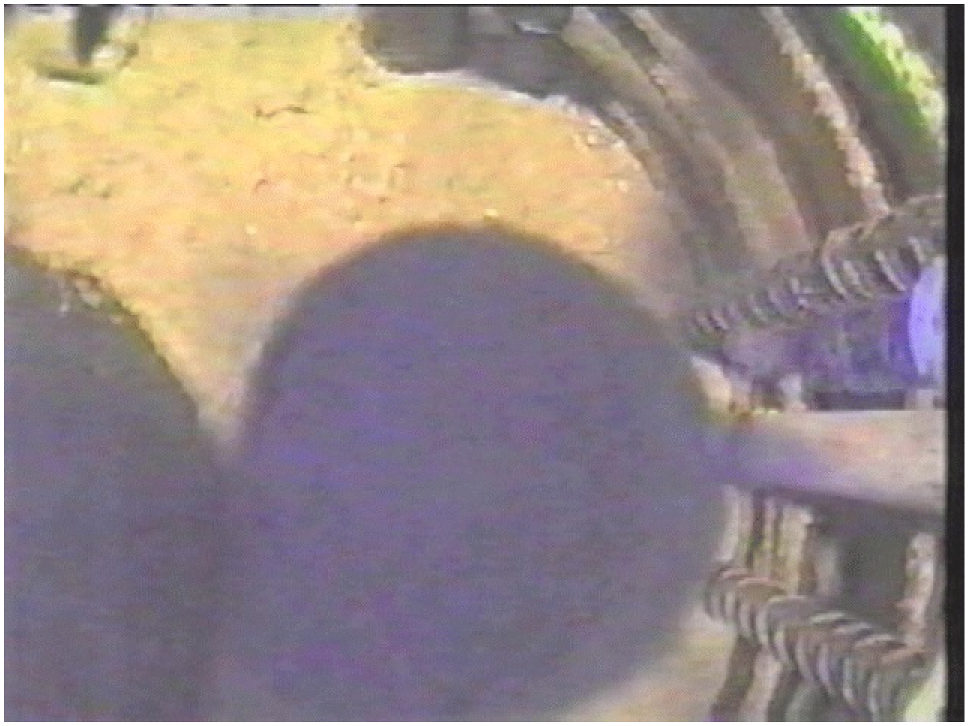

Figure 6-35. TS-5: SCC Mix Shows Even Flow and No Obvious Segregation

Figure 6-36. TS-5: SCC Mix Flow Shows Very Little Inner to Outer Cage Differential

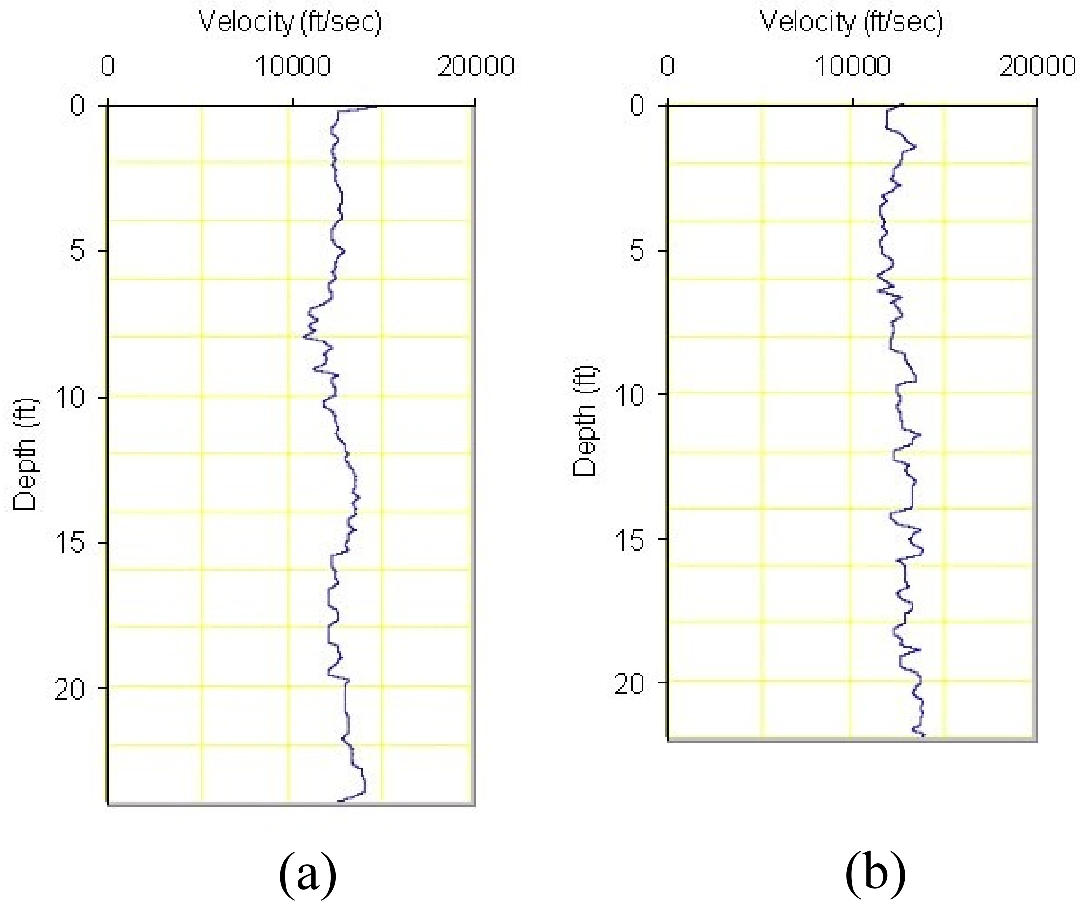

Figure 6-37. TS-3 CSL Data (a) Initial Readings and (b) Final Readings

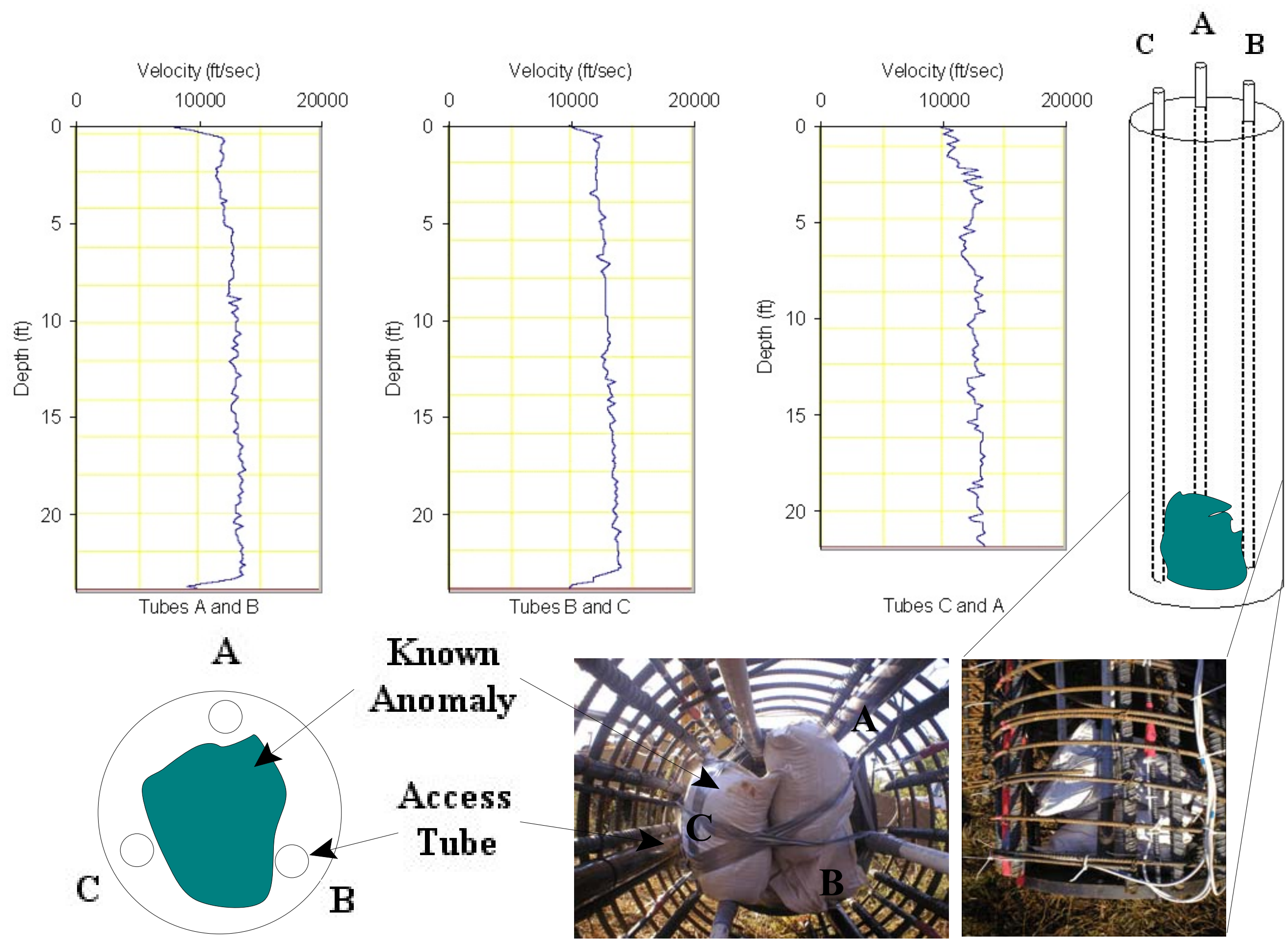

Figure 6-38. TS-1 CSL Data for All Access Tubes and Anomaly Location

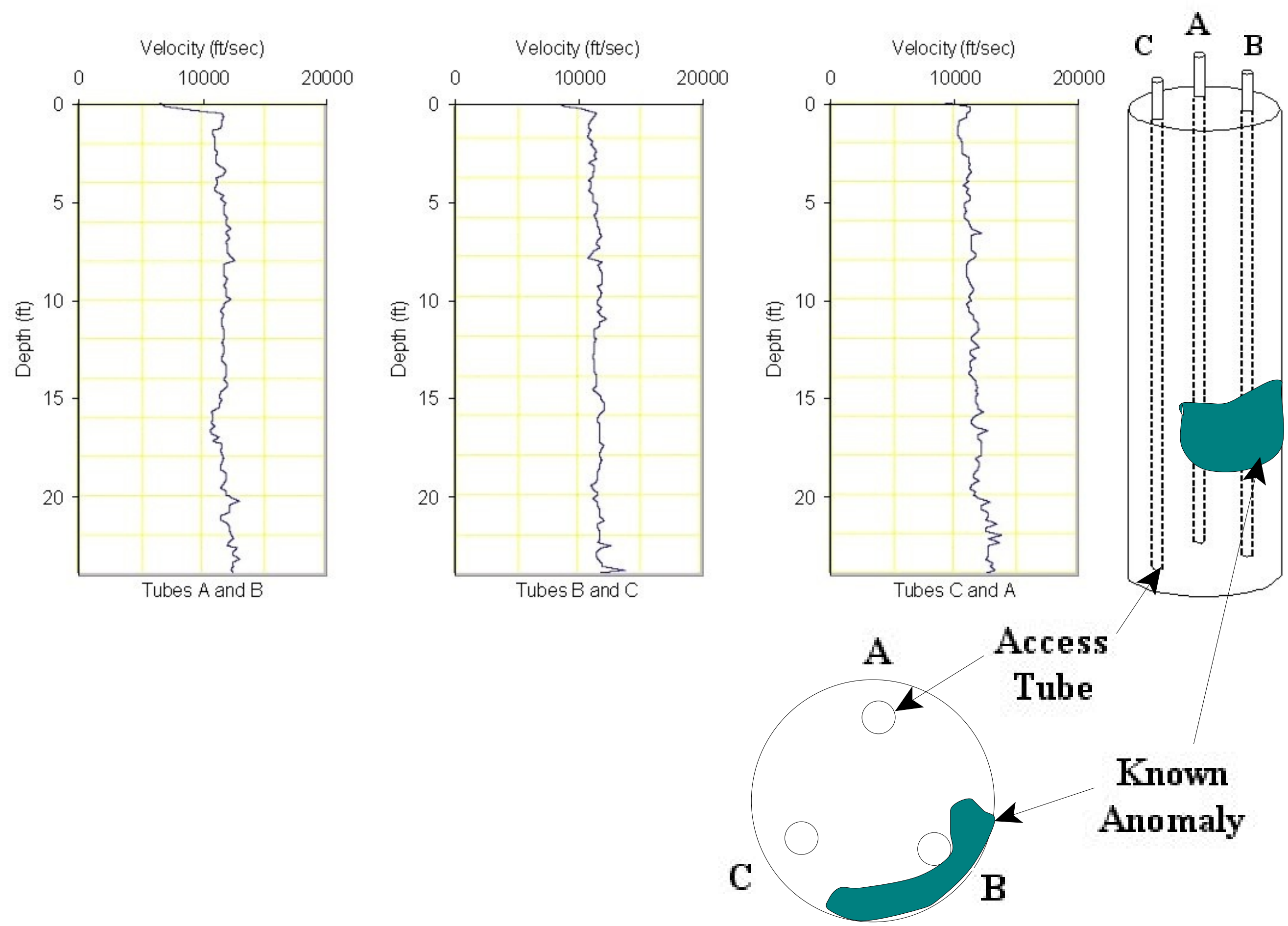

Figure 6-39. TS-2 CSL Data for All Access Tubes and Anomaly Location

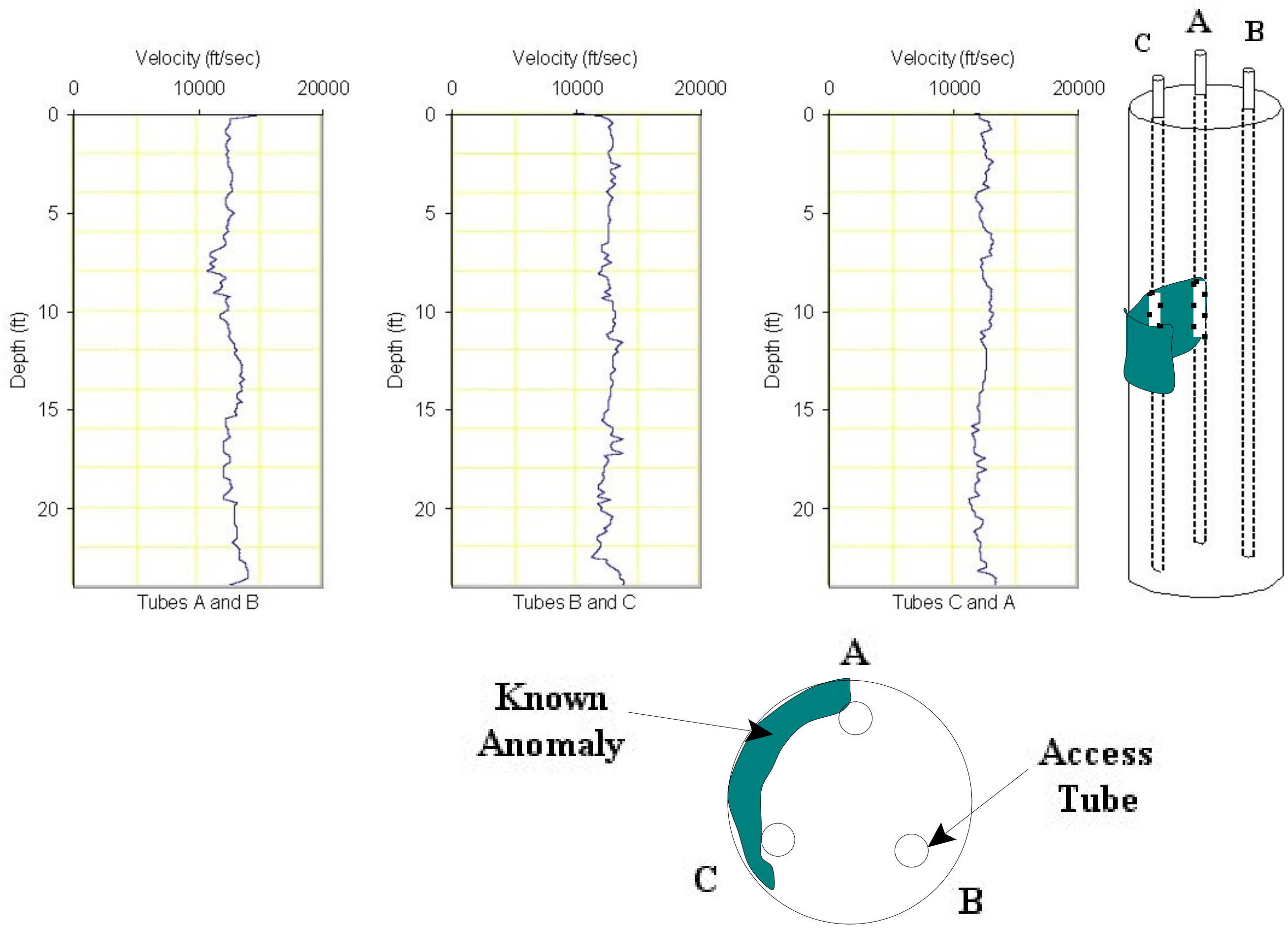

Figure 6-40. TS-3 CSL Data for All Access Tubes and Anomaly Location

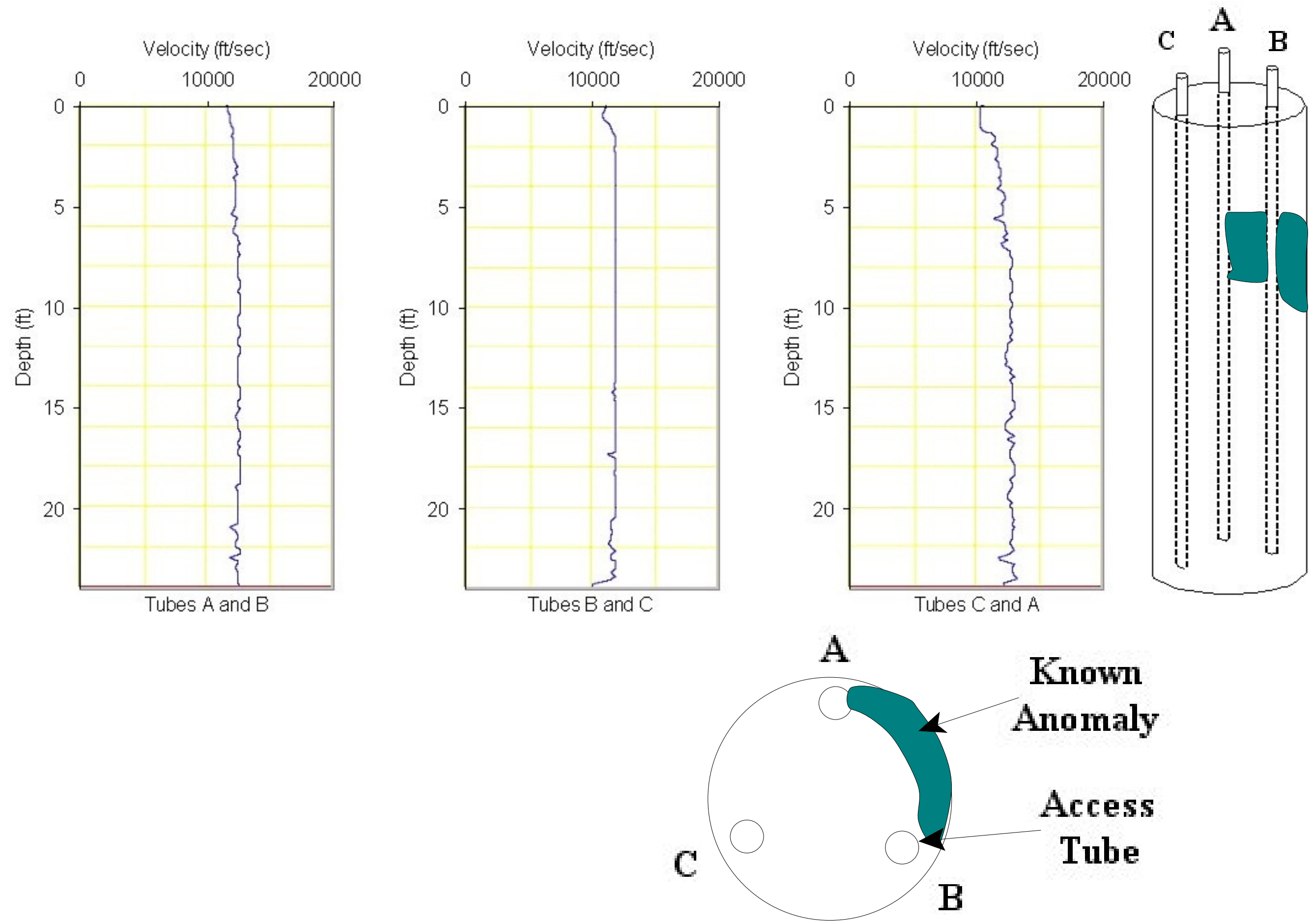

Figure 6-41. TS-5 CSL Data for All Access Tubes and Anomaly Location

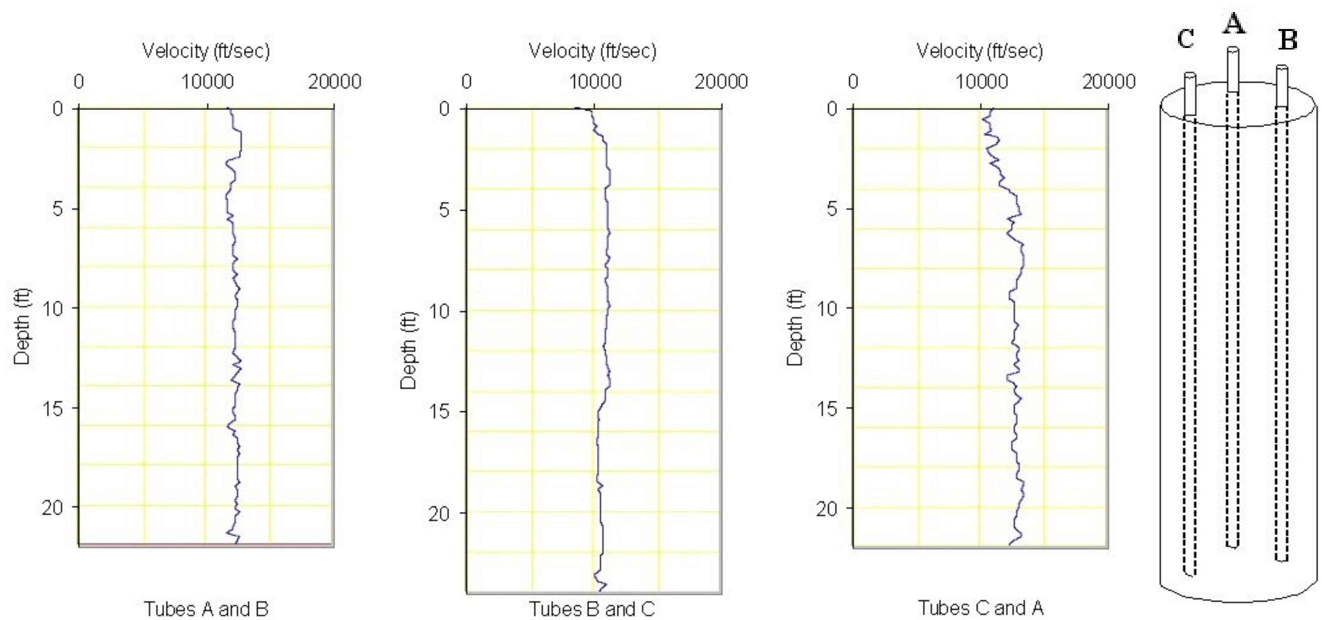

Figure 6-42. TS-4 CSL Data for All Access Tubes (Control Shaft, No Anomalies)

