USF UNIVERSITY OF SOUTH FLORIDA
Civil & Environmental Engineering

EVALUATION OF SELF CONSOLIDATING CONCRETE AND CLASS IV CONCRETE FLOW IN DRILLED SHAFTS

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FINAL REPORT

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
in.	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
AREA				
$in.^2$	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1,000 L shall be shown in m^3				

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
TEMPERATURE (exact degrees)				
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fL	foot-Lamberts	3.426	candela/m^2	cd/m^2

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in.^{2}	poundforce per square inch	6.89	kilopascals	kPa
kip	kilopound	4.45	kilonewtons	kN

APPROXIMATE CONVERSIONS TO SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
mm	millimeters	0.039	inches	in.
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		AREA		
mm^2	square millimeters	0.0016	square inches	in.^{2}
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/$in.^2$
kN	kilonewtons	0.225	kilopound	kip

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

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16. Abstract

The term “mattressing” is a relatively new term coined by a joint task force formed by the Deep Foundation Institute and European Foundation Federation Committee (DFI/EFFC). Mattressing is defined as a pattern of creases on the surface of drilled shafts or slurry walls where the pattern reflects the reinforcing cage layout and which gives the appearance of a quilted mattress top. The presence of creases does not substantially affect the volume of placed concrete and therefore is not detected by routine drilled shaft inspector logs where the anticipated/theoretical shaft volume is directly compared to the as-placed volume.

The presence of creases, when found, was thought to be an unusual occurrence caused by out-of-spec concrete flow (low or borderline slump) or slurry heavily laden with suspended soil particles (high sand content). Two studies leading up to this work identified that (1) concrete flow is radial and not vertically rising in the cover region, and (2) creases always formed when mineral slurry was used in large-scale laboratory shaft specimens where the concrete was tremie-placed and slurry displacing. This project was tasked with identifying the extent to which creases have formed in both laboratory and field conditions and the effect of their presence.

Laboratory findings showed that use of mineral slurry significantly affected rebar bond/development length requirements and corrosion resistance/durability provided by the cover concrete. Polymer slurry showed similar rebar bond effects but less frequently affected durability. The presence of water at the time of tremie-placing concrete had no adverse effect on rebar bond or durability.

Underwater inspection of selected in-service bridges showed similar results to the laboratory findings where the concrete surface behind removed casing coupons was visibly distressed for shafts using bentonite slurry, partially distressed for shafts using attapulgite slurry, and little to no distress for shafts using using water.

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Executive Summary

The term "mattressing" is a relatively new term coined by a joint task force formed by the Deep Foundation Institute and European Foundation Federation Committee (DFI/EFFC). Mattressing is defined as a pattern of creases on the surface of drilled shafts or slurry walls where the pattern reflects the reinforcing cage layout and which gives the appearance of a quilted mattress top. The presence of creases does not substantially affect the volume of placed concrete and therefore is not detected by routine drilled shaft inspector logs where the anticipated/theoretical shaft volume is directly compared to the as-placed volume.

The creases are formed as the concrete level inside the cage builds up enough pressure to push the concrete radially into the cover region and where the concrete separates as it passes around the rebar cage elements. Higher hydrostatic pressure and/or when the slurry is free from suspended solids minimize creasing, allowing the separated flow paths to visually recombine. The presence of creases, when found, was thought to be an unusual occurrence caused by out-of-spec concrete flow (low or borderline slump) or slurry heavily laden with suspended soil particles (high sand content). Two studies leading up to this work identified that (1) concrete flow is radial and not vertically rising in the cover region and (2) creases always formed when mineral slurry was used in large-scale laboratory shaft specimens where the concrete was tremie-placed and slurry displacing. The potential seriousness of the latter condition made it imperitive to determine if field-cast shafts developed the same degree of creasing as laboratory-cast shafts when mineral slurries were used.

The laboratory component of the project involved the examination of 59 large-scale, laboratory-cast shaft specimens for the visual presence of creases, the effect of the creases on concrete strength and rebar bond, and the resulting corrosion resistance or durability that could be expected from various drilling slurry types and consistencies. Slurry types included mineral (bentonite and attapulgite), polymer (three manufacturers), and natural slurry (water). Slurry viscosity of mineral and polymer slurries was varied over the widest expected range of use in field applications.

The surface texture of the shaft specimens, when cast against simulated steel casing, was found to be notably degraded when mineral slurries were used, regardless of slurry viscosity. This included the presence of creases and slurry products trapped between the concrete and the casing. Smooth surfaces where found for virtually all polymer and water specimens with some occurrences of hairline creases only visible by wetting and subsequent drying. The slurry trapping was identified as a probable cause of concrete-soil interface bond reductions previously attributed to filter cakes that form as mineral slurry flows into the surrounding soil and deposits the clay minerals along the side walls.

Concrete strength within the core of the shaft cage was compared to the cover regions via an extensive coring program designed to identify subtle variations in concrete strength and quality. For specimens cast in mineral slurry, an 18% reduction in strength was noted. For polymer and water conditions, the values varied by 14% and 8%, respectively, at the worst-case locations along a vertical crease. The rebar pullout bond was also examined as part of the strength effects investigation. The bond was found to be materially affected by the presence of mineral or polymer slurry but not for water conditions. A statistical evaluation of these tests identified the need for a

development length multiplier of 1.5-2 for slurry casting environments similar to that applied for epoxy coatings (1.2-1.5) or top vs.. bottom steel in horizontally cast beams.

Corrosion resistance provided by the cover concrete in tremie-placed, slurry-displaced foundation elements was also found to be significantly affected by slurry type and not nearly as much by consistency. Surface potential measurements were used as the primary mechanism for comparison where copper-copper sulfate electrodes were applied to the wetted concrete surface and the millivolt potential difference between the electrode, and the reinforcement cage was mapped over a large portion of the entire shaft surface. All bentonite cage specimens showed higher probability of corrosion where 82% were actively corroding. Polymer shafts had a lower percentage of corroding specimens (29%), and one water-cast (6%) specimens showed signs of active corrosion. Interestingly, the surface texture (roughness) was identified to be a direct indication of poor corrosion protection where specimens with a roughness greater than 6 in^3 of void volume per square foot of surface area were actively corroding.

The field component of the study included the underwater inspection of five bridges selected to be representative of shafts cast in water, bentonite, or attapulgite. Water is used when a casing is employed to stabilize the excavation; bentonite is used when fresh water conditions are present and slurry is required for stability; and attapulgite is used instead of bentonite in salt water conditions. Over-water bridges were selected due to the ability to check the concrete surface quality without soil excavation. However, most of the shafts inspected still had a majority of the permanent casings in place (required to form concrete up to the water surface). Therefore, 3 ft x 3 ft windows were cut in the casing to examine the concrete surface. Comparison of the electro-chemical failure ratios in the lab to the visual surface conditions in the field showed that the results were very similar: The percentage of degraded or creased shaft surfaces were 100% for bentonite-cast shafts, 45% for attapulgite-cast shafts, and 0% for water-cast shafts when single cages were used. For water-cast shafts cast with two concentric cages, all (3 out of 3) showed minor irregularities or creasing, indicating poorer concrete flow.

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Chapter One: Introduction

Drilled shafts are cylindrical, cast-in-place concrete, deep foundation elements that may be selected over driven piles on the basis of cost effectiveness, the soil strata encountered, or the need to control vibrations due to sensitive surroundings. In general, the process of constructing shafts involves the drilled excavation of soil or rock using large diameter augers to form a deep cylindrical void space. Within the excavation, placement of the necessary reinforcing steel is followed by concreting (Figure 1.1).

Figure 1.1 Shaft construction: excavation (left), cage placement (center) and concreting (right)

Unless permanent casing is used, the process requires the in situ soils to act as the formwork and thereby define the shape of the concrete. In Florida, the high water table often dictates the use of slurry to stabilize the excavation walls and concreting is always tremie-placed.

The two most common complications that arise during shaft construction stem from, and in the order of operation: (1) excavation stability and if successful, then (2) concrete-related flow properties. The latter of which is further complicated by the reinforcing cage congestion/spacing and the properties of the slurry that must be displaced by the concrete.

This study focuses on concrete flow-related effects on the constructed element performance and how the presence of drill slurry (mineral, polymer or natural) affects various aspects of the performance. Use of self-consolidating concretes (SCC) in lieu of standard Class IV shaft mixes is also addressed. As not all SCC mix designs use the same constituents or admixtures, the effects on the concrete performance in the presence of various slurry materials are at present unknown. Therefore, it is important that the behavior of both the concrete and slurry be fully understood.

1.1 Background

The stability of a drilled shaft excavation (prior to and during concreting) can be maintained mechanically, hydrostatically, or with a combination of both means. Mechanical stability implies the use of a full-length steel casing (a large diameter, thin-walled pipe) that holds the soil in place while the construction process is performed within. Upon completion of concreting, the casing is often fully extracted before the concrete cures, and the wet/fluid concrete flows/pushes out against the excavation walls. For this process to be successful, the concrete must still be fluid with sufficient flowability to move outward to the excavation walls upon casing extraction to promote side shear resistance. Alternately, slurry stabilization can be used where the freshly placed concrete comes into immediate contact with the soil strata but the concrete must fully displace the slurry.

Slurry Stabilization. Hydrostatic stabilization is the process of using fluid or slurry to stabilize the excavation. The slurry level is maintained higher than the surrounding ground water table, and thus, a net pressure (or flow) is always directed outward into the soil walls to prevent side wall collapse. The slurry can be natural ground water or sea water (natural slurry), or a mixture formed by mineral or polymer additives can be used. However, it is never an acceptable practice to allow the ground water to flow into the excavation as a means to introduce slurry (natural) as fluids flowing out of the soil will result in soil loosening, side wall collapse, and/or end bearing soil relaxation.

The selection of slurry products or additives is somewhat controversial as various states permit or restrict the use of some products. Most commonly, a powdered clay mineral called bentonite (sodium montmorillonite) is mixed with water to form a slurry with a density slightly higher than water, but with the added advantage of greatly slowing or completely sealing off flow into the surrounding soil while maintaining the pressure differential; this exerts a force against the soil that maintains stability. Alternately, polymer slurry products can be used that tend to only slow the inflow rate but do not seal off the excavation walls. In all cases, the slurry pressure / elevation within the excavation must be higher than that of the existing ground water.

At the time of concreting, the slurry properties should be clean or free from excess suspended solids (sand content < threshold) and it should be light enough to be easily displaced by the rising, tremie-placed concrete (density < threshold). The quality of the slurry composition also applies if a mineral or polymer slurry is used; therein the viscosity of the slurry should demonstrate proper flow properties and stay within a tight range of acceptance. So in almost all cases (in Florida), concrete placement is performed blindly below water or slurry and therefore the concrete constantly interacts with the slurry during concreting.

Drilled Shaft Concrete. FDOT state specifications require that the Class IV drilled shaft mixes have an initial slump between 7 and 9 in. at the onset of concreting and must maintain at least a 5-in. slump throughout the entire concreting process. When temporary casing is used, the flow properties of the concrete at the time of casing extraction (after all concrete has been placed) is of vital importance. If the concrete is unable to flow due to sudden loss in slump/flowability, then the casing may become stuck and abandoned as permanent casing. This lower slump threshold is a type of practical construction limit.

More startling, however, is when the casing can be extracted but the concrete does not flow outward. In this case, the shaft becomes essentially slip-formed in place with an annular gap between the shaft concrete and the surrounding soils. This results in unwittingly constructing a shaft with zero side shear resistance. Garbin (2003) conducted studies on 1/10th scale drilled shafts that were constructed over a wide range of slump conditions at the time of casing extraction and showed that a significant reduction in side shear occurred at the then-permitted 4-in. slump loss (Figure 1.2). A 5-in. slump loss was later adopted on the basis of these findings. However, even at 5-in. slump a marked reduction in capacity was observed. Another ramification from these findings could be to remove slump loss restrictions when no temporary casing is used, as the concrete would have already made contact with the soil strata at the onset of concreting.

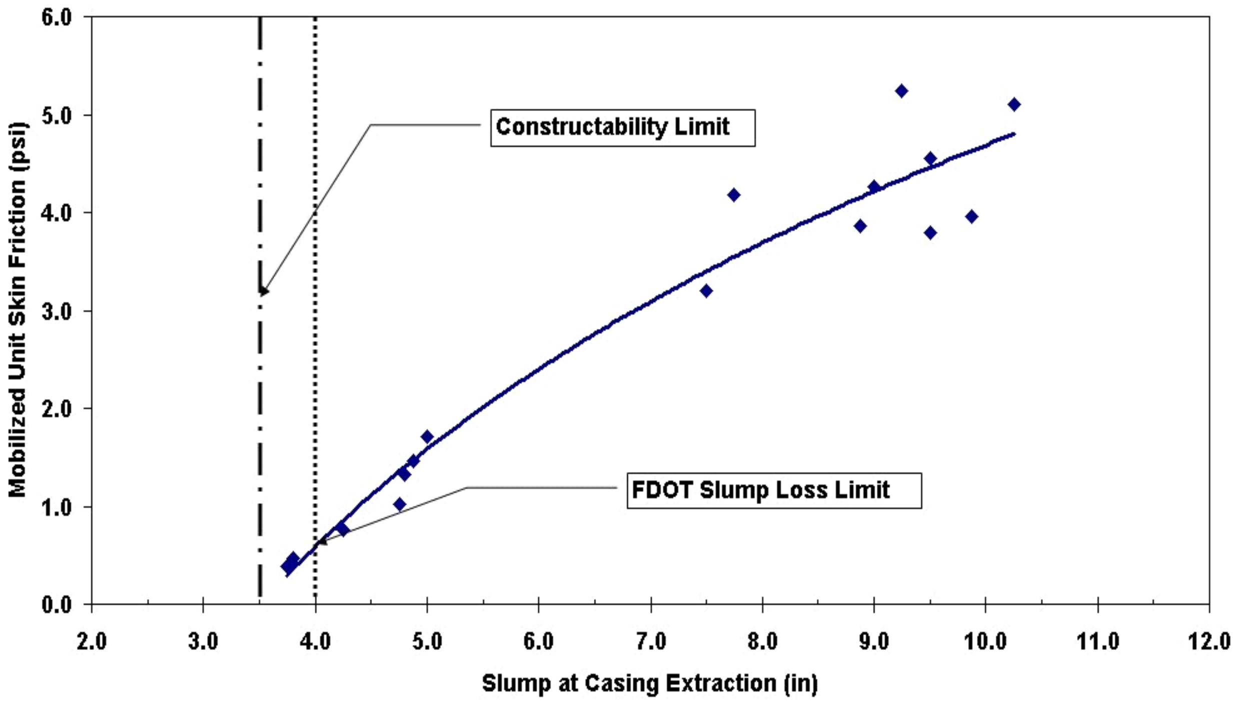


Figure 1.2 Side shear reduction caused by reduced slump (Garbin, 2003).

The flow of concrete in a shaft has been idealized as a rising fluid that displaces the lighter slurry effortlessly (e.g. oil on water). However, studies have shown that the rising concrete is drastically affected by the presence of the reinforcing cage (Mullins and Ashmawy, 2005; Deese, 2004; Deese and Mullins, 2005). Figure 1.3 shows a conceptual comparison of this misconception.

The studies showed that the tightness of the cage and fill/flow rate of the concrete were linked to the differential concrete level between the inside and outside of the cage. This differential was found to be the driving force for concrete to fill the cover region, and the head loss required for the concrete to pass through the cage had to be overcome with an equal magnitude of internal concrete head. These findings dismissed years of preconception that the rising concrete would scour the sidewalls and remove unwanted slurry clay buildup (filter cake) deposited by the outward flow of slurry into the surrounding soils.

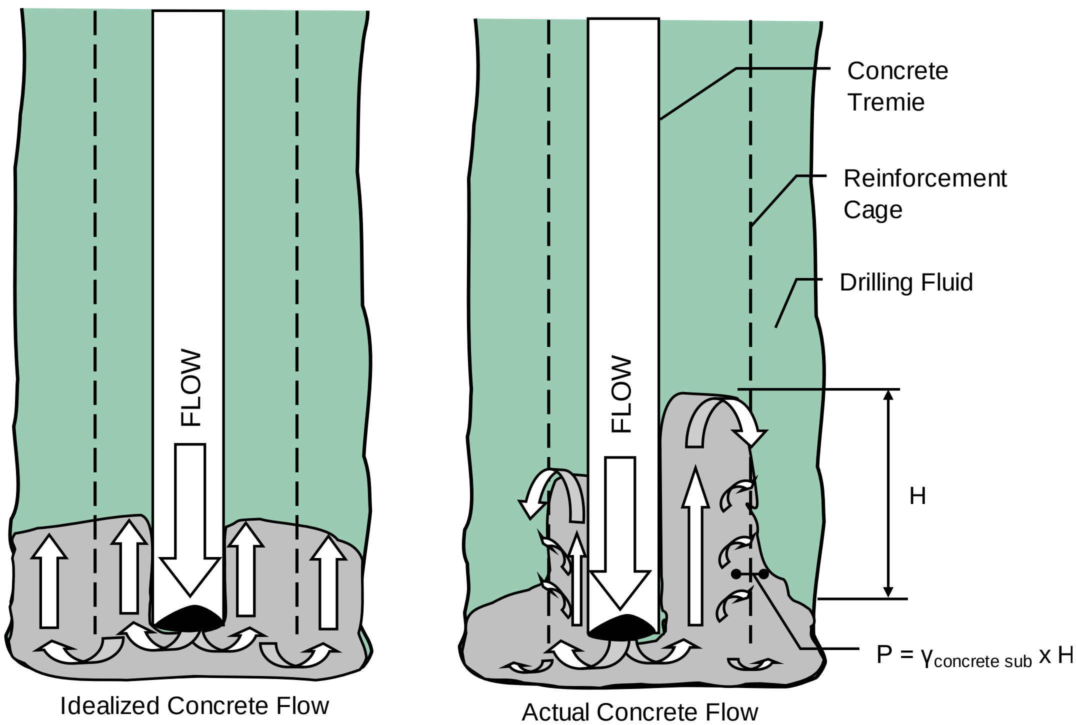


Figure 1.3 Comparison of idealized flow with observed (Mullins, 2005).

Small-scale lab specimens (Figure 1.4) showed concrete flows up beside the tremie in the remaining unobstructed portion of the cage. This was also observed in the field by weighted tape measurements taken from inside and outside the cage. For smaller-diameter shafts where the tremie diameter may occupy a large fraction of the interior cage region/volume, the differential may be even more drastic given a substantial increase in the upward concrete velocity from a standard concrete truck placement rate (e.g., 0.5 – 1 yd^3/min); smaller diameter shafts are filled more quickly where the rising vertical flow rate of the concrete is significantly higher than larger shafts. Similar to head loss computations in water distribution systems, the concrete head increase was shown to be proportional to the square of velocity (Figure 1.5).

Extending these findings, when the data in Figure 1.6 is sliced vertically at a constant flow rate (e.g., 1.5 ft/min) a practical cut off can be shown below which larger head differentials would occur. A cage spacing that produced a clear-spacing-to-maximum-aggregate-diameter ratio (CSD) smaller than 8 could result in more concrete buildup inside the cage, which in turn has a higher potential for inclusions (outside the cage) and may prevent concrete from sufficiently bonding to the surrounding soil.

Figure 1.4 Laboratory tests with small-diameter aggregate and tight reinforcement (CSD = 19).

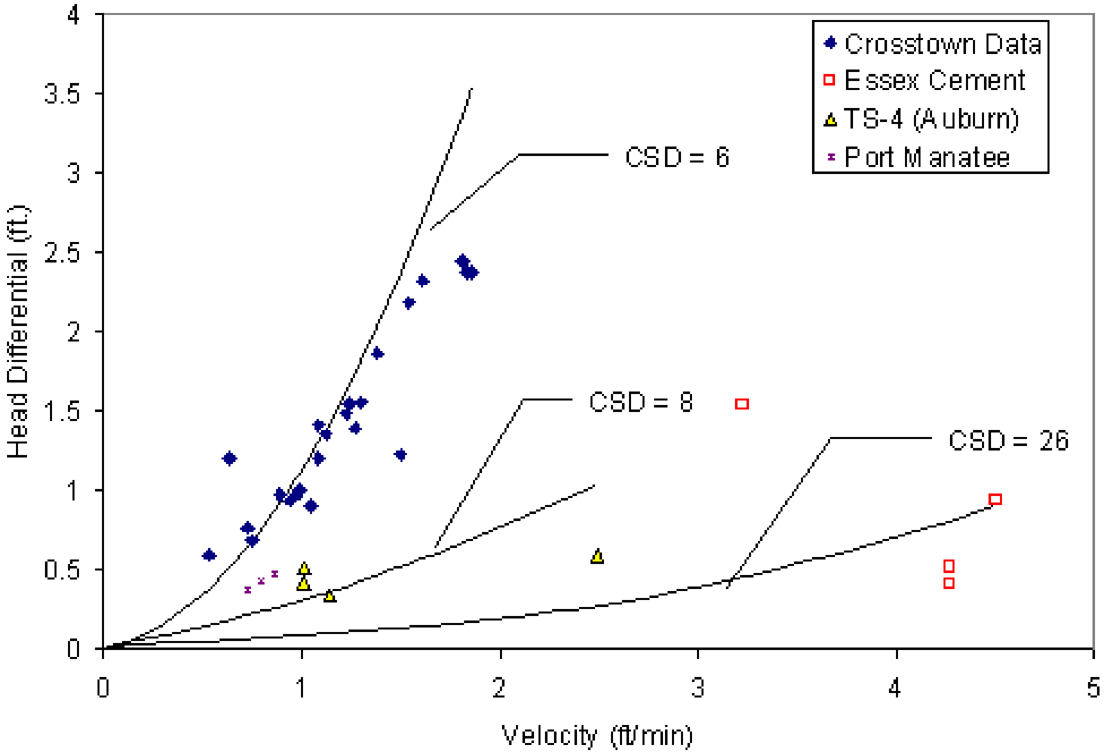


Figure 1.5 Inside-outside cage-head differential vs. upward concrete fill velocity (Deese and Mullins, 2005).

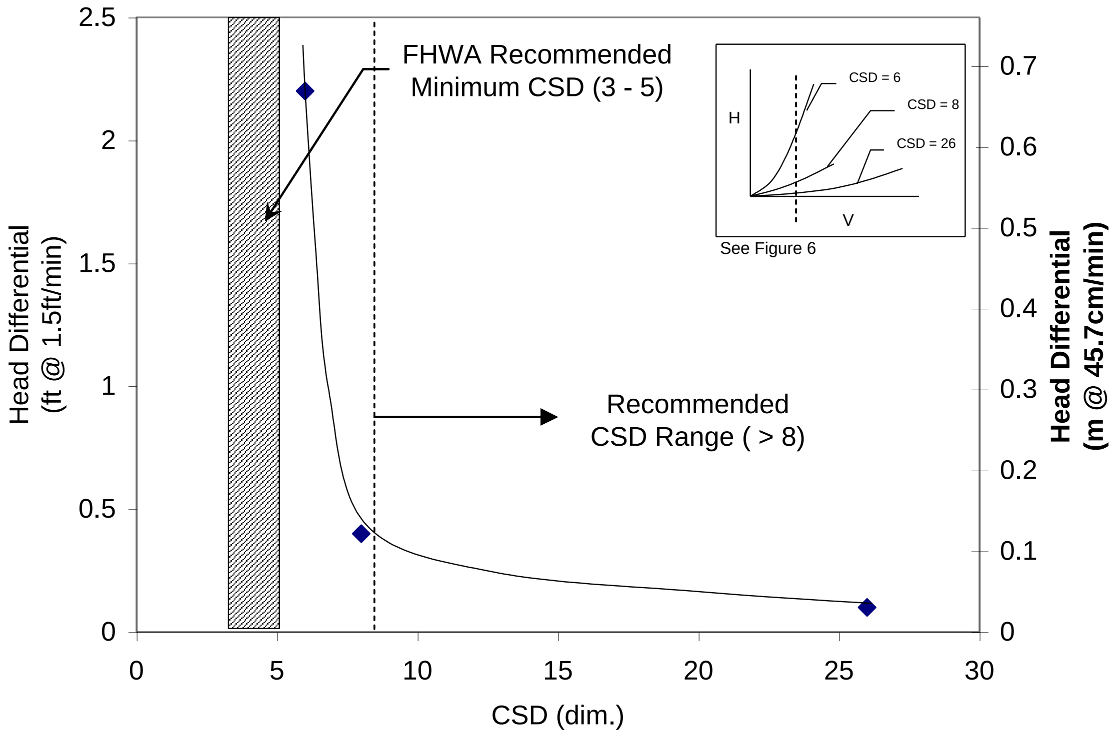


Figure 1.6 Recommended CSD ratio above which very little head differential was observed.

State and federal specifications have been established to control fresh concrete and slurry properties with the aim of circumventing the potential for problematic shafts. However, despite these efforts (specifications), problems persist. Figure 1.7 shows an example of a shaft that exhibited concrete flow problems, either from fresh concrete or slurry properties. Recent conversations with engineers in the UK indicated that this type of concrete flow (termed there as "quilting" or "mattressing") has been suspected but never confirmed. This image reinforced their fears. Since then and with this awareness, multiple occurrences have been documented.

Self-Consolidating Concrete. Self-consolidating concretes at first inspection seem to be natural candidates for drilled shaft applications as the poor flow that resulted in the Figure 1.7 observation might be alleviated. The first SCC prototypes were developed in the late 1980s (Osama et al, 1989) and hence are relatively new to construction as a whole. SCC is distinguished from conventional concrete by its water-like, free-flowing characteristics, which allow the mixes to flow into forms and around obstacles such as reinforcing steel under only the force of gravity (like shafts with no applied vibration). Recently, SCC concrete has become commonplace in precast yards (Figure 1.8) due to the reduced casting and finishing times.

Figure 1.7 Shaft exhumed to show poor concrete flow performance from slurry properties or fresh concrete properties (courtesy of the FDOT State Materials Office).

Figure 1.8 SCC used to cast prestressed piles requires almost no finishing as it pours out level.

For drilled shaft applications, several cases have been documented but the post construction evaluation methods (if any) are, at present, unclear. The evaluation of SCC performance in drilled shaft applications was one focus of this study.

1.2 Report Organization

The recurring finding of poor concrete flow through drilled shaft cages was the primary motivation for this study. The alarming condition of shafts found like that shown in Figure 1.7 raised the question, *how often does this occur?* This coupled with the findings of previous studies (Figures 1.3-1.6) further confirmed a general lack of understanding pertaining to concrete flow in tremie-placed, slurry displacing conditions. At the onset of this study, SMO/FDOT engineers along with USF researchers discussed the scope of the project and thereby narrowed the focus from *the development of a potentially viable SCC mix for shafts* to the present project, Part 1: an exploratory venture to identify the frequency/probability of poorly-cast underground structures and the effects of poor concrete flow on shaft performance and durability.

The ensuing eleven chapters discuss the efforts undertaken. Chapter 2 provides a literature review of SCC and Class IV shaft mix performance. Chapter 3 presents the construction of 58 large-scale shaft specimens cast in an outdoor research facility. Chapter 4 contains the rheological modeling completed to simulate tremie-placed, support-fluid-displaced concreting operations. Chapter 5 assesses the effects of support fluid on rebar bond through the evaluation of 227 pull out tests. Chapter 6 scrutinizes the lab specimens for corrosion resistance / durability and compares the effects of the various casting/slurry conditions and concrete types. Chapter 7 details the construction of an instrumented drill used to map concrete strength during coring operations. Chapter 8 presents the results of mercury porosimetry and x-ray diffraction tests of samples taken from the large-scale lab specimens. Chapter 9 looks at two methods used to quantify the surface condition of each of 52 laboratory-constructed shafts. Chapter 10 contains the findings from underwater evaluation of in-service bridges cast with bentonite, attapulgite, and natural (water) slurry conditions. Chapter 11 details the process for successfully operating the underwater laser scanner for evaluating the surface texture of submerged shafts. The data analysis procedure is also described. Chapter 12 provides a narrative summary of the results of all testing. The results from key data sets are then graphed together to reveal trends. Chapter 13 concludes the report with a summary of the findings and extended points of concern.

Appendices have been prepared corresponding to the various chapters and the respective data sets.

Chapter Two: Literature Review

2.1 Types of Deep Foundations

In terms of deep foundation construction, the most commonly used elements are drilled shafts or driven piles. However, other types of deep foundations exist such as micropiles and continuous flight auger (auger cast) piles/drilled displacement piles. These types of deep foundation elements are described in the following sections.

2.1.1 Driven Piles

Driven piles are prefabricated elements made of concrete, steel, or timber. Precast concrete elements are typically 12 to 36 in. in width or diameter and are installed using a pile driving hammer. Steel H piles and pipe piles can be installed using vibratory hammers and in some cases by water jetting. Driven piles cannot typically breach hard materials or rock. The most common types of piles used for transportation structures are steel H piles, pipe piles, and prestressed concrete piles. (Brown, et al., 2010)

2.1.2 Micropiles

Micropiles are smaller than driven piles, with diameters typically 12 in. or less. This type of pile is constructed using a high-strength steel rod or pipe, which is either driven or drilled into place. They are almost always grouted into the bearing strata once the desired depth/location is achieved. While this type of pile is a very small structural element, it can achieve a very high axial resistance and can be drilled into hard rock. In contrast, given the small cross sections, very little bending resistance can be developed. These elements are used extensively in structural repairs/foundation remediation where limited access is available for mobile drilling equipment.

2.1.3 Continuous Flight Auger (CFA) Piles and Drilled Displacement Piles

This form of pile is typically 12 to 30 in. in diameter and is only used in soils or weak rock. These elements are characterized by the installation procedure, which uses a full-length auger with continuous flights that allows the target foundation depth to be achieved in a single drilling stroke. Once the target depth has been achieved, concrete or grout is pumped down the center of the auger stem (which is hollow) and the auger is slowly extracted leaving a cast-in-place concrete or grout element. Reinforcing steel can be inserted immediately thereafter while the concrete/grout is still fluid. This, however, can limit the amount of steel and complexity of the reinforcing steel cage design.

2.1.4 **Drilled Shafts**

Drilled shafts, typically 3 to 12 ft in diameter, are cylindrical, cast-in-place, reinforced concrete elements constructed to depths of up to 300 ft. Being one of the largest diameter foundation options, significant bending and lateral resistance can be developed, which in some cases provides economic benefits. These elements differ from CFA piles as augers with only one or two flights are used, and the excavation process involves multiple trips down an open hole to remove material and reach the target depth. Hence, an open excavation is first created in which the reinforcing steel and concrete are then placed. Drilled shafts are often selected over other options as the drilling process to create the cast-in-place formwork that can penetrate stiff soil or rock strata not easily penetrated by driven pile or CFA options. Drilled shafts are the focus of the project and are therefore discussed in further detail.

2.2 **Drilled Shaft Construction**

Constructing drilled shafts involves three basic steps: (1) excavation, (2) installation of the reinforcing cage, and (3) concreting. For the purposes of this report, concreting is a primary focus. However, the mechanisms by which the excavation is held open/stable in the presence of ground water can greatly affect concrete placement and therefore is also discussed.

2.2.1 **Methods of Construction**

There are three methods of excavation used for drilled shafts: the dry method, the casing method, and the wet method. Once a method of construction has been determined, a concrete placement method can be selected (e.g. free-fall or tremie-placed).

Dry Method

The dry method of construction is the most favorable from an economic standpoint. This method can be utilized when soil and rock are located above the water table and will not cave in while the hole is being drilled or after. A homogeneous stiff clay for example, would be ideal for this method of drilling, while a loose sand would not. However, any type of soil is vulnerable to caving near the surface, thus a small, short piece of steel casing called a “surface casing” is typically inserted there. This form of casing may be temporary or permanent.

The construction process for the dry method is as follows. First, the shaft is excavated to the desired depth. This will most likely be completed using a simple rotary auger, which will also likely have teeth to break up the soil. This ensures the most amount of material possible per pass is removed. Next, the base of the shaft is cleaned using a bucket or flat bottom tool to remove any loose debris and potential water (Brown, et al., 2010). This is followed by the insertion of a reinforcement cage (for most projects), and then finished by placing the concrete. This can be done using the “free-fall” method of placing concrete where the flow is directed to the center of the hole in effort to not hit the reinforcement cage or sides of the borehole. Concrete can also be placed utilizing a short section of tremie pipe or centering device.

Casing Method

The casing method is not as simple as the dry method. It is most applicable in soils prone to caving or where rock deformation may occur during excavation. The casing method can also be used to extend shaft formation through water to reach a dry, stable formation. This method is commonly used with the addition of drilling fluid or water, however it is not always needed.

There are three general methods for the installation of casing. The first is to excavate an oversized hole via the dry method, then place the casing into the hole. However, this method is only acceptable in soils that are generally dry or with slow seepage. The second method is to use a drilling fluid to displace the soil while drilling the hole through the shallow permeable strata, the casing can then be placed and advanced into the bearing layer. Once the casing is sealed to a stable layer the drilling fluid can be removed from inside the casing. The third and most common method is to first install the casing through the soil strata and into the bearing layer, then excavate the shaft within the casing with or without fluid. The casing can be driven using methods such as a casing oscillator, impact or vibratory hammers, or a rotator with sufficient torque and downward force to advance the casing.

For all methods, while permanent casing can be used, most casing is recovered after the concrete is placed. In most cases as well the shaft excavation will continue past the bottom of the casing and thus it is important that the casing achieves a seal into the bearing layer to prevent caving and/or seepage. The use of a full-length reinforcement cage is generally required for all methods, as it is difficult to keep a partial cage in the proper orientation.

Wet Method

The wet method of construction is where the excavation is kept filled with a prepared drilling fluid, or slurry (discussed in detail in 2.3), designed to keep the borehole open by maintaining its stability, or filled with water if the hole is stable during the entire construction process. There are several cases where it is necessary to use the wet method, for example if the shaft is to be drilled into a sand or permeable layer that will collapse or demonstrate instability during excavation. Another case would be where the foundation is stable, however the shaft is to extend through caving or water-bearing soils that would be difficult to drive casing through because of the soils depth and thickness, the drilling fluid would be able to keep the excavation stable and prevent groundwater infiltration. Another circumstance is when full length casing is driven (method 3 of casing method), however the soil conditions at the base are permeable. This is an example of when plain water can be used instead of drilling fluid. The last instance for use of this method would be when the hole is cased into a stable rock, but the groundwater has an inflow of greater than 12 in. per hour. Thus, drilling fluid is used to prevent groundwater infiltration. (Brown, et al., 2010)

The use of drilling fluid works by forming a hydraulic gradient between the fluid in the borehole and the soil. This is done by keeping the drilling fluid elevation higher than that of the groundwater in the soil, thus the drilling fluid exerts a pressure causing it to try and flow out into the soil. This seepage pressure provides stability to the excavation sidewall. Common drilling fluids used are polymer and bentonite, which have the ability to hold higher levels of viscosity than water. A higher viscosity simultaneously reduces the rate of fluid loss within the excavation, which will result in the formation of the hydraulic gradient just discussed by keeping a hydraulic slurry head of at least 5 or more feet above the hydraulic head from the groundwater. (Brown, et al., 2010)

Construction for this method includes casing insertion, excavation and simultaneous filling with drilling fluid, reinforcement cage insertion, and concreting. Concreting is performed via tremie placement (section 2.2.2.2). By whatever means necessary it is also important to avoid potential inclusions of slurry or suspended sand into the concrete.

2.2.2 Concrete Placement

Placement of concrete into the drilled shaft is performed by two methods. The first method is free-fall concreting and the second is tremie placement. Both methods are discussed in the following sections.

Free-Fall

Free-fall concrete placement is only permitted for dry excavations, because if the shaft excavation is not completely dry the concrete and excess water will mix, resulting in a concrete mix with excessive water or even a zone of washed aggregate. To avoid this there should be less than 3 in. of water at the bottom of the excavation. Other precautions necessary when using this method are to avoid hitting the reinforcement cage and sides of the borehole. When the concrete hits the reinforcement cage the cage can get distorted, segregation in the concrete can also be produced. If the concrete hits the side of the borehole, this risks soil or debris being knocked into or mixing with the concrete.

To ensure that the concrete is placed in the center, a drop chute can be used. This can be composed of a short section of stiff or rigid pipe, a flexible hose should not be used as the flow may be difficult to direct. It should be noted that the Florida Department of Transportation (FDOT) still requires a tremie for depositing the concrete for dry excavations and states the free-fall of the concrete should be less than 5 ft at all times (FDOT, 2018); as such free-fall concreting is not discussed any further.

Tremie Placement

A gravity tremie places concrete by using a steel tube, typically with a hopper on top. Concrete can be placed directly from the concrete truck, from a pump, or by discharging from a bucket. Typical inside diameters of tremie pipes are 8 to 12 inches, however this is dependent on the diameter and depth of the excavation. It is important for the tremie pipe to be watertight. This will prevent drilling fluid from entering during placement. It is also critical for the tremie to be smooth and clean on the inside, minimizing drag forces.

There are two typical procedures that can be followed to minimize concrete contamination when slurry or water is used in the excavation. The first is to install a closed tremie where the bottom is sealed with a cover plate. The second method is to install an open tremie and insert a traveling plug ahead of the concrete. (Brown, et al., 2010)

Tremie placement delivers concrete first to the bottom of the shaft and then the tremie pipe is slowly raised. It is important to always keep the bottom of the tremie at least 10 ft below the rising surface of fresh concrete to prevent the concrete mixing with the slurry (Brown, et al., 2010; FDOT, 2018). However, there is always a time early-on when concrete has not yet filled significantly above the bottom of the tremie level. This report focuses on tremie-placed concrete.

2.3 Drilling Fluid

As discussed in section 2.2.1.3, drilling fluid can be used to press against the soil walls of the borehole in order to maintain stability. There are two main circumstances where drilling fluid is needed. The first is when casing is installed and sealed into an impermeable layer; once this is achieved, the drilling fluid can be pumped out from the inside of the casing; this is a rare case. The second scenario is anytime an excavation is performed below the water table and where casing is not used to maintain sidewall stability. When using this method, the concrete must be tremie-placed so that the drilling fluid is properly displaced. Figures 2.1 to 2.5 display the slurry types discussed being displaced by concrete, these figures also indicate the ease of displacement.

2.3.1 Natural (Water)

While not always possible, in some situations water can be used as a drilling fluid (Figure 2.1). The Federal Highway Administration (FHWA) drilled shaft manual recommends the use of water as a drilling fluid when the soil layers being penetrated are permeable, but unable to slough or erode when exposed to water in the borehole. For example a sandstone or cemented sand would work much better in comparison to a loose sand where water would rush in from the bottom faster than it could be pumped in, in turn also loosening the soil at the tip changing the SPT (standard penetration test) blow count. If water is chosen for use, the water level in the excavation must be kept above the piezometric surface so that seepage only occurs from excavation into the formation and not the opposite. (Brown, et al., 2010)

Figure 2.1 Natural slurry (water) exiting the form during concreting.

2.3.2 Mineral Slurry

Mineral slurry is the most common for the wet construction method. This slurry is formed by the combination of dry mineral clay powder (either sodium or calcium montmorillonite) and water. There are several types of mineral clay powder: attapulgite, sepiolite, and bentonite, the latter of which is typically used for construction.

Bentonite

In the United States, the majority of bentonite comes from the state of Wyoming. Bentonite is classified as a sodium montmorillonite. When bentonite clay powder and water are mixed, the clay particles form a suspension (O'Neill and Reese, 1999). This suspension is caused when the bentonite powder is bound by water, causing it to scatter into microscopic plate-like particles. These particles then go on to constantly repel each other, similar to magnets when the same poles are trying to touch. This allows for an almost indefinite particle suspension. The hydration of bentonite can take up to several hours to complete, once finished the slurry is ready for final mixing and use (Brown, et al., 2010). Figure 2.2 shows bentonite slurry displacing out of a construction form.

The primary need for use of bentonite (mineral) slurry is excavation stabilization. Bentonite slurry works in two ways to achieve this. The first is through what is known as a filter cake. The filter cake is a thin layer that is formed along the sidewalls by the slurry as the suspended clay particles are deposited onto the excavation walls, and the mix water migrates into the soils. This aids stabilization by reducing outflow into the soil; however, the filter cake has been shown to negatively affect shaft side shear when the bentonite sits in the excavation for eight hours or more prior to concrete placement (Allen, 2016). The second form of stabilization is through the exertion of a positive fluid (hydrostatic) pressure which acts against the filter cake membrane and borehole sidewalls (Brown, et al., 2010). This along with the filter cake also aids to prevent groundwater intrusion.

Figure 2.2 Bentonite slurry exiting the form during construction.

Attapulgite and Sepiolite

While attapulgite and sepiolite are mineral clay powders, they perform quite differently from bentonite. They are typically used where bentonite performs poorly, such as in marine environments which causes bentonite to flocculate. These clay minerals, in contrast to bentonite, are not hydrated by water and therefore have the ability to be used immediately after mixing. Figure 2.3 shows attapulgite slurry being displaced from a construction form. They also do not have the same suspension longevity as bentonite and therefore must be agitated frequently to ensure effectiveness. Another differing aspect is the filter cake, instead a soft clay layer is created on the walls of the excavation. This layer is an effective filter which is thought to be scoured off more easily by rising concrete, which is ideal considering this layer has a lower shear strength (O'Neill and Reese, 1999). However, the concept of "scour" as it pertains to excavation side walls is debatable and has been shown to not occur in tremie-placed application with standard to congested cages.

Figure 2.3 Attapulgite slurry exiting the form during concreting.

2.3.3 **Polymer**

Polymer slurries, where approved, are a relatively new alternative to mineral slurry, (Figures 2.4 and 2.5). Polymer slurries were introduced to the market around the 1980's. "The term polymer refers to any of numerous natural and synthetic compounds, usually of high molecular weight, consisting of individual units (monomers) linked in a chain-like structure" (Brown, et al., 2010). Polymer slurry is formed through the mixture of polyacrylamides and water. This mixture forms long chain-like molecules. These molecules are negatively charged, promoting molecular repulsion (O'Neill and Reese, 1999).

Polymer slurries, like mineral slurries, are used to provide excavation stability. Also like mineral slurries, polymers require a minimum amount of head to provide the needed hydrostatic pressure. Polymer slurries differ from mineral though, as their structure prevents the formation of a filter cake. Thus, to overcome the groundwater intrusion, a sufficient head differential must be maintained. To obtain excavation stability without the formation of a filter cake, polymer slurry continuously flows into the walls of the excavation. This is done at a slower rate due to the increased viscosity.

Figure 2.4 Polymer slurry exiting the form during concreting.

Figure 2.5 Close-up of a thicker viscosity polymer slurry exiting the form.

2.3.4 Blended Slurry

A blended slurry is the mixture of a mineral and polymer slurry, with bentonite typically used as the mineral slurry. This is done to take advantage of the benefits that each slurry type has to offer. However, blended slurries are not common and require expertise in the area beyond what is typically available. (Brown, et al., 2010)

2.3.5 **Marsh Funnel Test**

It has been established that slurry in general is used to provide excavation stability. This is generally executed by introducing the slurry to the excavation and maintaining it at an elevation of at least four feet above the groundwater table (FDOT, 2018). The slurry viscosity greatly affects the performance in maintaining stability. Slurry viscosity is tracked and measured through the Marsh funnel test method (API, 2009). This method works by timing how quickly a known volume of fluid discharges with falling head from a standardized funnel, Figure 2.6 (API, 2009). The unit of measurement for viscosity is seconds per quart. The thicker the slurry, the higher the Marsh funnel viscosity. For reference, the Marsh funnel viscosity of water is 26 sec/qt, this is the lower viscosity limit for drilling fluids. Typical workable ranges for mineral slurry are from 30 to 50 sec/qt (Brown, et al., 2010) and 50 to 90 sec/qt for polymer slurry (Mullins, et al., 2018).

Figure 2.6 A Marsh funnel viscosity test in progress.

2.4 **Concrete Selection for Drilled Shafts**

Placing concrete with a tremie pipe in a support-fluid-filled excavation is a unique construction method and as such the concrete used must possess certain specific basic characteristics: resistance to leaching and segregation, self-weight compaction, high durability (low porosity) and high workability. Of these, the workability is of primary importance as it determines the ability of a concrete to flow freely out of the tremie pipe and through the reinforcement cage to restore lateral stress against the sides of the excavation. The most effective way to accomplish this is with a highly fluid mix design. As mechanical vibration is impractical and could lead to unwanted mixing of concrete and support fluids, the mix design should be self-weight compacting. Additionally, drilled shaft concrete must maintain its fluid state throughout the depth of the excavation for the full time required to complete placement (Reese and O'Neill, 1999).

Concrete used for drilled shafts, considered Class IV concrete by FDOT, must be highly flowable so it can flow through the tremie pipe and fill the excavation. FDOT specifies a target slump of 8.5 in. and range of 7 to 10 in. (FDOT, 2018), whereas FHWA recommends a range of 7 to 9 in. (Brown, et al., 2010). FDOT further specifies that the concrete must maintain a minimum 5-in. slump throughout the entire concreting process (FDOT, 2018). In addition, FDOT mandates a minimum 28-day concrete compressive strength of 4,000 psi (FDOT, 2018).

Considering the large amounts of concrete used in the casting of a drilled shaft, construction temperature must be monitored, and measures should be taken to prevent excessive heat generation by the concrete (i.e. the concrete should have a low heat of hydration). Concrete must have low permeability to minimize corrosion potential and be cohesive in nature to resist leaching from the drilling fluid (Brown, et al., 2010).

2.5 Concrete Deficiencies

The realities of concrete flow in a drilled shaft (Figure 2.7) creates a situation for possible concrete deficiencies. As the concrete flows through the reinforcement cage it is cleaved by the vertical and horizontal rebar and then rejoins on the outside of the cage. This provides an inherit opportunity for laterally forming laitance interfaces to become trapped, creating creases in the cover concrete (Figure 2.7). These creases often extend to the surface and reflect the reinforcement cage layout (Figure 2.8); this phenomenon is termed *mattressing* by the Deep Foundations Institute (Beckhaus, 2016).

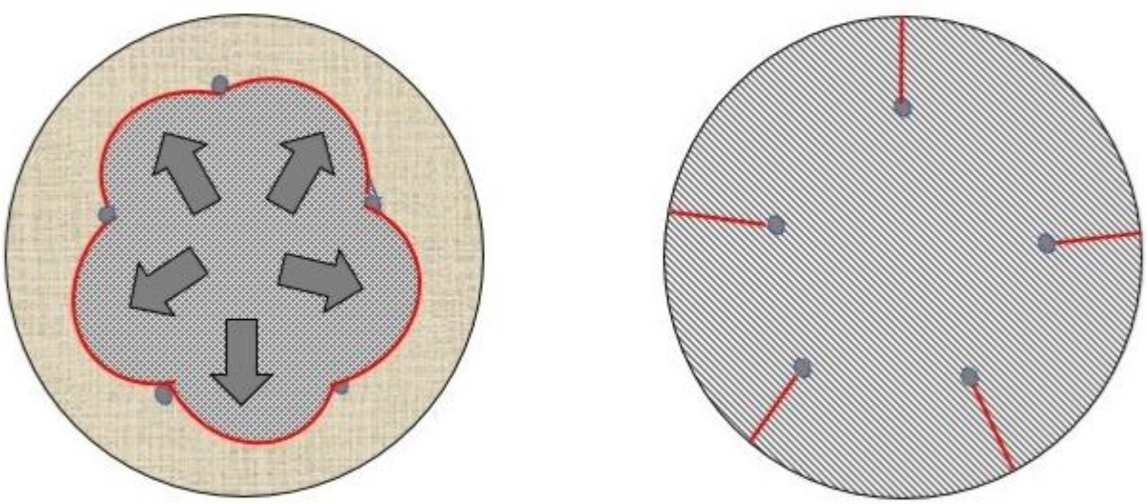


Figure 2.7 Laitance channel formation

Figure 2.8 Mattressing on a drilled shaft test specimen.

The potentially negative impacts of flow-induced anomalies and laitance crease formation are threefold: (1) laitance channels can create direct pathways for the transport of environmental chlorides to the reinforcing steel network, negating the protective qualities of the concrete cover and increasing the opportunity for corrosion, (2) the presence of trapped laitance can compromise the strength of the cover region and reduce structural capacity by cracking and reductions in the ultimate bending strength, and (3) the presence of trapped laitance and slurry along the shaft side walls has been attributed to filter cake formation, even in low-permeability soils. While filter cake formation is not the focus of this research, trapped laitance or slurry can be quantified by the surface texture for later consideration.

2.6 Self-Consolidating Concrete

Concrete flow remains one of the most significant issues when tremie-placing concrete in shafts. As a result, many studies have been performed to investigate the use of self-consolidating concrete (SCC) for drilled shafts. SCC is a highly flowable concrete, which contains viscosity modifying admixtures to prevent segregation. It is also classified by having a limited aggregate content and low water-to-powder ratio (Okamura and Ouchi, 2003). All studies examined thus far have concluded that SCC is an acceptable alternative to the standard FDOT Class IV drilled shaft concrete (Hodgson III, et al., 2005; Brown, et al., 2007; Rausche, et al., 2005); however, some have also decided it was not worth the cost as the same result could be accomplished using smaller diameter rounded aggregate (Brown, et al., 2007). SCC mixes also usually require higher amounts of binder, which are both expensive and heat producing. With shaft diameters greater than 4 ft, this presents new problems in controlling internal temperature levels and temperature distributions to prevent high temperature-induced cracking and decreases in durability (Mullins, et al., 2018).

2.7 Concrete Flow

The intended performance of fresh concrete is to expel and displace drilling fluid while the fluid concrete rises in the shaft. This is a blind process that is only recently being understood.

2.7.1 Misconception of Concrete Flow

In 2005, Deese and Mullins documented the misconception of concrete flow in drilled shafts. Previously, tremie-placed concrete was believed to rise in a uniform vertical layer across the shaft, displacing all slurry, and where upward flow in the cover would scour the soil side walls of slurry buildup as well as the reinforcement. Deese and Mullins showed instead that there was a buildup of concrete within the reinforcing cage before the concrete was then pushed radially through the cage into the cover region. For this to occur, a critical differential concrete height must be achieved between the center of the cage and cover region. This differential height was shown to be affected by clear spacing of the rebar, rate of concrete placement, and the maximum coarse aggregate size. This research showed that concrete does not displace slurry in a manner similar to oil over water, but instead slurry has the ability to become trapped in and/or against the sidewalls of the concrete. (Deese and Mullins, 2005)

Images of shafts have later shown the presence of this concrete buildup, but in cases where shafts were found to have been incompletely concreted. This further highlights that at the end of a concrete pour, when the concrete in the cage has reached the top of the cage, the cover concrete will still be lower and that only the toppling of concrete over the top edge of the cage will fill the final several feet of cover regions. This toppling action implies segregation of this concrete as it falls through the slurry in the cover. Figure 2.9 shows two shafts at different degrees of concrete completion when the tremie was extracted and where the buildup in the cage can be seen. In both cases, the tremie-placed portion of the concrete pour was assumed to be complete and a surface formwork was used to dry-pour the top of the shaft pedestal for a light mast fixture.

2.7.2 Impacts of Poor Concrete Flow and Slurry Environment

Summary of Studies

Numerous studies have shown that when compared to water- or polymer-cast shafts, the use of bentonite slurry in excavation support results in as much as a 50-percent reduction in concrete-to-soil bond (Majano, 1992; Brown, 2002; Lam, et al., 2014; Lam and Jefferis, 2015). Initial explanations credited this reduction in side shear to the filter cake that forms as the bentonite slurry flows into the surrounding soil and deposits clay particles on the excavation walls. While this is partially responsible, a far more significant mechanism for the deposition of bentonite along the side walls is from the trapping of bentonite due to the radial concrete flow (Caliari de Lima, 2017).

Figure 2.9 Concrete buildup inside the cage that provides lateral pressure to fill the cover.

Bowen (2013)

Bowen (2013) investigated the upper viscosity limit for bentonite slurry. However, during testing concrete flow issues were recognized. For this research, 18 small-scale shafts were cast that were 42-in. diameter and 24-in. tall using No.8 bars as reinforcement with 6-in. clear cover and spacing. A projection of the reinforcing cage could be seen on the concrete surface for almost all of the bentonite specimens (Figures 2.10 and 2.11). This was a result of laitance creases which form as radially flowing concrete fills the cover region, as depicted by Mullins and Deese.

While this research was focused on bentonite, several polymer shafts were cast as well. These shafts did not demonstrate the severity of mattressing noted with bentonite. Light crease lines were only noticed when closely examining the shafts. Bowen's work concluded by defining an upper viscosity limit for bentonite use in the state of Florida to be 40 sec/qt to prevent pronounced quilting/mattressing in bentonite cast shafts. (Bowen, 2013)

Figure 2.10 Shaft 7 30 sec/qt bentonite (left), shaft 8 40 sec/qt bentonite (right).

Figure 2.11 Shaft 4 55 sec/qt bentonite (left), shaft 5 90 sec/qt bentonite (right).

Lucas and Allen (2017)

Lucas and Allen (2017) evaluated the effects of slurry type and the time of exposure on the concrete soil interface. The motivation for the study was to define an upper time limit for slurry in-hole prior to concreting. The study involved casting 32, 1/10th scale shafts in sandy soil where the excavations were left open with slurry flowing into the surrounding soils for 2 to 96 hours. Full-scale shafts were also constructed where the open excavation times ranged from 1 to 48 hours. The small-scale shaft study showed polymer slurry had no time-related effects on the side shear (soil/concrete bond), but bentonite showed a marked reduction in side shear over the first 8 hours (33% with peak reduction at 8% per hour for the first 4 hours) and a more subtle reduction (0.1% per hour) thereafter (Figure 2.12).

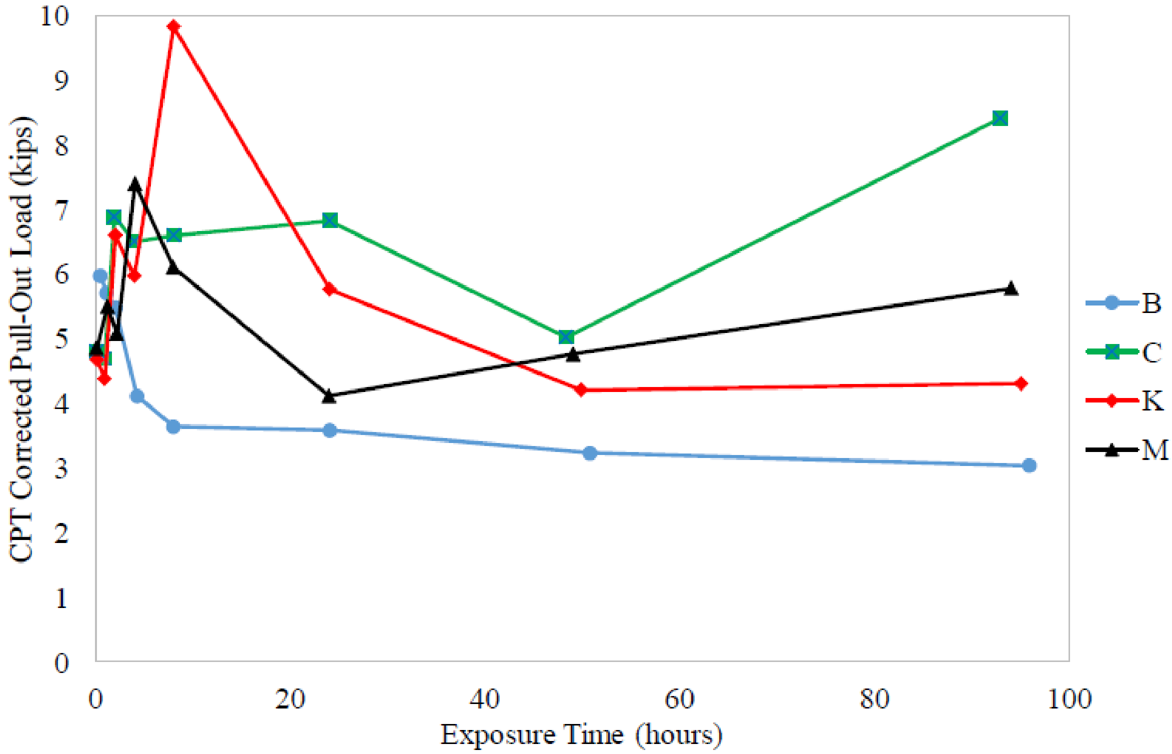


Figure 2.12 Effect of slurry type and exposure time on side shear pull-out resistance of small-scale shafts.

The full-scale results showed less drastic differences over the exposure time frame (2 to 48 hr) where the 2-hr and 48-hr bentonite shafts showed the same average side shear resistance (Figure 2.13). However, a reduction in side shear (16-26% at 1-inch displacement) was noted for bentonite-cast shafts when compared to polymer-cast shafts. As much as 38% reduction was noted between a high-viscosity polymer shaft (100 sec/qt) and the bentonite 40 sec/qt shaft (B0-40). In Figure 2.13, P denotes polymer; B, bentonite; 40, 60 and 100 describe the Marsh funnel viscosity in seconds per quart (Figure 2.6).

Unfortunately, both the small-scale and large-scale shafts tested did not have full reinforcing cages that Bowen (2013) showed to cause mattressing, and Deese (2004) noted the importance of building a head differential across the cage. In fact, Lucas and Allen conclude the lack of reinforcing cage (center bar only) may have increased the scouring effects in the large-scale test shafts, and therefore reduced the differences between shafts. Small-scale shafts, which showed up to a 55% reduction in side shear, were concreted with very low energy flow (1-2 ft of gravity head in a tremie) compared to the large-scale shafts that were pumped via concrete pump directly to the bottom of the shaft.

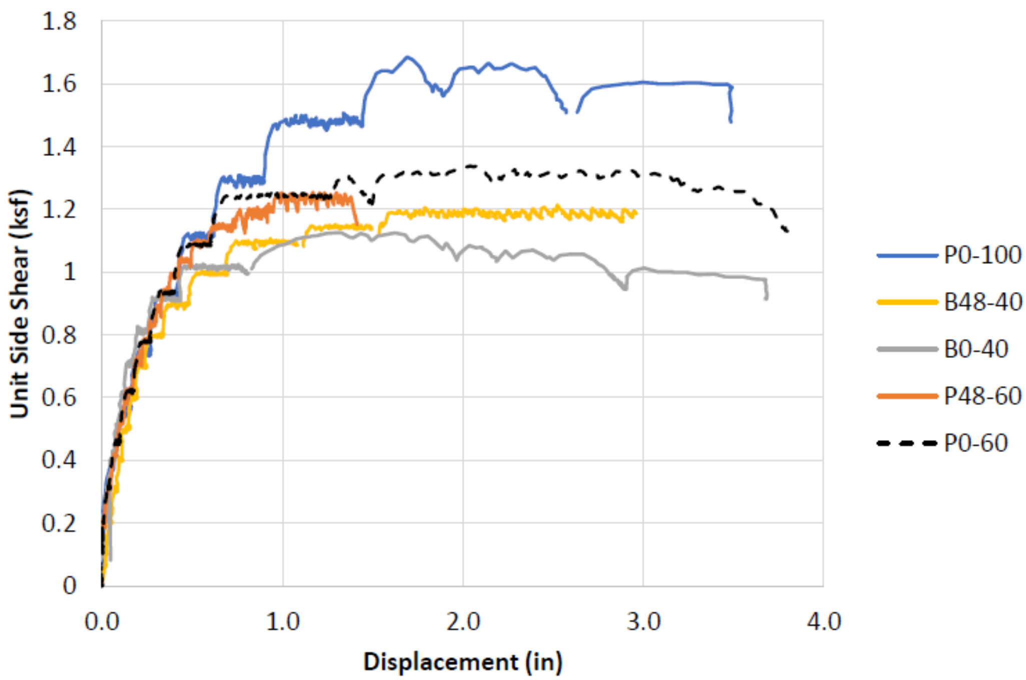


Figure 2.13 Results of full-scale shaft pull-out tests show 16-38% reduction in side shear from using bentonite.

2.8 Summary

This chapter provides the backdrop necessary to better understand concrete flow in tremie-placed, slurry displacing drilled shaft applications. In short, concrete flows radially into the annular cover region only after filling the inside of the cage to a sufficient height to overcome the head loss/flow restrictions of the cage lattice.

When fluid or slurry is present, the height necessary to produce the radial driving force is essentially doubled due to the buoyancy effects. Other factors affecting this height differential were shown to include the cage tightness and the speed at which the concrete is placed. In fact, head loss was shown to be proportional to the square of concrete flow, similar in theory and practice of pipe-flow head losses.

Radial flow is also the primary mode of laitance crease formations that lead to mattressing, which in turn provided the primary motivation and formed the main objective for this study: to define the effects of concrete flow on shaft performance.

Finally, the effects of slurry type were briefly introduced to show what is known with regards to the concrete-soil interface. Bentonite slurry, the most commonly used product and the most effective at maintaining soil-side-wall stability, has negative effects on side shear and those effects worsen with time of exposure.

Chapter Three: Test Specimens

The motivation for this study stemmed from observations of specimens cast for previous University of South Florida research projects (Costello, 2018, Mullins, et al, 2014; Bowen 2013). The first 22 shaft specimens were cast for a study in which the upper viscosity limit of bentonite and polymer support fluid was established (Bowen, 2013). Though previous work (Deese, 2005) introduced the concept of flow-related laitance creasing, it was only after the first set of forms was removed that mattressing was visually confirmed. The surface condition of several of the initial test shafts was alarming and warranted further verification. Thirty-six additional shafts were constructed using varied concrete types, support fluid characteristics, and reinforcement spacing in attempts to identify the variables pertinent to the problem. This study sought to quantify the magnitude of impacts resulting from mattressing.

Overall, a total of 58 test shafts were prepared. Each shaft measured 42 in. in diameter and 24 in. in height. Shaft specifics are provided in Table 3.1. Shaft 10 and shaft 44 were omitted from this study due to discontinuous reinforcement. Basic details for each concrete pour are included in Appendix A.

3.1 Casting of Test Specimens

This section details the construction of shafts 1 to 58. The details include form construction, reinforcing cage layout, concreting details, and concrete placement.

3.1.1 General Form Construction

The concrete forms used for casting the test specimens were prepared similarly for all concrete pours. First, collapsible children's pools of 6-ft. diameter and 15-in. height were laid out in the decided pour configuration as a means of slurry containment. This dimension was chosen as the volume of the pools was large enough to hold almost all (to be) displaced slurry, providing enough time for the extraction pump to prime and transport it to the slurry storage tank. Following this, a ¾-in. thick plywood sheet of 4 ft by 4 ft dimensions was carefully placed into the center of each pool. Next, the steel forms for the external walls of the shafts were prepared. The steel forms were fabricated using 18-gauge sheet steel rolled into a circular shape with angle iron welded to the ends to lock the forms. The ends of the forms were locked using three C-clamps on the upper, middle, and lower portions of the angles to form a circular shape. The dimensions of the enclosed steel forms resulted in shafts of 42-in. diameter and 24-in. height in all cases. The clamped steel forms were centered on the plywood.

Following form placement, a reinforcement cage, detailed in 3.1.2, was placed in the center of the steel form. Within each form, 2- x 4-in. wood blocks with a 1-in. holes cut out were attached from the cage to the form to aid in maintaining the circular nature of the forms and thus attaining the desired clear cover spacing along the perimeter of the shaft. These blocks were necessary to ensure proper clear cover was maintained during pouring operations.

To prevent fluid loss, silicone was used to seal the base of the steel forms to the plywood. Silicone was placed on both the inside and outside of the form, as well as along the seal where the angles were clamped together. Once the silicone was dry each form was tested to ensure that it was water tight. Figures 3.1 and 3.2 depict the overall form set up.

Figure 3.1 Forms set up prior to slurry and concrete placement.

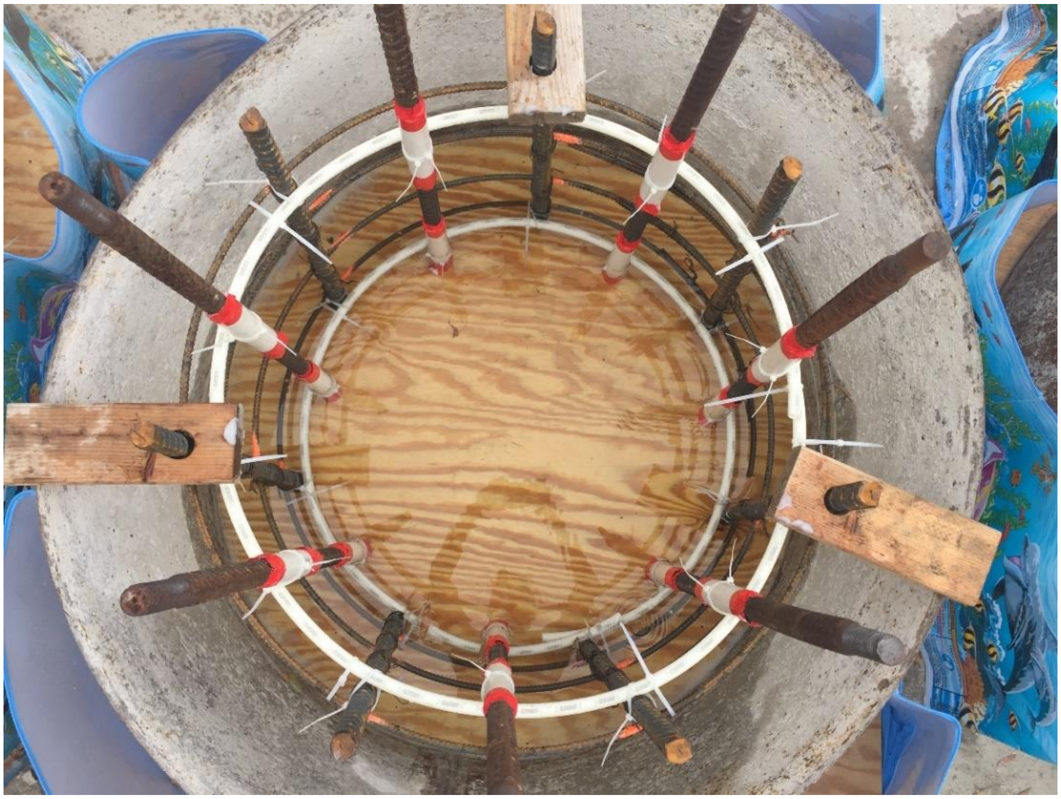


Figure 3.2 The inside of the form prior to slurry and concrete placement.

3.1.2 **Reinforcement Cage**

The reinforcing cage described in this section is nearly identical to that of shafts 1 to 18, previously cast by Bowen, aside from the varying bonded length. For construction, No. 8 reinforcement bars were used for the longitudinal steel and No. 3 stirrups were used for transverse steel. The cages were constructed to have 6 in. of clear spacing for the longitudinal and transverse steel, thus meeting the preferred tightest cage spacing criteria stated by the Florida Department of Transportation (FDOT, 2018). The clear cover used was also 6 in.

For cage construction the outer seven bars, which were used as structural reinforcement and not for pullout testing, were spaced and attached to the inside of the steel stirrups using steel tie-wire (Figure 3.3). The type of tie wire connection used was the quadruple-snap (Camilo Builes-Mejia, et al., 2014). Next, two rings of non-structural ½ in. polyethylene pipe (PEX pipe) were attached to the top and bottom hoops on the inside of the seven outer bars using plastic zip-ties. Finally, the debonded pullout bars were inserted and attached using zip-ties on the interior of the PEX pipe, this can also be seen in Figure 3.2. It should be noted that the steel stirrups did not come in contact with the longitudinal steel to be tested for pullout capacity.

Figure 3.3 Outer seven longitudinal bars attached to the inside of the transverse reinforcement.

The construction of the pullout specimens began by cutting to a length of 4 ft in order to accommodate the hydraulic ram and steel spacers used for testing. The bars were machined down to 7/8 in. diameter on the top and subsequently threaded to provide a point of resistance for the hydraulic ram during testing. Debonding on each bar was accomplished using 1-in. thin walled PVC pipe sealed with tape on the upper 8 inches and lower 10 inches of embedment length, resulting in a 6-in. bond length per bar (Figure 3.4).

Figure 3.4 The ~~six~~ six-inch debonded section of bar through the use of PVC pipe sections.

The 6-in. bonded length was determined by Bowen to be ideal for testing. For a deformed No. 8 bar the required development length according to ACI 318-14 is 47 in. (for 4,000 psi concrete); however, given the size of the test shafts, this dimension was unattainable. Bowen started with an initial bonded length of 18 in.; however, the pullout capacity was much higher than expected. He then gradually decreased the bonded length, finding the ideal length to be 6 in. (Bowen, 2013). Note: the 7/8-in. threaded ends on the 1-in. diameter bars resulted in a 60% reduction in bar area affecting the available tensile force for testing.

3.1.3 **Slurry**

For testing, varied slurry products and viscosities were used. As some of the slurry information is the proprietary property of the manufacturer, only basic descriptions have been provided. All sets of test shafts cast after the initial 18 by Bowen include a water-cast shaft as a control for the set.

Two mineral slurries were used for testing: bentonite and attapulgite. The bentonite used was a pure bentonite. The same product was used for all bentonite-cast specimens. The same attapulgite product was also used throughout the testing program. For polymer slurry, products from three different manufacturers were tested. For comparison purposes, the same number of pullout bars was prepared per manufacturer. Slurry viscosities were tested and recorded using the Marsh funnel method described in 2.3.5 prior to and on the day of concreting.

3.1.4 **Concreting**

Per FDOT specifications, slurry was always placed in the forms the morning of concreting no more than 8 hr before concreting (FDOT, 2018). All shafts were cast using tremie placement to simulate field conditions.

3.1.5 **Slump**

At the beginning of each pour, a concrete slump/flow test was performed. Standard slump test equipment was used for Class IV concrete and a wider base plate (spread board) was used for SCC concrete to measure the spread. This section will discuss both test procedures.

Standard Slump Test

The standard slump test (ASTM C143) is administered by first attaching the slump cone to a level base. The cone is then filled in three equal layers, with each layer being rodded 25 times. The top layer is then leveled off so that it is flush with the top of the cone. The cone is then lifted carefully off the base (within 3 to 5 seconds) to allow the concrete to spread or 'slump'. The removed cone is then flipped upside-down and placed on the base. The slump of the concrete is the measurement from the top of the 'slumped' concrete to the top of the cone. This process in shown in Figure 3.5.

Figure 3.5 The phases of performing a standard slump test; from left to right: rodding 25 times per layer, leveling the top of the cone, removing the cone to measure the slump value.

SCC Spread/Slump Test

The SCC slump is tested and measured in a different manner as SCC is highly fluid and does not hold the shape of the cone once removed. For this test, the same slump cone is used, however with a wider base plate (spread board). The slump cone is placed upside-down on the base plate and then completely filled with fresh concrete (Figure 3.6). The cone is once more carefully lifted allowing the concrete to spread (Figure 3.6). The diameter of the concrete is then measured at two perpendicular locations and the average is recorded as the concrete spread.

Figure 3.6 SCC slump/spread test where the standard slump cone is filled upside-down (left), the cone is lifted (center), and the diameter of the spread is measured (right).

Cylinder Preparation

Following slump testing, 4-in. by 8-in. cylinders were prepared to evaluate the concrete compressive strength. Preparation methods varied for Class IV concrete and SCC. Class IV concrete cylinders were made in accordance with ASTM C192 by adding concrete in two layers, rodding each layer 25 times, then leveling the concrete at the top, (Figure 3.7). SCC cylinders were simply filled and leveled, (Figure 3.8).

Figure 3.7 A Class IV concrete cylinder post leveling.

Figure 3.8 SCC cylinder preparation: pouring the concrete into cylinders (left) and leveling the tops (right).

Concrete Mix

The previously constructed 18 shafts were all cast with Class IV concrete from Preferred Materials, Inc. Five test shafts were constructed using self-consolidating concrete where two providers were used, Preferred Materials, Inc. and Argos USA.

Shafts 19 to 22

Shafts 19 to 22 were cast with Class IV concrete from Preferred Materials, Inc. The primary purpose of these samples was to test the pullout capacity of the second polymer manufacturer, thus the concrete properties had to be comparable. The slump of this concrete was measured to be 8.5 in. (Figure 3.9). Shaft properties can be found in Table 3.1.

Table 3.1 Slurry type, viscosity, concrete mix, and bonded length for shafts 19 to 22.

Shaft #	Concrete Mix	Slurry Type	Marsh Funnel Viscosity (sec/qt)	Bonded Length (in.)
19	Preferred Class IV	Polymer 2	63	6
20		Polymer 2	121	6
21		Bentonite	42	6
22		Water	26	6

Figure 3.9 Measurement of 8.5-in. slump for shafts 19 to 22.

Shafts 23 to 24

Shafts 23 and 24 were cast at the same time as 19 to 22; however, with Preferred SCC instead of Class IV concrete. Table 3.2 shows the slurry type and viscosity for each cast shaft in this pour and the previous Preferred SCC shafts. These shafts were cast prior to the purchasing of proper spread testing equipment and thus a spread was not recorded for these specimens.

Table 3.2 Slurry type, viscosity, and concrete mix for shafts 23 to 24.

Shaft #	Concrete Mix	Slurry Type	Marsh Funnel Viscosity (sec/qt)
23	Preferred SCC	Water	26
24		Bentonite	40

Shafts 31 to 36

Shafts 31 to 36 were cast with the same Class IV concrete from Preferred Materials, Inc. that was used for shafts 1 to 22. The purpose of these specimens was to test the varying viscosities of attapulgite slurry in terms of concrete flow and pullout capacity so that this data could be compared to the other slurry types. This pour also tested two more shafts using polymer manufacturer 2. The slump of this concrete was determined to be 7 in. by the standard slump test; (Figure 3.10). Table 3.3 displays shaft details such as slurry type, viscosity, and bonded length.

Table 3.3 Slurry type, viscosity, concrete mix, and bonded length for shafts 31 to 36.

Shaft #	Concrete Mix	Slurry Type	Marsh Funnel Viscosity (sec/qt)	Bonded Length (in.)
31	Preferred Type IV	Polymer 2	98	6
32		Water	26	6
33		PG Bentonite	39	6
34		Attapulgite	39	6
35		Attapulgite	200+	6
36		Polymer 2	47	6

Figure 3.10 Measurement of 7-in. slump for shafts 31 to 36.

Shafts 47 to 49

Test shafts 47 to 49 were cast with a different provider of SCC. The distributor selected was Argos USA. Slurry conditions were kept close to the SCC shafts already cast. The spread of this concrete was 28 in., Figure 3.11. Shaft details can be found in Table 3.4.

Table 3.4 Slurry type, viscosity, concrete mix, and bonded length for shafts 47 to 52.

Shaft #	Concrete Mix	Slurry Type	Marsh Funnel Viscosity (sec/qt)	Bonded Length (in.)
47	Argos SCC	Water	26	6
48		PG Bentonite	39	6
49		PG Bentonite	31	6

Figure 3.11 Measurement of spread for Argos SCC mix for shafts 47 to 52.

Shafts 53 to 58

Test shafts 53 to 58 were cast to investigate a third polymer manufacturer. To be consistent, Class IV concrete from Preferred Materials was used. The slump of this concrete was measured to be 9 in. by the standard slump test; (Figure 3.12). Table 3.5 displays shaft details such as slurry type, viscosity, and bonded length.

Table 3.5 Slurry type, viscosity, concrete mix, and bonded length for shafts 47 to 52.