Figure 6-43. TS-4 Exhumed Shaft Tip

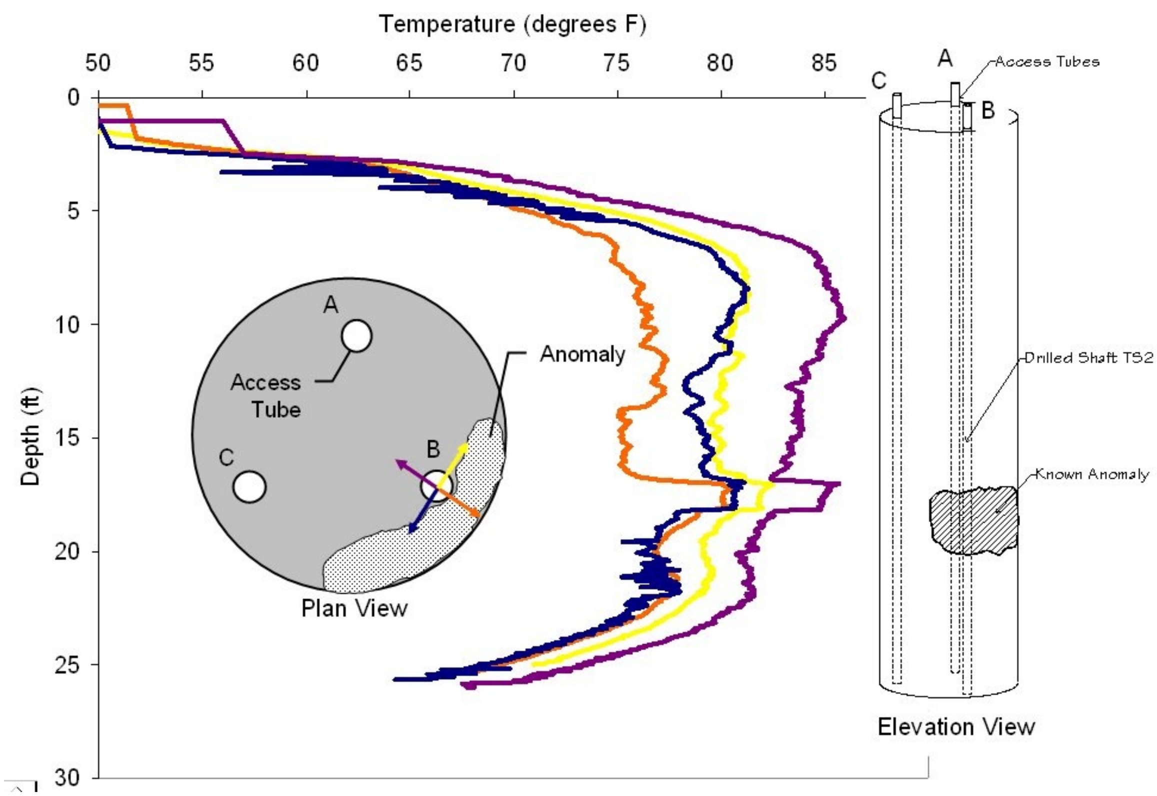

Figure 6-44. Thermal Integrity Testing of TS-2 (Tube B)

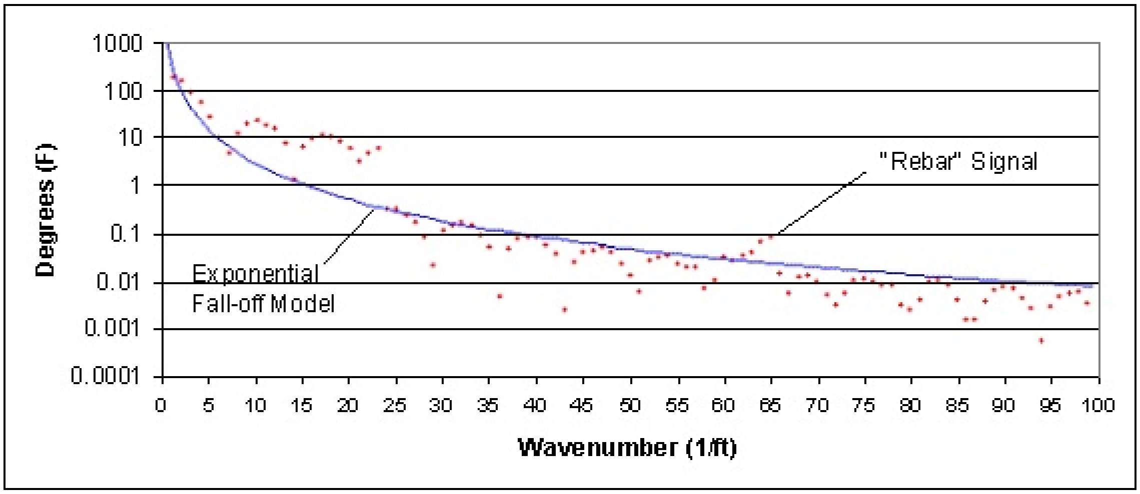

Figure 6-45. Fourier analysis of inward-facing sensor temperature data for shaft TS-3 (Tube A)

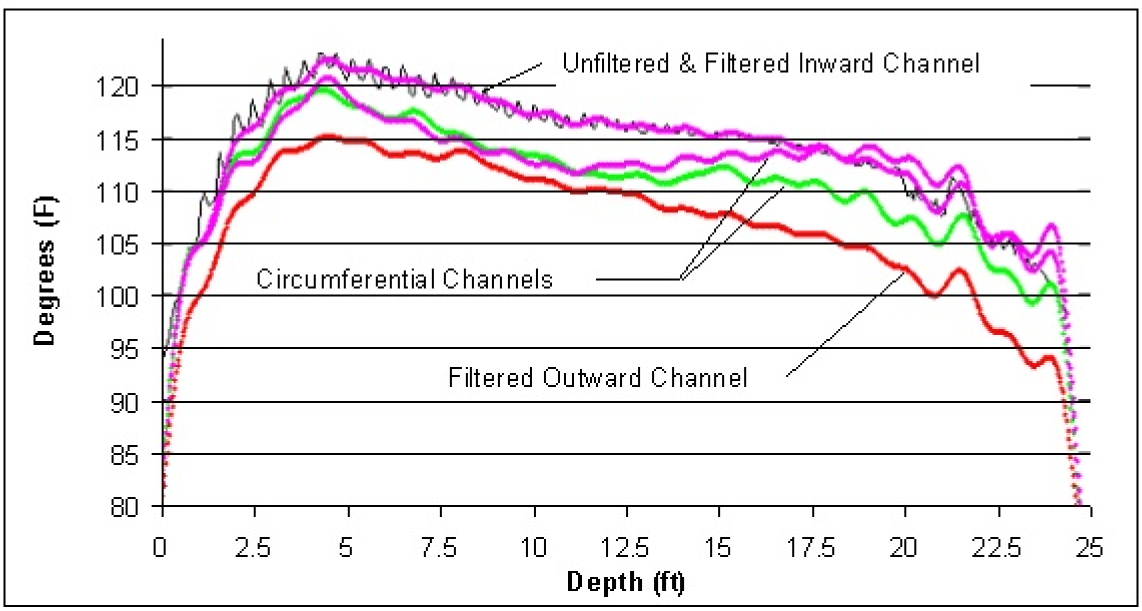

Figure 6-46. Low-passed ("rebar-filtered") temperature traces and one unfiltered trace of the inward-facing channel from shaft TS-3 (Tube A)

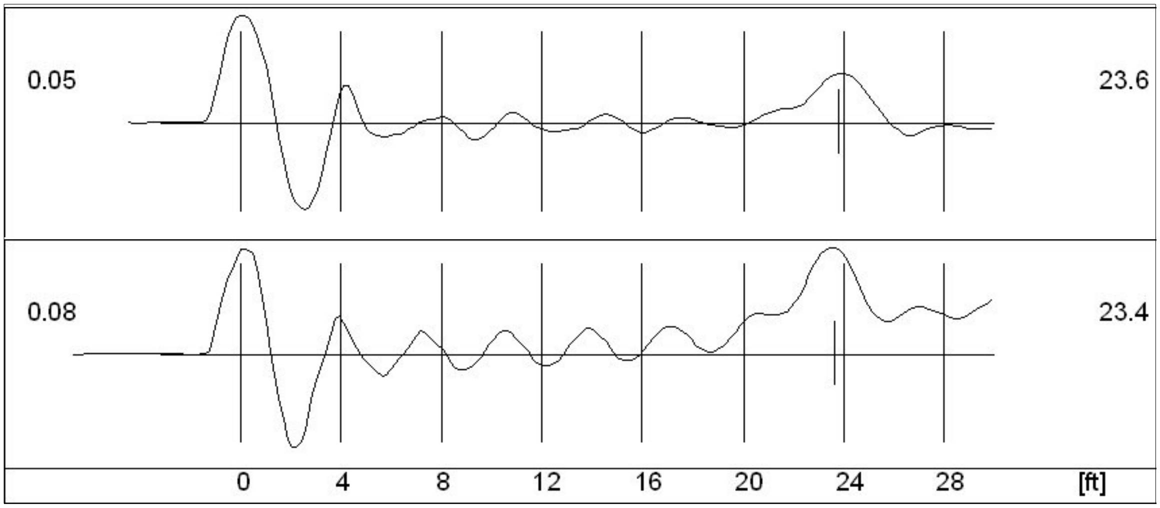

Figure 6-47. SIT data for TS-1 before grouting (top) and after load testing (bottom).

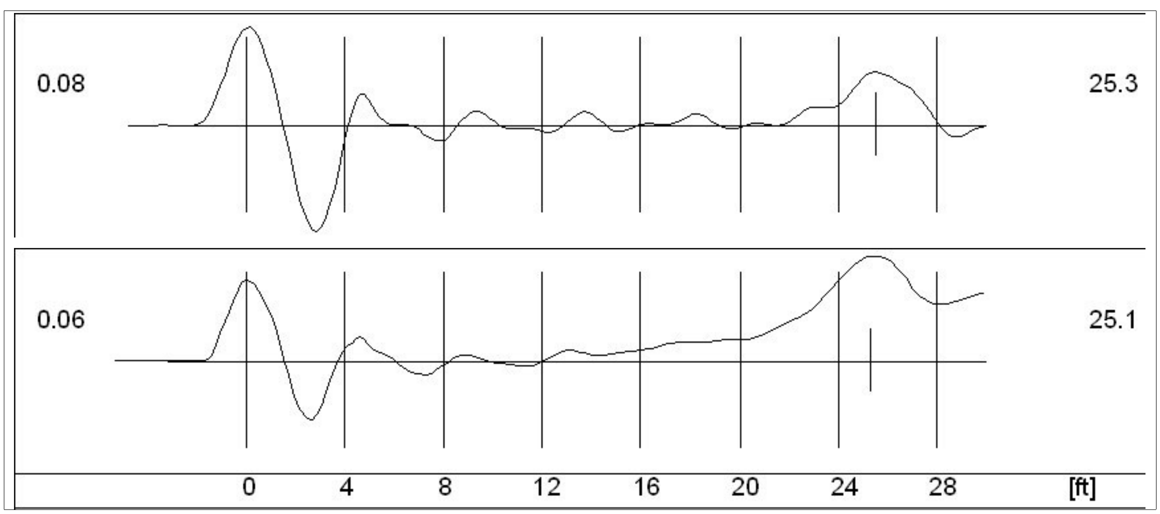

Figure 6-48. SIT data for TS-2 before grouting (top) and after load testing (bottom).

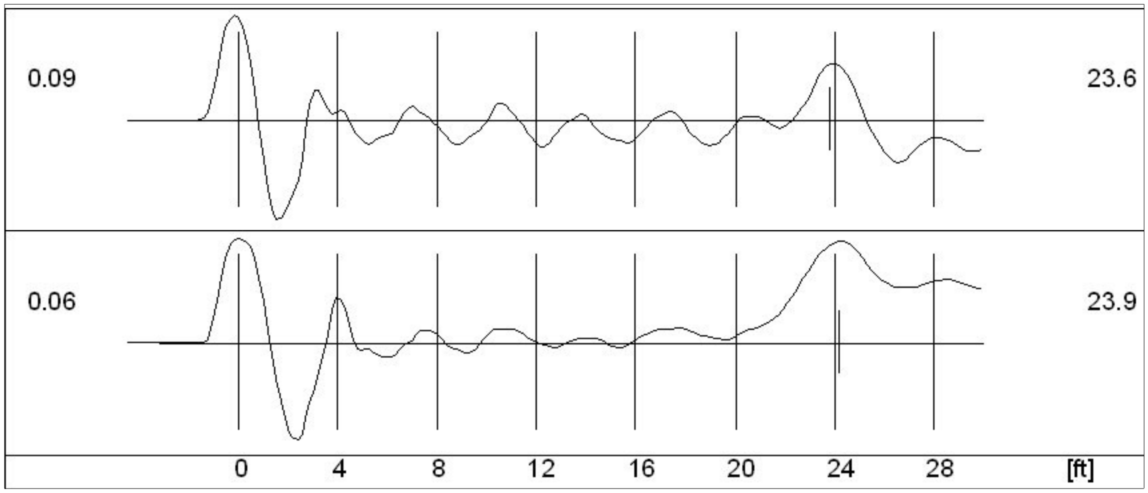

Figure 6-49. SIT data for TS-3 before grouting (top) and after load testing (bottom).

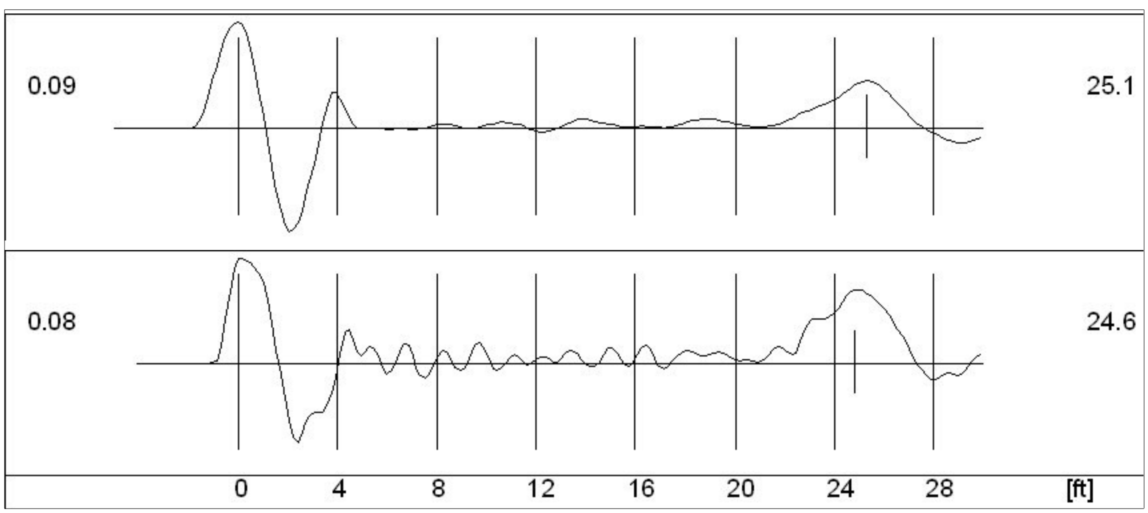

Figure 6-50. SIT data for TS-4 day of grouting (top) and after load testing (bottom).

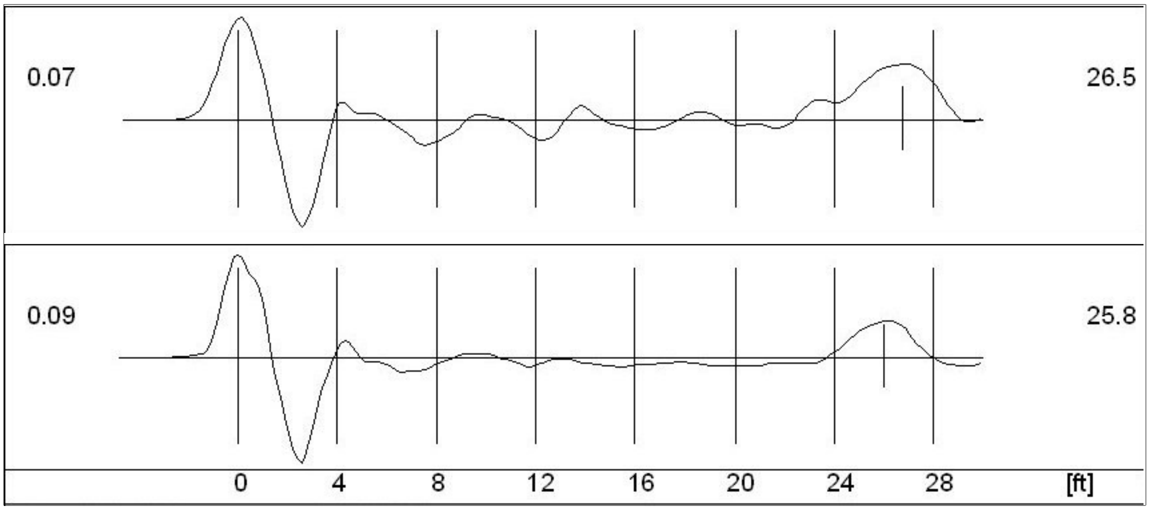

Figure 6-51. SIT data for TS-5 before grouting (top) and after load testing (bottom).