Shaft #	Concrete Mix	Slurry Type	Marsh Funnel Viscosity (sec/qt)	Bonded Length (in.)
53	Preferred Class IV	Polymer 3	49	6
54		Polymer 3	58	6
55		Polymer 3	120	6
56		Polymer 3	85	6
57		Bentonite	40	6
58		Water	26	6

Figure 3.12 Measurement of 9-in. slump for shafts 53 to 58.

Concrete Placement

Once the concrete slump/spread was approved, the concreting process was initiated. For each shaft the tremie pipe was prepared by capping the base with a metal plate, placing a plastic bag over the plate/base of the tremie pipe, and taping to seal/hold the connection (Figure 3.13). The capped tremie pipe was then inserted into the center of the shaft and lowered to the bottom of the form. Next the tremie pipe was filled with concrete from the chute. Once full, the tremie pipe was slightly elevated allowing the concreting process to begin and displacing the slurry from the shaft form (Figure 3.14). Once placement was completed the tops of the shafts were levelled and labeled. Post concreting, the samples were allowed time to achieve at least three-quarters of their design compressive strength before the steel forms were removed.

Figure 3.13 Tremie pipe preparation; placement of cap (left), covering of cap with plastic (center), taping of plastic around (not to) tremie pipe (right).

Figure 3.14 Displacement of slurry through the concreting process.

Table 3.6 Summary of all 58 shaft specimens

Shaft #	Concrete Mix	Support fluid Type	Viscosity	Average Pullout Strength (kip)	Average Concrete Compressive Strength (psi)
1	4KDS	PG Bentonite	44	57.234	6150
2	4KDS	PG Bentonite	105	49.704	6150
3	4KDS	PG Bentonite	40	36.894	4358
4	4KDS	PG Bentonite	55	32.697	4358
5	4KDS	PG Bentonite	90	38.094	4358
6	4KDS	Water	26	54.304	4358
7	4KDS	PG Bentonite	30	28.754	4530
8	4KDS	PG Bentonite	40	24.212	4530
9	4KDS	PG Bentonite	50	20.524	4530
10	4KDS	PG Bentonite	90	23.139	4530
11	4KDS	SP Polymer	65	32.338	4530
12	4KDS	SP Polymer	66	33.941	4530
13	4KDS	PG Bentonite	30	25.636	4753
14	4KDS	PG Bentonite	30	27.641	4753
15	4KDS	PG Bentonite	56	19.804	4753
16	4KDS	SP Polymer	85	24.077	4753
17	4KDS	SP Polymer	85	26.247	4753
18	4KDS	Water	26	34.042	4753
19	4KDS	KBI Polymer	63	20.9	4100
20	4KDS	KBI Polymer	121	19.3	4100
21	4KDS	PG Bentonite	42	20.7	4100
22	4KDS	Water	26	21.8	4100
23	SCC	Water	26	Not Tested	Not Tested
24	SCC	PG Bentonite	40	Not Tested	Not Tested
25	SCC	Attapulgite	40	14.3	5210
26	SCC	Water	26	Not Tested	5210
27	SCC	PG Bentonite	40	Not Tested	5210
28	SCC	SP Polymer	90	Not Tested	5210
29	SCC	SP Polymer	50	Not Tested	5210
30	SCC	PG Bentonite	30	Not Tested	5210
31	4KDS	KBI Polymer	98	42.6	6150
32	4KDS	Water	26	41.8	6150
33	4KDS	PG Bentonite	39	42.3	6150
34	4KDS	Attapulgite	39	43.6	6150
35	4KDS	Attapulgite	200+	37.6	6150
36	4KDS	KBI Polymer	47	48.9	6150
37	SCC	Water	26	Not Tested	Not Tested
38	SCC	SP Polymer	55	Not Tested	Not Tested

Table 3.6 (Continued)

Shaft #	Concrete Mix	Support fluid Type	Viscosity	Average Pullout Strength (kip)	Average Concrete Compressive Strength (psi)
1	4KDS	PG Bentonite	44	57.234	6150
40	SCC	PG Bentonite	41	Not Tested	Not Tested
41	SCC	PG Bentonite	28	Not Tested	Not Tested
42	SCC	Attapulgite	37	Not Tested	Not Tested
43	4KDS	PG Bentonite	37	Not Tested	Not Tested
44	4KDS	PG Bentonite	37	Not Tested	Not Tested
45	4KDS	PG Bentonite	37	Not Tested	Not Tested
46	4KDS	Water	26	Not Tested	Not Tested
47	SCC	Water	26	Not Tested	9128
48	SCC	PG Bentonite	39	Not Tested	9128
49	SCC	PG Bentonite	31	Not Tested	9128
50	SCC	SP Polymer	57	Not Tested	9128
51	SCC	SP Polymer	90	Not Tested	9128
52	SCC	Attapulgite	39	Not Tested	9128
53	4KDS	Matrix Polymer	49	Not Tested	5940
54	4KDS	Matrix Polymer	58	Not Tested	5940
55	4KDS	Matrix Polymer	120	Not Tested	5940
56	4KDS	Matrix Polymer	85	Not Tested	5940
57	4KDS	PG Bentonite	40	Not Tested	5940
58	4KDS	Water	26	Not Tested	5940
PG Bentonite- CETCO Puregold Gel ©					
SP Polymer – Shore Pac ®					
KBI Polymer – SlurryPro® CDP ™					
Matrix Polymer- Big Foot ®					

Chapter Four: Rheology Modeling

4.1 Overview

To better understand the phenomena causing the observed quilting in the cover region, it is necessary to examine how tremie-placed concrete flows into a reinforced, slurry-filled excavation. As stated previously, the commonly held concept that concrete displaces slurry like oil over water in a flat rising surface is a misconception. Deese and Mullins (2005) have shown that concrete, in fact, builds up a head inside the reinforcement cage prior to flowing radially into the annular cover region (Figure 4.1). While physical studies provide visual verification, numerical modeling is a cost-effective means by which to test variations in concrete material properties and reinforcement configuration. The modeling contained herein considers the use of a self-consolidating concrete as well as a typical FDOT Class IV drilled shaft mix design.

Self-consolidating concrete (SCC) is a highly flowable concrete mix that is increasingly chosen over standard mix designs due to its workability. There is a notable difference between the way self-consolidating concrete fills a drilled shaft excavation as opposed to other structural members like roof slabs, beams, and columns. The drilled shaft is cast below ground level, and the casting is done in the shaft that is formed by the drilled excavation of soil to the specified size and depth from the ground level. In most cases, the in situ soils act as the formwork and the excavation establishes the shape of the shaft concrete. In Florida, due to the presence of a high water table, drilling fluid is used to stabilize the excavation walls and concreting is always tremie-placed.

Considering the above factors, in the simulation of SCC flow in the drilled shaft, the following mechanisms are to be taken into account:

- Flow of SCC through the tremie pipe placed at the center of the shaft. The tremie extends from the top of the shaft to the bottom of the shaft leaving a gap of 6 to 10 inches from the excavation bottom.
- SCC flowing out from the tremie pipe spreads out radially throughout the entire cross section of the shaft, passing through the reinforcement steel and displacing the drilling fluid.
- Filling of SCC in the shaft from bottom to top and displacing the drilling fluid.

Figure 4.1 shows the above flow mechanisms in the drilled shaft in an idealized and actual flow pattern.

Even though the objective is to create a 3-D model simulation, 2-D modeling was initially performed as a precursor, given the challenges of the flow involving (1) resolution of the concrete-slurry interface, (2) the rheology of the fluids, and (3) the complex flows expected potentially characterized by pockets of slurry trapped within the concrete.

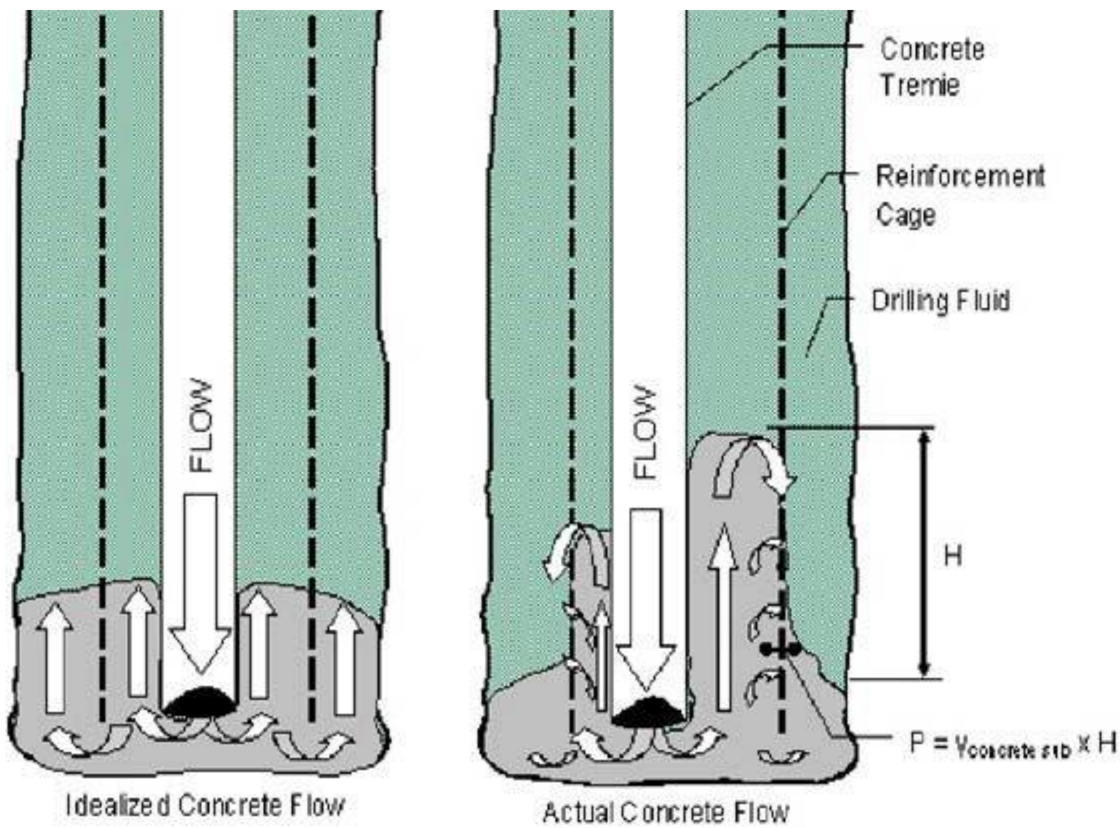


Figure 4.4.1 Flow Mechanism in Drilled Shaft: Idealized and Actual Flow Pattern (Deese and Mullins, 2005).

The numerical modeling and simulation of SCC flow in a drilled shaft can also be applied to normal concrete flow in a drilled shaft. Hence, in the following sections, the flow mechanism is referred to as concrete flow that is applicable to both normal drilled shaft concrete (NC) and SCC. Analysis is performed for both SCC and NC flows.

4.2 Modeling and Simulation in 2-D

2-D modeling and simulations have been performed with a finite element CFD (computational fluid dynamics) model built using COMSOL Multiphysics® software (COMSOL, 2016). The COMSOL CFD Module with incompressible laminar fluid flow has been considered for the modeling. The simulation of concrete flow in a drilled shaft involves two fluids consisting of the concrete flowing from the tremie pipe and the outgoing slurry from the excavation, displaced by the concrete. Tracking the moving interface between the two fluids is of prime importance in the study of the flow patterns of SCC and NC that is required for the evaluation of SCC for use in drilled shafts. Considering the axis symmetry of the shaft, an axisymmetric model was developed so that the problem size and computational expense could be greatly reduced. The modeling and simulation process can be divided into two parts:

Pre-processing, in which the model geometry is entered, the mesh is generated, boundary conditions are set, and the fluid properties are assigned. Computation, in which the fluid mechanics equations are numerically solved on the mesh.

4.2.1 Model Geometry

Geometry is formed as a combination of solid objects, mainly rectangular shapes, joined using Boolean operations like union, intersection, and difference. For this study, as shown in Figure 4.2, a shaft excavation of four-foot diameter and five-foot depth is modeled as a rectangular element. The steel rebars are modeled as horizontal elements (stirrups) with gaps that match the spacing in a full-scale shaft. In the geometry, the 10-inch diameter tremie pipe is modeled as a rectangular element. The tremie pipe element is provided at the center of the drilled shaft from the top of the shaft to 6 inches above the bottom of excavation. Note that the model shown in Figure 4.2 does not distinguish between the upward flow of concrete between the cage and tremie, and the upward flow between the cage and wall. The flow demonstrated is simply from the inlet and outlet shown.

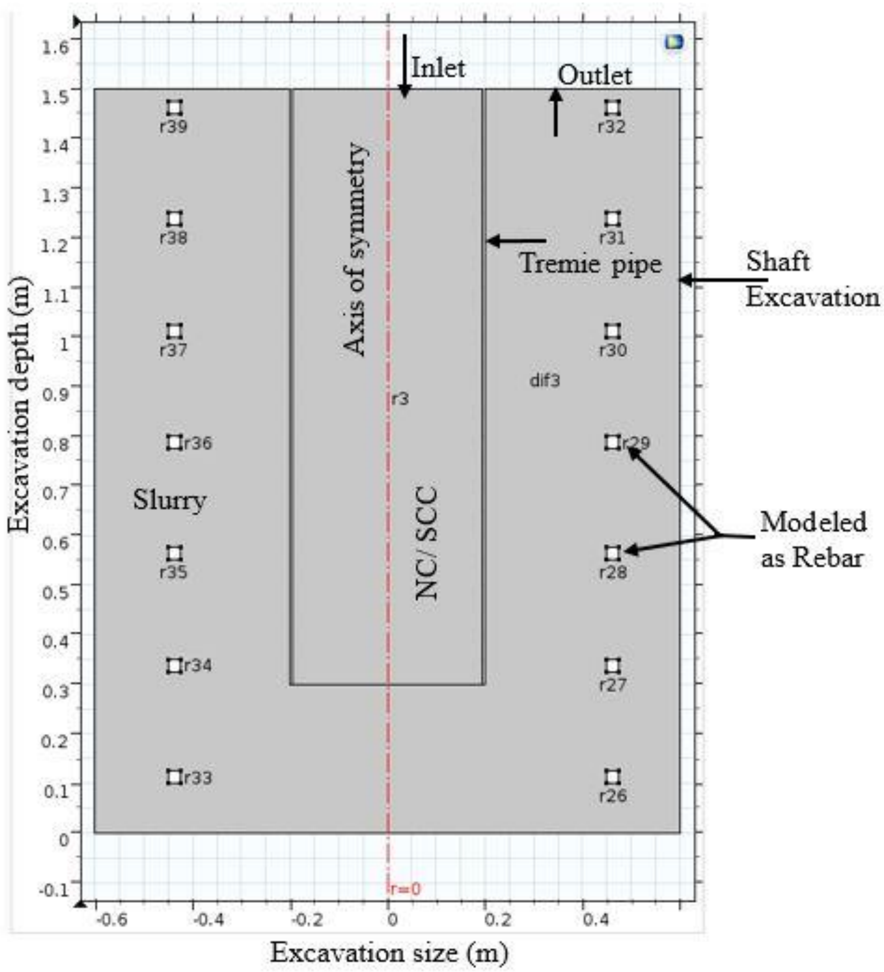


Figure 4.4.2 2-D Model in COMSOL: Shaft Model Geometry.

4.2.2 **Material Properties**

The material properties, used in the modeling, for SCC, NC, and the slurry are given in Table 4.1.

Table 4.1 Properties of Concrete, SCC, and Slurry.

Materials	Properties	
	Density in lb/ft^3	Viscosity Pa-s
NC	156 (2,400)	50 - 100
SCC	150 (2,300)	20 - 100
Slurry	75 (1,150)	0.10 - 0.5

4.2.3 **Boundary Conditions**

Boundary conditions have been set at the inlet and outlet of the axisymmetric model (see Figure 7.2). The inlet for the concrete flow is at the bottom of tremie pipe where the concrete flows out into the shaft excavation. The outlet is at the top of the excavation where the slurry flows out of the excavation. Velocity at the inlet and pressure at the outlet have been given as the boundary conditions. The inlet velocity value given is equivalent to the velocity of concrete flow in the tremie pipe, which was calculated from the concrete supply rate from the truck at the field. Considering a 10-cubic yard capacity truck discharging concrete in the shaft excavation in 20 min, the inlet velocity is calculated for the tremie size. The no-slip condition is enforced at the vertical side wall of the shaft excavation and at the bottom of the excavation. At the rebar surface, free slip is considered.

4.2.4 **Rheological Model**

Apart from initial computation that assumed Newtonian fluid behavior for both the concrete and the slurry, the axisymmetric model also considered non-Newtonian behavior for concrete and Newtonian behavior for slurry. With the CFD module, a predefined Carreau model has been used to input the viscosity values for concrete, and the model describes the variation of viscosity with shear rate (Andrade et al., 2007). For slurry, Newtonian behavior has been followed as the flow behavior is expected to be close to that of water.

4.2.5 **Governing Equations**

The fluid motion is governed by the Navier-Stokes equation, which is Newton's second law of motion for fluids (Hibbeler, 2015):

$$\rho \frac{\mathbf{Du}}{\mathrm{Dt}} = \rho\, \mathbf{g} - \nabla p + \nabla\cdot(\mu\, (\nabla \mathbf{u} + (\nabla \mathbf{u})^{T})) \tag{4.1}$$

The Navier-Stokes equation is a vector equation with components in x, y, and z directions. In these equations $\nabla = (\frac{\partial}{\partial x} + \frac{\partial}{\partial y} + \frac{\partial}{\partial z})$ and $\nabla^2 = (\frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} + \frac{\partial^2}{\partial z^2})$. Also, **u** is the velocity vector, **g** is gravity, **p** is pressure, **ρ** is density, and **μ** is viscosity. The left side of the equation represents acceleration. The three terms on the right side are the gravitational force, the pressure gradient, and the viscous term.

For an incompressible flow, the conservation of mass or continuity equation is

$$\nabla \cdot \mathbf{u} = 0 \tag{4.2}$$

which is solved in conjunction with the Navier-Stokes equation.

4.2.6 Level Set Method

In the case of two-phase or multi-fluid flow, the objective is to characterize the moving interfaces. The two important methods to characterize the moving interfaces are interface-tracking and interface-capturing techniques. In COMSOL, the interface is captured by using the Level Set method, which is an interface tracking technique. This method is widely adopted and is well-suited to applications where topological changes and/or sharp corners are present. In the Level Set method, a smooth function denoted as α is the level set function, which is used to represent the interface between the two fluids: the concrete (conventional shaft concrete or SCC) and drilling fluid (slurry or water). The interface is represented by the 0.50 contour of the function α, which divides the fluid region into zones, each corresponding to the two fluids. The zones where α is less than 0.5 correspond to concrete and the zones where α is greater than 0.5 correspond to slurry. For incompressible multi-fluid flows, the level set equation that is solved in the CFD module of COMSOL is as follows:

$$\frac{\partial \alpha}{\partial t} + \mathbf{u} \cdot \nabla \alpha = \gamma \, \nabla \cdot \left\{ \varepsilon \, \nabla \alpha - \alpha(1-\alpha) \frac{\nabla \alpha}{|\nabla \alpha|} \right\} \tag{4.3}$$

The left side of the equation defines the motion of the interface while the right side provides stabilization or reinitialization. The parameter ε defines the interface thickness and parameter γ introduces the intensity of reinitialization or stabilization. For example, if γ is too small, the thickness of the interface may not remain constant and parasitic oscillations may appear in the level set function α. If γ is too large, the interface may move incorrectly. As noted in the COMSOL manual, a recommended value of γ is the maximum absolute value of the flow velocity (COMSOL, 2016).

For incompressible two-phase flow, the above level set equation is coupled with the Navier-Stokes equation as shown below:

$$\rho \frac{\mathbf{Du}}{\mathrm{Dt}} = \rho \, \mathbf{g} - \nabla p + \nabla \cdot (\mu \, (\nabla \mathbf{u} + (\nabla \mathbf{u})^{T})) + \sigma K \delta n \tag{4.4}$$

where **σ** is the surface tension, **K** is curvature of the interface, **n** is the unit normal to the interface, and **δ** is the delta function. The expression σKδn denotes the surface tension force at the interface. Furthermore, the density and viscosity are defined as,

$$\rho = \rho_1 + (\rho_2 - \rho_1)\,\alpha \tag{4.5}$$

$$\mu = \mu_1 + (\mu_2 - \mu_1)\,\alpha$$

where, ρ_1 and ρ_2 are the densities of concrete and slurry, respectively, and μ_1 and μ_2 are the dynamic viscosities of concrete and slurry, respectively.

4.2.7 **Meshing**

The model geometry has been partitioned with a triangular element mesh consisting of 48,972 elements shown on Figure 4.3.

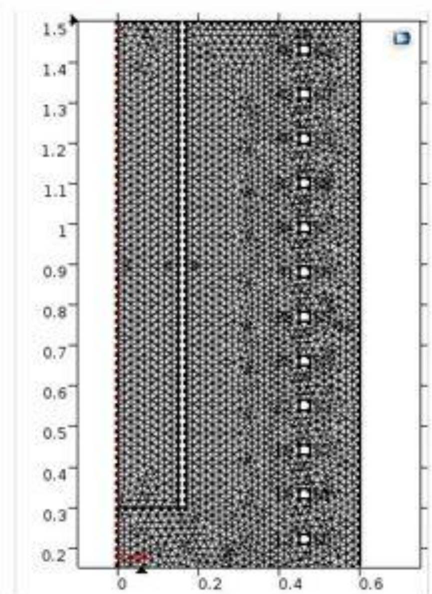


Figure 4.4.3 2-D Model in COMSOL: Shaft Model Meshing.

4.2.8 **Computations**

Computations are performed using the BDF (Backward Differentiation Formulas) time-dependent solver for a four-minute simulation time. The motion of the interface is captured and thus the concrete flow patterns are obtained. The flow patterns, in terms of the volume fraction of concrete, are saved at 15-second time intervals and the plots are extracted at the required time intervals. A typical volume fraction plot with the concrete flow pattern from the COMSOL output is shown in Figure 4.4. The flow pattern clearly shows a concrete head differential between inside and outside the reinforcement. This phenomenon has been observed in the field measurements carried out by Mullins and Ashmawy, (2005), as described earlier.

The computations are performed to study the concrete flow pattern for different concrete viscosities, different rebar spacing, and different concrete flow velocities. In the computations shown in Figure 4.5, concrete viscosities of 100 Pa-s at infinite shear rate and 1,000 Pa-s at zero shear rate, 7 rebars at a 9-in. vertical spacing, and a concrete flow velocity of 1.14 ft /min (0.0058 m/s) are considered. Figure 4.5 shows the concrete flow pattern at various flow time intervals. For example, the head differential is 10 in. (250 mm) at 210 s flow time. Figure 4.6 shows variation of concrete head differential with respect to the concrete inflow velocity when the other parameters, like viscosity values of concrete and drilling fluid, shaft size, and rebar arrangement, are kept the same.

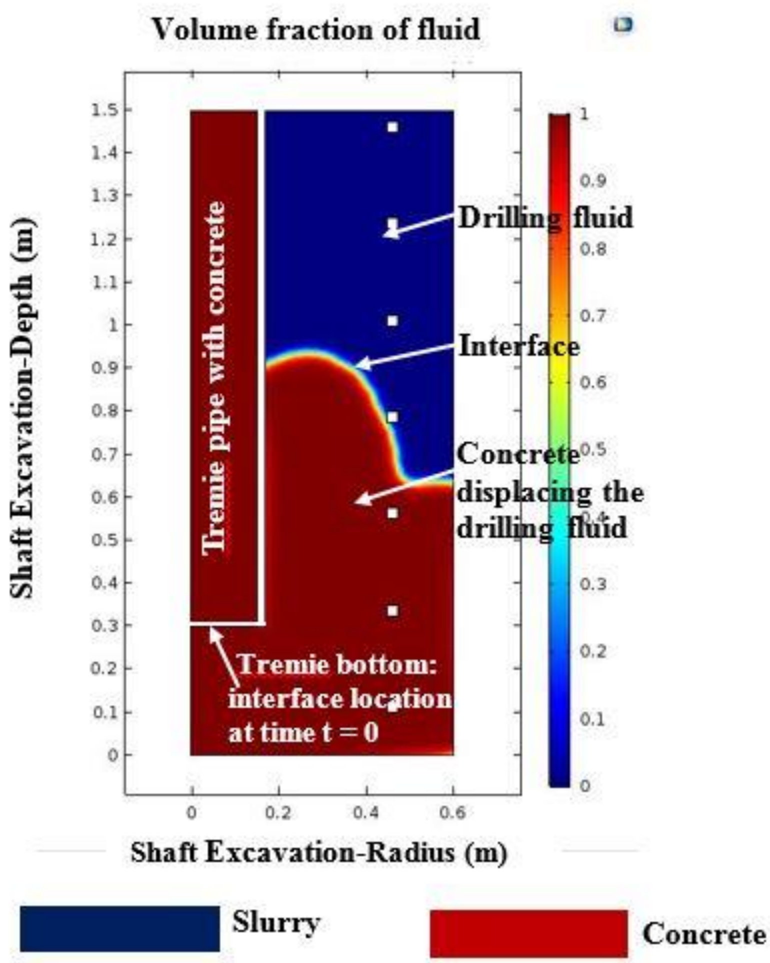


Figure 4.4.4 2-D Model in COMSOL: Typical Volume Fraction Plot, the color scale to the right represents α.

As expected from field measurements, the head differential, H_{diff}, increases with increasing concrete inflow velocity (Deese, 2005; Mullins and Ashmawy, 2005). Figure 4.7 shows H_{diff} plots for different rebar spacing. The values are taken at 210 s. When studying the effect of rebars, there is an increase in the concrete head as the rebar spacing reduces or as the number of bar increases. Similar scenarios were observed during shaft construction in the field by others where the concrete flow was affected by the rebar blockage (Hodgson et al., 2005).

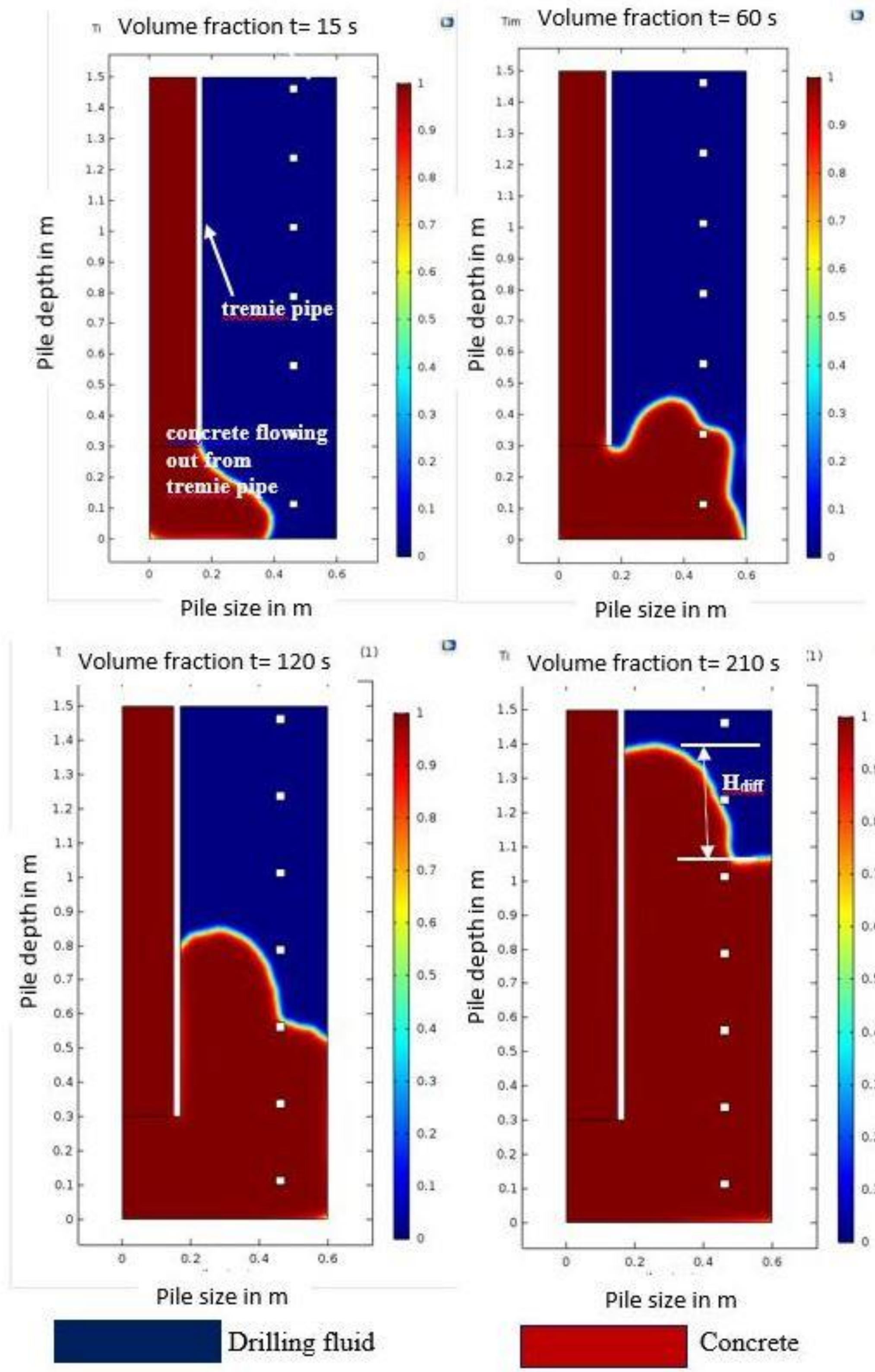


Figure 4.4.5 2-D Model in COMSOL: Concrete Flow Patterns at Different Flow Time Intervals.

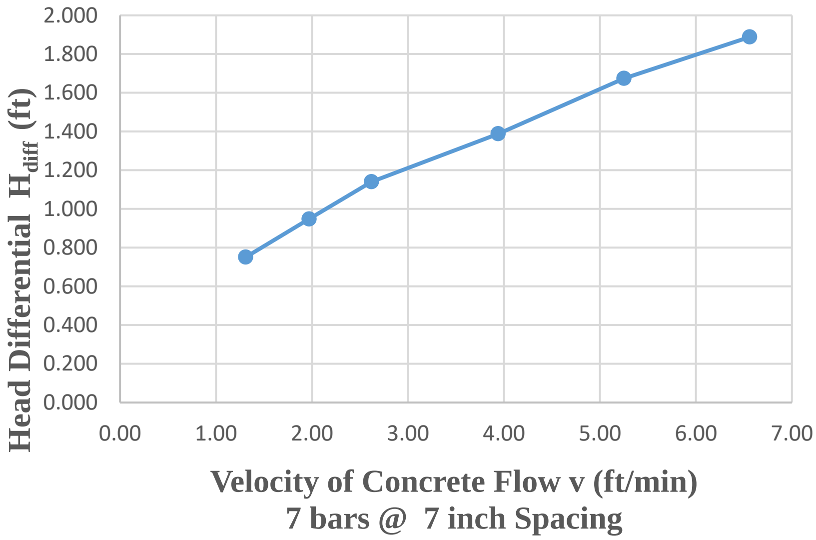


Figure 4.4.6 2-D Model in COMSOL: Concrete Flow Velocity (v) vs. Head Differential (H_{diff}).

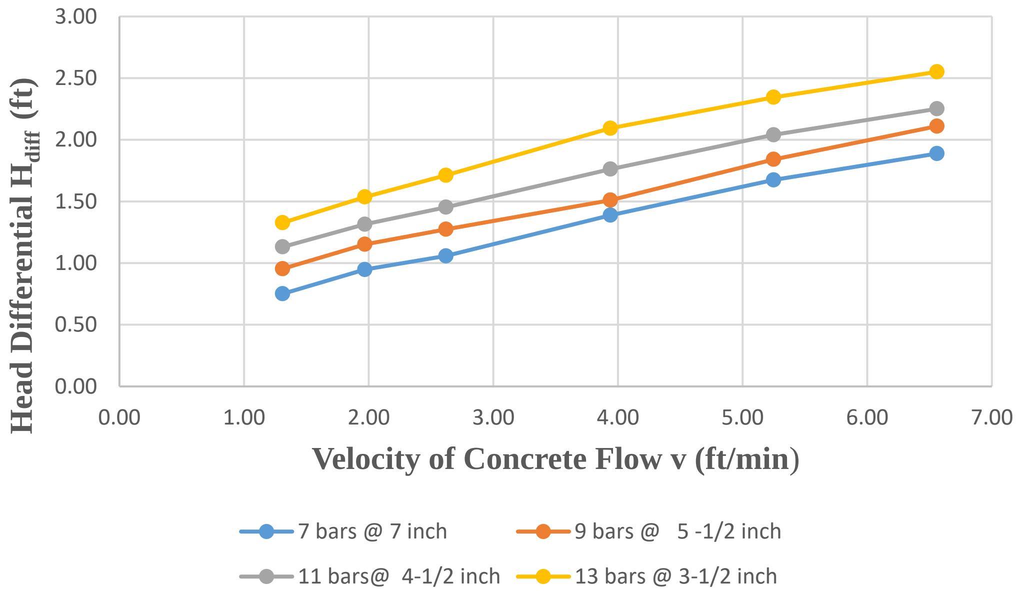


Figure 4.4.7 2-D Model in COMSOL: Concrete Flow Velocity (v) vs. Head Differential (H_{diff}) for Different Rebar Spacing.

4.3 Limitations of 2-D Simulations

Considerable approximations had to be made in the 2-D modeling. The concrete flow from the central tremie pipe to the excavation and the rebar arrangement are not fully representative of conditions in the field. For example, vertical rebars were not considered in the model geometry. In spite of these limitations and approximations, the flow pattern from the simulation shows the expected phenomena. However, performing the simulation using a 3-D shaft model was anticipated to provide a closer agreement with experimental data.

4.4 Modeling and Simulation in 3-D

4.4.1 3-D Simulation in COMSOL

In the case of 3-D modeling, the number of nodes and number of elements were expected to be larger when compared to the corresponding 2-D model developed. Hence, selecting the suitable shaft size for the 3-D model was important in order to perform the analysis in a reasonable amount of time and obtain reliable results. Thus, a somewhat unrealistic 3-D shaft model was initially used with an 18-inch diameter and 20-inch depth to perform the analysis. For this simulation, the mesh size was in the range of 0.08 to 1.10 in.

The tremie pipe size was four-inch diameter and centrally placed from the top of the shaft to six inches above the shaft bottom. Reinforcement cages were placed at four-inch cover. The vertical rebars were six No. 8 bars (one-inch diameter) placed at five-inch spacing. The horizontal ties were in the form of a helix using No. 4 (half-inch diameter) bars with a six-inch pitch. The shaft geometry was developed with cylindrical solid type objects and the meshing was auto-generated. The number of elements generated was 95,189. The shaft model geometry and the mesh generated are shown in Figure 4.8. The material properties were the same as considered in the 2-D analysis and given in Table 4.1. Similar to the 2-D analysis, simulations were performed with the finite element formulation in COMSOL Multiphysics®software with non-Newtonian behavior for the concrete and Newtonian behavior for slurry. The viscosity of concrete was set according to the non-Newtonian Carreau model.

Computations

Computations were performed in the same approach as in 2-D analysis using the BDF time-dependent solver method. The Level Set method was used to track the interface between concrete and the drilling fluid. The analyses were performed for 80-second flow times; the results were stored at prescribed time intervals, and the volume fraction plots were extracted to study the concrete flow patterns. Parametric studies were carried out for the re-initialization intensity parameter γ, surface tension σ, and interface thickness ε. The flow pattern obtained from the parametric study performed for γ, with values 0.5, 0.1, 0.05, and 0.01, are given in Appendix B. Based on the study, a γ value of 0.05 m/s (9.84 ft/min) was selected. Other values of γ led to generation of unphysical pockets of slurry within the concrete. For σ, flow patterns were studied for the values 0.05, 0.1, and 0.50 N/m, and a value of 0.1 N/m was selected because it showed a realistic flow pattern of the concrete in the excavation compared to other values. For ε, a value of 0.007 m (0.28 inches) was selected after studying the flow patterns for the values of 0.005, 0.007, 0.010, and 0.015 m. The flow patterns obtained from the parametric studies for σ and ε are also given in Appendix B.

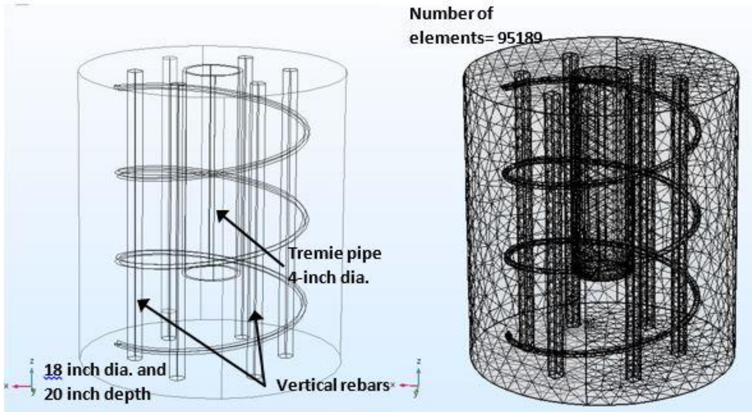


Figure 4.4.8 3-D Model in COMSOL: Shaft Geometry and Meshing

Figure 4.9 shows the concrete flow patterns simulated from the 3-D model, at different time intervals. For the shaft model of 18-inch diameter and 20-inch height, the computational solution time was 13 hr running on a desktop Dell computer. Even though the concrete filling was progressing in the excavation, the realistic concrete flow pattern with concrete head differential inside and outside the rebar cage was not apparent as had been observed in the 2D simulation presented earlier. Figure 4.10 shows the flow pattern at different horizontal planes at each time interval.

When observing the concrete flow at the rebar locations and at the concrete cover region in Figure 4.11, the simulation showed the concrete did not flow around the vertical rebars but flowed across the vertical rebars, which is not realistic and is attributed to poor resolution. To get a simulation with better resolution, it was essential to generate a finer mesh. Considering the large number of components, such as tremie pipe, vertical rebars, and the horizontal ties, involved in representing the geometry, and taking into account the number of elements needed to adequately represent the areas around the rebars and in the shaft cover region, it is estimated that at least four to five million elements would be required to accurately resolve the flow around the rebars. Moreover, these computations necessitate parallel computing using multiple parallel processors in order to be carried out in a reasonable amount of time. When observing the concrete flow at the rebar locations and at the concrete cover region in Figure 4.11, the simulation showed the concrete did not flow around the vertical rebars, rather it flowed across the vertical rebars, which is not realistic and is attributed to poor resolution.

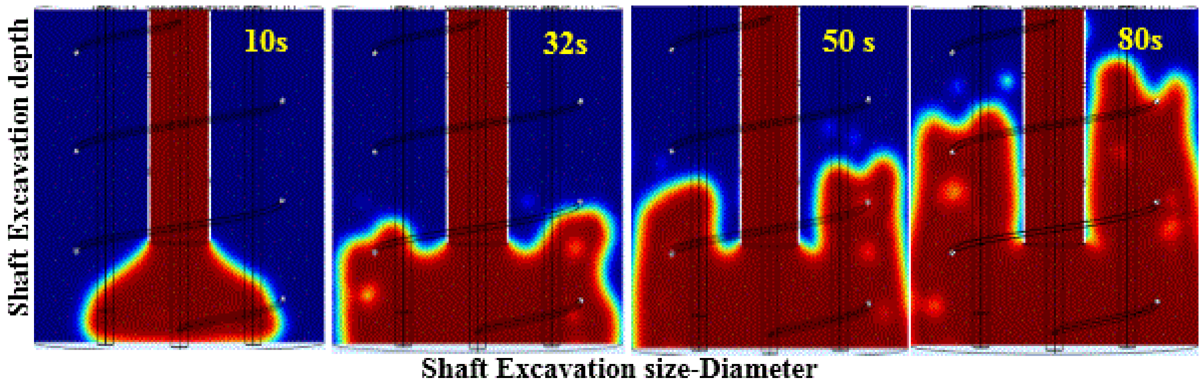


Figure 4.4.9 3-D Model in COMSOL: Flow Pattern on Vertical Planes at Different Time Intervals.

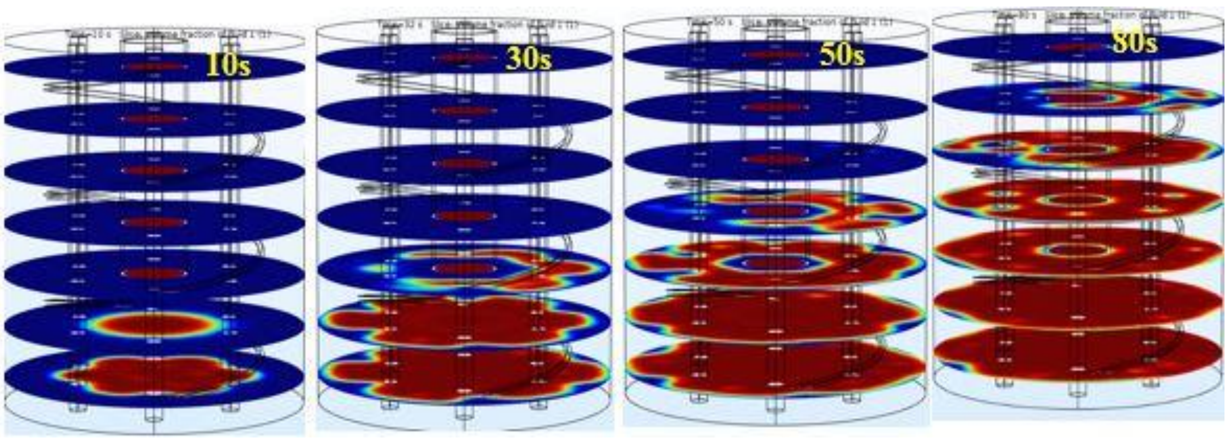


Figure 4.4.10 3-D Model in COMSOL: Flow Pattern on Horizontal Planes at Different Time Intervals.

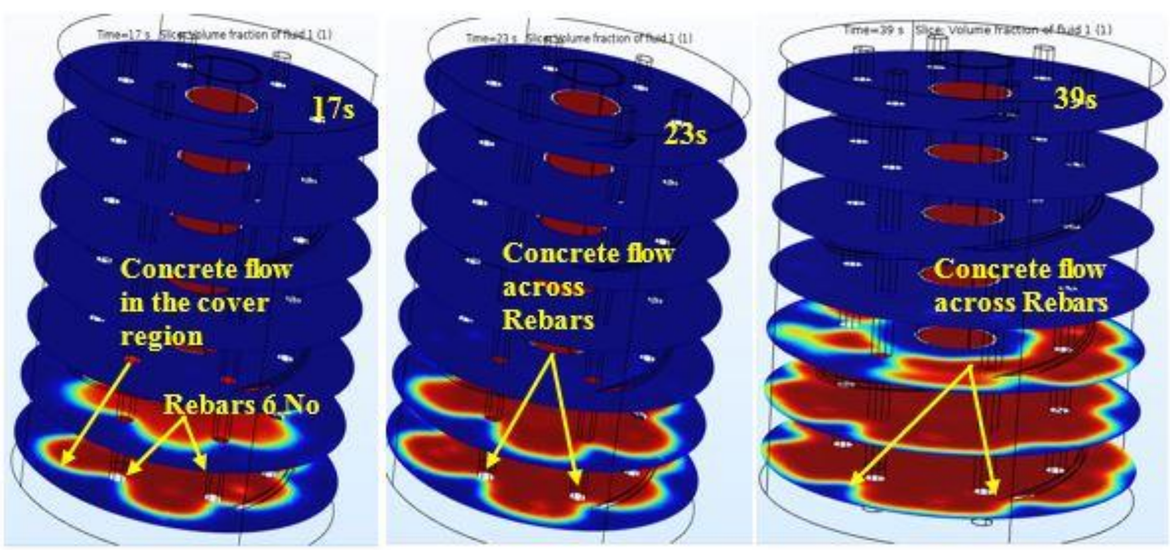


Figure 4.4.11 3-D Model in COMSOL: Concrete Flow Pattern at Rebar Locations.

In order to run the model on parallel computers available through CIRCE (Central Instructional Research Computing Environment) at University of South Florida, the model was transferred to ANSYS Fluent, which is a finite volume CFD code. Fluent implements the incompressible Navier-Stokes equations, while tracking the interface between the concrete and slurry using the Volume of Fluid method, which is based on the marker-and-cell method discussed earlier, making use of a volume fraction function tracked via an advection equation similar to the Level Set function in Eqn. 6.3. The non-Newtonian Carreau model is also available in Fluent. The Fluent user manual should be consulted for more information.

4.4.2 2-D Simulation with Simple Model

Before performing the shaft analysis in ANSYS-Fluent a preliminary analysis was carried out with a simple 2-D model. The objective was to verify the flow pattern and again estimate the mesh size required to be adopted in the 3-D model. A small size 2-D rectangular model, 12-inch width and 18-inch depth with only one rebar with 1.0-inch diameter was considered. The distance between the rebar center and the outlet edge was kept at six inches to simulate the normal dimension used for the shaft cover region. Since the geometry was small and had only one rebar, extremely fine meshing having a mesh size ranging from 0.05 inch near the rebar to 0.11 inch away from the rebar was generated. The concrete flow patterns obtained are shown in Figure 4.12. The concrete flow patterns obtained from the simulation show better flow behavior behind the rebar that is similar to that observed in the laboratory investigation carried out by Mullins (2015), as shown in Figure 4.13. This is in contrast to the behavior of the earlier 3-D simulations in which the concrete did not flow around the rebar, but rather crossed through it. Thus, in order to properly resolve the flow around the rebars, the mesh size (0.05 inch around the rebars) has to be much finer than the size of the rebars (1.0 inch).

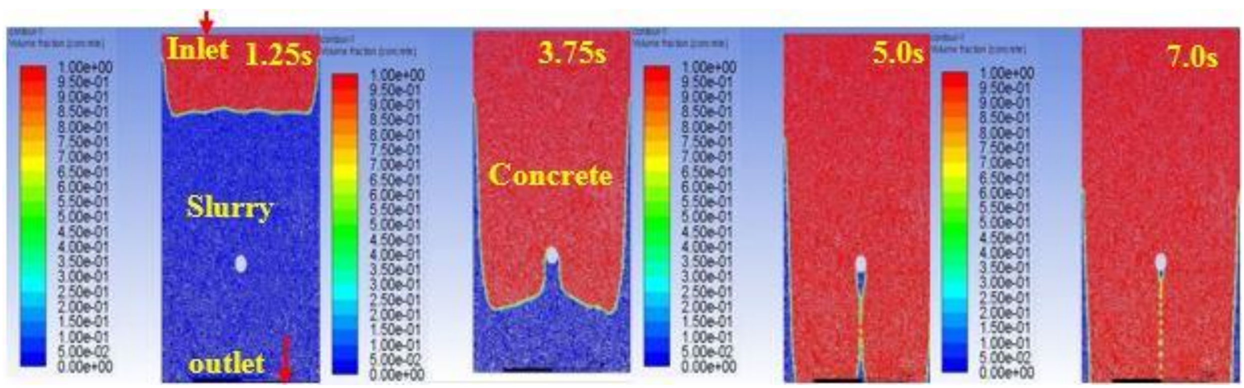


Figure 4.4.12 2-D Model in ANSYS-Fluent: Flow Pattern from a Simple Model Analysis.

Figure 4.4.13 Experimental Study with Cast Shafts at USF, (Mullins, 2015): Flow Pattern Observed.

4.4.3 3-D Simulation in ANSYS-Fluent

Three- and four-foot diameter shafts were considered for the simulations. These size shafts represent the smallest and most common FDOT shaft foundations for various structures. The tremie pipe sizes of ten and twelve inches were considered for the three- and four-foot shafts, respectively. The tremie was placed from the top of the shaft to six inches above the shaft bottom. Reinforcement cages were placed with a six-inch cover. The vertical rebars were one-inch diameter in size and the horizontal ties were 1/2-inch diameter in size. Two types of rebar and tie arrangements were considered to study the pattern of concrete flow under these conditions. The details of the different model geometries considered for the analysis are given in Table 4.2.

Table 4.2 Shaft and tremie sizes and reinforcement cage details

Shaft Size (in.)	Tremie Size (in.)	Vertical Rebar Spacing (in.)	Horizontal Tie Spacing (in.)
48	12	7	6
48	12	3.5	3.5
36	10	6.3	6
36	10	3.8	3.5

Model Geometry

3-D Model geometry was developed in SolidWorks (Planchard, 2015), and shaft sizes of 48-inch and 36-inch diameter were considered by creating a ninety-degree segmented model (Figure 4.14). The centrally placed tremie pipe, the vertical rebars, and the horizontal ties were incorporated in the geometry. The developed model was imported into ANSYS-Workbench to generate the mesh.

The geometries of 48-inch diameter and 36-inch diameter shaft models with the rebar details are shown in Figure 4.14.

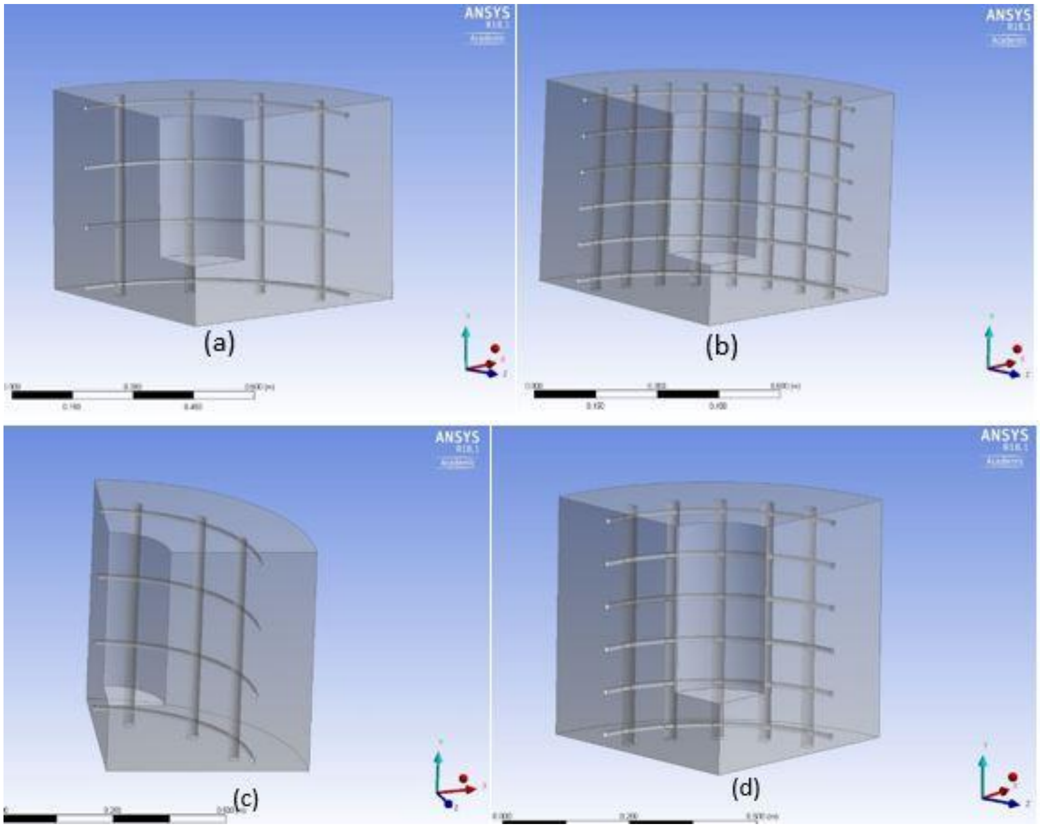


Figure 4.4.14 3-D Model in ANSYS-Fluent: Geometry of 48-inch and 36-inch diameter shafts with 20-inch depth: (a) 48-inch diameter shaft model 4 vertical rebars and 4 ties (b) 48-inch diameter shaft model: 8 vertical rebars and 6 ties (c) 36-inch diameter shaft model: 3 vertical rebars and 4 ties (d) 36-inch diameter shaft model: 5 vertical rebars and 6 ties

Meshing

Based on the mesh sizes adopted in the simple model discussed in the previous section, meshing was generated in the size range 0.033 to 0.13 inch. Since the model geometry consisted of shaft excavation, tremie pipe, vertical rebars, and horizontal ties, a mesh size within the range of 0.033 to 0.13 inch required on the order of 1.1 million nodes and 6.3 million elements (see Table 4.3). With this mesh system, the simulation could capture the flow pattern that exhibited the realistic flow behavior around the rebars as well as in the concrete cover region. Figures 4.15 and 4.16 show the meshes generated for 48-inch and 36-inch diameter shafts, respectively. Table 4.3 gives the details on the mesh sizes, the number of nodes, and the number of elements.

Table 4.3 Details of mesh sizes, number of nodes and number of elements.

No	Shaft Size	Reinforcement Details	Mesh size		Number of Nodes	Number of Elements
			Min. (in.)	Max. (in.)		
1	48-inch dia. 20-inch depth	4 Rebars and 4 Ties	0.033	0.12	1,188,233	6,366,030
2	48-inch dia. 20-inch depth	8 Rebars and 6 Ties	0.035	0.13	1,176,169	6,317,274
3	36-inch dia. 20-inch depth	3 Rebars and 4 Ties	0.03	0.11	99,4014	5,326,261
4	36-inch dia. 20-inch depth	5 Rebars and 6 Ties	0.03	0.11	1,069,758	5,718,755

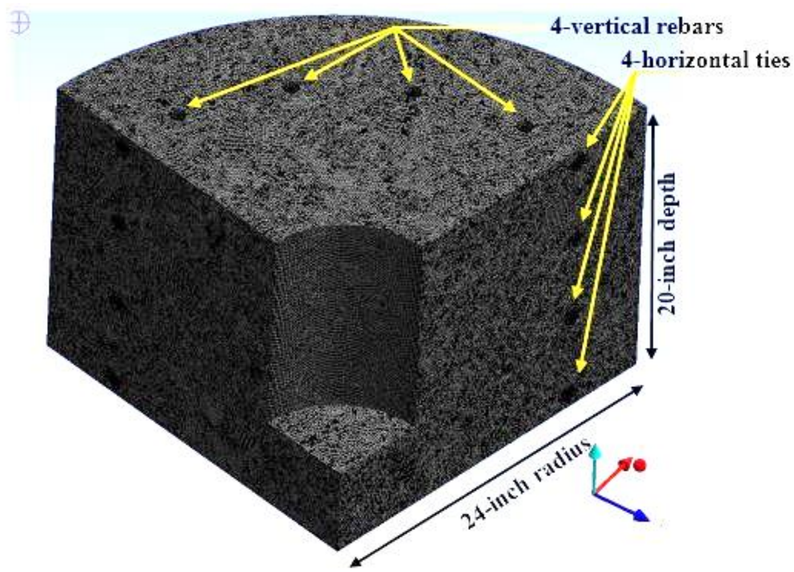


Figure 4.4.15 3-D Model in ANSYS-Fluent: Meshing for 48-inch Dia. Shaft.

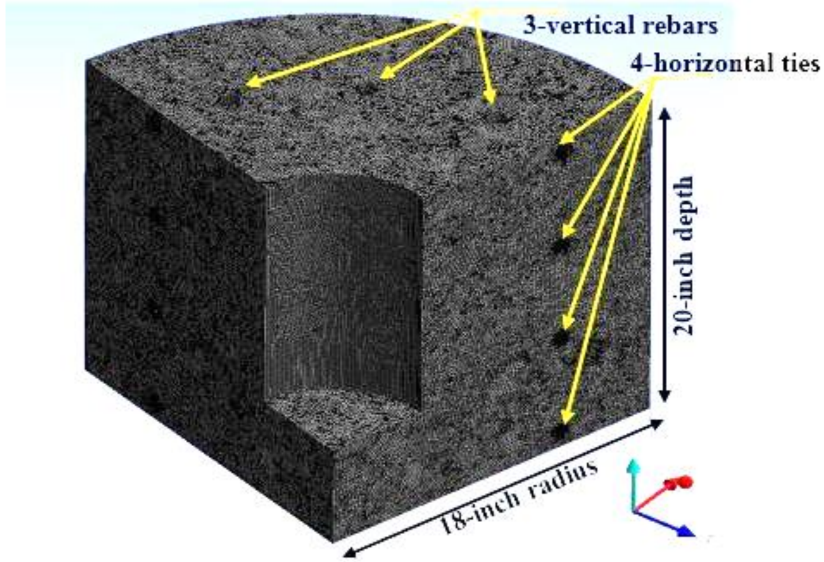


Figure 4.4.16 3-D Model in ANSYS-Fluent: Meshing for 36-inch Dia. Shaft.

Boundary Conditions

As described for the earlier 2-D simulations, boundary conditions were set at the inlet and outlet. The inlet for the concrete flow was at the bottom of the tremie pipe and the outlet was at the top of the excavation. In the earlier COMSOL simulation, the inlet was at the top of the tremie pipe, whereas in the current cases with Fluent, the inlet was set at the bottom of the tremie pipe in order to reduce the size of the domain and thus the number of grid points (see Figure 4.17). Velocity at the inlet and the pressure at the outlet were given as the boundary conditions. The inlet velocity value given was equivalent to the velocity of concrete flow in the tremie pipe, which was again calculated from the concrete delivery rate in the field, from a typical 10 yd^3 capacity truck in 20 minutes. Table 4.4 gives the inlet velocity for both 48-inch and 36-inch shafts. No-slip conditions were considered at the interface with the vertical face of the shaft excavation to simulate the zero velocity for the flow of concrete and the slurry along the excavation face. At the outlet, the flow was out of the excavation and the pressure was equal to the atmospheric pressure. Symmetry boundary conditions were assigned at the azimuthal ends of the 90-degree segmented domains in Fluent.

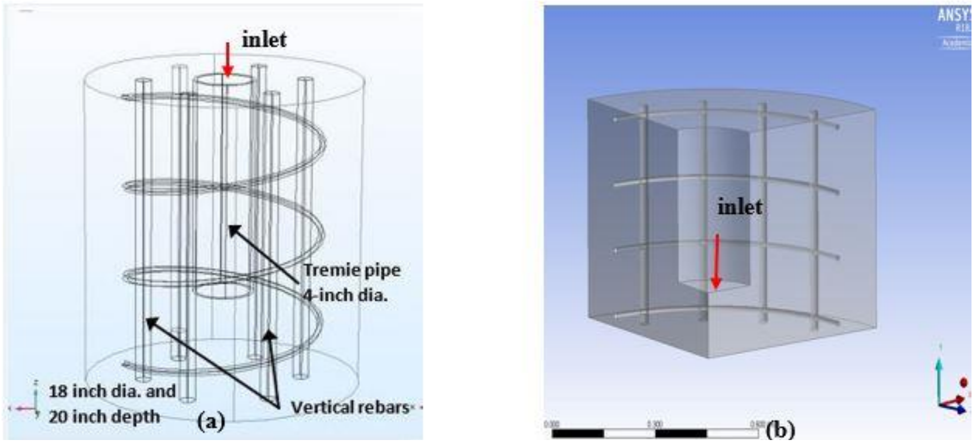


Figure 4.4.17 3-D Model Boundary Conditions at Inlet: (a) CFD Model Built in COMSOL (b) CFD Model in ANSYS-Fluent.

Table 4.4 Inlet velocity of concrete at tremie bottom.

Shaft size (in.)	Tremie size (in.)	Inlet velocity (ft/min)
48	12	17.7
36	10	25.6

Material Properties

In the Navier-Stokes equations, the viscosity of the concrete was given by the non-Newtonian Carreau model. For both SCC and normal concrete, the viscosity values at zero shear rate (μ_0) and at infinite shear rate (μ_∞) are given in Table 4.5. For density, constant values were assigned for SCC and normal concrete. For slurry, Newtonian behavior was followed and constant values of density and viscosity were considered. Analysis was also performed with water as the drilling fluid. The properties of SCC, normal shaft concrete, slurry, and water used in the computations are given in Table 4.5. A high viscosity value of 2500 Pa-s was considered for μ_0 for concrete so that, at very low shear rates, the material was nearly rigid. Very high values of initial viscosity in the range between 30,000 Pa-s to 160,000 Pa-s were chosen to input into the bi-viscosity model for a numerical simulation of concrete casting (also used by Vasilic et al. (2016)).

Table 4.5 Properties of SCC, concrete, slurry, and water used in the computations .

	Density Kg/m^3 (lb/ft^3)	Viscosity (μ) Pa-s	
		μ_0	μ_∞
SCC	2300 (143.58)	250	25
NC	2400 (149.83)	2500	100
Slurry	1150 (71.79)	0.5	
Water	1,000 (62.43)	0.01	

Computations

The numerical model prepared with the shaft geometry and the mesh generated with the ANSYS-Workbench was used to perform the simulations in ANSYS-Fluent. For the 48-inch and 36-inch diameter shafts, two types of reinforcement arrangements were considered to study the effect of rebar arrangement on the flow. Simulations were performed for both SCC and concrete, using slurry as the drilling fluid, to study and evaluate the flow performance for both the materials in the shaft excavation. Additional simulations were performed with water as a drilling fluid. The summary of the cases simulated and studied are given in Table 4.6 and Table 4.7 for 48-inch and 36-inch diameter shafts, respectively. To perform the computations, the number of time steps selected achieved the filling of the majority of the shaft to a depth of 20 inches.

Table 4.6 48-inch diameter and 20-inch depth segmental shaft: summary of simulations.

Case No	Material	Drilling Fluid	Types of Reinforcement	Model No
1	SCC	Slurry	4 Rebars and 4 Ties	5101
2	SCC	Slurry	8 Rebars and 6 Ties	5102
3	SCC	Water	4 Rebars and 4 Ties	5103
4	SCC	Water	8 Rebars and 6 Ties	5104
5	NC	Slurry	4 Rebars and 4 Ties	5105
6	NC	Slurry	8 Rebars and 6 Ties	5106
7	NC	Water	4 Rebars and 4 Ties	5107
8	NC	Water	8 Rebars and 6 Ties	5108

Table 4.7 36-inch diameter and 20-inch depth segmental shaft: summary of simulations

Case No	Material	Drilling Fluid	Types of Reinforcement	Model No
1	SCC	Slurry	3 Rebars and 4 Ties	6001
2	SCC	Slurry	5 Rebars and 6 Ties	6002
3	SCC	Water	3 Rebars and 4 Ties	6003
4	SCC	Water	5 Rebars and 6 Ties	6004
5	NC	Slurry	3 Rebars and 4 Ties	6005
6	NC	Slurry	5 Rebars and 6 Ties	6006
7	NC	Water	3 Rebars and 4 Ties	6007
8	NC	Water	5 Rebars and 6 Ties	6008

The time step size was chosen to ensure that the Courant-Friedrichs-Lewy (CFL) condition was satisfied. This is the condition for the relation to be maintained between the time step size and the mesh element size. The CFL condition is important to achieve the flow stability and for the solver to get solution for each element. The number of iterations was about 250 to 500 in order to achieve convergence for a single time step and still reduce residual levels to those recommended in the

Fluent user manual. The analysis was performed using the high-performance computing resources through the CIRCE cluster computer.

4.5 Results and Discussion

4.5.1 Concrete Flow Pattern from the Simulation

Simulations were performed for eight cases each for 48-inch and 36-inch diameter shafts and are summarized in Table 4.6 and Table 4.7, respectively. The results were processed using ANSYS-CFD Post, which is the post processing tool within ANSYS, and the concrete flow patterns were obtained on horizontal planes at different depths and on vertical sliced planes. A typical flow pattern obtained on horizontal planes at different depths in the shaft excavation is shown in Figure 4.18 (indicated by volume fraction coloration). This flow pattern was obtained from the simulation of NC flow when casting under slurry and at flow time of 35 s. In the flow pattern, the creases that developed behind the rebars in the concrete cover region can be seen through the interface between the concrete and the slurry. A similar flow behavior was observed during the study carried out at University of South Florida (Mullins, 2015) with 24 cast shafts, 42-inch diameter and 2-foot height (Figure 4.19).

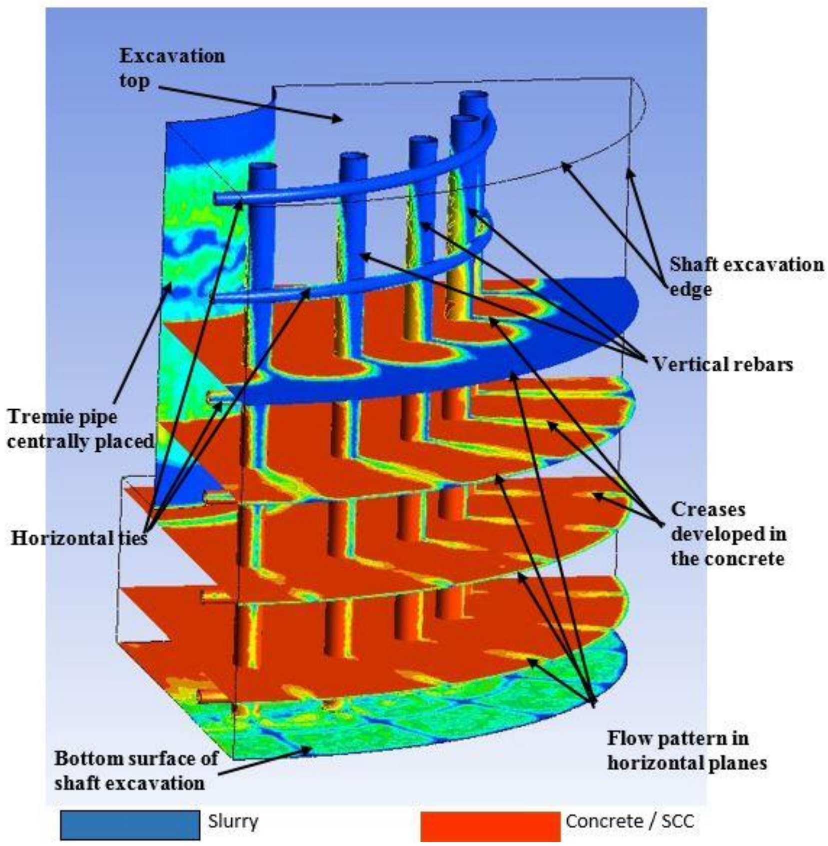


Figure 4.4.18 3-D Model in ANSYS-Fluent: Typical Flow Pattern on Horizontal Planes.

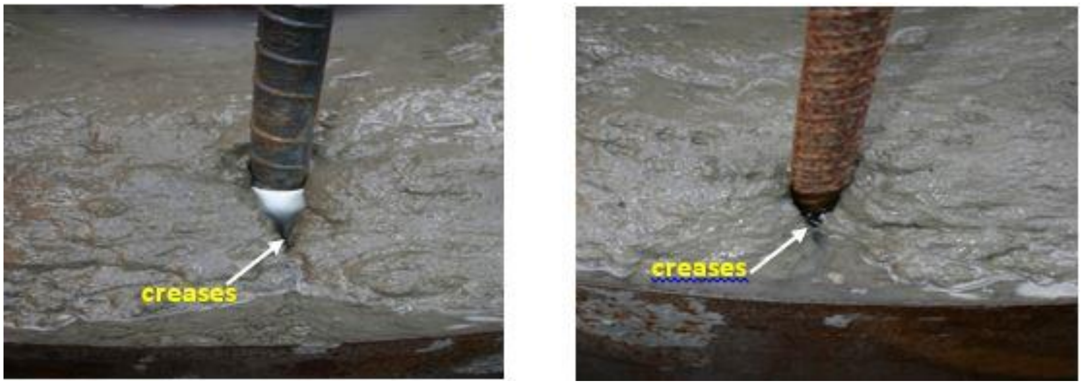


Figure 4.4.19 Experimental Study with Cast Shafts at USF: Creases Behind Rebars, (Mullins, 2015).

In addition, the volume fraction patterns from the simulations were extracted from concentric vertical slices near the reinforcing cage and nearest the shaft/soil sidewall interface (Figure 4.20). Both vertical and horizontal creases are clearly seen nearest the cage; close to the shaft sides the horizontal creases are very faint. Similar patterns of vertical and the horizontal creases were observed during the experimental study with cast shafts and are shown in Figure 4.21.

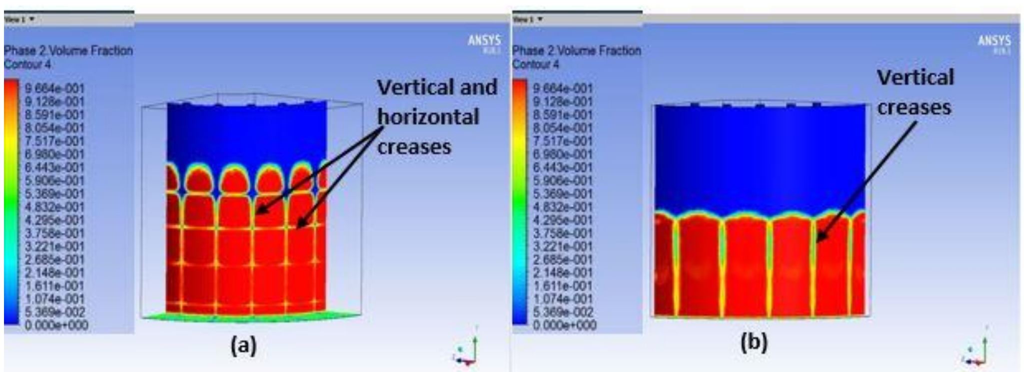


Figure 4.4.20 3-D Model in ANSYS-Fluent: Typical Volume Fraction Patterns - (a) Slice 1 in. outside the cage (b) Slice 1 in. from the Excavation Edge.

Figure 4.4.21 Experimental Study with Cast Shafts at USF: Vertical and Horizontal Creases Observed at the Edge of Cast Shaft, (Mullins, 2015).

Moreover, the flow pattern extracted from a vertical plane shows the concrete head differential between inside and outside the rebar cage in Figure 4.22. This similar pattern of concrete flow was obtained, as shown in Figure 4.23, from the research study carried out at USF with the Lateral Pressure Cell (LCP) developed by Mullins and Ashmawy, (2005). The objective of Mullins and Ashmawy, (2005) was to study the rheology of concrete as it flows from the tremie bottom and rises inside the cage within an excavation.

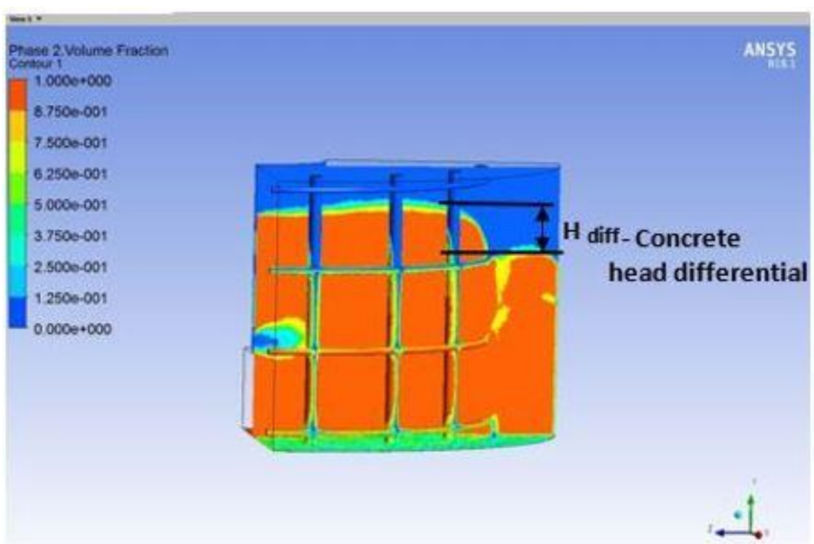


Figure 4.4.22 3-D Model in ANSYS-Fluent: Typical Flow Pattern on Vertical Plane.

Figure 4.4.23 Laboratory LCP Test at USF: Flow Pattern of Rising Mortar with Head Differential, (Mullins and Ashmawy, 2005).

Head differentials of rising mortar between inside and outside the reinforcement cage were measured by the LCP tests. Figure 4.23 shows the pattern of rising mortar observed in the LCP test. Field testing programs were also carried out and the concrete head differential was measured at Port of Tampa, Essex Cement Company Project, Crosstown Expressway Reversible Lanes Bridge site, and at Alagon Condominium project.

From the above comparison made between the concrete flow patterns obtained from the simulation and from the experimental study, it can be concluded that the 3-D model developed provides qualitatively reliable results. It is not possible to make a stronger comparison between the model and experiments because the flow time was not measured in the experiments. Furthermore, the viscosity of concrete given by the non-Newtonian Carreau model is for a homogeneous fluid, thus it does not consider the aggregates present in concrete.

4.5.2 Evaluation of Flow Performance of SCC and NC

Evaluation Based on Creases in the Concrete from Simulation

The flow patterns obtained from the simulations performed with SCC and NC for 48-inch diameter and 20-inch depth shaft models are shown in Figures 4.24 and 4.25, respectively. The flow patterns are again visualized on horizontal and vertical planes. The flow patterns on the horizontal planes are shown in Figures 4.24(a) and 4.25(a), and on the vertical planes in Figures 4.24(b) and 4.25(b).

In the flow pattern of NC extracted on the horizontal planes, creases formed behind the vertical rebars in the concrete cover regions were observed. Similar flow patterns were seen on the horizontal planes. Meanwhile, in the case of SCC, these creases were not observed.

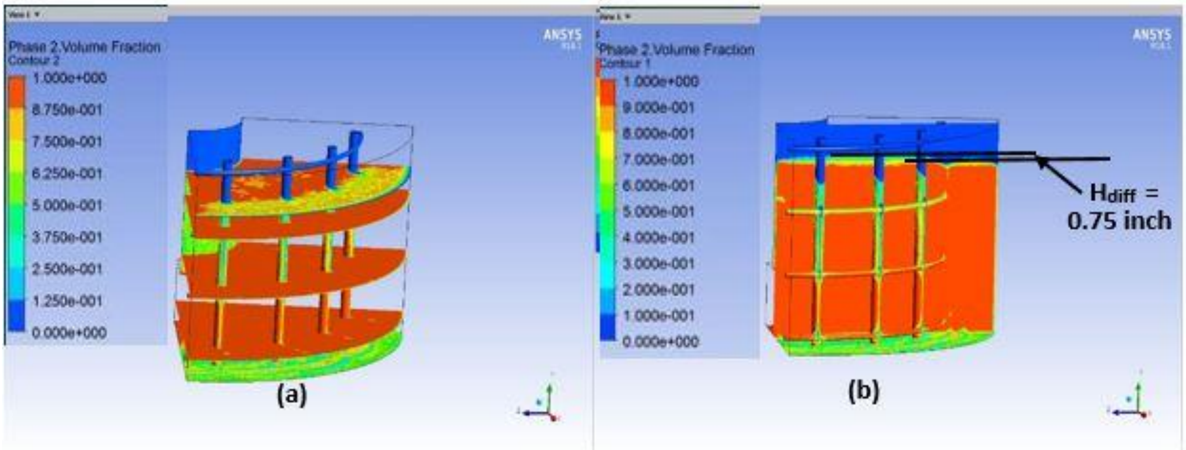


Figure 4.4.24 3-D Model in ANSYS-Fluent: Evaluation of Flow Performance of SCC-Shaft Size 48-inch Diameter and 20-inch Depth, t =100 s, (a) on Horizontal Planes (b) on Vertical Plane.