Figure 6-52. Mixing Grout in the Pump Prior to Post Grouting Each Shaft


Figure 6-53. Post Grouting Operation Injects Grout to Shaft Tip


Figure 6-54. Upward Movement of Shaft Measured with String Extensometers

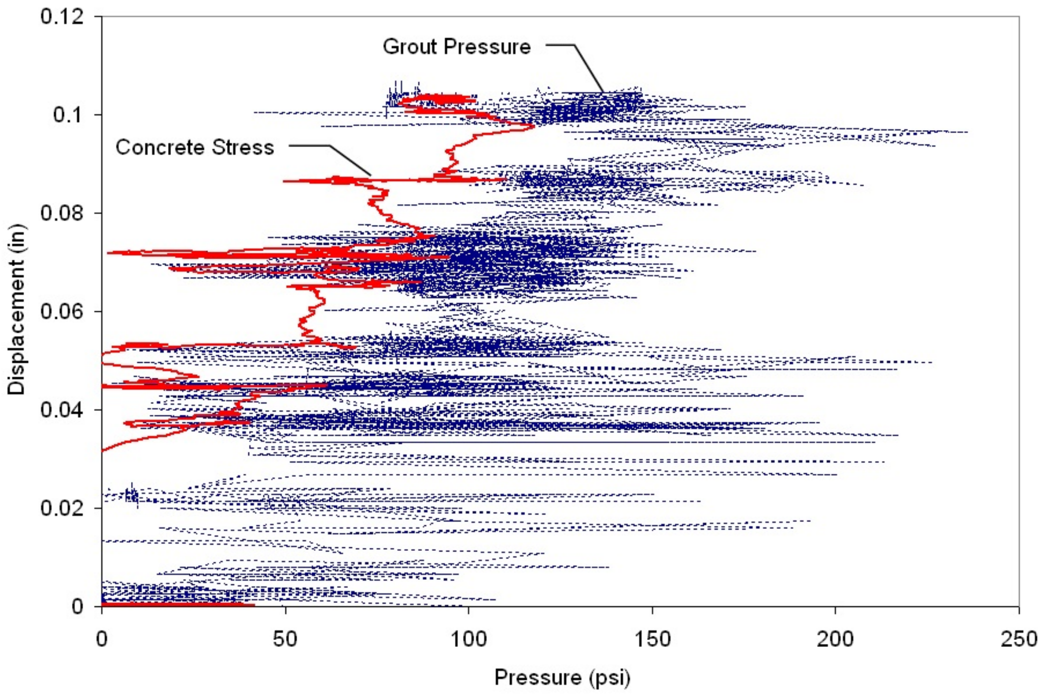

Figure 6-55. TS-1 Grout Pressure versus Displacement

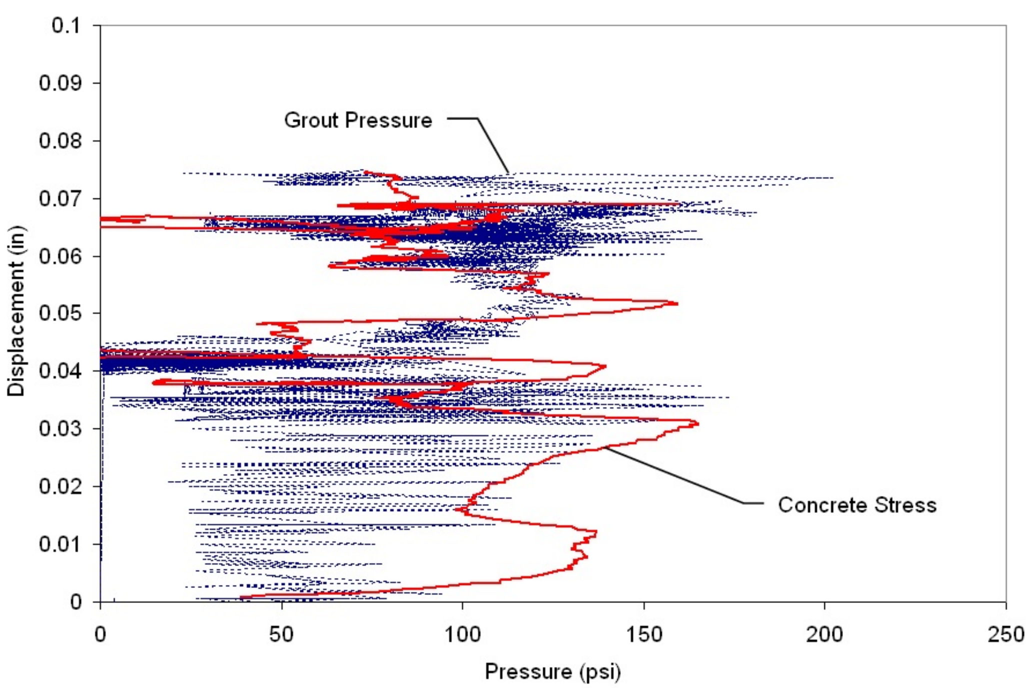

Figure 6-56. TS-2 Grout Pressure versus Displacement

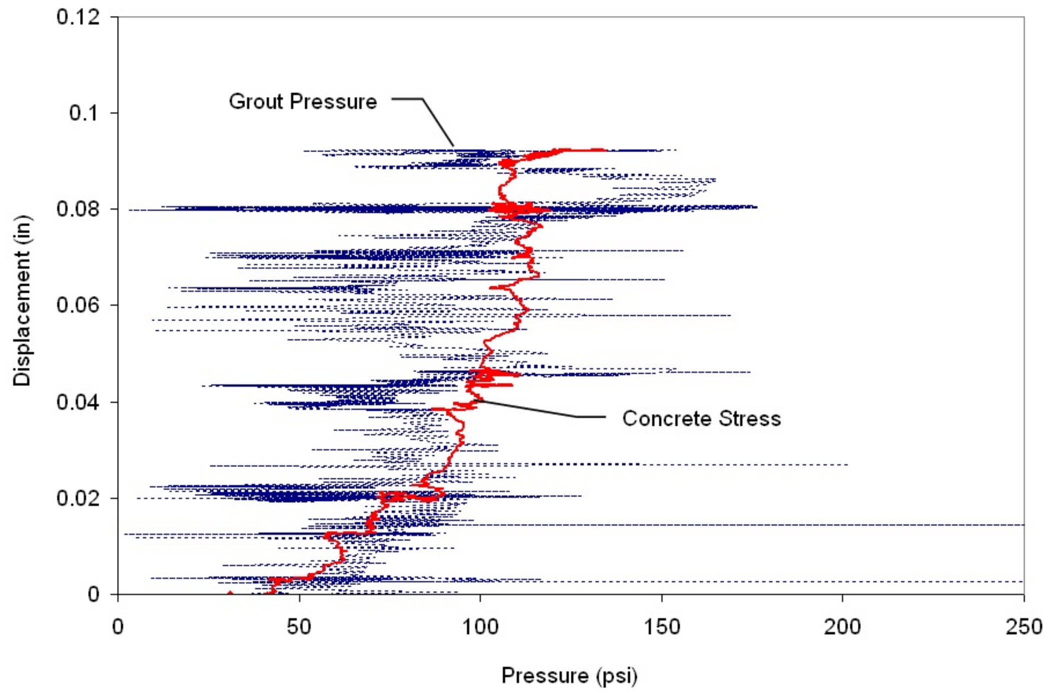

Figure 6-57. TS-3 Grout Pressure versus Displacement

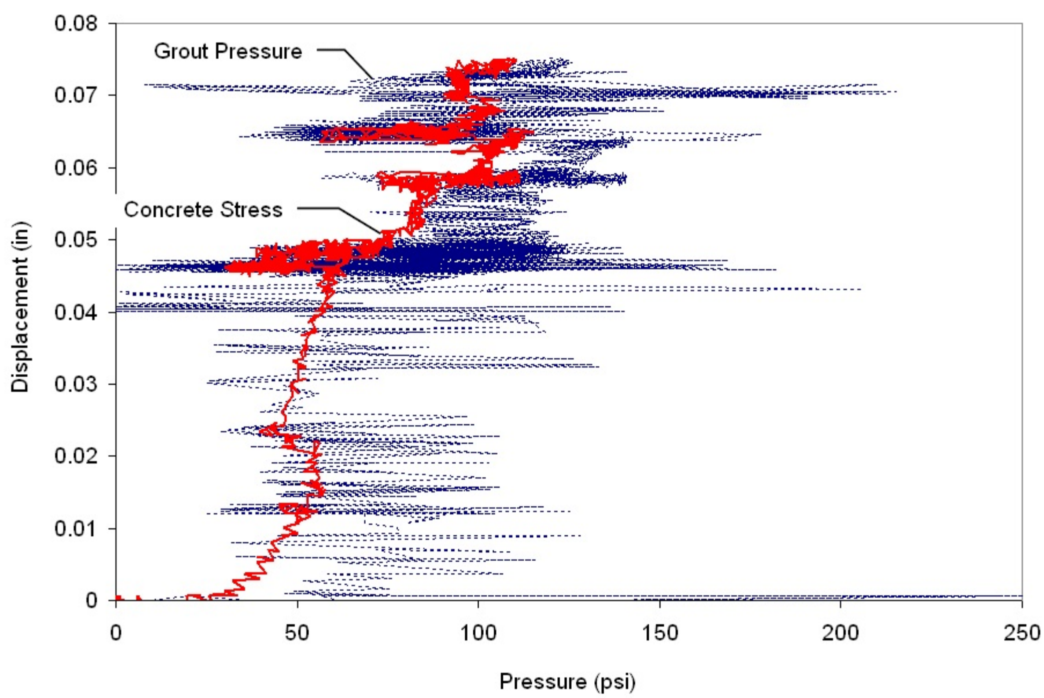

Figure 6-58. TS-5 Grout Pressure versus Displacement

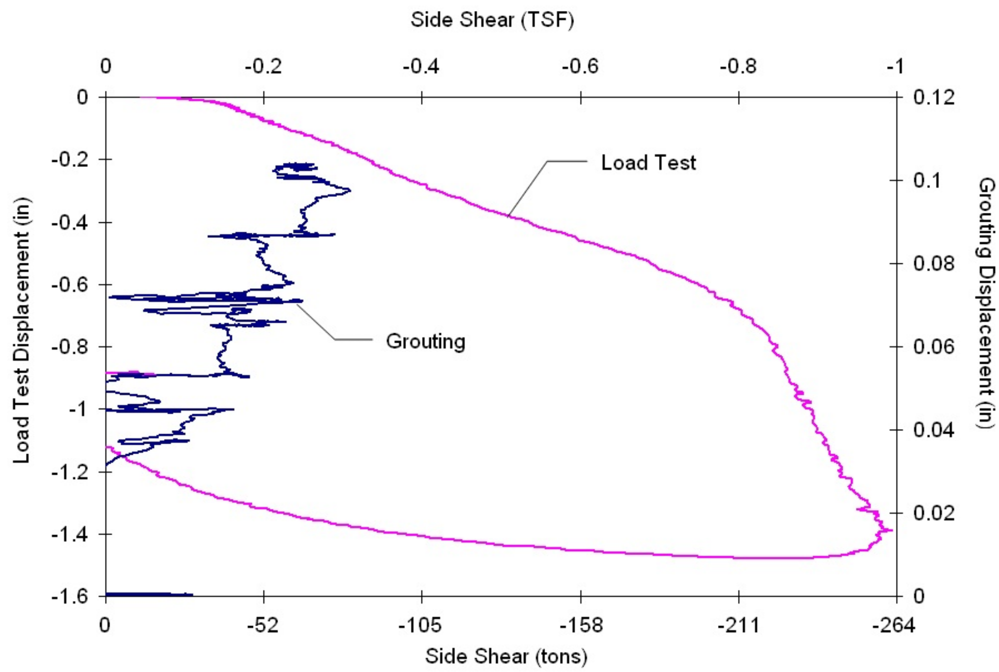

Figure 6-59. TS-1 Side Shear Plots

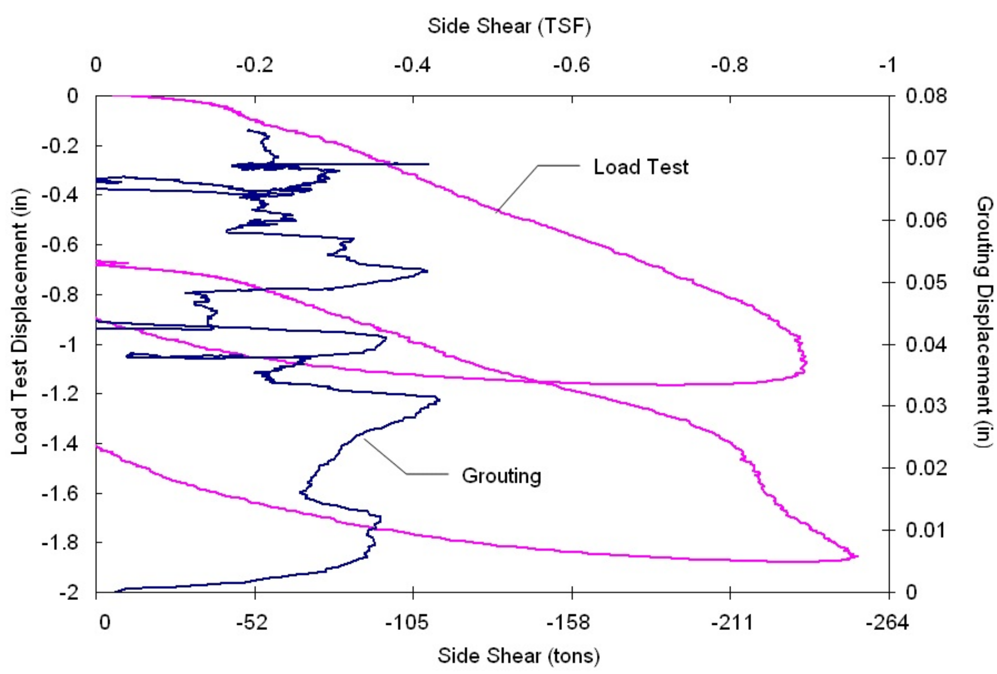

Figure 6-60. TS-2 Side Shear Plots

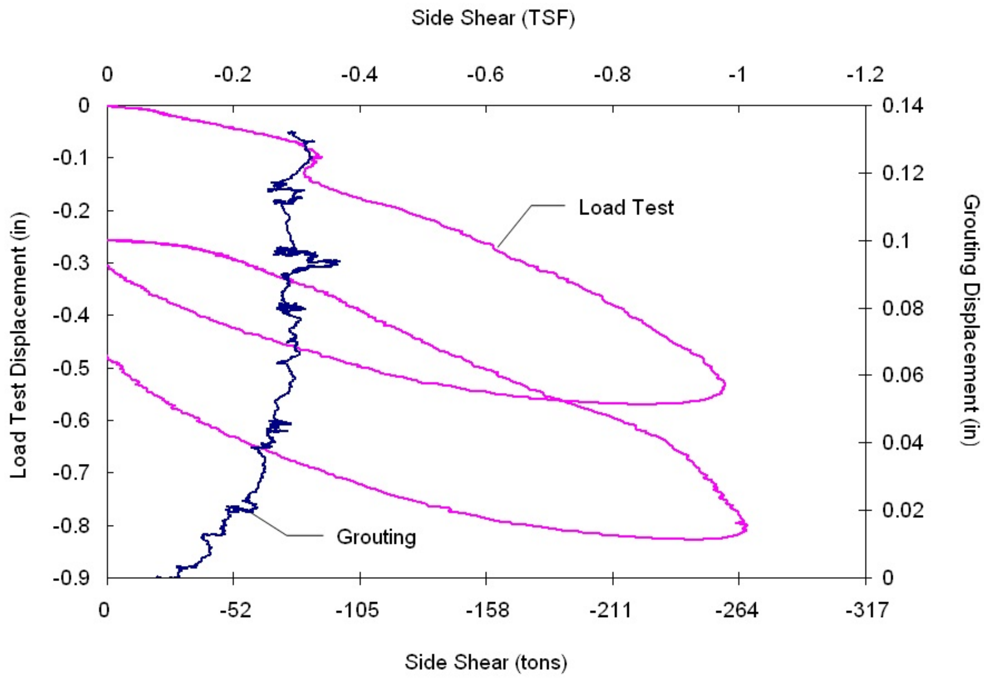

Figure 6-61. TS-3 Side Shear Plots

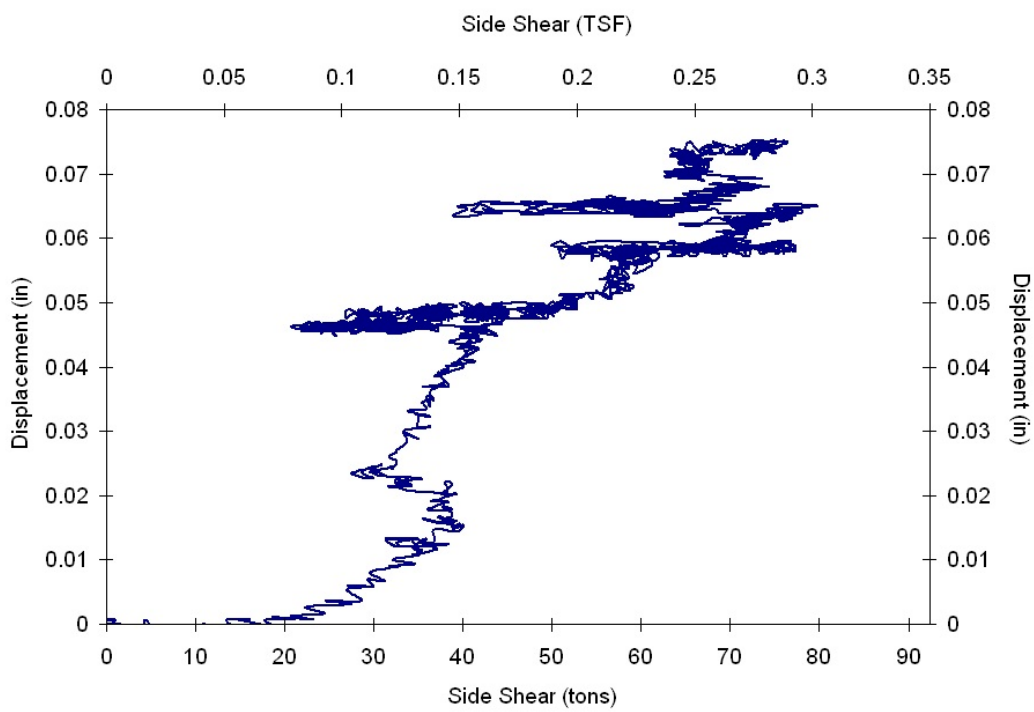

Figure 6-62. TS-5 Side Shear Plot during Grouting

Figure 6-63. 4MN Statnamic Load Test Setup

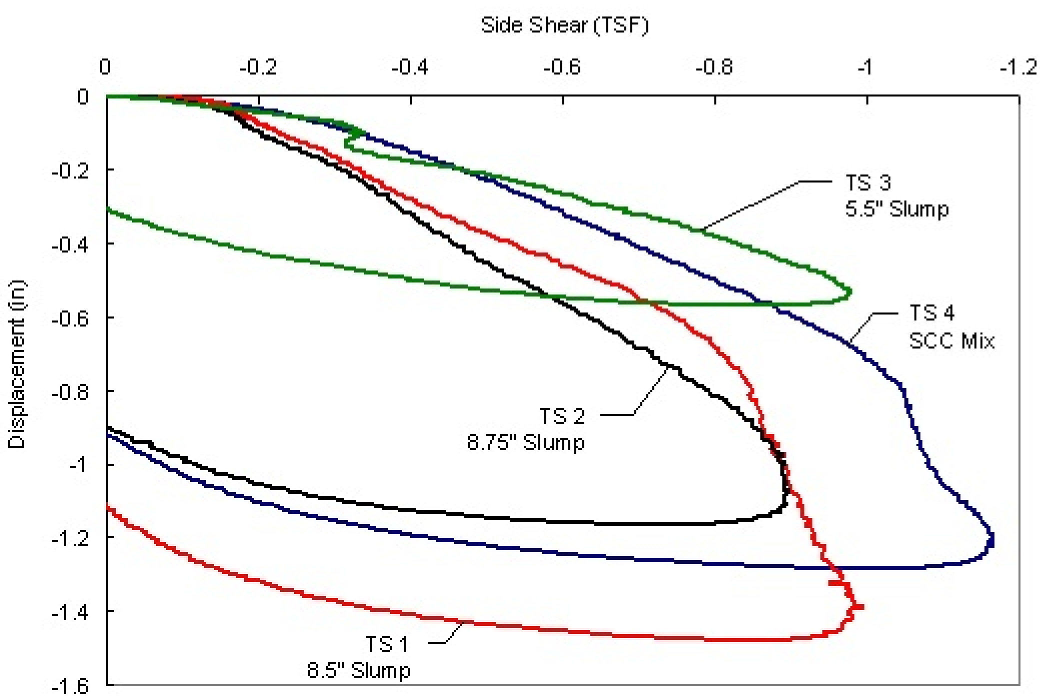

Figure 6-64. Side Shear Values for All Test Shafts (except TS-5)

Figure 6-65. Soil Excavation for Shaft Removal

Figure 6-66. Coring of Test Shafts for Permeability Testing

Figure 6-67. Saw Cutting of Test Shafts

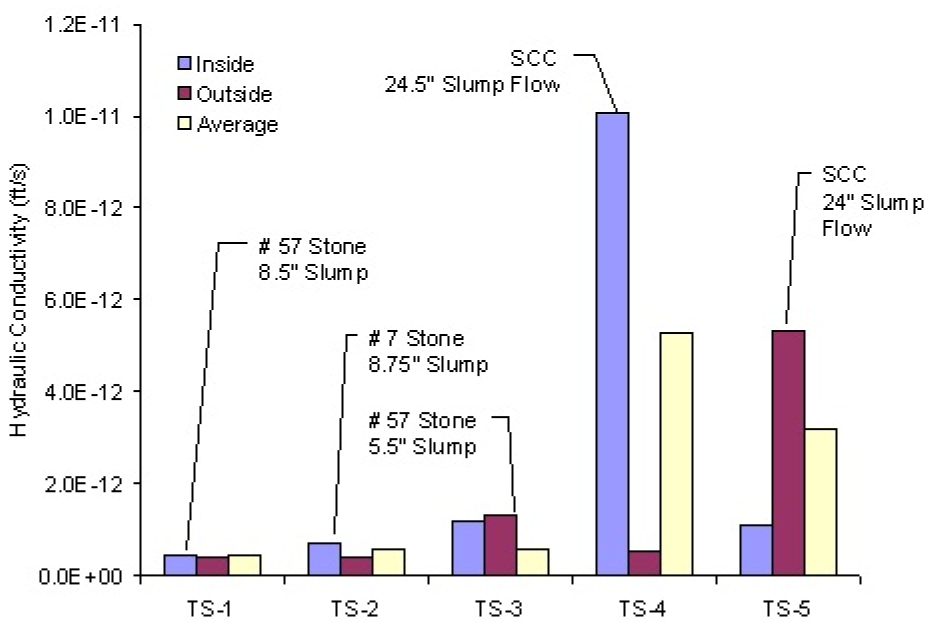

Figure 6-68. Permeability Results from Concrete Cores

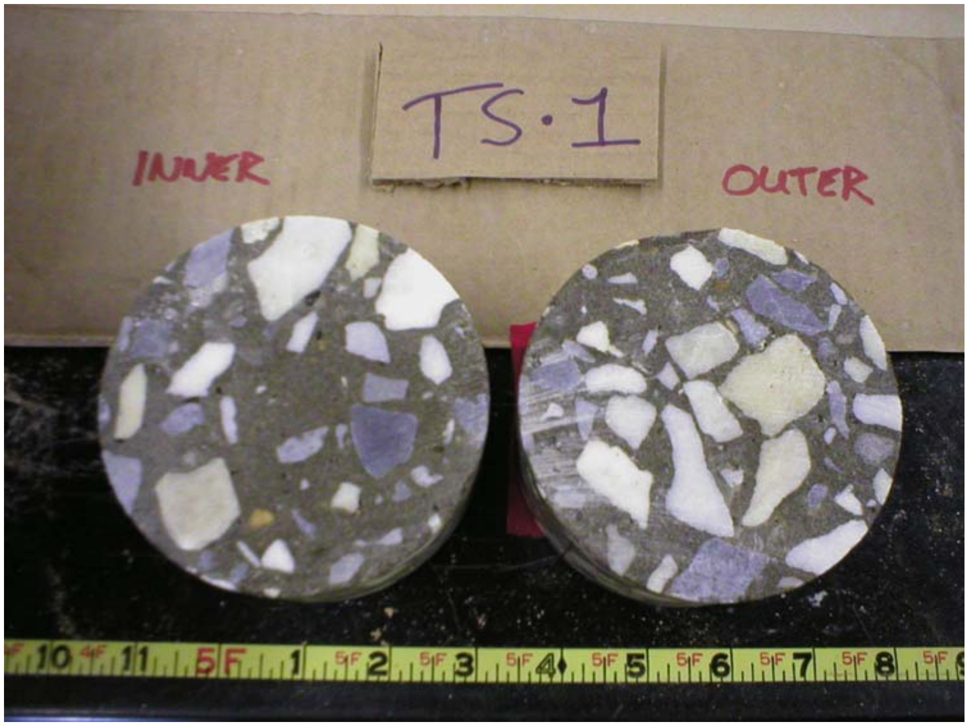

Figure 6-69. Core Sample from Shaft TS-1 Showing Material from Inner & Outer Cage

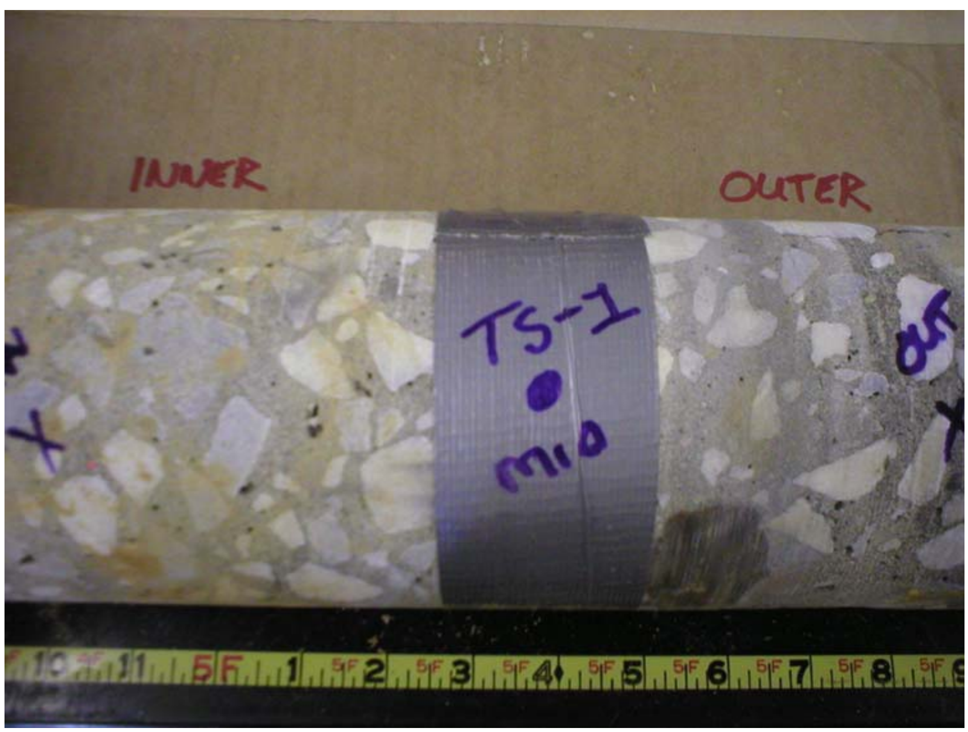

Figure 6-70. TS-1 Core Sample Before Splitting

Figure 6-71. Core Sample from Shaft TS-2

Figure 6-72. Core Sample from Shaft TS-3

Figure 6-73. Outer Cage Core Sample from Shaft TS-4

Figure 6-74. Core Sample from Shaft TS-5

Figure 6-75. TS-3 Saw Cut

Figure 6-76. TS-5 Saw Cut

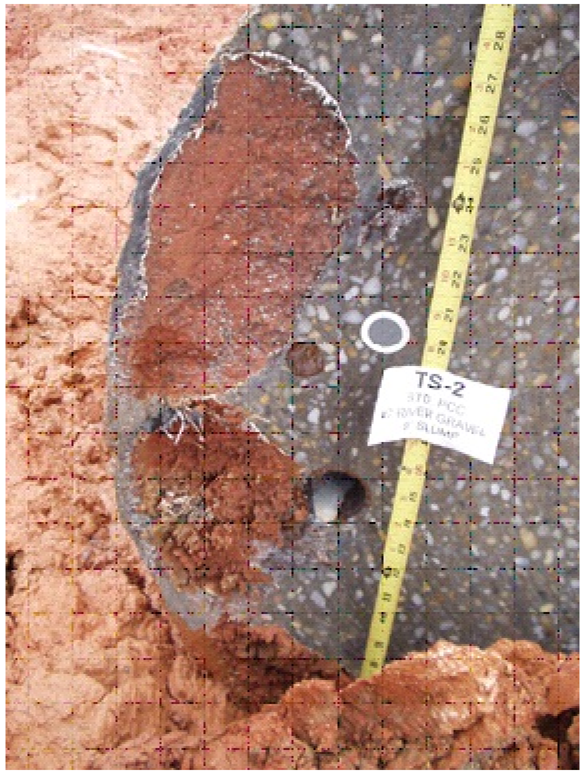

Figure 6-77. TS-2 Saw Cut

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7. FIELD HEAD DIFFERENTIAL MEASUREMENTS

7.1 Background

The discovery of a head differential in rising mortar between the inside and outside of the reinforcement cage during the LPC Testing Series III spawned the desire to better define and understand this behavior. To this end, a field testing program was established to survey if this behavior was observed in commercially constructed drilled shafts. The main objective of this testing was to quantify head differential behavior with variables such as CSD, rising concrete velocity, and concrete properties and borehole characteristics.

Multiple field visits across several sites were conducted to ascertain the amount of wet concrete head differential in various types of drilled shaft construction and to examine the correlation between this differential and rising concrete velocity. In order to obtain a variety of data, researchers deliberately chose sites with varying shaft diameters and CSD ratios. This field testing allowed researchers to obtain data for the primary objective, and also to survey commercial construction of drilled shafts not only for highway construction, but also private construction. For each site, researchers collected as much information as possible about its unique characteristics so that all possible correlations could be explored.

7.2 Drilled Shaft Testing Procedure

Upon each visit, it was necessary to collect as much data as possible about the individuality of each site as it pertains to shaft design, mix design, and construction procedure. Rebar cage spacing, cage length, shaft depth, casing diameter and concrete data were collected regularly, in addition to any other pertinent information. Acute observation of the construction process was critical to this research as well, and any unusual or exceptional practices were noted.

To accurately determine head differential, researchers used a weighted tape (Figure 7-1), similar to what drilled shaft inspectors use to measure rising concrete. Two tapes are dropped prior to the pouring of each truck, one inside the reinforcement cage and one outside, and initial readings taken with the top of the temporary casing as the reference. During pumping, head differential readings are taken every 30 seconds until the entire truck has been pumped, then the final height is measured. This process is repeated until the entire shaft has been poured.

7.3 Field Sites

In addition to the data collected from the Auburn test shafts (Chapter 6), three local Tampa bay area sites were visited. Figure 7-2 shows the locations of the each site visited. The construction of each site is detailed in the following sections.

7.3.1 Port of Tampa (Essex Cement)

Construction of cement silos at Berth 219 in the Port of Tampa (Figures 7-3 through 7-4) for Essex Cement Company demanded a foundation consisting of 177 drilled shafts with 3 foot diameters. The shafts were drilled to a depth of approximately 78 feet and utilized full length temporary casing with a sidewall thickness of 1/2 inch. The reinforcing cages were 52 feet in length and designed to terminate at the beginning of the rock socket. Stirrup spacing and mix specifications yielded a CSD of approximately 27. Each shaft was poured via a pump truck and had a volumetric requirement equivalent to 2 concrete trucks. Slump ranges for each truck fell between 8.5 and 9.5 inches. Due to the high water table, apparent in Figure 7-4, wet construction methods (natural slurry) were implemented.