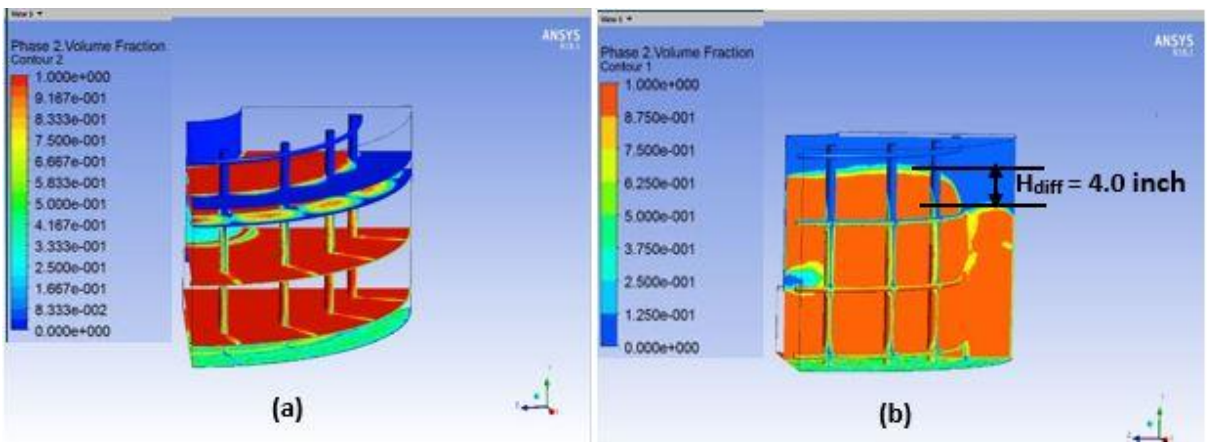


Figure 4.4.25 3-D Model in ANSYS-Fluent: Evaluation of Flow Performance of NC-Shaft Size 48-inch Diameter and 20-inch Depth, t = 83 s, on Horizontal Planes (b) on Vertical Plane.

Evaluation Based on Concrete Head Differential from Simulation

The concrete head differential between inside and outside the rebar cage was obtained from the simulation of both SCC and NC. For the evaluation of flow performance, the flow pattern of SCC and NC were compared by keeping the other parameters like shaft size, tremie pipe size, number and size of vertical rebars, and horizontal tie arrangements the same. The shaft details are 48-inch diameter with 12-inch diameter tremie pipe, four one-inch diameter vertical rebars and four 1/2-inch diameter horizontal ties in the 90-degree segmented shaft. For both the shaft models with SCC and NC, slurry was considered for the drilling fluid. The flow patterns obtained from the simulations extracted on the vertical plane are shown in Figure 4.24 (b) and Figure 4.25 (b) for SCC and NC, respectively. It can be seen that in the case of NC the head differential was about four inches, whereas for SCC, the head differential was negligible and less than an inch.

A similar evaluation was performed from the simulation obtained for 36-inch diameter shaft with SCC and NC. The flow pattern of SCC and NC extracted on the horizontal planes and vertical plane are shown in Figures 4.26 and 4.27.

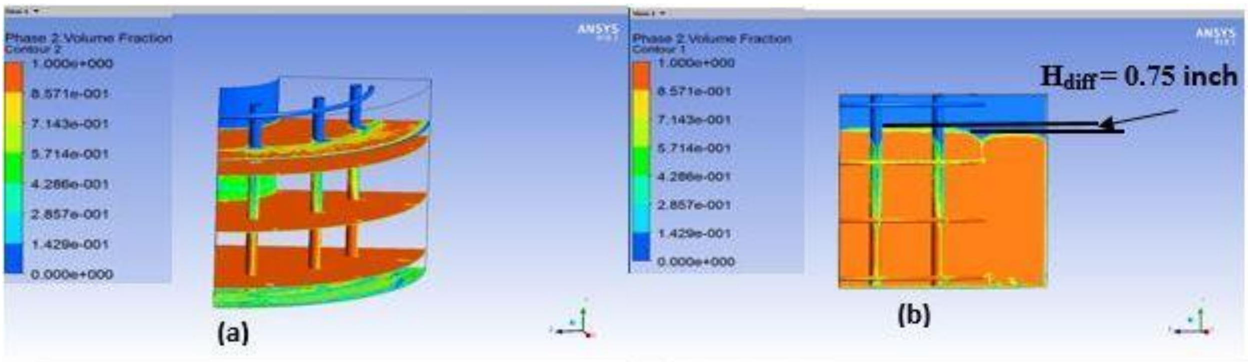


Figure 4.4.26 3-D Model in ANSYS-Fluent: Evaluation of Flow Performance of SCC-Shaft Size 36-inch Diameter and 20-inch Depth, t = 50 s, (a) on Horizontal Planes (b) on Vertical Plane.

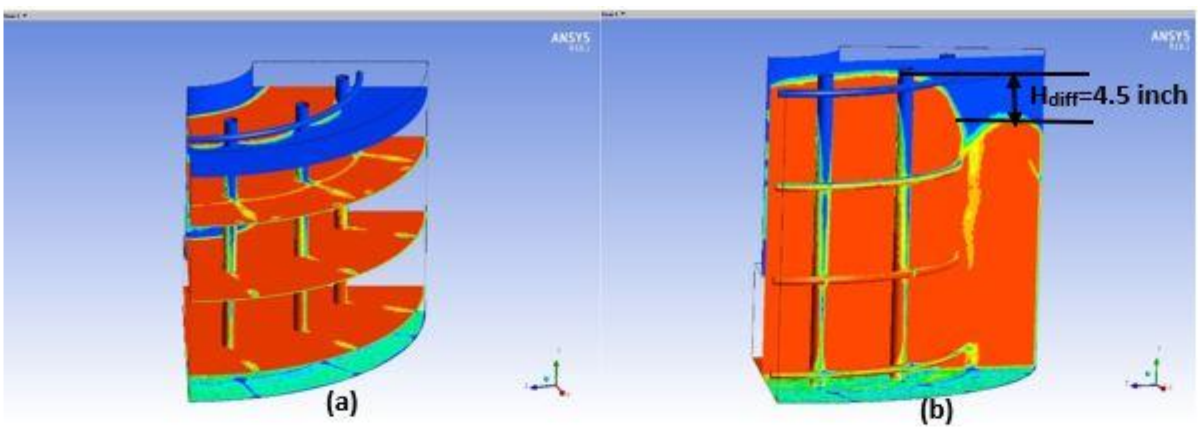


Figure 4.27 3-D Model in ANSYS-Fluent: Evaluation of Flow Performance of NC-Shaft Size 36-inch Diameter and 20-inch Depth, t = 50 s, (a) on Horizontal Planes (b) on Vertical Plane.

As observed in the simulation of 48-inch diameter shaft model, the flow pattern of SCC shows no creases and the concrete head differential is less than an inch in height. But the flow pattern of NC extracted on the horizontal plane shows creases in the concrete cover region and the flow pattern on the vertical plane exhibits a concrete head differential of about 4.5 inches.

Hence, from the simulations performed on both 48-inch diameter and on 36-inch diameter shaft models, it can be seen that, the flow behavior of SCC was free of creases, or if creases formed, they were negligible and significantly less concrete head differential formed. In contrast, the flow behavior of NC showed clear patterns of creases in the concrete cover region and with concrete head differentials on the order of four inches at the times the instantaneous solutions were plotted.

4.5.3 Effect of Shaft Sizes on the Flow Performance

The effect of shaft size on the flow performance was studied by comparing the results of the simulation with 48-inch and 36-inch diameter shaft models. The shaft depths were kept same at 20 inches for both the shafts. The head differential from the simulation, obtained for the shaft with SCC, was negligible, whereas for the shaft with NC, the head differential obtained was about four inches. Hence, to determine the effect of shaft sizes, only the simulations with NC were considered. The flow patterns extracted on horizontal planes and a vertical plane for both models are given in Figure 4.28 and in Figure 4.29. Creases are observed in the flow patterns of both the models, and no considerable differences in the crease patterns between the two models can be seen. However, subtle differences are observed between the concrete head differential encountered in both models. The concrete flow velocities in the 48-inch and 36-inch models were 1.15 ft/min and 2.07 ft/min, respectively. The concrete head differentials obtained were four inches and four and one-half inches for the 48-inch and 36-inch shaft models, respectively, at t = 83 s and 50 s. This is in keeping with velocity-dependent head losses in fluid mechanics and is therefore deemed to be inconsequential when considering shaft size. Deese (2005) concluded the head differential was most affected by cage spacing, aggregate size, and concrete velocity.

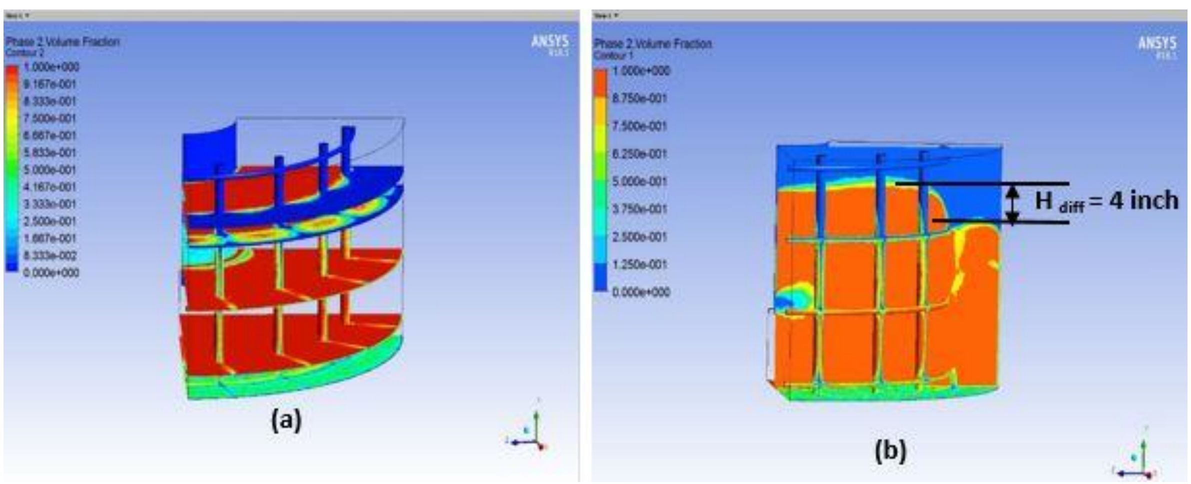


Figure 4.4.28 3-D Model in ANSYS-Fluent: Effect of Shaft Size on Flow Performance of NC-Shaft Size 48-inch Diameter and 20-inch Depth, t = 83 s, (a) on Horizontal Planes (b) on Vertical plane.

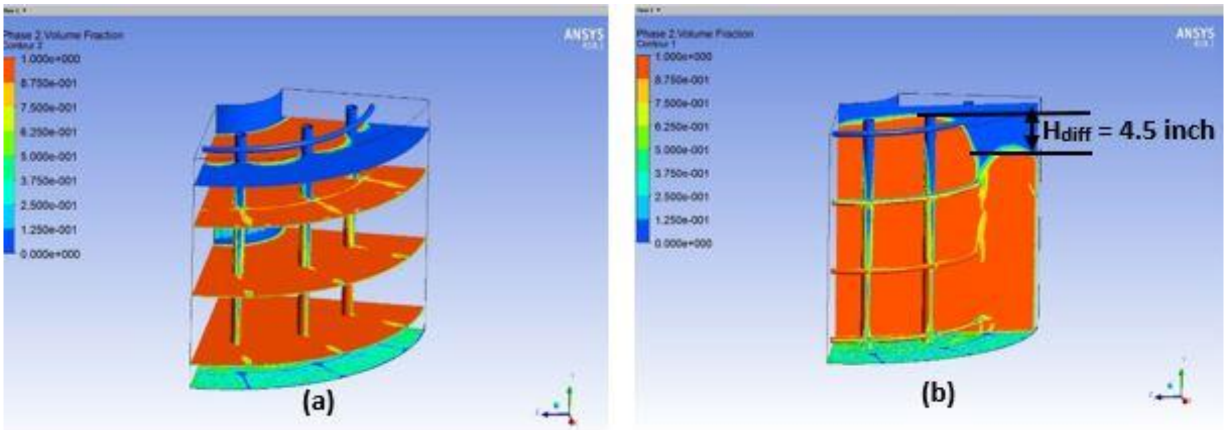


Figure 4.4.29 3-D Model in ANSYS-Fluent: Effect of Shaft Sizes on Flow Performance of NC-Shaft Size 36-inch Diameter and 20-inch Depth, t = 50 s, (a) on Horizontal Planes (b) on Vertical Plane.

4.5.4 **Effect of Rebar Arrangement on the Flow Performance**

The effect of rebar arrangement or the spacing of vertical rebars on the flow performance was also studied. The flow patterns extracted on horizontal planes and vertical plane for both models of 48-inch diameter shafts with four rebars at seven-inch spacing and with eight rebars at 3.5-inch spacing, (model No 5105 and model No 5106 as referred in Table 4.6) are given in Figure 4.30 and in Figure 4.31, respectively. Creases are observed in the flow patterns obtained from both of the reinforcement arrangements and there was some increase in the intensity of crease pattern obtained in the flow pattern of model 5106, shown in Figure 4.31, where the reinforcements were kept at a closer spacing of 3.5-inch. Moreover, the concrete head differentials obtained from the simulation performed in these two models were 4-inch and 10.75-inch, respectively. This shows that the head differential increases as the reinforcement spacing decreases, which is a realistic flow behavior. The concrete head differentials from these models are shown in Figure 4.30(b) and 4.31(b).

The effect of the rebar arrangement on the flow performance was observed from the flow patterns viewed on the surface elevation planes. The planes were extracted in the concrete cover region close to the reinforcement as well as close to the shaft edge. These flow patterns are shown in Figure 4.32 and Figure 4.33. Creases are observed in the flow patterns obtained from both of the reinforcement arrangements. However, considerable increase in the intensity of the crease pattern is observed when the reinforcements were kept at a closer spacing of 3.5-inch.

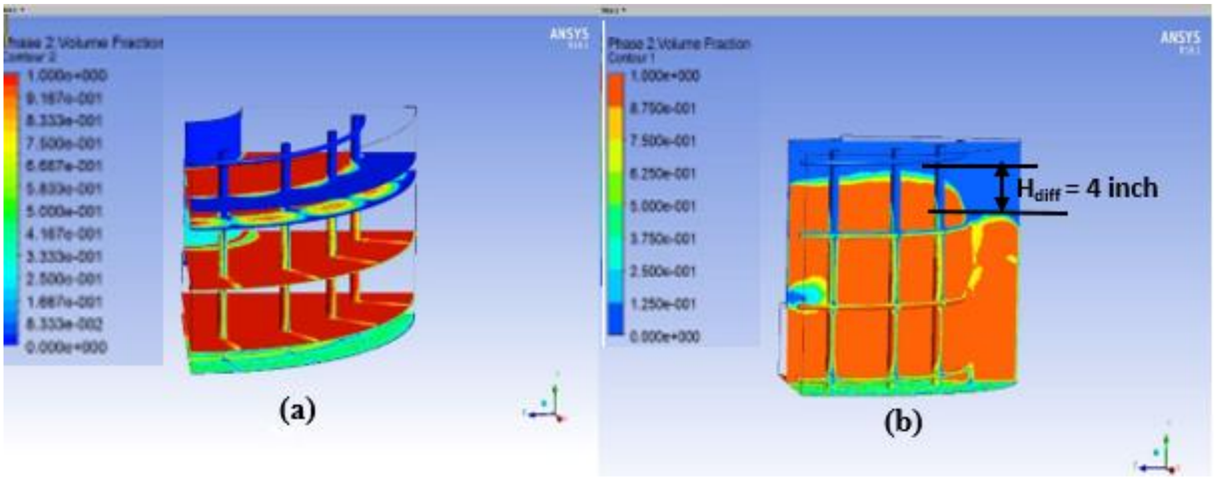


Figure 4.4.30 3-D Model in ANSYS-Fluent: Effect of Rebar Arrangement on Flow Performance of NC- Shaft Size 48-inch Diameter with 4 Rebars and 4 Ties, 7-inch spacing, t = 83 s, (a) on Horizontal Planes (b) on Vertical Plane.

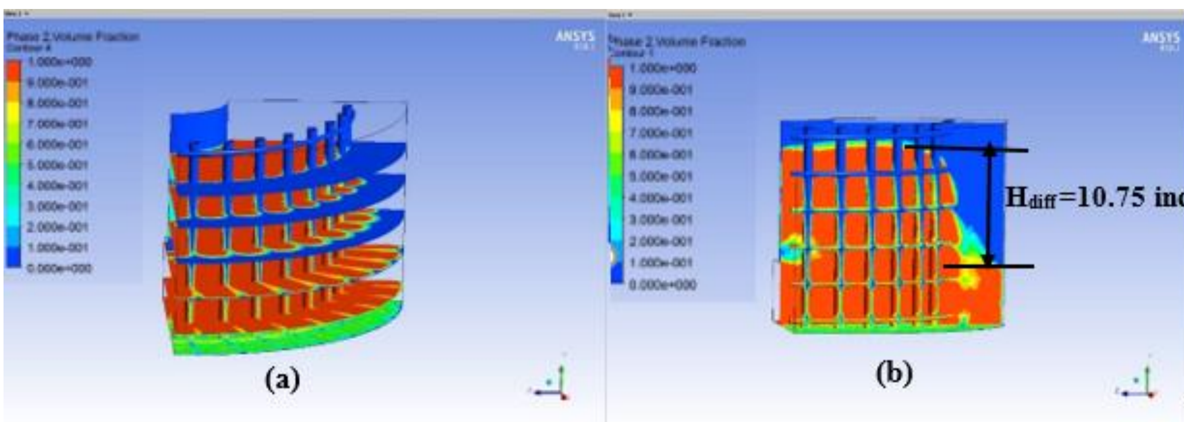


Figure 4.4.31 3-D Model in ANSYS-Fluent: Effect of Rebar Arrangement on Flow Performance of NC- Shaft Size 48-inch Diameter with 8 Rebars and 6 Ties, 3.5-inch spacing, t = 70 s, (a) on Horizontal Planes (b) on Vertical Plane.

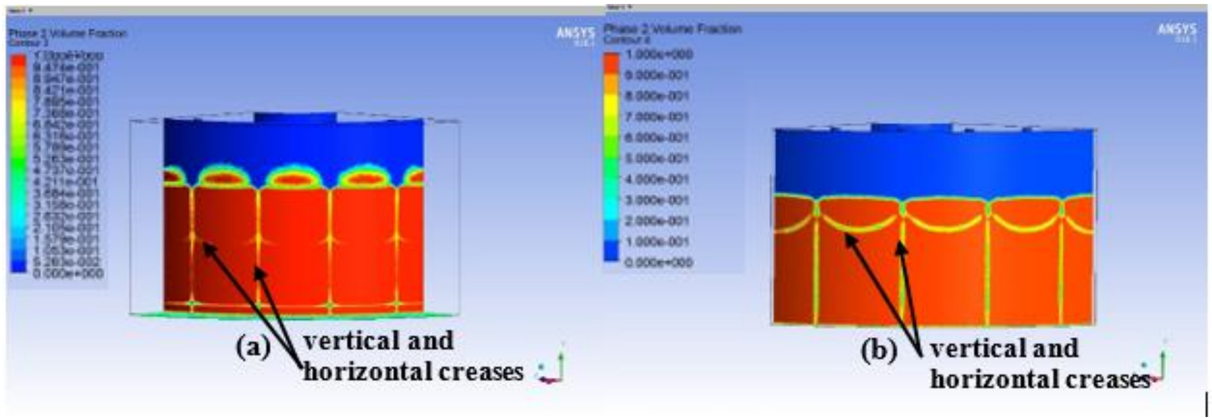


Figure 4.4.32 3-D Model in ANSYS-Fluent: Effect of Rebar Arrangement on Flow Performance of NC- Shaft Size 48-inch Diameter with 4 Rebars and 4 Ties, 7-inch spacing, t = 83 s, (a) on Surface Elevation at Concrete Cover Region (b) on Surface Elevation Close to the Shaft Edge.

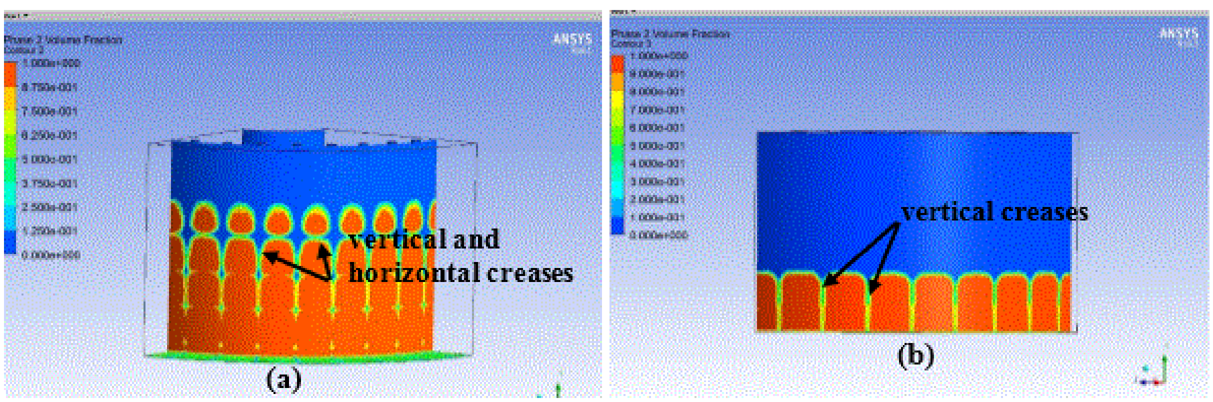


Figure 4.4.33 3-D Model in ANSYS-Fluent: Effect of Rebar Arrangement on Flow Performance of NC- Shaft Size 48-inch Diameter with 8 Rebars and 6 Ties, 3.5 inch spacing, t = 70 s, (a) on Surface Elevation at Concrete Cover Region (b) on Surface Elevation Close to the Shaft Edge.

4.5.5 Concrete Head Differential and the Concrete Flow Velocity

The concrete head differential encountered in the concrete flow for both SCC and NC were compared with the flow velocity in the shaft excavation. The plots of head differential vs. the flow velocity, for 48-inch diameter and 36-inch diameter shafts, are shown in Figure 4.34. The concrete flow velocities in the 48-inch diameter shaft with 12-inch diameter tremie pipe and in the 36-inch diameter shaft with 10-inch diameter tremie pipe were 1.15 ft/min and 2.07 ft/min, respectively. In these plots, the vertical rebar spacing is 7 inches in the case of the 48-inch diameter shaft and 6.3 inches in the case of the 36-inch diameter shaft. Even though only two results are available for this plot, the results show that as the flow rate increases, and thus the concrete flow velocity increases, there is an increase in the concrete head differential. This trend is again similar to the trend obtained in the experimental tests with the cast shafts of Mullins and Ashmawy (2005) shown in Figure 4.36. Similar plots of head differential vs. flow velocity, made for 48-inch diameter and 36-inch diameter shafts with rebars placed at 3.5-inch and at 3.8-inch spacings, respectively, were obtained for both SCC and NC as shown in Figure 4.35. The differential head values obtained from the models are given in Table 4.8.

The plots of the head differential vs. concrete flow velocity obtained for SCC and NC from the simulations are compared with the actual field values of head differential vs. concrete flow velocity in Figure 4.36 (previous shown in Figure 4.7). Note that the field data is for different CSD (cage spacing-to-maximum-aggregate-diameter ratio) values and the results from the simulations are for different viscosities and rebar spacing, thereby making a direct comparison difficult. Also note that the simulations do not take into account the effect of the aggregates. In Figure 4.36, it can be seen that for the field data, the steeper slope plots correspond to lower CSD values, which indicates closer rebar spacing and thus lower flowability. Meanwhile, in the case of the simulations, lower flowability is achieved by NC relative to SCC, as can be seen by the steeper slopes in the plots of the former (Figures 4.34-4.36).

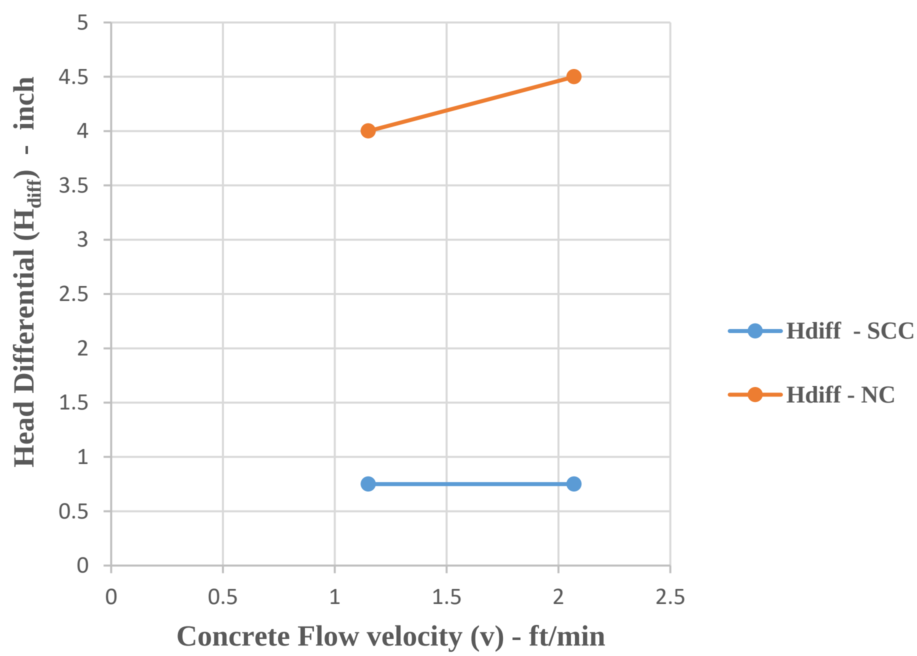


Figure 4.4.34 3-D Model in ANSYS-Fluent: Concrete Head Differential (H_{diff}) vs. Flow Velocity (v) - 48-inch Diameter Shaft with 4 Rebar and 36-inch Diameter Shaft with 3 Rebar.

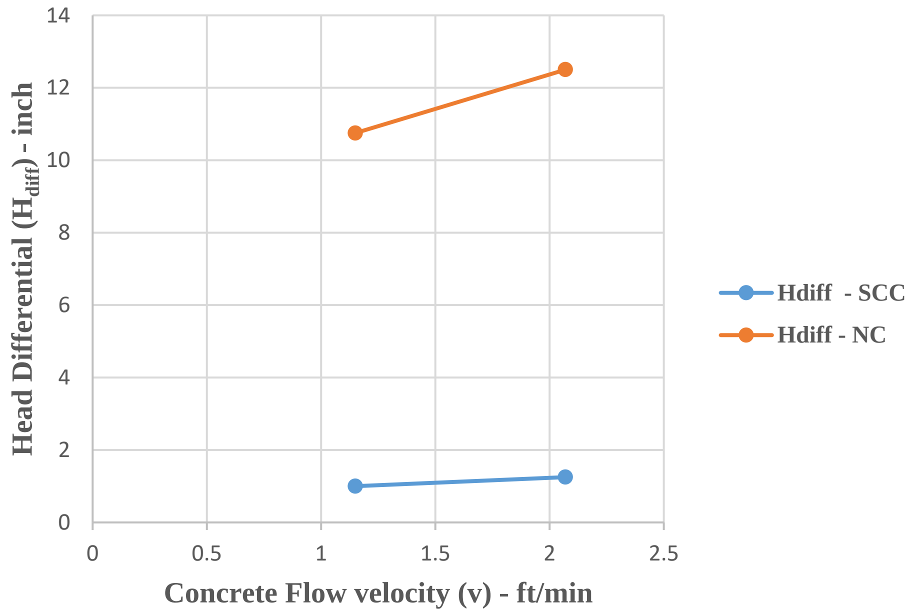


Figure 4.4.35 3-D Model in ANSYS-Fluent: Concrete Head Differential (H_{diff}) vs. Flow Velocity (v) - 48-inch Diameter Shaft with 8 Rebar and 36-inch Diameter Shaft with 5 Rebar.

Table 4.8 3-D Model in ANSYS-fluent: head differential (hdiff) obtained from simulations

Model No	**Shaft Diameter in.**	**SCC/NC - Slurry**	**Concrete Flow Velocity ft/min**	**H_{diff} in.**
5101	48	SCC-Slurry	1.15	0.75
5102	48	SCC-Slurry	1.15	1.00
5105	48	NC-Slurry	1.15	4.00
5106	48	NC-Slurry	1.15	10.75
6001	36	SCC-Slurry	2.07	0.75
6002	36	SCC-Slurry	2.07	1.25
6005	36	NC-Slurry	2.07	4.50
6006	36	NC-Slurry	2.07	12.50

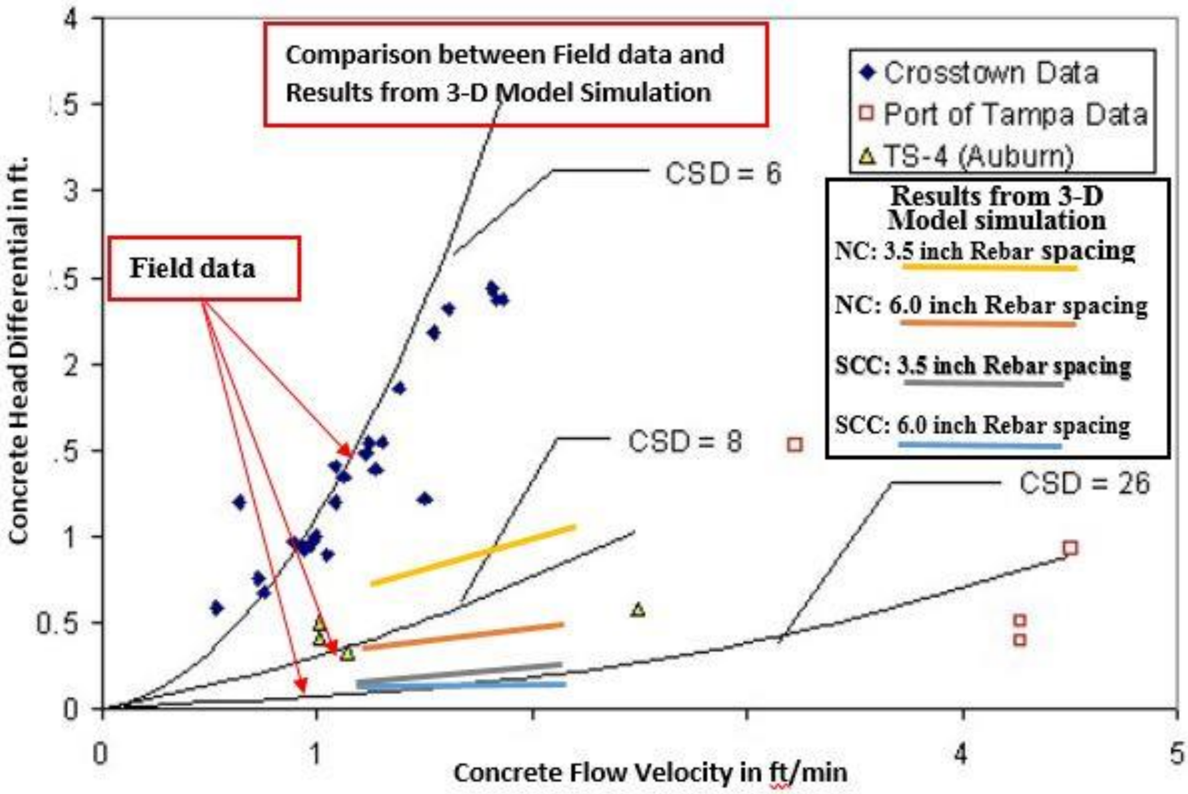


Figure 4.4.36 Concrete Head Differential vs. Flow Velocity: Comparison between Field Data, and Results from Simulation. Note: Field Data is from Mullins and Ashmawy (2005).

4.5.6 Flow Performance of Shafts Cast under Water as Drilling Fluid

Simulations were performed for the SCC shaft cast with water as a drilling fluid. The flow patterns were obtained on horizontal planes and on a vertical plane for a 36-inch diameter shaft model with three vertical rebars, and for this model with five vertical rebars, and are shown in Figures 4.37 and 4.38, respectively. The flow patterns obtained from the simulations show that the creases in the concrete cover region were insignificant and the concrete head differentials encountered were only 0.5 and 1.25 inch for the shaft models with three and five vertical rebars, respectively. These simulations indicated that the flow performance of SCC in the shaft excavation with water as the drilling fluid was similar to the performance of SCC with slurry as the drilling fluid. Similar flow performances of SCC were observed in the experimental shafts, cast with water as the drilling fluid, by Mullins (2015). The cast shafts in these experiments exhibited no visible creases as shown in Figure 4.39.

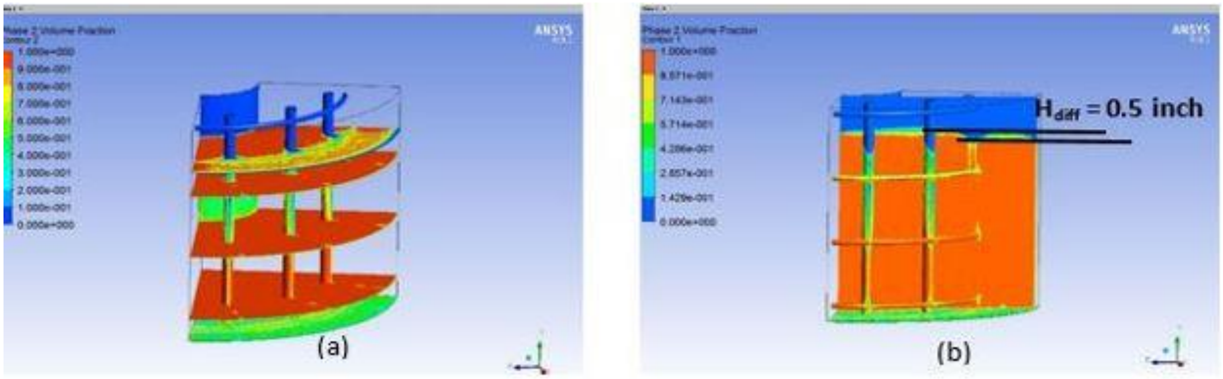


Figure 4.4.37 3-D Model in ANSYS-Fluent: Flow Performance of SCC Shaft Cast under Water- Shaft Size 36-inch Diameter with 3 Rebars and 4 Ties, t = 40 s, (a) on Horizontal Planes (b) on Vertical Plane.

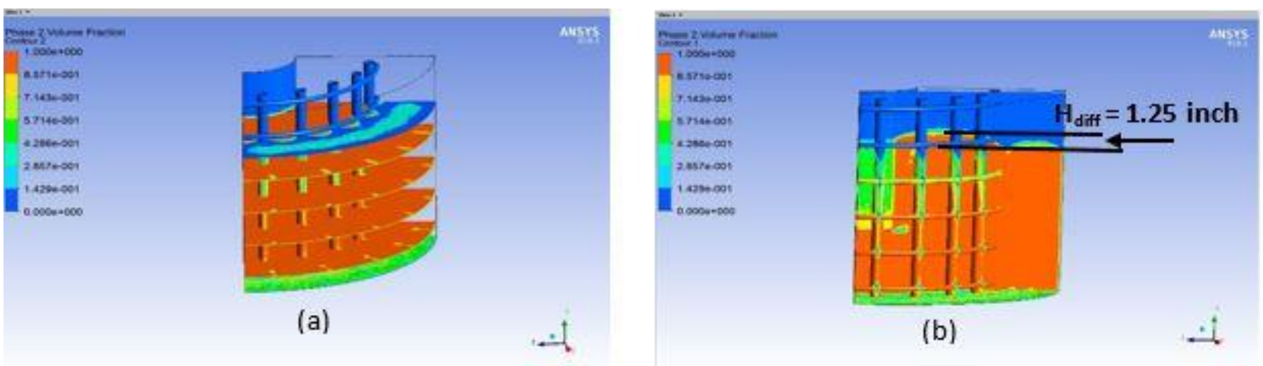


Figure 4.4.38 3-D Model in ANSYS-Fluent: Flow Performance of SCC Shaft Cast under Water- Shaft Size 36-inch Diameter with 5 Rebars and 6 Ties, t = 35 s, (a) on Horizontal Planes (b) on Vertical Plane.

Figure 4.4.39 Experimental Study with Cast Shafts at USF, (Mullins, 2015): Flow Performance of SCC Shaft Cast under Water- Shaft Size 48-inch Diameter.

4.6 **Chapter Summary**

This section discussed the flow patterns simulated from the 3-D model developed. In addition, it covered the qualitative validation of the 3-D model and the simulation. The validation was carried out by comparing the results from the model simulation with the experimental test data. The pattern of vertical and horizontal creases obtained from the simulations in the shaft concrete cover region were qualitatively compared with the creases developed in the experimental cast shafts. In addition, the concrete head differentials between inside and outside the reinforcement cage, observed in the flow pattern from the simulation were comparable to the results obtained at USF in laboratory experiments with LCP test apparatus (Mullins and Ashmawy, 2005).

Using this model as a tool, the performance of SCC for drilled shaft application was evaluated. It was observed that the flow pattern of SCC in a drilled shaft excavation produced negligible creases and the concrete head differentials between inside and outside the rebar cage were minimal. In the case of NC, there were creases of considerable depth in the shaft concrete cover region and the concrete head differential was four inches to twelve inches, depending on the shaft size and the reinforcement cage arrangement. The slope of the head differential vs.. velocity curves increased with decrease in CSD.

The normal concrete flow pattern from the simulation showed both horizontal and vertical creases developing from the splitting of the concrete flow as it passed around the rebars in the concrete cover region. In addition, the concrete head differentials encountered between the inside and outside of the reinforcement cage were observed from the simulations and were shown to be similar to those observed from earlier physical experiments carried out on the cast shafts at USF (Mullins, 2014). Various simulations were performed to study the effect of shaft sizes and the reinforcement arrangements on the flow performance.

Simulations of SCC and NC flows in drilled shaft concreting were studied in terms of creases and concrete head differential. In the flow pattern of SCC, very few creases were observed, compared to NC. In addition, in the flow pattern of SCC, the concrete head differential encountered was only about one inch, whereas in the case of NC, the concrete head differential observed in the 48-inch model simulation was 4 inches when the vertical rebars were spaced at 7 inches apart and 10 inches when the rebars were placed at 3.5 inches apart. Based on this numerical evaluation of flow performance, it was concluded that the flow performance of SCC is better than that of NC, as the flow of the former minimizes or eliminates anomalies in the concrete cover. No chemical reactivity was incorporated into any of the models.

The development of the present numerical model to simulate concrete flow in drilled shafts and subsequent evaluation of the concrete flow performance, as carried out in this research study, make use of state-of-the-art numerical techniques and physical modeling. For example, the model developed takes into account the non-Newtonian nature of concrete flow. However, it does not take into account the effect of suspended aggregates in the concrete flow.

Chapter Five: Bond Strength Considerations

5.1 Background

5.1.1 Bond Strength

Whereas development length is a practical parameter required to ensure proper reinforcing steel performance, its determination stems from studies focused on the bond stress (*u*) between concrete and reinforcing bars (Orangun, et al., 1975; Darwin, et al., 1992; Darwin, et al., 1996; Darwin, et al., 1998; Zuo and Darwin, 2000; Sozen and Moehle, 1990). In short, it is the minimum length a steel reinforcing bar must be embedded in concrete to ensure the full strength of the bar can be "developed." Intuitively, the development length is dependent on both the steel and concrete strength parameters.

5.1.2 ACI 318-14 Development of the Current Bond Strength and Development/Splice Length Equation

Currently there are two development length equations provided by the American Concrete Institute (ACI), one in ACI 318-14 and the other in ACI 408R-03. The origins of these equations are briefly described below under their respective sections, both stem from the works of Orangun et al. (1975). A primary difference between the two equations is that in ACI 318-14, the bond strength is proportional to the square root of the concrete compressive strength (f'_c), whereas in ACI 408R-03, the bond strength is proportional to the fourth root of the concrete compressive strength (when disregarding transverse reinforcement). Note that terms are defined as necessary and only redefined if the base definition is altered or base units vary.

Orangun et al. (1975)

The current ACI 318-14 equation initially stems from the work of Orangun et al. (1975). In this study an expression (Equation 5.1) was developed for the average bond stress at failure using statistical techniques. Note the expression was normalized to the square root of f'_c. This was completed through a regression analysis based on 62 beams, for bars not confined by transverse reinforcement. This equation relates the bond strength, u_c, to the length of bond, encapsulating concrete cover thickness, size of bar, and concrete strength.

$$\frac{u_c}{\sqrt{f'_c}} = 1.22 + 3.23\frac{c_{min}}{d_b} + 53\frac{d_b}{l_d} \qquad \text{(Eq. 5.1)}$$

c_{min} = smaller of minimum concrete cover or ½ of the clear spacing between bars, in.
l_d= development or splice length, in.
d_b= nominal diameter of bar being developed or spliced, in.

This expression was then altered slightly by rounding to the form of Equation 5.2:

$$\frac{u_c}{\sqrt{f'_c}} = 1.2 + 3\frac{c_{min}}{d_b} + 50\frac{d_b}{l_d} \qquad \text{(Eq. 5.2)}$$

Orangun et al. (1975) also examined bars confined by transverse reinforcement, which led to the inclusion of an additional term to account for the spacing and strength of confining steel (stirrups), Equation 5.3.

$$\frac{u_b}{\sqrt{f_c'}} = \frac{u_c + u_s}{\sqrt{f_c'}} = 1.2 + 3\frac{c_{min}}{d_b} + 50\frac{d_b}{l_d} + \frac{A_{tr} f_{yt}}{500 s n d_b} \quad \text{(Eq. 5.3)}$$

A_{tr}= area of transverse reinforcement normal to the plane of splitting through the anchored bars, in.[2]
f_{yt} = yield strength of transverse reinforcement, psi
s = spacing of transverse reinforcement, in.
n = number of bars or wires being developed or lap spliced along the plane of splitting

Sozen and Moehle (1990)

The other study influencing the ACI 318-14 equation is the work of Sozen and Moehle (1990). Therein, Sozen and Moehle completed a study investigating bond strength data from 16 various sources. Their goal was to outline a design procedure for determining required development/splice length. They concluded with an equation for allowable bond strength (Equation 5.4) as well as development length (Equation 5.6).

$$u = \frac{1}{\alpha_a}\frac{1}{\alpha_b}\frac{1}{\alpha_c}\frac{1}{\alpha_d} 6\sqrt{f_c'} \quad \text{(Eq. 5.4)}$$

where u = allowable bond strength for design, psi
f'_c = compressive strength of concrete (6x12 in. cylinder), psi
α_a = 1.0 if minimum concrete cover and clear bar separation are less than 2.5 times the bar diameter, or 2/3 if minimum concrete cover and clear bar separation are not less than 2.5 times the governing bar diameter
α_b = 1.0 if the amount of uniformly distributed reinforcement along require development or lap splice length does not satisfy Equation 5.5, or 2/3 if the amount of uniformly distributed transverse reinforcement along required development or lap splice length does satisfy Equation 5.

$$\frac{A_{tr} f_y}{N d_b s} \geq 3000 psi \quad \text{(Eq. 5.5)}$$

A_{tr} = total cross-sectional area of transverse reinforcement within a spacing s and perpendicular to the plane of the bars being spliced or developed, in.[2]
N = number of (transverse) bars being developed or spliced in a layer
s = spacing of transverse reinforcement along bars developed or spliced, in.
α_c = 1.3 if depth of concrete mix placed in one lift under horizontal reinforcing bar exceeds 12 in. or 1.0 if depth of concrete mix placed in one lift under horizontal reinforcing bar does not exceed 12 in.
α_d = 1.5 if the bar is coated with epoxy or 1.0 if the bar is not coated with epoxy

$$l_s = \alpha_a \alpha_b \alpha_c \alpha_d \frac{f_y d_b}{24\sqrt{f_c'}} \quad \text{(Eq. 5.6)}$$

Current Recommendation for the Development of Deformed Bars and Deformed Wires in Tension

As indicated in Equations 4 and 6, Sozen and Moehle also normalized bond strength with respect to the square root of the concrete strength, f'c, which has been carried through to form the development length equation currently used by ACI 318-14 (Equation 5.7).

$$l_d = \left(\frac{3}{40} \frac{f_y}{\lambda\sqrt{f_c'}} \frac{\Psi_t \Psi_e \Psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)} \right) d_b \quad \text{(Eq. 5.7)}$$

where the confinement term $\frac{c_b+K_{tr}}{d_b}$ shall not exceed 2.5. This is based on the condition that values above 2.5 will result in a pullout failure whereas under 2.5 splitting failures are likely to occur. (ACI 318-14)

where $K_{tr} = \frac{40A_{tr}}{sn}$

Ψ_e = coating on the reinforcement factor; uncoated or zinc galvanized is 1, 1.5 if epoxy coated or zinc and epoxy dual coated with clear cover less than 3d_b or clear spacing less than 6d_b, other epoxy coated is 1.2

Ψ_s = size of reinforcement factor; #7 or larger is 1

Ψ_t = casting position factor; more than 12 in. of fresh concrete placed below horizontal reinforcement is 1.3, other is 1

Table 5.1 shows the simplified development length equations due to the preselection of $\frac{c_b+K_{tr}}{d_b}$ value. The first case listed represents a preselected value of 1.5, whereas 'other cases' use a value of 1.0. (ACI Committee 318, 2014)

Table 5.1 Simplified development length equations for ACI 318-14. (ACI Committee 318, 2014)

Spacing and Cover	No. 6 and smaller bars and deformed wires	No. 7 and larger bars
Clear spacing of bars or wires being developed or lap spliced not less than d_b, clear cover at least d_b, and stirrups or ties throughout l_d not less than the Code minimum Or Clear spacing of bars or wires being developed or lap spliced at least 2d_b and clear cover at least d_b	$\left(\frac{f_y \Psi_t \Psi_e}{25\lambda\sqrt{f_c'}}\right) d_b$	$\left(\frac{f_y \Psi_t \Psi_e}{20\lambda\sqrt{f_c'}}\right) d_b$
Other cases	$\left(\frac{3f_y \Psi_t \Psi_e}{50\lambda\sqrt{f_c'}}\right) d_b$	$\left(\frac{3f_y \Psi_t \Psi_e}{40\lambda\sqrt{f_c'}}\right) d_b$

The development length used shall be the greater of the length calculated from one of these equations using all necessary modification factors, or 12 in. Modification factors are necessary for lightweight concrete (λ), epoxy (Ψ_e), bar size (Ψ_s), and casting position (Ψ_t).

5.1.3 ACI 408R-03 Development of Proposed Bond Strength and Development/Splice Length Equation

Darwin et al. (1992)

The development of the current ACI 408R-03 equations again stem from the work of Orangun et al. (1975), as seen in 5.1.2.1. Following their work was the work of Darwin et al. (1992), which included reanalyzing the Orangun et al. (1975) data for bars not confined by transverse reinforcement leading to Equation 5.8, where c_{min} and c_{max} were incorporated as well.

$$\frac{T_c}{\sqrt{f_c'}} = \frac{A_b f_s}{\sqrt{f_c'}} = 6.67 l_d (c_{min} + 0.5 d_b)\left(0.08\frac{c_{max}}{c_{min}} + 0.92\right) + 300 A_b \qquad \text{(Eq. 5.8)}$$

where A_b = area of bar being developed or spliced

f_s = steel stress at failure, psi

c_{max} = maximum of c_{bot} or c_s, ($c_{max}/c_{min} \leq 3.5$), in.

c_{bot} = bottom concrete cover for reinforcing bar being developed, in.

c_s = minimum of (c_{so}, c_{si} +0.25) or min (c_{so}, c_{si}), in.

c_{si} = ½ the bar clear spacing between bars, in.

c_{so} = side concrete cover for reinforcing bars, in.

Darwin et al. (1996)

Now using a larger data base, Darwin et al. (1996) completed a study with 133 specimens not confined by transverse reinforcement and 166 specimens which were confined, all of which were bottom-cast bars. It was determined through analysis of these specimens that using the fourth root minimized the spread in data from variations in concrete strength, f'_c, and thus it provided a better representation of concrete strength for purposes of development length. This study also included the effect of relative rib area, R_r. The 1996 study resulted in Equations 5.9 and 5.10 for bars not confined and bars confined by transverse reinforcement, respectively.

$$\frac{T_c}{f_c'^{1/4}} = \frac{A_b f_s}{f_c'^{1/4}} = [63 l_d (c_{min} + 0.5 d_b) + 2130 A_b]\left(0.1\frac{c_{max}}{c_{min}} + 0.9\right) \qquad \text{(Eq. 5.9)}$$

where $\left(0.1\frac{c_{max}}{c_{min}} + 0.9\right) \leq 1.25$

$$\frac{T_b}{f_c'^{\frac{1}{4}}} = \frac{A_b f_s}{f_c'^{\frac{1}{4}}} = [63 l_d (c_{min} + 0.5 d_b) + 2130 A_b]\left(0.1\frac{c_{max}}{c_{min}} + 0.9\right) + 2226 t_r t_d \frac{N A_{tr}}{n} + 66 \qquad \text{(Eq. 5.10)}$$

where T_b = total bond force of a developed or spliced bar

$= T_c + T_s$

t_r = term representing the effect of relative rib area on T_s
$= 9.6R_r + 0.28$
t_d = term representing the effect of bar size on T_s
$= 0.72d_b + 0.28$
N = number of transverse stirrups or ties within the development length

It should be noted that these equations are based on the best-fit or average expression. Hence, a reduction factor was suggested to avoid under-prediction half of the time. In 1998 Darwin et al. produced a publication analyzing this strength reduction factor for bond, φ_b. The process is summarized in the subsequent paragraphs.

Darwin et al. (1998)

For bond, there are three reduction factors (φ) to consider. The first is φ_b, the strength reduction factor for bond. Next is φ which is the strength reduction factor for the main loading (i.e. bending). Last is φ_d, the effective strength reduction factor used in calculating development/splice length ($\varphi_d = \varphi_b/\varphi$). (Darwin, et al., 1998)

The first step in this analysis is to choose the level of confidence or reliability index needed. For structures, a reliability index of 3.5 is commonly used, which correlates to a 1 in 4149 failure ratio. Using 3.5 also produces a probability of failure equal to about one-fifth of that obtained with 3.0, which is commonly used for reinforced concrete beams and columns (Darwin, et al., 1998).

It should be noted Darwin et al. (1998) used random variables to conduct Monte-Carlo simulations; however, not all variables need to be random to complete this calculation. There are several variables of importance. The first is the nominal ratio of live load to dead load, denoted as $(Q_L/Q_D)_n$. Values used for these calculations were 0.5, 1, and 1.5. Next, X(2) and X(3) were actual-to-nominal dead and live load random variables, respectively. Darwin et al. stated for reinforced concrete structures X(2) and X(3) are 1.03 and 0.975, respectively. V_{QD} and V_{QL} (the coefficient of variation for the dead load and live load) were also given as 0.093 and 0.25, respectively. Load factors used (γ_D, γ_L) depend on the code used. These variables impact the calculation of the mean random loading variable, $\bar{q}$, and the coefficient of variation of the random loading variable ($V_{\varphi q}$), Equations 5.11 and 5.12.

$$\bar{q} = \left[\frac{\overline{X(2)+X(3)\left(\frac{Q_L}{Q_D}\right)_n}}{\gamma_D+\gamma_L\left(\frac{Q_L}{Q_D}\right)_n}\right] \qquad \text{(Eq. 5.11)}$$

$$V_{\emptyset q} = \frac{\left\{\left[\overline{X(2)}V_{Q_D}\right]^2+\left[\overline{X(3)}\left(\frac{Q_L}{Q_D}\right)_n V_{Q_L}\right]^2\right\}^{1/2}}{\overline{X(2)}+\overline{X(3)}\left(\frac{Q_L}{Q_D}\right)_n} \qquad \text{(Eq. 5.12)}$$

The random resistance variables, including mean test-prediction ratio ($\bar{r}$) and the coefficient of variation of the resistance random variable V_r can be randomly generated as well or calculated from test data. When using a set of data for analysis, $\bar{r}$ and V_r can be found by taking the mean of the measured/prediction ratio ($\bar{r}$), finding the standard deviation, and then dividing the standard deviation by the mean to solve for the coefficient of variation (V_r).

Using all variables discussed above the strength reduction factor for bond can be calculated using Equation 5.13.

$$\emptyset_b = \frac{\bar{r}}{\bar{q}} e^{-\left(V_r^2+V_{\emptyset q}^2\right)^{1/2}\beta} \quad \text{(Eq. 5.13)}$$

Once the bond reduction factor has been found, it is then divided by the strength reduction factor for loading (φ) to yield the effective strength reduction factor, φ_d. This reduction factor is then applied to applicable development length and bond stress equations.

Zuo and Darwin (2000)

Continuing to build upon past work, Zuo and Darwin released another paper on bond strength in 2000. For this research a total of 64 beam splice specimens were tested with various reinforcing bar sizes, concrete properties (strength and aggregate), and with or without the utilization of stirrups. While analyzing this data in conjunction with past data (171 specimens), a new form of equation 5.9 (equation 5.14) was produced, differing only slightly in that 63 decreased to 59.8 and the coefficient for A_b decreased to 2350 from 2130. (Zuo and Darwin, 2000)

$$\frac{T_c}{f_c'^{1/4}} = \frac{A_b f_s}{f_c'^{1/4}} = [59.8 l_d(c_{min} + 0.5 d_b) + 2350 A_b]\left(0.1\frac{c_{max}}{c_{min}} + 0.9\right) \quad \text{(Eq. 5.14)}$$

Regarding splices with transverse reinforcement, it was concluded that while the fourth root function of f'$_c$ for the concrete contribution (T_c) remained the most effective, the three-quarter power was most accurate for the transverse reinforcement (T_s). This led to the development of Equation 5.15. (Zuo and Darwin, 2000)

$$\frac{T_s}{f_c'^{3/4}} = 31.14 t_r t_d \frac{N A_{tr}}{n} + 3.99 \quad \text{(Eq. 5.15)}$$

t_d = term representing the effect of bar size on T_s
= $0.78 d_b + 0.22$

Combining Equations 5.14 and 5.15 results in Equation 5.16.

$$\frac{T_b}{f_c'^{1/4}} = \frac{A_b f_s}{f_c'^{1/4}} = [59.8 l_d(c_{min} + 0.5 d_b) + 2350 A_b]\left(0.1\frac{c_{max}}{c_{min}} + 0.9\right) + (31.14 t_r t_d \frac{N A_{tr}}{n} + 3.99)\sqrt{f_c'} \quad \text{(Eq. 5.16)}$$

Current Recommendation for the Development of Deformed Bars and Deformed Wires in Tension

ACI Committee 408 has since made minor changes to Equations 14 and 16, as seen in ACI 408R-03 (2003), to produce Equations 5.17 and 5.18.

$$\frac{T_c}{f_c'^{1/4}} = \frac{A_b f_s}{f_c'^{1/4}} = [59.9 l_d(c_{min} + 0.5 d_b) + 2400 A_b]\left(0.1\frac{c_{max}}{c_{min}} + 0.9\right) \quad \text{(Eq. 5.17)}$$

$$\frac{T_b}{f_c'^{1/4}} = \frac{A_b f_s}{f_c'^{1/4}} = [59.9 l_d(c_{min} + 0.5 d_b) + 2400 A_b]\left(0.1\frac{c_{max}}{c_{min}} + 0.9\right) + (30.88 t_r t_d \frac{N A_{tr}}{n} + 3)\sqrt{f_c'} \quad \text{(Eq. 5.18)}$$

In terms of development length, Equation 5.19 was produced from Equations 5.17 and 5.18. To ensure a low probability of failure, reduction factors were then found per the Darwin et al. 1998 process. ACI Committee 408 recommends using a strength reduction factor of 0.82 for dead and live load factors of 1.2 and 1.6, respectively, corresponding to a 0.9 reduction factor for bending. This was then applied to Equation 5.19 to yield Equation 5.20, the design equation recommended by ACI Committee 408. (ACI Committee 408, 2003)

$$\frac{l_d}{d_b} = \frac{\left(\frac{f_y}{\emptyset f_c'^{1/4}} - 2400\omega\right)\alpha\beta\lambda}{76.3\left(\frac{c\omega + K_{tr}}{d_b}\right)} \qquad \text{(Eq. 5.19)}$$

$$\frac{l_d}{d_b} = \frac{\left(\frac{f_y}{f_c'^{1/4}} - 1970\omega\right)\alpha\beta\lambda}{62\left(\frac{c\omega + K_{tr}}{d_b}\right)} \qquad \text{(Eq. 5.20)}$$

β = coating factor; uncoated reinforcement is 1, epoxy-coated is 1.5 with cover less than $3d_b$ or clear spacing less than $6d_b$, other epoxy-coated is 1.2

λ = lightweight concrete factor; normal weight concrete is 1

α = reinforcement location factor; 1.3 if reinforcement placed so 12 in. or more of fresh concrete is cast below the development length, other is 1.0

$\omega = \frac{c_{max}}{c_{min}} + 0.9 \le 1.25$

Note here the bond/development length concrete compressive strength uses the fourth root instead of the square root. The term $\frac{c\omega + K_{tr}}{d_b}$ must be less than or equal to 4.0. Modification factors taken into account are lightweight concrete (λ), epoxy (β), and casting position (α).

5.1.4 American Association of State Highway and Transportation Officials (AASHTO)

The current recommendations for the American Association of State Highway and Transportation Officials (AASHTO) are adapted from ACI 318-14, with the primary difference being that ksi is used instead of psi for concrete and steel strength (f'_c and f_y, respectively). The expression recommended has two parts, Equations 5.21 and 5.22.

$$l_d = l_{db}\left(\frac{\lambda_{ri}\lambda_{cf}\lambda_{rc}\lambda_{er}}{\lambda}\right) \qquad \text{(Eq. 5.21)}$$

$$l_{db} = 2.4 d_b \frac{f_y}{\sqrt{f_c'}} \qquad \text{(Eq. 5.22)}$$

Modification factors included in this equation are lightweight concrete (λ), epoxy (λ_{cf}), reinforcement location or casting position (λ_{ri}), and reinforcement confinement (λ_{rc}). The reinforcement confinement term is satisfied by the equation $\frac{d_b}{c_b + K_{tr}}$ where $k_{tr} = 40A_{tr}/(sn)$ and $0.4 \le \lambda_{rc} \le 1.0$, this is equivalent to the 2.5 limitation placed on the inverse of this term, seen in ACI 318-14. (AASHTO, 2017)

5.1.5 **Slurry Effects on Bond Strength**

None of the above codes (ACI 318-14, ACI 408R-03, AASHTO 2017) mention the potential effects of slurry on bond strength. ACI 408R-03 does mention bar cleanliness, where "*reinforcement must be free of mud, oil, and other nonmetallic coatings*" (ACI Committee 408, 2003). The FHWA manual, which references AASHTO's equations, does provide a section on bond and slurry. It states, *"Fleming and Sliwinski (1977) report that the general opinion is that there is no significant reduction of bond between concrete and the reinforcing steel in drilled shafts constructed under bentonite slurry"* (Brown, et al., 2010). They go on to give an acknowledgement from the Federation of Piling Specialists to increase bond, but not more than 10% of the value for plain bars. However the conclusions made in this section are predominantly based on testing completed by Butler and are as follows: *"The current state of knowledge on this topic suggests that the use of mineral and polymer slurries for drilled shaft construction does not reduce the bond resistance between concrete and reinforcing bars. There is currently no reason to account for the use of drilling fluids when considering development length of rebar in drilled shafts"* (Brown, et al., 2010).

Seal Slab to Pile Bond

Research conducted on tremie-placed seal slabs around steel H-piles and prestressed concrete piles revealed that the slurry or fluid to be displaced by the concrete can have an effect on the bond between the pile and seal slab (Sosa, 1999; Mullins, et al., 1999; Mullins, et al., 2001; Mullins, et al., 2002). Prior to this research, seal slab design assumed bond to be negligible or was fully discounted. Test results demonstrated that significant bond could be expected in some cases; however, when the fluid displaced was bentonite slurry, a notable reduction in bond was seen. The effects of fluid type on tremie-placed seal-slab-to-steel-pile bond are shown in Figure 5.1 below. For this study, full-scale seal slabs were cast around W14x90 steel pile sections and 14-in. prestressed concrete square piles located in cofferdams that were flooded with water or bentonite slurry. Dry conditions were also tested as controls. The concrete piles and steel pile tests gave similar results. Specifically for the steel piles, when compared to the dry construction values, water conditions produced average bond values 4% lower, and bentonite slurry showed up to a 54% reduction in bond by comparison. Polymer slurry was not tested.

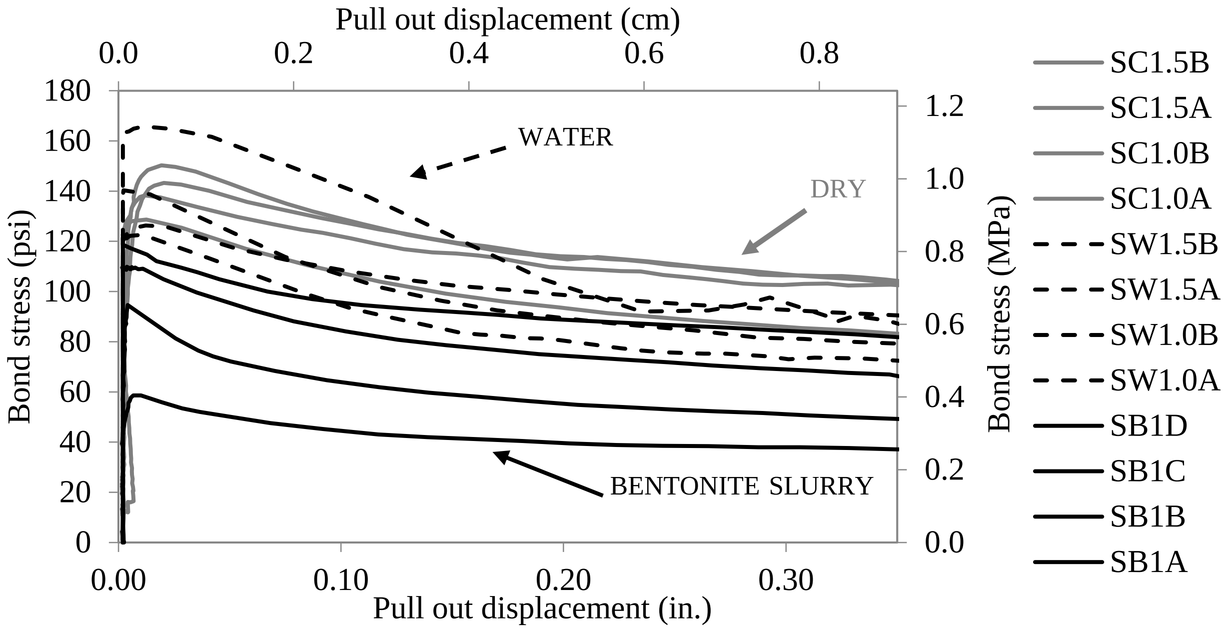


Figure 5.1 Steel pile to seal slab bond.

Jones and Holt (2004)

Jones and Holt investigated the effects of bentonite and polymer slurries on bond strength through previously performed laboratory and field testing. The first set of test data inspected were from the Construction Industry Research and Information Association (CIRIA) laboratory testing in 1967. Piles were cast by injecting concrete into the base of the form to displace the drilling fluid, hence the No. 7 ribbed reinforcement bar was vertical for casting. It should be noted that no transverse reinforcement was used in this study. Three casting conditions were tested: air, bentonite, and bentonite with added clay and sand. Six samples were produced per drilling condition, yielding 18 total samples. For testing, specimens were loaded in the direction of the concrete flow.

The results of this study showed that the average control (air) bond stress were 1.5 times higher than either bentonite average bond stress. Testing for slip was also performed and showed there was more slip for bentonite cast specimens at any given stress. Also included in this testing program were identical tests for straight and twisted bars. These bar types showed no significant loss of bond or stiffness and thus it was concluded that the flow of concrete sufficiently removes the bentonite in those cases. However, the same cannot be said for ribbed bars, where a bentonite residue was left on the tops of the ribbing, producing a reduction in bond stress.

The next study considered was site testing by Rail Link Engineering (RLE) in 2000. In this program, two 7-m-long piles were constructed per drilling condition. Each pile contained 6 No. 10 bars debonded along their length and anchored at varying depths. Thus 60 total samples were tested, 12 per drilling condition. The casting conditions tested were air, water, bentonite, polymer 60, and polymer 100; where 60 and 100 represent the target viscosity for polymer in sec/qt. The bentonite Marsh funnel viscosity was kept above 30 sec/qt, but below 50 sec/qt. Minimal transverse reinforcement were used, and it was assumed that concrete was filled from the base, thus resulting in loading in the direction of concrete flow.

Based on the assumed characteristic yield strength, anything below this value was assumed to be bond failure and anything above assumed to be the bar yielding. While air and bentonite showed bond stresses above the yield stress at all depths, water and polymer performed poorly at the base of the pile. While water was only poor at the base, polymer performance was variable throughout the entire length of the pile. This trend was worse in the polymer 100 than polymer 60.

While the poor behavior seen at the bottom could be from contamination of the base and position of the reinforcement, it was unclear why this poor performance was only seen in water and polymer, and not bentonite. Additionally, data was recorded to monitor slip however no consistent trends were noted. It was concluded that the accuracy of the testing may not be sufficient to produce reliable data.

The last set of test data examined was laboratory data from the Building Research Establishment (BRE) in 2001. This testing was prepared similarly to CIRIA as concrete was injected at the base and no transverse reinforcement was used. The primary difference was that the piles were cast so that testing could be performed on both ends to investigate the impact of testing direction with regards to concrete flow. The piles were cut in two with debonded and bonded regions on both sides of the cut. There were 10 reinforcement bar test samples per casting condition, 5 top and 5 bottom, making for 40 total test samples. The drilling conditions tested were air, bentonite, polymer 60, and polymer 100.

This testing concluded that top loading (in the direction of concrete flow) gave 10% lower bond stresses on average for slurry conditions than bottom loading (the opposite direction of concrete flow), which gave higher mean bond stresses. Load slip data was taken again with this experimentation but produced unreliable results.

While CIRIA indicated a significant drop in bond with the use of bentonite, using corrected bond values BRE showed all specimens were performing above what had been assumed from the code. Site testing from RLE showed no adverse effects from the use of bentonite slurry. It is important to recognize that both CIRIA and BRE testing did not include the use of transverse steel and RLE testing only included a minimal amount. The use of transverse steel will improve bond strength, however it also has the potential to make it worse.

CIRIA testing showed that in ideal-flow concreting, slurry contamination is removed from the bottom of the reinforcement ribbing, however pockets of slurry are trapped on the tops. As exposed by Deese and Mullins, tremie-placed concrete flows radially, not uniformly and not vertically, and under these conditions Jones and Holt warn *"is it possible for voids full of bentonite to be formed between the main bars and pile wall.*" Jones and Holt also acknowledge quilting that may be seen on the surface and state that "*in this situation the loss of bond capacity seen may be significantly greater and unlikely to be covered by a simple factor on the bond length.*" They go on to recommend further research in this area. (Jones and Holt, 2004)

Bowen (2013)

As mentioned above in section, Bowen performed research investigating the upper viscosity limit for bentonite slurry. One of many methods used to achieve this was through pullout capacity testing. While the procedure for construction of these samples can be found in Chapter 3 of this report, a few key points of his findings will be summarized here. Bowen cast the initial 18 shafts whose data are used in this report. These samples were not all cast with a bonded length of 6 in., as Bowen initially tested 18-in. and decreased to a 6-in. bonded region after finding the bond was too strong. Table 5.2 summarizes the qualities of these 18 shafts, which resulted in 126 pullout test specimens. Testing showed that rebar bond degraded by as much as 70% when cast with bentonite slurry, which was considered acceptable by most state construction specifications at the time. This issue can be related back to concrete flow issues as the concrete does not fully encapsulate the bar, leaving potential voids and potential pockets of trapped slurry.

Table 5.2 Summary of 18 shafts previously cast and tested by Bowen (2013).

Shaft #	Concrete Mix	Slurry Type	Viscosity	Bonded Length (in.)	Average Pullout Strength (kip)	Average Concrete Compressive Strength (psi)
1	4KDS	Bentonite	44	18	57.234	6,150
2	4KDS	Bentonite	105	18	49.704	6,150
3	4KDS	Bentonite	40	10	36.894	4,358
4	4KDS	Bentonite	55	10	32.697	4,358
5	4KDS	Bentonite	90	10	38.094	4,358
6	4KDS	Water	26	8, 10, 12	54.304	4,358
7	4KDS	Bentonite	30	6	28.754	4,530
8	4KDS	Bentonite	40	6	24.212	4,530
9	4KDS	Bentonite	50	6	20.524	4,530
10	4KDS	Bentonite	90	6	23.139	4,530
11	4KDS	Polymer 1	65	6	32.338	4,530
12	4KDS	Polymer 1	66	6	33.941	4,530
13	4KDS	Bentonite	30	6	25.636	4,753
14	4KDS	Bentonite	30	6	27.641	4,753
15	4KDS	Bentonite	56	6	19.804	4,753
16	4KDS	Polymer 1	85	6	24.077	4,753
17	4KDS	Polymer 1	85	6	26.247	4,753
18	4KDS	Water	26	6	34.042	4,753
Polymer 1- Polymer manufacturer one						

Another highlight of his findings was the examination of radial cores taken at the intersection of these creases that revealed the concrete was not contiguous at the creases. This was made exceedingly apparent when the high viscosity bentonite slurry sample was cored, it separated into four pieces defined by laitance crease locations. All four pieces were found to be coated with trapped bentonite slurry. Results from the coring can be seen in Figures 5.2 to 5.6, which show cores from a water-cast shaft, 60 sec/qt polymer-cast shaft, 30 sec/qt bentonite-cast shaft, 40 sec/qt bentonite-cast shaft, 50 sec/qt bentonite-cast shaft, and 90 sec/qt bentonite-cast shaft. On the water-cast shaft, there is a clean bond interface where the concrete meets the reinforcement bar. The polymer sample displayed a light slurry coating around the reinforcement bar.

Figure 5.2 Radial coring set up.

Figure 5.3 An empty core hole showing the surface crease extending to the reinforcing steel.

Figure 5.4 A water core (left) and 60 sec/qt polymer core (right).

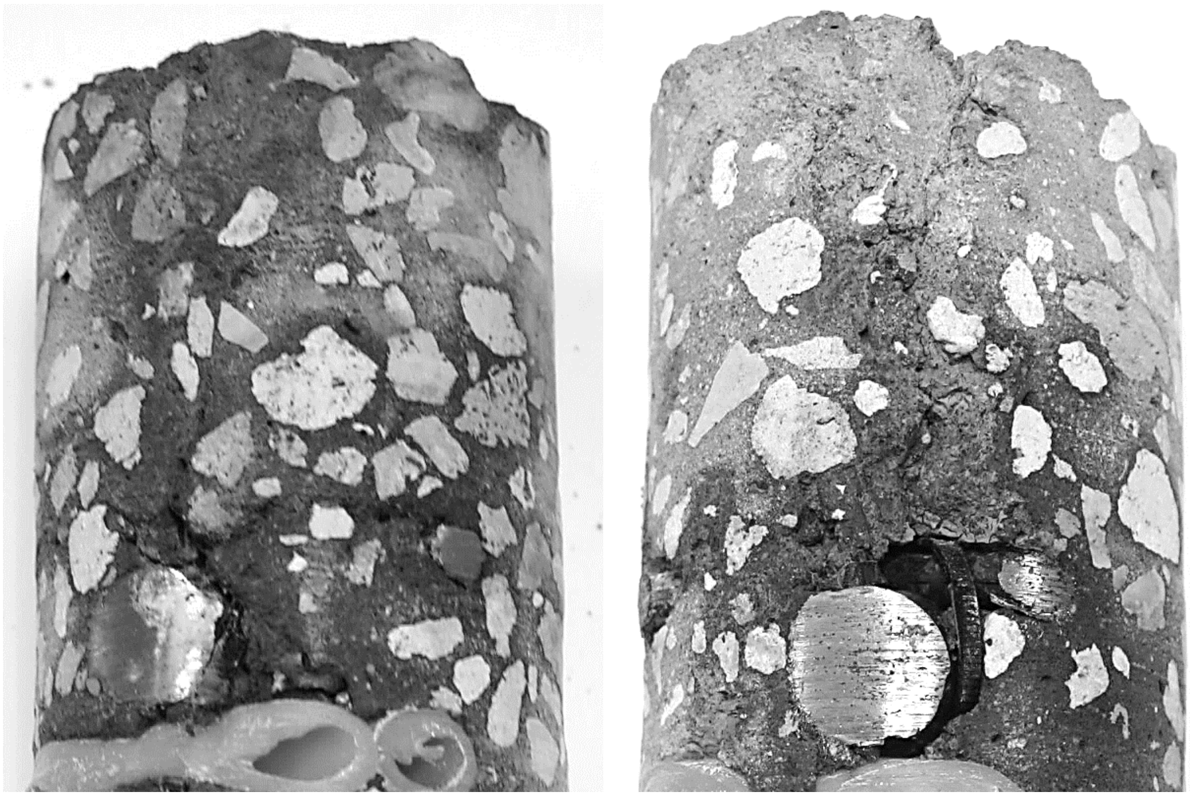


Figure 5.5 A 30 sec/qt bentonite core (left) and 40 sec/qt bentonite core (right).

Figure 5.6 Pieces from a 50 sec/qt bentonite core (left) and a 90 sec/qt bentonite core (right).

5.1.6 **Previously Accepted Pullout Capacity Testing Methods**

This section discusses two testing procedures to investigate bond strength completed by Butler (1973) and Smith-Emery Laboratories, Inc. (2015). Both methods of testing were flawed and not representative of construction practices. While Smith-Emery Laboratories testing was for Matrix polymer slurry used in the state of California, Butler's tests are currently referenced in the FHWA drilled shaft manual leading to the conclusion that there is no reason to account for drilling fluids when considering bond.

Butler (1973)

The research performed by Butler only included two pullout tests on No.8 reinforcement bars in bentonite-cast, tremie-placed concrete. One of the specimens failed from cover spalling through poor jack alignment and the other was a bond splitting failure with 2.25 in. of cover in a 30-in. diameter shaft. Additionally, the bentonite used for this testing was not pure bentonite, but a bentonite-polymer blend denoted as a high-yield bentonite. This product has half the amount of suspended solids as pure bentonite and is often not accepted for bridge construction applications (FDOT, 2018). The viscosity of the slurry was not reported, but it can be approximated to be 30 sec/qt based on the documented mix ratio of 0.21 lb/gal (Butler, 1973) and mix-ratio-to- viscosity correlations known for this type of slurry (Mullins, et al., 2013).

In addition to the two pullout tests performed on full-scale specimens, Butler also prepared 12 laboratory samples. Six of the samples were straight bars and the remaining 6 were deformed bars. From each set of six, three bars were cast in dry conditions and the other three were *"coated with mud slurry before casting in concrete,"* but not tremie-placed, slurry displacing concreting (Butler, 1973). Thus, there were only five applicable tests conducted upon which the FHWA guidelines for slurry effects on bond are based. In addition, none of these tests are representative of the construction practices.