The collection of 4 data sets were completed for this site. Figure 7-5 presents a graph of the CSD ratio versus measured head differential. It is interesting to note that the head differentials vary regardless of a constant CSD ratio. This suggests that another variable, rising concrete (uphole) velocity, could be key in measuring wet concrete behavior. Figure 7-6 shows the uphole velocity versus measured head differential, however, it is difficult to derive any trend from such a small amount of data.

7.3.2 Crosstown Expressway Reversible Lanes Bridge

Construction on the Crosstown Expressway Reversible Lanes Bridge began in 2003 (Figures 7-7 and 7-8). The bridge is designed to facilitate 3 lanes of traffic westward into Tampa during morning hours, then reverse flow eastward during the afternoon. The bridge utilizes a mono-pier foundation system, meaning that each column rests atop a single large diameter drilled shaft. Shaft diameters range from 4 to 8 feet with depths up to 80 feet. By nature of the design, the drilled shafts require a large amount of reinforcement with tightly-spaced cages; CSD ratios for all shafts were 6 and slumps ranged from 7 to 9 inches.

Three shafts were investigated at two points along the route that offered a significant variation in the construction atmosphere. The first two shafts (167 and 156) were 6 and 8 feet in diameter, respectively, and located within close proximity to an already-existing roadway (Site 1). The third shaft (18) was 8 feet in diameter and positioned over a waterway (Site 2). Construction methods were similar to those used at the Port of Tampa in that a full length temporary casing was vibrated to the rock layer, the reinforcing cages were designed to terminate at the rock socket, and concrete was pumped from concrete trucks through a tremie. The large diameter shafts required upwards of 15 concrete trucks and 5 hours to complete, affording several data sets per shaft.

Plotting the recorded head differential against the uphole velocity reveals a trend between shafts of similar CSD ratios (Figure 7-9). This evidence suggests that perhaps a family of curves exists for different ranges of CSD ratios. Also, it is evident that another variable may affect the range of head differential fluctuation within a particular shaft, for it is easily seen that the differential range in shaft 156 is twice as large as the range for shafts 18 and 167.

While waiting between the arrival of concrete trucks, differential measurements were taken in the stagnant boreholes. In time periods of up to one hour, it was observed that the concrete differential did not decrease appreciably.

7.3.3 Alagon Condominiums

The Alagon is a 21-story condominium overlooking Hillsborough Bay. The foundation of this luxurious high-rise consists of 140 drilled shafts ranging from 2 to 5 feet in diameter with CSD ratios of 10 and slumps of 8.5 to 9 inches. Shaft lengths vary depending on the elevation to rock (26 to 40 ft). The shafts were constructed in a similar fashion to those of the Port of Tampa and the Crosstown Expressway, with the exception that concrete was placed with a hopper instead of a pump truck (Figure 7-10). This deviation afforded an opportunity to examine very high uphole velocities due to the relatively small shaft diameters and large capacity of the hopper.

Differential heights were measured at the end of each bucket pour due to safety concerns, thereby altering the standard measuring procedure. Since uphole velocities were unable to be calculated, data collected for this site was not included in the final analysis. Also, most shafts unexpectedly required more than two bucketfuls. Differential readings taken during these pours revealed that the surface of the advancing concrete actually fell between buckets. Since cased construction method was used, voids within the rock socket may have opened and allowed concrete to escape from the borehole.

7.4 Head Differential Summary

Figure 7-11 summarizes the data collected from the Port of Tampa, Crosstown, and NGES site (TS-4) as a function of the CSD. Clearly the CSD is not the only parameter affecting the build-up of concrete head inside the reinforcing cage. Figure 7-12 shows the same data as a function of velocity for each group of common CSDs observed. Second-order trends appear to exist for different CSD values, verifying the concept that head pressure is directly proportional to the square of the velocity head. Given a constant uphole velocity, a drastic difference in head differential occurs as the CSD increases from 6 to 8; once the CSD increases beyond 8, the head differential does not significantly decrease (Figure 7-13).

Since there is little alternate configurations for a given reinforcement cage design and concrete flow rate is highly uncontrollable, it is more rational to adjust the coarse aggregate size so as to minimize concrete build-up inside the cage. This is preferable given the ease in which a thick layer of sediment can be deposited at sand contents approaching 4%.

Figure 7-1. Weighted Tape (left) used in Taking Head Differential Measurements (right)

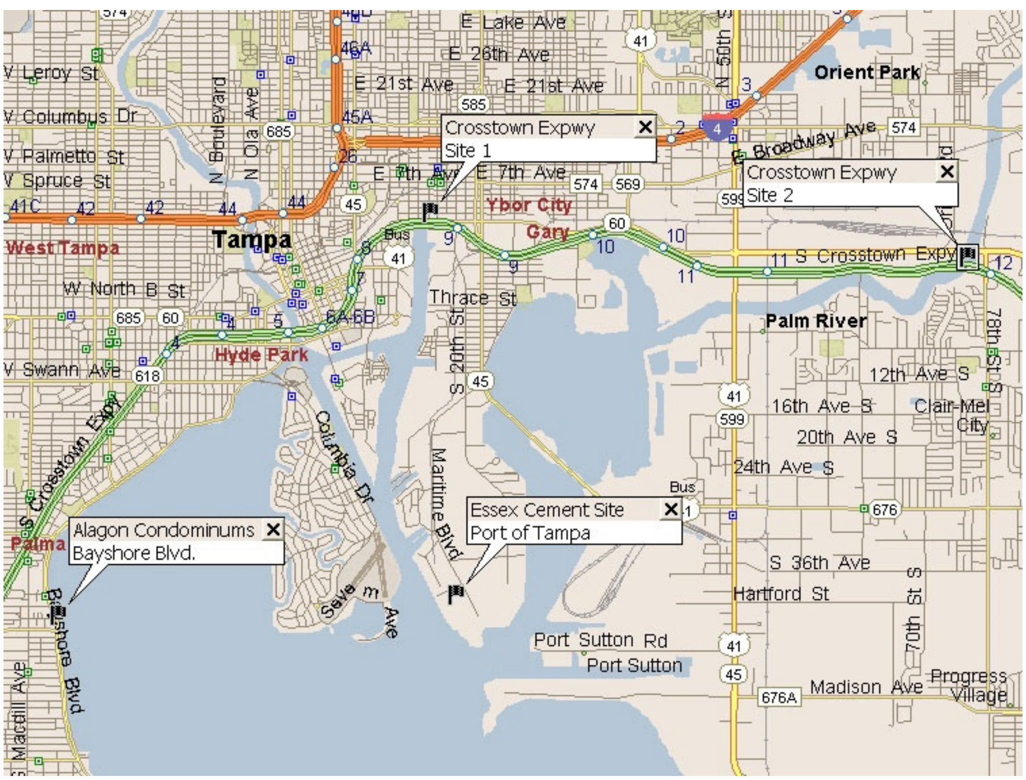

Figure 7-2. Location of the 4 Sites Visited for Head Differential Readings

Figure 7-3. Cage Installation at the Port of Tampa

Figure 7-4. High Water Table Visible at the Port of Tampa

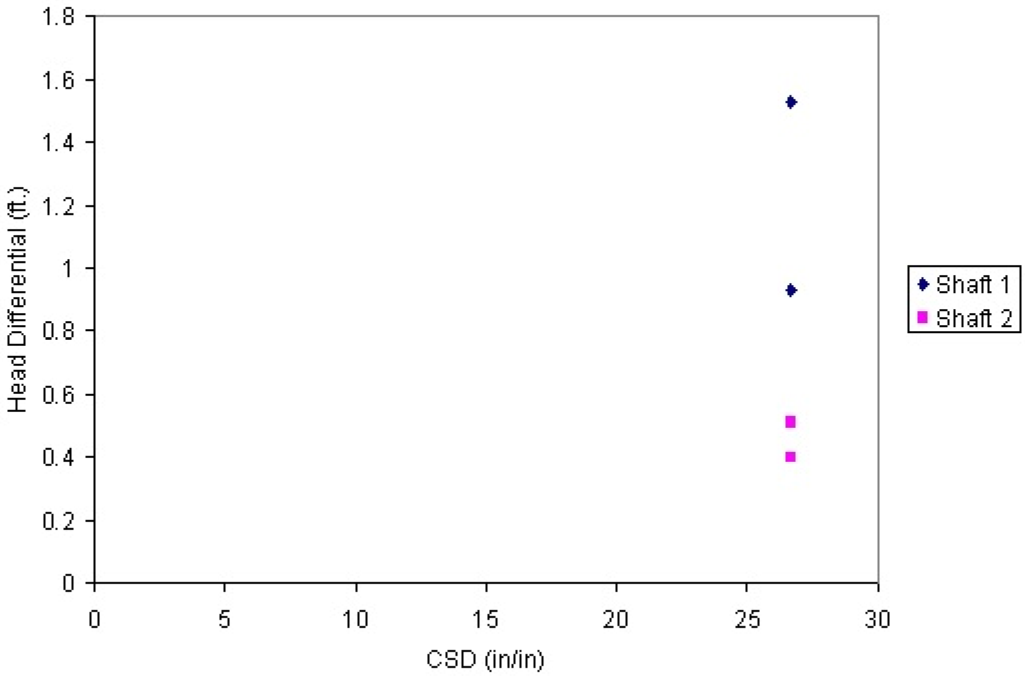

Figure 7-5. Head Differential as a Function of the CSD Ratio for the Port of Tampa

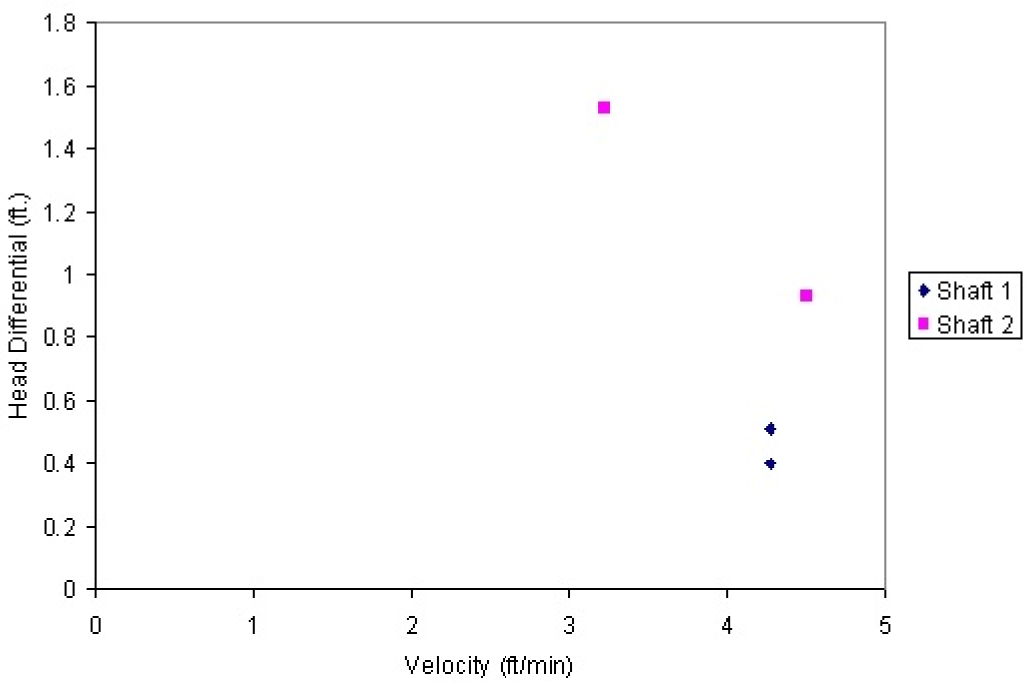

Figure 7-6. Head Differential as a Function of the Velocity for the Port of Tampa

Figure 7-7. Mono-pier Cage Placement at the Crosstown Expressway

Figure 7-8. Head Differential Measurements at the Crosstown Expressway

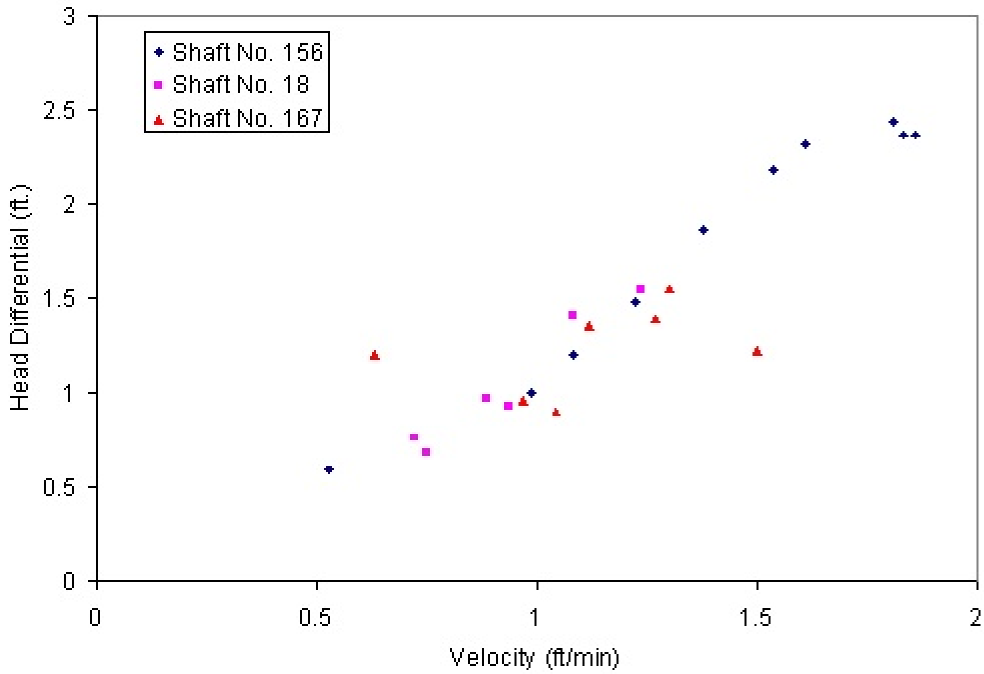

Figure 7-9. Head Differential as a Function of Velocity for the Crosstown Expressway

Figure 7-10. Alagon Bucket Pours (left) and Field Measurements (right)

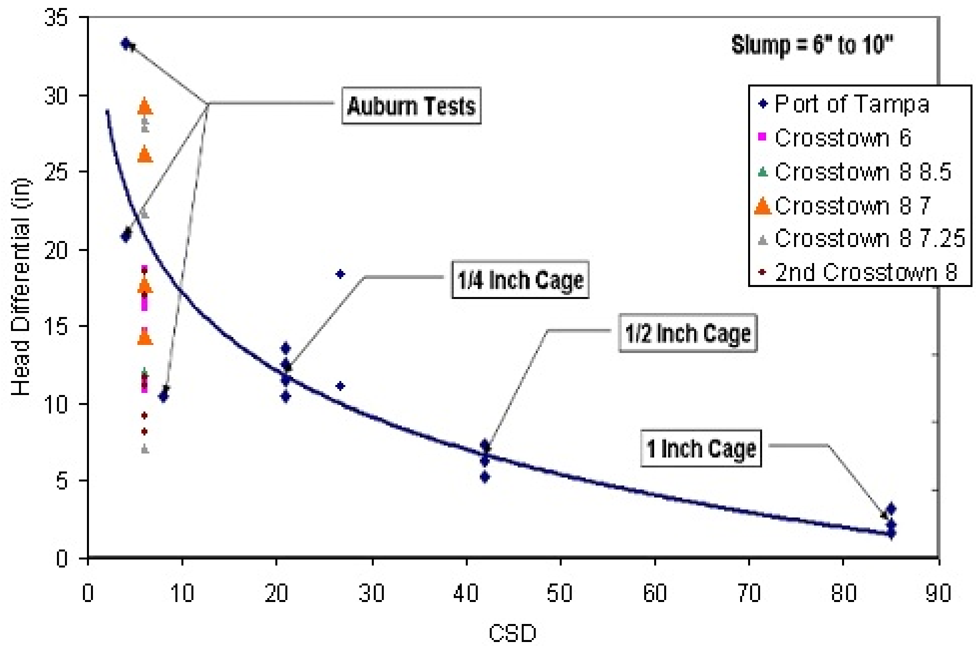

Figure 7-11 Field Measurements Combined with Lab Data

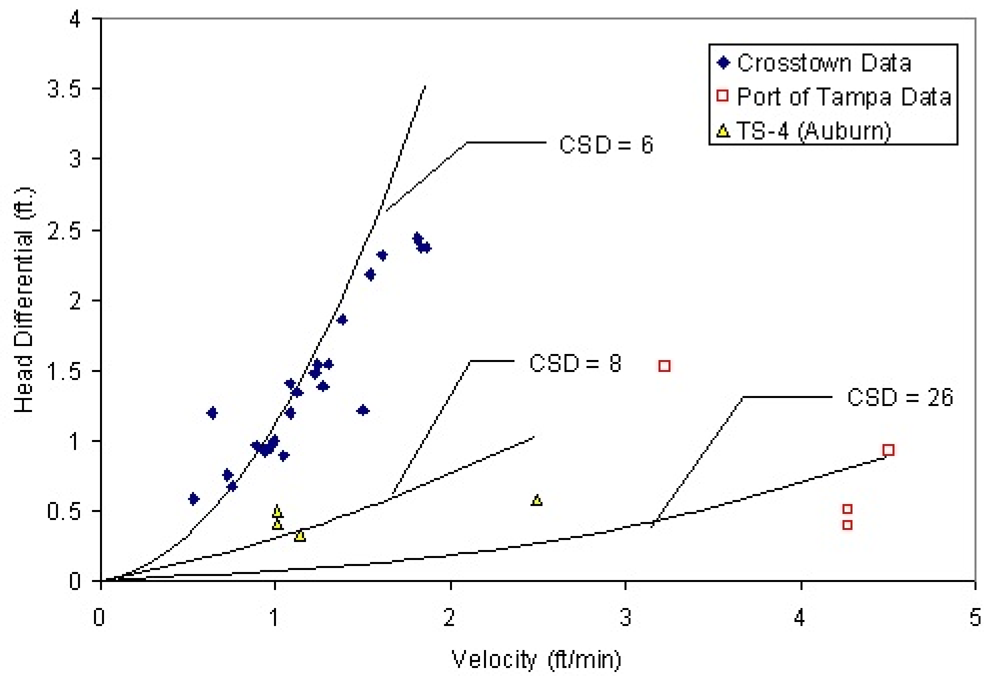

Figure 7-12. Summary of Head Differential as a Function of Velocity

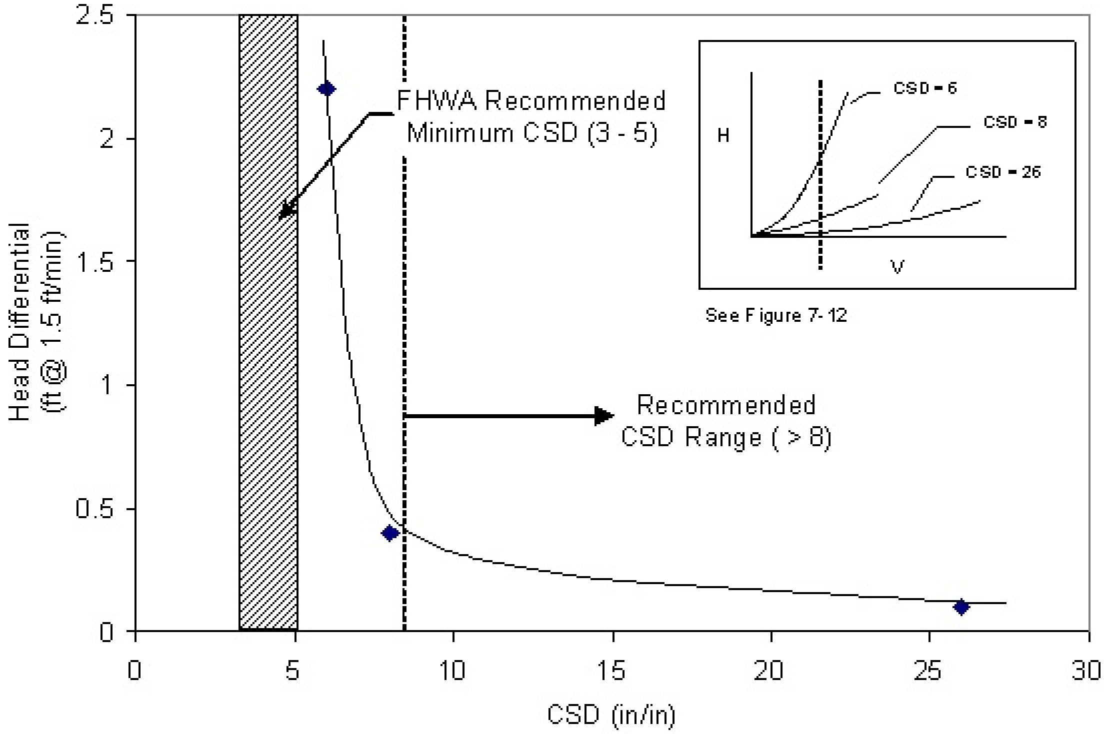

Figure 7-13. Recommended CSD Range to Minimize Head Differential

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General Conclusions

The results from this study revealed that the location of the water table was not the primary cause of anomalies in the drilled shafts, but rather concrete properties and slurry properties showed more significant effects. However, the presence of the water table requires sufficient slurry head over and above the water table and cannot be dismissed as being an important factor. As such, the scope of the project was widened to investigate the effects of other factors which would cause anomalies within drilled shafts. The following parameters were investigated: borehole cleanliness, construction techniques, slump loss during concrete placement, reinforcement cage spacing, slurry properties, sand content, and borehole open time. The project was divided into five different laboratory and field testing phases: (1) Lateral Pressure Cell testing, (2) Frustum Confining Vessel testing, (3) Slurry/Sand Fallout testing, (4) Full-scale Drilled Shaft testing, and (5) Full-scale Concrete Head Differential measurements. A significant finding which potentially affects each of the above areas was the way in which concrete flows from the tremie pipe. Figure 8-1 helps to illustrate the difference between the previously conceived flow and that observed in this study.

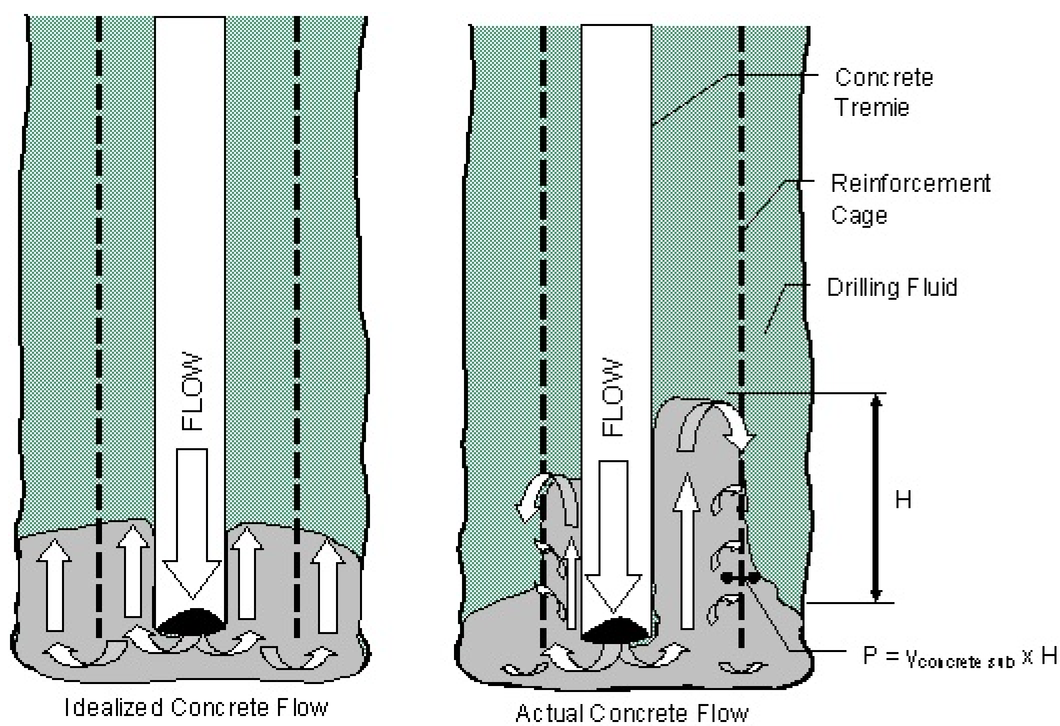

Figure 8-1. Comparison of Idealized Concrete Flow with Observed

8.1.1 Lateral Pressure Cell Testing

Three series of Lateral Pressure Cell testing were conducted to investigate: (1) the effect of slump on lateral pressures, (2) reinforcement cage spacing and wet construction method, and (3) concrete head differentials inside versus outside the reinforcement cage. Series I of testing showed that the lateral pressure of a final pour does not vary significantly with slump (from 7 to 11 inches). Series I and II of tests revealed the complexity of the flow of concrete out of the tremie and into the borehole. Although it is commonly assumed in practice that concrete rises in a uniform manner, this was never observed in the LPC. To the contrary, the material between the tremie pipe and the reinforcing cage always rose to a higher level than the material on the outside of the cage. This concrete head differential is affected by the geometry of the reinforcing cage and also by the size and shape of the largest particles in the concrete mix. Looking at the ratio of the smallest clear spacing in the cage to the largest particle diameter of the mix, the CSD ratio was addressed. It was found that for CSD values that fall well within the currently acceptable design guidelines, large head differentials would develop. Compounding this problem is that under the influence of a slurry head in the borehole, the concrete leaving the tremie is further impeded and this head differential grows significantly larger. As the lab scale study only qualitatively addressed the relationship, field testing was subsequently conducted to quantify the occurrence.