Smith-Emery Laboratories (2015)

Currently Florida and California are two of the strictest states regarding polymer slurries. Polymers must be tested and approved prior to usage in these states; however, testing methods used are often inadequate (FDOT, 2018). An example of this can be demonstrated from Matrix polymer slurry testing for use in California. While the report from Smith-Emery Laboratories references ASTM A944 and C192 for bond strength and casting/curing concrete test specimens, respectively, it also states that modifications to these standards were necessary. Thus, instead of following the procedure for casting of the test specimen outlined in ASTM A944, a modification was made. For their testing, 18-inch long No.5 reinforcement bars were placed in 6-in. by 12-in. cylinders. The samples were prepared by first soaking the bottom 6 inches of the bars in the slurry to be tested for 12 hours. Then one soaked bar was embedded 6 inches below the top of the cylinder mold in the center of the test sample, hence the tip of the slurry-soaked portion was 6 inches below the cylinder's surface. This differs from ASTM A944, as it states that the specimen shall be "*cast in a block of reinforced concrete*" 23 to 25 in. long by 7.5 to 8.5 in. + d_b wide and a minimum of d_b + c_b + l_e +2.5 in. tall (ASTM A944). On dimensions alone, if the test was to be conducted with the cylinder on its side, the length is approximately half of the recommended value and the diameter is less than the recommended width and height. ASTM A944 also suggests the use of stirrups. Additionally, the use of PVC pipes as bond breakers is mentioned to "*avoid a localized cone-type failure of the concrete at the loaded end on the specimen*" (ASTM A944). This testing did not make use of debonded regions and was therefore susceptible to this type of failure.

The concrete used was Quickrete 5000 Concrete Mix. Based on the compressive strengths in the report, the concrete strength was just beyond the recommended strength for testing, 4500 to 5500 psi. The concreting process of the cylinders was performed using ASTM C192, where the concrete is added in three equal layers and rodded 25 times per layer. This differs from ASTM A944 as vibration is the stated method of consolidation to be used. In either method, this will not produce results comparable with tremie placement, as the concreting process is wholly different. By adding concrete in three equal layers and rodding or vibrating each layer, the slurry that would be trapped on the top of the ribbing due to tremie placement, as noted by Jones and Holt, would most likely not be there. As this residue is what primarily caused a fluctuation in bond stress with tremie-placed concrete (Jones and Holt, 2004), this test fabrication cannot be stated as equivalent. For pullout capacity testing, a station was set up similar to what is depicted in ASTM A944; however, instead of the unit performing a horizontal test, the sample was placed upside-down for testing.

Overall, these accepted test methods do not reflect field conditions for two substantial reasons. The creation and casting of the specimen should have been in tremie-placed conditions to simulate a realistic environment, and a debonded region should have been utilized to avoid a localized cone-type failure of the concrete.

During the RLE testing, performance results from polymer slurries proved to be variable and at times insufficient (Jones and Holt, 2004). In short, present test methods to verify slurry performance are inadequate.

The information stated above shows a clear research gap where some codes at least acknowledge bar cleanliness (under which slurry-coated reinforcement could fall), others do not acknowledge slurry effects on reinforcement at all. It is further concerning that FDOT would require such testing on new slurry products if slurry did not propose an issue to bond strength. This report focuses on the analysis of slurry effects on rebar bond where concrete is tremie-placed and slurry displacing.

5.2 Testing

This section outlines the testing procedures performed. The first testing procedure outlined is the process whereby the pullout bars were tested for capacity. A numerical model has been included which was created to confirm the credibility of the method used to complete pullout tests.

5.2.1 Pullout Testing

ACI 408R-03 has four different pullout testing configurations defined: (a) the pullout specimen, (b) the beam end specimen, (c) the beam anchorage specimen, and (d) the splice specimen (ACI Committee 408, 2003). None of the given configurations were practical for the specimens tested in this report, therefore a combination of cases (a) and (b) (Figure 5.7), was adopted to allow for a direct pullout of vertically cast specimens with debonded regions, case (c) (Figure 5.7).

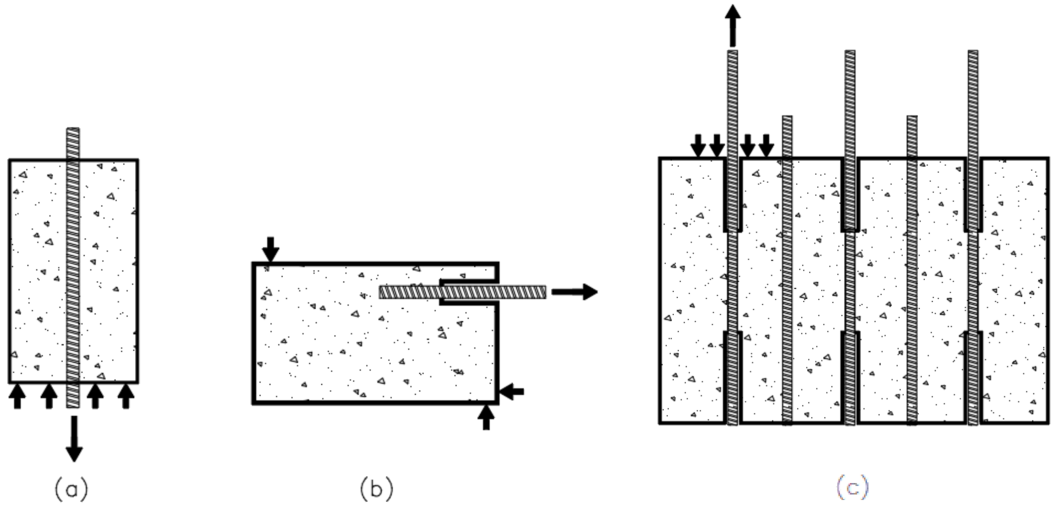


Figure 5.5.7 Pullout testing methods (a) pullout specimen and (b) beam end specimen and (c) the pullout testing method for tremie-placed specimens used in this study.

ACI 408R-03 notes case (a) to be the least realistic option as the stress fields produced rarely match construction (ACI 408R-03, 2001). To reduce this effect, precautions were taken during construction of the samples through the debonded regions. As discussed above, during construction, the upper 8 in. and lower 10 in. of each pullout sample were debonded. Debonding of the upper 8 in. of the bars was designed to reduce the effects of the jacking-induced compressive stress seen at the surface (ACI 408R-03, 2001). This was confirmed through numerical modeling to drastically reduce the compression stress in the bonded region. The lower debonded portion of the bar allowed for adjustments in bonded length.

Numerical Model of Test Method

A numerical model was generated to confirm the reduction of compressive stresses in the bonded region. This model was fabricated using Comsol 5.2a software. A general description and build of the model is as follows: First the type of model was selected, a three-dimensional, stationary structural model. Two models were generated for comparison, one with a 6-in. bond at the top of the shaft followed by an 18-in. debonded region, and the other using an 8-in. debonded top section followed by a 6-in. bond followed by a 10-in. debonded region, to simulate the test specimens. Both models depict 42-in. diameter, 24-in. tall shafts with 7 equally spaced No. 8 reinforcing bars. Only the pullout bars were incorporated into the model, not the entire reinforcing cage configuration. The other bars and transverse reinforcement were excluded as they do not have any influence on the pullout bar stresses.

Debonding was achieved by leaving a small space (0.05 in.) around the bars. The fabricated model can be seen in Figure 5.8 from multiple views. Note that the model shown has 8-in. and 10-in. debonded regions. The model was separated material wise into concrete and reinforcing steel. Figure 5.9 shows the highlighted section representing concrete, and the highlighted bars selected as reinforcing steel. Properties of the steel used for the pullout bars and of the concrete used for the shaft can be found in Tables 5.3 and 5.4, respectively. Note a concrete compressive strength of 4 ksi was used for this model.

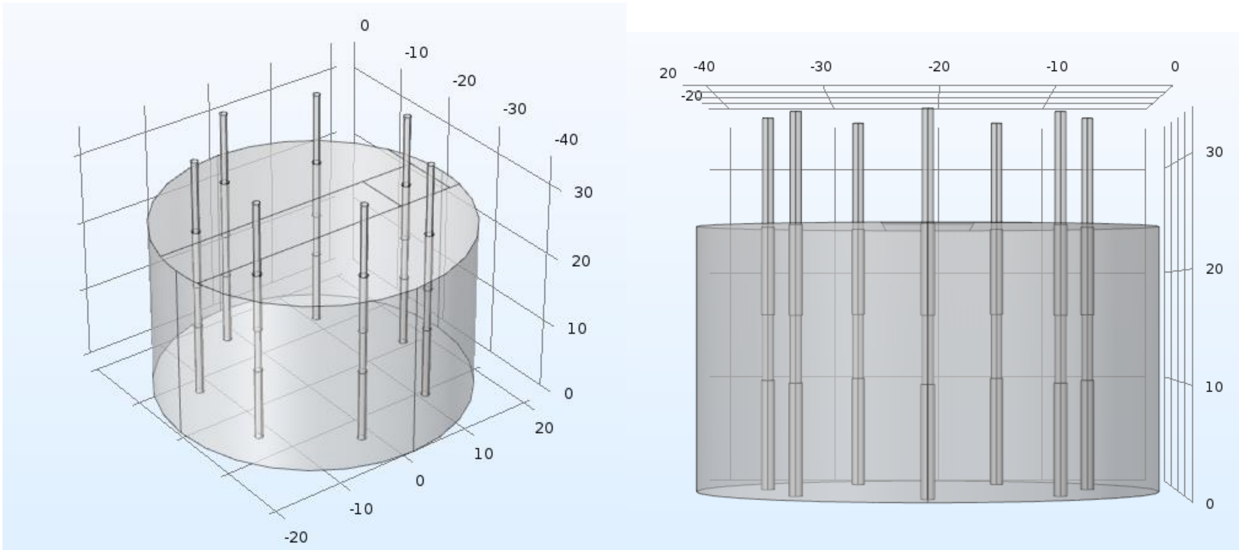


Figure 5.8 The generated model in Comsol 5.2a software.

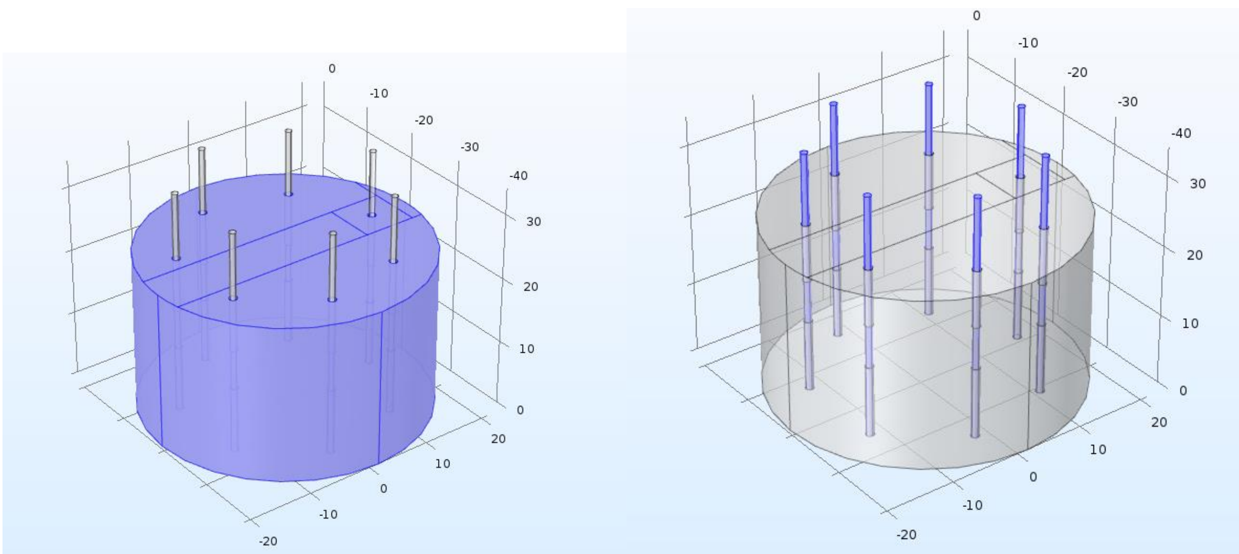


Figure 5.9 Material properties for models: highlighted sections represent concrete (left) and steel (right).

Table 5.3 Input material properties for reinforcing steel.

Property	Value
Density	7850 kg/m^3
Young's Modulus	29,000 ksi
Poisson's Ratio	0.30

Table 5.4 Input material properties for shaft concrete.

Property	Value
Density	2,300 kg/m^3
Young's Modulus	3,600 ksi
Poisson's Ratio	0.20

As seen in Figure 5.10 (left), cut planes were inserted to isolate an 8-in. by 8-in. square to represent the equal and opposite force applied to the concrete by the loading jack. For ease of modeling, loads were made input parameters as part of a parametric study, where the load at the tip of the tested reinforcing bar was *P* and the square region of concrete was loaded to –*P* (Figure 5.10).

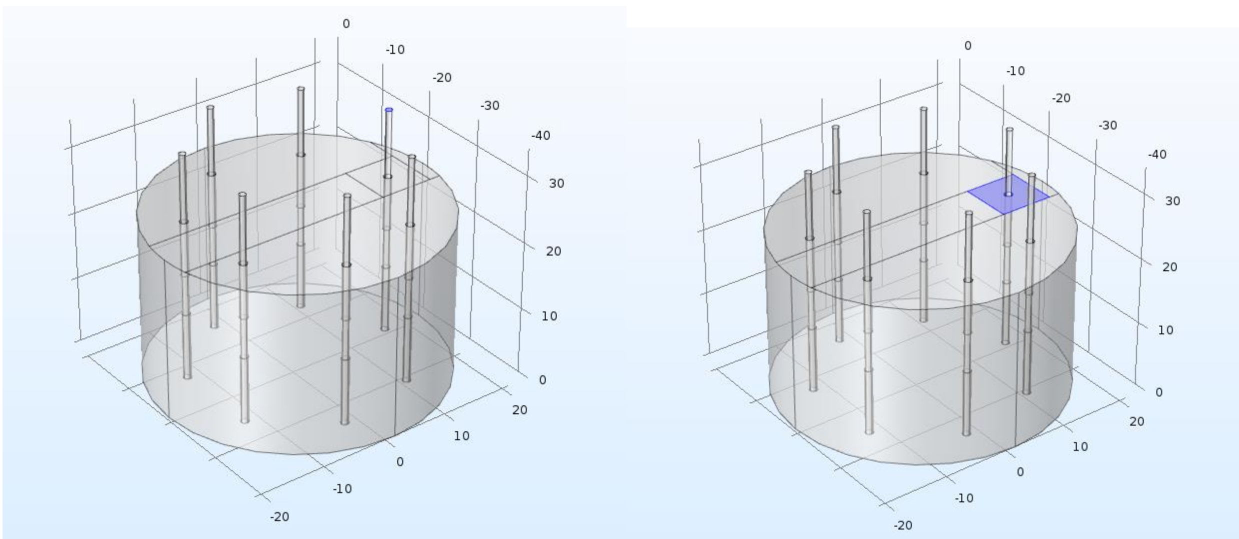


Figure 5.10 Loading parameters of model: load applied at tip of reinforcement tested, *P* (left), load applied to area concrete impacted by loading jack, -*P* (right).

The model was fixed on the side of the concrete walls, then a physics-controlled finer mesh was applied. The model was then run at values of 10, 20, 30, 40, and 50 kip; (1 kip = 1,000 lbf). A side-by-side comparison of a 6-in. bond directly below the concrete surface versus debonding the top 8 in. has been provided in Figure 5.11.

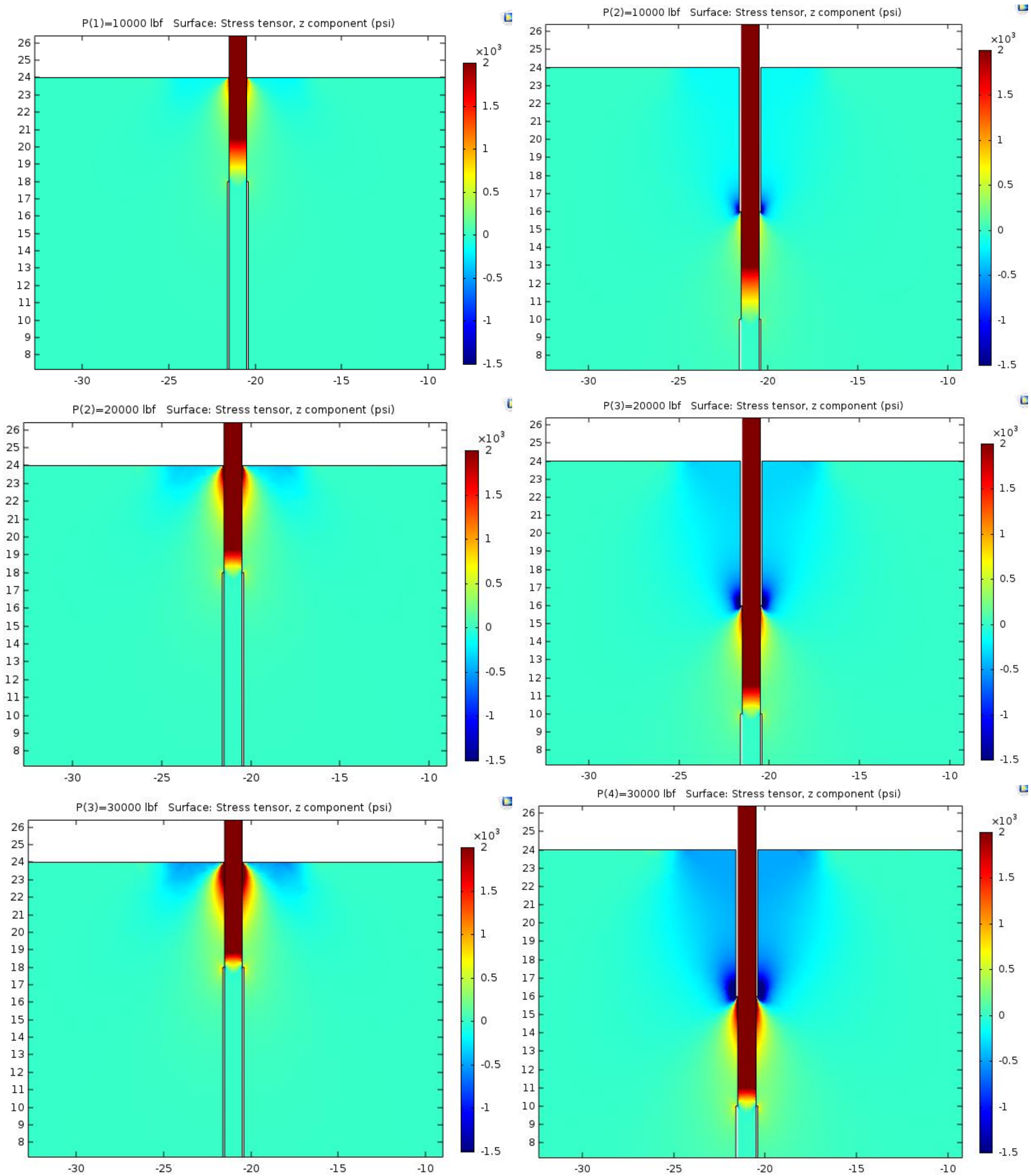


Figure 5.11 The loading of each model at 10, 20, 30, 40, and 50 kip; the top 6-in. bonded (left), the top 8-in. debonded (right).

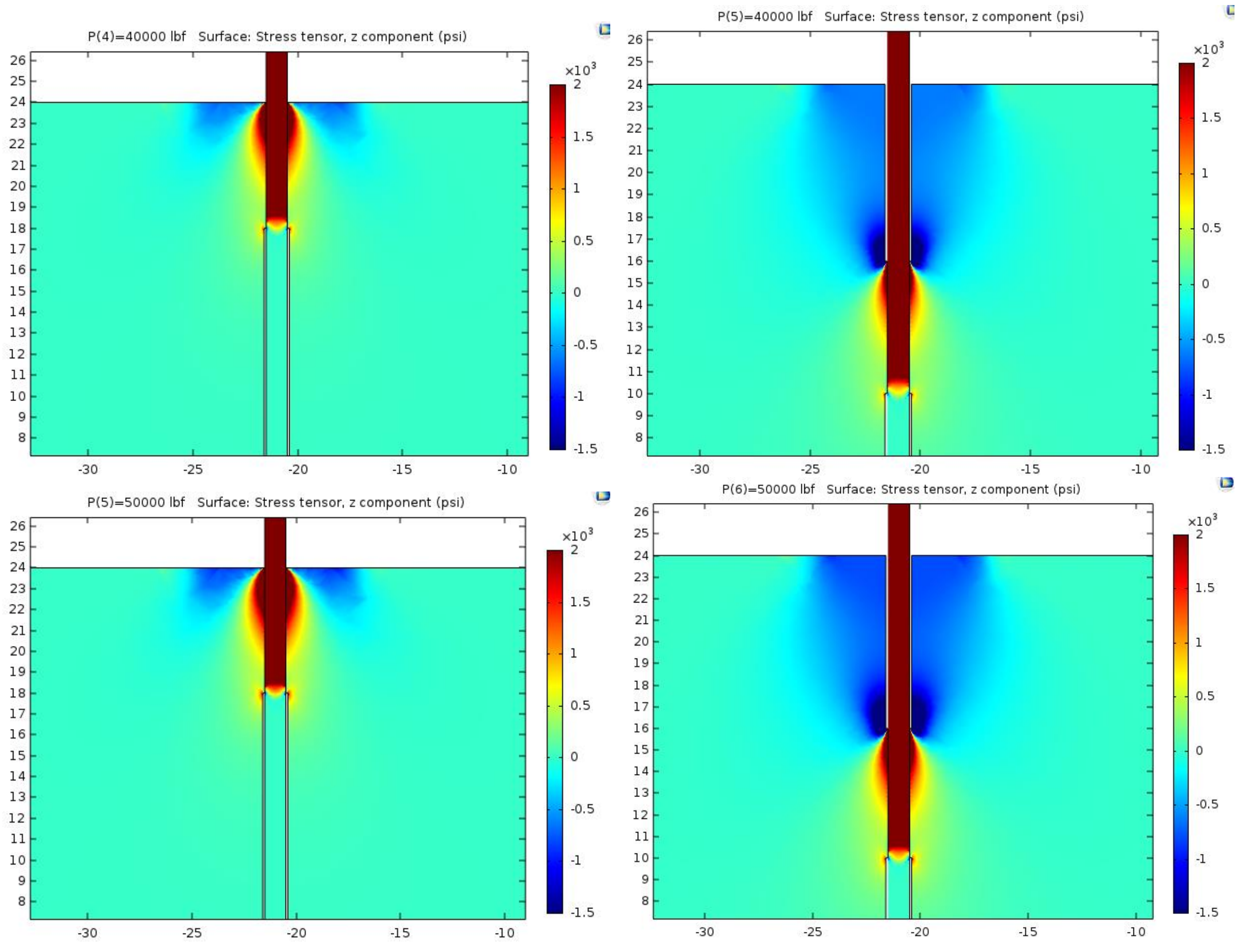


Figure 5.11 (Continued)

For modeling purposes, 50 kip was assumed to be the greatest magnitude encountered. The scales seen on the right side of the images in Figure 5.11 are identical for all, representing positive 2000 psi to -1,500 psi, tension to compression, respectively, where zero is neutral. Thus, the darker royal blue colors noted are the extreme end of the modeled compression scale. Note that the compressive stress seen on the concrete from the loading jack is generated in the same region as the tensile stress generated from the loading of the pullout bar when the 6-in. bond is next to the concrete surface (case (a)) (Figure 5.12), versus an almost linear division of compressive and tensile stresses when the bonded region is 8-in. below the concrete surface (Figure 5.13). Note regions of tension and compression have been exaggerated to illustrate the stress fields.

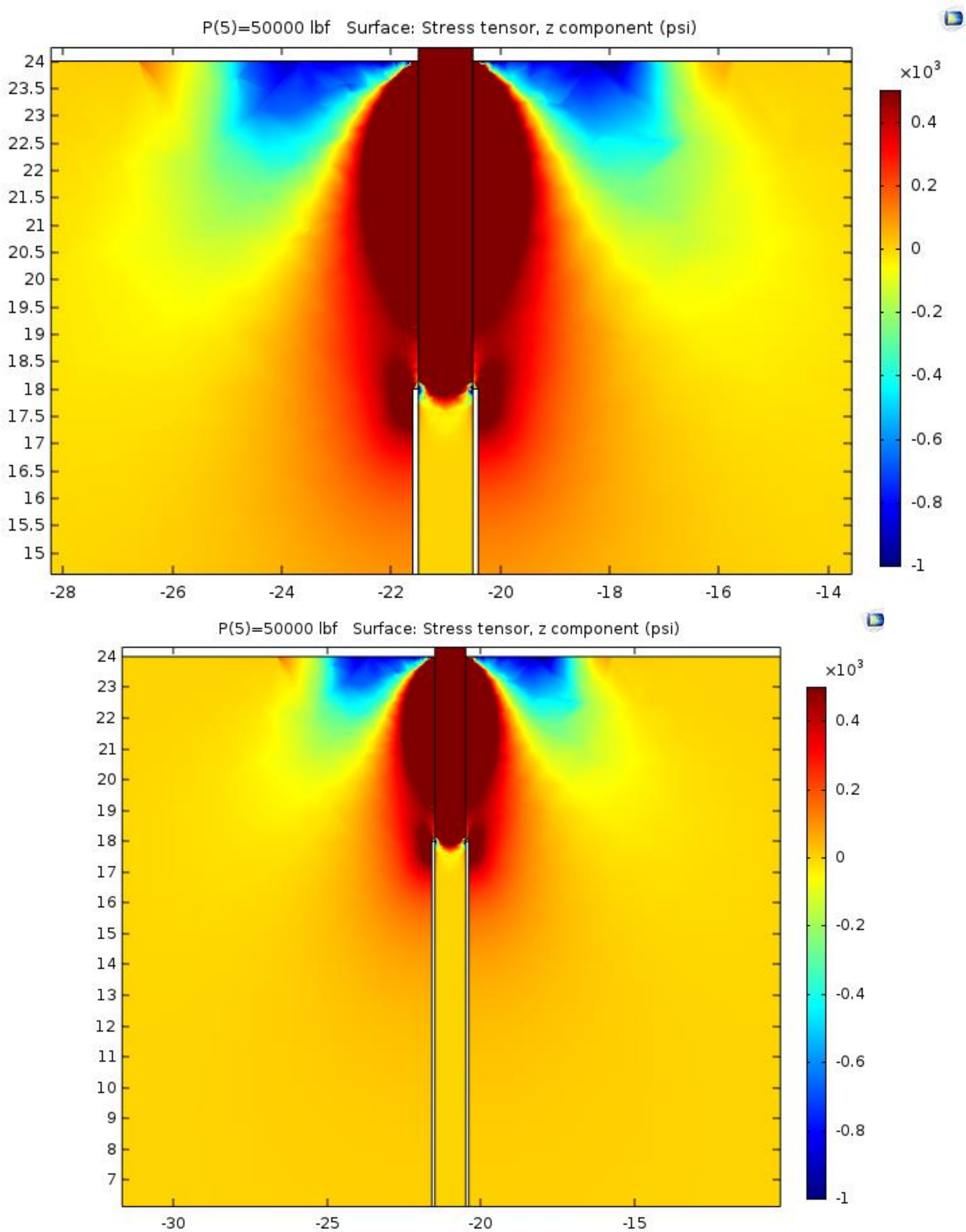


Figure 5.12 Zoomed in tension and compression fields for 6 in.-bond at the top.

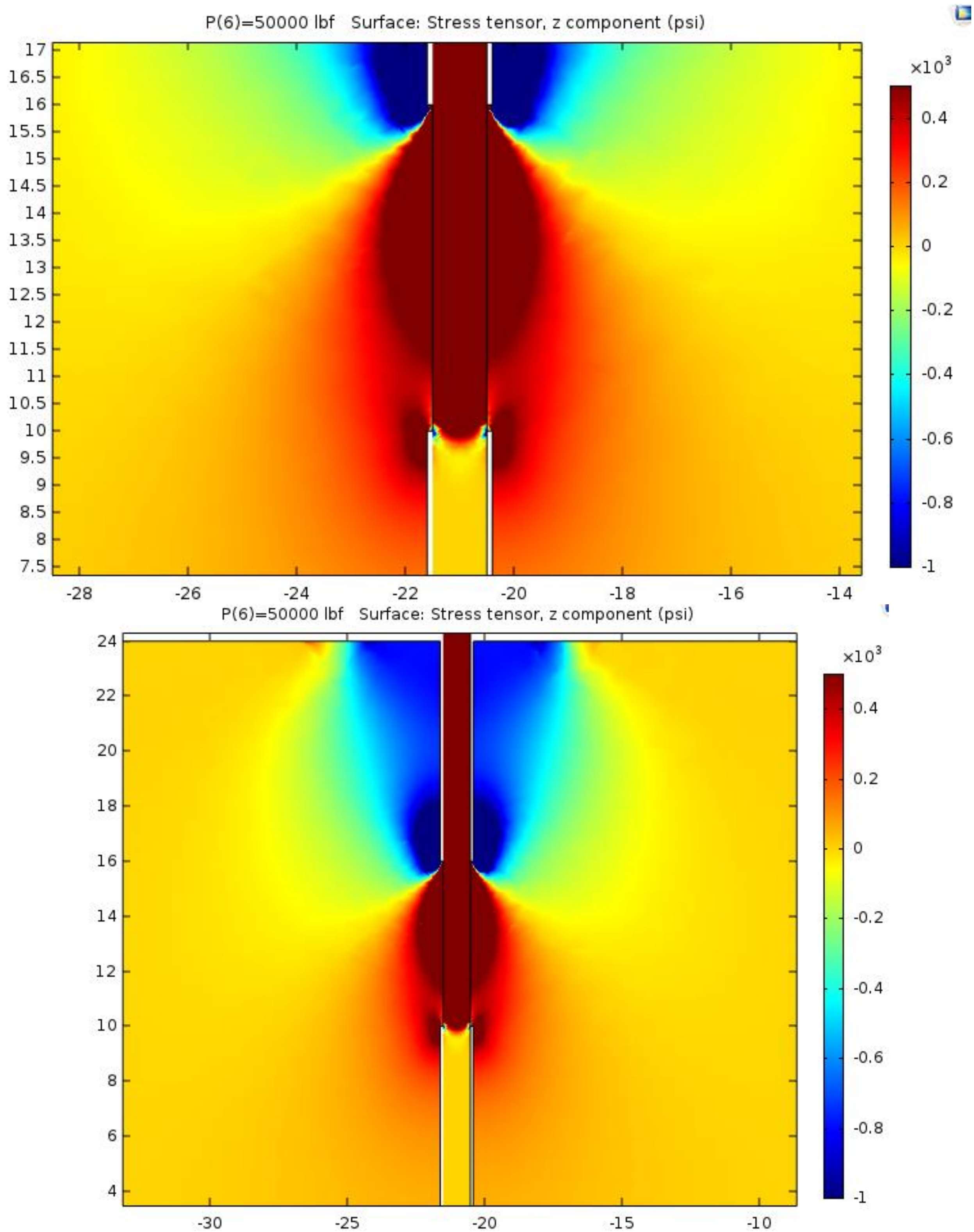


Figure 5.13 Zoomed in tension and compression stress fields for reinforcing bar pullout test method.

Thus this modeling has confirmed that the proposed pullout test procedure combining cases (a) and (b), is acceptable as the jacking-induced compressive stress seen is drastically reduced in the bonded region when the upper 8-in. of the bar are debonded.

Pullout Testing Procedure

The procedure for pullout testing conducted on the first 18 shafts can be found in Bowen's thesis document Bowen (2013). The procedure used for all other specimens is almost identical with subtle variances in the equipment. For pullouts conducted on the new shaft specimens, the procedure and equipment used were as follows.

A 60-ton capacity hollow-core hydraulic ram was placed over the bar to be tested and onto lead plates used to level the concrete surface. An 8-in. diameter load cell was then placed over the bar along with spacers and an upper steel plate. Double nuts at the top of the bar were used to secure the loading assembly and distribute the load along the entire threaded region. A displacement transducer was also attached to the side of the hydraulic ram to monitor movement. This assembly is displayed in Figure 5.14. A manually-operated hydraulic pump was used to conduct pullout testing, as it was able to apply a slow loading rate of approximately 100 lb/s.

Using a computerized data acquisition system, the load cell and displacement transducer were monitored at a sampling rate of 10 Hz, ensuring peak force would be captured (Figure 5.15). Testing was only performed once the concrete reached a minimum compressive strength of 4 ksi. All testing was completed on the same day as cylinder compressive strength testing. In total, including pullout specimens from Bowen's testing, 268 tests were performed. This included 138 tests in mineral slurry, 82 in polymer slurry, and 47 in water.

Figure 5.14 General assembly used for pullout testing: loading jack, load cell, steel plate, double nuts, and a displacement transducer.

Figure 5.15 Testing of the pullout bars: as the bar is being loaded the computer is being monitored.

5.3 **Pullout Testing Initial Results**

As detailed in above, pullout testing was performed only after the concrete reached the desired compressive strength. Compressive strength testing was performed using 4-in. by 8-in. concrete cylinders that were cast using the same concrete batched for the shafts. On the day of pullout capacity testing, two concrete cylinders were tested; the average of the concrete compressive strength results can be found in Table 5.5.

Table 5.5 Average concrete compressive strength determined from concrete test cylinders.

Shaft Grouping	Shaft Numbers	Average Compressive Strength (psi)
19 to 22	19 to 22	6,130
23 to 24	23, 24	6,130
31 to 36	31, 34 to 36	6,130
31 to 36	32, 33	6,160
47 to 52	47 to 49	9,130
53 to 58	53, 54	5,950
53 to 58	55 to 58	6,020

Results of the pullout strength testing have been divided by concrete placement date/shaft grouping. Shafts 19 to 24, 31 to 36, 47 to 49, and 53 to 58 are shown below in Tables 5.6, 5.7, 5.8, and 5.9, respectively. In Tables 5.6 through 5.9, slurry type and viscosity are denoted by the initial of the slurry type used during the casting process (i.e. B, P, W, A stands for bentonite, polymer, water, attapulgite, respectively) followed by the measured viscosity. Results highlighted in red indicate that the bar broke during testing, this can also be stated as a bar failure. At the bottom of each table the maximum and minimum pullout capacity, along with the average and standard deviation have been provided. It should be noted only 6 pullout bars per shaft were tested for shafts 47, 48, and 49.

Table 5.6 Pullout data from shafts 19 to 24.

	Class IV Concrete				Preferred SCC	
	Shaft 19 P63	Shaft 20 P121	Shaft 21 B42	Shaft 22 W26	Shaft 23 W26	Shaft 24 B40
Bar #	Max Load (kip)	Max Load (kip)	Max Load (kip)	Max Load (kip)	Max Load (kip)	Max Load (kip)
1	26.47	9.57	19.91	32.17	57.22	15.07
2	24.24	19.40	20.80	29.54	44.20	23.26
3	20.34	23.52	18.31	22.52	44.57	29.87
4	17.53	17.13	19.22	27.05	53.50	17.69
5	17.71	18.05	20.31	27.99	56.39	23.73
6	21.26	25.90	20.26	21.83	52.29	16.47
7	18.74	21.32	26.36	24.70	49.59	17.21
Maximum	26.47	25.90	26.36	32.17	57.22	29.87
Minimum	17.53	9.57	18.31	21.83	44.2	15.07
Average	20.90	19.27	20.74	26.54	51.11	20.47
Standard Deviation	3.39	5.27	2.61	3.76	5.25	5.33

Table 5.7 Pullout data from shafts 31 to 36.

	Class IV Concrete					
	Shaft 31 P98	Shaft 32 W26	Shaft 33 B39	Shaft 34 A39	Shaft 35 A200+	Shaft 36 P47
Bar #	Max Load (kip)	Max Load (kip)	Max Load (kip)	Max Load (kip)	Max Load (kip)	Max Load (kip)
1	52.62	55.01	35	34.22	38.71	59.25
2	48.52	49.64	41.96	39.13	39.87	58.76
3	28.77	43.03	41.15	48.13	50.22	59.71
4	13.68	49.36	34.96	46.71	35.68	43.38
5	42.06	56.13	42.78	37.4	32.68	37.73
6	47.41	42.87	57.74	41.15	34.54	44.24
7	41.38	40.92	42.5	49.06	31.68	39.13
Maximum	52.62	56.13	57.74	49.06	50.22	59.71
Minimum	13.68	40.92	34.96	34.22	31.68	37.73
Average	39.21	48.14	42.30	42.26	37.63	48.89
Standard Deviation	13.59	6.07	7.61	5.77	6.30	9.95

Table 5.8 Pullout data from shafts 47 to 49.

	Argos SCC		
	Shaft 47 W26	Shaft 48 B39	Shaft 49 B31
Bar #	Max Load (kip)	Max Load (kip)	Max Load (kip)
1	-	36.33	52.06
2	27.82	50.35	51.10
3	48.17	44.13	51.27
4	46.65	45.15	48.76
5	51.26	39.21	54.90
6	55.11	37.22	48.95
Maximum	55.11	50.35	54.90
Minimum	27.82	36.33	48.76
Average	45.80	42.06	51.17
Standard Deviation	10.56	5.42	2.26

Table 5.9 Pullout data from shafts 53 to 58.

	Class IV Concrete					
	Shaft 53 P49	Shaft 54 P58	Shaft 55 P120	Shaft 56 P85	Shaft 57 B40	Shaft 58 W26
Bar #	Max Load (kip)	Max Load (kip)	Max Load (kip)	Max Load (kip)	Max Load (kip)	Max Load (kip)
1	56.60	59.05	45.88	48.58	45.86	58.68
2	54.05	48.49	44.24	48.66	48.91	54.60
3	56.22	36.17	36.97	49.59	38.04	51.58
4	-	50.67	33.91	49.28	53.54	47.52
5	50.41	49.55	37.78	47.53	36.70	49.67
6	45.59	57.10	47.18	42.36	54.49	51.99
7	54.05	56.80	46.01	51.58	43.42	49.21
Maximum	56.60	59.05	47.18	51.58	54.49	58.68
Minimum	45.59	36.17	33.91	42.36	36.70	47.52
Average	52.82	51.12	41.71	48.23	45.85	51.89
Standard Deviation	4.17	7.79	5.34	2.87	7.00	3.76

5.4 **Analysis**

This section details the methods of analysis used for the testing program. The predicted pullout capacities were analyzed using two methods meant to confirm the findings, the level of reliability and Monte Carlo simulation. Using these analyses, resistance factors were generated for all slurry types aside from attapulgite in Class IV concrete in accordance with ACI 318-14 and ACI 408R-03, as well as comments in AASHTO. Bentonite slurries were then examined by varying viscosity groupings, the polymer slurries by manufacturer, and water versus bentonite in terms of SCC. SCC and attapulgite slurry are also discussed separately in their respective sections.

5.5 **Level of Reliability**

This method of resistance factor determination uses the desired level of reliability (reliability index) and an equation for the calculation of resistance factors. Details of this procedure can be found in Darwin et al. 1998, however this section will present the general calculation.

The equation used for resistance factor determination is:

$$\emptyset_b = \frac{\bar{r}}{\bar{q}} e^{-\left(V_r^2 + V_{\emptyset q}^2\right)^{1/2} \beta} \qquad \text{(Eq. 5.13)}$$

presented from Darwin et al. 1998, where $\emptyset_b$ is the bond resistance factor, $\bar{q}$ is the mean loading random variable, $V_{\varphi q}$ is the loading coefficient of variation, $\bar{r}$ is the mean resistance test-prediction ratio, V_r is the resistance coefficient of variation, and β is the reliability index.

The calculation of the mean loading random variable and coefficient of variation have previously been discussed using equations 5.11 and 5.12. Resistance variables ($\bar{r}$ and V_r) are the mean bias and coefficient of variation calculated using the pullout data. The desired reliability index is 3.5, which translates to a failure ratio of 1 in 4149. While this has been deemed a reasonable value, the actual failure ratio of structural elements is far less.

5.6 **Monte Carlo Simulation**

A Monte Carlo simulation is a randomly-generated probability model. In this report, Monte Carlo analyses were used to predict the probability of failure by generating one million random values for both the load (Q) and resistance (R). The simulation used in this report works in the following manner.

Failure occurs when the resistance is less than the loading. The calculation of the number of failures is based upon the equation $\emptyset R_N \geq R_U$ where R_N is the nominal resistance and R_U, the factored load, is equivalent to $1.2DL + 1.6LL$. In this simulation the mean loading was considered an input value, thus solving for the mean resistance using the following process.

$$\emptyset R_N \geq R_U$$

$$\emptyset R_N = 1.2DL + 1.6LL \quad \text{(Eq. 5.23)}$$

$$R_N = \frac{R}{r} \quad \text{(Eq. 5.24)}$$

$$\emptyset \frac{R}{r} = 1.2DL + 1.6LL$$

$$R = \frac{r(1.2DL + 1.6LL)}{\emptyset}$$

The following equation gives the mean resistance:

$$\bar{R} = \frac{\overline{r(1.2DL + 1.6LL)}}{\emptyset}$$

Here $\emptyset$, the strength reduction factor for the loading under consideration, is equivalent to the bond reduction factor. The variable r is the bias value for the resistance. This formulation was derived from Darwin et al. 1998.

Now that a formula for mean resistance has been determined, it must be considered that the load input is a mean value. This predicates a mean load factor (LF) must be used as well. For this calculation a dead-to-live-load ratio of 2 was used:

$$\overline{LF} = \frac{1.2DL+1.6LL}{DL+LL} = \frac{1.2(2)+1.6(1)}{2+1} = \frac{4}{3} = 1.33 \quad \text{(Eq. 5.25)}$$

Thus the final equation for mean resistance is then:

$$\bar{R} = \frac{\overline{r(LF*Q)}}{\emptyset} \quad \text{(Eq. 26)}$$

With mean values for resistance and load determined, the standard deviation can now be calculated. The standard deviation is equivalent to mean value times the coefficient of variation. For load, with a dead-to-live-load ratio of 2, a 0.102 coefficient of variation (COV) stays constant for all simulations. In terms of resistance, the same calculation is applied; however, the COV changes based on the data set under investigation (for this report, the casting condition).

Considering the data follows a log-normal distribution, the calculated mean and standard deviation values for load and resistance were converted to fit a log-normal distribution using the following equations:

$$\mu_{lnR,Q} = ln\mu_{R,Q} - \frac{1}{2}\sigma^2_{lnR,Q} \quad \text{(Eq. 5.27)}$$

$$\sigma^2_{lnR,Q} = \ln\left[1 + \left(\frac{\sigma_{R,Q}}{\mu_{R,Q}}\right)^2\right] \quad \text{(Eq. 5.28)}$$

where μ is equivalent to the mean and σ the standard deviation of either R or Q.

Now that all variables needed for the simulation have been established, the Monte Carlo simulation can be generated in a few simple steps. In excel, establish columns for one hundred thousand generations of two random variables (X and Y, i.e. one for load and one for resistance). Each column will have the same formulation to create random variables X and Y. For the creation of a normal distribution, the excel function is norminv, which requires the probability (the random variable), mean (zero is used here), and standard deviation (one). The function in excel looks like this: "=norminv(rand(),0,1)".

To generate the random variable used for the failure ratio, the standard deviation of the load or resistance is now multiplied by X or Y, respectively, and added to the mean. While this is the general formula, as stated above this data follows a log-normal distribution and thus, the standard deviation and mean used for this process are in log form, established from equations 5.27 and 5.28 above. To return the data to a normal distribution, the exponent of the randomly generated variable for failure ratio was taken.

These two values were then compared so that if load was greater than the resistance, a failure would occur. The failures were then totaled. Ten simulations were run per condition to accumulate one million data points. To achieve the failure ratio the, total number of failures (of all ten trials) was divided by one million, then the inverse was taken, resulting in one in the numbert of failures calculated. An example of this sheet for further clarification can be found in Appendix C.

5.7 Analysis of Pullout Capacity Data

Interpretation of this data was performed in several stages using a statistical analysis. Pullout results from the first 18 shafts cast by Bowen were included in this analysis. First the predicted pullout capacity was calculated using Equations 5.7 and 5.18, representing ACI 318-14 and ACI 408R-03. Note Equation 5.7 was rearranged to solve for bond force yielding Equation 5.29:

$$T_b = 10.472\emptyset_b\sqrt{f_c'}\left(c_b + \frac{40A_{tr}}{sn}\right)l_d \qquad \text{(Eq. 5.29)}$$

The resistance factor for these calculations was taken to be 1.0. An example of this calculation is shown below using data for shaft 34, variables can be found in Table 5.10.

Table 5.10 Variables for example.

ACI 318 and 408R-03	
f'_c	6,130 psi
l_d	6 in.
A_{tr}	0.11 in.^{2}
n	1
ACI 318	
c_b	6 in.
s	6 in.
ACI 408R-03	
c_{min}	3.25 in.
c_{max}	3.25 in.
R_r	0.07[1]
d_b	1 in.
A_b	0.79 in.^{2}
N	1

Example calculation for ACI 318-14 is:

$$T_b = 10.472\emptyset_b\sqrt{f_c'}\left(c_b + \frac{40A_{tr}}{sn}\right)l_d$$

$$T_b = 10.472(1)\sqrt{6130\text{ psi}}\left(6\text{ in.} + \frac{40(0.11\ in.^2\ )}{6\ in.* 1}\right)6\text{ in.} = 33124\text{ lb} = 33.12\text{ kip}$$

Example calculation for ACI 408R-03 is:

$$T_b = (f_c')^{1/4}\left[[\emptyset_b 59.9 l_d(c_{min} + 0.5d_b) + \emptyset_b 2400 A_b]\left(0.1\frac{c_{max}}{c_{min}} + 0.9\right) + \left(30.88 t_r t_d \frac{NA_{tr}}{n} + 3\right)f'^{\frac{1}{2}}_c\right]$$

$$t_r = 9.6R_r + 0.28 = 0.952$$

$$t_d = 0.78d_b + 0.22 = 1$$

[1] (Darwin, Zuo, Tholen, & Idun, 1996)

$$T_b = (6130\ psi)^{1/4}\left[\left[(1)59.9(6\ in.)\big(3.25\ in. + 0.5(1\ in.)\big) + (1)2400(0.79\ in.^2)\right]\left(0.1\frac{3.25\ in.}{3.25\ in.} + 0.9\right) + \left(30.88(0.952)(1)\frac{(1)(0.11\ in.^2)}{1} + 3\right)(6130\ psi)^{1/2}\right] = 33{,}020\ lbs = 33.02\ kips$$

Once the calculations were completed for all samples, the results were then compared to the measured values. This comparison can be seen in Figures 5.16 and 5.17. Generally, the mean experimental values agreed with the predicted capacities. Given that most specimens had a similar concrete strength and bond length, many of the predicted capacity values were similar, resulting in vertical banding.

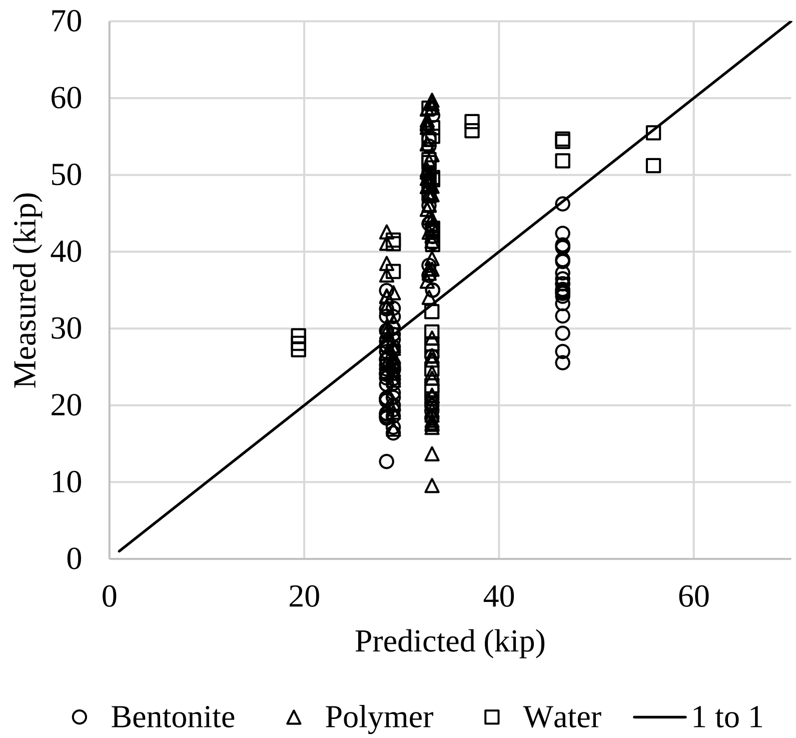


Figure 5.16 Measured strength versus predicted strength for ACI 318-14.

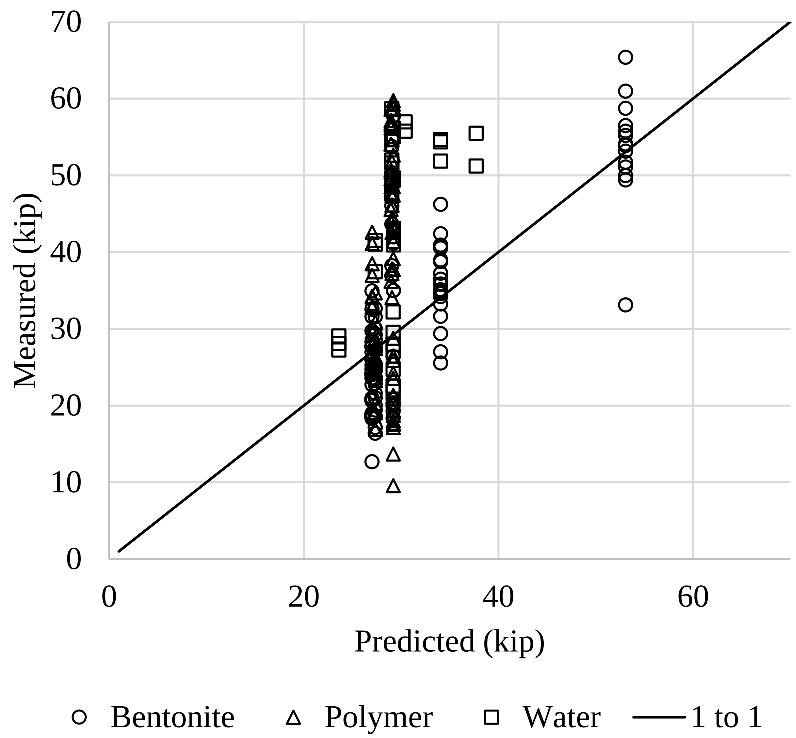


Figure 5.17 Measured strength versus predicted strength for ACI 408R-03.

Using the measured and predicted capacities, the bias (measured/predicted) for each sample was found. This was then plotted against the Marsh funnel viscosity for ACI 318-14 and ACI 408R-03, in Figures 5.18 and 5.19, respectively. The lines seen on Figures 5.18 and 5.19 indicate the mean bias value per casting condition where ACI indicates dry conditions. A general trend can be noted on both figures of decreasing bias and therefore pullout capacity with increasing slurry viscosity. Similar to Figures 5.16 and 5.17, Figures 5.18 and 5.19 still demonstrate significant variability, but now for a given viscosity.

With regards to slurry viscosity and soil type, higher viscosity slurry is requisite for more porous, free-flowing soil types, whereas lower viscosity slurry is appropriate for fine-grained, low permeability soils. Thus while lower viscosity slurry, which is typically closer to the viscosity of water, was seen to perform better in bond, it is not practical to restrict the use of higher viscosity slurry. Hence, a statistical evaluation of slurry effects was performed. For this the data was divided up by slurry types of water, polymer, and bentonite. Attapulgite was excluded as there was not enough data for an accurate analysis.

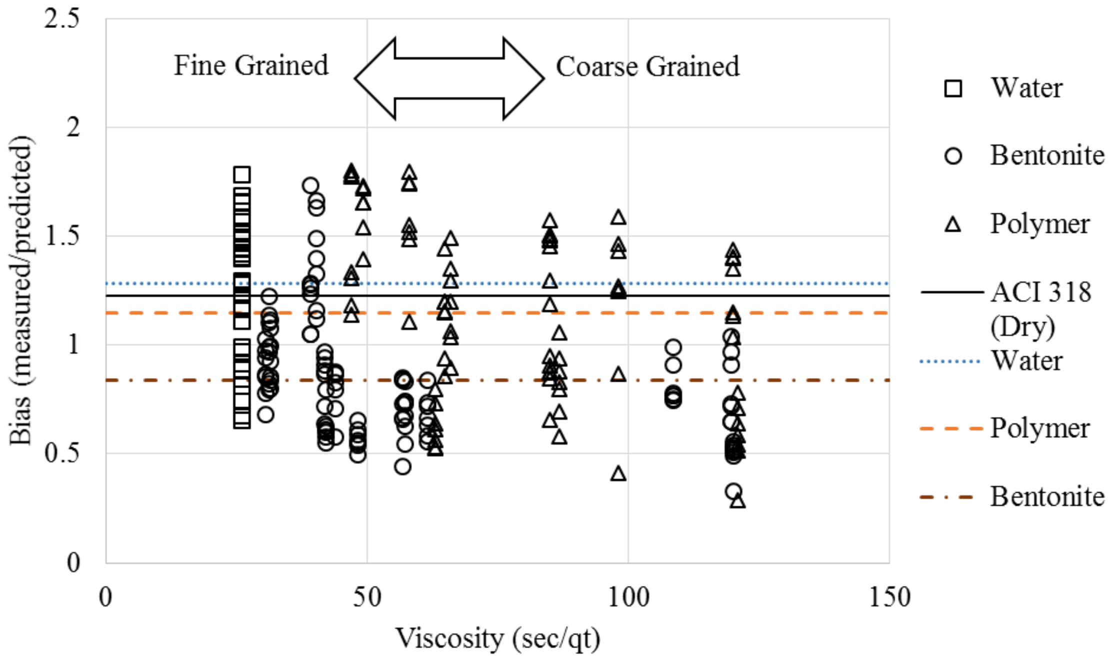


Figure 5.18 Bias versus slurry viscosity for ACI 318-14.

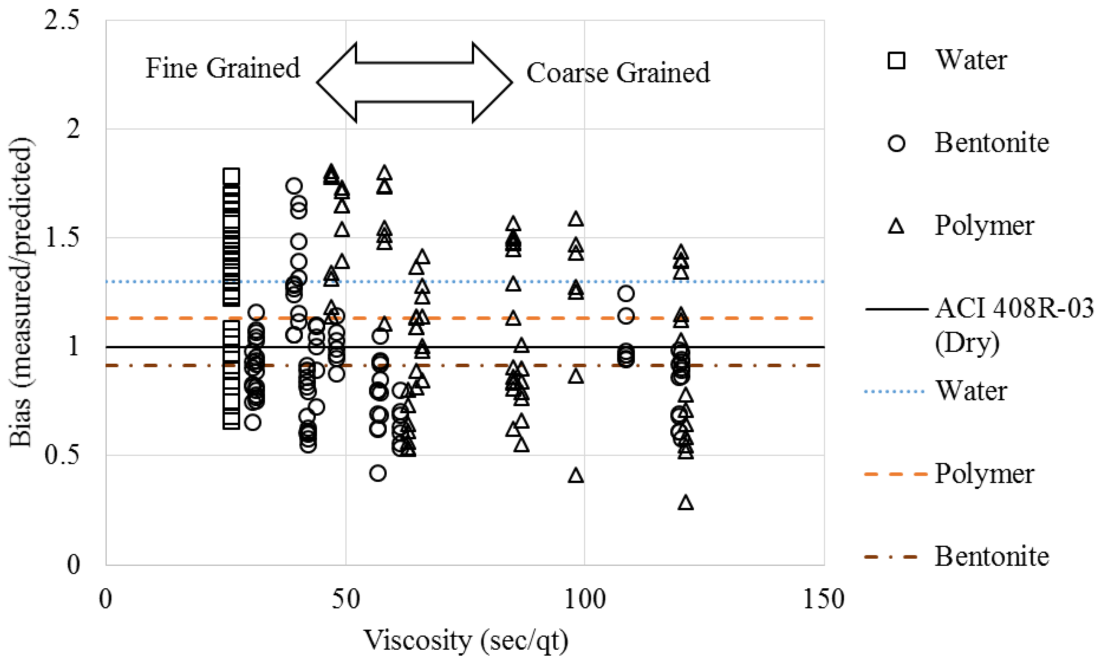


Figure 5.19 Bias versus slurry viscosity for ACI 408R-03.

After finding the bias, the standard deviation and coefficient of variation were determined for each slurry type. This information can be found in Table 5.11, the values for dry conditions were taken as those for ACI 318 and ACI 408R-03 recorded in ACI 408R-03 (ACI Committee 408, 2003).

Table 5.11 Mean bias, standard deviation, and CoV values for various conditions using ACI 318 and ACI 408R-03.

	Dry (ACI)		Water		Bentonite		Polymer	
ACI Eq.	318	408R-03	318	408R-03	318	408R-03	318	408R-03
Mean Bias ($\bar{r}$)	1.23	1.00	1.28	1.30	0.84	0.91	1.15	1.13
Standard Deviation	0.30	0.12	0.32	0.32	0.27	0.25	0.39	0.40
CoV (V_r)	0.24	0.12	0.25	0.25	0.32	0.27	0.34	0.35

Using the mean bias and standard deviation values shown in Table 5.11, log-normal probability density curves were generated for the two prediction methods (Figures 5.20 and 5.21). Note that as stated above a resistance factor of 1 was used in these equations, so there is no resistance factor effect seen in these probability density curves. A vertical line was placed at 1.0 to show the threshold, where above or equal to 1.0, the measured capacity is generally acceptable; however, below 1.0 the measured capacity is unacceptable.

Also noted in the legends of Figures 5.20 and 5.21 are the current failure ratios for each casting condition. In order to determine these failure ratios, Monte Carlo simulations were conducted for each casting condition (water, bentonite, polymer, and dry). Failure ratios for ACI 318 ranged from 1:2.4 for bentonite slurry to 1:39.6 for natural slurry conditions, all of which can be seen in Figure 5.20. For ACI 408R-03 failure ratios ranged from 1:3.6 for bentonite slurry to 1:45.3 for natural slurry conditions, Figure 5.21 states the others. It should be noted that the failure ratio is not assigned on the basis of the fraction of bias below 1.0, but rather where the random variations in load and strength result in a strength/load ratio below 1.0.

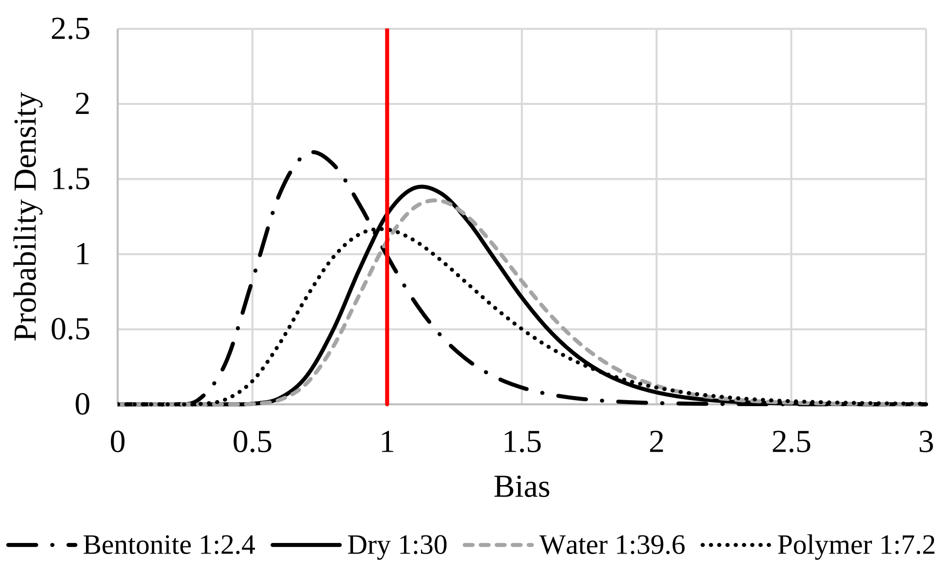


Figure 5.20 Lognormal probability density curve for ACI 318.

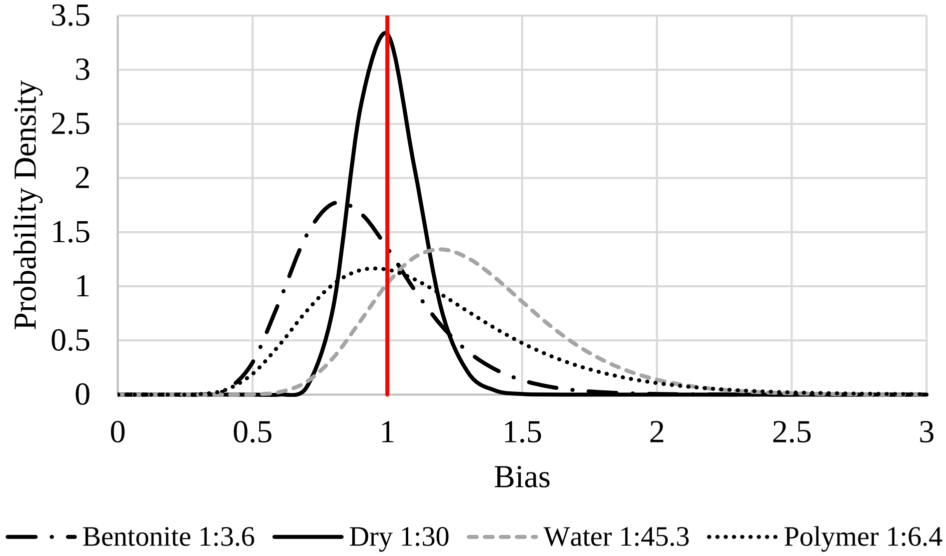


Figure 5.21 Lognormal probability density curve for ACI 408R-03.

5.8 Resistance Factor Generation

5.8.1 ACI 318-14 and ACI 408R-03

Using the information in Table 5.11, the procedure outlined previously for the calculation of bond resistance factors by Darwin et al. (1998) was followed. All parameters, aside from the mean bias and coefficient of variation (found in Table 5.11), used for Equations 5.11 to 5.13 are shown in Table 5.12. Note that the dead-load (DL)-to-live-load (LL) ratio has three differently colored values. The different colors correspond to the random loading variable ($\bar{q}$) and coefficient of variation for the random loading variable, as their value is dependent on the DL/LL ratio.

Table 5.12 Parameters used in equations 11, 12, and 13.

Load Factor, DL	1.2
Load Factor, LL	1.6
DL/LL Ratio	0.67, 1, 2
Reliability Index	3.5
Load Bias, DL	1.03
Load Bias, LL	0.975
Load CoV, DL	0.093
Load CoV, LL	0.25
$\bar{q}$	0.693, 0.716, 0.759
$V_{\varphi q}$	0.152, 0.131, 0.102

Resistance factors were first calculated for all DL/LL ratios, then the worst case was selected, which happened to always correspond to a DL/LL ratio of 2, aside from the dry ACI 408R-03 condition where a DL/LL ratio of 0.67 controlled. The calculated bond resistance factors are displayed in Table 5.13.

Table 5.13 Resistance factors for all casting conditions.

Slurry Type	Water	Polymer	Bentonite	Dry
ACI 318	0.666	0.435	0.341	0.65
ACI 408R-03	0.677	0.418	0.441	0.74

5.8.2 **AASHTO**

In terms of predicting capacity, only ACI 318-14 and ACI 408R-03 are examined to this point of the report. However, AASHTO load factors and other parameters are also examined given that they differ from ACI values. Thus, this process was performed using corresponding AASHTO parameters as well. Parameters used for the calculation of AASHTO resistance factors are found in Table 4.5. These values were used in the same equation(s) used to calculate the ACI resistance factors, Equations 11 to 13. Again varying DL/LL ratios were calculated and the worst case was chosen. The predominant worst case, exactly as above, was a DL/LL ratio of 2, aside from dry conditions for ACI 408R-03, where the worst case was again a DL/LL ratio of 0.67. It should be noted that the AASHTO development length equation is equivalent to the one found in ACI 318-14. Thus, the predicted capacities are equivalent to the values calculated for ACI 318-14 above.

Table 5.14 AASHTO parameters.

Load Factor, DL	1.25
Load Factor, LL	1.75
DL/LL Ratio	2
Reliability Index	3.5
Load Bias, DL	1.05
Load Bias, LL	1.15
Load CoV, DL	0.1
Load CoV, LL	0.2

While the ACI and AASHTO load parameters vary rather significantly from one another, the results are strikingly similar. Table 5.15 displays the bond resistance factor calculated using AASHTO variables. The only difference that can be noted in this table relates to ACI 408R-03 where the bond resistance factor increases from 0.74 to 0.75 using AASHTO factors.

Table 5.15 AASHTO bond resistance factors, ACI included for comparison purposes.

Parameters	ACI	Water	Polymer	Bentonite	Dry
AASHTO	318	0.667	0.435	0.341	0.65
	408R-03	0.677	0.417	0.441	0.75
ACI	318	0.666	0.435	0.341	0.65
	408R-03	0.677	0.418	0.441	0.74

5.9 Resistance Factor Application

After calculating the bond resistance factors, they were applied to Equations 5.18 and 5.28 and the predicted pullout capacities were recalculated resulting in new bias values. New probability density curves were then generated, which incorporated the new bond resistance factors, and are shown in Figures 5.22 and 5.23. Monte Carlo simulations were performed once more to confirm the bond resistance factors level of reliability. New failure ratios are noted in their respective figures, all exceeding the intended acceptable level of a 3.5 reliability index, which corresponds to a failure ratio 1:4149.

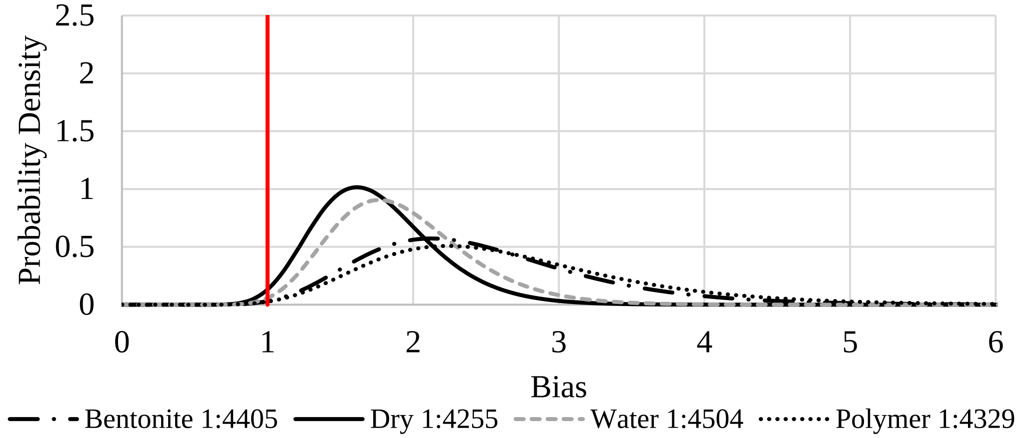


Figure 5.22 Lognormal probability density for ACI 318 after applying resistance factors.

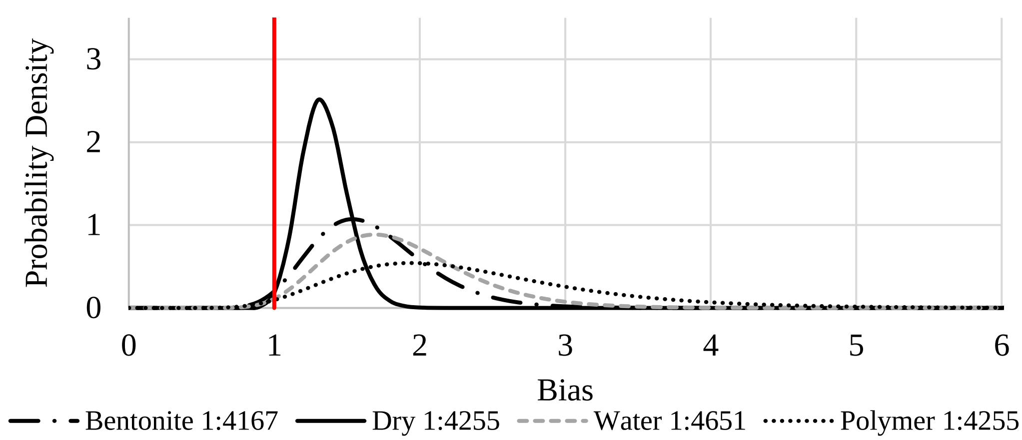


Figure 5.23 Lognormal probability density for ACI 408R-03 after applying resistance factors.

5.10 Splitting Failure Limitation

The analysis presented above suggests that the development length equations currently have unacceptable failure ratios, and that these ratios can be decreased to an acceptable level of reliability through the use of resistance factors. However, during the above analysis, no limitation was placed on the term $(c_b + K_{tr})/d_b$, which is limited by ACI 318-14 to a value of 2.5 or less and by ACI 408R-03 to 4.0 or less "to prevent pullout failures" (ACI Committee 318, 2014; ACI Committee 408, 2003).

5.10.1 **ACI 318-14**

To account for this limitation, parts of the analysis completed above were repeated for ACI 318-14 with the 2.5 limitation placed on this term. Table 5.16 shows the recalculated bias, standard deviation, and coefficient of variation for all casting conditions aside from dry. As the data used for dry conditions was not available, dry conditions were not included in this analysis. Notice the bias values significantly increase using this limitation.

Table 5.16 Mean bias, standard deviation, and CoV values for various conditions using ACI 318-14 and the 2.5 limitation.

	Water	Bentonite	Polymer
Mean Bias ($\bar{r}$)	3.46	2.26	3.10
Standard Deviation	0.85	0.72	1.06
CoV (V_r)	0.25	0.32	0.34

Monte Carlo simulations were again performed to see the failure ratio prior to a resistance factor. Water casting conditions showed zero failures in one million trials, indicating that the 2.5 limit is more conservative than the target reliability index of 3.5 requires it to be. Bentonite and polymer slurries gave a failure ratios of 1:1642 and 1:24390, respectively. While the polymer failure ratio is acceptable based on the target reliability index of 3.5, the bentonite ratio is not.

Using the equation for the level of reliability and the values from Table 4.3, new resistance factors were generated (Table 5.17). The calculated resistance factor for water indicates how conservative the predicted capacity already is. Monte Carlo simulations were completed for polymer and bentonite slurries, confirming that the use of the calculated resistance factors lead to an acceptable level of reliability. From the calculated resistance factors it can be noted that water casting conditions are 1.795 times more conservative than required and polymers are 1.171 times more conservative.

Table 5.17 Resistance factors using 2.5 limitation.

Slurry Type	Water	Polymer	Bentonite
ACI 318-14, 2.5 limitation	1.795	1.171	0.919

5.10.2 **ACI 408R-03**

As stated above, for ACI 408R-03, the $(c_b + K_{tr})/d_b$ term is limited to 4.0 or less (ACI Committee 408, 2003). In ACI 318-14, this term is fairly straightforward, where c_b "*is a factor that represents the least of the side cover, the concrete cover to the bar or wire, or one-half the center-to-center spacing of the bars or wires*" (ACI Committee 318, 2014). Just as simple, $K_{tr} = 40A_{tr}/sn$ and d_b is simply the diameter of the bar (ACI Committee 318, 2014). Considering this term is kept in its $(c_b + K_{tr})/d_b$ form in the bond strength equation, limiting this value is fairly easy. ACI 408R-03 provides more of a challenge. Aside from d_b, which is the same as defined previously, c and K_{tr} are now defined as:

$$c = (c_{min} + 0.5d_b)(0.1\frac{c_{max}}{c_{min}} + 0.9)$$

$$K_{tr} = \frac{0.52t_r t_d A_{tr}}{sn}\sqrt{f_c'}$$

Note ACI 318-14 references c as c_b. This makes the limiting $(c + K_{tr})/d_b$ equation:

$$\frac{1}{d_b}\left[(c_{min} + 0.5d_b)\left(0.1\frac{c_{max}}{c_{min}} + 0.9\right) + \frac{0.52t_r t_d A_{tr}}{sn}\sqrt{f_c'}\right] \leq 4.0 \quad \text{(ACI Committee 408, 2003)}$$

In order to limit this term for bond strength the following manipulation of Equation 5.18 was applied:

$$T_b = (f_c')^{1/4}\left[[59.9l_d(c_m + 0.5d_b) + 2400A_b]\left(0.1\frac{c_M}{c_m} + 0.9\right) + \left(30.88t_r t_d\frac{NA_{tr}}{n} + 3\right)\sqrt{f_c'}\right]$$
$$= (f_c')^{1/4}\left[59.9l_d(c_m + 0.5d_b)\left(0.1\frac{c_M}{c_m} + 0.9\right) + 2400A_b\left(0.1\frac{c_M}{c_m} + 0.9\right) + 30.88t_r t_d\frac{NA_{tr}}{n}\sqrt{f_c'} + 3\sqrt{f_c'}\right]$$

Substituting: $N = l_d/s$, $c = (c_{min} + 0.5d_b)(0.1\frac{c_{max}}{c_{min}} + 0.9)$:

$$= (f_c')^{1/4}\left[59.9l_d(c) + 2400A_b\left(0.1\frac{c_M}{c_m} + 0.9\right) + 59.9\left(0.52t_r t_d\frac{l_d A_{tr}}{sn}\sqrt{f_c'}\right) + 3\sqrt{f_c'}\right]$$

Combining like terms and substituting in K_{tr} yields:

$$T_b = (f_c')^{1/4}\left[59.9l_d(c + K_{tr}) + 2400A_b\left(0.1\frac{c_M}{c_m} + 0.9\right) + 3\sqrt{f_c'}\right]$$

Last both sides must be divided by d_b:

$$\frac{T_b}{d_b} = \frac{(f_c')^{1/4}}{d_b}\left[59.9l_d(c + K_{tr}) + 2400A_b\left(0.1\frac{c_M}{c_m} + 0.9\right) + 3\sqrt{f_c'}\right]$$

This finally forms Equation 5.30:

$$T_b = d_b(f_c')^{1/4}\left[\left(59.9l_d\left(\frac{c+K_{tr}}{d_b}\right)\right) + \left(\frac{2400A_b*\left(0.1\frac{c_{max}}{c_{min}}+0.9\right)}{d_b}\right) + \left(\frac{3\sqrt{f_c'}}{d_b}\right)\right] \qquad \text{(Eq. 5.30)}$$

After determining Equation 5.30, the 4.0 limitation was applied and the same analysis was performed once more. The mean bias, standard deviation, and coefficient of variation for water, bentonite, and polymer are noted in Table 5.18. Immediately it can be seen that the mean bias values are not as inflated as those seen from ACI 318-14.

Table 5.18 Mean bias, standard deviation, and CoV values for various conditions using ACI 408R-03 and the 4.0 limitation.

	Water	Bentonite	Polymer
Mean Bias ($\bar{r}$)	1.35	0.94	1.18
Standard Deviation	0.33	0.25	0.41
CoV (V_r)	0.24	0.27	0.35

First, the level of reliability method to find resistance factors, again using the parameters from Table 5.12, was applied. Resistance factors found are shown in Table 5.19. The calculated resistance factors using the 4.0 limitation are not nearly as conservative as what is observed with the ACI 318-14 2.5 limitation.

Table 5.19 Resistance factors using 4.0 limitation.

Slurry Type	Water	Bentonite	Polymer
ACI 408R-03, 4.0 limitation	0.707	0.449	0.434

Monte Carlo simulations were run pre- and post- resistance factor to generate the number of failures. Results confirmed that the limiting factor for ACI 408R-03 makes the equation only mildly more conservative, as water showed a failure ratio of 1 in 28, bentonite 1 in 3.8, and polymer 1 in 3.8. Using the calculated resistance factors in Table 5.19 yields improved failure ratios of 1 in 4484, 1 in 4673, and 1 in 5291 for water, bentonite, and polymer, respectively.

5.11 **Bentonite Viscosity Ranges**

In the above analysis, all bentonite samples were averaged. In this portion, the samples were divided into their respective viscosity categories. The categories were 30 to 40 sec/qt, 40 to 50 sec/qt, 50 to 70 sec/qt, and 90+ sec/qt. Viscosities were separated this way as FDOT only allows bentonite viscosities in the range of 30 to 40 sec/qt for drilling; however, other states have varied upper viscosity limits (FDOT, 2018). The last two categories were divided based on sample size and available data.

The analyses presented in the above sections were performed (the reliability index method and Monte Carlo simulations) for samples grouped by viscosity. Tables 5.20 and 5.21 display the mean bias, standard deviation, coefficient of variation, initial failure ratios, determined resistance factors, and final failure ratios for ACI 318-14 without and with the splitting failure limitation, respectively.

Table 5.20 Statistical information and results from analysis for ACI 318-14 with no limitation for bentonite viscosity ranges.

Bentonite Viscosity (sec/qt)	30 to 40	40 to 50	50 to 70	90+
Mean Bias ($\bar{r}$)	1.03	0.85	0.70	0.71
Standard Deviation	0.22	0.32	0.11	0.19
CoV (V_r)	0.21	0.38	0.16	0.27
Initial Failure Ratio	1 in 10	1 in 2.3	1 in 1.5	1 in 1.6
Resistance Factor (RF)	0.596	0.283	0.47	0.346
RF Failure Ratio	1 in 4219	1 in 4202	1 in 6098	1 in 4329

Table 5.21 Statistical information and results from analysis for ACI 318-14 with 2.5 limitation for bentonite viscosity ranges.

Bentonite Viscosity (sec/qt)	30 to 40	40 to 50	50 to 70	90+
Mean Bias ($\bar{r}$)	2.77	2.29	1.89	1.91
Standard Deviation	0.58	0.87	0.30	0.51
CoV (V_r)	0.21	0.38	0.16	0.27
Initial Failure Ratio	0 in 1,000,000	1 in 349	1 in 1,000,000	1 in 1,715
Resistance Factor (RF)	1.606	0.763	1.288	0.932
RF Failure Ratio	1 in 4,310	1 in 4,219	1 in 5,236	1 in 4,329

As expected, without the splitting failure limitation, all specimens need the use of a resistance factor to reach the desired reliability; however, it can be seen that bentonite viscosities of 30 to 40 sec/qt provide a resistance factor of almost double what was generated for overall viscosities, (0.341), thus notably performing better. When assessing the mean bias per viscosity grouping of Table 5.20 and comparing it with the generated value for all viscosities in Table 5.11, it can be seen that 30 to 40 sec/qt and 40 to 50 sec/qt fall above the mean bias for all viscosities (0.84), versus 50 to 70 sec/qt and 90+ sec/qt which are below. While it would seem fitting for the trend of better to worse to follow throughout, this is not the case. The primary reason behind this being the coefficient of variation. For viscosities from 40 to 50 sec/qt the calculated coefficient of variation was 0.38, which is higher than all of the others. This value indicates that this data is prone to more scatter, as also evident by the highest standard deviation of all groupings. Thus, when calculating a resistance factor or failure ratio this heavily impacts the performance. It should also be noted, that the sample size of this viscosity range is the highest, which can have an influence on this as well (more data can equal more scatter, but also be more realistic).

The same result is found when the 2.5 limitation is used, however this is much more critical. With a 2.5 limitation, the calculated resistance factor is still 0.763 for 40 to 50 sec/qt. The only other viscosity range not meeting the desired reliability index was 90 + sec/qt, which gave a resistance factor of 0.932. However, viscosity ranges 30 to 40 sec/qt and 50 to 70 sec/qt provided adequate or conservative resistance factors of 1.606 and 1.288, respectively. Note: computed resistance values greater than 1.0 are generally capped at 1.0.

To maintain consistency, the same analysis was performed for ACI 408R-03 conditions. All statistical information and results from the reliability index method and Monte Carlo simulations can be found in Tables 5.22 and 5.23, without and with the splitting failure limitation, respectively.

The same predicament occurs where the standard deviation and coefficient of variation are much higher for the bentonite viscosity range of 40 to 50 sec/qt; however, the difference is not as drastic for ACI 408R-03. Overall, the calculated resistance factor values seem to scatter above or below the generated value for all viscosities presented above; this applies for both with and without the limitation.

Table 5.22 Statistical information and results from analysis for ACI 408R-03 with no limitation for bentonite viscosity ranges.

Bentonite Viscosity (sec/qt)	30 to 40	40 to 50	50 to 70	90+
Mean Bias ($\bar{r}$)	1.00	0.97	0.74	0.89
Standard Deviation	0.23	0.29	0.15	0.17
CoV (V_r)	0.23	0.30	0.21	0.19
Initial Failure Ratio	1 in 6.9	1 in 4.1	1 in 1.8	1 in 4.4
Resistance Factor (RF)	0.544	0.420	0.432	0.553
RF Failure Ratio	1 in 4255	1 in 4237	1 in 4310	1 in 4831

Table 5.23 Statistical information and results from analysis for ACI 408R-03 with 4.0 limitation for bentonite viscosity ranges.

Bentonite Viscosity (sec/qt)	30 to 40	40 to 50	50 to 70	90+
Mean Bias ($\bar{r}$)	1.04	0.99	0.76	0.90
Standard Deviation	0.24	0.30	0.15	0.17
CoV (V_r)	0.23	0.30	0.20	0.19
Initial Failure Ratio	1 in 8.7	1 in 4.5	1 in 2.0	1 in 5.0
Resistance Factor (RF)	0.560	0.426	0.454	0.567
RF Failure Ratio	1 in 4255	1 in 4444	1 in 4630	1 in 4739

For this set of data, the sample count was 28, 35, 21, and 21, for ranges 30 to 40 sec/qt, 40 to 50 sec/qt, 50 to 70 sec/qt, and 90+ sec/qt, respectively. As an adequate sample size for analysis is typically taken as 30, only one category of this data fulfills this guideline.

5.12 Performance VariabilityBetween Polymers

During the course of the testing for this report, three polymers from different manufacturers were examined. As company names are confidential, they have been labeled 1, 2, and 3, consistent with previously stated polymers 1, 2, and 3 in Chapter 3. Currently polymer performance can vary heavily between manufacturers, as there can be many varying properties. The testing results presented here confirm the variability seen in the industry. By first considering ACI 318-14 with no limitation, all statistical data and failure ratios have been noted in Table 5.24. Looking primarily at the mean bias and coefficient of variation, each polymer manufacturer is slightly different. Polymer 1 seems to best represent the "average", polymer 2 has a similar mean bias, but has a coefficient of variation more than double that of polymer 1, and polymer 3 has a smaller coefficient of variation than polymer 1, and also has a much higher mean bias. From polymer manufacturer 1 to 3, the initial failure ratios are 1 in 7.8, 1 in 2.7, and 1 in 30303, respectively. The resistance factors calculated are 0.557, 0.232, and 1.068 for 1, 2, and 3, respectively, corresponding to massive performance gaps.

Table 5.24 Statistical information and results from analysis for ACI 318-14 with no limitation for polymers 1 to 3.

Polymer Manufacturer	1	2	3
Mean Bias ($\bar{r}$)	1.01	0.97	1.48
Standard Deviation	0.23	0.46	0.20
CoV (V_r)	0.23	0.48	0.14
Initial Failure Ratio	1 in 7.8	1 in 2.7	1 in 30303
Resistance Factor (RF)	0.557	0.232	1.068
RF Failure Ratio	1 in 4425	1 in 4545	1 in 5917

When analyzing this data included the splitting failure limitation (Table 5.25), the same general trends are noted between mean bias and coefficient of variation. This critical aspect seen in Table 5.25 is the initial failure ratio. While polymers 1 and 3 yield zero failures in one million trials, polymer 2 shows a 1 in 155 failure ratio. Thus, the 2.5 limitation does not make this product conservative enough to achieve a 3.5 reliability index on its own.

Table 5.25 Statistical information and results from analysis for ACI 318-14 with a 2.5 limitation for polymers 1 to 3.

Polymer Manufacturer	1	2	3
Mean Bias ($\bar{r}$)	2.73	2.61	3.98
Standard Deviation	0.62	1.24	0.55
CoV (V_r)	0.23	0.48	0.14
Initial Failure Ratio	0 in 1,000,000	1 in 155	0 in 1,000,000
Resistance Factor (RF)	1.501	0.625	2.876
RF Failure Ratio	1 in 4587	1 in 4202	1 in 5780

For ACI 408R-03 the same analysis was again prepared, the results can be found in Tables 5.26 and 5.27 for values without and with a limitation, respectively. The results were very similar to ACI 318-14 where polymer 3 performed best, followed by polymer 1 and lastly polymer 2.

Table 5.26 Statistical information and results from analysis for ACI 408R-03 without limitation for polymers 1 to 3.

Polymer Manufacturer	1	2	3
Mean Bias ($\bar{r}$)	0.96	0.97	1.47
Standard Deviation	0.22	0.46	0.20
CoV (V_r)	0.22	0.48	0.14
Initial Failure Ratio	1 in 5.8	1 in 2.7	1 in 18182
Resistance Factor (RF)	0.535	0.233	1.062
RF Failure Ratio	1 in 4854	1 in 4566	1 in 6494

Table 5.27 Statistical information and results from analysis for ACI 408R-03 with 4.0 limitation for polymers 1 to 3.

Polymer Manufacturer	1	2	3
Mean Bias ($\bar{r}$)	1.00	1.02	1.54
Standard Deviation	0.22	0.48	0.21
CoV (V_r)	0.22	0.48	0.14
Initial Failure Ratio	1 in 7.4	1 in 3.0	1 in 10,0000
Resistance Factor (RF)	0.556	0.243	1.111
RF Failure Ratio	1 in 4608	1 in 4149	1 in 5682

5.13 **Attapulgite**

While initially the intention was to use attapulgite data to generate a mineral slurry factor, this data did not follow the trends noted with bentonite slurry. A data analysis has been performed (Table 5.28); however, considering the sample size is only 14, more testing should be performed to have a better understanding of attapulgite's performance. Based on the information provided in Table 5.28, it can be seen that attapulgite performs quite well. When the ACI 318-14 2.5 limitation is applied, the level of conservativeness is above that calculated for water, which could be misleading. One possible element that may change greatly with a large sample size is the coefficient of variation, determined for all cases to be a value of 0.16, which is a relatively small value. As more data is accumulated, there may be more scatter, possibly raising the coefficient of variation and lowering the calculated resistance factors.

Table 5.28 Statistical information and results from analysis for ACI 318-14 and ACI 408R-03 for attapulgite.

	ACI 318-14		ACI 408R-03	
Limitation	None	2.5	None	4.0
Mean Bias ($\bar{r}$)	1.21	3.25	1.21	1.26
Standard Deviation	0.19	0.51	0.19	0.20
CoV (V_r)	0.16	0.16	0.16	0.16
Initial Failure Ratio	1 in 180	0 in 1,000,000	1 in 179	1 in 370
Resistance Factor (RF)	0.827	2.22	0.827	0.865
RF Failure Ratio	1 in 5618	1 in 5618	1 in 5650	1 in 5263

5.14 **Self-Consolidating Concrete**

As with attapulgite, the sample sizes for self-consolidating concrete are only 12 and 19 for water and bentonite, respectively, and therefore this analysis should be viewed as preliminary. While the sample size is small, it provides insights to a potential issue between bentonite slurry and SCC. Note the viscosity range for the bentonite slurry evaluated is 31 to 42 sec/qt. With regards to ACI 318-14 Tables 5.29 and 5.30 provide data without and with the limitation. While in both circumstances SCC water specimens seem to outperform Class IV water specimens, SCC bentonite is underperforming. Under the 2.5 limitation, bentonite in SCC still provides a failure ratio of 1 in 42, generating a resistance factor of 0.56 to reach the desired 3.5 reliability index. SCC water however proves to be extremely conservative, generating a resistance factor of 2.151.

Table 5.29 Statistical information and results for ACI 318-14 without limitation for SCC.

Slurry	Water	Bentonite
Mean Bias ($\bar{r}$)	1.37	0.63
Standard Deviation	0.29	0.24
CoV (V_r)	0.21	0.38
Initial Failure Ratio	1 in 185	1 in 1.4
Resistance Factor (RF)	0.799	0.208
RF Failure Ratio	1 in 4237	1 in 4115

Table 5.30 Statistical information and results for ACI 318-14 with limitation for SCC.

Slurry	Water	Bentonite
Mean Bias ($\bar{r}$)	3.70	1.70
Standard Deviation	0.78	0.66
CoV (V_r)	0.21	0.38
Initial Failure Ratio	0 in 1,000,000	1 in 42
Resistance Factor (RF)	2.151	0.560
RF Failure Ratio	1 in 4405	1 in 4149

In terms of ACI 408R-03, Tables 5.31 and 5.32 provide the analysis data generated. The same general trends are seen, where SCC water out performed Class IV water-casting conditions and SCC bentonite underperforms, for both cases. Resistance factors generated without a limitation were 0.878 and 0.329 for water and bentonite and with a limitation were 0.933 and 0.333, respectively.

Table 5.31 Statistical information and results for ACI 408R-03 without limitation for SCC.

Slurry	Water	Bentonite
Mean Bias ($\bar{r}$)	1.41	0.82
Standard Deviation	0.27	0.27
CoV (V_r)	0.19	0.32
Initial Failure Ratio	1 in 575	1 in 2.2
Resistance Factor (RF)	0.878	0.329
RF Failure Ratio	1 in 5263	1 in 4329

Table 5.32 Statistical information and results for ACI 408R-03 with limitation for SCC.

Slurry	Water	Bentonite
Mean Bias ($\bar{r}$)	1.48	0.85
Standard Deviation	0.27	0.28
CoV (V_r)	0.19	0.33
Initial Failure Ratio	1 in 1453	1 in 2.4
Resistance Factor (RF)	0.933	0.333
RF Failure Ratio	1 in 5556	1 in 4167

5.15 **Chapter Summary**

Rebar bond and development length are key reinforcement design parameters. As engineering practice has developed and moved from ASD to LRFD design methods, resistance factors have become integral to the design process. This chapter detailed the procedure for testing rebar bond and then analyzed the resulting data using several empirical equations currently accepted as standard. Those results were then scrutinized using statistical methods with the final outcome being a recommended resistance factor for the presence of support fluid during concreting operations.

Chapter Six: Electrochemical Testing

Corrosion is the process through which metal deteriorates as it reacts with the environment. If left unprotected, metals in wet environments will uniformly corrode, meaning a chemical or electrochemical reaction is occurring over a large surface area. This reaction will continue to deteriorate the metal, thinning it to the point of failure. Though a monumentally destructive force, uniform corrosion is not a major industry concern because it is commonly preventable and characteristically predictable (Fontana, 1967).

Civil engineering structures are designed to resist corrosion. Exposed metals vulnerable to corrosion are often coated, painted or protected by some means. In the case of reinforced concrete, the concrete theoretically acts as a barrier between the reinforcing steel and the surrounding environment. Corrosion resistance in reinforced concrete is a designable parameter dictated by concrete quality and cover thickness. However, the presence of possible pathways leading directly to the reinforcing steel is of great importance and yet rarely addressed. This chapter focuses on the electrochemical properties of 52 large-scale, lab-cast drilled shaft specimens. Perhaps most important, all specimens were cast using the tremie placement method where some fluid type was displaced by the rising fluid concrete.

The testing was multifaceted: (1) establish electrical continuity of the reinforcement system, (2) conduct baseline half-cell potential measurements, and (3) conduct additional half-cell potential measurements after a set increment of time. This second set of tests allowed for the analysis of changes in half-cell potential over time.

6.1 Corrosion Rate Expressions

In the field of corrosion engineering, metals are often compared on the basis of their corrosion resistance. This comparison can only be made meaningful through quantification. There are several expressions used to quantify corrosion resistance, many of which simply describe the amount of material lost, or thinning, over a specific period of time. The expression mils per year (mpy) is the one most commonly used in engineering as it uses whole numbers (non decimals) and can be further applied to structural lifespan prediction. The formula is stated as follows:

$$mpy = \frac{534W}{\rho AT} \tag{6.1}$$

wherein:

	W	weight loss (mg)
	ρ	density of specimen (g/cm^3)
	A	area of specimen (in.^{2})
	T	exposure time (hr)
	534	unit conversion factor

6.2 Corrosion Lifespan Analysis

Corrosion is best understood using the basic model of an electrolytic cell. An electrolytic cell is a system involving two electrodes (an anode and a cathode) and two types of chemical reactions, one to supply electrons and one to consume them (Carino, 1999). Generally, these systems are built using two dissimilar metals in an electrolyte solution. In the case of reinforced concrete, heterogeneities in the surface of the steel and the variable nature of concrete means a single piece of reinforcing steel acts as both the anode and the cathode, creating a short- circuited electrolytic cell with corrosion occurring at the anode. In order for concrete to act as a sufficient electrolyte to initiate corrosion, the dissolved chloride ions in the pores of the paste must surpass the critical concentration conditions needed to destroy the passive coating on the steel. Without the introduction of environmental chlorides, this process is unlikely. As a result, the life expectancy and serviceability of a drilled shaft (or any reinforced concrete element) is dependent on several parameters focused on the idea of staving off the initiation of the corrosion process. These factors are: concrete quality, concrete cover, the surrounding conditions, and the ability of the embedded reinforcement to withstand aggressive environments. These parameters can be defined as:

C_s	Concentration of chloride ions at the concrete surface (environment)
x_{cover}	Concrete cover
D	Apparent diffusion coefficient (concrete quality)
C_T	Chloride threshold at which corrosion initiates (steel type dependent)

The corrosion process is destructive and as such, the amount of time between the construction of a structure and the initiation of corrosion can be directly correlated to the life expectancy of the structure. While some chloride ions can be inherently present in the moist concrete pore fluid, the concentration is rarely sufficient to initiate corrosion; thus, corrosion initiation time (t_i) is most commonly defined as the time period necessary for concentrated chloride ions from the concrete surface to diffuse through the cover region and reach the reinforcing steel. This diffusion time can be estimated using the parameters listed above (Sagüés, 2002; Mullins, et al., 2009). The corrosion initiation time is commonly computed using an error function wherein C_s, x_{cover}, D, and C_T are all inputs.

$$C_T = C_s(1 - erf\,\frac{x}{2\sqrt{Dt_i}}) \tag{6.2}$$

Of those parameters, only x_{cover} and D are designable elements. Where D is a by-product of the mix design, which for drilled shafts is somewhat constant. The cover thickness (x_{cover}), however is specified by the Florida Department of Transportation to be 6 inches for a drilled shaft with a 1.5-inch reinforcing steel offset tolerance, meaning 4.5-inches of cover is permissible. Assuming a worst-case scenario diffusion coefficient and a conservative chloride concentration, the resulting time to initial corrosion is more than 500 years. That being said, if the cover region is cracked or mattressing creases exist, the cover thickness goes to zero and corrosion can initiate immediately.

6.3 Anomalies and Corrosion Potential

Design lifespan computations assume a contiguous concrete cover. As noted in Chapter 2, field and laboratory observations have shown *mattressing* (laitance channel formation) in shaft specimens constructed in wet conditions, where concrete is tremie-placed and the support fluid is displaced by the heavier fluid concrete. *Mattressing* introduces the possibility of direct ground or sea water access to the reinforcing cage. This concept was demonstrated in a study conducted at the University of South Florida (Campbell, 2014). When a tank was attached to the surface of a drilled shaft test specimen and filled partially with water (Figure 6.1), the water began to leak out along the surface creases (Figure 6.2) and immediately poured out below the encased reinforcement (Figure 6.3). This shows any protection provided by the concrete has been negated. This in essence results in a zero cover thickness. Quantifying the effects of *mattressing* or other surface anomalies forms the basis for much of the efforts in this chapter.

Figure 6.1 Water filled tank attached to concrete surface

Figure 6.2 Water draining from the tank into the creases

Figure 6.3 Water leaking below reinforcement in the bottom (left) and the top (right) of the shaft

6.4 Establishing Electrical Continuity

Corrosion in reinforced concrete is electrochemical in nature. The corrosion process involves the flow of electrons from an anodic site to a cathodic site on the reinforcing steel system. Corrosion requires four basic elements: anode, cathode, electrolyte and metallic path. The anode is the site of the corrosion and constitutes the source of current flow. The cathode is corrosion free and receives the flow of current. The electrolyte is a medium capable of conducting electrical current. In reinforced concrete, the fluid-filled pores serve as the electrolyte. The metallic path is the connection between the anode and the cathode that allows the return of current. In the simplest terms, cathodic protection is the process of converting anodic sites to cathodic sites through an applied current. Establishing a metallic path, hereafter referred to as electrical continuity as per industry nomenclature, is essential to this process.

Many organizations have published cathodic protection installation guidelines stating that electrical continuity must exist. However, specifications regarding a procedure to establish continuity are vague, varied, or non-existent. Literature suggests continuity guidelines roughly fall into two general categories: undefined and poorly defined. Undefined guidelines state continuity is required without referencing a procedure (Sagues, 1995; NACE, 2007; NACE, 2016). Poorly defined guidelines require continuity be established through the testing of electric potential (SIKA, 2010; DTI, 1981; Kentucky, 2011; Clear, 1993). This test makes voltage measurements between two rebar in an effort to assess the electrical connectivity and where less than a 1mV threshold is used to delineate when connectivity exists (greater than 1mV it does not). Mutual resistance is a similar test but where a 1-ohm threshold is used.

The Strategic Highway Research Program published the *New Cathodic Protection Installation* guide (Clear, 1993), which references an AASHTO standard that was still in draft form (AASHTO, 1994). The referenced specification was mislabeled by Clear as AASHTO TF29-650.37 as it was only a draft and an incomplete document. Today it can only be found in AASHTO TF29-650.30.15 from the published Task Force 29 report, Guide to Specifications for Cathodic Protection of Concrete Bridge Decks. This volume, currently out of print, established the following requirements for electrical continuity: "Electrical continuity exists between reinforcing bars or between reinforcing bars and other metal items when the millivolt difference between them is no more than 1.0 mV, the DC resistance is less than 1 ohm and the DC resistance measured in the forward and reverse directions does not exceed 1 ohm," (AASHTO, 1994). Though Clear states the AASHTO procedure was utilized to establish electrical continuity, no indication was given where resistance measurements were taken, and relied on millivolt data (not resistance) to support assertions of continuity. Nevertheless, Clear (1993) may be the only work that cites the now-out-of-print AASHTO recommended procedure.

This inconsistency in specifications creates uncertainty regarding a satisfactory practice. Further, few procedures have been established, and there are no justifications given for a particular practice (e.g. the rationale for less than 1mV potential difference). A previous study conducted at the University of South Florida (Mobley, 2017) consisted of a series of experiments performed to determine the statistical validity of methods used to establish electrical continuity and provide justification for implementation of a common practice. A brief summary of the experimental process and results is herein presented.

6.4.1 Experimental Procedure

The 2017 study conducted mutual potential and mutual resistance tests on all vertical reinforcement bars for each of twenty-three drilled shaft specimens. Prior to testing, the exposed end of each vertical rebar was drilled, tapped, and a stainless-steel screw was installed to establish a satisfactory electrical connection (Figure 6.4). The seven vertical rebar on each shaft were labeled, and testing was completed between all bar combinations (21 combinations per shaft).

Figure 6.4 Stainless steel connection

When the mutual potential is graphed against the mutual resistance, the data is banded along the zero-millivolt potential line horizontally, and above 100-Ω the data is banded vertically. The data points scattered along the potential axis between zero and five are indicative of a well-connected system, as this reading reflects a negligible potential difference. The data points above five millivolts that are also above 100 Ω would generally indicate a poorly connected system as they exceed the inherent resistance between two pieces of reinforcing steel and exhibit a loss in potential across the system. The points of particular interest are the points positioned at the base of the vertical band in the data (Figure 6.5). These points have a resistance over 100-Ω but show negligible loss of potential.

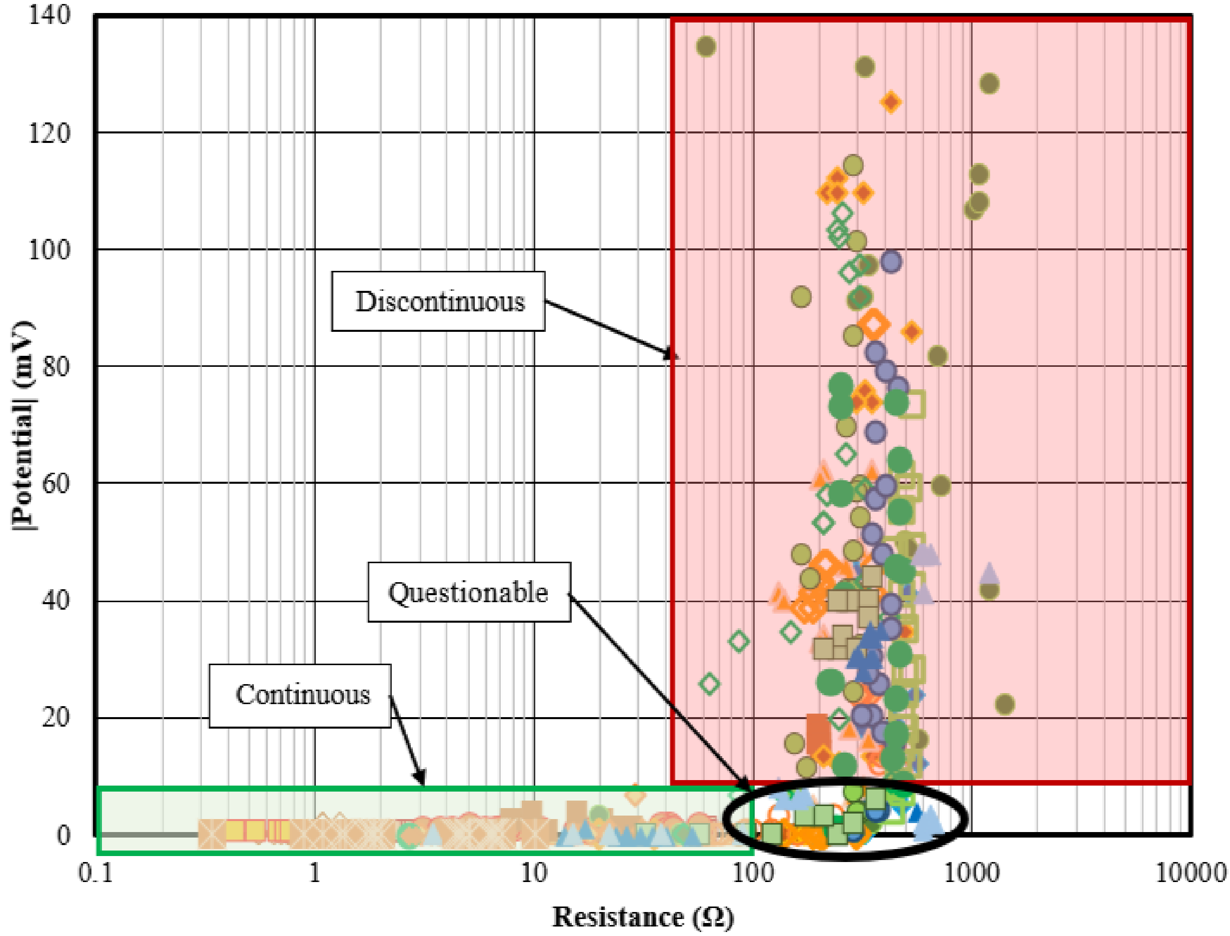


Figure 6.5 Mutual potential vs.. mutual resistance

In Figure 6.5, the area in red indicates a discontinuous rebar system, meaning that the potential value is higher than expected. The area shown in green contains readings with low potential values, and resistance values ranging from very low to less than about 100 ohms, indicating a continuous rebar system. The area in question is circled and contains the values with high resistances and low potentials.

The initial assumption that these high resistance, low potential points were statistical scatter was disproven when the sample data distribution for all points over 100 Ω was plotted against the normal standard distribution curve for the same data range (Figure 6.6). The normal distribution curve, created from a set of 300 randomly generated numbers, shows a 5% occurrence of points within the -5mV to 5mV range.

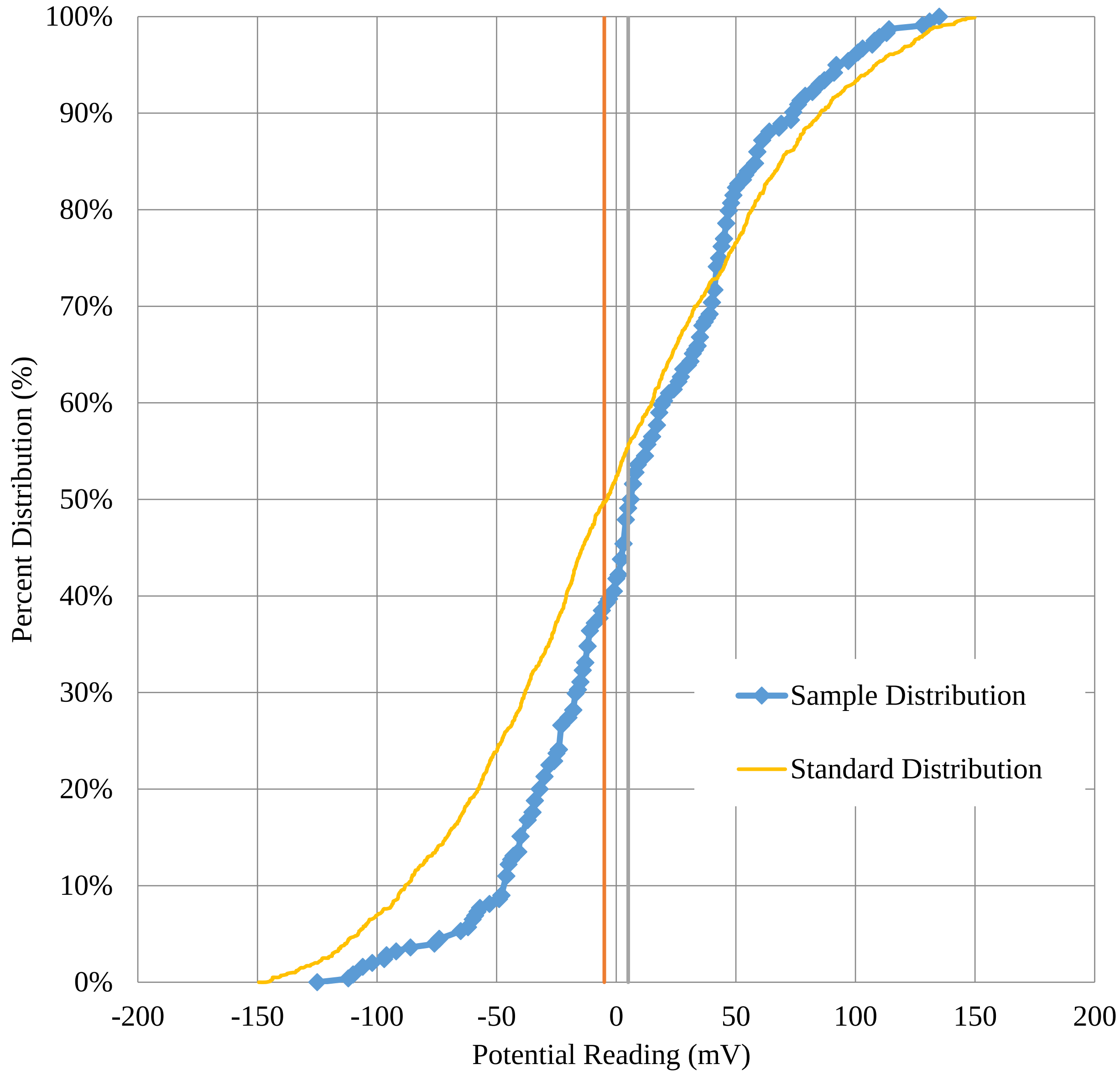


Figure 6.6 Potential distribution for measurements with resistances greater than 100 Ω

The sample data distribution shows the actual percentage of points in that range is 10% (Figure 6.7). This is twice the expected distribution meaning that there is a 50% chance the readings are valid and constitute connectivity and a 50% chance the readings are erroneous scatter and signify a discontinuous system. Without both the mutual potential and mutual resistance data it would be difficult to accurately diagnose the system. For that reason, the present findings support the use of both mutual potential and mutual resistance when establishing electrical continuity.

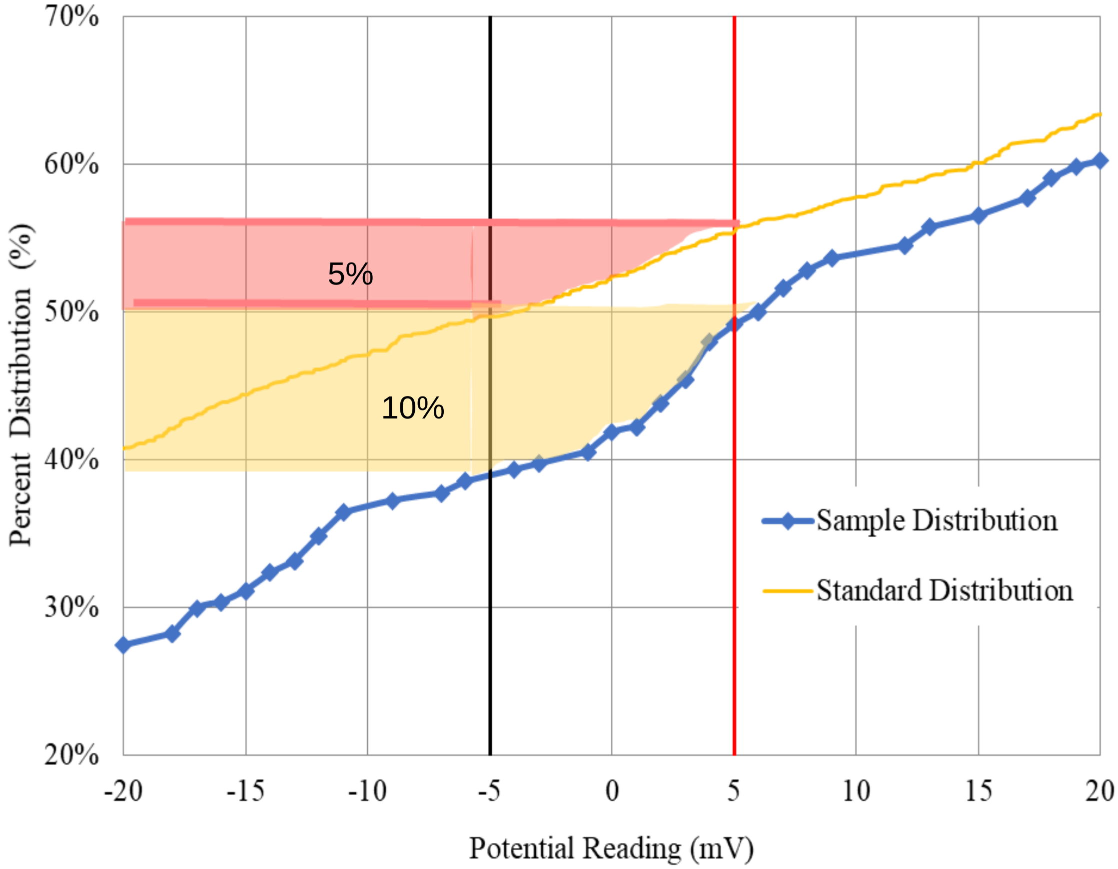


Figure 6.7 Statistical importance for measurements with resistances greater than 100 Ω

6.5 Multipoint Surface Mapping

Half-cell potential measurements can be an effective indicator of active corrosion within a reinforced concrete structure. This is performed by measuring the relative voltage potential between the reinforcing steel and a copper-copper sulfate electrode in contact with the concrete surface several inches away from the reinforcing steel. The original 23 specimens from the 2017 study were augmented in this study with an additional 33 specimens cast with different polymer support fluids along with mineral, water, and attapulgite support fluid. Surface potential measurements of the original 2017 specimens were again taken to show the effects of time on these measurements.

The half-cell potential of each shaft specimen was mapped evenly over the surface using a prescribed grid. A grid template was made out of a single piece of 21-inch by 27-inch rubberized plastic sheeting. A sharpened 1.5-inch diameter pipe was used to punch holes through the plastic in rows with a 3-inch center-to-center spacing in both directions (Figure 6.8). This resulted in 80 measurement locations for each shaft. Half-cell potential testing was then conducted per ASTM C876-09: *Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete*, using a copper-copper sulfate reference electrode and a standard multi-meter.

Figure 6.8 Completed template

Testing was conducted with the multimeter on the 2000 mV setting. The negative (COM) port was connected to a copper-copper sulfate reference electrode and the positive terminal was connected to the reinforcing steel via an alligator clip attached to a previously established secure electrical connection, the reinforcing steel having been interconnected with a stainless steel wire to ensure connectivity. In order to maintain an electrical junction between the porous tip of the reference electrode and the concrete surface, a wet sponge was wrapped around the tip of the reference electrode. Additionally, the concrete surface was soaked for a minimum of 24 hours prior to testing. This step ensured minimization of fluctuations in test measurements. Once saturated, measurements were taken at all 80 points in the test template grid. The readings were recorded to the nearest millivolt. See Appendix D for a sample data collection form. These tests were repeated after several months to analyze the effect of prolonged environmental exposure.

6.6 Results

The data from Shaft 1 is shown in Table 6.1 as an example; the complete data sets for two tests on all shafts are included in Appendix E.

Table 6.1 Sample half-cell potential data collected from shaft 1 (mV)

Circumferential position (in.)	Vertical Position (in) Bottom (0 in.) to Top (24 in.)							
	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0	-266	-289	-333	-337	-313	-298	-296	-289
3	-279	-292	-316	-317	-311	-301	-300	-293
6	-290	-302	-312	-312	-308	-306	-307	-303
9	-290	-308	-315	-312	-309	-310	-308	-307
12	-307	-316	-315	-318	-317	-314	-311	-302
15	-309	-317	-319	-325	-324	-322	-320	-311
18	-314	-323	-324	-332	-330	-328	-327	-320
21	-323	-330	-330	-338	-338	-342	-331	-322
24	-327	-330	-338	-344	-345	-348	-337	-329
27	-328	-333	-338	-344	-347	-349	-338	-337

All of the data was also graphed topographically using three-dimensional mapping software. Using a color-coding system and standardized contour spacing, the topographic surface maps illustrate the corrosion potential of each shaft. Lighter colors denote low corrosion probability; darker colors high. Figures 6.9 to 6.11 show examples of the range of variation in results. Complete results can be found along with the data tables for each test in Appendix E.

Figure 6.9 Shaft 6, water, test 1

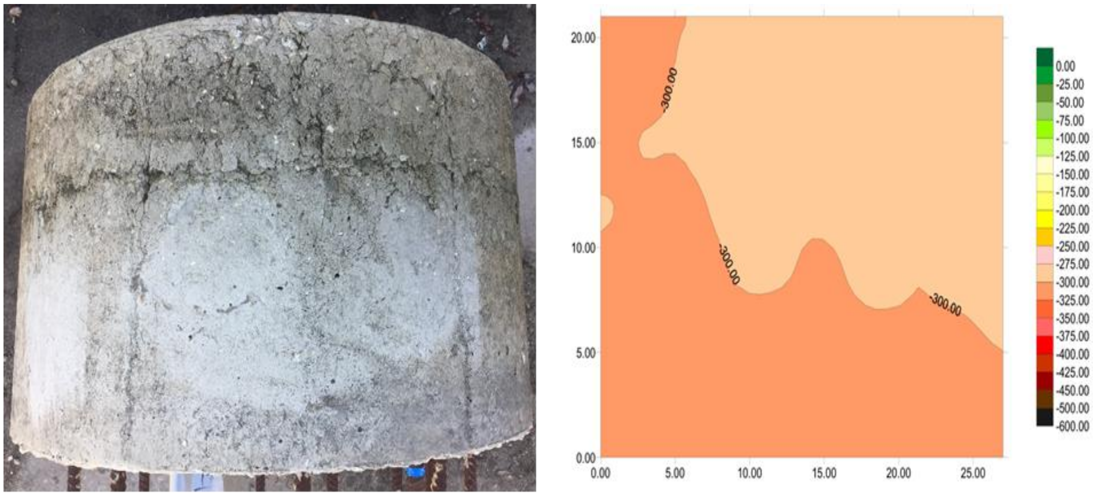


Figure 6.10 Shaft 17, polymer, test 1

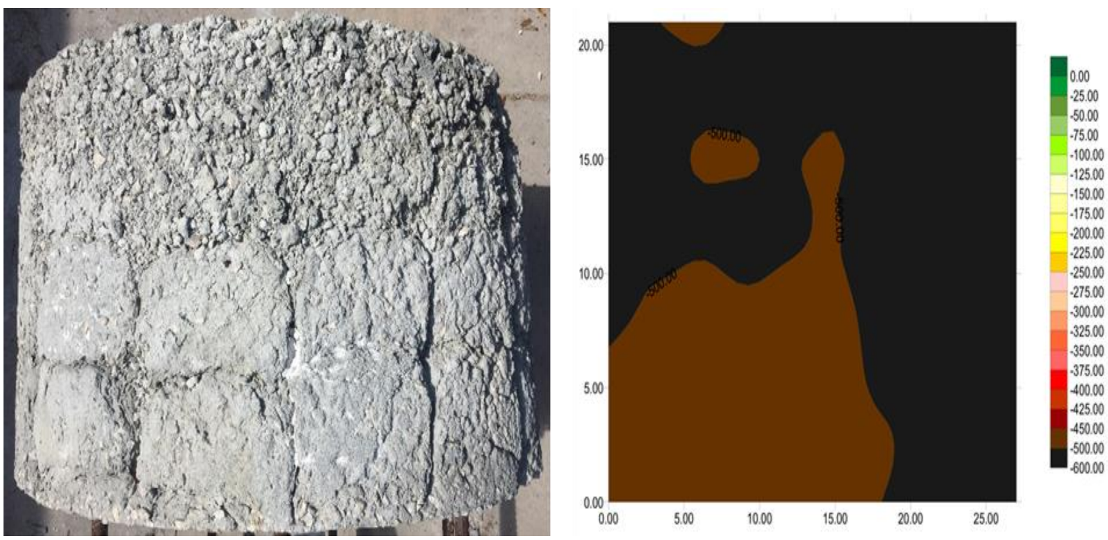


Figure 6.11 Shaft 21, bentonite, test 1

The 80 values collected during each test set were plotted on a standard distribution (Figure 6.12) using a rank and percentage analysis. The median (potential at 50% ranking) or the E_{50} value was taken as a single point representative of each test shaft for comparative plotting purposes. This is the preferred industry approach for such evaluations. The minimum value for each test set (E_{min}) was also tabulated. The E_{min} value represents the worst case for each data set and is used as such herein. ASTM C876-09 states a potential reading below -350mV indicates a high probability of corrosion. For the purpose of this work, corrosion potential is being used as an indicator of cover concrete quality, therefore the -350mV threshold has been applied as the distinguishing line between effective and ineffective concrete cover. The Figure 6.12 data shows the entire shaft to not be corroding.

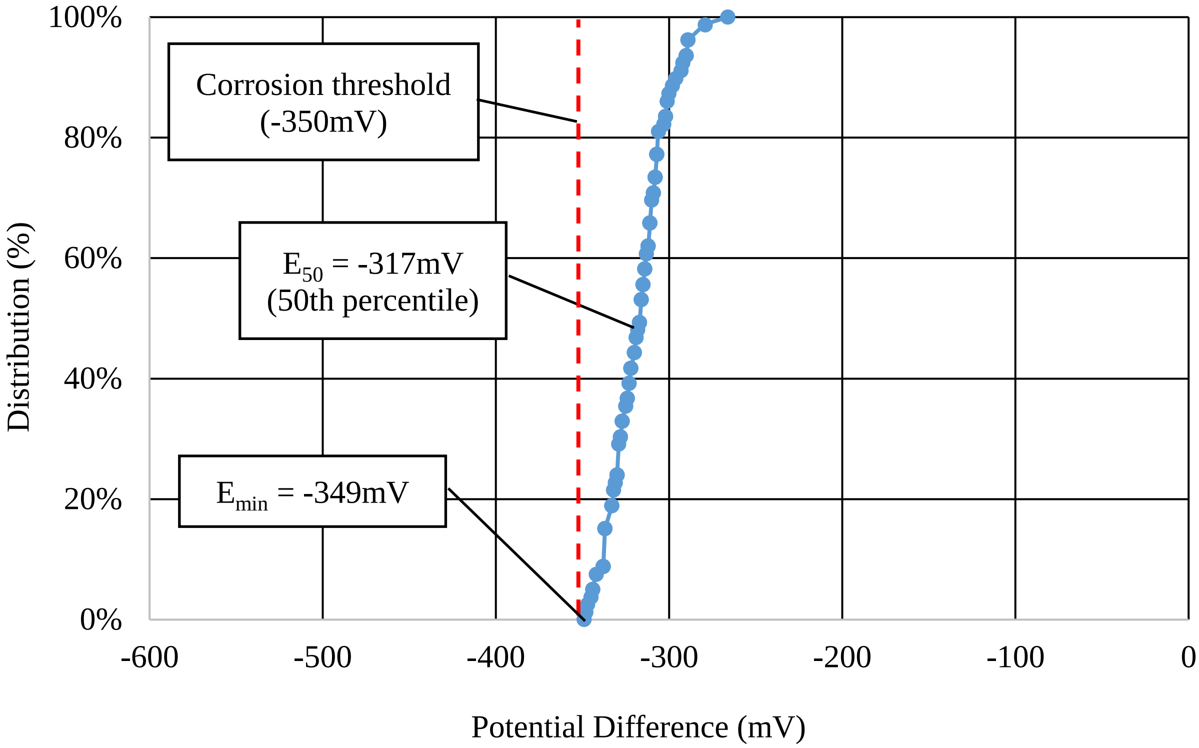


Figure 6.12 Half-cell potential mapping data distribution

E_{50} potential data for all shafts ranged from -571mV to -155mV with a standard deviation of 99mV. A total of 32% of the test shafts had an E_{50} potential below -350mV, and all save three of that 32% were constructed using mineral support fluid. The E_{50}, and E_{min} values for each test can be found in Table 6.2. The shafts are numbered consecutively by date of construction with the age of the specimen at the time of each test given in years. Additionally, the support fluid type is included wherein P indicates polymer, B indicates bentonite, W indicates water, and A indicates attapulgite mineral support fluid. Values more negative than the -350 mV threshold are highlighted.

Table 6.2 Summary of multipoint grid testing results

			Test 1			Test 2		
Shaft #	Slurry	Mix	Age (yrs)	E_{50}(mV)	E_{min} (mV)	Age (yrs)	E_{50}(mV)	E_{min} (mV)
1	B	4KDS	3.63	-317	-350	6.15	-390	-422
2	B	4KDS	3.64	-449	-492	6.11	-384.5	-449
3	B	4KDS	3.43	-373	-396	5.93	-519	-569
4	B	4KDS	3.44	-443	-474	5.93	-520	-553
5	B	4KDS	3.43	-447	-494	5.94	-498	-538
6	W	4KDS	3.43	-155	-176	5.96	-209	-246
7	B	4KDS	3.43	-372	-480	5.93	-380	-403
8	B	4KDS	3.30	-225	-352	5.83	-358	-477
9	B	4KDS	3.32	-383	-430	5.85	-421	-442
11	P	4KDS	3.32	-285	-366	5.85	-317	-359
12	P	4KDS	3.33	-190	-226	5.83	-247	-337
13	B	4KDS	3.06	-289	-351	5.59	-335	-360
14	B	4KDS	3.08	-282	-371	5.57	-348	-402
15	B	4KDS	3.06	-335	-362	5.59	-399	-450
16	P	4KDS	3.07	-279	-298	5.56	-465	-494
17	P	4KDS	3.07	-300	-322	5.59	-347	-384
18	W	4KDS	3.08	-293	-307	5.56	-337	-379
19	P	4KDS	1.41	-243	-275	3.98	-269	-318
20	P	4KDS	1.41	-242	-259	3.93	-470	-509
21	B	4KDS	1.41	-508	-596	3.98	-490	-518
22	W	4KDS	1.43	-250	-281	3.97	-264	-302
23	W	SCC	1.43	-258	-283	3.95	-368	-454
24	B	SCC	1.46	-425	-469	3.98	-450	-475
25	A	SCC	0.37	-415	-472	1.68	-393	-476
26	W	SCC	0.20	-326	-403	1.70	-334	-408
27	B	SCC	0.35	-246.5	-352	1.66	-285	-357
28	P	SCC	0.35	-286.5	-347	1.66	-267.5	-349
29	P	SCC	0.35	-410.5	-551	1.66	-436.5	-563
30	B	SCC	0.35	-410	-500	1.70	-493.5	-611
31	P	4KDS	0.15	-303.5	-321	1.46	-292	-316
32	W	4KDS	0.18	-279	-304	1.50	-301	-332
33	B	4KDS	0.18	-221	-378	1.48	-360	-398
34	A	4KDS	0.15	-267.5	-318	1.48	-352	-461
35	A	4KDS	0.15	-571.5	-623	1.47	-579.5	-631
36	P	4KDS	0.15	-279	-307	1.46	-332	-362
37	W	SCC	0.48	-236.5	-292	1.32	-232	-288

Table 6.2 Summary of multipoint grid testing results

			Test 1			Test 2		
Shaft #	Slurry	Mix	Age (yrs)	E_{50}(mV)	E_{min} (mV)	Age (yrs)	E_{50}(mV)	E_{min} (mV)
38	P	SCC	0.48	-258	-331	1.31	-254.5	-358
39	P	SCC	0.48	-245	-293	1.31	-240	-298
40	B	SCC	0.48	-251	-358	1.32	-263	-412
41	B	SCC	0.48	-226	-257	1.31	-211	-244
42	A	SCC	0.48	-202	-226	1.31	-194.5	-216
43	B	4KDS	0.31	-397.5	-483	1.15	-521	-626
45	B	4KDS	0.31	-439.5	-636	1.17	-533	-659
46	W	4KDS	0.31	-226	-258	1.17	-266	-362
47	W	SCC	0.27	-265.5	-313	1.11	-286	-351
48	B	SCC	0.27	-257.5	-293	1.11	-281	-325
49	B	SCC	0.27	-274	-314	1.11	-274	-320
50	P	SCC	0.27	-262	-279	1.13	-262	-285
51	P	SCC	0.27	-252	-314	1.13	-343	-404
52	A	SCC	0.93	-265	-316	1.11	-263	-335
53	P	4KDS	0.34	-183	-205	0.53	-241	-260
54	P	4KDS	0.34	-182	-215	0.53	-235	-252
55	P	4KDS	0.34	-187	-209	0.53	-279	-383
56	P	4KDS	0.34	-167	-181	0.53	-236	-252
57	B	4KDS	0.34	-183	-230	0.53	-288	-425
58	W	4KDS	0.34	-170.96	-215	0.53	-222	-247

The age of the specimens at the time of testing varied from just under 2 months to just over 6 years. By plotting the E_{50} values for each test versus the age of the specimen, there is an overall trend toward increasingly negative corrosion potential values (Figure 6.13). The data is separated into the three main support fluid types (bentonite, polymer, water).

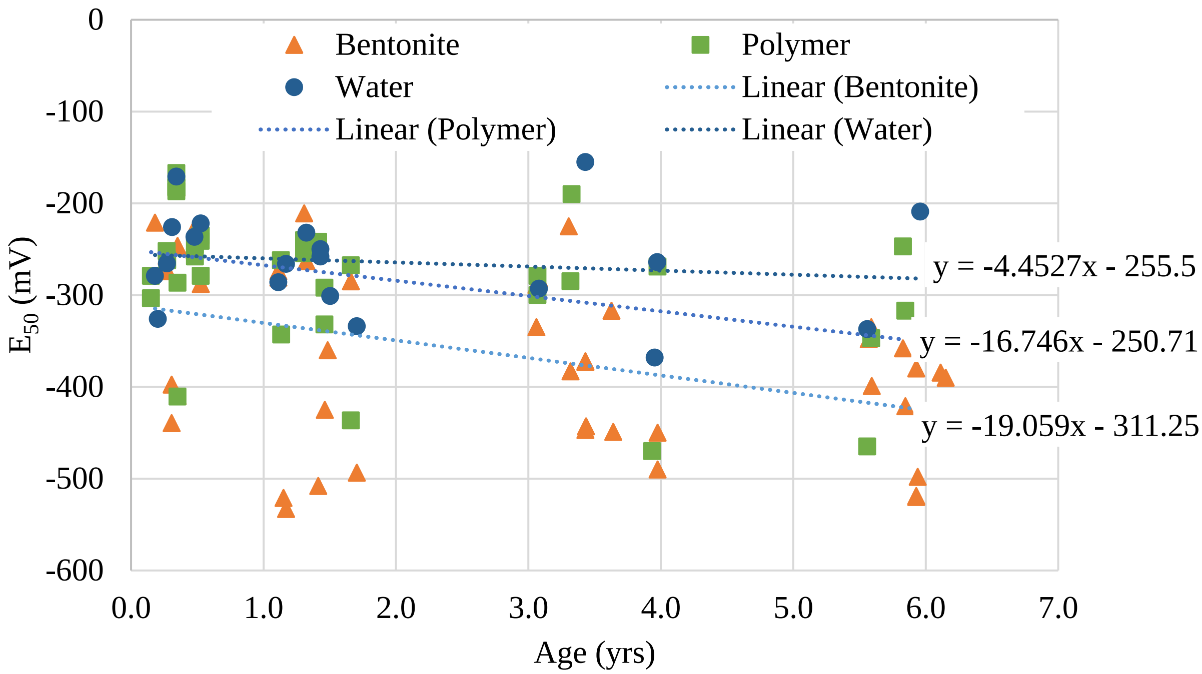


Figure 6.13 E_{50} vs. age at the time of testing

6.7 Chapter Summary

The results of the surface potential and electrical conductivity were presented from 52 large-scale, lab-cast drilled shaft specimens where concrete for each specimen was tremie-placed, support-fluid-displaced. The dominant variable between specimens was the support fluid type displaced at the time of casting. Visible creases were the most common anomalies that in some cases formed free-flowing channels from the concrete surface to the rebar and along the length of the reinforcing steel. Surface potential measurements indicated a startling effect of the creases, which was most pronounced when mineral support fluid was present at the time of concreting.

Chapter Seven: Strength Profiling

The goal of this testing was to devise an instrument capable of providing a strength profile of concrete in real time. In concept, it is a concrete penetrometer that measures the instantaneous concrete strength from concrete coring resistance via a fully-instrumented concrete coring drill motor. The novelty of the device is not the individual components (which are all off-the-shelf), but rather in the simultaneous collection of the core drill data followed by computational conversion to the in situ concrete strength profile. A significant advantage is to show locally weak or strong portions often missed in traditionally-sized specimens.

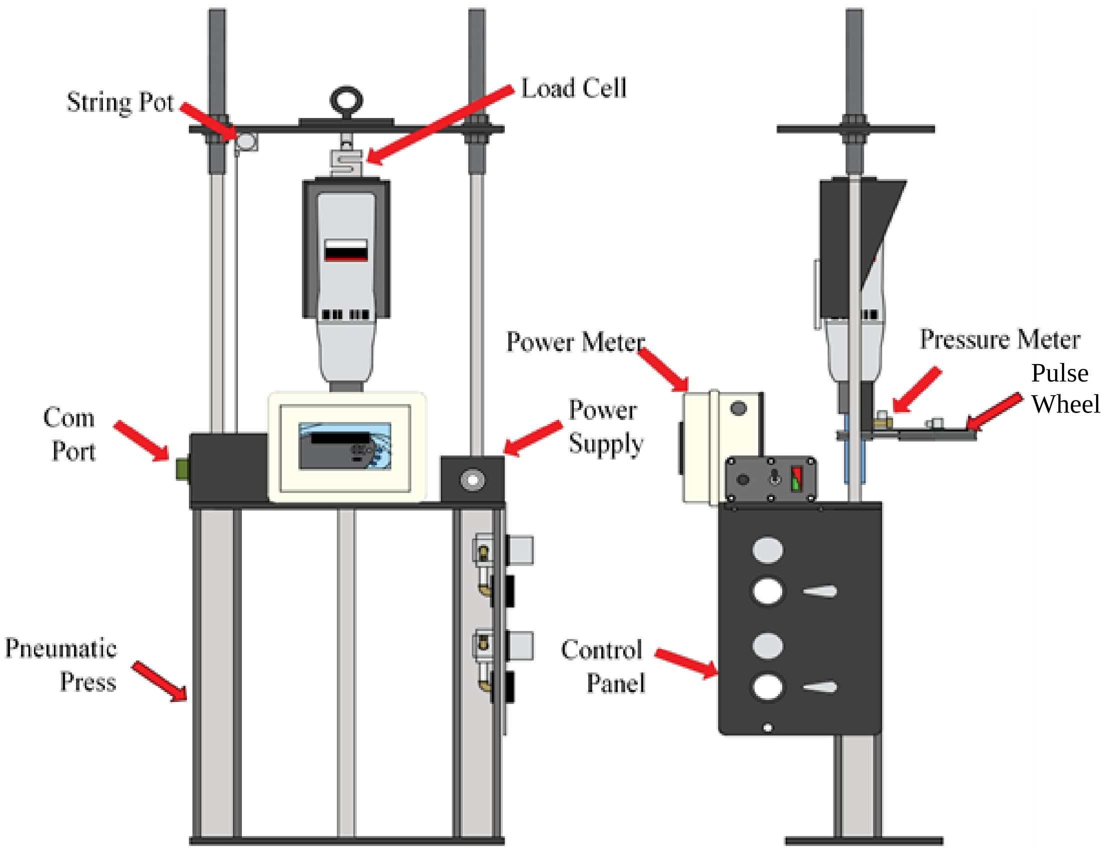


Figure 7.1 Line drawing schematic of the concept machine

Drilling resistance is a well-understood aspect of petroleum engineering where many aspects of the process are monitored to increase production. While the strength of material is the predominate cause of drilling resistance, advancement rate and not actual strength is the focus.
The platform of the machine was a Milwaukee 4049, 20-amp manually-operated coring machine fitted with a 1-inch inner diameter, diamond tip, core barrel. This is a wet-core drill that lowers and lifts the core barrel with a linear gear / rack and pinion configuration, wherein turning the crank controls crowd and advances or retracts the drill with a manually-applied, variable force (Figure 7.2).

Figure 7.2 Standard manually operated core drill motor

7.1 Instrumentation

Producing usable and replicable data necessitated the isolation of variables affecting coring efficiency, including force on the core barrel, rotational velocity, advancement rate, and fluid pressure. Mechanically, the motor and core barrel were removed from the crank/gear system and installed on a custom frame wherein the drill operation is controlled by two Parker 4MA, 24-in. stroke, 2.5-in. diameter, double-acting pneumatic cylinders. The pneumatic cylinders allow for complete user control of applied force by using four air pressure regulators controlling the downward crowd or upward extraction force independently. The exact force applied to the drill motor and core barrel was monitored using an Omega, LCCD-2K, 2000-lb capacity load cell connected between the top plate and the coring drill motor using a universal joint.

A Celesco SP2-50 string pot displacement transducer with a 50-inch range was used to record the depth of coring, and by recording with the associated time, the vertical advancement rate could also be determined. The rotational velocity (rpm) was measured with a BEI, H20 incremental rotary encoder mounted to a 2:1 ratio set of pulse wheels. As fluid was also used to flush cuttings from the annulus around the core barrel and in turn affects drilling performance, the fluid pressure was monitored using a Honeywell Model AB/HP 6-psi pressure transducer.

The variations in power resulting from additional crowd and drilling resistance was monitored directly using a General Electric PQMII Power Quality Meter. This meter combines voltage with current taken using an Omega RCT151205A current coil to produce a power output while taking into account the effects of phase shift. All data was monitored and recorded using a Model

BMS16HR-53 Titan Mini-Recorder computerized data acquisition system from Mars Labs. Data was collected at a 128 Hz sampling rate.

The result of this instrumentation was a drilling machine with the ability to provide dynamic force, velocity, pressure, power, and rpm data (Figures 7.3-7.5). In post-processing, this data could then be used to determine the resistive force and strength of the concrete. This data analysis process is outlined in the results section.

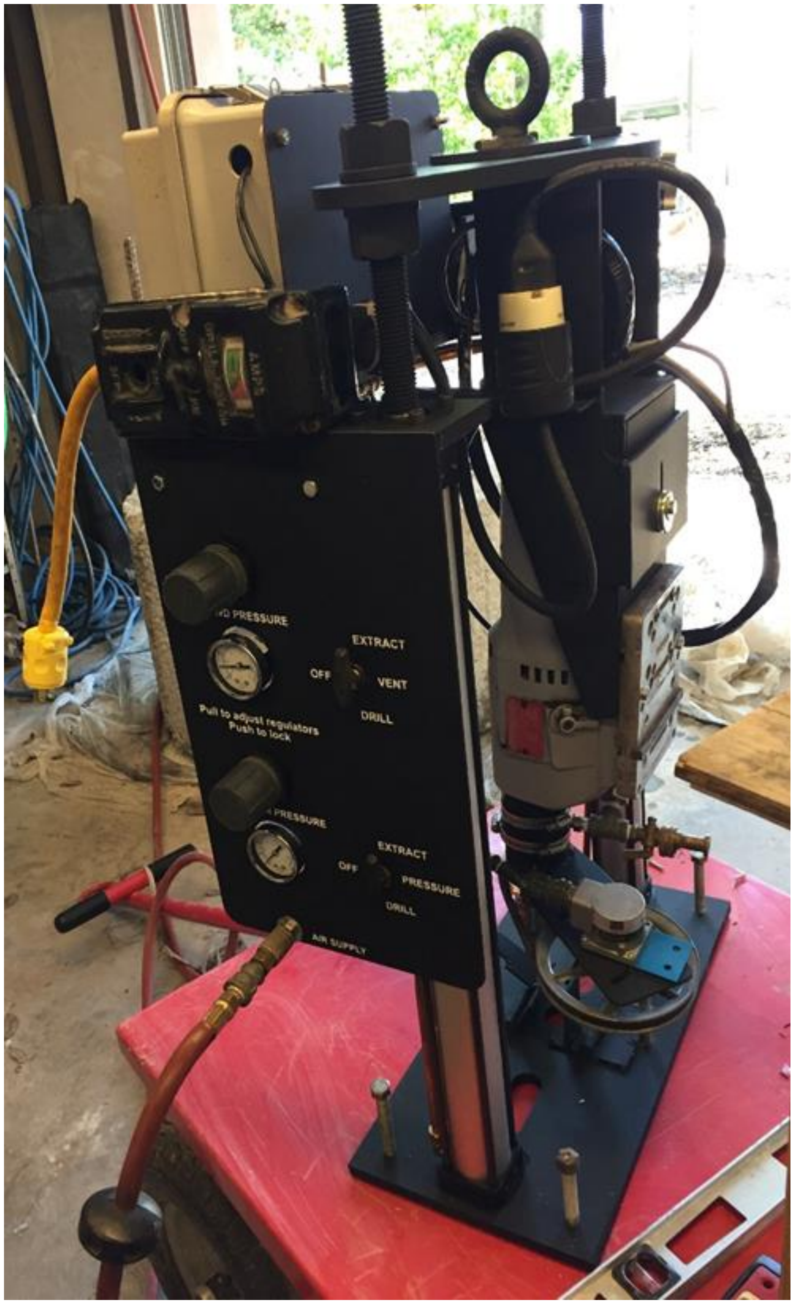


Figure 7.3 Control panel

Figure 7.4 Back view

Figure 7.5 Front view

Preliminary verification tests were conducted using the new coring system where each of the transducer outputs was checked. The results of that testing are detailed below.

7.1.1 Rotational Velocity

Rotational velocity can serve as a quality control check for the final calculations based on the idea that, given a constant applied drilling force, the rate of rotation should increase or decrease with corresponding changes in concrete strength. Rotation was calibrated through use of the totalizer option on the data acquisition system. The core barrel was turned manually and the rotations tallied, this result was then compared to the total rotations recorded. Good agreement was noted.

7.1.2 Power

In lieu of measuring torque and multiplying it by rpm to compute power, an electrical power meter was implemented to simultaneously measure the current, voltage, and phase angle. The importance lies in the phase angle measurement not previously measured, and where the actual power draw is the product of voltage, current, and the cosine of the phase angle. Previous measurements assumed a phase angle to be small and where the power factor (cosine of the phase angle) was taken as a constant of 0.9 based on spot-checked values. The new power meter is thought to have a significant advantage by removing this assumption.

7.1.3 Displacement

While somewhat trivial compared to the other transducers, the string line transducer was confirmed to register the full 18-in. stroke of the pneumatic cylinder. Like rotational velocity, the advancement rate was then computed using the timestamps associated with each data point.

7.1.4 Pressure

Pressure of the drilling fluid, if appreciable, could reduce the net force on the cutting edge of the core barrel. However, the anticipated pressure range was small and the 6-psi transducer range made simple calibration checks possible by using a simple column of water and comparing the hydrostatic pressure with that registered. Good agreement was noted.

7.1.5 Coring Procedure

The process of coring was standardized to provide baseline measurements of crowd, displacement, rpm, and power prior to making contact with the concrete surface. The base plate of the core rig was equipped with slots to allow for the installation of a mechanical rebar splice as a means to secure the machine to the shaft surface.

Steps used to perform the coring were as follows:

1. Set data acquisition system to scan
2. Balance transducers
3. Turn on water
4. Power core drill
5. Wait 5 seconds
6. Begin recording data
7. Wait 3 seconds
8. Slowly turn the vent knob from the “OFF” to the “DRILL” position allowing the core barrel to come in contact with the concrete surface gently.
9. Turn the Pressure knob from the “OFF” to the “DRILL” position
10. Monitor transducer readings during drilling operations
11. Once drilling operations are complete allow the drill to run for an additional 3 seconds
12. Turn off power
13. Stop recording data
14. Carefully, extract core barrel and check for any concrete prior to setting up at the next drilling location.

7.2 Data Analysis

The machine was instrumented to provide all necessary information used to calculate the specific energy as developed by Teale (1965). This, in turn, is correlated to the strength of rock or cemented materials during rotary, non-percussive drilling operations. Therein, crowd, rpm, torque, and displacement measurements are required (equation 7.1).

$$e = \frac{F}{A} + \frac{2\pi}{A}\left(\frac{NT}{u}\right) \tag{7.1}$$

wherein: e = Specific Energy (psi)
F = Crowd (lb)
A = Core Bit Area (in^2)
N = Rotational Speed (rev/min)
T = Torque (lb-in)
u = Penetration Rate (in/min)

The concrete penetrometer used the same equation, but power, P, was measured directly (equation 7.2).

$$P = TN2\pi \tag{7.2}$$

By combining equations (7.1) and (7.2), Teale's expression simplifies into equation (7.3):

$$e = \frac{F}{A} + \frac{P}{Au} \tag{7.3}$$

The specific energy is then equated to compressive strength (f'c) using an empirical relationship (equation 7.4) where the coefficients a and b are both functions of the penetration rate.

$$f'c = \frac{b + \sqrt{(b^2 - 4ae)}}{2a} \tag{7.4}$$

wherein: *f'c* = Compressive Strength (psi)

7.3 Results

The strength is then averaged every 1/16 inch of penetration and graphed against depth. Figure 7.6 shows the results from two coring tests. The first test was taken inside the reinforcement cage of shaft five, and the second test was also taken on shaft five, but this time the core barrel was aligned with a vertical crease outside of the reinforcement cage. Plotted strength profiles for all coring operations can be found in Appendix F.

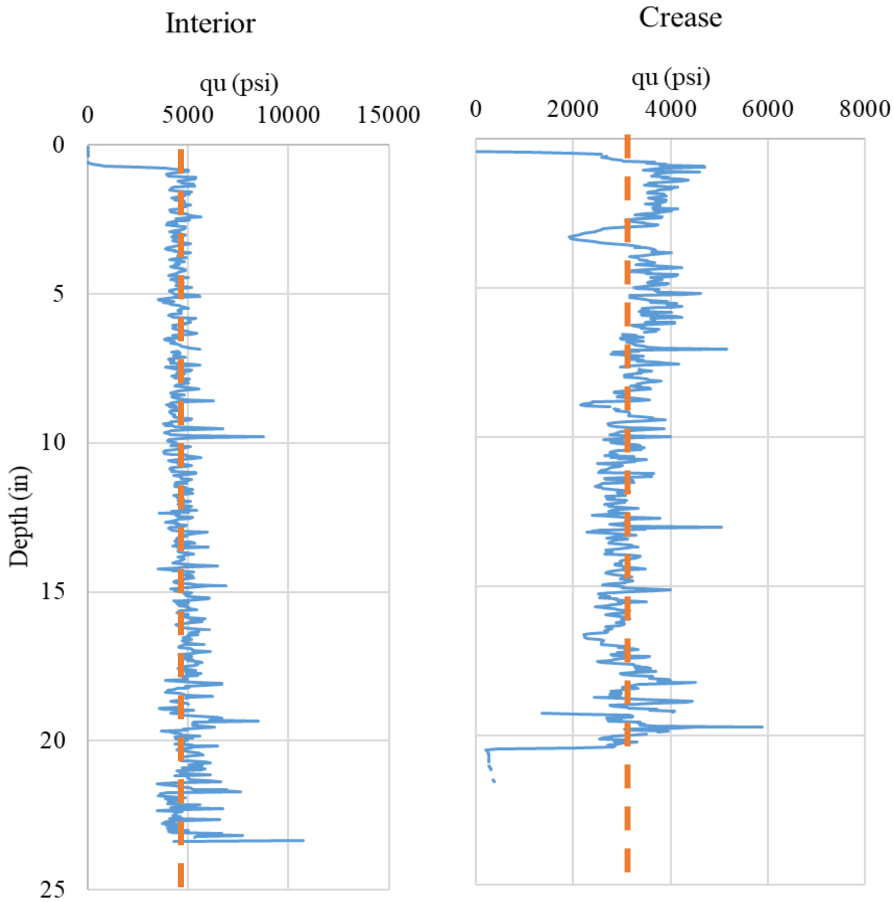


Figure 7.6 Shaft 5 strength profiles (interior and crease)

The dotted line on figure 7.6 represents the average calculated strength for each test. Thirty of the fifty-eight drilled shaft specimens have been profiled to date. This process involves drilling five to six cores per shaft; at least two inside the reinforcement cage (interior), two in the cover region, and one aligned with a vertical crease (Figure 7.7). The average strength per test is then computed and tabulated. The results for shaft five are given in table 7.1.

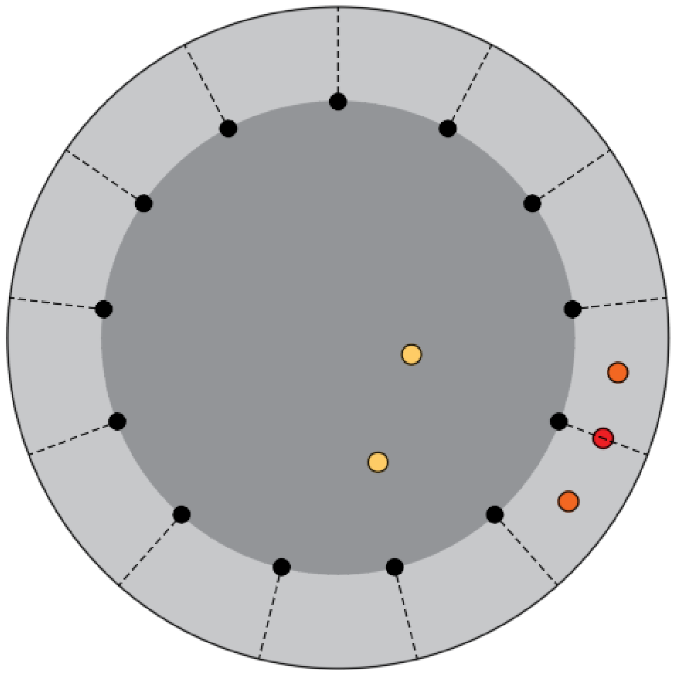


Figure 7.7 Plan view of sample core locations

Table 7.1 Average concrete strength summary for Shaft 5

Shaft 5		
Core #	Location	qu (psi)
1	Crease	3043
2	Cover	4711
3	Cover	4134
4	Interior	4312
5	Interior	5023
6	Interior	4640
	Baseline	**5023**

A summary table of average values similar to Table 7.1 was generated for each shaft in order to organize the data in a way that highlights strength differentials between the interior concrete and concrete in the cover region. The largest interior value was used as a baseline for numerical comparisons. Concrete strength is routinely determined via the compressive testing of cast concrete cylinders. These cylinders represent a perfect condition, thus providing the maximum possible concrete strength. As perfect conditions are nearly impossible to replicate in tremie-placed concrete, the largest interior average represents a value associated with the conditions nearest to perfect. The average strength in the cover region, including the strength determined from coring a vertical crease, was divided by the baseline strength on a per shaft basis. The resulting normalized data sets have been separated into three groups relating to the support fluid used during shaft construction (Tables 7.2-7.4).

Table 7.2 Comparison of interior (baseline) and normalized crease and cover strengths - bentonite

Bentonite					
Shaft #	Viscosity	Baseline	Crease	Cover 1	Cover 2
	(s)	(psi)			
1	44	4688	0.82	0.81	0.74
3	40	4296	0.86	1.03	0.94
4	55	4268	1.06	1.08	0.91
5	90	5023	0.61	0.94	0.82
7	30	4304	0.67	0.68	0.88
8	40	4532	0.88	0.91	0.91
9	57	5627	0.74	0.67	0.88
10	90	5142	0.77	0.76	0.80
13	30	3796	0.93	0.94	0.94
14	30	4512	0.68	0.93	0.90
15	56	4742	1.05	0.92	0.95
21	42	5092	0.73	0.81	0.95
45	37	4847	0.83	0.84	0.89
Average			**0.82**	**0.87**	**0.89**

Table 7.3 Comparison of Interior (baseline) and normalized crease and cover strengths - polymer

Polymer					
Shaft #	Viscosity (s)	Baseline Strength (psi)	Crease	Cover 1	Cover 2
11	65	4220	1.04	1.22	1.06
12	66	5976	0.77	0.96	0.96
16	85	4045	0.87	0.89	0.94
17	85	4345	0.91	0.91	0.93
19	63	5739	0.84	0.86	0.97
20	121	4720	0.76	1.01	0.96
Average			**0.86**	**0.98**	**0.97**

Table 7.4 Comparison of Interior (baseline) and normalized crease and cover strengths - water

Water					
Shaft #	Viscosity (s)	Baseline Strength (psi)	Crease	Cover 1	Cover 2
6	26	4752	0.85	0.87	0.96
18	26	3957.3	0.84	1.12	1.12
22	26	4597.6	0.88	1.02	1.03
32	26	4956.7	1.15	0.91	0.92
46	26	5341.3	0.86	0.95	1.01
Average			**0.92**	**0.97**	**1.01**

7.4 Chapter Summary

The results of coring-based strength profiling were presented from 30 large-scale, lab-cast drilled shaft specimens where concrete for each specimen was tremie-placed and support-fluid-displaced. The dominant variable was support fluid type at the time of casting. A minimum of five strength profiles was completed for each specimen. The average strength from each profile was tabulated and the strength values in the cover region were compared to the maximum average strength of the interior cores on a shaft-by-shaft basis. Additional comparisons were made between the maximum interior strength and the average strength of a profile aligned with a vertical crease in the cover region. The results of these comparisons showed a marked reduction in strength in the cover region, with the greatest impacts being attributed to specimens cast in mineral support fluid.