While the existence of a head differential between the inner and outer cage material is not itself a problem, the likelihood that anomalies will occur in the finished shaft increases as the magnitude of this differential increases. This is due to the motion of the concrete column. Instead of a uniform surface rising from the bottom of the excavation, a differential surface exists. For all cases studied, the material inside the cage and in contact with the tremie rose, in an arched fashion, to a much higher level that the material outside the cage. At some critical differential, the material on the inside would slough downward and outward through the cage. Segregation of the concrete constituents as well as entrapment of loose debris in the borehole each become a major concern.

8.1.2 Frustum Confining Vessel Testing

Three series of Frustum Confining Vessel testing were conducted to investigate: (1) the effects of water table location, (2) extraction rate of the temporary casing, and (3) the effects of slump loss during concrete placement. The first series of testing showed that the water table location had little to no effect on the quality of the drilled shaft. The second series of tests revealed the higher extraction rates caused anomalies to form in the shaft (tapered shafts) and impacted the final dimension (diameter) of the shafts. The third series of tests demonstrated that the skin friction capacity of a drilled shaft is very sensitive to slump loss that occurs during construction. Although currently the FDOT allows the slump of drilled shaft concrete to fall to as low as 4 inches during construction, data presented in this study shows that below a slump of 5 inches there is significant reduction in the mobilized skin friction of the model shafts (Figure 4-28). A 5 inch slump caused a 50% reduction and a 4 inch slump caused a 75% reduction. All of the shafts presented herein were constructed at values of slump either within the FDOT 7 to 9 inch recommendation, or above. However, the slump of several of the shafts was allowed to decrease prior to pulling the temporary casing. The magnitude of this decrease varied, but slumps as low as 3.5 inches were obtained. Relating the skin friction capacity of these shafts as a function of the slump at the time of casing extraction, it is evident that the FDOT 4 inch lower bound is too low. Such low slump conditions produce a slip-forming effect when the casing is extracted leaving an annular void similar in shape and volume of the now-absent casing.

8.1.3 Slurry/Sand Fallout Testing

Four series of Concrete Pour Simulator testing were conducted to investigate sand fallout in bentonite slurry as a function of wait time, pour velocity and slurry properties. The first series of testing was conducted on slurry with 1% sand content and demonstrated that uphole velocity had minimal effect on sand fallout. The second series of testing was conducted on slurry with 2% sand content and showed that low viscosity (approximately 30-32 second Marsh cone) slurry had much higher sand fallout relative to higher viscosity mixes (39-42 second Marsh cone). It also establishes that longer wait times increase fallout of sand as finer material is able to settle in the borehole. The third series of tests used slurry with a sand content of 4%. These tests showed that wait time had minimal effect on fallout, with approximately the same amount for periods of up to 12 hours. The fourth series of tests, conducted on slurry with 8% sand content confirmed the trend of constant fallout regardless of wait time. It is apparent, as well, that as slurry sand content increases, the amount of fallout relative to the total amount of sand increases. In general, an undisturbed column of slurry deposited most of the material within the first 2 hours (which was up to 50% of the total suspended sand).

8.1.4 NGES Full-Scale Testing

The field testing at the NGES site test involved construction and quality monitoring of five full scale drilled shafts. The parameters investigated, included three different concrete mix designs, construction method (dry versus wet hole), and reinforcement cage spacing. Several non-destructive tests (cross-hole, impact echo, thermal, and post grouting) were performed on each test shaft to show the effectiveness of detecting known anomalies placed in the shafts. The drilled shafts served to reinforce what was found in the LPC testing. Video footage of the complex flow of concrete out of the tremie and through the cage was obtained along with tape measurements to the top of the concrete (inside and outside the reinforcement cage). This footage agreed well with the information recorded in the LPC testing program. Differentials as large as 2 feet were measured and the arching and sloughing of the rising concrete column was recorded using the borescope. With the strong agreement between lab and field results, the argument can be made that more care must be taken during the design stage to ensure the CSD ratio is adequate to avoid large head differentials during construction. By making some very small changes in either clear rebar spacing or maximum coarse aggregate size, this CSD ratio could be adjusted such that the construction techniques employed in the field have much less effect on the integrity of the finished shaft.

8.1.5 Full-Scale Head Differential Testing

Full-scale head differential measurements were conducted (involving over 40 data sets) at three different construction sites in the Tampa area. The objective of this testing was to quantify head differential as a function of CSD ratio, uphole velocity, and concrete properties. The full scale testing first showed that CSD ratio did not have a direct correlation with head differential. Instead, it was found that as upward velocity increased, larger head differentials were observed. It was apparent that shafts with similar CSD ratios also showed similar head differentials with respect to velocity. The largest head differentials (upwards of 2.5 feet) were observed for shafts with CSD ratios of approximately 6. As the CSD ratio decreased, the head differential increased. Head differential measurements taken during wait times between concrete trucks revealed that minimal decrease occurred for periods of up to one hour. This suggests that, in conjunction with rapid sand fallout, inclusions may form when pouring resumes, trapping settled material in the lower level concrete cover / annular areas.

8.2 Recommendations

(1) CSD > 5. Current FHWA recommendations suggest that a CSD as low as 3 is reasonable. However, concrete flow observed in this study support an increase in this recommended value. This applies to structural, geotechnical, and materials engineers alike. As the size and configuration of rebar in drilled shaft cages cannot always be altered, smaller maximum aggregate diameter may be appropriate. Smaller-sized coarse aggregate such as #7 stone should be considered for tremie-placed drilled shaft construction. This would help to increase the CSD and thereby lower the potential head differential that could develop. This means less concrete back pressure would be needed to adequately penetrate the cage. Where practical, this limit should be applied to the worst case in the cage such as spliced cage segments.

(2) Slump Loss > 4 inches. Slump during construction should be carefully monitored. The FDOT 7 to 9 inch criteria seems acceptable for concrete placement, but the 4 inch lower bound on slump loss may be too lax. The lab study showed that a final slump of 3.5 inches to 4 inches were on the verge of being unable to construct, and produced near zero side shear. Further, due to the interactive nature of the FCV's constant bladder pressure, 4 inch slump may be under-conservative for field applications. Perhaps a more stringent value of 4.5 to 5 inches would be more appropriate for full-scale, cased construction. Considering the availability of admixtures on the market, maintaining a slump of 7 to 9 inches throughout the entire pour is not unreasonable, and perhaps this should be considered as well.

(3) Slurry Sand Content < 1%. When constructing shafts using slurry stabilized constructions, the sand content at the time of concreting should be reduced to a more restrictive value of 1% from the present value of 4%. The concrete pour simulator used in this study showed that large amounts of sand can be suspended in the slurry but an almost equal amount can fall out of suspension within the first 2 hours (e.g. a 60ft excavation at 4% sand content and with a 38% fallout would deposit up to 11inches of accumulation). Further, other Southeastern states have recently adopted similar requirements. This can be met by either de-sanding or by maintaining two separate slurry tanks, one with clean "concreting slurry" and a second with "excavating slurry." The "excavating slurry" is then exchanged with the "concreting slurry" prior to concreting. Slurry properties during excavation are far less important with regard to sand content provided that sufficient slurry head is maintained. Failure to maintain a stable borehole results in sloughing and a reduction in soil strength. In such cases, the anticipated design capacity is unrelated to actual capacity.

(4) Pre-charge Tremie with Mortar. At the onset of concreting the tremie charged with concrete is lifted to begin concrete flow. The first several feet of concrete rise at the base of the excavation segregates to an unknown degree due to the violent mixing action in the zone surrounding the end of the tremie. In some cases this segregation can cause blockage as the course aggregates clump together, but in all cases some segregation occurs. A solution to this occurrence involves pre-charging the tremie with neat cement or a fine-aggregate mortar mix before beginning the concreting. In many instances, when pump trucks are used, a slicking, neat cement mix is run through prior to concrete pumping which would suffice.

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APPENDICES

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Appendix A. Section 455 A and C with Commentary

SECTION 455 A and C with Commentary
STRUCTURES FOUNDATIONS

Appendix A (Continued).

Index

A. GENERAL

455-1 General Requirement.

The Contractor may examine available soil samples and/or rock cores obtained during the soil boring operations at the appropriate District Materials Office.

Appendix A (Continued).

455-1.1 Protection of Existing Structures: When the plans require foundation construction operations in close proximity to existing structures, take all reasonable precautions to prevent damage to such structures. The requirements described herein apply to all types of structures (on or off the right-of-way) that may be adversely affected by foundation construction operations (including phase construction) due to vibrations, ground loss, ground heave, or dewatering. Protect utilities as described in 7-11.6.

Monitor structures for settlement in a manner approved by the Engineer, recording elevations to 0.001 foot [0.5 mm]. Monitor the following structures:

(1) shown in the plans.

(2) within a distance, in feet, of pile driving operations equal to 0.5 times the square root of the hammer energy, in foot-pounds [in meters, of pile driving operations equal to 4.14 times the square root of the hammer energy, in kilojoules]. Take required measurements before the initiation of driving and then daily on days when driving occurs or as indicated in the plans and weekly for two weeks after driving has stopped.

(3) within a distance of ten shaft diameters or the estimated depth of excavation, whichever is greater.

(4) within a distance of three times the depth of excavation for the footing.

Obtain the Engineer's approval of the number and location of monitoring points. Take elevation;

(1) before beginning construction,

(2) daily during the driving of any casings, piling, or sheeting,

(3) weekly for two weeks after

stopping driving,
(4) during excavation,
(5) during blasting,
(6) or as directed by the Engineer.

Notify the Engineer of any movements detected and immediately take any remedial measures required to prevent damage to the existing structures.

Except as noted herein, employ a qualified Specialty Engineer to survey all structures, or portions thereof, within:

(1) a distance, in feet, of pile driving operations equal to 0.25 times the square root of the hammer energy, in foot-pounds [in meters, of pile driving operations equal to 2.07 times the square root of the hammer energy, in kilojoules]

(2) a distance of ten shaft diameters or the estimated depth of excavation, whichever is greater

(3) three times the excavation depth

(4) or as shown in the plans

The Department will make the necessary arrangements to provide right-of-way entry for the Contractor's engineer to survey. Adequately document the condition of the structures and all existing cracks with descriptions and pictures. Prepare two reports documenting the condition of the structures: one report before beginning foundation construction operations and a second report after completing foundation construction operations. The Department will take ownership of both reports. Do not perform pre-driving and post-driving surveys of the condition of bridges owned by the Department except when shown in the

Appendix A (Continued).

Contract Documents.

When shown in the Contract Documents, employ a qualified Specialty Engineer to monitor and record vibration levels during the driving of casings, piling, sheeting, or blasting operations. Provide vibration monitoring equipment capable of detecting velocities of 0.1 in/s [2.5 mm/s] or less.

Upon detecting settlement of 0.005 foot [1.5 mm], vibration levels reaching 0.5 in/s [13 mm/s], levels otherwise shown in the Contract Documents, or damage to the structure, immediately stop the source of vibrations, backfill any open drilled shaft excavations, and contact the Engineer for instructions.

When the plans require excavations for construction of footings or caps, the Contractor is responsible for evaluating the need for, design of, and providing any necessary features to protect adjacent structures. Construct sheeting and shoring as detailed in the plans. When sheeting and shoring are not detailed in the plans, employ a Specialty Engineer to design the sheeting and shoring, and to sign and seal the plans and specification requirements. Send these designs to the Engineer for his record before
beginning construction.

Also, when shown in the Contract Documents or when authorized by the Engineer, install the piling to the depth required to minimize the effects of vibrations or ground heave on adjacent structures by approved methods other than driving (preformed holes, predrilling, jetting, etc.). In the event the Department authorizes the use of preformed pile holes to meet this requirement, the Department will pay for this work as described in 455-5.9.3.

Do not drive piles within 200 feet [60 m] of concrete less than two days old unless authorized by the Engineer.

Also if not otherwise provided in the plans, the Contractor is responsible for evaluating the need for, design of, and providing all reasonable precautionary features to prevent damage, including, but not limited to, selecting construction methods and procedures that will prevent damaging caving of the shaft excavation and monitoring and controlling the vibrations from construction activities, including driving of casings, driving of sheeting, and blasting.

When shown in the plans or directed by the Engineer, install a piezometer near the right-of-way line and near any structure that may be affected by lowering the ground water when dewatering is required. Monitor the piezometer and record the ground water elevation level daily. Notify the Engineer of any ground water lowering near the structure of 12 inches [300 mm] or more.

Appendix A (Continued).

455-1.2 Excavation: Complete all excavation of the foundations prior to installing piles or shafts unless otherwise authorized by the Engineer. After completing pile/shaft installation, remove all loose and displaced materials from around the piles/shafts, leaving a clean, solid surface. Compact the soil surface on which concrete is to be placed or which will support the forming system for the concrete to a density not less than 90% of the maximum density as determined by AASHTO T 180, and which will support the load of the plastic concrete without settling or causing the concrete to crack, or as shown in the Contract Documents. The Engineer will not require the Contractor to compact for excavations made below water for seals or when the footing or cap or forming system (including supports) does not rest on the ground surface.

455-1.2.1 Abutment (End Bent) Fill: Place and compact the fill before installing end-bent piling/shafts, except when:

(1) driving specified test piling in end bents or,

(2) the plans show uncased piles through proprietary retaining wall fills.

When installing piles/shafts or casing prior to placing fill, take necessary precautions to prevent displacement of piles/shafts during placing and compacting fill materials within 15 feet [4.5 m] of the piles/shafts or casing. Reference and check the position of the piles/shafts or casing at three approximately equal intervals during construction of the embankment.

Place embankment material in 6 inch [150 mm] loose lifts in the 15 foot [4.5 m] area around the piles/shafts or casing. Compact embankment material within the 15 foot [4.5 m] area adjacent to the piles/shafts or casing to the required density with compaction equipment weighing less than 1,000 pounds [450 kg]. When installing piles/shafts prior to the completion of the surrounding fills, do not cap them until placing the fills as near to final grade as possible, leaving only the necessary working room for construction of the caps.

Provide permanent casings for all drilled shafts through mechanically stabilized fills (for example, behind proprietary retaining walls). Provide permanent casings for conventional fills 15 foot [4.5 m] or greater in height except when shown otherwise in the plans. Do not provide permanent casings for conventional fills less than 15 foot [4.5 m] in height except when shown in the plans or when directed by the Engineer. Install temporary casings through the completed conventional fill when permanent casings are not required.

Provide permanent casings, if required, before the fill is placed extending a sufficient distance into the existing ground to provide stability to the casings during construction of the abutment fill.

Appendix A (Continued).

455-1.3 Cofferdams: Construct cofferdams as detailed in the plans. When cofferdams are not detailed in the plans, employ a Specialty Engineer to design cofferdams, and to sign and seal the plans and specification requirements. Send the designs to the Engineer for his records before beginning construction.

Provide a qualified diver and a safety diver to inspect the conditions of the foundation enclosure or cofferdam when the Contract Documents require a seal for construction. Equip these divers with suitable voice communications, and have them inspect the foundation enclosure and cofferdam periphery including each sheeting indentation and around each piling or drilled shaft to ensure that no layers of mud or other undesirable materials were left above the bottom of seal elevation during the excavation process. Also have the divers check to make sure the surfaces of the piles or drilled shafts are sufficiently clean to allow bond of the concrete down to the minimum bottom of seal elevation. When required, ensure that there are no mounds of stone, shell, or other authorized backfill material left after placement and grading. Assist the Engineer as required to ensure that the seal is placed as specified and evaluate the adequacy of the foundation soils or rock. Correct any deficiencies found by the divers. Upon completion of inspection by the divers, the Department may also elect to inspect the work before authorizing the Contractor to proceed with subsequent construction operations. Furnish the Engineer a written report by the divers indicating the results of their underwater inspection before requesting authorization to place the seal concrete.

Appendix A (Continued).

455-2 Static Compression Load Tests.

455-2.1 General: Employ a professional testing laboratory, or Specialty Engineer with prior load test experience, to conduct the load test in compliance with these Specifications, to record all data, and to furnish reports of the test results to the Engineer except when the Contract Documents show that the Department will supply a Geotechnical Engineer to provide these services.

Use a load for the test that is three times the design load, the maximum load shown in the plans or as designated by the Engineer (within the limits of the test equipment provided), or the failure load, whichever occurs first.

Do not apply test loads to piles sooner than 48 hours (or the time interval shown in the plans) after driving of the test pile or reaction piles, whichever occurs last.

Allow up to four weeks after the last load test for the analysis of the load test data and to

provide all the estimated production drilled shaft tip elevations. If the Contractor is willing to construct production shafts in areas designated by the Engineer, he shall set shaft tip elevations as required to keep him working, beginning one week after the final load test.

Do not begin static load testing of drilled shafts until the concrete has attained a compressive strength of 3,400 psi [23.5 MPa]. The Contractor may use high early strength concrete to obtain this strength at an earlier time to prevent testing delays.

Load test piles/shafts in the order

directed by the Engineer. The Department will furnish
certain load test equipment and/or personnel when shown in the plans. Inspect all equipment to be furnished by the Department at least 30 days prior to use, and notify the Engineer of any equipment that is not in satisfactory operating condition. The Department will consider any necessary repairs ordered by the Engineer to place the equipment in satisfactory operating condition as Unforeseeable Work. Provide the remainder of the equipment and personnel needed to conduct the load tests. Unless shown otherwise in the Contract Documents, provide all equipment, materials, labor, and technical personnel required to conduct the load tests, including determination of anchor reaction member depths. In this case, provide a loading apparatus designed to accommodate the maximum load plus an adequate safety factor.

While performing the load test, provide safety equipment, and employ safety procedures consistent with the latest approved practices for this work. Include with these safety procedures adequate support for the load test plates and jack to prevent them from falling in the event of a release of load due to hydraulic failure, test shaft failure, or any other cause.

Include in the bid the cost of transporting load test equipment and instrumentation supplied by the Department from their storage location to the job site and back. Handle these items with care. The Contractor is responsible for the safe return of these items. After completion of the static load tests, return all Department furnished equipment in satisfactory operating condition. Repair all damage to the test equipment furnished by the Department to the satisfaction of the Engineer. Clean all areas of rust on structural steel items, and repaint those areas in accordance with Section 561. Return all load test equipment supplied by the Department within 30 days after completing the load tests.

The Contractor is responsible for the equipment from the time it leaves its storage area until the time it is returned. During this time, insure the equipment against loss or damage for the replacement cost thereof (the greater of $150,000 or the amount shown in the plans) or for the full insurable value if replacement cost insurance is not available.

Notify the Engineer at the preconstruction conference or no later than 30 days before beginning test pile installation of the proposed testing schedule so that items supplied by the Department may be reserved. Notify the Department at least ten working days before pick-up or return of the equipment. During pick-up, the Department will complete a checklist of all equipment placed in the Contractor's possession. The Department will later use this checklist to verify that the Contractor has returned all equipment. Provide personnel and equipment to load or unload the equipment at the Department's storage location. Provide lifting tongs or nylon slings to handle Department owned test girders. Do not perform cutting, welding, or drilling on Department owned girders, jacks, load cells, or other equipment.

Appendix A (Continued).

455-2.2 Loading Apparatus: Provide an apparatus for applying the vertical loads as described in one of the following:

(1) As shown and described in the Contract Documents.

(2) As supplied by the Contractor, one of the following devices designed to accommodate a load at least 20% higher than that shown in the Contract Documents or described herein for test loads:

(a) Load Applied by Hydraulic Jack Acting Against Weighted Box or Platform: Construct a test box or test platform, resting on a suitable support, over the pile, and load it with earth, sand, concrete, pig iron, or other suitable material with a total weight greater than the anticipated maximum test load. Locate supports for the weighted box or platform at least 6 feet [2 m] or three pile/shaft diameters, whichever is greater, measured from the edge of the pile or shaft to the edge of the supports. Insert a hydraulic jack with pressure gauge between the test pile or shaft and the underside of the reaction beam, and apply the load to the pile or shaft by operating the jack between the reaction beam and the top of the pile or shaft.

(b) Load Applied to the Test Pile or Shaft by Hydraulic Jack Acting Against Anchored Reaction Member: Construct reaction member anchorages as far from the test piles/shafts as practical, but in no case closer than the greater of 3 pile/shaft diameters or 6 feet [2 m] from the edge of the test pile/shaft. Attach a girder(s) of sufficient strength to act as a reaction beam to the upper ends of the anchor piles or shafts. Insert a hydraulic jack with pressure gauges between the head of the test pile/shaft and the underside of the reaction beam, and apply the test load to the pile/shaft by operating the jack between the reaction beam and the pile/shaft head.

If using drilled shafts with bells as reaction member anchorages, locate the top of the bell of any reaction shaft anchorage at least three shaft diameters below the bottom of the test shaft.

(c) Combination Devices: The Contractor may use a combination of devices (a) and (b), as described above, to apply the test load to the pile or shaft.

(d) Other Systems Proposed by the Contractor and Approved by the Engineer:
When necessary, provide horizontal supports for loading the pile/shaft, and space them so that the ratio of the unsupported length to the minimum radius of gyration of the pile does not exceed 120 for steel piles, and the unsupported length to the least cross-section dimension does not exceed 20 for concrete piles or drilled shafts. Ensure that horizontal supports provide full support without restraining the vertical movement of the pile in any way.

When required by the Contract Documents, apply a horizontal load to the shaft either separately or in conjunction with the vertical load. Apply the load to the test shaft by hydraulic jacks, jacking against Contractor provided reaction devices. After receiving the Engineer's approval of the proposed method of load application, apply the horizontal load in increments, and relieve it in decrements as required by the Contract Documents.

Appendix A (Continued).

455-2.2.1 Modified Quick Test:

(a) Loading Procedure - Piles: Place the load on the pile continuously, in increments equal to approximately 5% of the maximum test load specified until approaching the failure load, as indicated by the measuring apparatus and/or instruments. Then, apply increments of approximately 2.5% until the pile "plunges" or attains the limiting load. The Engineer may elect to stop the loading increments when he determines the Contractor has met the failure criteria or when a settlement equal to 10% of the pile width or diameter is reached. Apply each load increment immediately after taking and verifying the complete set of readings from all gauges and instruments. Apply each increment of load within the minimum length of time practical, and immediately take the readings. Complete the addition of a load increment and the completion of the readings within five to 15 minutes. The Engineer may elect to hold the maximum applied load up to one hour.

Remove the load in decrements of about 10% of the maximum test load. Remove each decrement of load within the minimum length of time practical, and immediately take the readings. Complete the removal of a load decrement and the taking of the readings within five to 15 minutes. The Engineer may also require up to two reloading cycles with five loading increments and three unloading decrements. Record the final recovery of the pile until movement is essentially complete for a period up to one hour after the last unload interval.

(b) Failure Criteria and Safe-Load: Use the criteria described herein to

Appendix A (Continued).

establish
the failure load. The failure load is defined as the load that causes a pile top deflection equal to the calculated elastic compression plus 0.15 inch [3.8 mm] plus 1/120 of the pile minimum width or the diameter in inches [millimeters] for piles 24 inches [610 mm] or less in width, and equal to the calculated elastic compression plus 1/30 of the pile minimum width or diameter for piles greater than 24 inches [610 mm] in width. Consider the safe allowable load of any pile so tested as either 50% of the maximum applied load or 50% of the failure load, whichever is smaller.