Chapter Eight: Porosity and Hydration Products Determinations

8.1 Overview

To better understand the phenomena causing the observed quilting in the cover region, it is necessary to examine concrete quality. This quilting is a product of the interface that is formed between radially flowing concrete and displaced slurry during tremie placement. This interface is referred to as a laitance. While common practice is to over-pour the drilled shaft, meaning to continue pouring concrete after the excavation is full, with the idea that all laitance is removed at the top of the shaft, recent studies have shown that in fact there is a dominant radial component of concrete flow that fills the interior cage and supplies the volume that subsequently exits radially into the cover.

The elements that make up the laitance include the concrete and the slurry. As the focus of this chapter is the concrete, it is important that the nature of the slurry is also understood. Three types of slurry were used during test specimen construction: mineral, polymer and natural (water). The slurries were all mixed the day prior to construction and were placed in the concrete forms 12 hours before concrete placement.

Mineral Slurry
Mineral slurry is the combination of water and a dry clay powder (usually sodium or calcium montmorillonite). The most commonly used clay is known as bentonite, though attapulgite, sepiolite, and other naturally occurring clay minerals are also used. Bentonite is the common name for packaged, processed, clay powder made primarily of sodium montmorillonite. Bentonite slurry works two-fold during the excavation and construction processes: (1) with the slurry level higher than the ground water, the differential hydrostatic slurry pressure pushes against the excavation walls preventing cave-ins and (2) the gel strength of the clay suspends soil particles long enough to be transported out of the excavation during the concreting process.

When bentonite slurry is introduced into an excavation, the slurry permeates the walls of the excavation and deposits clay particles as they are filtered out of suspension. The resulting layer of clay on the side walls, called a filter cake, further stabilizes the soil matrix from fluctuations in local slurry pressure that accompany the auger passing by the walls. Filter cake formation occurs relatively quickly where, within 4 – 8 hr, flow into the surrounding soil can completely cease. Though generally beneficial to stability, the filter cake can have negative effects on the side shear (concrete / soil interface) of the shaft.

Viscosity is the best measure of slurry quality and is monitored via the American Petroleum Institute (API) test method known as the Marsh funnel test. While the test does not measure viscosity in the traditional sense (shear stress / shear rate), it provides an indication of slurry consistency by measuring the time required for 1 quart of fluid to pass through a standard orifice at the base of a standard funnel. For bentonite slurry to function properly, state and federal specifications require the slurry to fall between 28 and 50 sec/qt, depending on the state. In Florida, the range is 30 to 40 sec/qt. As a point of reference, water has a Marsh funnel viscosity of 26 sec/qt, so a 28- or 30-sec/qt slurry has very little bentonite powder in suspension (0.11-0.15 lb/gal,); 50-sec/qt bentonite slurry requires 0.65-0.8 lb/gal. (Mullins et al., 2011).

Polymer Slurry

Polymer slurry is the combination of water and proprietary blends of polyacrylamides. These slurries form long, hair-like, chain molecules that have been negatively charged to promote molecular repulsion (Reese and O'Neill, 1999). Like bentonite, polymer slurry requires a head differential sufficient to overcome the force of the groundwater inflow. The molecular structure of polymer slurries prohibits the formation of a filter cake (no particulates) and continuous filtration is required to maintain the stability of the excavation. This requires a higher head differential for polymer slurry than that needed for mineral slurry and more reserve volume.

Quilting

As the timeframe for the formation of the radial / horizontal flow is relatively brief, it is likely that the degree of deterioration of concrete or slurry/concrete interface thickness of the laitance is minimal. Regardless, previous chapters have shown that the laitance does form. As vibratory consolidation of the concrete is rarely implemented in slurry-type drilled shaft construction, the region outside the cage is highly likely to contain veins of poorly cemented or diluted, high w/c ratio material. In the cases where bentonite is used, these veins contain trapped bentonite (Figure 8.1).

Figure 8.1 Radial concrete flow responsible for filling cover region.

Quilting describes vertical or horizontal planar features emanating primarily from reinforcing bars (Figure 8.2). Concrete is always placed inside the cage such that flow must go outward through the reinforcement cage into the cover region. As the concrete flows around the reinforcement, a separation occurs whereby two faces are cleaved by passage around the rebar and coated with slurry, commonly referred to as laitance interfaces, which must recombine outside the rebar by pressing these interfaces together. This creates visible or microscopic interfaces of altered concrete that may appear on the side of the shaft surface as creases in the form of a quilted grid pattern. The depth of the creases can extend to the reinforcing steel and this presents significant durability issues as the openings facilitate the corrosion process by providing transport pathways for the ingress of detrimental materials existing in the surrounding soil. Figure 8.3 shows a conceptual view of the radial concrete flow and the vertical creases that form. This hypothesis would explain the poor corrosion performance noted in visually perfect specimens. Figure 8.4 shows the same phenomenon that forms horizontal creases simultaneously.

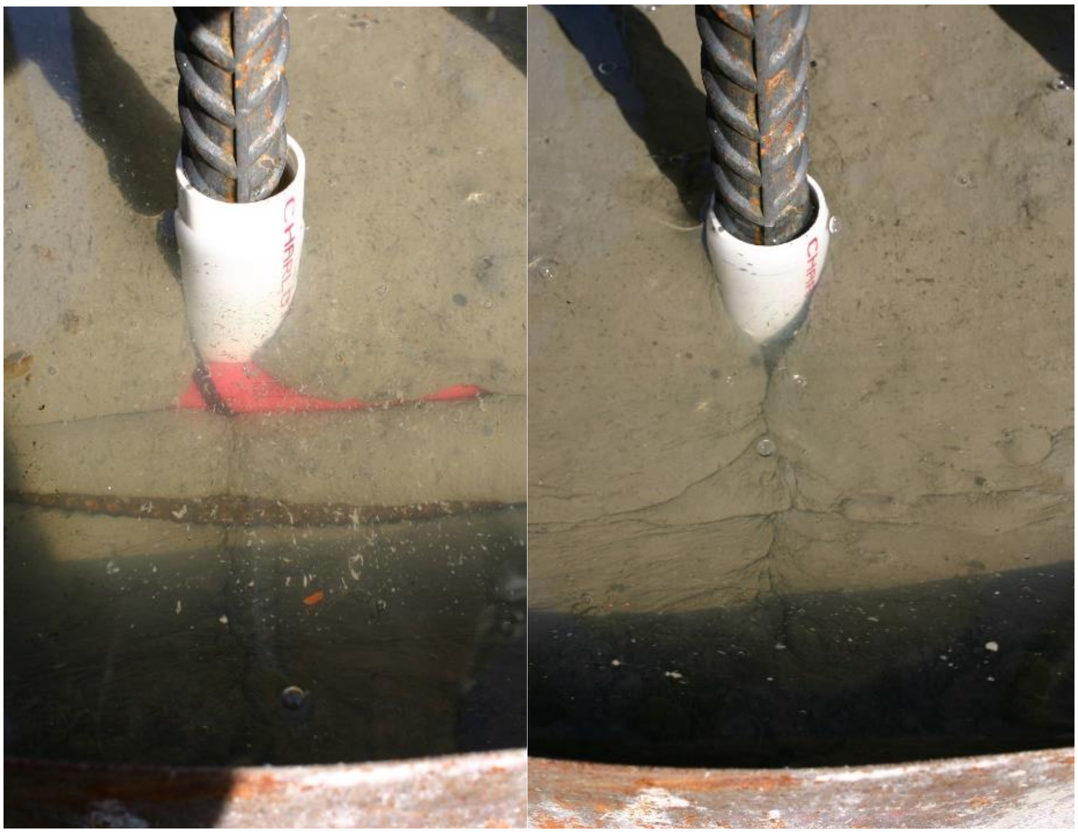


Figure 8.2 The concrete flowing radially and forming quilting.

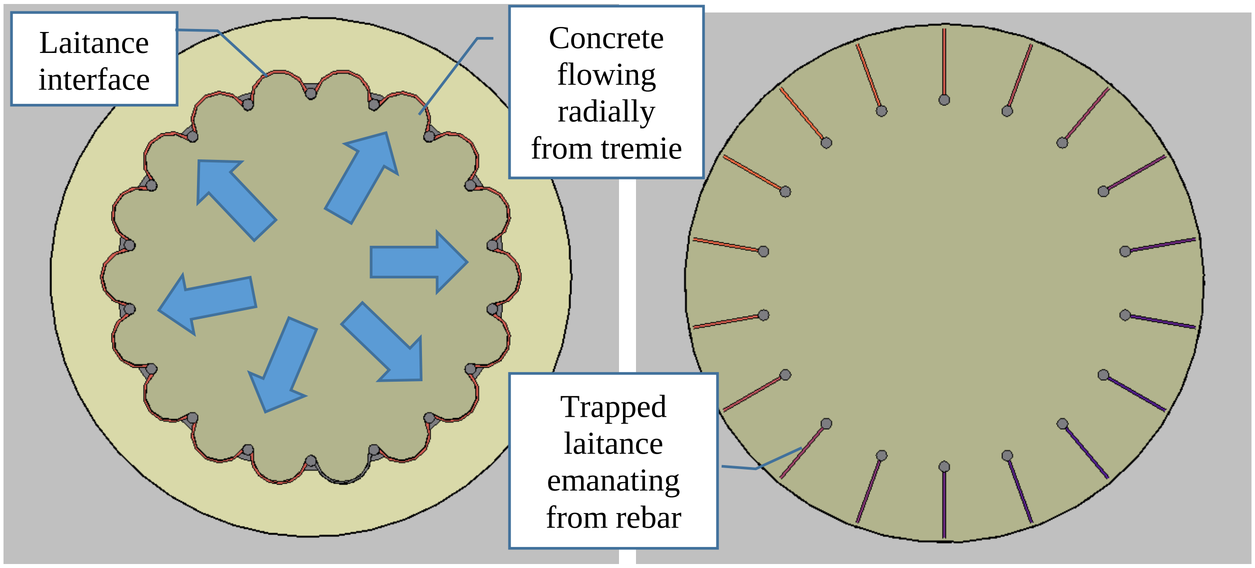


Figure 8.3 Top view of concrete flowing radially through cage (initial left; final right).

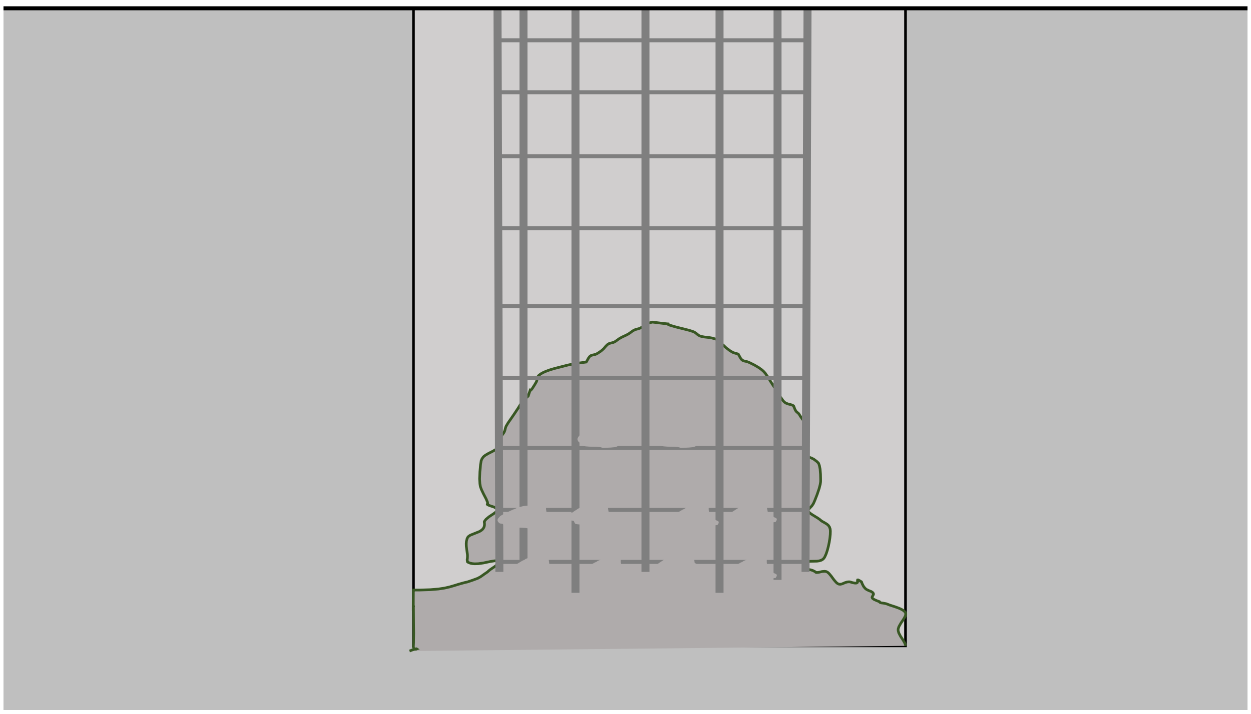


Figure 8.4 Profile view of radial concrete flow through stirrups.

This final report will focus on evaluations using compressive strength, Mercury Intrusion Porosimetry (MIP), and X-Ray Diffraction (XRD) testing. Three shafts were selected for testing from the 52 specimens that were cast, one each from the three slurry types used (water, bentonite, and polymer). In addition, paste cubes were constructed to further analyze the impact that bentonite slurry can have on the integrity of the "quilted" surfaces in the cover region, and on the properties of the concrete when it is intermixed with slurry.

This Task undertook three different types of testing to assess the properties of the concrete inside and outside the reinforcing cage. The premise of the program was that the act of concrete flowing through the cage (for submerged-tremie placement) causes the concrete to mix with the slurry, producing poorer quality / contaminated concrete. The three tests included: (1) mercury intrusion porosimetry, (2) x-ray diffraction, and (3) strength testing. The latter was performed on extracted cores and purpose-built specimens with various proportions of cement paste and slurry.

8.2 Coring (Specimen Retrieval)

Of the 52 specimens, shafts 6, 9, and 11 were chosen for the testing in this Task. Shaft 6 was placed in water, shaft 9 in a 50-sec/qt bentonite, and shaft 11 in a 65-sec/qt polymer. The overall test program structure provided for non-destructive electrochemical testing first, followed by selective coring. Cores were then divided for mercury intrusion porosimetry, x-ray diffraction, and strength testing (the concrete penetrometer was used to further evaluate strength variations).

Cores were labeled using the following format: The first number was the shaft number. The second number denoted the region of the shaft from which the core was taken (the shafts were divided into four quadrants). The letter *a* or *b* indicated whether the core was outside or inside the reinforcement cage, respectively; *a* means the core was taken from outside the reinforcement cage in the cover region, and *b* denotes a core sample from within the reinforcement cage. The following number in parenthesis was the segment of the core. The last number was the section number cut from the noted core segment. For example 6-1a(2)1 denotes shaft 6, quadrant 1, outside of the reinforcing cage (a), second piece of the recovered core, and the first/uppermost sample from that core. Figure 8.5 shows the coring locations of Shaft 6 as well as the profile of the core.

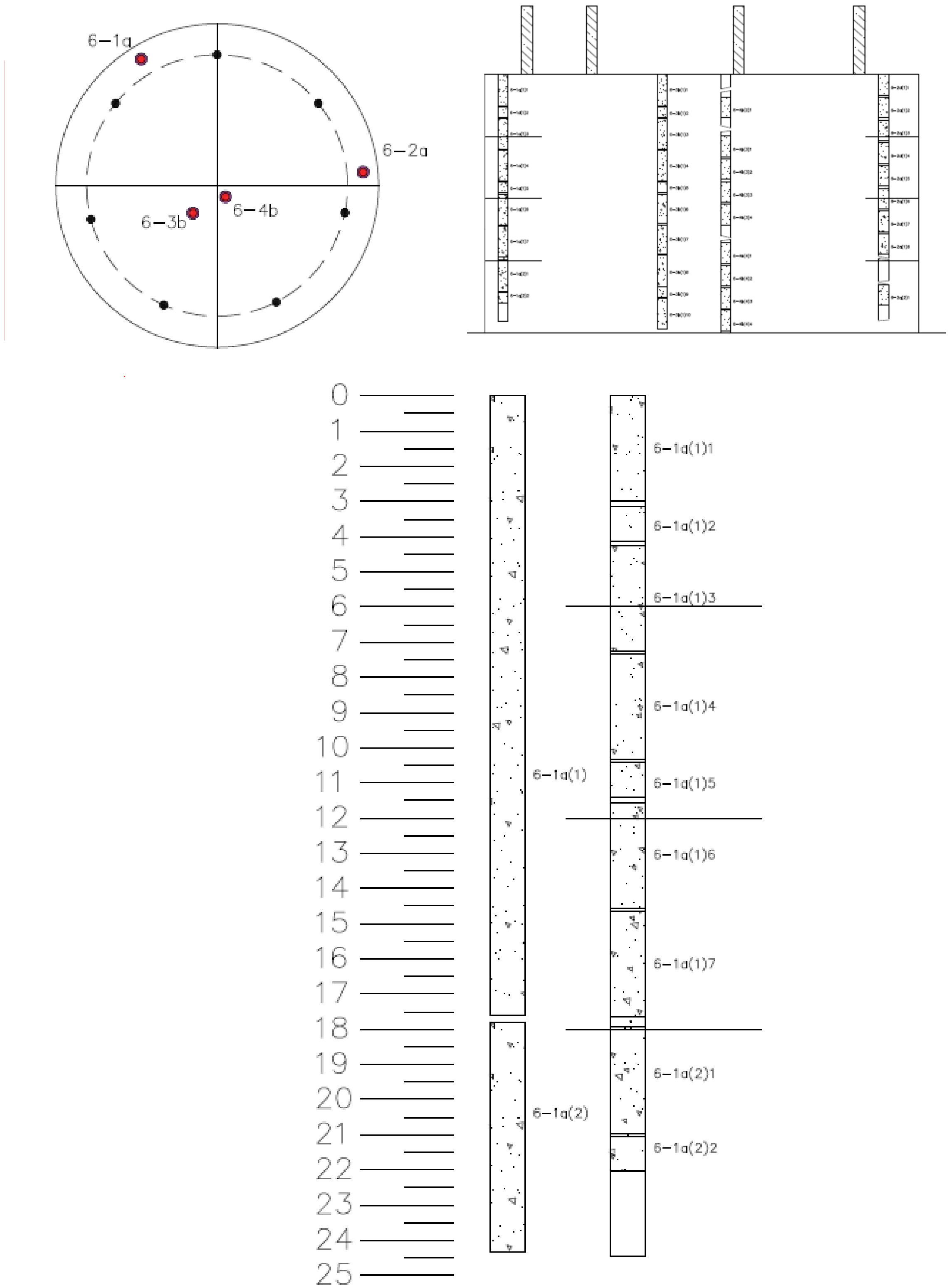


Figure 8.5 Core location and profile for 6-1a.

8.3 Mercury Intrusion Porosimetry

Mercury Intrusion Porosimetry (MIP) is a useful tool when characterizing porous materials like concrete. The test method can examine pore sizes between 3.5 nm to 500 μm in diameter and provide information such as the pore size distribution, total pore volume or porosity, specific surface area of the sample, and the skeletal and apparent density. While this information is helpful, the use of this testing technique is not without limitations; all pores and pore structure emanate from the sample surface and where the largest pores are assumed to be at the surface. The method cannot identify the actual size and volume of interior pores, nor can it analyze closed pores, as the mercury has no way of entering.

Mercury is uniquely suited for this type of test due to its high surface tension and incompressible nature (no dissolved air gases typical of other fluids). The high surface tension increases the resistance to fill voids that would be immediately filled by lower surface tension fluids at atmospheric pressure. This allows the test to discriminate between void diameters using a pressure / void size relationship which yields a pore size distribution. Table 8.1 shows the surface tension of typical fluids.

The assumption of most importance for this testing method is pore shape. Using the Washburn equation (Equation 1), a modified Young-Laplace equation, practically all instruments assume a cylindrical pore geometry.

$$\Delta P = \gamma \left(\frac{1}{r_1} + \frac{1}{r_2}\right) = \frac{2\gamma cos\theta}{r_{pore}} \tag{8.1}$$

Where

r_1 and r_2 – describe the curvature of the interface
γ – surface tension of the mercury
r_{pore} – pore size
θ – contact angle between the solid and mercury
ΔP – pressure difference

The Washburn equation relates all of the above elements. However, it should be noted that the real pore shape can be quite different from the cylindrical assumption made. This can lead to discrepancies with analysis and reliability. (Giesche, 2006)

Pore distribution is determined on the basis of the amount of mercury that intrudes as a function of pressure, where the volumes intruded at lower incremental pressures define larger pore sizes, and further refinement of the distribution is obtained as the pressure is increased, progressively forcing the mercury into smaller and smaller pores.

Table 8.1 Surface tension of common fluids relative to mercury.

Source: http://www.dataphysics.de/fileadmin/user_upload/pdf/DataPhysics_Surface_Tension_Energy.pdf

Fluid Name	Surface tension (mN/m) @ 20°C
Ethanol	22.10
Isopropyl Alcohol	23.00
Acetone	25.20
Gasoline	29.20
Engine Oil	31.0 (15°C)
Glycerol	63.28
Water	72.80
Mercury	425.41

For this study MIP was conducted on cores extracted from three tremie-cast shaft specimens where water, polymer slurry, or bentonite slurry was displaced during concreting.

8.3.1 Preparation of MIP specimens

The specimens for MIP testing came from cores 6-1a, 6-3b, 9-3a, 9-1b, 11-1a, and 11-3b. Recall that (a) means the core was taken from outside the reinforcement cage in the cover region and (b) denotes a core sample from within the reinforcement cage. Cores were 1-in. diameter concrete and were cut to a height/length of 1.0 using a concrete saw and oven-dried at 105°C for 24 hours to constant weight. MIP tests were performed on the samples using Quantachrome Poremaster 60. The system was operated at the high-pressure range, 0-60,000 psi. The adopted mercury surface tension was 480 mN/m and the contact angle between the mercury and the solid surface was 140°.

8.3.2 MIP Test Results

Mercury porosimetry curves for shafts 6, 9, and 11 can be seen in Figures 8.6, 8.7, and 8.8, respectively. Intruded volume is measured in cubic centimeter per gram of concrete. A very minor trend is seen in these graphs where it seems the cover region concrete is more porous than the interior concrete. This is best demonstrated in Shaft 11, polymer, and very subtly seen in Shafts 6 and 9, water and bentonite, respectively. Please note it is suspected that shaft 6 sample 6-1a(1)2 is only accurate to a pore diameter of 100 nm due to the anomalous results; analysis for the remainder of the report only uses data from the smaller pores. This shaft is being re-cored and retested to eliminate any suspicion.

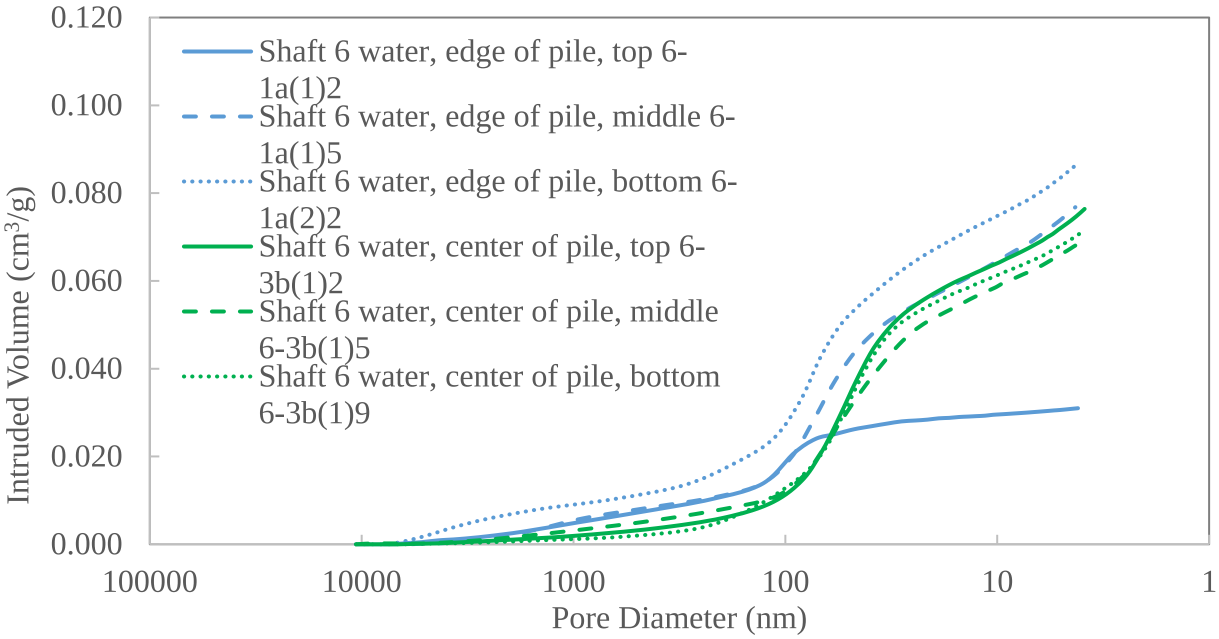


Figure 8.6 Mercury porosimetry curve for samples from Shaft 6.

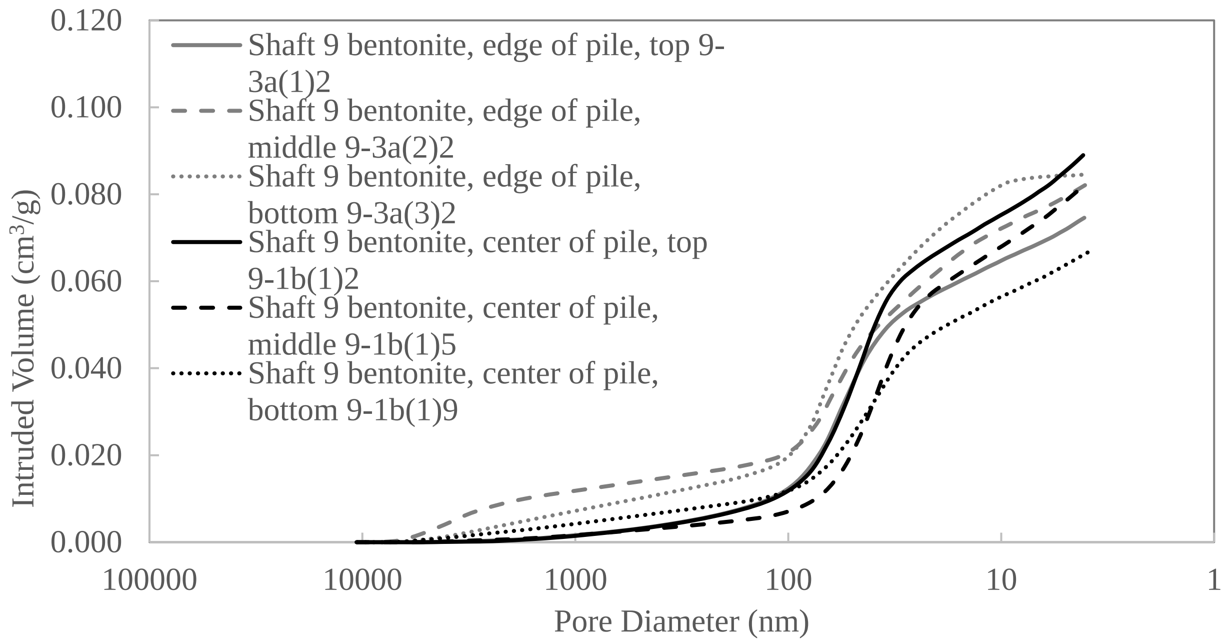


Figure 8.7 Mercury porosimetry curve for samples from Shaft 9.

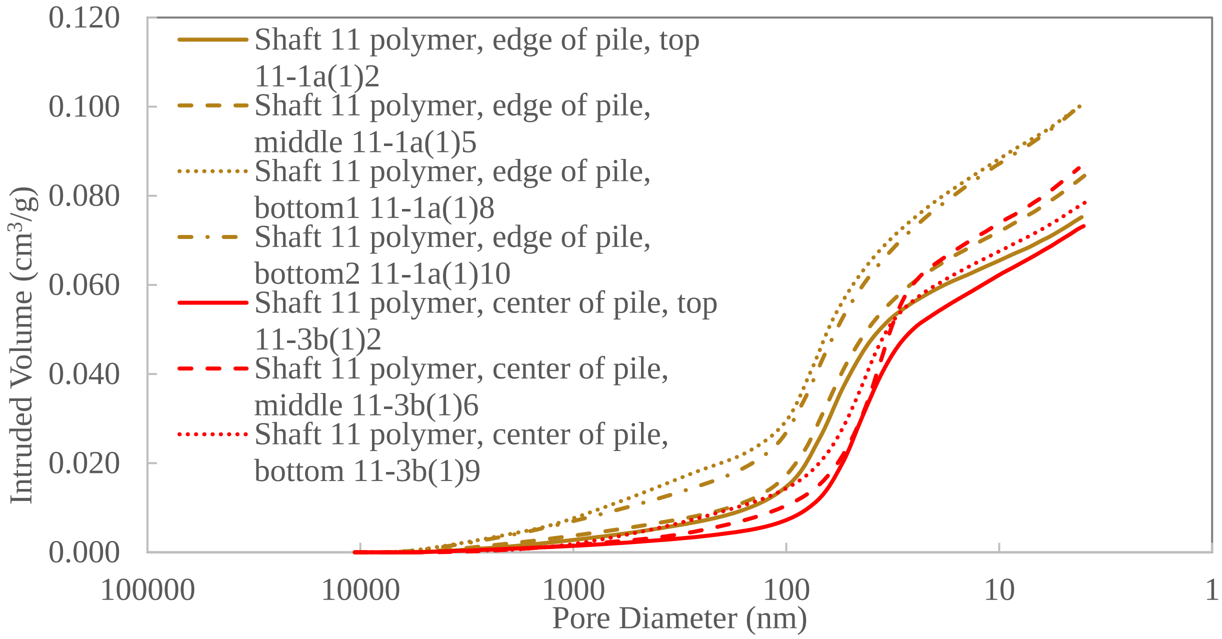


Figure 8.8 Mercury porosimetry curve for samples from Shaft 11.

Next the pore size distribution was plotted for all samples (Figures 8.9-8.11). Again, while there is a suggested trend, further testing is needed to corroborate and expound on this data. This trend can be seen where the highest peak of the samples from outside the reinforcement cage in general are higher than the main peak shown from samples inside the reinforcement cage and suggests a shift in the pore size distribution for the concrete outside of the reinforcement cage. Thus initial findings would suggest the cover region concrete has a higher porosity. All samples were within 3-in. of a crease.

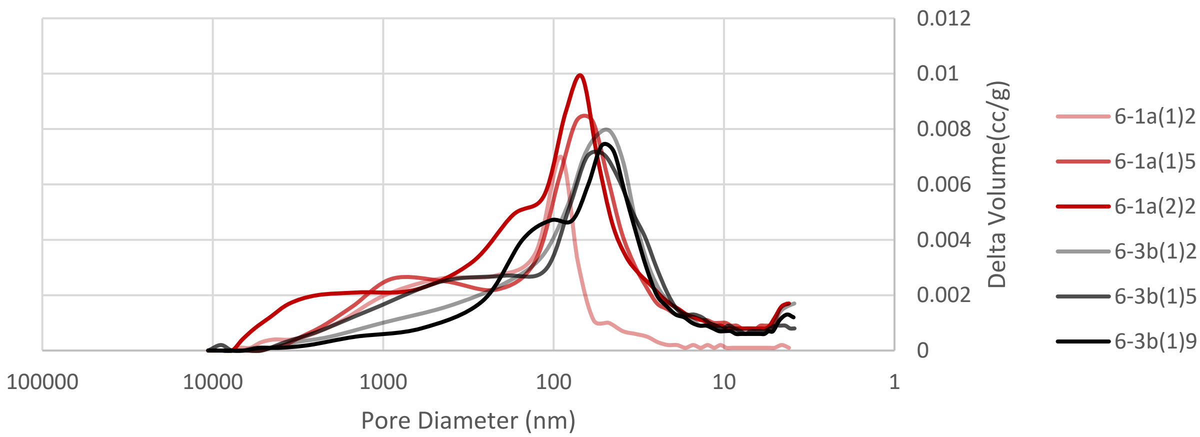


Figure 8.9 Pore size distribution from MIP testing for cores from shaft 6 (water).

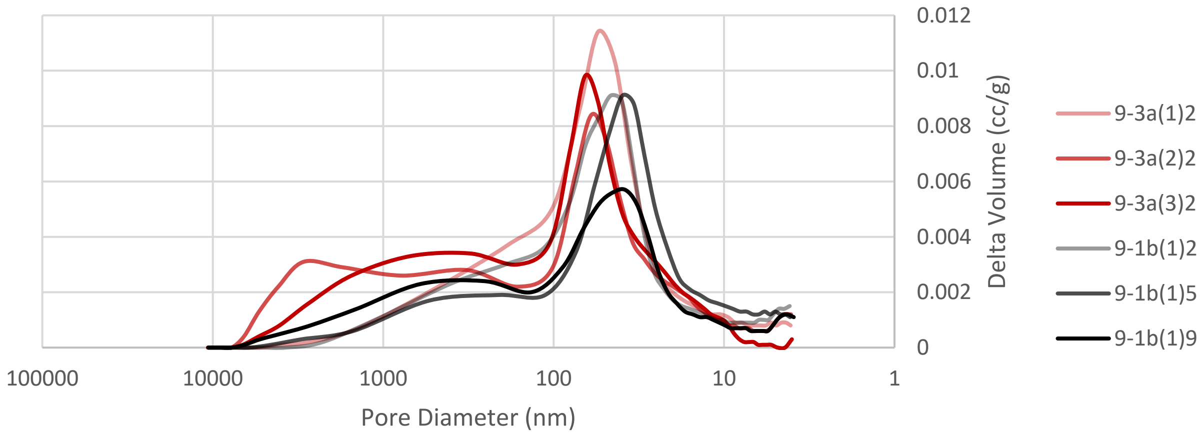


Figure 8.10 Pore size distribution from MIP testing for cores from shaft 9 (bentonite, viscosity 50 sec/qt).

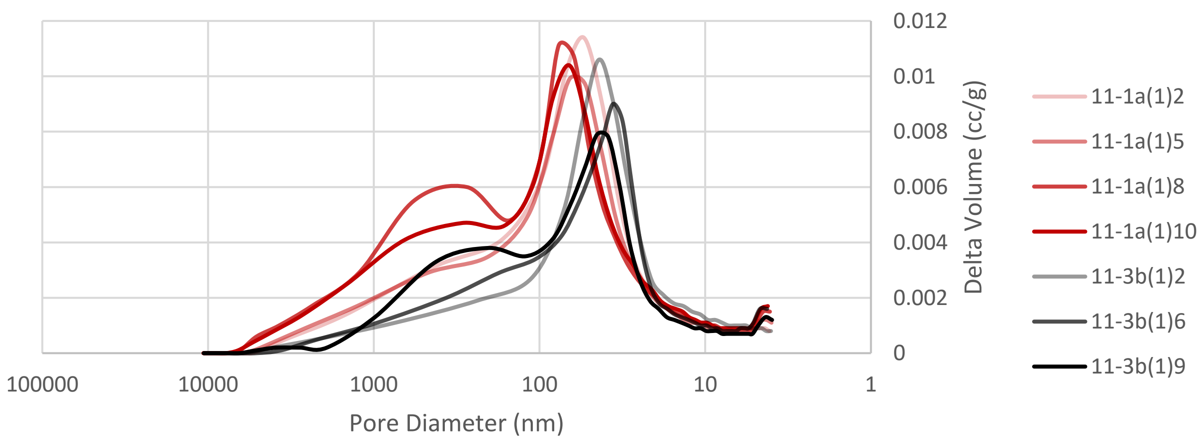


Figure 8.11 Pore size distribution from MIP testing for cores from shaft 11 (polymer, viscosity 65 sec/qt).

Next data was analyzed from a perspective of total intruded volume as a function of depth, Figure 8.12. This is also the end point (farthest right) of Figures 8.6 - 8.8. While somewhat inconclusive, a small trend can be seen where Shaft 6, water, has the lowest total intruded volume in comparison to the bentonite- and polymer-cast shafts, 9 and 11, respectively.

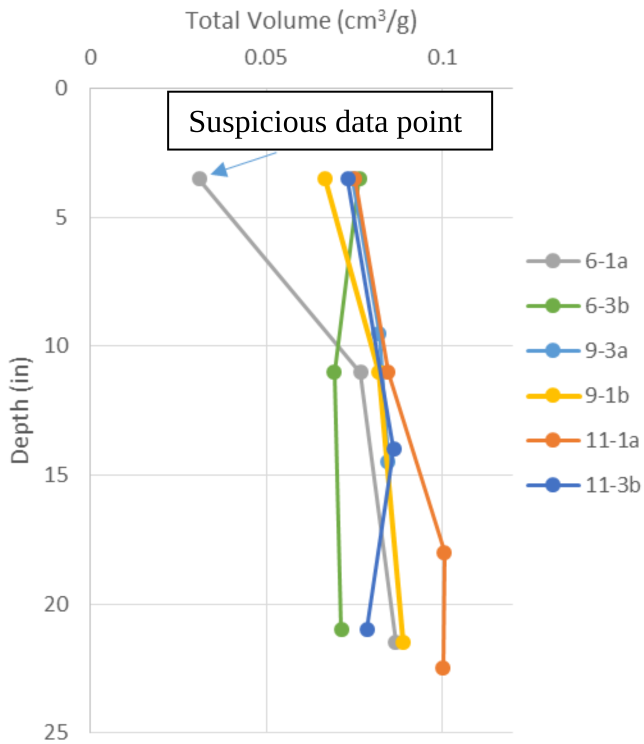


Figure 8.12 Total intruded volume for samples tested from shafts 6, 9, and 11.

Correlating with the pore size distribution, data was then broken into four groupings: 1,000 to 10,000 nm, 100 to 1,000 nm, 10 to 100 nm, 1 to 10 nm and plotted versus depth, Figure 8.13. Again, referring to Figures 8.6-8.8, this is the difference in volume from the 10,000, 1,000, 100, 10, and 1 locations. For example, 10 to 100 nm graphs show the value at 10 nm minus the value at 100 nm. Here again it can be noted that the 10 to 100 nm range has a higher intruded volume. This is also the trend seen in Figures 8.10-8.12. No significant variation is seen as a function of vertical position in the shaft.

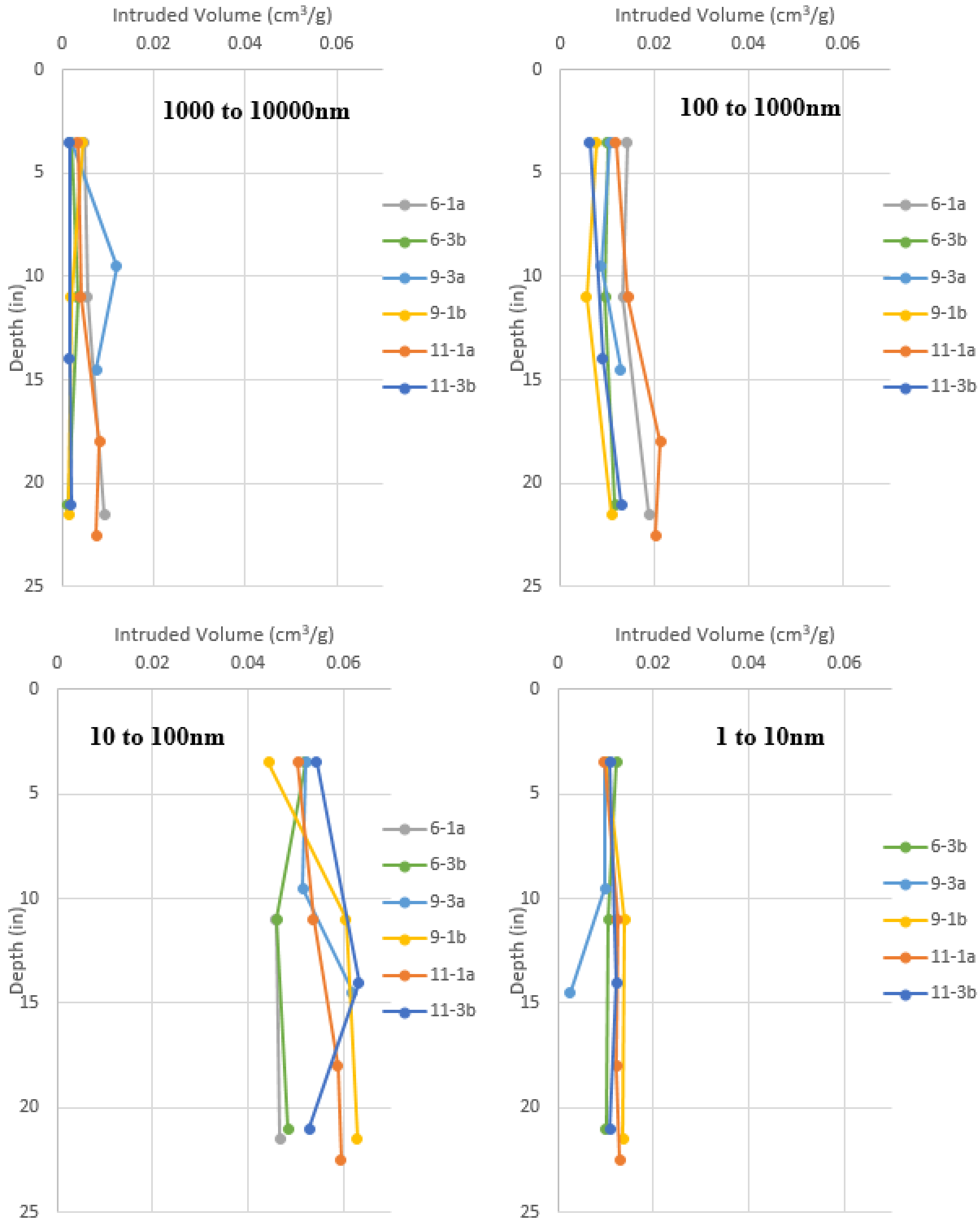


Figure 8.13 The four groupings of intruded volume.

While a subtle trend can be seen where water-cast shafts are the least porous and polymer-cast shafts are most porous, very large voids in the surface of a specimen that may be typical of tremie-placed concrete may go undetected as the volume of these voids would be discounted as the baseline zero-pressure mercury volume required to start the test.

8.4 X-Ray Diffraction

Quantitative X-ray diffraction analysis (QXRD) of concrete consists of three main steps: preparation of a powdered sample, data collection, and Rietveld refinement analysis for phase identification and quantification.

8.4.1 Sample Preparation

The preparation of concrete specimens for XRD has some specific demands and features. First, the powder sample preparation for QXRD should not contain aggregate, which requires the manual separation of limestone (calcite, $CaCO_3$) from the hydrated cement. This minimizes the amount of limestone that can interfere with hydration phase quantification. The grinding of concrete should also be gentle/light to separate or minimize the amount of sand (quartz, SiO_2) in the powdered sample. Aggregate content minimization increases the accuracy of the weight fraction determination of the crystalline hydration products and the remaining cement phases in concretes. Back-loading technique (Taylor et al., 2002) of powder into the sample holder minimizes preferred orientation effects.

Several techniques for Rietveld analysis have been proposed in the literature (Aranda et al., 2012; Le Saoût et al., 2011; Snellings et al., 2014; Wilson et al., 2014). These techniques have some common and specific features in the order and number of parameters that should be refined; namely, scale factors, lattice parameters, zero shift or specimen displacement, polynomial or Chebyshev polynomial coefficients for background or automatic background fitting, preferred orientation for some specific phases, and peak shape parameters.

For this study, the as-received concrete core specimens were crushed into small size pieces to separate as much of the limestone (light-color pieces) as possible from the mortar (gray-color pieces). Then, the selected gray-color pieces were gently/lightly ground using a mortar and pestle to obtain a powder finer than 45 microns (passing #325 sieve) and to remove as much sand as possible.

8.4.2 Mixtures of Concrete Powders with Internal Standard Material

For a complete analysis of concrete, the amorphous content should be determined in addition to the determination of the crystalline phases. Due to the incomplete separation of the limestone and sand from the hydrated cement paste, the internal standard method was adopted for the amorphous content determination. Highly crystalline (95 wt%) calcium fluorite (CaF_2) was used as an internal standard material. The concrete powder was mixed with 10 wt% fluorite using a mortar and pestle.

8.4.3 XRD Data Collection and Analysis.

XRD scans for all concrete were collected in accordance with ASTM C1365-06 specifications. The following diffractometer settings were used in data collection:

Instrument Settings:

-	Diffractometer:	Phillips X'Pert PW3040 Pro
	Goniometer:	θ - 2θ , radius 240 mm
	Source:	CuKα radiation, line focus
	Generator:	45 kV, 40 mA
-	Sample:	
	Surface diameter:	16 mm
	Spinning rate (rpm)	30
	Preparation:	Back-loading
-	Incident optics:	
	Programmable divergence slit:	5 mm (automatic)
	Soller slit:	0.04 radians
	Mask (horizontal divergence slit):	10 mm
-	Receiving optics:	
	Programmable anti-scatter slit:	5 mm (automatic)
	Soller slit:	0.02 radians
	Detector:	X'Celerator Scientific
-	Scan info:	
-	Angular range (2θ):	5 - 70°
	Step (2θ):	0.0167°
	Length of linear detector (2θ):	2.122°
	Counting time per step (s):	130.2

Automatic slits were used in the incident and receiving optics to reduce background scattering from the metal holder, which can be observed for fixed-slit settings at low diffraction angles ranging between 5 and 20° (Pecharsky and Zavalij, 2009). This undistorted range is important for correct phase-fraction determination of ettringite, monocarboaluminate, and portlandite. The Rietveld refinement analyses of the collected x-ray scans were performed using HighScore Plus v4.5 software.

8.4.4 Rietveld Refinement Analysis

Rietveld refinement (Rietveld, 1967), (Rietveld, 1969) is based on the iterative comparison of the experimentally-collected diffraction pattern with the calculated pattern for a mixture of known phases. According to the literature, the following parameters should be refined: scale factor; lattice parameters; zero shift (Aranda et al., 2012), (Snellings et al., 2014), (Jadhav and Debnath, 2011; Gualtieri et al., 2012; De Schepper et al., 2014; Snellings et al., 2010; Speakman, 2010); preferred orientation for portlandite (001), calcite (104), monocarboaluminate (100), and ettringite (001) (Aranda et al., 2012; Snellings et al., 2014; Wilson et al., 2014; Le Saoût et al., 2007; Gualtieri et al., 2012); and peak shape parameters (Le Saoût et al., 2011; Wilson et al., 2014). The background was chosen in the automatic mode with a bending factor of 5 and granularity of 25. The following crystalline phases, Table 8.2, were detected and refined (note that no alite was detected):

Table 8.2 Crystalline phases in XRD refinement.

Phase	Formula	Crystal System	ICSD Code
Belite	Ca_2SiO_4	Monoclinic/β	81096
Ferrite	Ca_2AlFeO_5	Orthorhombic	9197
Calcite	$CaCO_3$	Rhombohedral	80869
Quartz	SiO_2	Rhombohedral	41414
Portlandite	$Ca(OH)_2$	Rhombohedral	15471
Ettringite	$Ca_6Al_2(OH)_{12}(SO_4)_3$ $26H_2O$	Hexagonal	155395
Monocarboaluminate	$Ca_4Al_2(OH)_{12}(CO_3)$ $5H_2O$	Triclinic	59327
Fluorite	CaF_2	Cubic	44618

The results of the Rietveld refinement of concrete specimens are shown in Table 8.3. The results are given after excluding any quartz (sand) and calcite (limestone) from the analysis.

Table 8.3 QXRD Phase analysis for shafts prepared with water, polymer and bentonite slurries (weight percentages); samples from shaft 6 were 3 years, 8 months, 2 weeks, and 2 days old, samples from shafts 9 and 11 were 3 years, 8 months, and 3 days old

Sample	Belite	Ferrite	Portlandite	Ettringite	Monocarboaluminate	Amorphous Content
6-1a(1)1	0.6	0.2	1.0	1.6	1.0	95.6
6-1a(1)4	0.0	0.2	2.0	1.6	1.4	95.0
6-1a(2)1	0.0	0.0	2.0	1.8	1.1	95.1
6-3b(1)1	1.0	0.4	1.1	1.7	1.0	95.0
6-3b(1)4	0.7	0.5	1.7	2.0	1.5	93.7
6-3b(1)8	0.2	0.2	1.1	1.1	1.1	96.3
9-1b(1)1	0.3	0.5	1.4	3.1	1.5	92.9
9-1b(1)4	1.1	0.8	1.1	2.8	1.9	92.4
9-1b(1)8	0.0	0.2	0.7	1.1	1.7	96.3
9-3a(1)1	0.2	0.2	0.7	2.7	1.8	94.3
9-3a(2)1	0.2	0.2	1.0	1.7	2.1	95.2
9-3a(3)1	0.5	0.5	0.9	2.7	2.3	93.1
11-1a(1)1	0.5	0.4	0.5	2.2	1.8	94.6
11-1a(1)4	0.0	0.4	0.9	2.3	1.8	94.8
11-1a(1)9	0.2	0.3	0.6	1.9	2.5	94.5
11-3b(1)1	0.2	0.2	0.8	2.0	2.0	94.7
11-3b(1)5	0.3	0.3	0.9	1.9	1.2	95.3
11-3b(1)8	0.6	0.5	1.1	2.1	1.2	94.6

A typical XRD scan and the refinement fitting curve are shown in Figure 8.14 for the concrete sample 9-1b(1)4.

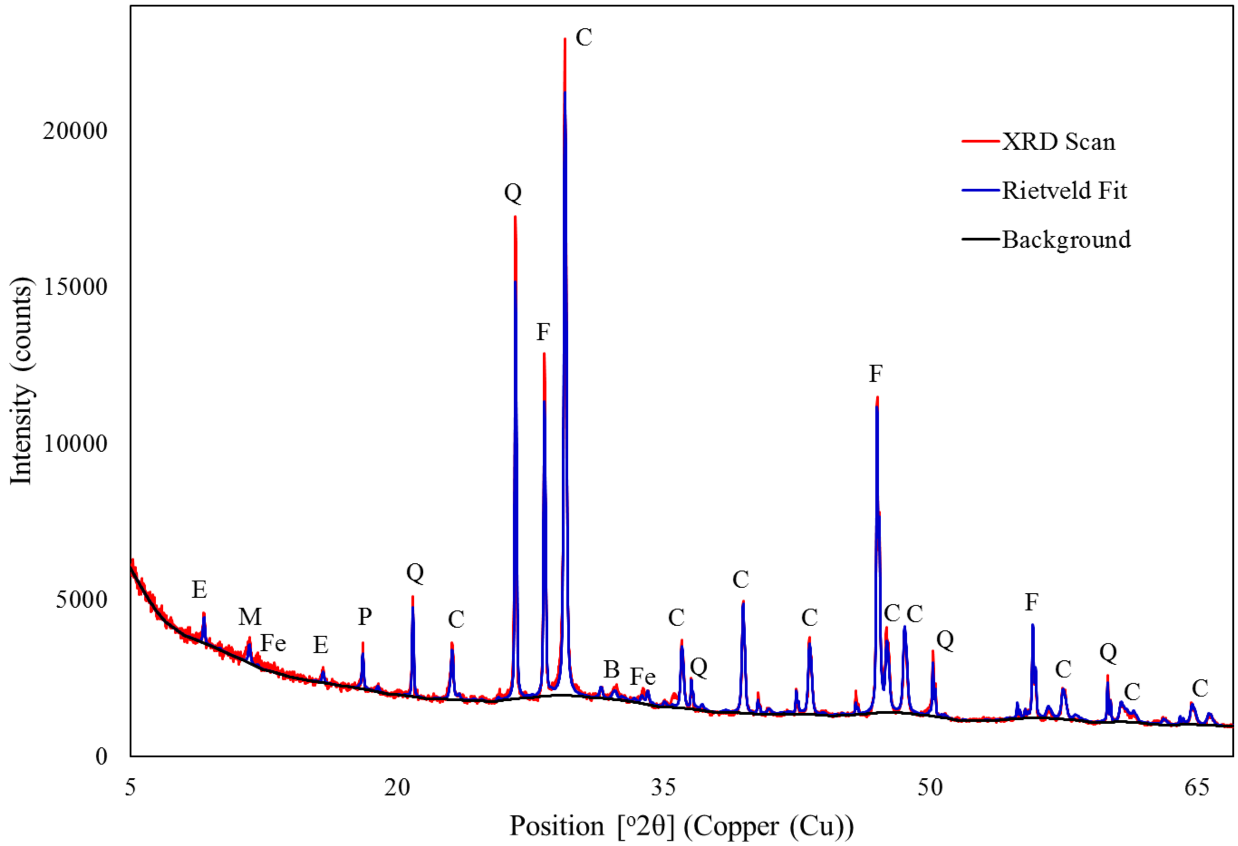


Figure 8.14 XRD scan and refinement fitting of the concrete sample 9-1b(1)4. C – calcite, Q – quartz, B – belite, Fe – ferrite, P – portlandite, E – ettringite, M – monocarboaluminate.

The data in Table 8.3 was also plotted against depth for each crystalline phase (Figures 8.15-8.17) to show spatial variations, if any; the strength profiles from the next section have been included as well for reference. The strength versus amorphous content is plotted in Figure 8.18.

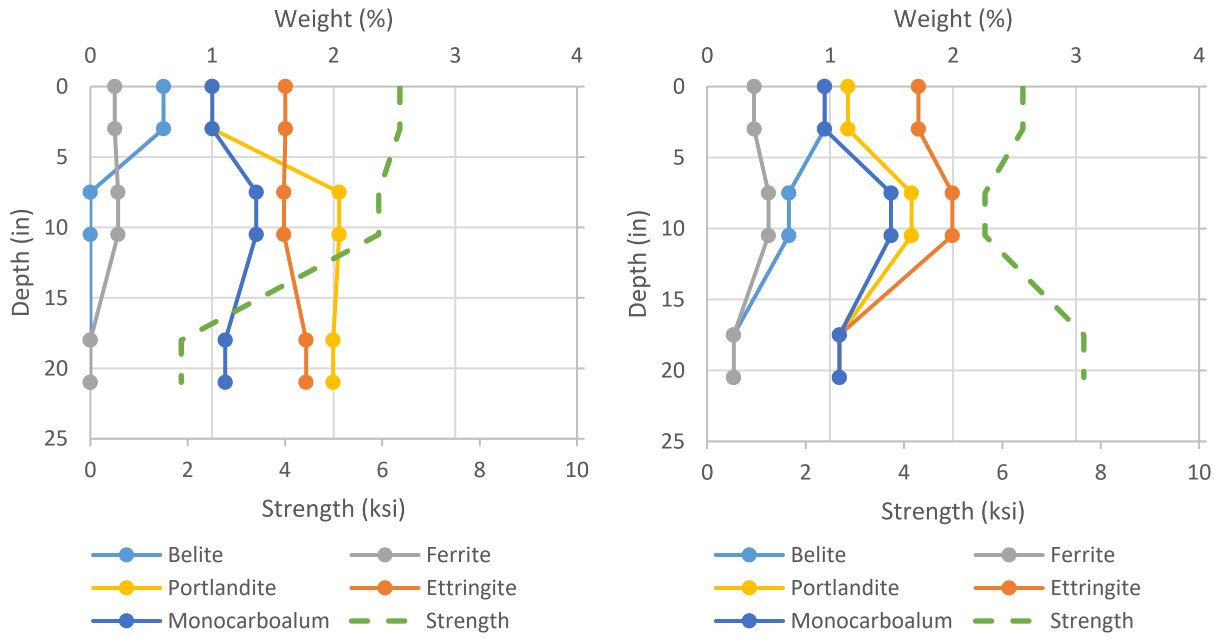


Figure 8.15 Weight percentage of crystalline phases by depth 6-1a outside (left) 6-3b inside (right); water shaft.

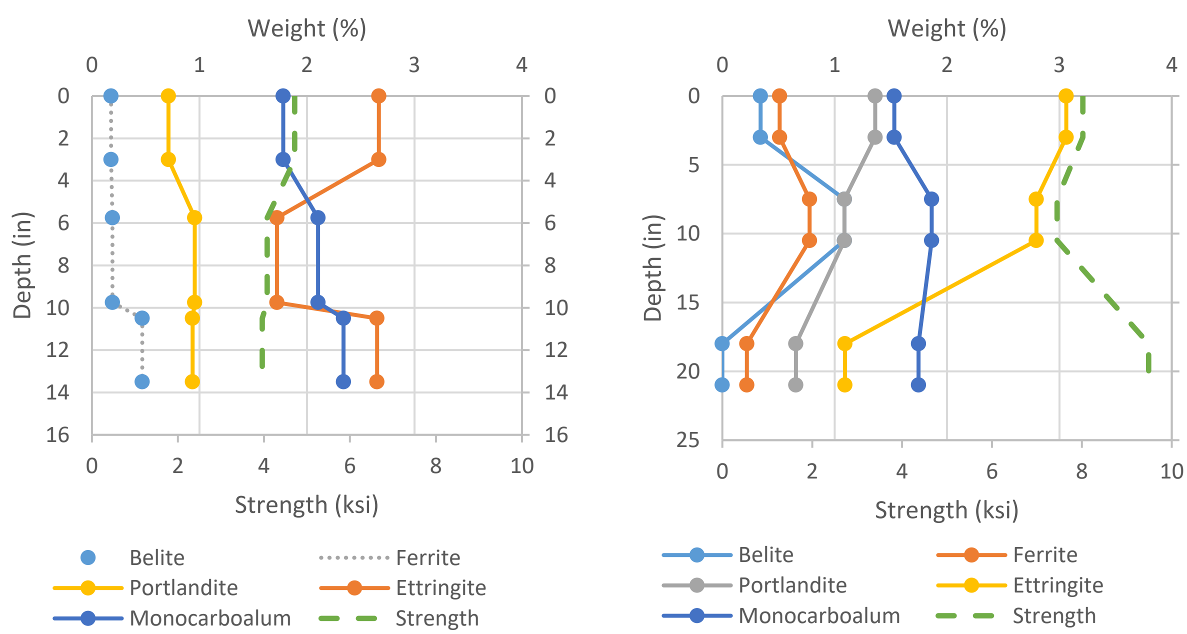


Figure 8.16 Weight percentage of crystalline phases by depth 9-3a outside (left) 9-1b inside (right); bentonite shaft.

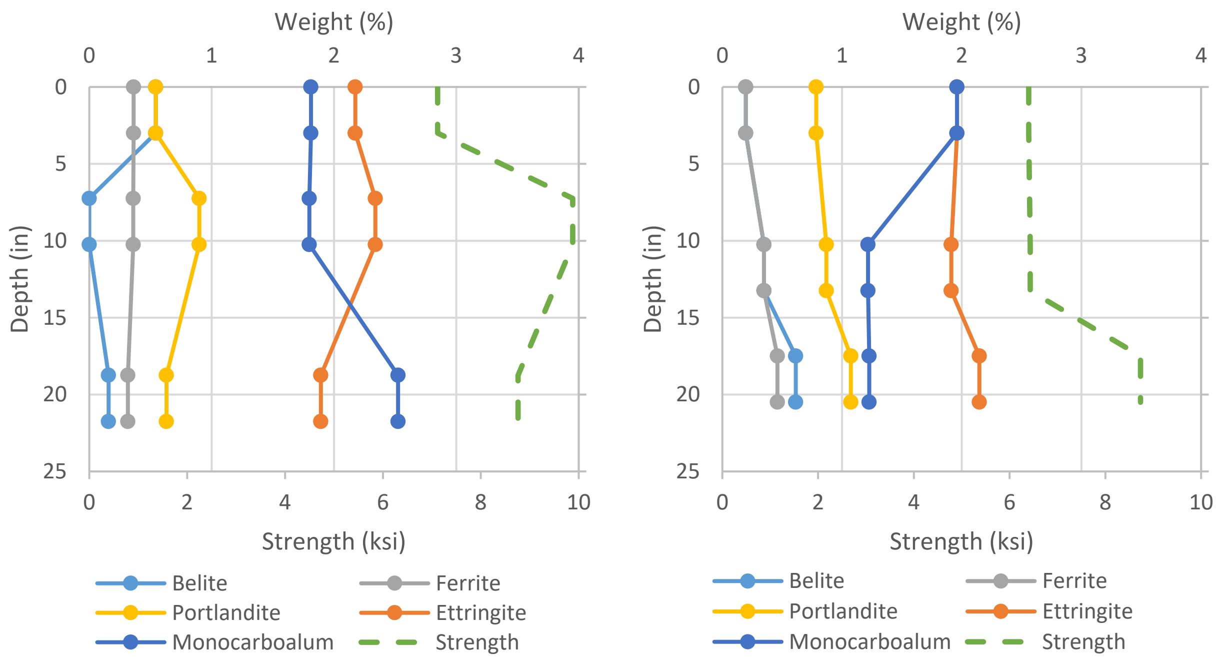


Figure 8.17 Weight percentage of crystalline phases by depth 11-1a outside (left) 11-3b inside (right); polymer shaft.

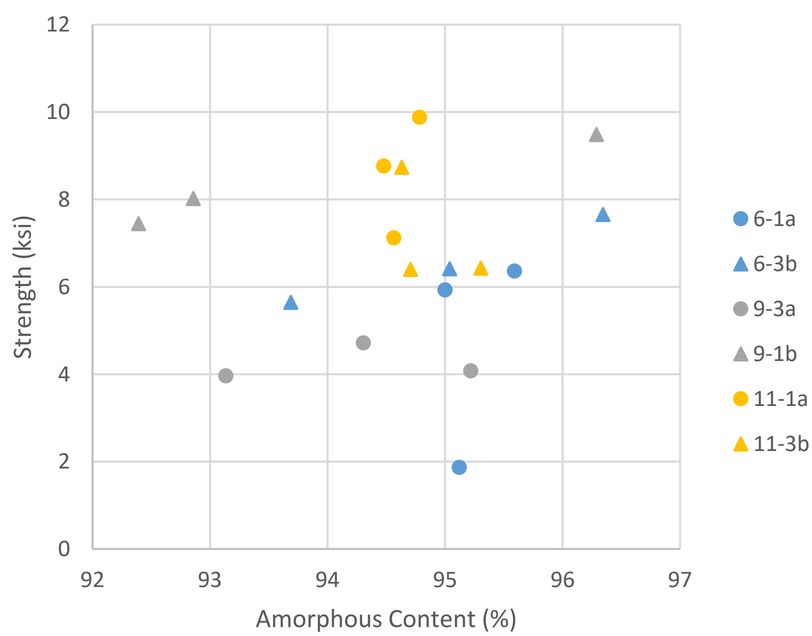


Figure 8.18 Weight percentage of amorphous content by strength.

The average XRD data was then plotted for each core against strength (Figures 8.19-8.21); hence, there are two data points per shaft, one for inside the reinforcement cage and one for the cover region.

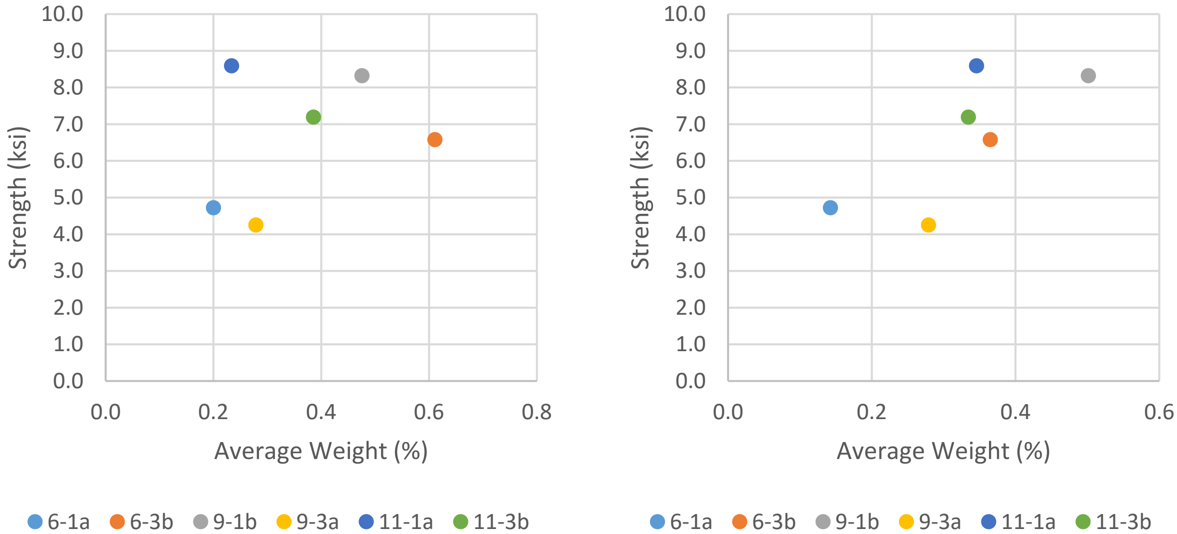


Figure 8.19 Average weight percentage vs. average strength for belite (left), average weight percentage vs. average strength for ferrite (right).

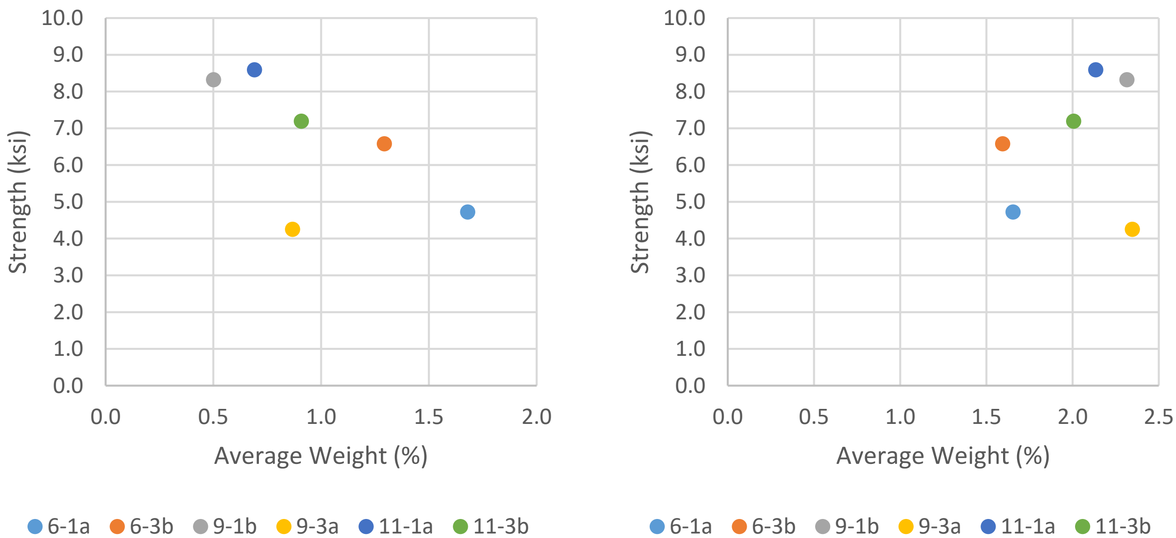


Figure 8.20 Average weight percentage vs. average strength for portlandite (left), average weight percentage vs. average strength for ettringite (right).

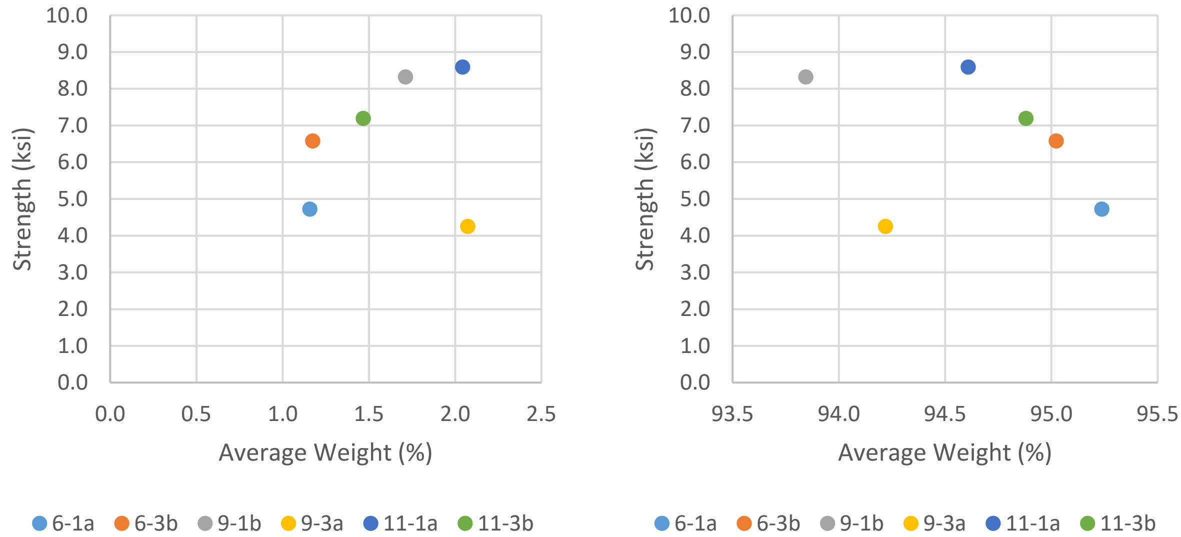


Figure 8.21 Average weight percentage vs. average strength for monocarboaluminate (left), average weight percentage vs. average strength for amorphous content (right).

While no trend was indicated for belite, ferrite showed a possible linear trend between all slurry types. Portlandite, ettringite, and monocarboaluminate, all seemed to show linear trends when excluding the cover-region bentonite core (9-3a), the strength of which was significantly lower compared to the others, suggesting the bentonite cover specimen was somehow different.

When excluding the cover-region bentonite core, portlandite seemed to support a trend of a higher strength with a lower weight percentage. Ettringite and monocarboaluminate showed the opposite trend with a lower strength correlating to a lower weight percentage. Interestingly, even though the cover-region bentonite core produced the most ettringite and monocarboaluminate, its strength was the lowest.

The amorphous content seemed to possibly support a linear trend, excluding bentonite (cover and center) cores. Strengths of samples from polymer- and water-slurry shafts showed a linear trend with higher strengths for lower amorphous contents, potentially indicating a higher degree of reaction.

In short, Figures 8.19-8.21 suggest correlations existed when the cover-region sample of the bentonite shaft was excluded. However, the magnitude of variation was large, making these trends insignificant and/or coincidental.

8.5 Unconfined Compression Strength Testing (cores from shafts 6, 9, and 11)

Cores taken from locations inside and outside the reinforcing cage from all three test shafts were also tested for unconfined compression strength. As core diameters were 1 in., the lengths of the cores were cut to 2 in. to provide suitable length-to-diameter (l/d) ratios. The strength of the cores was not expected to be truly representative of $f'c$ due to the relative dimensions of the aggregate and the shaft diameter; however, the goal was to identify variations. The unconfined compression strength profiles for all six cores are shown in Figure 8.22 for shafts 6, 9, and 11. Shafts 6 and 9 (water and bentonite) showed a decrease in cover strength relative to the core concrete strength. Shaft 11 (polymer) was relatively unaffected.

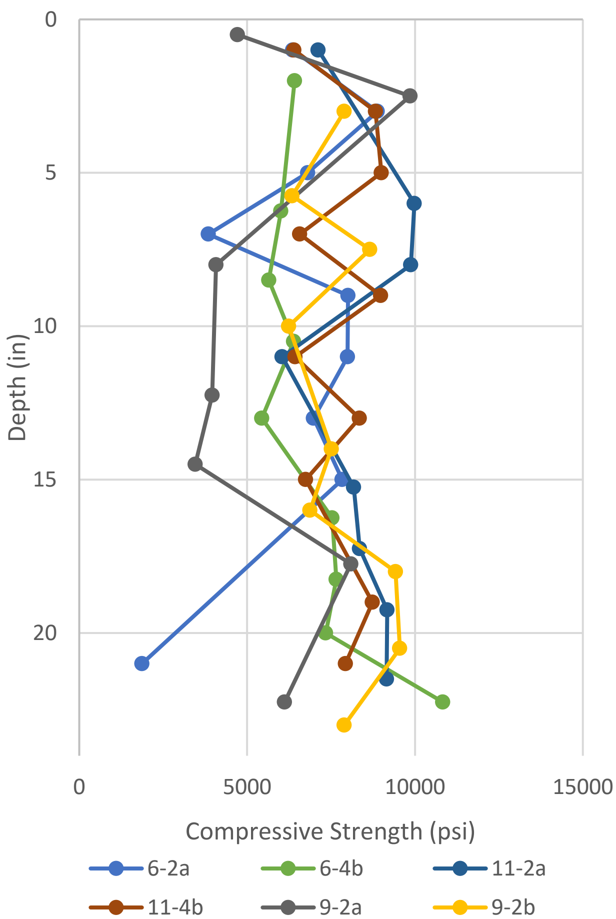


Figure 8.22 Compressive strength profiles.

8.6 Bentonite Effects on Concrete Strength

8.6.1 Literature Search for Bentonite Effects on Concrete Strength

In 2011, a study was completed at the Federal University of Technology in Akure, Nigeria. This study tested and examined the effects of bentonite powder on ordinary portland cement with a variety of mix proportions. For testing, 150- x 150- x 150-mm cubes were prepared in wooden molds. The bentonite used in this study was manufactured by Changsha May Shine Chemical Co., Ltd; in Hunan, China. Aggregates used were natural sand as fine aggregate and 19- mm granite as coarse aggregate.

Several mix design proportions were used; 1:2:4, 1:3:6, and 1:1.5:3 (cement:sand:course aggregate). These were completed at two water-to-cement ratios (w/c), 0.7 and 0.8. For each w/c ratio, there was a cement control, a 10% bentonite blend, a 20% bentonite blend, and a 30% bentonite blend by cement mass. Bentonite control mixes were also completed for each design proportion using a w/c of 1.5. The bentonite control did not include cement powder.

Figures 8.23 to 8.28 show the various compressive strengths versus curing time. By quick inspection, it can be seen that the addition of bentonite caused strength reduction. For example, when looking at the 1:2:4 w/c of 0.7 mix 28-day strength, the overall strength reduction from the control for 10%, 20%, and 30% was 39%, 46%, and 56%, respectively. Table 8.4 lists the strength reduction percentage for each mix.

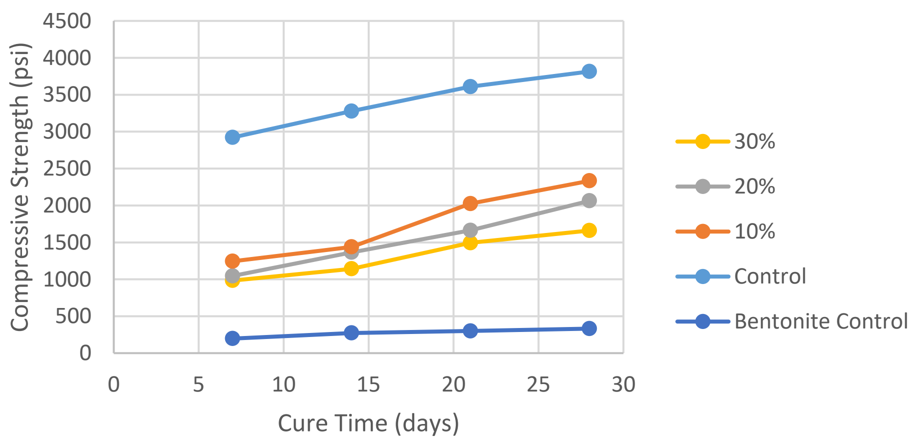


Figure 8.23 Strength versus cure time for mix proportion 1:2:4 at w/c of 0.7.

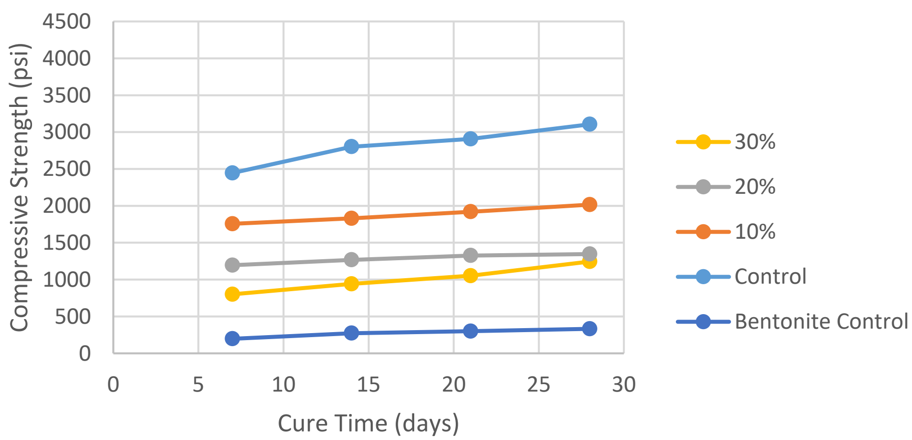


Figure 8.24 Strength versus cure time for mix proportion 1:2:4 at w/c of 0.8.

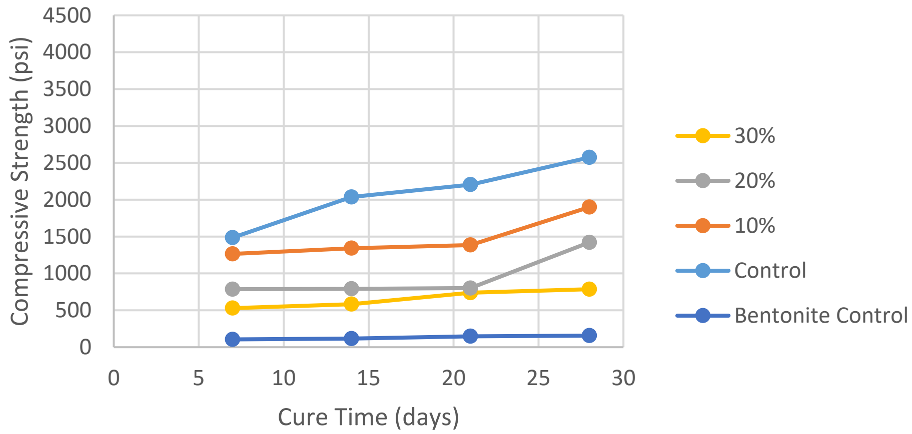


Figure 8.25 Strength versus cure time for mix proportion 1:3:6 at w/c of 0.7.

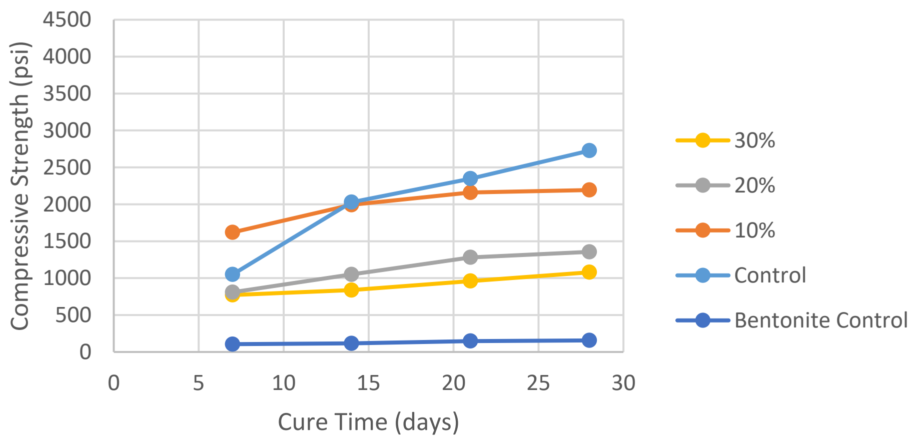


Figure 8.26 Strength versus cure time for mix proportion 1:3:6 at w/c of 0.8.

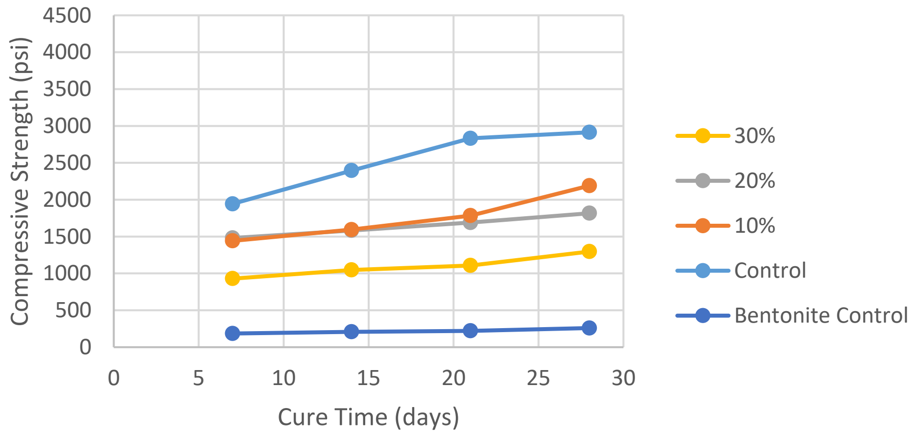


Figure 8.27 Strength versus cure time for mix proportion 1:1.5:3 at w/c of 0.7.

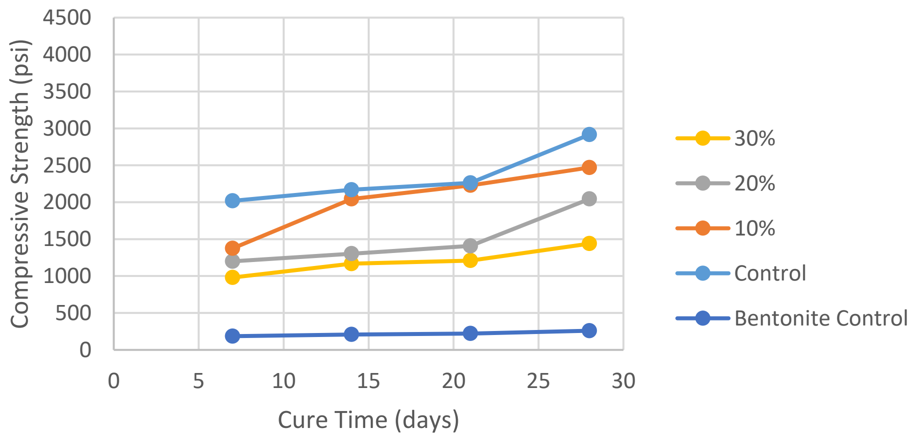


Figure 8.28 Strength versus cure time for mix proportion 1:1.5:3 at w/c of 0.8.

Table 8.4 Percentage of strength reduction per mix.

1:2:4 w/c 0.7		1:2:4 w/c 0.8	
Mix	Strength Reduction	Mix	Strength Reduction
10%	39%	10%	35%
20%	46%	20%	57%
30%	56%	30%	60%
1:3:6 w/c 0.7		1:3:6 w/c 0.8	
Mix	Strength Reduction	Mix	Strength Reduction
10%	26%	10%	20%
20%	45%	20%	50%
30%	69%	30%	60%
1:1.5:3 w/c 0.7		1:1.5:3 w/c 0.8	
Mix	Strength Reduction	Mix	Strength Reduction
10%	25%	10%	15%
20%	38%	20%	30%
30%	56%	30%	51%

The strengths for the mixes decreased with bentonite content. This decrease ranged from 15% to 69% for the various mix designs. From this data, it was concluded that the addition of bentonite to concrete decreases its strength. The magnitude of the decrease depends on mix proportions and percentage of cement replaced.

8.6.2 Cement Paste Cube Testing

In order to gain a deeper understanding of the interaction occurring in the cover region between bentonite slurry and cement during the casting process, paste cubes were prepared where bentonite slurry and cement paste were intentionally mixed at varying proportions. The mixes included a pure paste (control), a 25 volume percent replacement of paste with bentonite slurry, and a 50 volume percent replacement of paste with bentonite slurry.

Sample Preparation (Cube Mixes)

The prepared cubes were 2 in. x 2 in. x 2 in. and were cast in brass molds. The paste was made with a water-to-cement (w/c) ratio of 0.4 for all mixes. The bentonite slurry was made using PureGold Gel from Cetco with a target viscosity of 33 sec/qt. This called for 0.5 lb addition of bentonite to 1 gallon of water, resulting in a water-to-bentonite (w/b) weight ratio of 16.

The bentonite slurry was made using the following steps:

- Measure out the needed amount of water (Figure 8.29)
- Add soda ash to water to raise pH to 9-10 pH per manufacturer recommendations (6.9 grams/3 gallons of water) and mix (Figures 8.30 and 8.31)
- Measure out needed amount of bentonite powder and slowly add to water mixture, mix until homogenous (Figure 8.32)
- Measure viscosity using Marsh funnel (Figure 8.33)

Figure 8.29 The water being measured out for the slurry.

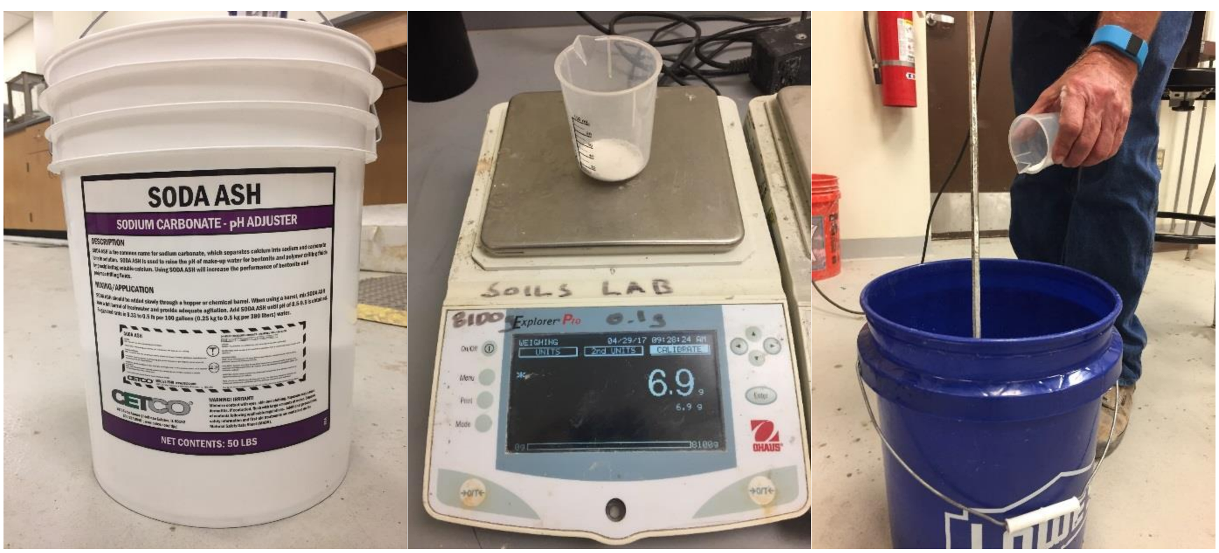


Figure 8.30 The soda ash (left) being measured out (middle) and added to the water (right).

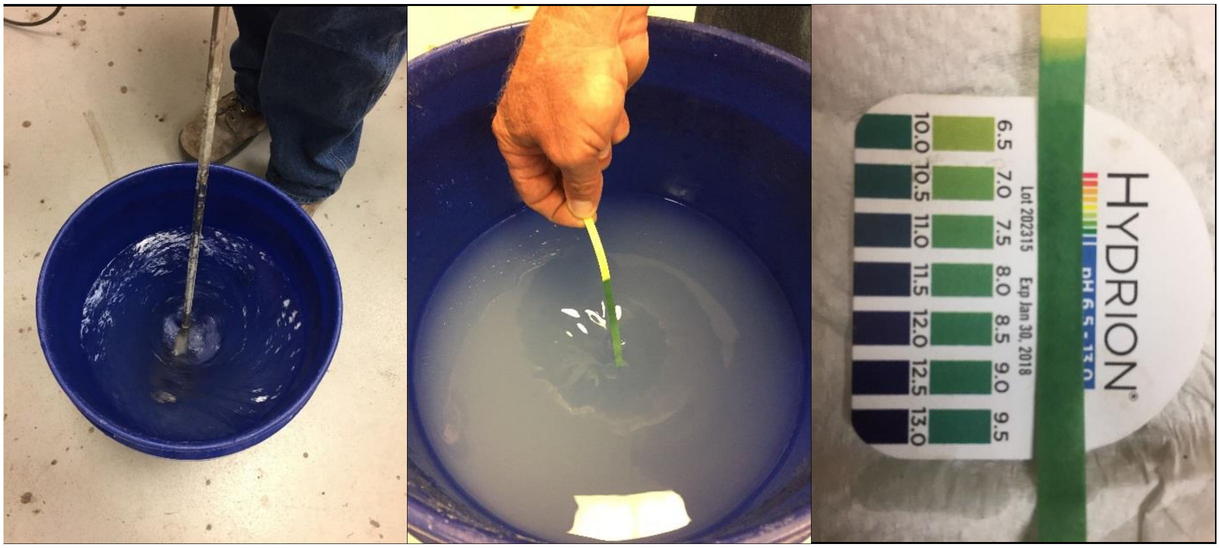


Figure 8.31 The pH being tested to ensure the water is at 9-10 pH.

Figure 8.32 The bentonite powder slowly being added to the water (left) to form the slurry (middle and right).

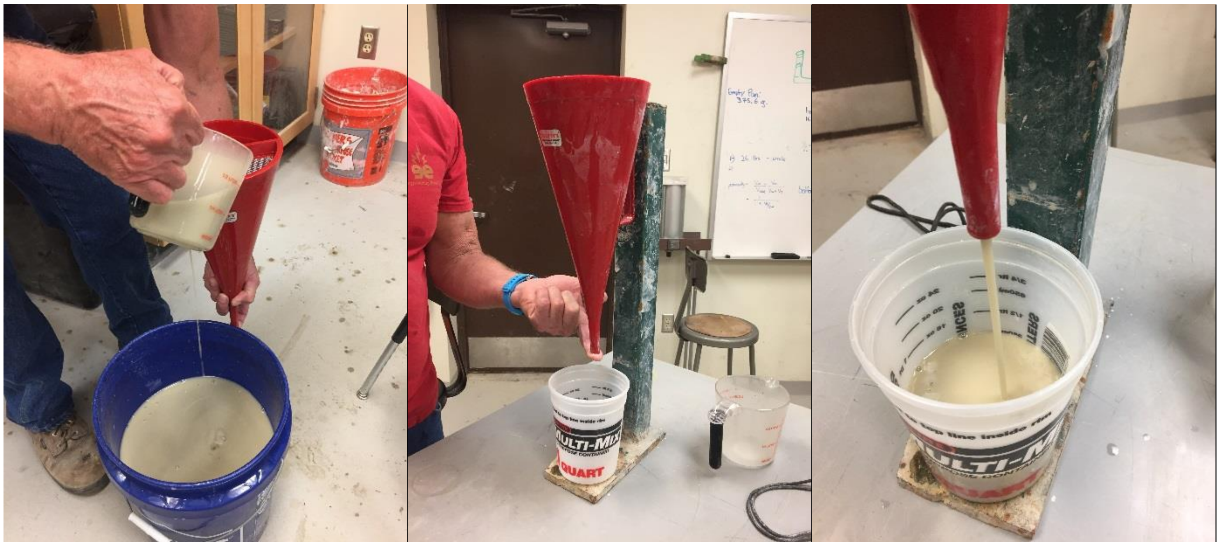


Figure 8.33 Testing of the bentonite slurry viscosity using the Marsh funnel.

The bentonite slurry was prepared 24 hours prior to use to allow for full hydration. For the paste, type I/II cement and deionized water were used. Mixing was done in accordance with ASTM C 305-12 section 7 for paste mixes and C109-08 for the tamping procedure. Note that mixing procedure does vary as the addition of bentonite slurry had to be taken into account and there is no ASTM for this step.

The general procedure for all mixes is as follows:

- Weigh out all components of mix (cement, deionized water, bentonite slurry if applicable)
- Grease molds using WD-40 (Figure 8.34)
- Follow mixing procedure from ASTM C 305-12 section 7 for paste mixes
- All water was placed into mixing bowl
- The cement was added to the water and allowed to sit for 30 seconds for the absorption of the water
- The mixer was turned on for 30 seconds at speed 1 (slow)
- The mixer was stopped and the bowl was scraped as needed for 15 seconds
- The mixer was turned on for 60 seconds at speed 2 (medium)
- If adding bentonite slurry, add slurry and mix on speed 1 for an additional 30 seconds
- Fill cube molds (Figure 8.34) with paste and follow tamping procedure from C109-08 (Figure 8.35)
- Move full cube molds to moisture cabinet for storage
- After 3 days remove specimens from mold
- Test for 3-, 7-, and 28-day strengths following C109-08 qualifications for loading rate

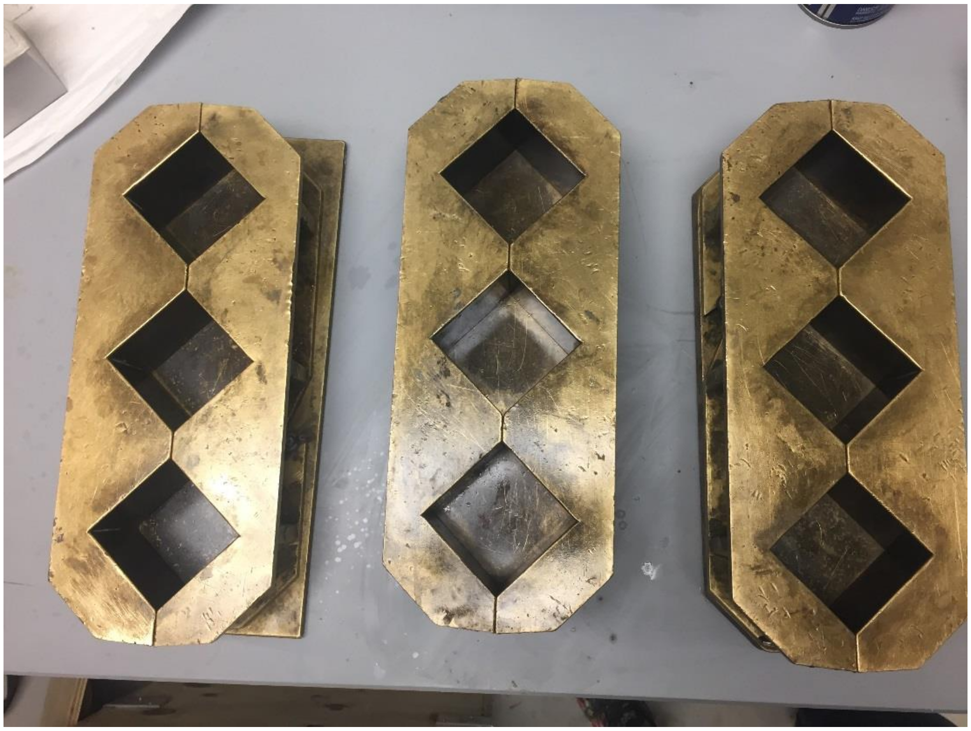


Figure 8.34 Greased cube molds used for construction of samples.

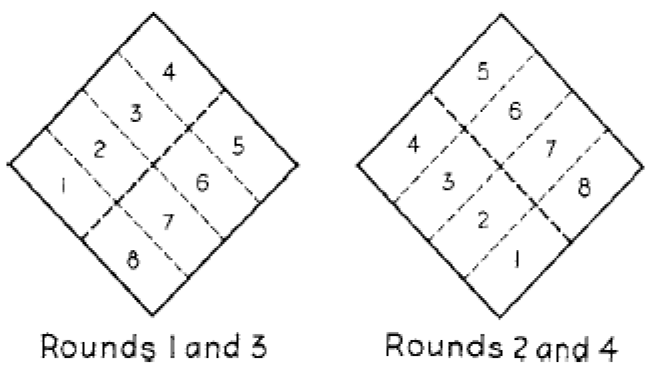


Figure 8.35 Tamping order followed from ASTM C109 (ASTM, 2008).

Mix Designs

Mix 1: Control

The control mix consisted only of water and cement to form the paste (Figure 8.36). Mixing proportions were designed for a 0.4 w/c ratio and can be seen in Table 8.5. The mixing procedure for mix 1 follows ASTM C305-12 section 7 as there were no other variables to take into account for this mix design (Figure 8.37).

Table 8.5 Mix proportions for mix 1.

	Design		Actual	
Ingredient	lb	grams	lb	grams
Water	1.812	821.91	1.812639	822.2
Cement	4.53	2054.77	4.530053	2054.8
Bentonite Slurry	0	0	0	0

Figure 8.36 All measured out ingredients for mix 1.

Figure 8.37 Mixing the paste.

Mix 2: 25% slurry replacement by volume

The second mix was a 25% replacement by volume of paste with the bentonite slurry. The paste was still made at a 0.4 w/c ratio; however, adding the bentonite slurry raised the water-to-solid ratio (w/s) ratio to 0.615. The mixing proportions for mix 2 can be seen in Table 8.6. The viscosity of the bentonite slurry at time of mixing was 36 sec/qt.

Table 8.6 Mix proportions for mix 2.

	Design		Actual	
Ingredient	lb	grams	lb	grams
Water	1.36	616.89	1.360251	617
Cement	3.4	1542.21	3.400185	1542.3
Bentonite Slurry	0.80445	364.89	0.804466	364.9

The mixing procedure for mix 2 varied slightly as there is no ASTM specification that gives a mixing procedure that takes into account the addition of slurry. ASTM C305-12 was followed as close as possible. The mixing procedure was as follows:

- Measure out all ingredients (Figure 8.38)
- All water was placed into mixing bowl
- The cement was added to the water and allowed to sit for 30 seconds for the absorption of the water
- The mixer was turned on for 30 seconds at speed 1 (slow)
- The mixer was stopped and the bowl was scraped as needed for 15 seconds
- The mixer was turned on for 60 seconds at speed 2 (medium)
- Bentonite slurry was added to the cement paste and mixed at speed 1 (slow) for 30 seconds (Figure 8.39)

Following this, the paste was distributed to the molds in thirds, where the tamping procedure shown in Figure 8.35 was used.


Figure 8.38 All measured out ingredients for mix 2.


Figure 8.39 Bentonite slurry being added to mix 2.

Mix 3: 50% replacement by volume

The third mix was a 50% replacement by volume of cement paste with bentonite slurry. Again, the paste was made with a 0.4 w/c ratio. The addition of the bentonite slurry raised the w/s ratio to 1.03. Thus, a decrease in strength was expected because of the higher w/s ratio. Mixing proportions for mix 3 can be seen in Table 8.7. The viscosity at time of mixing for the bentonite slurry was 36 sec/qt.

Table 8.7 Mixing proportions for mix 3.

	Design		Actual	
Ingredient	lb	grams	lb	grams
Water	0.906	410.95	0.906319	411.1
Cement	2.265	1027.39	2.265027	1027.4
Bentonite Slurry	1.609	729.83	1.609373	730

The mixing procedure for the third mix followed the procedure for mix 2 and was as follows:

- Measure out all ingredients (Figure 8.40)
- All water was placed into mixing bowl
- The cement was added to the water and allowed to sit for 30 seconds for the absorption of the water (Figure 8.41)
- The mixer was turned on for 30 seconds at speed 1 (slow)
- The mixer was stopped and the bowl was scraped as needed for 15 seconds
- The mixer was turned on for 60 seconds at speed 2 (medium)
- Bentonite slurry was added to the cement paste and mixed at speed 1 (slow) for 30 seconds (Figure 8.42)

Following this, the paste was distributed to the molds in thirds, where the tamping procedure shown in Figure 8.35 was again used.

Figure 8.40 All measured out ingredients for mix 3.

Figure 8.41 The addition of cement to water (left) and the mixing process (right).

Figure 8.42 The addition of bentonite slurry to the mix (left) and the mixing process (right).

While the paste w/c stayed constant in all three mixes at 0.4, the overall w/c ratio increased in mixes 2 and 3 due to the water in the bentonite slurry. Table 8.8 shows mixing proportions, including the overall w/s ratio, and shows the actual amount of bentonite powder and water in each mix.

Table 8.8 Mixing proportions for mixes 1-3.

Mix 1: Control						
Paste			Bentonite Solution			Mix
Water (g)	Cement (g)	w/c	Water (g)	Bentonite Powder (g)	w/b	w/s
821.9	2054.8	0.4	0.0	0.0	0.0	0.4

Mix 2: 25% Bentonite Solution Replacement						
Paste			Bentonite Solution			Mix
Water (g)	Cement (g)	w/c	Water (g)	Bentonite Powder (g)	w/b	w/s
616.9	1542.2	0.4	344.3	20.6	16.7	0.6

Mix 3: 50% Bentonite Solution Replacement						
Paste			Bentonite Solution			Mix
Water (g)	Cement (g)	w/c	Water (g)	Bentonite Powder (g)	w/b	w/s
411.0	1027.4	0.4	688.6	41.2	16.7	1.0

Results

In total, 27 cubes were made, 9 per mix. For testing, cubes 1-3 were tested at 3 days, 4-6 at 7 days, and 7-9 at 28 days. Cubes were tested and results analyzed in accordance with ASTM C109-08. Figure 8.43 shows the testing of a control sample.

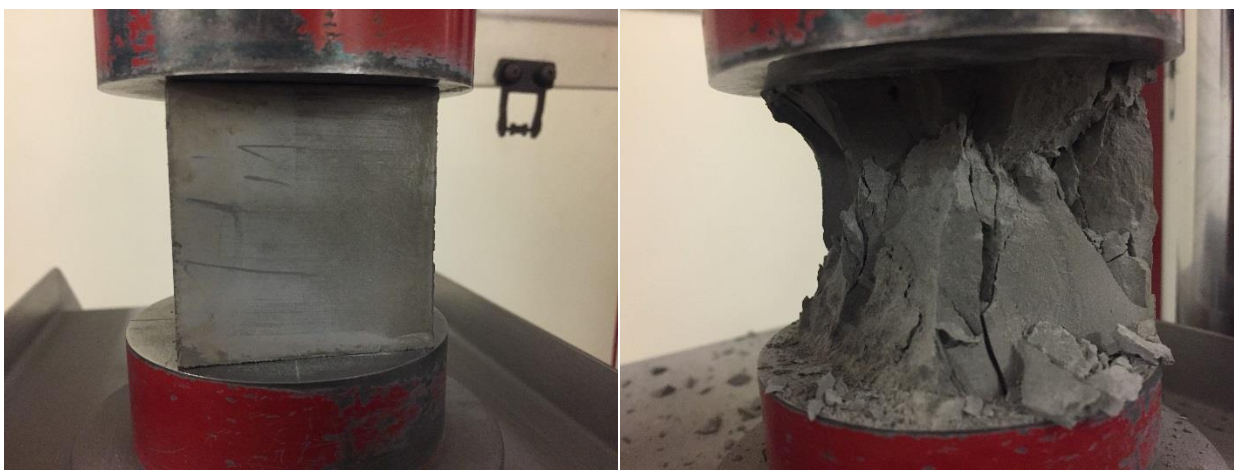


Figure 8.43 A sample tested from mix 1 at 7 days.

Mix 1 Control

Table 8.9 shows all compressive strength results from mix 1. Again, cubes 1-3 represent compressive strength at 3 days, 4-6 at 7 days, and 7-9 at 28 days. Figure 8.44 displays average compressive strength of the 3-, 7-, and 28-day breaks.

Table 8.9 Mix 1 results; Cubes 1-3 3-day, 4-6 7-day, 7-9 28-day.

Cube	Age (days)	Weight (g)	Thickness (in)	Width (in)	Height (in)	Compressive Strength(psi)	% diff	
1	3	251.61	1.994	1.966	1.955	6480	4%	okay
2	3	251.52	2.031	1.968	1.975	6070	-2%	okay
3	3	246.63	2.006	1.961	1.931	6080	-2%	okay
4	7	246.17	2.003	2.001	2.011	8160	3%	okay
5	7	246.03	2.002	2.005	2.003	7830	-2%	okay
6	7	246.52	2.013	2.007	2.034	7880	-1%	okay
7	28	242.83	2.017	1.992	1.998	9220	0%	okay
8	28	244.73	2.019	2.001	1.993	9830	6%	okay
9	28	244.78	2.027	1.997	2.055	8700	-6%	okay

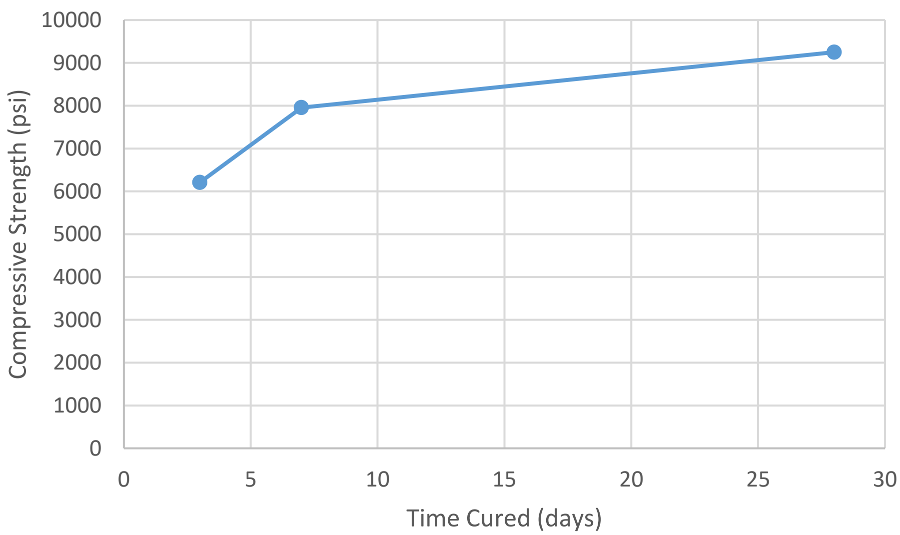


Figure 8.44 Mix 1 average compressive strength versus curing time.

Mix 2: 25% replacement by volume

Table 8.10 displays the results of all breaks from mix 2. The average strengths for 3, 7, and 28 days of curing have been plotted in Figure 6.45. The average of two cubes was taken instead of three for samples 1-3 tested at 3 days and 7-9 tested at 28 days. This is because one or more cubes exceeded the acceptable 8.7% difference range. Averaging two resulted in the cubes being within the acceptable 7.6% difference range. Note that from ASTM C109-08, 8.7% is the maximum permissible difference between 3 cubes and 7.6% is the maximum between 2 cubes.

Table 8.10 Mix 2 results; Cubes 1-3 3-day, 4-6 7-day, 7-9 28-day.

Cube	Weight (g)	Thickness (in)	Width (in)	Height (in)	Compressive Strength (psi)	% diff		% diff	
1	215.95	2.017	1.924	1.94	1960	-4%	okay	3%	okay
2	218.6	2.009	1.942	1.932	2310	13%	average two		
3	210.74	2.009	2.007	2.01	1860	-9%	okay	-3%	okay
4	214.6	2.009	2.011	2.01	3340	8%	okay		
5	208.45	2.019	2.005	2.014	2920	-5%	okay		
6	210.19	2.018	2.005	2.019	2990	-3%	okay		
7	193.63	2.015	2.007	2.009	3470	2%	okay	-3%	okay
8	189.63	2.017	1.986	1.998	3030	-11%	average two		
9	199.59	2.021	2	2.009	3720	9%	average two	3%	okay

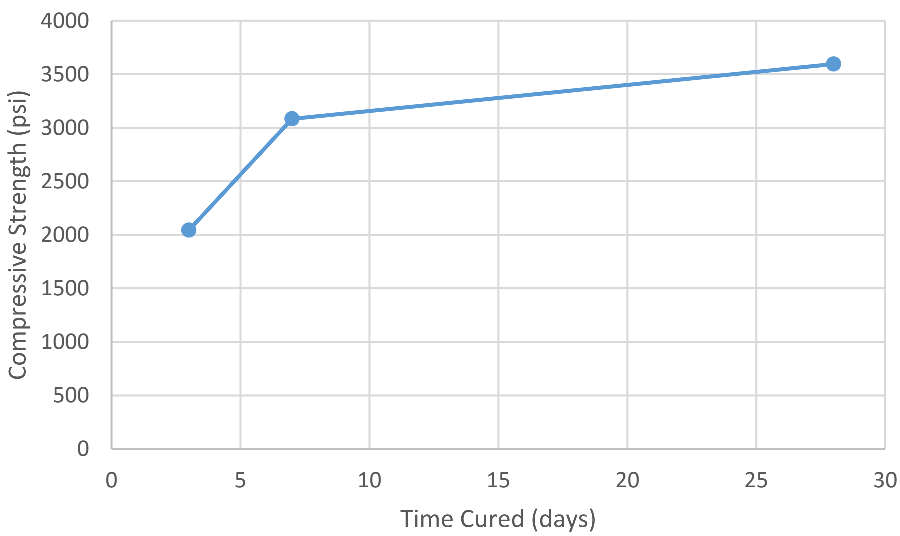


Figure 8.45 Mix 2 average compressive strength versus curing time.

Mix 3: 50% replacement by volume

The results of strength testing for mix 3 can be seen in Table 8.11. Figure 8.46 shows the average compressive strength versus curing time for mix 3. The average of two specimens was taken for cubes 4-6 and 7-9, as one or more cubes fell outside the 8.7% permissible difference. While samples 7-9 satisfied the 7.6% allowable difference for two cubes, samples 4-6 did not meet this qualification. The smallest percent difference observed between two cubes was 9%, thus those two were averaged for samples 4-6 at 7 days.

Table 8.11 Mix 3 results; Cubes 1-3 3-day, 4-6 7- day, 7-9 28-day.

Cube	Weight (g)	Thickness (in.)	Width (in)	Height (in)	Compressive Strength (psi)	% diff		% diff		% diff	
1	179.34	1.994	1.839	1.863	460	-4%	okay				
2	186.58	2.006	1.906	1.879	500	4%	okay				
3	185.62	1.998	1.997	2.02	480	0%	okay				
4	176.96	2.002	2.002	2.005	530	-19%	average two	-10%	average two		
5	194.05	2.007	2.003	2.003	780	19%	average two			9%	average two
6	183.6	2.004	2.007	2.007	650	-1%	okay	10%	average two	-9%	average two
7	171.03	2.01	2.002	2.009	1300	9%	okay	3%	okay		
8	175.78	2.006	2.018	2.022	1070	11%	average two				
9	173.79	2.001	2.01	2.018	1220	2%	okay	-3%	okay		

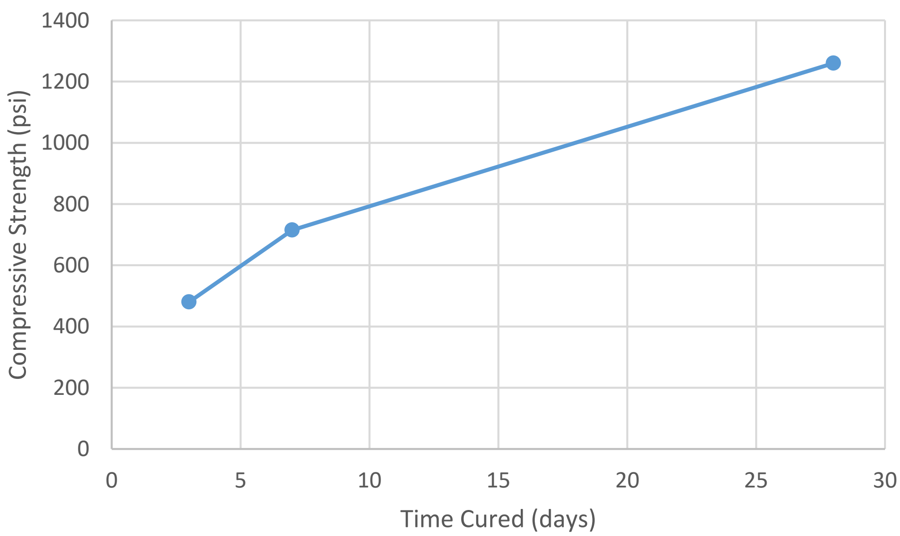


Figure 8.46 Mix 3 average compressive strength versus curing time.

8.6.3 Discussion

As seen in Figures 8.44-8.46, all mixes follow the general trend of strength gain over time, which is expected. However, when the results are plotted on one graph, the differences between the mixes become apparent (Figure 8.47). The control mix is much stronger than the other two containing bentonite slurry. When looking at 3-day strengths, mix 2, 25% replacement, has a 67.1% decrease in strength and mix 3, 50% replacement, shows a 92.3% decrease in strength.

For an accurate comparison, supplemental control paste mixes were prepared with equivalent w/c ratios. Their mix proportions can be seen in Appendix G. The first set of supplemental mixes was to see what the strength of paste would be at an equivalent w/c ratio. Thus, two mixes were made with the targeted w/c ratios of 0.62 and 1.07. These mixes were composed of deionized water and the same type I/II cement used for mixes 1-3. For assessing the effect of bentonite at similar w/c ratios, the inclusion of 1.3% bentonite, by weight of solids, caused a 3-day strength reduction of 20.1% and 7-day of 11.7%. Similarly, the inclusion of 3.9% bentonite, by weight of solids, caused a 3-day strength reduction of 59.1% and a 7-day of 64.2%.

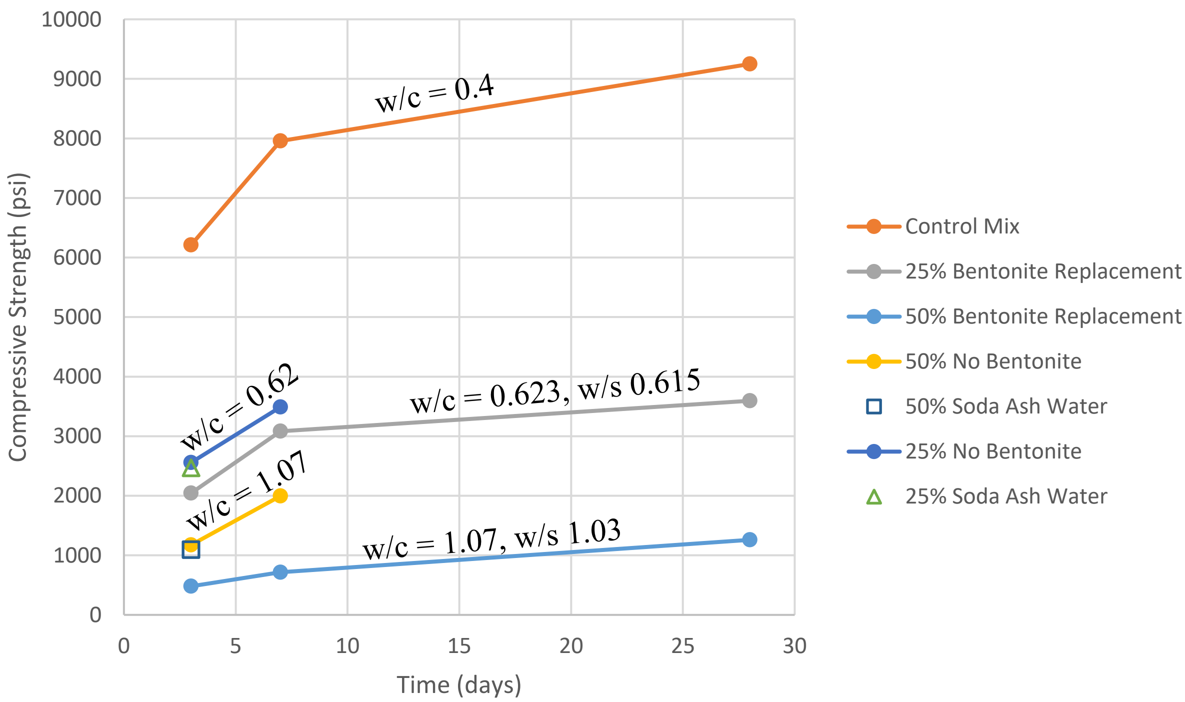


Figure 8.47 Compressive strength versus time for all cube mix designs, mix 1-3 as well as supplemental mixes.

Next, the effect on strength of soda ash, which is mixed into the water used to hydrate the bentonite slurry, was investigated. To test this, instead of deionized water, the soda ash water, which is typically only used with the bentonite, was utilized. It can be seen in Figure 8.47 that there is almost no difference in strength when the cubes were cast with the soda ash water. A 3.4% loss in strength was seen in mix 2, and a 6.8% loss was seen for mix 3. While there is a small loss, the soda ash can be eliminated as the main cause of strength loss.

These tests identify bentonite as the primary source for strength loss. Based on the proportions used in mixes 2 and 3, bentonite powder only accounted for 0.8% and 1.9% of the mix, respectively. This test demonstrates that even a small amount of bentonite can cause a drastic strength loss.

8.7 Summary

For this chapter, testing was completed on cores from miniature shaft specimens, as well as paste cubes. The testing included strength testing, mercury intrusion porosimetry (MIP), and x-ray diffraction (XRD).

The current results from shafts 6, 9, and 11 (water, bentonite, and polymer, respectively) for MIP indicate that 1) the cover concrete was more porous than the interior concrete, and 2) the shafts cast in water were the least porous, while those cast in polymer slurry were the most porous.

In terms of the XRD testing on shafts 6, 9, and 11, possible linear trends were seen for several crystalline phases and the amorphous content. For portlandite, ettringite, and monocarboaluminate, these trends were only significant when data from the cover of the bentonite core (9-3a) were excluded.

The second phase of strength testing used paste cube mixes, testing the interaction between the bentonite and cement that occurs in the cover region. Results showed that as the amount of bentonite increased, the strength of the sample decreased. Supplementary testing was performed to ensure the strength loss was indeed being caused by the bentonite. While the additional water accounts for the majority of strength loss, 43.8% for mix 2 with 25% replacement and 67.8% for mix 3 with 50% replacement, the inclusion of bentonite accounts for an additional 11.7% loss for mix 2 and 64.2% for mix 3 at 7 days. This resulted in a total strength loss of 61.2% and 91% for mixes 2 and 3, respectively.

While the cube study provided conclusive results in identifying the source of strength loss (bentonite), XRD testing was still performed on the cube samples. Unfortunately, the XRD testing did not show conclusive results as the peaks best matched that of water and the tests hardly showed any bentonite (pure bentonite was tested also so it could be identified). XRD was performed as it eliminates the difficulty of removing the aggregate from the samples, which was an issue seen from the samples prepared from the coring process.

Concrete surface roughness affects both the corrosion protection and side shear performance of drilled shafts cast in a support-fluid-filled excavation. Prior to this study, many researchers assumed filter action of mineral support fluid (slurry) particles depositing on to the side walls was responsible for degradation of side shear performance, this perceived cause was even extended to impervious soils where filtration action would not occur. Brown (2002) reported a 50 percent reduction in side shear in bentonite cast shafts when compared to polymer or cased shaft. Exposure times were limited to one hour. Figure 9.1 shows the results of load tests performed on two subject shafts, one cast in bentonite support fluid and the other cast in polymer support fluid. The maximum load for the polymer-cast shaft is nearly twice the load born by bentonite cast shaft. What this proves, again is not all of the support fluid is expelled from the shaft during concreting. Rather, a trapped support fluid interface remains.

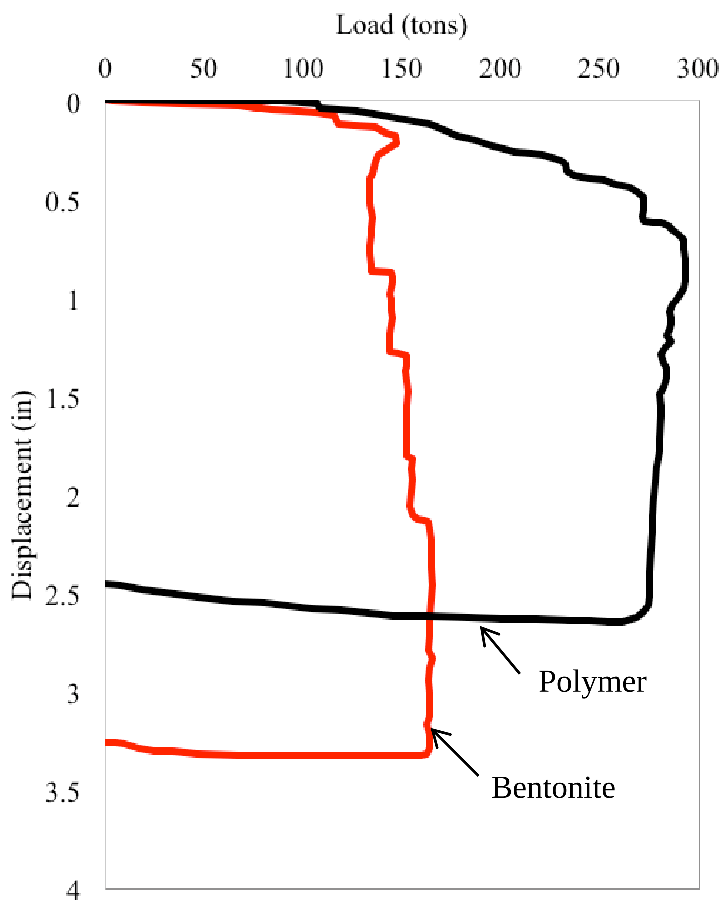


Figure 9.1 Effects of support fluid on side shear

Bowen, 2013 showed bentonite support fluid trapped between the surface of the concrete and the steel casing. This was revealed upon casing removal which would be inaccessible in field/ underwater conditions such as the conditions for Brown (2002.) Figure 9.2 shows a bentonite-cast shaft immediately after form removal. Mattressing is visible on the surface and there is a thick layer of bentonite caked to the concrete (Figure 9.3). After pressure washing the surface of the test shafts, the mattressing become more apparent and the surface roughness was revealed. The state of deterioration on the shaft shown in Figure 9.4 was not uncommon in bentonite cast shafts.

Figure 9.2 Visible mattressing upon form removal.

Figure 9.3 Bentonite caked to the concrete surface trapped between concrete and casing.

Figure 9.4 Visible mattressing after pressure washing

Quantifying the surface roughness then became a primary focus of this work as surface roughness was suspected to be an indication of concrete quality (cover protection), which would then make external physical assessment a direct link to the internal health of the structural steel reinforcement and the structure as a whole. Visual inspection of the lab cast specimens immediately revealed variations in the surface texture that were in part a by-product of trapped support fluid between the outward flowing concrete and the simulated casing. Figures 9.5-9.7 demonstrate the differences seen between a water, bentonite, and polymer cast shafts.

Figure 9.5 Shaft 6: f’c 4358 psi; drilled shaft mix; water cast; smooth; faint channeling

Figure 9.6 Shaft 9: f’c 4530 psi; drilled shaft mix; 50 sec/qt bentonite; rough; with well-defined creases

Figure 9.7 Shaft 11: f'c 4530 psi; drilled shaft mix; 65 sec/qt polymer; smooth; no creases

Much of the focus of this set of tests revolved around identifying physical surface features that may indicate concrete flow problems and potential adverse effects on the longevity of the structure. To this end, the surface condition of the shafts was an obvious variable for consideration. One method to classify the surface condition of the individual shafts was through approximate quantification of the surface roughness. This was accomplished by two means: physical and digital.

9.1 Physical Surface Void Volume Determination

The surface roughness was assessed by measuring the surface void volume through a procedure developed for this project wherein a 6-inch square of the shaft surface was filled with a putty of a known density and finished in such a manner as to approximate a smooth shaft surface. With the weight of the putty required a void volume was calculated and then extrapolated to approximate the void volume for the entire shaft surface. This assumed the original outer surface of the shaft was still in part present and severe radial reduction had not taken place.

Initially a testing area grid was created by cutting a 6-inch by 6-inch hole out of a sheet of thick plastic sheeting (Figure 9.8). This hole was used as a template for setting the limits of the testing area.

Figure 9.8 Template construction

Next a beaker with a known volume was filled with drywall putty. Then all of the testing equipment was placed in a tray and the weight was recorded (Figure 9.9). The template was then placed on the surface of the shaft (Figure 9.10). This test was conducted two times on each shaft, care was taken to place the template on an area representative of the overall surface condition.

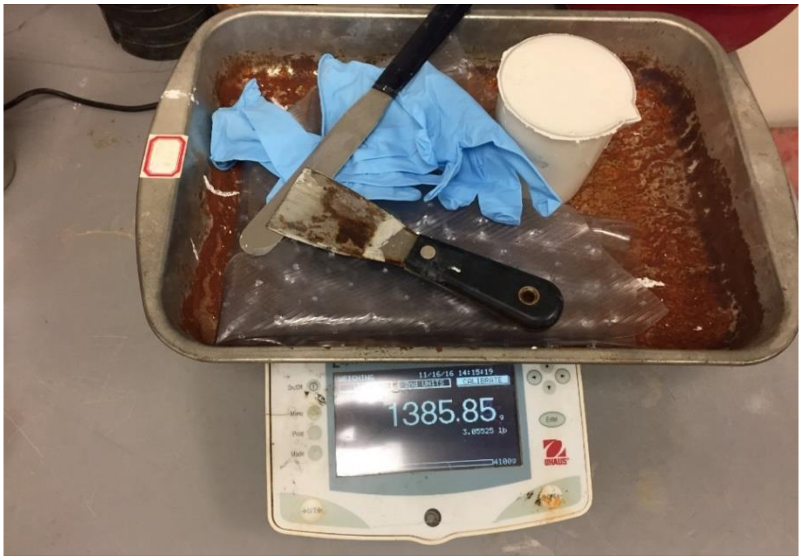


Figure 9.9 Weighing of testing equipment. Tray includes: plastic template, rubber gloves, two putty knives, beaker full of drywall putty.

Figure 9.10 Testing template placed on shaft surface.

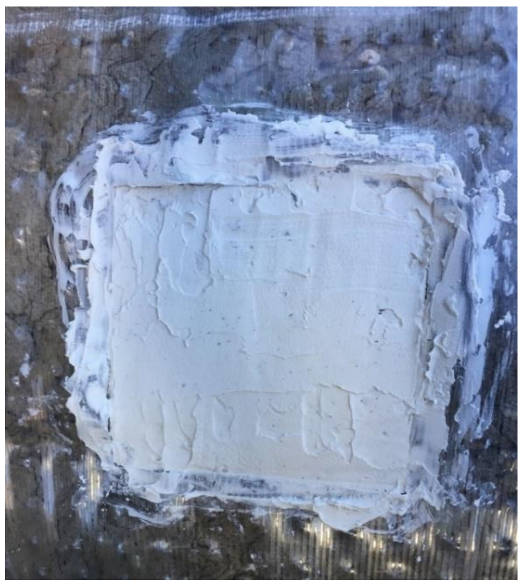


Figure 9.11 Testing template filled to approximate a void free surface

After putting on the rubber gloves, the putty knife was used to contour the concrete surface in the template area to approximate a smooth, void-free surface (Figures 9.11 and 9.12). Special care was taken to ensure all putty stayed on the tools, in the tray or on the concrete surface. The gloves, template, beaker with the remaining putty and all of the tools were then put back into the tray and reweighed (Figure 9.13). The difference in weight was then converted to a volume using the calculated density of the putty. Data tables can be found in Appendix H.

Figure 9.12 Testing the second location on shaft 10

Figure 9.13 Weighing testing equipment after test is complete

9.2 **Digital Surface Void Volume Determination**

Shafts 1-24 were scanned on one side using an Artec Eva 3D scanner (Figure 9.14). An Artec Eva is a structured light device used to make texture and accurate 3D models of medium-sized objects such as the selected portions of the shafts. The handheld scanner captures precise measurements in high resolution, and can be used for multiple applications. Shafts 25-58 were scanned on one side using an Artec Leo 3D scanner (Figure 9.15). The chief difference in the two systems is that the Artec Leo offers onboard automatic processing.

Figure 9.14 Scanning shaft 20

Figure 9.15 Scanning shaft 56

A structured-light 3D scanner is a scanning device for measuring the three-dimensional shape or surface of an object using an established light pattern and a camera system. Structured light is a method of projecting a known pattern of light on to a surface. The manner and extent in which this pattern of light is distorted or altered when it strikes a surface allows the system to calculate information about the depth and surface detail of the objects in the scan. Factory calibration ensures a minimum accuracy of 100 microns. Figure 9.16 shows a sample of the data collected, this surface profile was generated using up to 15 million data points. Selected shaft renderings can be found in Appendix I.

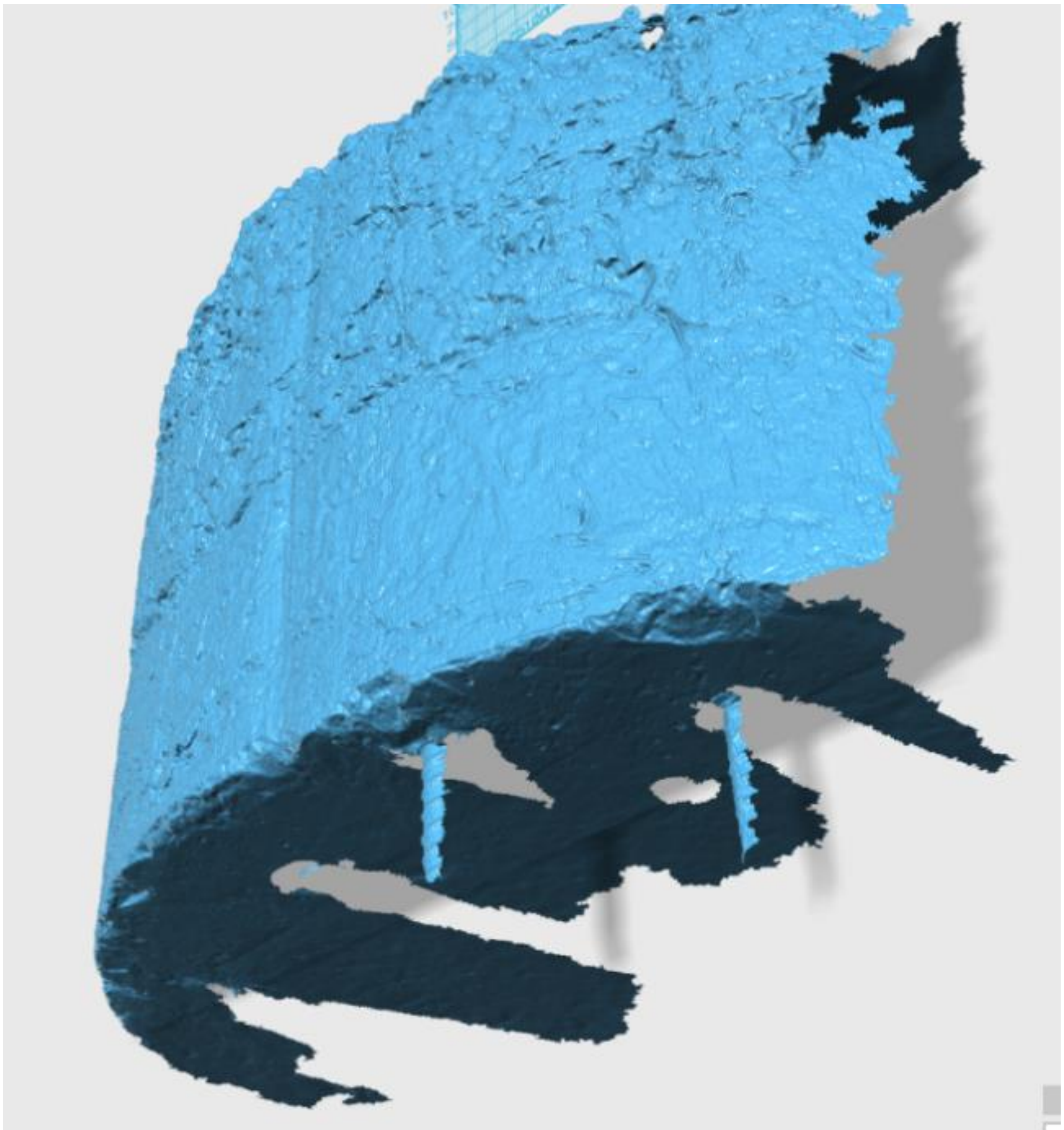


Figure 9.16 Scan data detail

9.3 Results

The analysis of the digital scans is a lengthy process requiring enormous computational capabilities. This section will provide a brief summary of the process used to analyze the scan data, but the full step-by-step procedure can be found in Appendix J. The scan data for each shaft has been collected and stored in an object file. As figures 9.17, 9.18, and 9.19 show, the data represents the surface condition of each shaft with a high degree of detail.

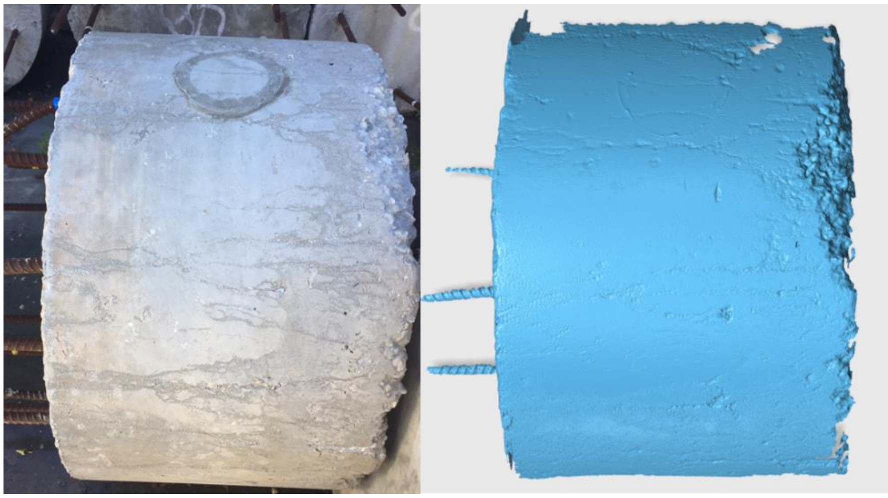


Figure 9.17 Shaft 6 photo and scan comparison

Figure 9.18 Shaft 9 photo and scan comparison

Figure 9.19 Shaft 11 photo and scan comparison

Due to constraints in the AutoCAD software, the data had to be reduced prior to analysis. Each file was reduced from its original size of 10-30 million points to a manageable 2.5 million points using a free, web-sourced, software called MeshLab and reducing by no more than 50% in each stage of the process. The surface mesh was then exported from MeshLab as an xyz point file capable of being uploaded into AutoCAD Civil3d for analysis.

By attaching the point file to a topographic surface definition, a surface was then generated for each shaft. Choosing a 1-foot by 1-foot representative test area allowed for the direct determination of surface roughness without extrapolation. If the surface showed signs of creasing, then care was taken to represent those creases in the test area. A profile was taken horizontally across the center of each shaft to identify the surface shape and condition (a circumferential slice). The ideal surface profile was approximated using a three-point curve in the profile creation tools menu (Figure 9.20). This function allowed for variation in the shaft radius due to the nature of construction and any distortion away from circular that may have been induced by the flexible forms.

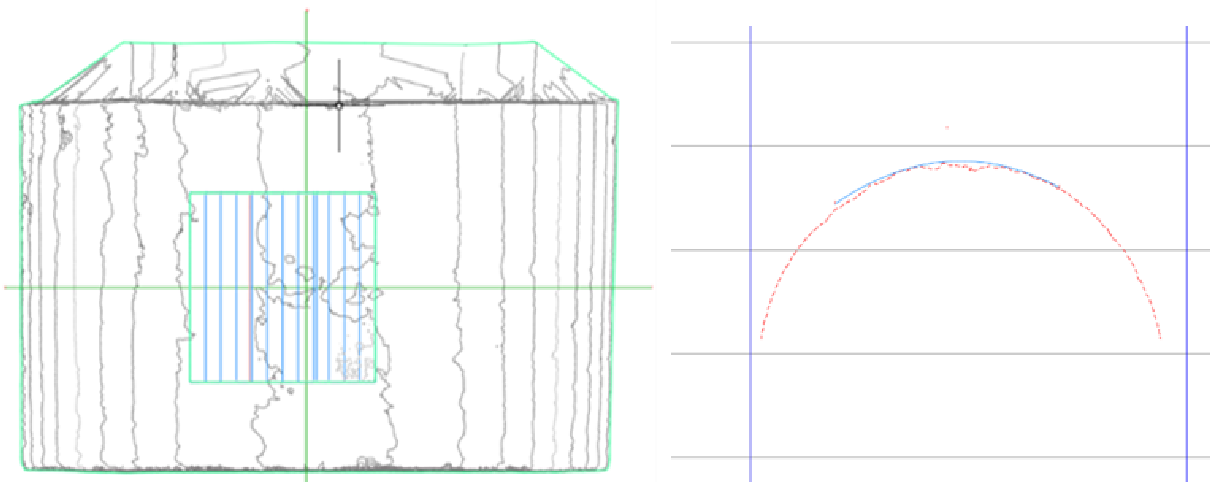


Figure 9.20 Shaft 9 surface data analysis plan and profile

The ideal surface profile elevation was set to match the highest points in the region of interest (Figure 9.21). Using a flat assembly with perfectly vertical side slopes, the ideal surface profile was used to create a corridor and the surface from the corridor was compared with the existing surface using cut and fill tools to generate the digital surface void volume data. Surface roughness was computed for each shaft (Figure 9.22) and the results were then tabulated with the results from the physical testing (Table 9.1).

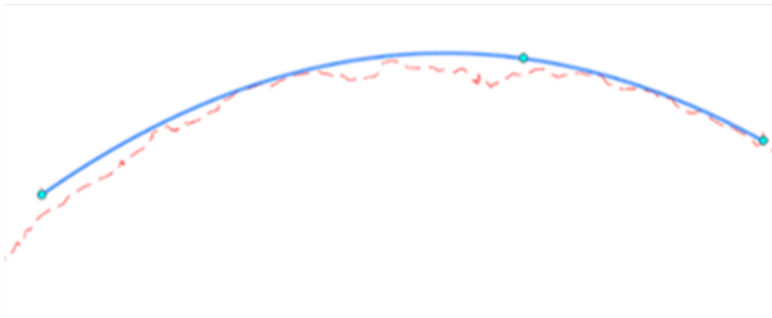


Figure 9.21 Shaft 9 existing and finished grade.

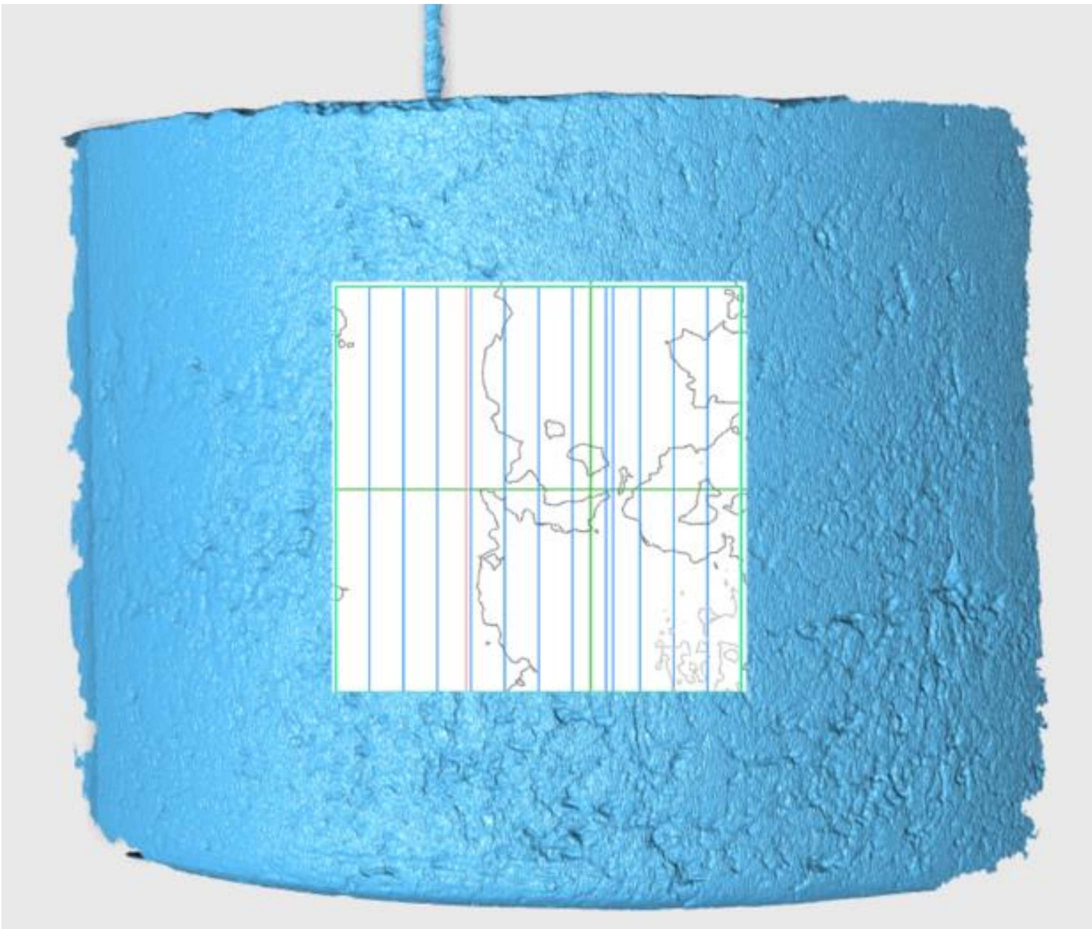


Figure 9.22 Shaft 9 Digital void volume determination.

Table 9.1 Surface roughness data summary.

Shaft #	Support fluid Type	Physical Surface Roughness (in.^{3}/ft^2)	Digital Surface Roughness (in.^{3}/ft^2)	Percent difference (%)
1	B44	26.93	27.36	2%
2	B105	22.35	20.64	-8%
3	B40	9.25	10.32	10%
4	B55	14.24	11.04	-29%
5	B90	11.54	12.24	6%
6	water	1.91	2.16	12%
7	B30	22.15	20.4	-9%
8	B40	9.79	10.08	3%
9	B50	11.83	12.24	3%
10	P90	24.56	23.04	-7%
11	P65	3.25	3.36	3%
12	P66	2.70	2.64	-2%
13	B30	7.68	6.96	-10%
14	B30	13.62	13.2	-3%
15	B56	11.97	10.8	-11%
16	P85	3.30	4.8	31%
17	P85	1.74	3.84	55%
18	water	2.16	2.64	18%
19	P60	1.97	2.88	32%
20	P130	1.49	2.64	44%
21	B40	26.27	18.24	-44%
22	water	1.35	1.44	7%
23	water	2.19	2.88	24%
24	B40		31.2	
25	A40	7.59	5.49	-38%
26	water	2.88	1.53	-88%
27	B40	6.83	7.47	9%
28	P90	2.36	3.33	29%
29	P50	6.16	5.4	-14%
30	B30	15.51	13.32	-16%
31	P98	2.22	1.908	-16%
32	water	1.75	1.89	8%
33	B39	5.54	7.47	26%
34	A39	7.25	8.37	13%
35	A200+	14.90	15.93	6%
36	P47	3.28	3.96	17%
37	water	2.29	3.24	29%
38	P55	6.82	5.67	-20%
39	P73	5.28	5.76	8%
40	B41	16.35	13.41	-22%

Shaft #	Support fluid Type	Physical Surface Roughness (in.^{3}/ft^2)	Digital Surface Roughness (in.^{3}/ft^2)	Percent difference (%)
41	B28	3.32	4.59	28%
42	A37	6.44	n/a	n/a
43	B37	7.57	7.38	-3%
45	B37	6.67	7.74	14%
46	water	3.81	2.25	-69%
47	water	2.93	3.42	14%
48	B39	5.86	5.85	0%
49	B31	4.63	3.69	-25%
50	P57	2.49	1.17	-113%
51	P90	1.60	1.35	-18%
52	A39	4.07	4.95	18%
53	P49	4.81	4.68	-3%
54	P58	3.92	3.24	-21%
55	P120	3.38	2.79	-21%
56	P85	3.94	4.59	14%
57	B40	6.23	5.04	-24%
58	water	3.65	2.193	-66%

Digital surface void volume determination resulted in a higher quantity than the physical surface void volume method used in most cases. The percent difference shown above illustrates the conservative nature of the physical method. This could be due to the reference ideal surface used in both cases as the datum. The physical void volume ideal surface is created without a template and as such is left to the judgement of the technician performing the test. The digital void volume uses the highest elevation along the selected profile to determine the ideal surface elevation. This method is also reliant on the judgement of the technician performing the analysis. That being said, Figure 9.23 shows the results of the two testing methods following a linear relationship thus supporting the validity of either when reviewed against the other.

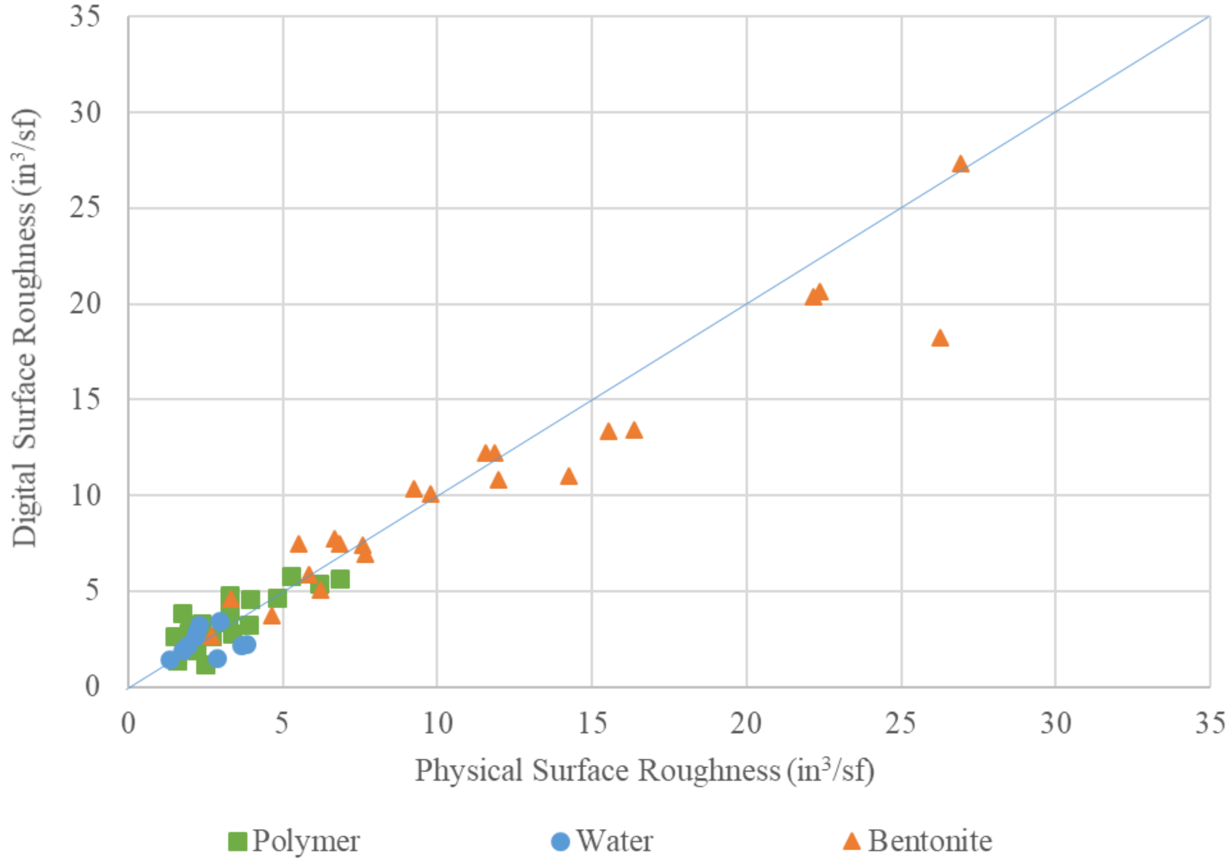


Figure 9.23 Digital surface roughness vs. physical surface roughness.

9.3.1 **Radius Reduction**

The surface roughness is more than just an indication of texture, it is a quantification of a volume of missing concrete. This volume results in an effective reduction in the radius of the shaft. In tables 9.2- 9.4 this radius reduction is tabulated by support fluid type. Also included is the correlating cover thickness reduction as a percentage of total cover.

Table 9.2 Radius reduction in bentonite cast shafts.

Bentonite		Surface Roughness	Equivalent Concrete Lost	Effective Radius Reduction	Cover Reduction
Shaft #	Viscosity				
	(s)	(in.^{3}/ft^2)	(in.^{3})	(in.)	(%)
1	44	27.36	601.7	0.1909	3.18%
2	105	20.64	453.9	0.1438	2.40%
3	40	10.32	226.9	0.0718	1.20%
4	55	11.04	242.8	0.0768	1.28%
5	90	12.24	269.2	0.0852	1.42%
7	30	20.4	448.6	0.1421	2.37%
8	40	10.08	221.7	0.0701	1.17%
9	50	12.24	269.2	0.0852	1.42%
13	30	6.96	153.1	0.0484	0.81%
12	66	2.64	58.1	0.0183	0.31%
15	56	10.8	237.5	0.0751	1.25%
21	40	18.24	401.1	0.1271	2.12%
27	41	7.47	164.3	0.0519	0.87%
30	30	13.32	292.9	0.0927	1.55%
33	39	7.47	164.3	0.0519	0.87%
40	41	13.41	294.9	0.0933	1.56%
41	28	4.59	100.9	0.0319	0.53%
43	37	7.38	162.3	0.0513	0.86%
45	37	7.74	170.2	0.0538	0.90%
48	39	5.85	128.6	0.0407	0.68%
49	31	3.69	81.1	0.0256	0.43%
57	40	5.04	110.8	0.0350	0.58%

Table 9.3 Radius reduction in polymer cast shafts.