Appendix A (Continued).

455-2.2.2 Loading Procedure - Shafts: Apply vertical loads concentric with the longitudinal axis of the tested shaft to accurately determine and control the load acting on the shaft at any time.

Place the load on the shaft continuously in increments equal to approximately 5% of the maximum test load specified until approaching the failure load, as indicated by the instruments. Then, apply increments of approximately 2.5% until the shaft "plunges" or attains the limiting load. The Engineer may elect to stop the loading increments when he determines that the failure criteria has been met or a settlement equal to 10% of the shaft width is reached. Apply each load increment immediately after taking and verifying the complete set of readings from all gauges and instruments. Apply each increment of load within the minimum length of time practical, and immediately take the instrument system readings. Complete the addition of a load increment and the taking of the instrument system readings within five to fifteen minutes. The Engineer may elect to hold the maximum applied load up to one hour.

Remove the load in decrements of about 10% of the maximum test load. Remove each decrement of load within the minimum length of time practical and take the instrument system readings immediately. Complete the removal of a load decrement and the taking of the instrument system readings within five to fifteen minutes. The Engineer may also require up to two reloading cycles with five loading increments and three unloading decrements. Record the final recovery of the shaft until movement is essentially complete for a period up to one hour after the last unload interval.

Use the criteria described herein to establish the failure load unless shown otherwise in the Contract Documents. The failure load is defined as follows:

(1) for shafts with diameters up to 24 inches [610 mm], the load that causes a shaft top deflection equal to the calculated elastic compression, plus 0.15 inch [4 mm], plus 1/120 of the shaft diameter in feet [millimeters],

(2) for shafts with diameters larger than 24 inches [610 mm], the load that causes a shaft top deflection equal to the calculated elastic compression, plus 1/30 of the shaft diameter.

Consider the safe allowable load of any shaft so tested as either 50% of the maximum applied load or 50% of the failure load, whichever is smaller.

Appendix A (Continued).

455-2.3 Measuring Apparatus: Provide an apparatus for measuring movement of the test piles/shafts that consists of all of the following devices:

(1) Wire Line and Scale: Stretch a wire as directed by the Engineer between two supports located at a distance at least:

(a) 10 feet [3 m] from the center of the test pile but not less than 3.5 times the pile diameter or width.

(b) 12 feet [3.7 m] from the centerline of the shaft to be tested but not less than three shaft diameters.

Locate the wire supports as far as practical from reaction beam anchorages. At over-water test sites, the Contractor may attach the wire line as directed by the Engineer to the sides of the service platform. Mount the wire with a pulley on one support and a weight at the end of the wire to provide constant tension on the wire. Ensure that the wire passes across the face of a scale mounted on a mirror attached to the test pile/shaft so that readings can be made directly from the scale. Use the scale readings as a check on an average of the dial readings. When measuring both horizontal and vertical movement, mount separate wires to indicate each movement, horizontal or vertical. Measure horizontal movements from two reference wires set normal to each other in a horizontal.

(2) Wooden Reference Beams And Dial Gauges: Attach wooden reference beams as detailed in the plans or approved by the Engineer to independent supports. For piles, install the greater of 3.5 times the pile diameter or width or 10 feet [3 m] from the centerline of the test pile. For drilled shafts install the greater of three

shaft diameters or 12 feet [3.7 m] from the centerline of the shaft to be tested. Locate the reference beam supports as far as practical from reaction beam anchorages. For over-water test sites, the Contractor may attach the reference beams as directed by the Engineer between two diagonal platform supports. Attach dial gauges, with their stems resting either on the top of the pile/shaft or on lugs or similar reference points on the pile/shaft, to the fixed beams to record the movement of the pile/shaft head. Ensure that the area on the pile/shaft or lug on which the stem bears is a smooth surface which will not cause irregularities in the dial readings.

For piles, the minimum acceptable method for measuring vertical movement is two dial gauges, each with 0.001 inch [0.025 mm] divisions and with 2 inch [50 mm] minimum travel, placed at 180 degrees or at the diagonal corners of the pile.

For shafts, ensure that three dial gauges, each with 0.001 inch [0.025 mm] divisions and with 2 inch [50 mm] minimum travel, placed at 120-degree intervals around the shaft, are the minimum acceptable method for measuring vertical movement. Ensure that four dial gauges, each with 0.001 inch [0.025 mm] divisions and with 2 inch [50 mm] minimum travel, placed at 90 degree intervals
are the minimum required for measuring horizontal movement.

(3) Survey Level: As a check on the dial gauges, determine the elevation of a point near the top of the test pile/shaft (on plan datum) by survey level at each load and unload interval during the load test. Unless approved otherwise by the

Engineer, level survey precision is 0.001 foot [0.3 mm]. Alternately, the surveyor may read an engineer's 50 scale attached near the pile/shaft head. Determine the first elevation before applying the first load increment; make intermediate readings immediately before a load increment or an unload decrement, and after the final unload decrement that completely removes the load. Make a final reading at the time of the last recovery reading or as directed by the Engineer.

For over-water test sites, when shown in the plans or directed by the Engineer, the Contractor shall drive an H pile through a 36 inch [914 mm] casing to provide a stable support for the level and to protect it against wave action interfering with level measurements. Provide a suitable movable jig for the surveyor to stand. Use a jig that has a minimum of three legs, has a work platform providing at least 4 feet [1.2 m] width of work area around the casing, and is approved by the Engineer before use. The described work platform may be supported by the protective casing when approved by the Engineer.

Appendix A (Continued).

455-2.4 Load Test Instrumentation:

(1) General: The intent of the load test instrumentation is to measure the test load on top of the pile and, when provided in the Contract Documents, its distribution between side friction and end bearing to provide evaluation of the preliminary design calculations and settlement estimates and to provide information for final pile/shaft length design. Ensure that the instrumentation is as described in the Contract Documents.

When requested by the Engineer, provide assistance during installation of any instrumentation supplied by the Department. Supply 110 V, 60 Hz, 30 A of AC electric power in accordance with the National Electric Code to each test pile/shaft site during the installation of the instrumentation, during the load testing, and during any instrumented redrives ordered by the Engineer.

Place all of the internal instrumentation on the rebar cage before installation in the test shaft. Construct the rebar cage at least two days before it is required for construction of the test shaft. Provide assistance during installation of instrumentation supplied by the Department, including help to string, place, and tie the instrumentation and any assistance needed in moving or repositioning the cage to facilitate installation. Place the rebar cage in one segment complete with its instrumentation. The Engineer may require multiple lift points and/or a suitable "stiffleg" (length of H pile or other suitable section) to get the cage in a vertical position without causing damage to the instrumentation. Successfully demonstrate the lifting and handling procedures before the installing instrumentation.

(2) Hydraulic Jack and Load Cell: Provide hydraulic jack(s) of adequate size to deliver the required test load to the pile/shaft unless shown otherwise in the plans. Before load testing begins, furnish a certificate from a reputable testing laboratory showing a calibration of gauge readings for all stages of jack loading and unloading for jacks provided. Ensure that the jack has been calibrated within the preceding six months unless approved otherwise. Recalibrate the jack after completing load testing if so directed by the Engineer. Ensure that the accuracy of the gauge is within 5% of the true load.

Provide an adequate load cell approved by the Engineer that has been calibrated within the preceding six months. Provide an approved electrical readout device for the load cell. Before beginning load testing, furnish a certificate from a reputable testing laboratory showing a calibration of readings for all stages of loading and unloading for load cells furnished by the Contractor. Ensure that the accuracy of the load cell is within 1% of the true load.

If the Department supplies the Contractor with the jack and/or load cell, have the equipment calibrated and include the cost in the cost for static load test.

(3) Telltales: When shown in the Contract Documents, provide telltales that consist of an unstressed steel rod placed, with appropriate clearance and greased for reducing friction and corrosion, inside a constant-diameter pipe that rests on a flat plate attached to the end of the pipe at a point of interest shown in the plans.

Construct telltales in accordance with details shown in the Contract Documents. Install dial gauges reading to 0.001 inch [0.025 mm] with 1 inch [25 mm] minimum travel as directed by the Engineer to measure the movement of the telltale with respect to the top of the pile/shaft.

(4) Embedded Strain Gauges: When shown in the Contract Documents, provide strain gauges which shall be placed in the test shaft to measure the distribution of the load. Ensure that the type, number, and location of the strain gauges are as shown in the plans or as directed by the Engineer. Use strain gauges that are waterproof and have suitable shielded cable that is unspliced within the shaft.

455-2.5 Support Facilities: Furnish adequate facilities for making load and settlement readings 24 hours per day. Provide such facilities for the instrumented area, and include lighting and shelter from rain, wind, and direct sunlight.

Appendix A (Continued).

455-2.6 Load Test Personnel Furnished by the Contractor: Provide a certified welder, together with necessary cutting and welding equipment, to assist with the load test setup and to make any necessary adjustments during the load test. Provide personnel to operate the jack, generators, and lighting equipment, and also provide one person with transportation to assist as required during load test setup and conducting of the load tests. Provide personnel required to read the dial gauges, take level measurements, and conduct the load test, except when the Contract Documents show that the Department will provide these personnel.

455-2.7 Cooperation by the Contractor: Cooperate with the Department, and ensure that the Department has access to all facilities necessary for observation of the conduct and the results of the test.

Appendix A (Continued).

455-2.8 Required Reports: Submit a preliminary static load test report to the Engineer within five days after completing the load test. When the Contract Documents do not require internal instrumentation, submit the final report within ten days after completing the load test. Furnish the final report of test results for internally instrumented shafts within 30 days after completing the load test. Include in the report of the load test the following information:

(1) A tabulation of the time of, and the amount of, the load and settlement readings, and the load and recovery readings taken during the loading and unloading of the pile.

(2) A graphic representation of the test results, during loading and unloading of pile top movement as measured by the average of the dial gauge readings, from wireline readings and from level readings.

(3) A graphic representation of the test results, when using telltales, showing pile compression and pile tip movement.

(4) The estimated failure and safe loads according to the criteria described herein.

(5) Remarks concerning any unusual occurrences during the loading of the pile.

(6) The names of those making the required observations of the results of the load test, the weather conditions prevailing during the load test, and the effect of weather conditions on the load test.

(7) All supporting data including jack and load cell calibrations and certificates and other equipment requiring calibration.

(8) When the Contract Document

requires internal instrumentation of the shaft, furnish all of the data taken during the load test together with instrument calibration certifications. In addition, provide a report showing an analysis of the results of axial load and lateral load tests in which soil resistance along and against the shaft is reported as a function of deflection.

Provide the necessary report(s) prepared by a qualified Geotechnical Engineer registered in Florida as a Specialty Engineer except when the Contract Documents show that the Department will provide a Geotechnical Engineer.

C. DRILLED SHAFTS

455-13 Description.

Construct drilled shaft foundations consisting of reinforced, or unreinforced when indicated in the plans, concrete drilled shafts with or without bell footings.

455-14 Materials.

455-14.1 Concrete: For all concrete materials, meet the requirements of Section 346. Use concrete that is specified in the plans.

C455-14.1: The concrete design should be in accordance with the plans, but a typical water-cement ratio is less than or equal to 0.41. Section 346-3.2 states that the initial slump must be from 7” to 9”. The required concrete strength varies, but should be specified in the plans. Typically, 3400 psi is used in slightly aggressive environments and 4000 psi in moderate to extremely aggressive environments. High-early cement is typically used in the mix if the shaft must be load tested before the strength reaches 3400 psi (455-2.1). Air contents range from 3 to 6 percent.

Appendix A (Continued).

455-14.2 Reinforcing Steel: Meet the reinforcing steel requirements of Section 415. Ensure that reinforcing steel is in accordance with the sizes, spacing, dimensions, and the details shown in the plans.

455-15 Construction Methods and Equipment.

455-15.1 General Requirements:

455-15.1.1 Templates: Provide a fixed template, adequate to maintain shaft position and alignment during all excavation and concreting operations, when drilling from a barge. Do not use floating templates (attached to a barge). The Engineer will not require a template for shafts drilled on land provided the Contractor demonstrates satisfactorily to the Engineer that shaft position and alignment can be properly maintained. The Engineer will require a fixed template, adequate to maintain shaft position and alignment during all excavation and concreting operations, for shafts drilled on land when the Contractor fails to demonstrate satisfactorily that he can properly maintain shaft position and alignment without use of a template.

Appendix A (Continued).

455-15.1.2 Drilled Shaft Installation Plan: At the preconstruction conference or no later than 30 days before beginning drilled shaft construction, submit a drilled shaft installation plan for approval by the Engineer. Include in this plan the following details:

C455-15.1.2: The purpose of a Drilled Shaft Installation Plan is to detail the method of construction and develop a contingency should obstacles arise during construction. It should provide sufficient information to the Engineer so that he/she may evaluate the construction equipment and methods, as well as the experience of key personnel. The plan should be submitted no later than 30 days prior to installation of any test hole, test shaft, or production shaft. This allows the Engineer and Contractor sufficient time to review the plan for potential problems before construction begins.

The plan may initially be tentative, but it should be refined during test shaft construction. Once the plan is refined to a working state, strict adherence to the plan should be followed.

1. Name and experience record of drilled shaft superintendent or foreman in responsible charge of drilled shaft operations. Ensure that the person in responsible charge of day to day drilled shaft operations has prior experience constructing shafts similar to those described in the Contract Documents. Final approval by the Engineer will be subject to performance in the field.

In most cases, the success or failure of a drilled shaft installation depends on the experience of the person in charge, despite having proper equipment and a feasible plan. The Contractor should provide a resume which includes a summary of experience installing similar drilled shafts. An important factor is knowing how well the Contractor handles issues such as unusual site conditions and equipment breakdown. Suitable references should be provided to help answer these questions.

Appendix A (Continued).

2. List and size of proposed equipment, including cranes, drills, augers, bailing buckets, final cleaning equipment, desanding equipment, slurry pumps, core sampling equipment, tremies or concrete pumps, casings, etc.

This allows the Drilled Shaft Inspector and the contractor an itemized list to verify that all necessary equipment is present at the beginning of construction and that the equipment complies with Section 455-15.1.3.

All technical information and specifics on the equipment should be included to allow the Engineer to make a preliminary evaluation of the equipment's ability to perform in compliance with the specifications.

3. Details of sequence of construction operations and sequence of shaft construction in bents or shaft groups.

Sufficient details should be provided for the Engineer to fully understand the order of the Contractor's anticipated operation and to evaluate whether the specified times will be met.

Simultaneous construction of adjacent shafts should be avoided in areas where limestone formations are present. Difficulties in placing the concrete may arise if unforeseen caverns connecting the excavations allow concrete to flow from one hole to another. If concrete flowing into the adjacent excavation freefalls, aggregate segregation or soil entrapment may occur. Also, the concrete level may drop and increase the risk of a breached tremie.

4. Details of shaft excavation methods.

Sufficient detail of the Contractor's proposed excavation methods should be provided to the Engineer for evaluation of the adequacy of the tools and whether the time schedule will be upheld.

Appendix A (Continued).

5. Details of slurry, including proposed methods to mix, circulate, desand, test methods, and proposed testing laboratory to document test results.

The type of slurry, when it will be introduced, what head will be maintained during construction, and storage procedures should be included. Refer to Section 455-15.8.1.

As specified in Section 455-15.8.2, contractors must supply individuals from a testing laboratory if mineral slurries are specified in the Drilled Shaft Installation Plan, but the contractor may supply an experienced individual to conduct the tests if natural slurries are specified.

6. Details of proposed methods to clean shaft after initial excavation.

The Contractor should provide the details of the shaft cleaning operations. This should include the use of any special cleaning buckets, airlifts (where suitable), submersible pumps, etc. See Section 455-15.11.4.

7. Details of shaft reinforcement, including methods to ensure centering/required cover, cage integrity during placement, placement procedures, cage support, and tie downs.

8. Details of concrete placement, including proposed operational procedures for concrete tremie or pump, including initial placement, raising during placement, and overfilling of the shaft concrete. Also provide provisions to ensure proper final shaft cutoff elevation.

9. Details of casing removal when removal is required, including minimum concrete head in casing during removal.

The minimum intended concrete head should be indicated if the casings will be extracted prior to an overpour.

10. Required submittals, including shop drawing and concrete design mixes.

Slump loss data, the laboratory that performed the tests, and the date that they were performed should be submitted along with the mix design.

Appendix A (Continued).

11. Details of any required load tests, including equipment and procedures, and recent calibrations for any jacks or load cells.

Unless the Department supplies the load test equipment, design calculations and drawings should be included. Erection details or assembly procedures should always be included.

12. Methods and equipment proposed to prevent displacement of casing and/or shafts during placement and compaction of fill.

13. Details of environmental control procedures used to prevent loss of slurry or concrete into waterways or other protected areas.

Special permit requirements and proposed methods of slurry disposal must be addressed for the Engineer's evaluation.

14. Other information shown in the plans or requested by the Engineer.

Additional things to consider in the installation plan are:

· What if the concrete pump breaks in the middle of a pour? When faced with the option of removing concrete from the hole and pouring again or preparing the surface of the cold joint, the contractor may decide to provide an extra pump.
· What if the concrete trucks are delayed in the middle of a pour?
· What if the crane breaks during the pour?

Appendix A (Continued).

The Engineer will evaluate the drilled shaft installation plan for conformance with the Contract Documents. Within 20 days after receipt of the plan, the Engineer will notify the Contractor of any additional information required and/or changes that may be necessary in the opinion of the Engineer to satisfy the Contract Documents. The Engineer will reject any part of the plan that is unacceptable. Submit changes agreed upon for reevaluation. The Engineer will notify the Contractor within seven days after receipt of proposed changes of their acceptance or rejection. All approvals given by the Engineer are subject to trial and satisfactory performance in the field.

It is usually helpful for the Engineer to meet with the Contractor and discuss the results of the review. Discussing the results may save time by decreasing the number of resubmittals before an acceptable Drilled Shaft Installation Plan is formulated.

Demonstrate the adequacy of methods and equipment during construction of the first drilled shaft which shall be an out-of-position test hole generally constructed as an unreinforced shaft. Drill this test hole in the position shown in the plans or as directed by the Engineer and drill to the maximum depth for any production shaft shown in the plans. Failure to demonstrate the adequacy of methods or equipment to the Engineer is cause for the Engineer to require appropriate alterations in equipment and/or method by the Contractor to eliminate unsatisfactory results. Provide any additional test holes required to demonstrate the adequacy of methods or equipment at no expense to the Department. Make no changes in methods or equipment after initial approval without the consent of the Engineer.

Test holes are generally constructed without reinforcement (refer to C455-18), away from the foundation area, and preferably in the area of greatest concern (i.e. work site's worst conditions, suspected artesian conditions, and suspected cavities in rock).

A separate test hole is not required for drilled shafts installed under mast arms, cantilever signs, overhead truss signs, high mast light poles or other miscellaneous structures. The first production shaft will serve as a test hole for determining acceptability of the installation method.

The methods and equipment used in the construction of the test shafts should not practically vary from what is used for the production shafts. A drilled shaft's capacity is largely dependent on the construction methods and equipment, so any variation could result in a shaft with a lower capacity than what is expected.

455-15.1.3 General Methods and Equipment: Perform the excavations required for the shafts and bell footings, through whatever materials encountered, to the dimensions and elevations shown in the Contract Documents, using methods and equipment suitable for the intended purpose and the materials encountered. Provide equipment capable of constructing shafts supporting bridges to a depth equal to the deepest shaft shown in the plans plus 15 foot [4.5 m] or plus three times the shaft diameter, whichever is greater, except when the plans require equipment capable of constructing shafts to a deeper depth. Provide equipment capable of constructing shafts supporting non-bridge structures, including mast arms, signals, signs and light supports to a depth equal to the deepest shaft shown in the plans plus 5 feet [1.5 m].

Construct drilled shafts according to the Contract Documents using generally either the dry method, wet method, casing method, or permanent casing method as necessary to produce sound, durable concrete foundation shafts free of defects. Use the permanent casing method only when required by the plans or authorized by the engineer. When the plans describe a particular method of construction, use this method except when permitted otherwise by the Engineer after field trial. When the plans do not describe a particular method, propose a method on the basis of its suitability to the site conditions and submit it for approval by the Engineer.

Set a suitable temporary removable surface casing. The minimum surface casing length is the length required to prevent caving of the surface soils and to aid in maintaining shaft position and alignment. The Engineer may require predrilling with slurry and/or overreaming to the outside diameter of the

C455-15.1.3:

If it becomes necessary to excavate deeper because of unexpected soil conditions or exceeding the time limit of slurry in the hole, providing equipment that is able to construct shafts deeper than what is specified in the plans can save time and money. Prior planning can possibly prevent poor performance. Refer to 455-15.11.5

casing to install the surface casing at some sites.

455-15.2 Dry Construction Method: Use the dry construction method only at sites where the ground water table and soil conditions, generally stiff to hard clays or rock above the water table, make it feasible to construct the shaft in a relatively dry excavation and where the sides and bottom of the shaft are stable and may be visually inspected by the Engineer prior to placing the concrete.

In applying the dry construction method, drill the shaft excavation, remove accumulated seepage water and loose material from the excavation and place the shaft concrete in a relatively dry excavation.

Use the dry construction method only when shaft excavations, as demonstrated in a test hole, have 12 inches [300 mm] or less of seepage water accumulated over a four hour period, the sides and bottom remain stable without detrimental caving, sloughing, or swelling for a four hour period, and the loose material and water can be satisfactorily removed prior to inspection and prior to placing concrete.

Use the wet construction method or the casing construction method for shafts that do not meet the requirements for the dry construction method.

Provide temporary surface casings to aid shaft alignment and position and to prevent sloughing unless the Engineer determines by demonstration that the surface casing is not required.

C455-15.2: Though dry excavations are the easiest to construct and inspect, due to Florida's high water table, this method is inappropriate for most sites. However, if the dry construction method is appropriate for a site, the Contractor should still have a contingency plan for the use of wet methods should conditions change from hole to hole.

Appendix A (Continued).

455-15.3 Wet Construction Method: Use the wet construction method at all sites where it is impractical to provide a dry excavation for placement of the shaft concrete.

The wet construction method consists of drilling the shaft excavation below the water table, keeping the shaft filled with fluid (mineral slurry, natural slurry or water), desanding and cleaning the mineral slurry and final cleaning of the excavation by means of a bailing bucket, air lift, submersible pump or other approved devices and placing the shaft concrete (with a tremie or concrete pump extending to the shaft bottom) which displaces the water or slurry during concreting of the shaft excavation. Provide temporary surface casings to aid shaft alignment and position and to prevent sloughing of the top of the shaft except when the Engineer declares that the surface casing is not required.

Where drilled shafts are located in open water areas, construct the shafts by the wet method using exterior casings extending from above the water elevation into the ground to protect the shaft concrete from water action during placement and curing of the concrete. Install the exterior casing in a manner that will produce a positive seal at the bottom of the casing so that there is no intrusion or extrusion of water or other materials into or from the shaft excavation.

If proposed, demonstrate in a test hole, that split casings can produce a positive seal for their entire length which will prevent intrusion of water into the shaft or extrusion of concrete or other materials from the shaft.

C455-15.3: Soil conditions dictate the necessity for using either mineral or natural slurry when drilling using the wet method. Water may be sufficient for shafts constructed in limerock or cemented soils, but mineral slurry is required when drilling through noncohesive soils or loose sands. Some soils may have sufficient clay so that a slurry capable of stabilizing the hole is formed during the drilling process.

A positive head of water or mineral slurry must be maintained at all times when using the wet construction method. Failure to do so may cause the hole to become unstable and the walls may collapse. Refer to C455-15.8.1. Earlier commentaries state that "several feet" must be maintained, but the recent specification defines 4 feet to be the minimum maintainable head when using mineral slurry.

Appendix A (Continued).

455-15.4 Casing Construction Method: Use the casing method at all sites where it is inappropriate to use the dry or wet construction methods without the use of temporary casings other than surface casings. In this method, the hole is advanced through caving material by the wet method as described above. When a formation is reached that is nearly impervious, place a casing in the hole and seal in the nearly impervious formation. Proceed with drilling as with the dry method to the projected depth. Proceed with the placement of the concrete as with the dry method except withdraw the casing after placing the concrete. In the event seepage conditions prevent use of the dry method, complete the excavation and concrete placement using wet methods.

Where drilling through materials having a tendency to cave, advance the excavation by drilling in a mineral slurry. In the event that a caving layer or layers are encountered that cannot be controlled by slurry, install temporary removable casing through such caving layer or layers. The Engineer may require overreaming to the outside diameter of the casing. Take whatever steps are required to prevent caving during shaft excavation including installation of deeper casings. If electing to remove a casing and replace it with a longer casing through caving soils, adequately stabilize the excavation with slurry or backfill the excavation. The Contractor may use soil previously excavated or soil from the site if backfilling the excavation. The Contractor may use other approved methods which will control the size of the excavation and protect the integrity of the foundation soils to excavate through caving layers.

Before withdrawing the casing, ensure that the level of fresh concrete is at such a

C455-15.4: It should be understood that though the specifications define separate construction methods for the use of temporary and permanent casing, casing is usually implemented with the wet and dry methods. Because of this, it is arguable that only two methods of construction exist, wet and dry, each implementing casing when necessary.

Refer to C455-15.10.4 and C455-17.2.

level that the fluid trapped behind the casing is displaced upward. As the casing is withdrawn, maintain the level of concrete within the casing so that fluid trapped behind the casing is displaced upward out of the shaft excavation without mixing with or displacing the shaft concrete.

The Contractor may use the casing method, when approved by the Engineer, to construct shafts through weak caving soils that do not contribute significant shaft shear resistance. In this case, place a temporary casing through the weak caving soils before beginning excavation. Conduct excavation using the dry construction method where appropriate for site conditions and the wet construction method where the dry construction method is not appropriate. Withdraw the temporary casing during the concreting operations unless the Engineer approves otherwise.

455-15.5 Permanent Casing Method: Use the permanent casing method when required by the plans. In this method, place a casing to the prescribed depth before beginning excavation. If the Contractor cannot attain full penetration, the Engineer may direct the Contractor to excavate through the casing and advance the casing until reaching the desired penetration. In some cases the Engineer may require the Contractor to overream the outside diameter of the casing before placing the casing.

Cut the casing off at the prescribed elevation upon reaching the proper construction sequence and leave the remainder of the casing in place.

C455-15.5: If the option is available, it is preferable to use temporary casing instead of permanent casing. Not only is cost savings incurred from reusing a temporary casing, but shaft side capacity is likely to be higher with a concrete/soil interface as opposed to a steel/soil interface. However, the capacity of a smooth interface left behind by an extracted casing may altogether be considerably less than a shaft having a rough interface left behind by the scarring of an auger.

Appendix A (Continued).

455-15.6 Excavations: The Contractor may extend drilled shaft excavations deeper by extra depth excavation when the Engineer determines that the material encountered while drilling the shaft excavation is unsuitable and/or is not the same as anticipated in the design of the drilled shaft.

Take cores when shown in the plans or directed by the Engineer to determine the character of the material directly below the shaft excavation. Cut the cores with an approved core barrel to a minimum depth of 5 feet [1.5 m] below the bottom of the drilled shaft excavation when completing the shaft excavation. The Engineer may require the Contractor to cut any core below the 5 foot [1.5 m] minimum depth and up to a total depth of 20 feet [6 m] below the bottom of the drilled shaft excavation.

The Engineer will inspect the cores and determine the depth of required excavation. When considered necessary by the Engineer, take additional cores.

When shown in the plans, prior to excavation, take a core (Shaft Excavation) through part or all of the shaft, to a depth up to 20 feet [6 m] below that shaft's planned tip elevation.

Use a core barrel designed:

(a) to cut a core sample from 4 to 6 inches [100 to 150 mm] in diameter,

(b) so that the sample of material cored can be removed from the shaft excavation

and the core barrel in an undisturbed state, and

(c) in sufficient length to provide core samples, as directed by the Engineer up to a depth of 20 feet [6 m] below the bottom of the drilled shaft excavation.

When called for in the plans, substitute Standard Penetration Tests (SPT) for coring.

In such cases, supply these tests at no additional cost per foot [meter] to the Department above that bid for core (shaft excavation).

Maintain a drilling log during shaft excavation and during coring operations that contains information such as the description of and approximate top and bottom elevation of each stratum encountered, depth of penetration, drilling time in each of the various strata, material description, and remarks. Classify, measure, and describe core samples in the drilling log. Place the core samples in suitable containers, identified by shaft location, elevation from and to, and job number, and deliver to the Department within 48 hours after cutting. Furnish two copies of the drilling log, signed by a designated representative of the Contractor and co-signed by a designated representative of the Department, to the Department at the time the shaft excavation is completed and accepted.

Provide areas for the disposal of unsuitable materials and excess materials as defined in 120-5 that are removed from shaft excavations, and dispose of them in a manner meeting all requirements pertaining to pollution.

When shown in the plans, excavate bells to form a bearing area of the size and shape shown. Bell outlines varying from those shown in the plans are permissible provided the bottom bearing area equals or exceeds that specified. If the diameter of the bell exceeds three times the shaft diameter, drill the excavation deeper as directed and form a new bell footing. Excavate bells by mechanical methods.

Furnish the additional drilled shaft concrete over the theoretical amount required to complete filling any excavations for bells and shafts which are larger than required by the plans or authorized by the Engineer, at no expense to the Department.

455-15.7 Casings: Ensure that casings are metal, or concrete when indicated in the plans, of ample strength to withstand handling and driving stresses and the pressure of concrete and of the surrounding earth materials, and that they are smooth and water tight. Ensure that the inside diameter of casing is not less than the specified size of shaft except as provided below. The Department will not allow extra compensation for concrete required to fill an oversize casing or oversize excavation.

The Engineer will allow the Contractor to supply casing with an outside diameter equal to the specified shaft diameter (O.D. casing) provided he supplies additional shaft length at the shaft tip.

Determine the additional length of shaft required by the following relationship:

$$\textbf{Additional Length} = \frac{(D_1 - D_2)L}{D_2}$$

C455-15.7:

Appendix A (Continued).

where:

D_1 = casing inside diameter specified = shaft diameter specified.

D_2 = casing inside diameter provided (D_2 = D_1 minus twice the wall thickness).

L = authorized shaft length below ground.

Appendix A (Continued).

Bear all costs relating to this additional length including but not limited to the cost of extra excavation, extra concrete, and extra reinforcing steel.

Remove all casings from shaft excavations except those used for the Permanent Casing Method. Ensure that the portion of casings installed under the Permanent Casing Method of construction below the shaft cut-off elevation remains in position as a permanent part of the Drilled Shaft. The Contractor may leave casings if in the opinion of the Engineer the casings will not adversely affect the shaft capacity in place. When casings that are to be removed become bound in the shaft excavation and cannot be practically removed, drill the shaft excavation deeper as directed by the Engineer to compensate for loss of capacity due to the presence of the casing. The Department will not compensate for the casing remaining. The Department will pay for the additional length of shaft under Item No. 455-88 [Item No. 2455-88] and the additional excavation under Item No. 455-125 [Item No. 2455-125].

Since temporary casing extraction usually occurs after the concrete is placed and the reinforcing cage is already in the hole, it may be impossible to excavated deeper to remove bound casing.

When the shaft extends above ground or through a body of water, the Contractor may form the portion exposed above ground or through a body of water, with removable casing except when the Permanent Casing Method is specified (see 455-23.10). When approved, the Contractor may form drilled shafts extending through a body of water with permanent or removable casings. However, for permanent casings, remove the portion of metal casings between an elevation 2 feet [0.6 m] below the lowest water elevation and the top of shaft elevation after the concrete is cured.

Appendix A (Continued).

Dismantle casings removed to expose the concrete as required above in a manner which will not damage the drilled shaft concrete. Dismantle removable casings in accordance with the provisions of 455-17.5.

Generally when removal of the temporary casing is required, do not start the removal until completing all concrete placement in the shaft. The Engineer will permit movement of the casing by rotating, exerting downward pressure, and tapping it to facilitate extraction, or extraction with a vibratory hammer. Extract casing at a slow, uniform rate with the pull in line with the axis of the shaft. Withdraw temporary casings while the concrete remains fluid.

When conditions warrant, the Contractor may pull the casing in partial stages. Maintain a sufficient head of concrete above the bottom of the casing to overcome the hydrostatic pressure of water outside the casing. At all times maintain the elevation of the concrete in the casing high enough to displace the drilling slurry between the outside of the casing and the edge of the hole while removing the casing.

The Contractor may use special casing systems in open water areas, when approved, which are designed to permit removal after the concrete has hardened. Design special casings so that no damage occurs to the drilled shaft concrete during their removal.

Though no maximum extraction rate is specified, once the casing extraction begins, it should progress in a continuous motion with no pauses. Pausing during a casing extraction can produce anomalies on the surface of the shaft.

Appendix A (Continued).

455-15.8 Slurry and Fluid in Excavation at Time of Concrete Placement:

C455-15.8: The term "slurry" is intended to represent the water/clay mixture that has not yet been placed into the excavation and introduced to cuttings. The term "fluid" refers to the water/clay mixture after it is placed in the excavation and contains cuttings.

455-15.8.1 Slurry: When slurry is used in an excavation, use only mineral slurry of processed attapulgite or bentonite clays. The Engineer will not allow polymer slurries. Use slurry having a mineral grain size such that it will remain in suspension and having sufficient viscosity and gel characteristics to transport excavated material to a suitable screening system. Use a percentage and specific gravity of the material to make the suspension sufficient to maintain the stability of the excavation and to allow proper placement of concrete. Ensure that the material used to make the slurry is not detrimental to concrete or surrounding ground strata. During construction, maintain the level of the slurry at a height sufficient to prevent caving of the hole. In the event of a sudden significant loss of slurry such that the slurry level cannot practically be maintained by adding slurry to the hole, delay the construction of that foundation until an alternate construction procedure has been approved.

Thoroughly premix the mineral slurry with clean fresh water prior to introduction into the shaft excavation. Ensure that the percentage of mineral admixture used to make the suspension is such as to maintain the stability of the shaft excavation. The Engineer will require adequate water and/or slurry tanks when necessary to perform the work in accordance with these Specifications. The Engineer will not allow excavated pits on projects requiring slurry tanks without the

C455-15.8.1: Only 12 states currently allow the use of polymer slurries (as of 2003). Though prior research has shown that they leave no filter cake and are environmentally friendly, undocumented instances of borehole collapse due to induced vibrations have discouraged their usage. In fact, very high head differentials between the slurry and the GWT must be maintained under normal conditions to maintain borehole stability (FHWA recommends at least 2 meters).

written permission of the Engineer. The Engineer will require adequate desanding equipment when shown in the Contract Documents. However, the Engineer will not require desanding equipment for drilled shafts for sign post or lighting mast foundations unless shown in the Contract Documents. Take the steps necessary to prevent the slurry from "setting up" in the shaft, including but not limited to agitation, circulation, and/or adjusting the composition and properties of the slurry. Provide suitable offsite disposal areas and dispose of all waste slurry in a manner meeting all requirements pertaining to pollution.

Provide a qualified professional soil testing laboratory approved by the Engineer to perform control tests using suitable apparatus on the mineral slurry mixture to determine the following parameters:

(a) Freshly mixed mineral slurry: Measure the density of the freshly mixed mineral slurry regularly as a check on the quality of the suspension being formed using a measuring device calibrated to read within ±0.5 lb/ft3 [±8 kg/m3].

(b) Mineral slurry supplied to the drilled shaft excavation: Perform the following tests on the mineral slurry supplied to the shaft excavation and ensure that the results are within the ranges stated in the table below:

Appendix A (Continued).

Item to be measured	Range of Results at 68ºF (20ºC)	Test Method
Density - in freshwater - in saltwater	64 to 73 lb/ft^3 (1030 to 1170 kg/m^3) 66 to 75 lb/ft^3 (1060 to 1200 kg/m^3)	Mud density balance FM 8-RP13B-1
Viscosity	28 to 40 seconds	Marsh Cone Method FM 8-RP13B-2
pH	8 to 11	Electric pH meter or pH indicator paper strips FM 8-RP13B-4
Sand Content	4 % or less	FM 8-RP13B-3

Appendix A (Continued).

The Contractor may adjust the limits in the above table(s) when field conditions warrant as successfully demonstrated in a Test Hole or with other methods approved by the Engineer. The Engineer must approve all changes in writing before the Contractor can continue to use them.

Perform tests to determine density, viscosity, and pH value to establish a consistent working pattern, taking into account the mixing process and blending of freshly mixed mineral slurry and previously used mineral slurry. Perform a minimum of four sets of tests to determine density, viscosity, and pH value during the first 8 hours mineral slurry is in use.

When the results show consistent behavior, discontinue the tests for pH value, and only carry out tests to determine density and viscosity during each four hours mineral slurry is in use. If the consistent working pattern changes, reintroduce the additional tests for pH value for the time required to establish consistency of the test values within the required parameters.

(c) Furnish reports of all mineral slurry tests required above, signed and sealed by a Specialty Engineer, representing the soil testing laboratory to the Department on completion of each drilled shaft.

(d) The Department may perform comparison tests as determined necessary during the mineral slurry operations.

During construction, maintain the level of mineral slurry in the shaft excavation within the excavation and at a level not less than 4 feet [1.2 m] above the highest expected piezometric water pressure along the depth of a shaft.

When using slurry, a minimum positive head of four feet should be maintained above the highest piezometric head. This will ensure that a minimum confining pressure of 1.7 psi acts on the shaft walls to prevent sloughing or caving. The calculations are based on the use of fresh water for the drilling slurry. The confining pressure for slurries with higher

At any time the wet construction method of stabilizing excavations fails, in the opinion of the Engineer, to produce the desired final result, discontinue this method of construction, and propose modifications in procedure or alternate means of construction for approval.

Slurry testing is not required for drilled shafts installed under mast arms, cantilever signs, overhead truss signs, high mast light poles or other miscellaneous structures.

unit weights (salt water or mineral slurries) will be slightly larger, thus more conservative.

Besides helping to suspend solids and increasing the confining pressure, mineral additives are used in slurries to reduce the permeability of various soil strata. This can reduce the loss of drilling fluid, making it easier to maintain a positive head.

If the slurry head is allowed to fall below the piezometric head, or the drilling fluid is introduced into the excavation after the water table is reached, an inward pressure may cause the shaft sides to cave. Once the walls of the excavation begin to cave, the confining pressure may not be sufficient to stabilize the “ceiling” of the cave. The soil may continue to cave, especially in cohesionless soils, until:

- The hole is lost
- Nearby structures are affected
- It interferes with construction of surrounding shafts

It is not required that premixed drilling slurry used for miscellaneous structures be tested prior to placement in the excavation, but once the fluid enters the hole, it must be tested in accordance with Section 455-15.8.2.

Appendix A (Continued).

455-15.8.2 Fluid In Excavation At Time Of Concrete Placement: Prior to placing concrete in any shaft excavation, ensure that heavily contaminated suspensions, which could impair the free flow of concrete from the tremie pipe, have not accumulated in the bottom of the shaft. Take samples of the fluid in the shaft from the base of the shaft and at intervals not exceeding 10 feet [3 m] up the shaft, using an approved sampling tool. Ensure that the density of the fluid in the shaft excavation prior to concreting is less than 75 lb/ft^3 [1,200 kg/m^3]. The Engineer will not require tests for pH and viscosity when mineral slurry is not used in the excavation. Ensure that projects that require desanding equipment have a sand content not greater than 4% as determined by FM 8-RP13B-3. Take whatever action is necessary to modify the fluid in the shaft excavation prior to placing the concrete to bring the fluid within the specification requirements.

When using mineral slurry, the applicable density test method and reporting requirements described in 455-15.8.1 apply to tests of slurry in the shaft prior to placing the concrete. Such tests shall be performed by an approved soil testing laboratory engaged by the Contractor in the presence of a representative of the Department. When mineral slurry is not used, testing may be performed by an experienced person furnished by the Contractor and approved by the Engineer. The Department may also perform comparison tests. Provide equipment for such comparison tests when requested by the Engineer.

C455-15.8.2: More recent extracts from the 2000 version of these standards and specifications used in teaching the Drilled Shaft Inspector's Qualification Course reiterate that "any shaft excavation" includes shafts under miscellaneous structures. A major culprit of shaft anomalies is a "dirty hole", meaning that the excavation contains a large amount of suspension. Though standards and specifications for miscellaneous structures are typically more lax, removing heavily contaminated suspension from the bottom of the excavation is a crucial step in ensuring the production of a good quality shaft. Refer to C455-15.10.4.

Reconsideration should be given to the maximum sand content of 4% when dealing with large diameter drilled shaft excavations. Though the theoretical volume of sand per unit slurry is the same, 4% by volume of a large shaft (9 ft diameter by 120 ft long) can reach a total sand weight as large as 14 tons. Further research is required to define a more reasonable limit on the sand content of larger shafts.

455-15.9 Tremies:

455-15.9.1 General: The requirements of 400-8.3 will apply when using a tremie to place drilled shaft concrete. The requirements of 400-7.7 will apply when using a pump to place drilled shaft concrete.

455-15.9.2 Dry Excavations: Ensure that the tremie for depositing concrete in a dry drilled shaft excavation consists of a tube of solid construction, a tube constructed of sections which can be added and removed, or a tube of other approved design. The Contractor may pass concrete through a hopper at the top of the tube or through side openings as the tremie is retrieved during concrete placement. Support the tremie so that the free fall of the concrete is less than 5 feet [1.5 m] at all times. If the free falling concrete causes the shaft excavation to cave or slough, control the movement of concrete by reducing the height of free fall of the concrete and/or reducing the rate of flow of concrete into the excavation.

455-15.9.2:

Free fall is considered as the distance the concrete has to fall to reach the hopper. The distance through the tremie pipe is not included in the 5 ft. limit.

455-15.9.3 Wet Excavations: Construct the tremie or pump line used to deposit concrete beneath the surface of water so that it is water-tight and will readily discharge concrete. Construct the discharge end of the tremie or pump line to prevent water intrusion and permit the free flow of concrete during placement operations. Ensure that the tremie or pump line has sufficient length and weight to rest on the shaft bottom before starting concrete placement.

C455-15.9.3:

The discharge end of the tremie must be plugged to prevent water or slurry from entering. If water or slurry is allowed to enter the tremie, the concrete may segregate (cement washes off of the aggregate) as the tremie is charged, or initially filled. Segregation of the concrete will cause a pile of gravel to be left at the bottom. As a result, shaft capacity will be reduced, and larger settlements may result before reaching capacity.

When the tremie is lowered to the bottom of the excavation, it is crucial to allow the discharge end to rest on the bottom. Once the tremie is charged, it should be lifted only a few inches. If the end is raised too high (several feet), the segregation problems discussed above may occur.

Currently, traveling plugs are not allowed when sealing the tremie. A common practice for sealing the tremie is to cut a circular plate slightly larger than the diameter of the discharge end. Plastic wrap is used to hold the plate on and seal the seam. Duct tape can be wrapped around the tremie to hold the plastic in place. When the tremie is lowered into the wet excavation, hydrostatic pressure will help keep the plug on and sealed. The plate will also prevent soil from plugging the end of the tremie when resting the discharge end on the bottom. Once the tremie is charged and lifted a few inches, the weight of the concrete will break the seal.

Appendix A (Continued).

Ensure that the discharge end of the tremie or pump line is entirely immersed in concrete at all times during placement operations. Ensure that the free fall of concrete into the hopper is less than 5 feet [1.5 m] at all times. Support the tremie so that it can be raised to increase the discharge of concrete and lowered to reduce the discharge of concrete. The Engineer will not allow rapid raising or lowering of the tremie to increase the discharge of the concrete. Maintain a continuous flow of concrete and a positive pressure differential of the concrete in the tremie or pump line at all times to prevent water or slurry intrusion into the shaft concrete.

Throughout placement, ensure that the tremie does not raise out of the concrete. If the concrete is allowed to free fall through slurry, pockets of slurry or soil may become trapped and create voids in the shaft.

455-15.10 Excavation and Drilling Equipment:

455-15.10.1 General: All shaft excavation is Unclassified Shaft Excavation and extra depth excavation is Unclassified Extra Depth Excavation. The Engineer will require Drilled Shaft Sidewall Overreaming when inspections show it to be necessary. These terms are defined in 455-15.10.2, 455-15.10.3, and 455-15.10.4, respectively.

Use excavation and drilling equipment having adequate capacity, including power, torque, and downthrust, and excavation and overreaming tools of adequate design, size, and strength to perform the work shown in the plans or described herein. When the material encountered cannot be drilled using conventional earth augers and/or underreaming tools, provide special drilling equipment, including but not limited to rock augers, core barrels, rock tools, air tools, blasting materials, and other equipment as necessary to continue the shaft excavation to the size and depth required. In the event blasting is necessary, obtain all necessary permits. The Contractor is responsible for the effects of blasting on already completed work and adjacent structures. The Engineer must approve all blasting.

Appendix A (Continued).

455-15.10.2 Unclassified Shaft Excavation: Unclassified Shaft Excavation is defined as all processes required to excavate a drilled shaft of the dimensions shown in the Contract Documents to the depth indicated in the plans or directed by the Engineer, completed and accepted. Include in the work all shaft excavation, whether the material encountered is soil, rock, weathered rock, stone, natural or man-made obstructions, or materials of other descriptions.

455-15.10.3 Unclassified Extra Depth Excavation: Unclassified Extra Depth Excavation is defined as all processes required to excavate a drilled shaft of plan dimensions below the elevation of the bottom of the shaft as indicated on the plans.

455-15.10.4 Drilled Shaft Sidewall Overreaming: Drilled Shaft Sidewall Overreaming is defined as the unclassified excavation required to roughen its surface or to enlarge the drilled shaft diameter due to softening of the sidewalls or to remove excessive buildup of slurry cake when slurry is used. Increase the shaft radius a minimum of 1/2 inch [15 mm] and a maximum of 3 inches [75 mm] by overreaming. The Contractor may accomplish overreaming with a grooving tool, overreaming bucket, or other approved equipment.

Meet the limit for depth of sidewall overreaming into the shaft sidewall material and the elevation limits between which sidewall overreaming is required.

C455-15.10.4:

Failure to remove excess slurry cake buildup on the walls of an excavation can lead to a significant decrease in shaft side capacity. Research has shown that the “plug flow” concept, where a rising concrete column scours the slurry off of the walls of the excavation, is not a good representation of how the concrete actually flows.

As the concrete column rises, the concrete tends to “mushroom up” between the outside of the tremie and the inside of the reinforcing cage. If the concrete has a relatively low slump (close to the 4" minimum) a head differential can develop from the inside to outside of the cage. Concrete will tend to fall through the cage and press into the walls of the excavation. If the sand content of the slurry is high, the bottom of the excavation is not clean, or a slurry cake is allowed to develop on the walls of the excavation, debris may become trapped between the concrete and soil as the concrete presses outward. Voids left on the perimeter of the shaft can invite corrosion and ultimately destroy the shaft.

Appendix A (Continued).

455-15.11 Inspection of Excavations:

455-15.11.1 Dimensions and Alignment: Provide equipment for checking the dimensions and alignment of each permanent shaft excavation. Determine the dimensions and alignment of the shaft excavation under the observation and direction of the Department. Generally check the alignment and dimensions by any of the following methods as necessary:

(a) Check the dimensions and alignment of dry shaft excavations using reference stakes and a plumb bob.

(b) Check the dimensions and alignment of casing when inserted in the excavation.

(c) Insert a casing in shaft excavations temporarily for alignment and dimension checks.

(d) Insert a rigid rod or pipe assembly with several 90-degree offsets equal to the shaft diameter into the shaft excavation for alignment and dimension checks.

Insert any casing, rod or pipe assembly, or other device used to check dimensions and alignment into the excavation to full depth.

455-15.11.2 Depth: Generally reference the depth of the shaft during drilling to appropriate marks on the Kelly bar or other suitable methods. Measure final shaft depths with a suitable weighted tape or other approved methods after final cleaning.

Appendix A (Continued).

455-15.11.3 Shaft Inspection Device: The Department, when shown in the plans, may use a shaft inspection device (SID) comprised of a television camera sealed inside a water-tight jacket to inspect the bottoms of the shafts. The Department may also use a sidewall sampler attached to the shaft inspection device to sample the sides of the shafts. Cooperate with the Department in using this device, including placing the device in position for inspection and removing it after the inspection. Furnish 110 V single phase current (minimum 30 A service), 220 V single phase current (minimum 15 A service), and a 150 psi [1.0 MPa] compressor (8 cfm [0.0038 m^3/s] minimum) to operate the SID. Include all cost related to the inspection device in the cost of drilled shaft items.

Provide the projected drilled shaft construction schedule to the Engineer at the preconstruction conference or no later than 30 days before beginning drilled shaft construction so that the SID may be scheduled. Include in the bid the cost of transporting the SID from its storage location to the job site and back. Notify the Department at least ten days prior to the desired pick-up date. During pick-up, the Department will complete a checklist of all equipment placed in the Contractor's possession. The Department will later use this checklist to verify that the Contractor has returned all equipment.

The Contractor is responsible for the device from the time it leaves its storage area until the time it is returned. During this time, insure the device against loss or damage for the replacement cost thereof (the greater of $400,000 or the amount shown in the plans) or for the full insurable value if replacement cost insurance is not available.

Return the device in good working

condition to its proper location within 30 days after completing the drilled shafts. Notify the Department at least ten working days prior to returning the SID.

455-15.11.4 Shaft Cleanliness Requirements: Adjust cleaning operations so that a minimum of 50% of the base of each shaft will have less than 1/2 inch [13 mm] of sediment at the time of placement of the concrete. Ensure that the maximum depth of sedimentary deposits or any other debris at any place on the base of the shaft excavation does not exceed 1 1/2 inches [40 mm]. The Engineer will determine shaft cleanliness by visual inspection for dry shafts, using divers or SID or other methods the Engineer deems appropriate for wet shafts.

When using slurry, meet the requirements of 455-15.8 at the time of concrete placement.

Ensure that the depth of sedimentary deposits or other debris does not exceed 1 inch [25 mm] over the base of the shaft when installing drilled shafts under mast arms, overhead truss signs, high mast light poles or other miscellaneous structures.

C455-15.11.4: Refer to C455-15.10.4.

Appendix A (Continued).

455-15.11.5 Time of Excavation: Any unclassified excavation work lasting more than 36 hours (measured from the beginning of excavation for all methods except the Permanent Casing Method, which begins at the time excavation begins below the casing) before placement of the concrete may require overreaming the sidewalls to the depth of softening or removing excessive slurry cake buildup as indicated by samples taken by the sidewall sampler or other test methods employed by the Engineer. Ensure that the minimum depth of overreaming the shaft diameter is 1/2 inch [13 mm] and the maximum depth is 3 inches [75 mm]. Provide any overreaming required at no expense to the Department when exceeding the 36-hour limit unless the time limit is exceeded solely to accomplish Unclassified Extra Depth Excavation ordered by the Engineer. The Department will pay the Contractor for authorized overreaming resulting from softening or excessive slurry cake buildup which is indicated by sidewall samples or other test methods employed by the Engineer during the initial 36-hour time period. The Department will pay the Contractor for authorized overreaming when sidewall samples indicate softening or excessive filter cake buildup in shaft excavations which exceed the 36-hour time limit in order to accomplish Unclassified Extra Depth Excavation ordered by the Engineer.

When using slurry, adjust excavation operations so that the maximum time that slurry is in contact with the bottom 5 feet [1.5 m] of the shaft (from time of drilling to concreting) does not exceed 12 hours. If exceeding the 12-hour time limit, overream the bottom 5 feet [1.5 m] of shaft at no additional expense to the Department prior to performing other operations in the shaft.

C455-15.11.5:

Refer to C455-15.10.4.

For drilled shafts installed under mast arms, cantilever signs, overhead truss signs, high mast light poles or other miscellaneous structures, all references to a 36-hour time limit is changed to a 12-hour time limit.

455-16 Reinforcing Steel Construction and Placement.

455-16.1 Cage Construction and Placement: Completely assemble and place as a unit the cage of reinforcing steel, consisting of longitudinal bars, ties, and cage stiffener bars, immediately after the Engineer inspects and accepts the shaft excavation and immediately prior to placing concrete. Tie all intersections of drilled shaft reinforcing steel with cross ties or "figure 8" ties. Use double strand ties or ties with larger tie wire when necessary. The Engineer will give final approval of the cage construction and placement subject to satisfactory performance in the field.

455-16.2 Splicing Cage: If the bottom of the constructed shaft elevation is lower than the bottom of the shaft elevation in the plans, extend a minimum of one half of the longitudinal bars required in the upper portion of the shaft the additional length. Continue the tie bars for the extra depth, spaced on 2 foot [0.6 m] centers, and extend the stiffener bars to the final depth. The Contractor may lap splice these bars or use unspliced bars of the proper length. Do not weld bars to the planned reinforcing steel unless shown in the Contract Documents.

Appendix A (Continued).

455-16.3 Support, Alignment, and Tolerance: Tie and support the reinforcing steel in the shaft so that the reinforcing steel will remain within allowable tolerances as specified in 455-8 and Section 415.

Use concrete wheels or other approved noncorrosive spacing devices near the bottom and intervals not exceeding 15 feet [4.5 m] up the shaft to ensure concentric spacing for the entire length of the cage. Do not use block or wire type spacers. Use a minimum of one spacer per 30 inches [750 mm] of circumference of cage with a minimum of three at each level. Provide concrete spacers, constructed as shown in the Contract Documents, at the bottom of the drilled shaft reinforcing cage to maintain the specified distance between the bottom of the cage and the bottom of the shaft is maintained. Use the number of bottom spacers as shown in the Contract Documents. Use spacers constructed of approved material equal in quality and durability to the concrete specified for the shaft. The Engineer will approve spacers subject to satisfactory performance in the field.

Check the elevation of the top of the steel cage before and after placing the concrete. If the rebar cage is not maintained within the specified tolerances, correct it as directed by the Engineer. Do not construct additional shafts until modifying the rebar cage support in a manner satisfactory to the Engineer.

Appendix A (Continued).

455-17 Concrete Placement.

455-17.1 General: Place concrete in accordance with the applicable portions of Sections 346 and 400, Standard Operating Procedures for Quality Control of Concrete, Subarticles 455-15.2, 455-15.3, 455-15.4, 455-15.5, 455-15.8, 455-15.9, and the requirements herein.

Place concrete as soon as possible after completing all excavation, cleaning the shaft excavation, inspecting and finding it satisfactory, and immediately after placing reinforcing steel. Continuously place concrete in the shaft to the top elevation of the shaft. Continue placing concrete after the shaft is full until good quality concrete is evident at the top of the shaft. Place concrete through a tremie or concrete pump using approved methods.

If the pressure head is lost during concrete placement for any reason, the Engineer may direct the Contractor to perform integrity testing at no expense to the Department.

455-17.2 Placement Time Requirements: The elapsed time for placing drilled shaft concrete includes the concrete mixing and transit time, the concrete placement time, and the time required to remove any temporary casing that causes or could cause the concrete to flow into the space previously occupied by the casing. Maintain a minimum slump of 4 inches [100 mm] throughout the elapsed time. Use materials to produce and maintain the required slump through the elapsed time that meets the class of concrete specified.

Provide slump loss tests that demonstrate to the Engineer that the concrete will maintain a 4 inch [100 mm] or greater slump for the anticipated elapsed time before beginning drilled shaft construction.

C455-17.2: In the 1991 version of this specification, placement was restricted to two hours unless the contractor could demonstrate that the mix would maintain a slump of 4” or greater over the longer placement time. Placement time was defined as the time from when the concrete entered the tremie or pump to removal of any temporary casings. The specifications no longer give a two hour time limit, but the concrete must still maintain a slump of 4” or greater throughout placement.

Since atmospheric conditions greatly affect the slump of concrete, slump loss demonstration tests should be performed under conditions similar to what is expected at the time of placement.

Though the specifications allow for a minimum slump of 4”, research shows a drastic reduction in shaft capacity, particularly in side friction, when the slump falls near or below 4”. The most probable explanation is the inability of a stiff concrete to flow outward and occupy the void left behind by an extracted casing. If this happens, it is possible that the stiff concrete will not apply the same lateral pressure on the walls of the excavation as if it were more fluid. If concrete strength requirements do not interfere with the mix design, the initial slump should be targeted as closely to the maximum 9” as possible.

Appendix A (Continued).

455-17.3 Forms: When the top of shaft elevation is above ground, form the portion of the shaft above ground with a removable form or another approved method to the dimensions shown in the plans.

When the shaft extends above the ground through a body of water, the Contractor may form the portion through the water with removable forms except when the Permanent Casing Method is specified.

When approved, the Contractor may form the portion through the water with permanent forms, provided the forms are removed from 2 feet [0.6 m] below the lowest water elevation to the top of shaft elevation.

C455-17.3:

When forming concrete shafts through water, an alternative to forming with permanent casing is to use two concentric temporary casings. The inner casing provides the form for the concrete. The annular region between the inner and outer casing is filled with sand, so that when the inner casing is removed after placement, the outer casing remains to protect the shaft against water and wave action. The sand acts as a barrier and prevents the concrete from bonding with the casing. After the concrete is cured, the outer casing is removed.

455-17.4 Riser Blocks: The Contractor may cast a riser block of equal diameter as the column and of a maximum height of 6 inches [150 mm] at the top of the completed shaft. When this option is chosen, extend any dowel steel above the top of shaft an additional 6 inches [150 mm].

455-17.5 Curing: Cure the top surface in accordance with 400-16, and construct any construction joint area as shown in the plans. Protect portions of drilled shafts exposed to a body of water from the action of water by leaving the forms in place for a minimum of seven days after casting the concrete. The Contractor may remove forms prior to seven days provided the concrete strength has reached 2,500 psi [17 MPa] or greater as evidenced by cylinder breaks.

455-18 Test Holes.

The Engineer will use the construction of test holes to determine if the methods and equipment used by the Contractor are sufficient to produce a shaft excavation meeting the requirements of the Contract Documents. During test hole excavations, the Engineer will evaluate the ability to control dimensions and alignment of excavations within tolerances; to seal the casing into impervious materials; to control the size of the excavation under caving conditions by the use of mineral slurry or by other means; to properly clean the completed shaft excavation; to construct excavations in open water areas; to establish elevations for belling; to determine the elevation of ground water; to place concrete meeting the requirements of these Specifications within the prescribed time frame; and to execute any other necessary construction operation. Revise the methods and equipment as necessary at any time during the construction of the test hole when unable to satisfactorily carry out any of the necessary operations described above or when unable to control the dimensions and alignment of the shaft excavation within tolerances.

Drill test holes out of permanent position at the location shown in the plans or as directed by the Engineer. Ensure that the diameter and depth of the test hole or holes are the same diameter and depth as the production drilled shafts. Do not reinforce the test hole, but fill the test hole with concrete in the same manner that production reinforced shafts will be constructed. The Contractor may backfill the test holes with suitable soil in a manner satisfactory to the Engineer. Leave the concreted test holes in place, except remove the top of the shaft to a depth of 2 feet [0.6 m] below the ground line. Use the same

C455-18.

The requirement to reinforce test holes varied from edition to edition of this specification, but the current edition does not require a reinforced test hole. The driving factor behind an unreinforced hole is the cost savings, but reinforcing a test hole may help the inspector and contractor identify potential concrete placement problems associated with tightly spaced cages, Osterberg cells, and multiple access tubes (CSL).

procedure for shafts constructed in water. Restore the disturbed areas at the sites of test holes drilled out of position as nearly as practical to their original condition. When the Contractor fails to demonstrate to the Engineer the adequacy of his methods or equipment, and alterations are required, he shall provide additional test holes at no expense to the Department. Include the cost of all test holes in the cost of the Drilled Shafts.

455-19 Test Bells.

Ream the bells at specified test holes to establish the feasibility of belling in a specific soil strata. Use the diameter and shape of the test bell shown in the plans or as approved in writing.

455-20 Construction Tolerances.

Meet the following construction tolerances for drilled shafts:

(a) Ensure that the top of the drilled shaft is no more than 3 inches [75 mm] laterally from the position indicated in the plans.

(b) Ensure that the vertical alignment of the shaft excavation does not vary from the alignment shown in the plans by more than 1/4 in/ft [20 mm/m] of depth.

(c) After placing all the concrete, ensure that the top of the reinforcing steel cage is no more than 6 inches [150 mm] above and no more than 3 inches [75 mm] below plan position.

(d) Ensure that the reinforcing cage is concentric with the shaft within a tolerance of 1 1/2 inches [40 mm]. Ensure that concrete cover is 6 inches ± 1 1/2 inches [150 ± 40 mm] unless shown otherwise in the plans.

(e) All casing diameters shown in the plans refer to I.D. (inside diameter) dimensions. However, the Contractor may use casing with an outside diameter equal to the specified shaft diameter if the extra length described in 455-15.7 is provided. In this case, ensure that the I.D. of the casing is not less than the specified shaft diameter less 1 inch [25 mm]. When approved, the Contractor may elect to provide a casing larger in diameter than shown in the plans to facilitate meeting this requirement. When casing is not used, ensure that the minimum diameter of the drilled shaft is 1 inch [25 mm] less than the specified shaft diameter. When conditions are such that a series of telescoping casings are used, provide the casing sized to maintain the minimum shaft diameters listed above.

(f) Excavate the bearing area of bells to the plan bearing area as a minimum.

Ensure that the diameter of the bells does not exceed three times the specified shaft diameter. The Contractor may vary all other plan dimensions shown for the bells, when approved, to accommodate his equipment.

(g) Ensure that the top elevation of the drilled shaft concrete has a tolerance of +1 and - 3 inches [+25 and -75 mm] from the top of shaft elevation shown in the plans.

(h) The dimensions of casings are subject to American Pipe Institute tolerances applicable to regular steel pipe.

(i) Use excavation equipment and methods designed so that the completed shaft excavation will have a flat bottom. Ensure that the cutting edges of excavation equipment are normal to the vertical axis of the equipment within a tolerance of ±3/8 in/ft [±30 mm/m] of diameter.

455-21 Drilled Shaft Excavations Constructed out of Tolerance.

Do not construct drilled shaft excavations in such a manner that the concrete shaft cannot be completed within the required tolerances. The Contractor may make corrections to an unacceptable drilled shaft excavation by any combination of the following methods:

(a) Overdrilling the shaft excavation to a larger diameter to permit accurate placement of the reinforcing steel cage with the required minimum concrete cover.

(b) Increasing the number and/or size of the steel reinforcement bars.

(c) Enlargement of the bearing area of the bell excavation within tolerance allowed.

When the tolerances are not met, the Contractor may request design changes in the caps or footings to incorporate shafts installed out of tolerance. The Contractor shall bear the costs of redesign and Unforeseeable Work resulting from approved design changes to incorporate shafts installed out of tolerance. Employ a Specialty Engineer to perform any redesign and who shall sign and seal the redesign drawings and computations. Do not begin any proposed redesign until it has been reviewed for acceptability and approved by the Engineer.

Backfill any out of tolerance shafts in an approved manner when directed by the Engineer until the redesign is complete and approved. Furnish additional materials and work necessary, including engineering analysis and redesign, to effect corrections of out of tolerance drilled shaft excavations at no expense to the Department.

Appendix A (Continued).

455-22 Static Compression Load Tests.

455-22.1 General: When the plans include load testing, perform all load tests in accordance with 455-2.

455-22.2 Disposition of Loading Material: After completing all load tests, clean, remove all rust on structural steel, repaint all areas having damage to the paint in accordance with Section 561, and return all load test equipment supplied by the Department to its designated storage area. Notify the Department at least ten working days in advance so that arrangements can be made to unload the equipment. The Contractor shall remove all equipment and materials which remains his property from the site. Clean up and restore the site to the satisfaction of the Engineer.

455-22.3 Disposition of Tested Shafts: After completing testing, cut off the tested shafts and any reaction shafts at an elevation 24 inches [610 mm] below the finished ground surface. Take ownership of the shaft cut-offs and provide areas for their disposal.

Appendix A (Continued).

$$F = \frac{2D_2 - D_1}{D_2}$$

455-23 Method of Measurement.

455-23.1 Drilled Shafts: The quantity to be paid for will be the length, in feet [meters], of the reinforced concrete drilled shaft of the diameter shown in the plans, completed and accepted. The length will be determined as the difference between the top of shaft elevation as shown in the plans and the final bottom of shaft elevation as authorized and accepted. When the Contractor elects to provide outside diameter (O.D.) sized casing rather than inside diameter (I.D.) sized casing as allowed in 455-15.7, the pay quantity measured as described above will be multiplied by a factor (*F*) determined as follows:

where:

F = factor to adjust pay quantities to compensate for smaller shafts.

D_1 = casing inside diameter specified = shaft diameter specified.

D_2 = casing inside diameter provided (D_2 = D_1 minus twice the wall thickness).

Appendix A (Continued).

455-23.2 Drilled Shafts (Unreinforced): The quantity to be paid for will be the length, in feet [meters], of unreinforced concrete drilled shaft of the diameters shown in the plans, completed and accepted. The length will be determined as the difference between the top of shaft elevation as shown in the plans and the final bottom of shaft elevation as authorized and accepted. When the Contractor elects to use O.D. casing, the quantity as determined above will be multiplied by the factor *"F"* determined as described in 455-23.1.

455-23.3 Unclassified Shaft Excavation: The quantity to be paid for will be the length, in feet [meters], of unclassified shaft excavation of the diameter shown in the plans, completed and accepted, measured along the centerline of the shaft from the ground surface elevation to the plan bottom of shaft elevation authorized and accepted. When drilled shafts are constructed through fills placed by the Contractor, the original ground surface before the fill was placed will be used to determine the quantity of unclassified shaft excavation. When the Contractor elects to use O.D. casing, the quantity as determined above will be multiplied by the factor *"F"* determined as described in 455-23.1.

Appendix A (Continued).

455-23.4 Unclassified Extra Depth Excavation: The quantity to be paid for will be the length, in feet [meters], of unclassified shaft excavation of the diameter shown in the plans measured along the centerline of the shaft from the bottom of shaft elevation shown in the plans to the final authorized bottom of shaft elevation when below the plan bottom of shaft elevation. When the Contractor elects to use O.D. casing, the quantity as determined above will be multiplied by the factor *"F"* determined as described in 455-23.1.

455-23.5 Drilled Shaft Sidewall Overreaming: The quantity to be paid for will be the length, in feet [meters], of drilled shaft sidewall overreaming authorized, completed and accepted, measured between the elevation limits shown in the plans or authorized by the Engineer. When the Contractor elects to use O.D. casing, the quantity as determined above will be multiplied by the factor *"F"* determined as described in 455-23.1.

455-23.6 Bell Footings: The quantity to be paid for will be the number of bells of the diameter and shape shown in the plans, completed and accepted.

455-23.7 Test Holes: The cost of all test holes will be included in the cost of Drilled Shafts.

455-23.8 Test Bells: The quantity to be paid for will be the number of test bells, completed and accepted.

Appendix A (Continued).

455-23.9 Core (Shaft Excavation): The quantity to be paid for will be the length, in feet [meters], measured from the bottom of shaft elevation to the bottom of the core-hole, for each authorized core drilled below the shaft excavation, completed and accepted. When the Engineer authorizes Core (Shaft Excavation) extending through part or all of the shaft, prior to excavation, to some depth below the shaft bottom, the quantity will be the length in feet [meters], measured from the top elevation to the bottom elevation authorized by the Engineer, completed and accepted. When SPT tests are substituted for coring as provided in 455-15.6, the quantity will be determined as described above for coring.

Appendix A (Continued).

455-23.10 Casings: The quantity to be paid for will be the length, in feet [meters], of each size casing as directed and authorized to be used. The length will be measured along the casing from the top of the shaft elevation or the top of casing whichever is lower to the bottom of the casing at each shaft location where casing is authorized and used, except as described below when the top of casing elevation is shown in the plans. Casing will be paid for only when the Permanent Casing Method is specified, when the plans show a casing that becomes a permanent part of the shaft, or when the Engineer directs the Contractor to leave a casing in place which then becomes a permanent part of the shaft. No payment will be made for casings which become bound or fouled during shaft construction and cannot be practically removed. The Contractor shall include the cost of all temporary removable casings for methods of construction other than that of the Permanent Casing Method in the bid price for Unclassified Shaft Excavation item.

When the Permanent Casing Method and the top of casing elevation are specified, the casing will be continuous from top to bottom. Authorization for temporary casing will not be given unless the Contractor demonstrates that he can maintain alignment of the temporary upper casing with the lower casing to be left in place during excavation and concreting operations. When artesian conditions are or may be encountered, the Contractor shall also demonstrate that he can maintain a positive water-tight seal between the two casings during excavation and concreting operations.

Appendix A (Continued).

When the top of casing elevation is shown in the Contract Documents, payment will be from the elevation shown in the plans or from the actual top of casing elevation, whichever is lower, to the bottom of the casing. When the Contractor elects to use an approved special temporary casing system in open water locations, the length to be paid for will be measured as a single casing as provided above.

455-23.11 Protection of Existing Structures: The quantity to be paid for will be at the lump sum price.

455-23.12 Static Load Tests: The quantity to be paid for will be the number of load tests conducted.

455-23.13 Instrumentation and Data Collection: The quantity to be paid for will be at the lump sum price.

455-24 Basis of Payment.

455-24.1 Drilled Shafts: Price and payment will be full compensation for all drilled shafts, including the cost of concrete and reinforcing steel, including all labor, materials, equipment, and incidentals necessary to complete the drilled shaft.

455-24.2 Drilled Shafts (Unreinforced): Price and payment will be full compensation for all drilled shafts (unreinforced), including the cost of concrete and all labor, equipment, materials, and incidentals necessary to complete the drilled shaft.

Appendix A (Continued).

455-24.3 Unclassified Shaft Excavation: Price and payment will be full compensation for the shaft excavation (except for the additional costs included under the associated pay items for casing); removal from the site and disposal of excavated materials; restoring the site as required; cleaning and inspecting shaft excavations; using slurry as necessary; using drilling equipment; blasting procedures, special tools and special drilling equipment to excavate the shaft to the depth indicated in the plans; and furnishing all other labor, materials, and equipment necessary to complete the work in an acceptable manner.

455-24.4 Bell Footings: Price nd payment will be full compensation for forming and excavating the bell beyond the diameter of the drilled shaft, furnishing and casting additional concrete necessary to fill the bell outside the shaft together with any extra reinforcing steel required, removing excavated materials from the site, and all other expenses necessary to complete the work.

455-24.5 Test Holes: No separate payment will be made for Test Hole. All cost of Test Holes will be included in the cost of Drilled Shafts.

455-24.6 Test Bells: Price and payment will be full compensation for forming the test bell, providing inspection facilities, backfilling the bell when the test hole is drilled out of position, and all other expenses necessary to complete the work.

Appendix A (Continued).

455-24.7 Core (Shaft Excavation): Price and payment will be full compensation for drilling and classifying the cores, delivering them to the Department, furnishing drilled shaft concrete to fill the core hole, and all other expenses necessary to complete the work. When SPT tests are substituted for coring as provided in 455-15.6, they will be paid for at the price per foot [meter] for coring.

455-24.8 Casings: Price and payment will be full compensation for additional costs necessary for furnishing and placing the casing in the shaft excavation above the costs attributable to the work paid for under associated pay items for Unclassified Shaft Excavation and Unclassified Extra Depth Excavation.

455-24.9 Protection of Existing Structures: Price and payment will include all cost of work shown in the plans or described herein for protection of existing structures. When the Contract Documents do not include an item for protection of existing structures, the cost of settlement monitoring as required by these Specifications will be included in the cost of Unclassified Shaft Excavation; however, work in addition to settlement monitoring will be paid for as Unforeseeable Work when such additional work is ordered by the Engineer.

455-24.10 Static Load Tests: Price and payment will include all costs related to the performance of the load test.

Appendix A (Continued).

455-24.11 Instrumentation and Data Collection: Price and payment will include all labor, equipment, and materials incidental to the instrumentation and data collection, and, when required, the load test report.

455-24.12 Payment Items: will be made under:

Item No. 455- 18- Protection of Existing Structures - lump sum.
Item No. 2455- 18- Protection of Existing Structures - lump sum.
Item No. 455- 88- Drilled Shaft - per foot.
Item No. 2455- 88- Drilled Shaft - per meter.
Item No. 455- 90- Bell Footings - each.
Item No. 2455- 90- Bell Footings - each.
Item No. 455- 92- Test Bells - each.
Item No. 2455- 92- Test Bells - each.
Item No. 455-107- Casing - per foot.
Item No. 2455-107- Casing - per meter.
Item No. 455-111- Core (Shaft Excavation) - per foot.
Item No. 2455-111- Core (Shaft Excavation) - per meter
Item No. 455-119- Test Loads - each.
Item No. 2455-119- Test Loads - each.
Item No. 455-122- Unclassified Shaft Excavation - per foot.
Item No. 2455-122- Unclassified Shaft Excavation - per meter.
Item No. 455-129- Instrumentation and Data Collection - lump sum.
Item No. 2455-129- Instrumentation and Data Collection - lump sum.