Polymer Shaft #	Viscosity	Surface Roughness	Equivalent Concrete Lost	Effective Radius Reduction	Cover Reduction
	(s)	(in.^{3}/ft^2)	(in.^{3})	(in.)	(%)
11	65	3.36	73.9	0.0233	0.39%
12	66	2.64	58.1	0.0183	0.31%
16	85	4.8	105.6	0.0334	0.56%
17	85	3.84	84.4	0.0267	0.44%
19	60	2.88	63.3	0.0200	0.33%
20	121	2.64	58.1	0.0183	0.31%
28	59	3.33	73.2	0.0231	0.39%
29	46	5.4	118.8	0.0375	0.63%
31	98	1.908	42.0	0.0133	0.22%
36	47	3.96	87.1	0.0275	0.46%
38	55	5.67	124.7	0.0394	0.66%
39	73	5.76	126.7	0.0400	0.67%
50	57	1.17	25.7	0.0081	0.14%
51	90	1.35	29.7	0.0094	0.16%
53	49	4.68	102.9	0.0325	0.54%
54	58	3.24	71.3	0.0225	0.38%
55	120	2.79	61.4	0.0194	0.32%
56	85	4.59	100.9	0.0319	0.53%

Table 9.4 Radius reduction water cast shafts.

Water Shaft #	Viscosity	Surface Roughness	Equivalent Concrete Lost	Effective Radius Reduction	Cover Reduction
	(s)	(in.^{3}/ft^2)	(in.^{3})	(in.)	(%)
6	26	2.16	47.5	0.0150	0.25%
18	26	2.64	58.1	0.0183	0.31%
22	26	1.44	31.7	0.0100	0.17%
23	26	2.88	63.3	0.0200	0.33%
26	26	1.53	33.6	0.0106	0.18%
32	26	1.89	41.6	0.0131	0.22%
37	26	3.24	71.3	0.0225	0.38%
46	26	2.25	49.5	0.0156	0.26%
47	26	3.42	75.2	0.0238	0.40%
58	26	2.193	48.2	0.0152	0.25%

9.4 Chapter Summary

Physical and digital methods for evaluating surface roughness were developed and implemented on 52 large-scale, lab-cast drilled shaft specimens where concrete for each specimen was tremie-placed and support-fluid-displaced. The calculated surface roughness for each test method was tabulated for each shaft. The equivalent radius reduction was also computed and tabulated. The results of comparing the values with regards to support fluid type showed a marked distinction in surface roughness, with the greatest impacts being attributed to specimens cast in mineral slurry.

Chapter Ten: Field Exploratory Evaluation of Existing Bridges with Drilled Shaft Foundations

With extreme-event loading states often controlling pier/foundation designs for overwater bridges, there has been an almost complete change from bridge bents to cap and column footings. The net effect was to make all foundation elements (piles or shafts) work in concert to resist vessel impact loads. As a result, most new bridge piers have fully submerged foundation elements. Figure 10.1 shows the two variants comprising the east and west bound bridges in the Gandy Bridge corridor of Tampa Bay.

Figure 10.1 Newer cap and column pier design (left); older pile bent piers (right).

Overwater shaft construction employs steel casing through the water and embedded in the soil that allows the shaft concrete to be poured up to the cut-off elevation, which is often near or below sea level. This casing is left in place until the concrete has cured sufficiently to proceed with footing construction, and at which time the casing can be removed (cut off) down to the level of the mudline. This Task targeted overwater bridges where the casing was fully or partially removed to assess the shaft surface conditions. This approach was adopted in lieu of partial excavation around on-land shafts that would also reveal the shaft surface, but may have also required washing and been costly.

The approach was multi-faceted (1) identify an inventory/listing of bridges built on shafts, (2) obtain plan sets detailed enough to screen candidate bridges, (3) obtain biennial inspection reports complete with diver notes to focus on which shafts of which piers may be fruitful, and (4) conduct underwater evaluations of those bridges where the casing was in part removed, revealing the concrete surface. Ideally, candidate bridges would be constructed using all stabilization methods including: full length temporary casing (natural slurry), bentonite slurry, and attapulgite slurry. Recall from Chapter 3 that none of the 24 laboratory-cast samples were tremie-placed in attapulgite.

10.1 Bridge Identification

Florida is home to more than 12,000 bridges (FHWA, 2016), many of which are over water. Close coordination with District maintenance engineers, past and present central office personnel, bridge inspectors, and CEI consultants was required to draft a list of likely candidate bridges. Ongoing efforts to identify bridges that match the construction method in question have thus far produced a list of 14 bridges (Table 10.1, Figure 10.2).

Table 10.1 List of bridges reviewed to date.

Bridge Name	Bridge Number	Location	Year Built
Bridge of Lions	780074	St. Augustine, FL	1927
Clearwater Memorial Causeway	150244	Clearwater, FL	2005
Clearwater Pass Bridge	155522	Clearwater, FL	1995
Fuller Warren Bridge	720629*	Jacksonville, FL	1995
Gandy Bridge	100585	Tampa, FL	1924
John Ringling Causeway	170176	Sarasota, FL	1926
Overland Bridge	720627	Jacksonville, FL	2017
Santa Fe River Bridge	260112	Gainesville, FL	2002
SR2 Choctawhatchee Bridge	520145	Caryville, FL	1940
SR10 Choctawhatchee Bridge	520149	Caryville, FL	1927
SR20 Blountstown Bridge	470052	Blountstown, FL	1998
SR61 Lost Creek Bridge	590048	Wakulla, FL	1991
SR63 Ochlockonee Bridge	500124 - 500127	Ochlockonee River, FL	2001
Victory Bridge	530951	Chattahoochee, FL	1996

*Indicates main bridge span.

Not all necessary information could be obtained to warrant on-site investigations, but if any evidence suggested that exposed shaft concrete could be found, then those bridges were slated for underwater evaluation. A summary of each candidate bridge is provided for completeness.

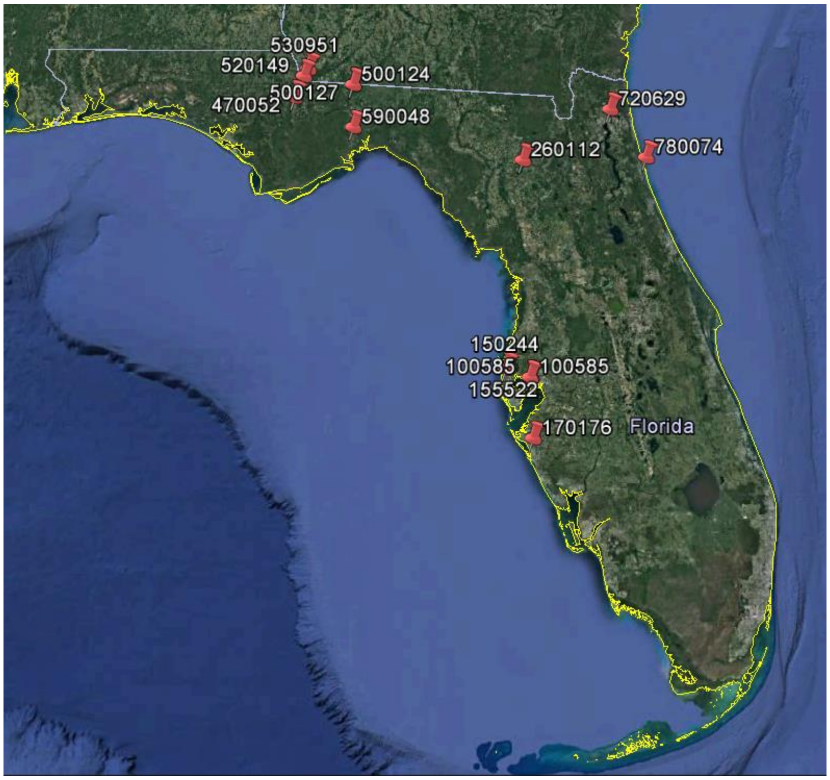


Figure 10.2 Bridge locations.

10.1.1 **Bridge of Lions**

A part of State Road A1A, the Bridge of Lions spans the intracoastal waters of St. Augustine, connecting Anastasia Island with downtown St. Augustine. The bridge is iconically known for the two lions that have guarded the bridge since its construction in 1927. These lions are Carrara marble Medici Lions that are identical to those in Loggia dei Lanzi in Florence, Italy. Prior to the bridge of lions there was a wooden bridge, built in 1895, known as the "South Beach Railroad Bridge" or as "The Bridge to Anastasia Island". This bridge was renovated in 1904 and was able to accommodate a trolley. Known as the father of the Bridge of Lions, Henry Rodenbaugh initiated the construction of the bridge in 1925 through his funding efforts. Completed in 1927 with its extravagant art and style, the Bridge of Lions has been regarded as a symbol of the nation's oldest city. The Bridge of Lions underwent an 80 million dollar renovation in 2006. A temporary bridge was constructed and the lions were removed for the time being. After work was done on the bascule towers and the steel girders, the Bridge of Lions was reopened in March of 2010 and the Lions were brought back in March of 2011, marking the completion of a long renovation project.

Table 10.2 Bridge of Lions.

Bridge Name	Bridge of Lions
Bridge Number	780074
Year Built	1927
Slurry Type	Mineral
Shaft Diameters	3 ft-8 ft

Bridge of Lions is a double-leaf bascule bridge that stands on 25 piers (Figure 10.3). These piers are supported by drilled shafts ranging from 3 to 8 ft in diameter (Table 10.2). Inspection records show that the drilled shafts on piers 10 and 11 have steel casings extending up from the ground line to within 6 ft of the bottom of the footing. This is not a clear indicator that exposed concrete will be available for inspection. Inspection report photographs show delamination above the water line and minor damage to the fender system. The underwater photos do not indicate that any vegetation was removed for inspection and as such cannot be used to confirm or deny the presence of casing. Based on review of the information provided for this bridge, further on-site evaluation was warranted.

Figure 10.3 Bridge Number 780074 (Bridge of Lions) plan and elevation images.

10.1.2 Clearwater Memorial Causeway

Clearwater Memorial Causeway (Figure 10.4) is a fixed span structure that connects downtown Clearwater and Clearwater Beach, passing over the intracoastal waterway. It is a part of State Road 60, a road that goes from Clearwater Beach to Vero Beach. The first Memorial Causeway Bridge opened in the 1920s. It was a two-lane flat span concrete bridge. This bridge was used for approximately forty years before the second Memorial Causeway Bridge. The second was a bascule bridge, opened in the 1960s. A portion of the original bridge was then opened as a fishing pier. The third bridge became fully operational in 2005. This bridge is 2540 ft long and stands on 10 piers. These piers are supported by drilled shafts of 4 and 6 ft in diameter (Table 10.3).

Table 10.3 Clearwater Memorial Causeway.

Bridge Name	Clearwater Memorial Causeway
Bridge Number	150244
Year Built	2002
Slurry Type	Natural
Shaft Diameters	4 ft, 5 ft

The underwater inspection report from 2016 indicates that the steel casing is still in place for all shafts. The report also indicates that the casings exhibit light pitting and minor corrosion. The presence of casings eliminates this bridge from the list of possible field inspections. Information provided for this bridge at that time was deemed insufficient or inappropriate to warrant further on-site evaluation.

Figure 10.4 Bridge Number 150244 (Clearwater Memorial Causeway) plan and elevation images.

10.1.3 **Clearwater Pass Bridge**

Clearwater Pass Bridge (Figure 10.5) is a fixed span structure that carries Gulf Blvd. across Clearwater pass from Clearwater Beach to Sand Key Public Park. The current bridge opened in 1995 and replaced a drawbridge that had been in service since the 1960s. The Clearwater Pass bridge has a vertical clearance of 74 ft and as such eliminates the need for drawbridge functionality. The bridge is 2520 ft long and stands on 22 piers. These piers are supported by drilled shafts of 3 and 6 ft in diameter (Table 10.4).

Table 10.4 Clearwater Pass Bridge.

Bridge Name	Clearwater Pass Bridge
Bridge Number	155522
Year Built	1995
Slurry Type	Natural
Shaft Diameters	3 ft, 6 ft

The underwater inspection report from 2017 indicates that the scour has exposed the concrete surface below the casing on 12 shafts. The depth of exposure varies from 7 to 42 inches. The concrete is noted as “irregular with no exposed steel.” This bridge was constructed using natural slurry, but the concrete irregularities may still warrant field verification. Based on review of the information provided for this bridge, further on-site evaluation was warranted.

Figure 10.5 Bridge Number 155522 (Clearwater Pass Bridge) plan and elevation images

10.1.4 **Fuller Warren Bridge**

The Fuller Warren Bridge (Figure 10.6) is a prestressed concrete girder structure that carries interstate 95 across the St. Johns River. This bridge, which became fully operational in 2002, was built to replace the deteriorating steel bascule bridge that opened in 1954. The steel bridge remained in place, though out of service, until it was demolished with explosives in 2007. The new structure retains the namesake of former Florida Governor Fuller Warren, is eight lanes wide, 7,500 ft long, and has a 75-ft vertical clearance at midspan.

Table 10.5 Fuller Warren Bridge Sections.

Bridge 1	720154
Detour over college street	720158
Bridge 2	720627
Ramp C Bridge	720628
Bridge 3	720629
Bridge 4	720630
Ramp F Bridge	720631
Ramp E Bridge	720632
Ramp D Bridge	720633
Ramp I Bridge	720645

The plans for the Fuller Warren Bridge were divided into 10 parts, each corresponding to a different bridge section. Each bridge section has a unique bridge ID number. The main span is labeled as Bridge #3 in the plans but corresponds to bridge ID 720629. The full list of bridge labels and numbers are given in Table 10.5. Based on initial information, the plans were only requested for bridges 720627, 720628 and 720633, which correspond to Bridge 2, Ramp C Bridge, and Ramp D Bridge, respectively. These three plan sets show a total of 44 drilled shafts in the water ranging from 3 to 6 ft in diameter (Table 10.6). The inspection reports indicate that these shafts are all fully cased. Information on the other seven bridges may provide further illumination regarding the drilled shafts in the main span; of particular interest is 720629, Bridge 3. Information provided for this bridge at that time was deemed insufficient or inappropriate to warrant further on-site evaluation.

Table 10.6 Fuller Warren Bridge.

Bridge Name	Fuller Warren
Bridge Number	See Table 10.5
Year Built	2002
Slurry Type	Natural
Shaft Diameters	3 ft, 4 ft, 6 ft

Figure 10.6 Bridge Number 720629 (main) (Fuller Warren Bridge) plan and elevation images.

10.1.5 **Gandy Bridge**

The Gandy Bridge corridor is the first major water crossing between Hillsborough and Pinellas counties in Old Tampa Bay. The corridor, stretching 2.4 miles, has been the site for four bridges dating back to 1924 when the first two-lane low-level draw bridge was built. In 1956, a second bridge, which carried westbound traffic, was built to the north of the original bridge, which carried two eastbound lanes. In 1975, the third Gandy Bridge was opened to the south of the 1924 bridge and took over east bound traffic. In 1996 the fourth bridge (Figure 10.7) was opened, which is the bridge of interest to this project. It was built on the original alignment of the 1924 bridge that had been fully removed. At that time, the 1956 bridge was converted to a pedestrian trail and all west bound traffic was routed over the newest bridge. The 1956, 1975 and 1996 bridges were all high-level (45 ft clearance) with no moving components. Today, the bridge is still part of US 92.

The bridge is technically in FDOT District 7, but was built in 1996 under District 1 oversight prior to the creation of District 7. The bridge has 96 spans supported by 97 Piers, 94 of the piers are in the water. Each water pier has a cap-and-column design, where a single hammerhead-type pier cap, column, and footing are supported by four drilled shafts with shaft diameters ranging from 4 ft to 7 ft (Table 10.7), and with both single- and double-concentric steel reinforcing cage configurations. All shafts were constructed with natural slurry and a combination of temporary and permanent casings.

Table 10.7 Gandy Bridge (Westbound).

Bridge Name	Gandy Bridge (Westbound)
Bridge Number	100585
Year Built	1997
Slurry Type	Natural
Shaft Diameters	4 ft, 6 ft, 7 ft

Inspection records for this bridge are highly detailed, and show which shafts had their casings removed and which have voids and/or honeycombing. This is one of the few bridges seen with casings removed. Based on review of the information provided for this bridge, further on-site evaluation was warranted.

Figure 10.7 Bridge Number 100585 (Gandy Bridge) plan and elevation images.

10.1.6 **John Ringing Causeway**

The original bridge was built in 1925 by John Ringling, who owned land on both Lido and Longboat keys. Wanting to develop the land in the future, Ringling connected the two keys with the mainland. Shortly after, the bridge was donated to the city in 1927. In 1951, the State Road Department started to build a four-lane drawbridge which opened in 1959, and the original bridge was demolished. The same thing occurred in 2000 and a fixed high-span bridge was completed in 2003 (Figure 10.8).

John Ringling Causeway has 10 piers and two end bent systems. Each pier has two 9-ft drilled shafts (Table 10.8). Each end bent system has a total of four drilled shafts, each with a diameter of 4 ft. The drilled shafts for this bridge were cast using natural slurry. Review of the most recent inspection report indicated that all shafts are still fully cased. Information provided for this bridge at that time was deemed insufficient or inappropriate to warrant further on-site evaluation.

Table 10.8 John Ringling Causeway.

Bridge Name	John Ringling Causeway
Bridge Number	170176
Year Built	2003
Slurry Type	Natural
Shaft Diameters	4 ft, 9 ft

Figure 10.8 Bridge Number 170176 (John Ringling Causeway) plan and elevation images.

10.1.7 Overland Bridge

The Overland Bridge (Figure 10.9) is the elevated section of I-95 before the split into three bridges (the Fuller Warren Bridge, the Acosta Bridge and the Main St. Bridge). This bridge is currently under construction and as such, no as- built plans were available. However, as an over-land structure there will be no piers in the water, eliminating the possibility for underwater inspection. Basic bridge information can be found in Table 10.9. Information provided for this bridge at that time was deemed insufficient or inappropriate to warrant further on-site evaluation.

Table 10.9 Overland Bridge.

Bridge Name	Overland Bridge
Bridge Number	720627
Year Built	2017
Slurry Type	Bentonite
Shaft Diameters	unknown

Figure 10.9 Bridge Number 720627 (Overland Bridge) plan and perspective images.

10.1.8 Santa Fe River Bridge

The Santa Fe River Bridge (Figure 10.10) is located in High Springs, FL on US41/441. Its purpose is to carry the highway over the Santa Fe River. The bridge has a length of 369 ft. While it does not have any historical significance, this bridge marks the start of a 26-mile paddle-boarding trail from High Springs to Bransford at the Suwannee River.

This bridge has two end bent piers and three intermediate piers with shaft diameters of 3 and 5 ft, respectively (Table 10.10), and each pier is supported by two shafts. Inspection reports for this bridge note honeycombing on columns. Shafts were cast with bentonite slurry (contractor was given the choice of casing or bentonite). Based on the river depth profiles, the top of shafts can potentially be seen. Based on review of the information provided for this bridge, further on-site evaluation was warranted.

Table 10.10 Santa Fe River Bridge.

Bridge Name	Santa Fe River Bridge
Bridge Number	260112
Year Built	2002
Slurry Type	Bentonite
Shaft Diameters	3 ft, 5 ft

Figure 10.10 Bridge Number 260112 (Santa Fe River Bridge) plan and elevation images.

10.1.9 SR-2 Choctawhatchee Bridge

This bridge carries SR 2 over the Choctawhatchee River (Figure 10.11). It is 2559 ft long and holds no historic significance. Review of the plans for this bridge indicate that there are 21 piers in the waterway or flood plain. Those piers are supported by 42 drilled shafts 5 ft in diameter (Table 10.11). Inspection reports show that this bridge is fully cased. Information provided for this bridge at that time was deemed insufficient or inappropriate to warrant further on-site evaluation.

Table 10.11 SR2 Choctawhatchee Bridge.

Bridge Name	SR2 Choctawhatchee Bridge
Bridge Number	520145
Year Built	2001
Slurry Type	Natural
Shaft Diameters	5 ft

Figure 10.11 Bridge Number 520145 (SR2 Choctawhatchee Bridge) plan image.

10.1.10 SR-10 Choctawhatchee Bridge

Known as the Caryville Bridge or the George L. Dickenson Bridge (Figure 10.12), this bridge was constructed in 1927 as an effort to connect Washington and Holmes County. The original bridge had four Warren deck trusses and a double leaf bascule section. Between 1944 and 1952, the bridge underwent a reconfiguration into a fixed deck design that had a wider roadway.

Review of the newer bridge plans indicates that there are 24 piers. Those piers are supported by 48 drilled shafts, 5 and 6 ft in diameter (Table 10.12). Permanent casing was used during installation. Information provided for this bridge at that time was deemed insufficient or inappropriate to warrant further on-site evaluation.

Table 10.12 SR10 Choctawhatchee Bridge.

Bridge Name	SR10 Choctawhatchee Bridge
Bridge Number	520149
Year Built	2000
Slurry Type	Mineral Slurry
Shaft Diameters	5 ft, 6 ft

Figure 10.12 Bridge Number 520149 (SR10 Choctawhatchee Bridge) plan and elevation images.

10.1.11 **SR-20 Blountstown Bridge.**

The bridge that carries SR20 over the Apalachicola river is commonly known as the Trammel Bridge (Figure 10.13), named after the three members of the Trammell family: (1) U.S Senator Park M. Trammel; (2) Member of the Florida Legislature John D. Trammell; and (3) Robert D. Trammel, a representative of the Blountstown area in the Florida legislature. These men helped either pass legislation that called for the construction of the bridge or were involved in securing funding. The original bridge was opened in 1938 and now carries westbound traffic whereas the eastbound span is a concrete high-rise bridge that was opened in 1998. Interestingly, the ends of the bridges do not share the same time zone. The east end is in the Eastern Time zone and the west end is in the Central Time Zone.

Review of the plans shows that there are 60 drilled shafts within the waterway or flood plain, four of which are in the Apalachicola River. Shafts range in diameter from 5 to 9 ft (Table 10.13). The inspection reports received to date do not include the underwater reports. However, the plans indicate casing should be removed down to an elevation 30 ft, and as such it is assumed that the casings were removed. Based on review of the information provided for this bridge, further on-site evaluation was warranted.

Table 10.13 SR20 Blountstown Bridge.

Bridge Name	SR 20 Blountstown
Bridge Number	470052
Year Built	1998
Slurry Type	Bentonite
Shaft Diameters (ft)	5, 6, 7, 9

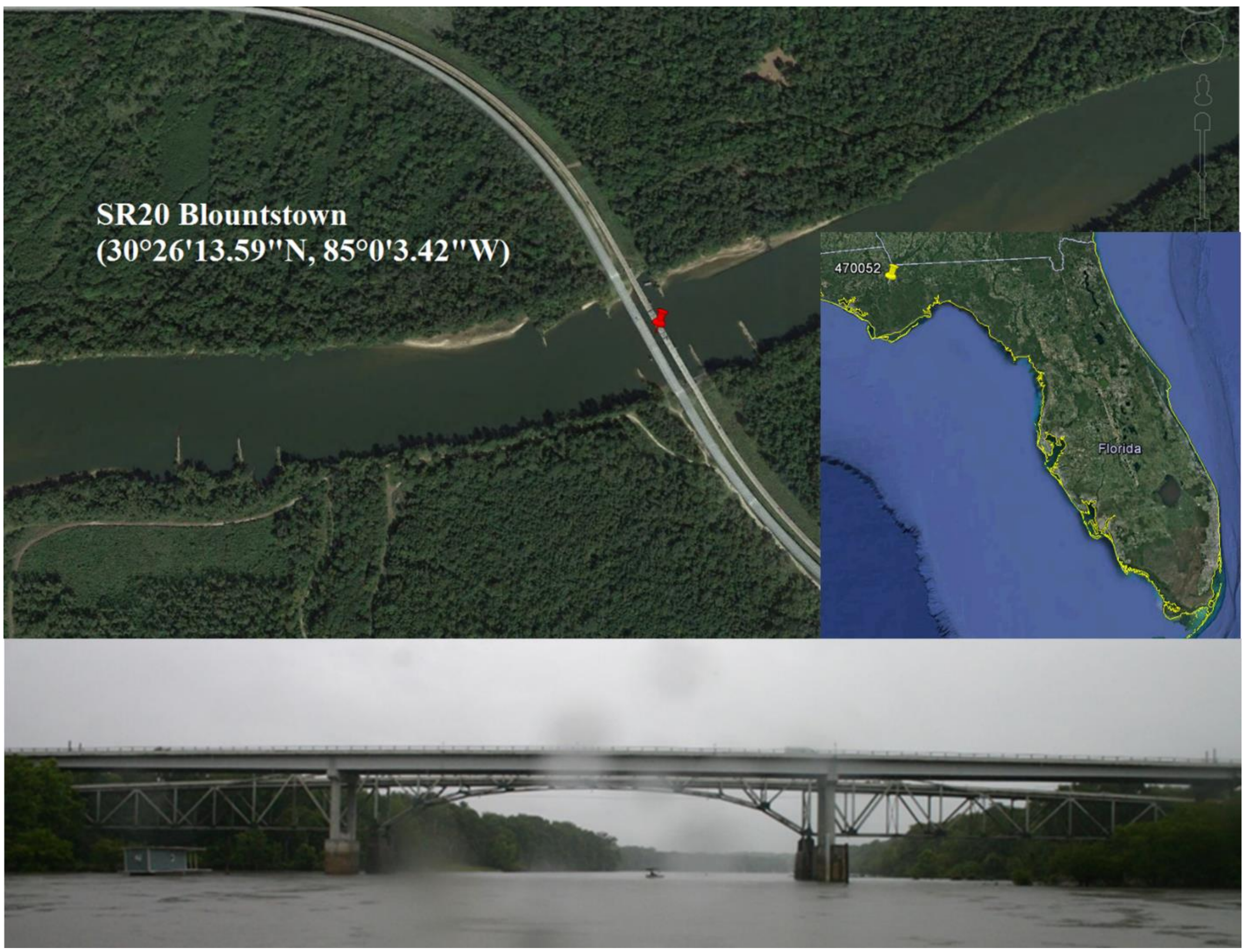


Figure 10.13 Bridge Number 470052 (SR20 Blountstown Bridge) plan and elevation images.

10.1.12 **SR-61 Lost Creek Bridge**

The Lost Creek Bridge (Figure 10.14) carries state road 61 over Lost Creek. This bridge is located 1.2 miles south of Crawfordville, FL. It has no historical significance.

There are 4 piers within the waterway with a total of 8 drilled shafts 3 ft in diameter (Table 10.14). Initial information indicated that the steel casings had been removed from the drilled shafts; however, there did not appear to be any water access to perform inspection. Thus, information provided for this bridge at that time was deemed insufficient or inappropriate to warrant further on-site evaluation.

Table 10.14 SR 61 Lost Creek Bridge.

Bridge Name	SR61 Lost Creek Bridge
Bridge Number	590048
Year Built	1991
Slurry Type	Bentonite
Shaft Diameters	3 ft

Figure 10.14 Bridge Number 590048 (Lost Creek Bridge) plan image.

10.1.13 **SR-63 Ochlockonee Bridge**

The Ochlockonee Bridge on US 27 (SR 63) and is located one half mile north of Leon County (Figure 10.15). This bridge connects the highway over the Ochlockonee River, hence the name. This bridge has no historical significance.

The bridge is made up of four main sections each with their own bridge number. The northbound bridges are 500124 and 500126. The southbound bridges are 500125 and 500127. The portion of this bridge constructed using drilled shafts includes bridges 500124 and 500127. Review of the plans indicates that there are 20 piers within the waterway or flood plain. These piers are supported by 40 drilled shafts 3 ft in diameter (Table 10.15). Inspection reports are unclear as to the presence of casing on the shafts. Information provided for this bridge at that time was deemed insufficient or inappropriate to warrant further on-site evaluation.

Table 10.15 SR63 Ochlockonee Bridge.

Bridge Name	SR 63 Ochlocknee Bridge
Bridge Number	500124/500127
Year Built	2001
Slurry Type	Bentonite
Shaft Diameters	3 ft

Figure 10.14 Bridge Numbers 500124/500127 (SR63 Ochlockonee Bridge) plan image.

10.1.14 **Victory Bridge**

The Victory Bridge carries US90 over the wetlands/flood plain west of the Apalachicola River, immediately downstream of the Jim Woodruff Dam. The original bridge was built in 1927 and is no longer used, having been replaced in 1996 by the high level bridge that is in service currently. Review of the construction plans indicates that there are 22 piers, each supported by 2 drilled shafts 4 ft in diameter (Table 10.16). While inspection reports do not provide sufficient detail to determine what is visible, there is no access to the waterway to allow for inspection. Information provided for this bridge at that time was deemed insufficient or inappropriate to warrant further on-site evaluation.

Table 10.16 Victory Bridge.

Bridge Name	Victory Bridge
Bridge Number	530111
Year Built	1996
Slurry Type	Natural
Shaft Diameters	4 ft

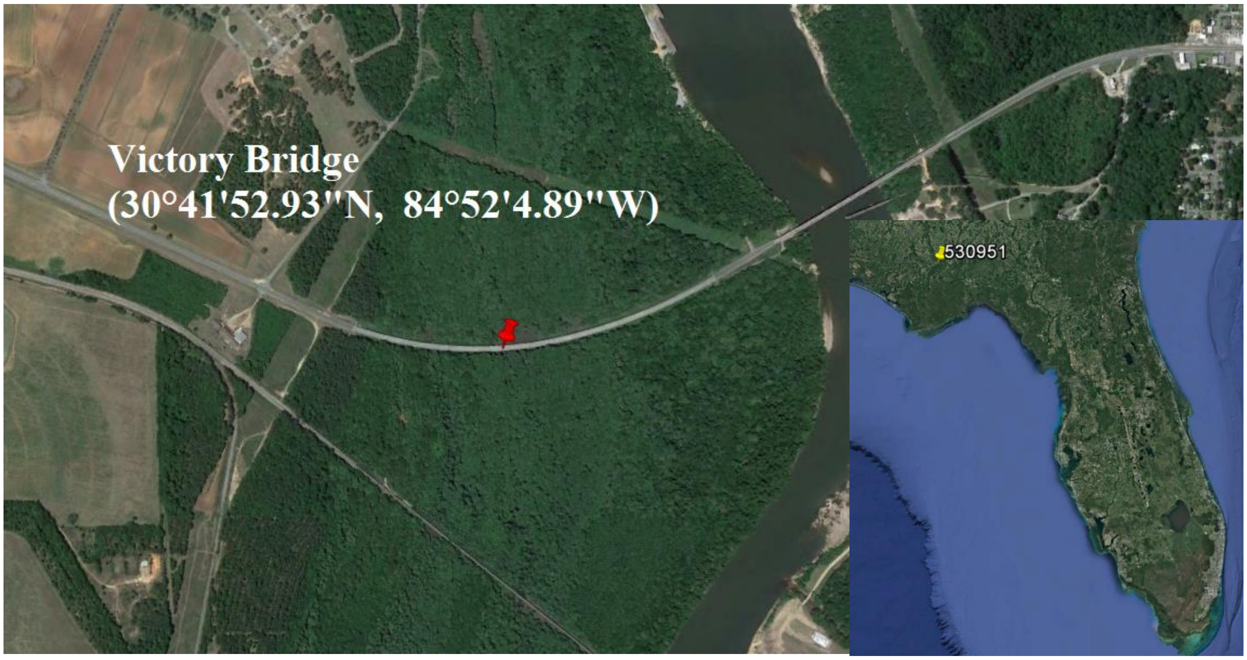


Figure 10.16 Bridge Number 530111 (Victory Bridge) plan image.

10.2 Underwater Evaluations

Of the 14 bridges screened and discussed earlier, five were selected for on-site, underwater evaluation. These bridges were selected to represent bridges constructed with bentonite, attapulgite and natural slurry types. To be appropriate for this stage, plans, construction records, or inspection records had to indicate that the permanent casing (required for over water construction) had been removed at least in part, allowing for direct access and visual evaluation. Later, a dive team was contracted to remove 3-ft by 3-ft windows of casing, as after initial inspection it was found that most shafts were fully-cased with limited, if any, exposed concrete. Thus, casing removal is also included in this section per bridge.

10.2.1 Gandy Bridge 100585 (US-92 over Old Tampa Bay)

Drilled shaft construction records (logs) for the Gandy Bridge were not available through the normal information request protocol as that type of field record was not deemed important enough to store (hard copies). However, load testing reports from Statnamic tests provided the first real information about the construction process and were the only drilled shaft construction logs that could be found. These documents revealed that natural slurry was used for construction making this an ideal control site. Only a few shafts were included in the load testing report. Figure 10.17 shows both the eastbound (pile) and westbound (shaft) Gandy Bridges as well as the diving setup.

Figure 10.17 Eastbound and westbound Gandy Bridge (looking west).

For dive inspections, shafts were selected based on the dive inspection reports provided by FDOT from Bolt Underwater Services. Uncased shaft locations were provided, as well as the presence of any voids or honeycombing. Piers with voids and honeycombing were selected in addition to piers without deficiencies. Later, to provide a means for observation of the encased concrete surfaces and for comparison with concrete surfaces that were not directly exposed to the surrounding environment, windows were cut in the casings of selected shafts. Figure 10.18 shows the plan view of the eastern portion of the bridge and has the inspected piers circled. Figure 10.19(a) shows the drilled shaft numbering protocol, where shafts are labeled 1-4, with 1 being NW, 2 SW, 3 NE, and 4 SE. Figures 10.19 (b) to (d) show the shaft sizes and layouts of reinforcement cages used. More detail can be found in Appendix K. A summary of piers inspected along with their shaft sizes, reinforcement cages, and comments made by the dive inspectors can be found in Table 10.17.

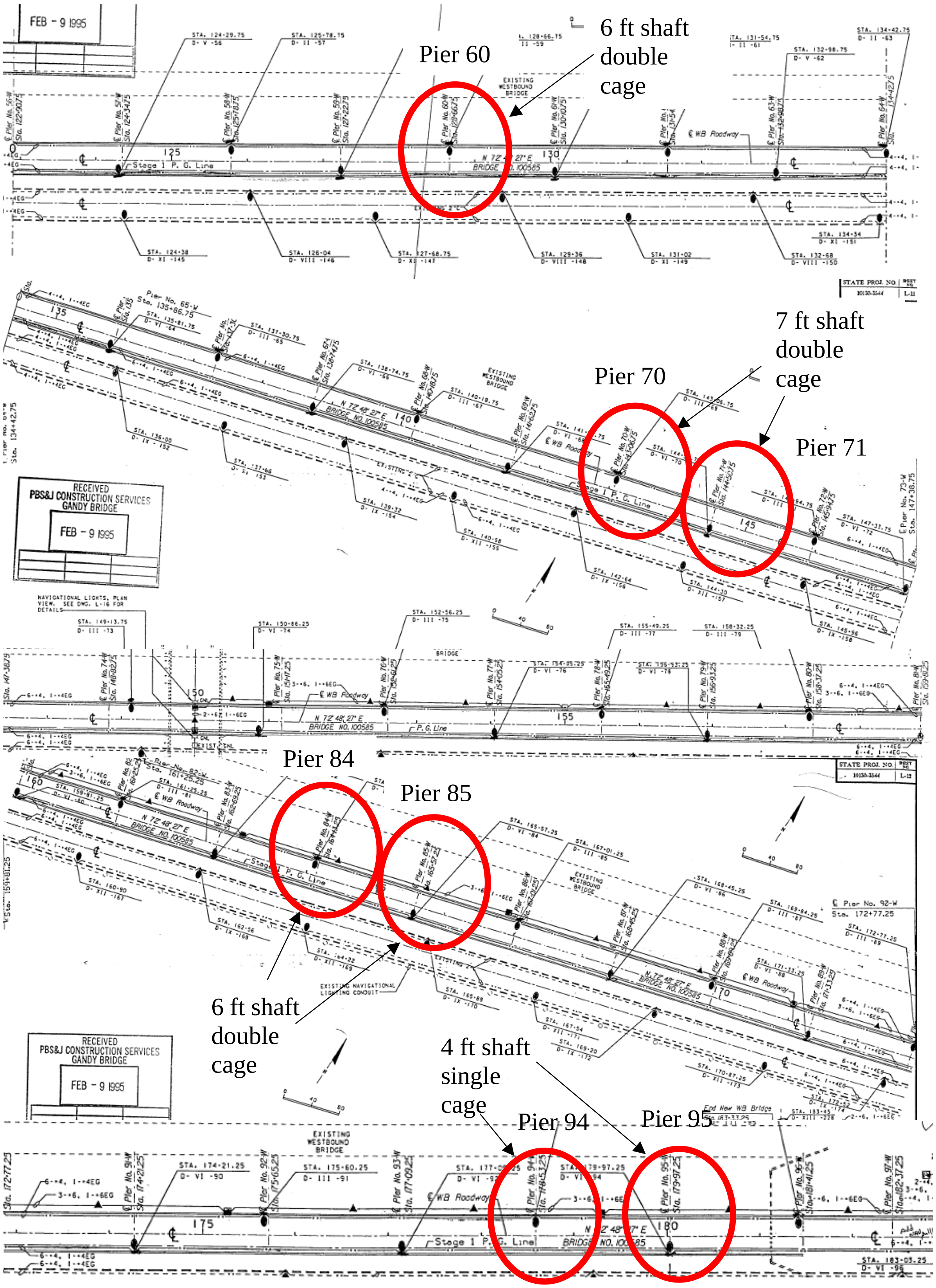
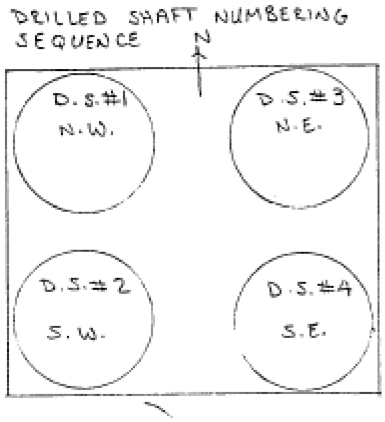
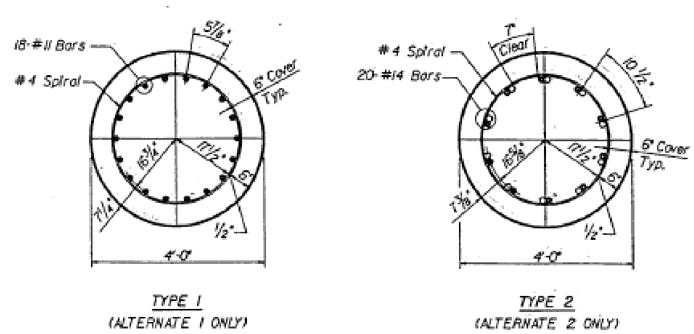


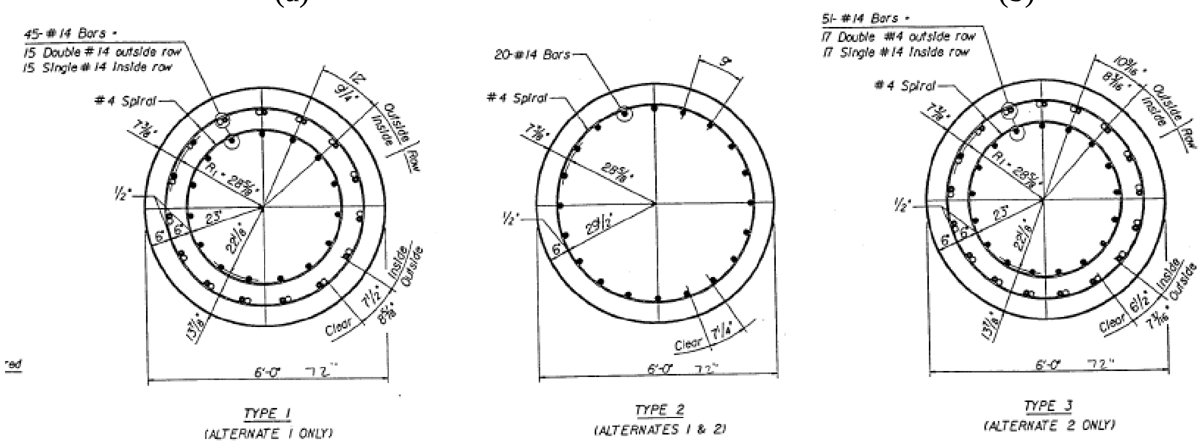
Figure 10.18 Plan view of eastern portion of Gandy Bridge. The piers that were inspected have been circled.



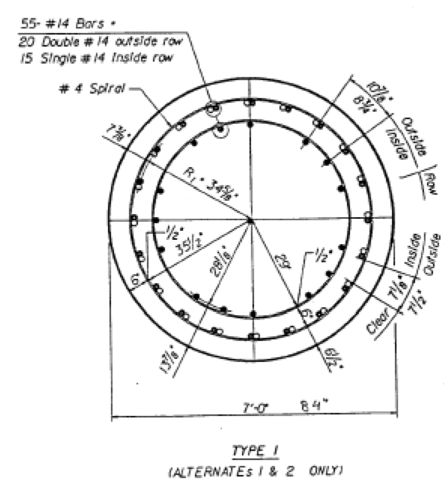
(a)



(b)



(c)



(d)

Figure 10.19 (a) Drilled shaft numbering sequence, (b) 4-ft diameter shaft reinforcement cage layouts, (c) 6-ft diameter shaft reinforcement cage layouts, (d) 7-ft diameter shaft reinforcement cage layout.

Table 10.17. Summary of shafts inspected at Gandy Bridge.

Shaft #	Shaft Diameter (ft)	Cage Type	Diver Comment
Initial Inspection: Previously cut casing, concrete was cleaned for observation			
71-1	7	Double[a]	Void, 5 in. diameter x 3 in. deep
71-4	7	Double[a]	Void, 14 in. H x 14 in. W x 2 in. D
84-1	6	Double[b]	Void, 2 in. H x 9 in. W x 2 in. D
84-2	6	Double[b]	Void, 2 in. H x 9 in. W x 2 in. D
84-3	6	Double[b]	8 in. H x 3 in. W x 3 in. D
84-4	6	Double[b]	2 Voids, Up to 14 in. H x 5 in. W x 4 in. D
85 (all)	6	Double[b]	N/A
94 (all, with casing)	4	Single[c]	N/A
95-1	4	Single[c]	Void, 32 in. H x 12 in. W x 2 in. D
95-2	4	Single[c]	Void, 2 ft H x 18 in. W x 3 in. D
Secondary Inspection: These shafts were selected to have casing removed			
60-2	6	Double[b]	Fresh concrete, clean
60-4	6	Double[b]	Fresh concrete, clean
70-1	7	Double[a]	Fresh concrete, clean with small defects
70-3	7	Double[a]	Fresh concrete, uneven surface
91-2	4	Single[c]	Fresh concrete, clean
91-4	4	Single[c]	Fresh concrete, clean

[a]See Figure 10.19(d)
[b]See Figure 10.19(c)- Type 1
[c]See Figure 10.19(b)- Type 1

Pier 71 (7-ft diameter, double cage)

Pier 71 contains 7-ft diameter Type 1 shafts (Figure 10.19(d)). Two shafts 71-1 and 71-4 were listed as having voids in the dive report, and shaft 71-4 was chosen for cleaning and inspection. Unfortunately, no images of the shaft were taken before cleaning; however, it looked similar to shaft 84-4 (Figure 10.24), covered in a layer of barnacles. Figure 10.20 shows each side of the cleaned portion of the shaft. The 7-ft shafts had small voids in the cleaned concrete surface (Figure 10.21). Sections of cleaned concrete can be seen up-close in Figure 10.22; note that this concrete was the roughest seen on this bridge. Figure 10.23 shows a long crevice in 71-4.

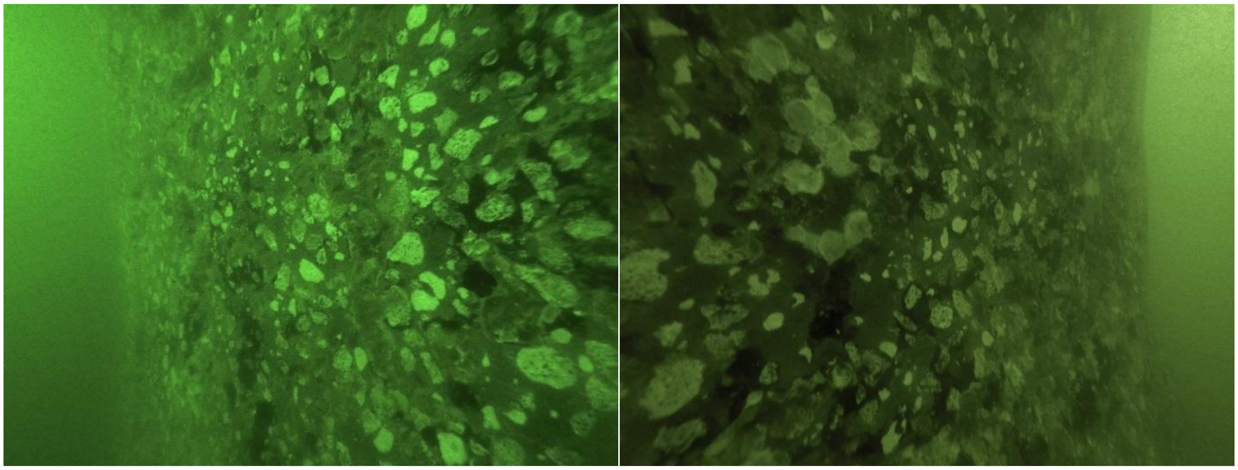


Figure 10.20 Shaft 71-4 post-cleaning.

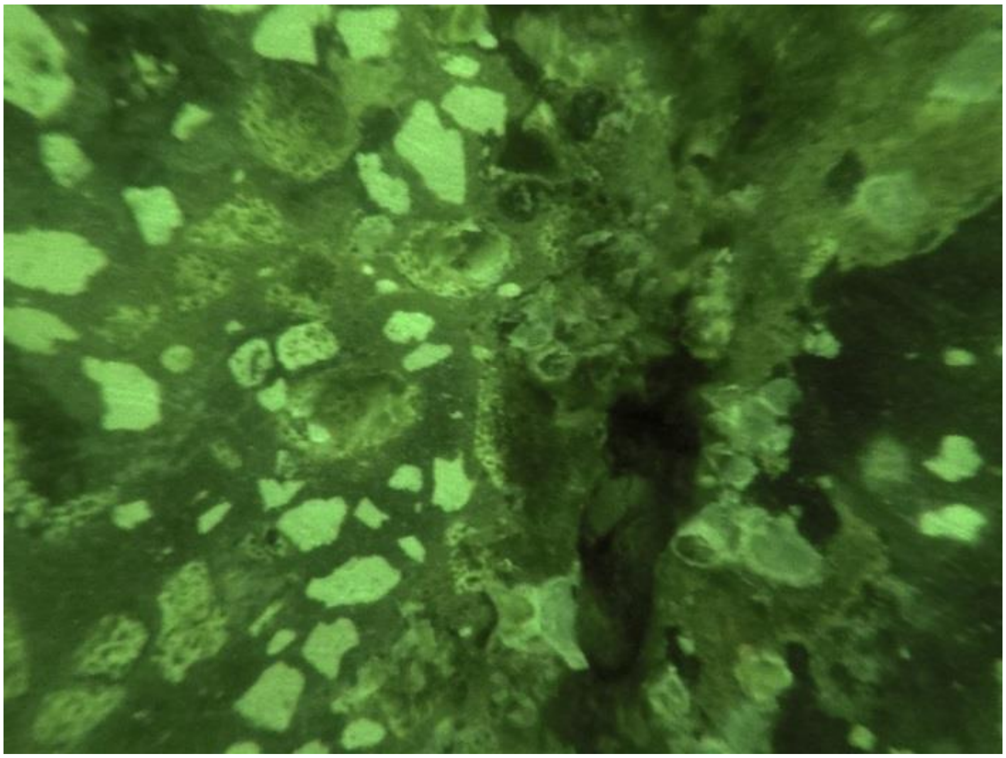


Figure 10.21 Close up of a small void on 71-4.

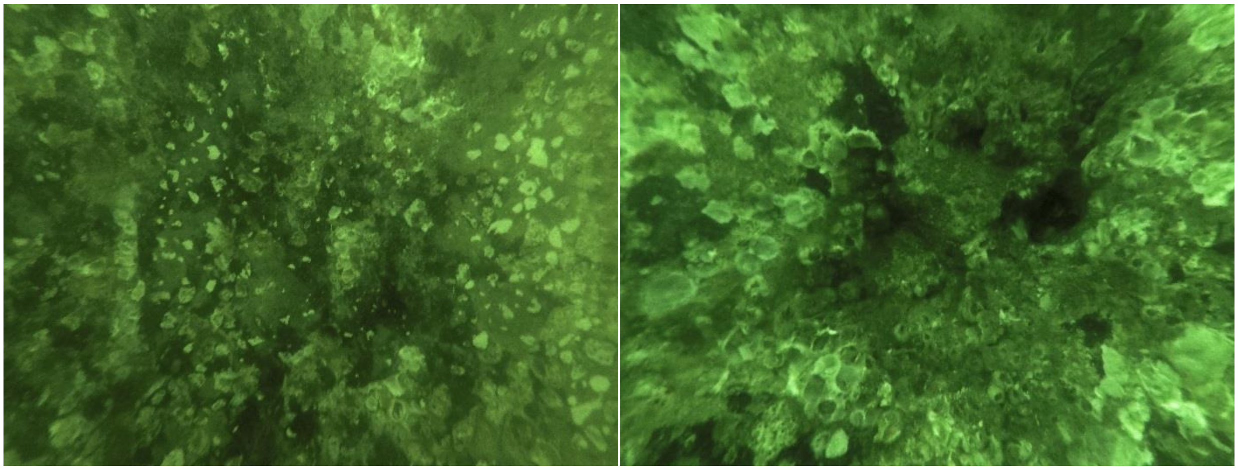


Figure 10.22 Cleaned concrete on 71-4, close-up pictures.

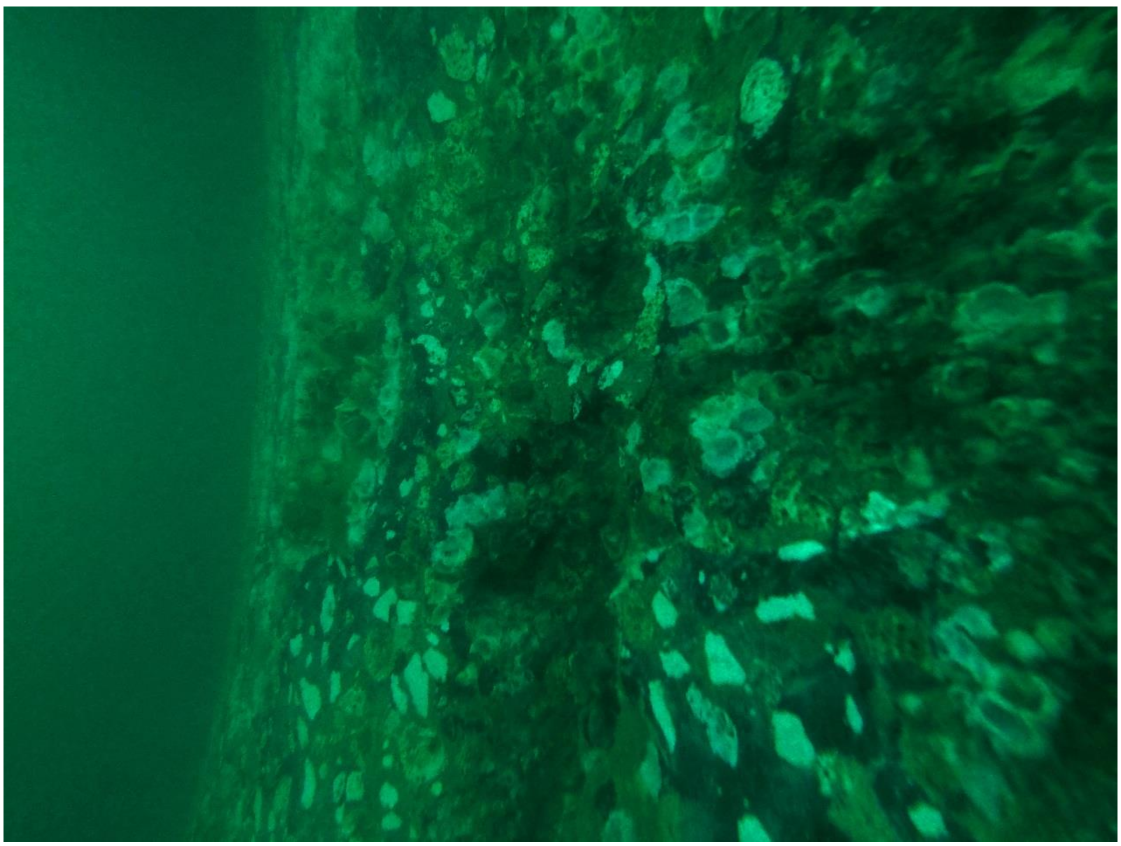


Figure 10.23 Large void found on the south side of 71-4.

Pier 84 (6-ft diameter, double cage)

Pier 84 was noted to have voids on all four shafts and contains 6-ft diameter Type 1 reinforced shafts (Figure 10.19(c)). Shaft 84-4 was chosen for cleaning and inspection as it was listed as having two voids. Figure 10.24 shows 84-4 with a thick layer of barnacles prior to cleaning. This layer is better viewed in Figure 10.25 where the cleaned concrete transitions into the barnacle layer. Figures 10.26 and 10.27 show the cleaned portions of the shaft from east and southeast sides. It can be seen in these figures that the concrete is not completely smooth. The circled red areas on Figure 10.26 represent areas of the cleaned concrete that are filled with barnacles. Therefore the 6-ft Type 1 shafts seem to experience at least small voids in the concrete. This is suspected to have been caused by the double reinforcement cage. Figure 10.28 shows the layers found when cleaning the concrete. Consistently throughout the examination of the Gandy shafts, encrustations were found consisting of barnacles and miscellaneous plant life/organisms. Upon initial cleaning, barnacles were knocked/scraped off and the remnants of the barnacle attachment were then removed. Below this layer, a thin black layer was encountered which was difficult to remove. In cases of smooth surfaces with no significant voiding, pristine concrete could clearly be seen. Rougher surfaces with voids retained the plant life, barnacles and black layer. Except for isolated instances, there was no attempt to dig into voids and determine the full depth.

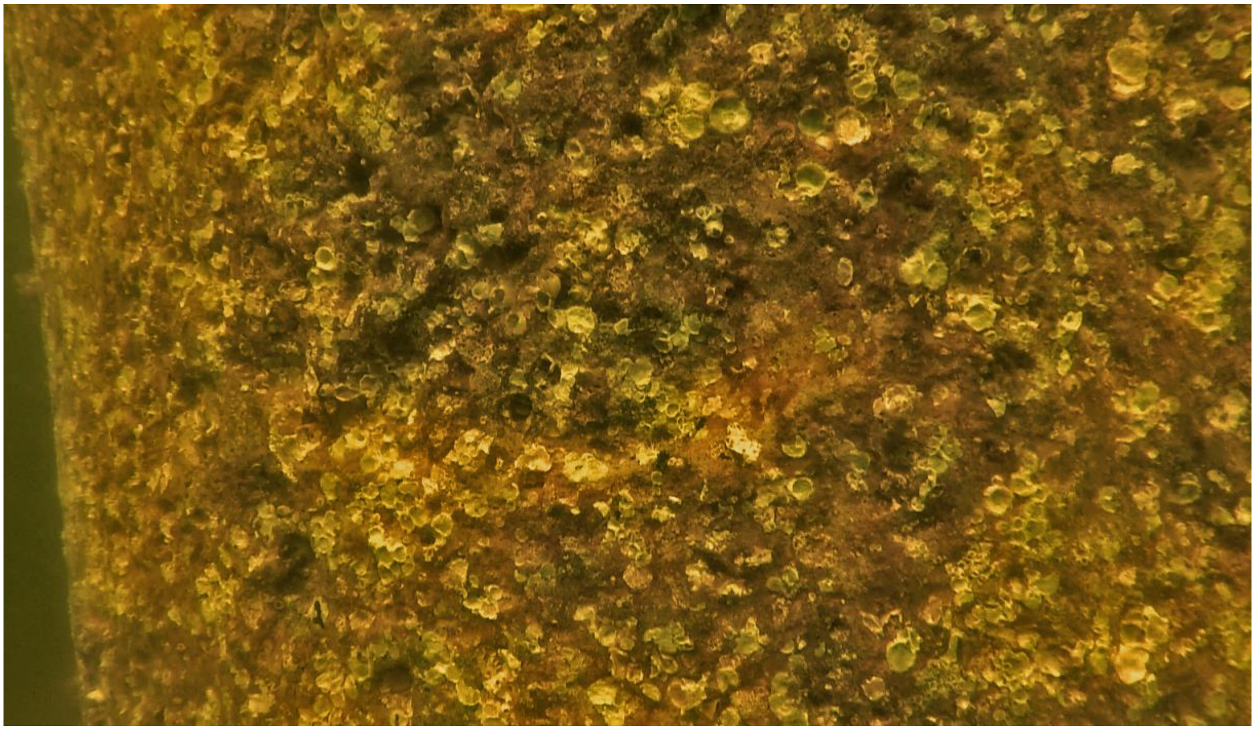


Figure 10.24 Shaft 84-4 prior to cleaning.

Figure 10.25 Shaft 84-4 partially cleaned, note clean concrete going into layer of barnacles.

Figure 10.26 Cleaned portion of 84-4 on the southeast side of the shaft, circled sections indicate small voids in concrete filled by barnacles.

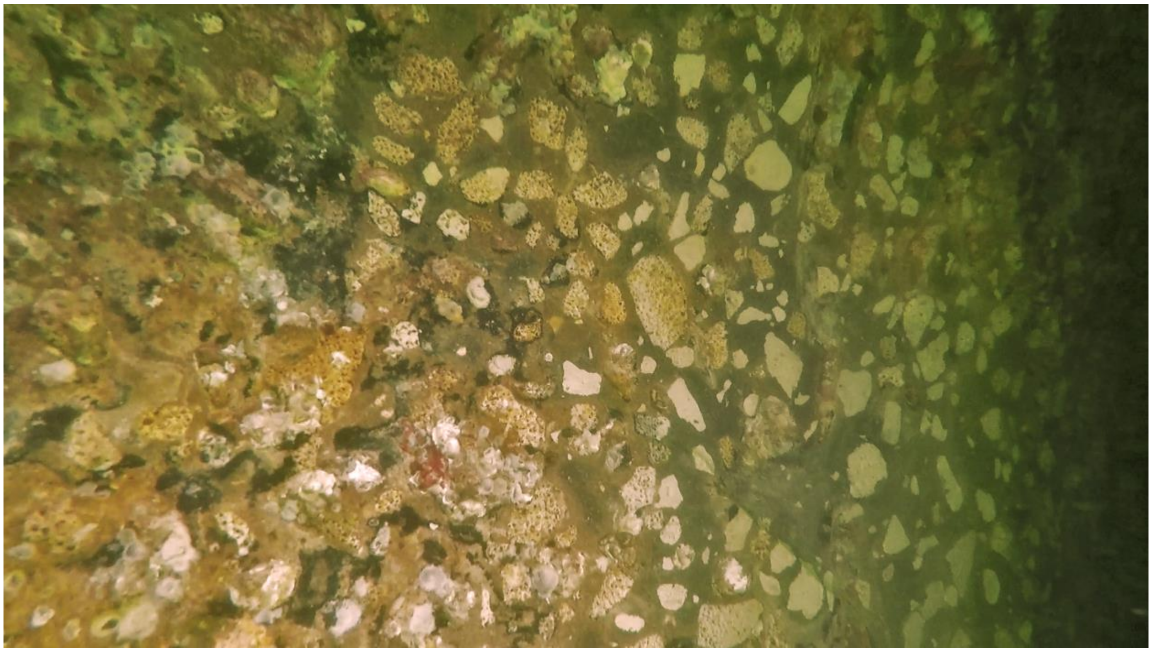


Figure 10.27 Cleaned portion of 84-4, east side of shaft.

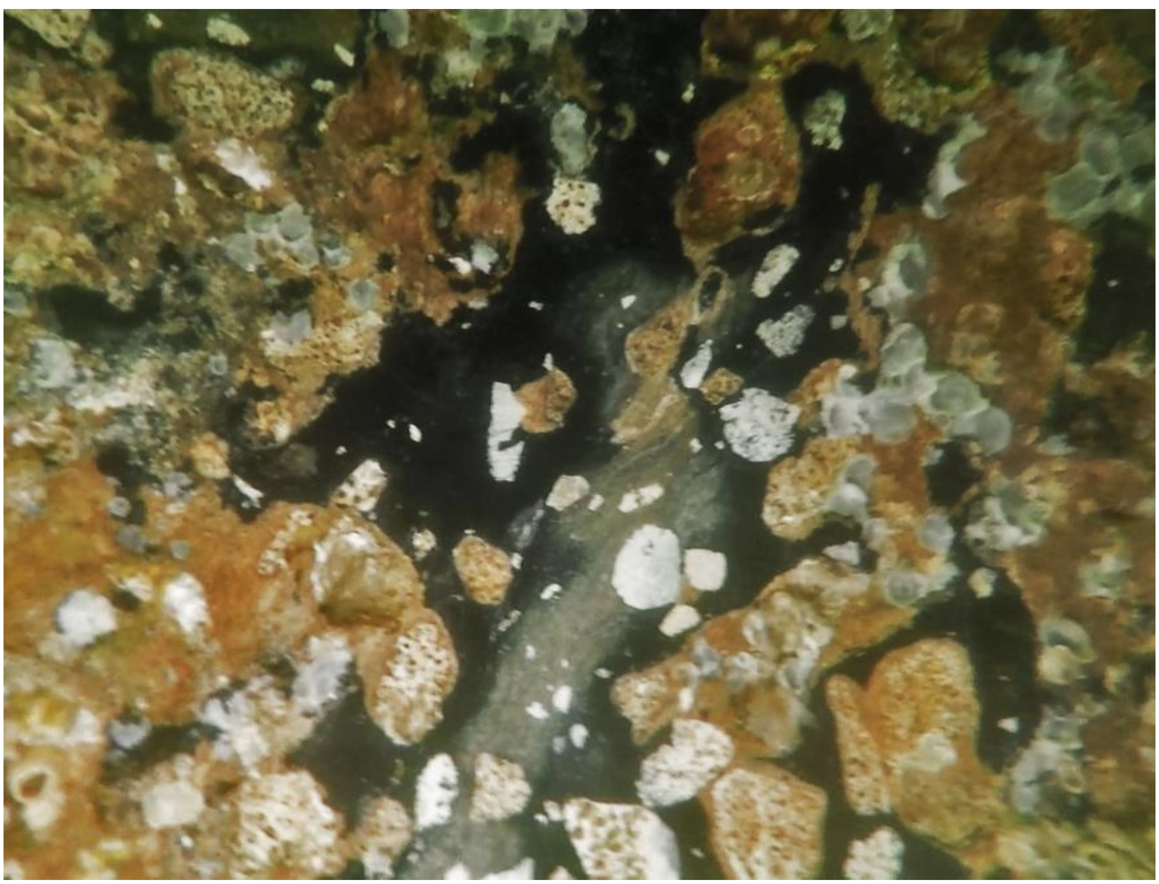


Figure 10.28 Layers from barnacles to black unknown layer to concrete.

Pier 85 (6-ft diameter, double cage)

Pier 85 was chosen as an example of a typical 6-ft Type 1 reinforcement cage (Figure 10.19(c)). The dive report does not mention this pier as having voids; however, it does state that two of the shafts, 85-2 and 85-3, do not have visible casings. Therefore, 85-2 was selected for cleaning and inspection.

Unfortunately there is no image of 85-2 before cleaning; however, it looked similar to others covered in barnacles (Figure 10.24). While the cleaned section of the shaft overall shows a smooth surface (Figure 10.29), there are small crevices. As seen in Figures 10.30 and 10.31, barnacles and sea-life fill in these voids. As noted above, the small voids/crevices are believed to have been caused by the double reinforcement cage; note this was not observed on the 4-ft diameter, single reinforcement cage shafts inspected at Pier 95.

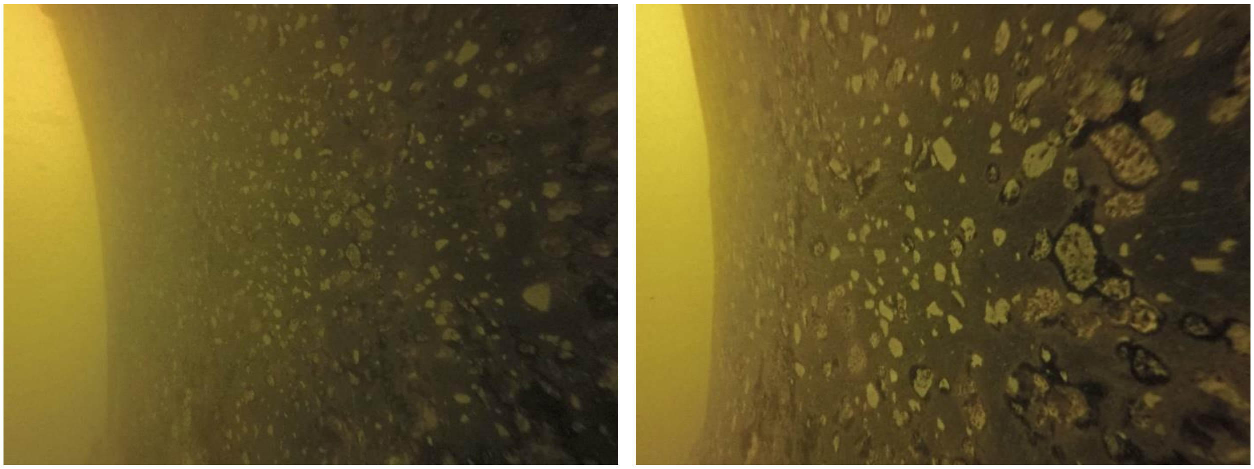


Figure 10.29 Shaft 85-2 after cleaning.

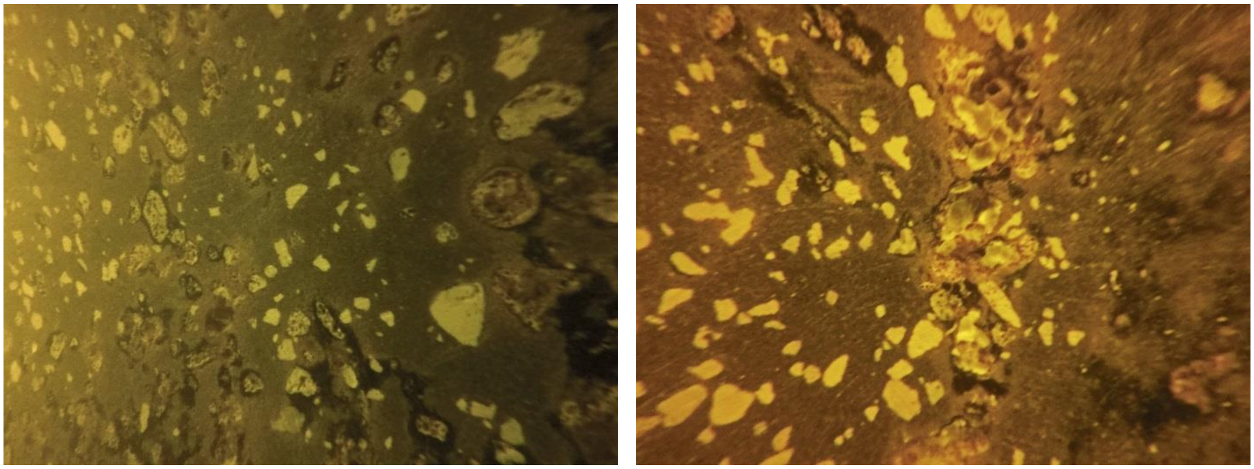


Figure 10.30 Crevices in 85-2 filled with barnacles.

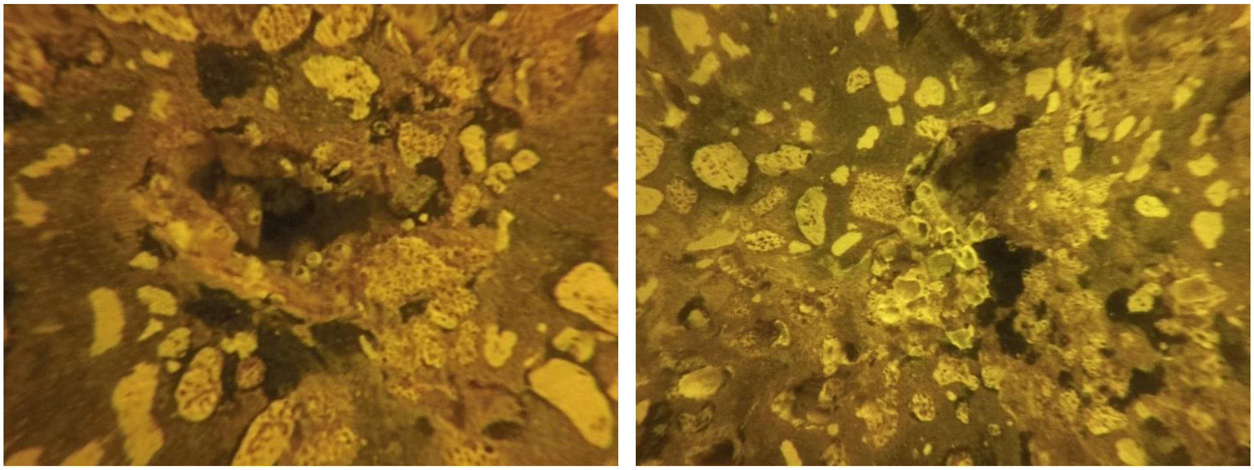


Figure 10.31 Small voids in 85-2 filled with sea-life and barnacles.

Pier 94 (4-ft diameter, single cage, cased and uncased)

Pier 94 was not mentioned in the diver report, and therefore it was thought to be a good candidate to have a casing. Upon initial inspection, it can be difficult to tell if the shaft has a casing or not because of the thick layer of barnacles attached to the shaft concrete or its casing. Therefore, shaft 4 from Pier 94 (94-4) was chosen for cleaning to demonstrate what steel casing looks like versus the concrete. Figure 10.32 shows 94-4 prior to cleaning and Figure 10.33 shows 94-4 during cleaning. In Figure 10.34 the steel from the casing is visible. As seen from Figure 10.34, there is a black layer underneath the encrustation and on top of the steel casing.

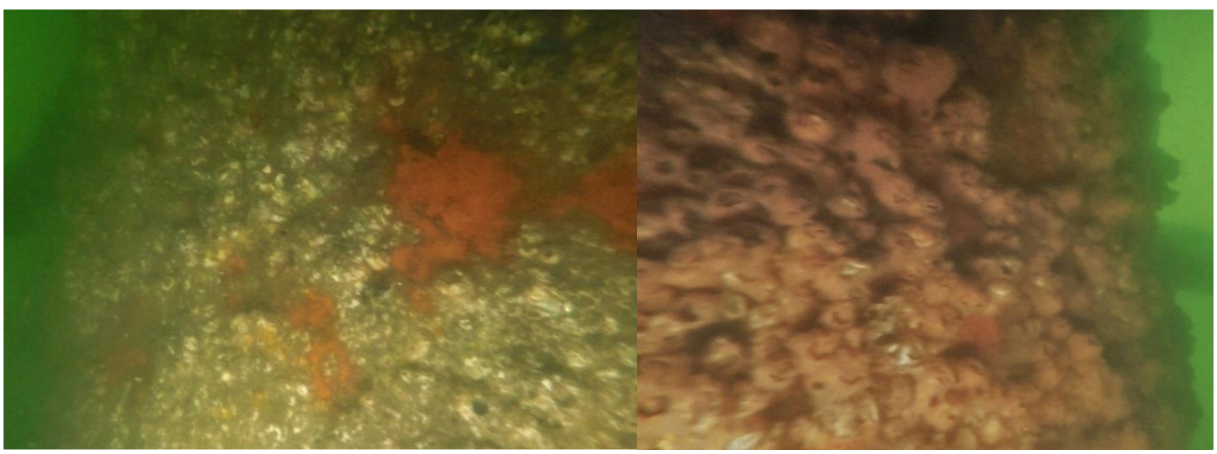


Figure 10.32 Shaft 94-4 pre-cleaning.

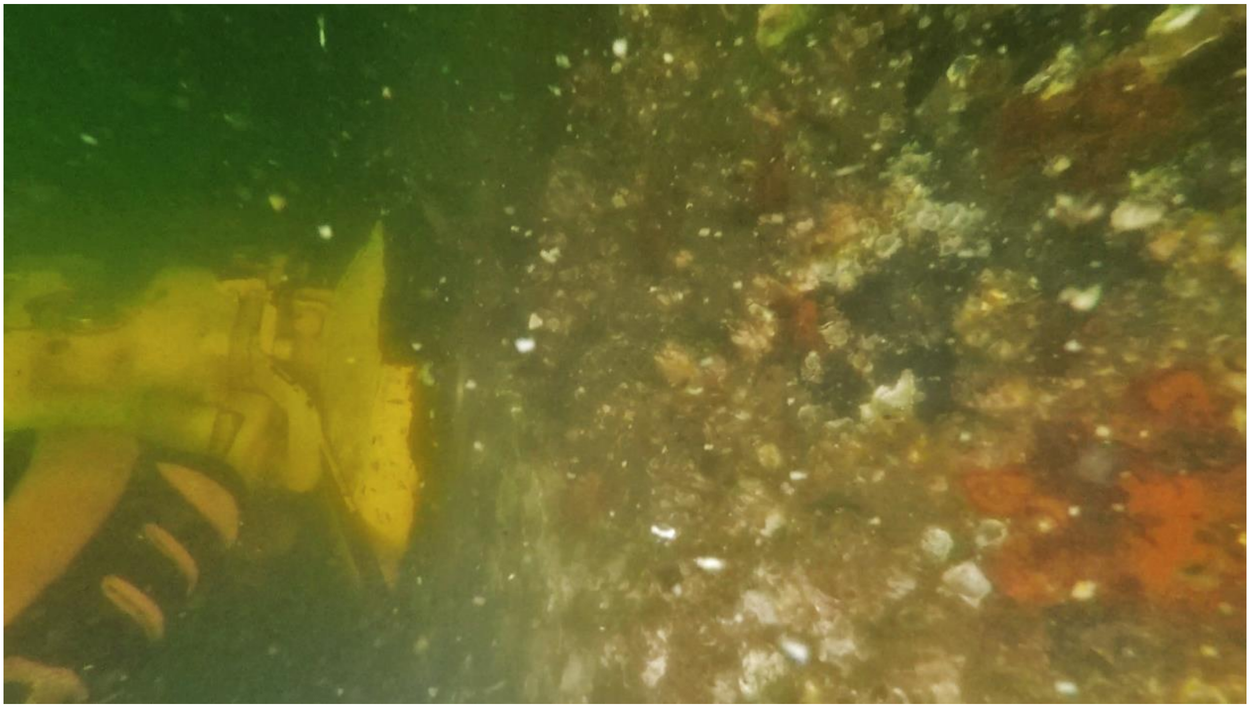


Figure 10.33 Shaft 94-4 during cleaning with the grinder.

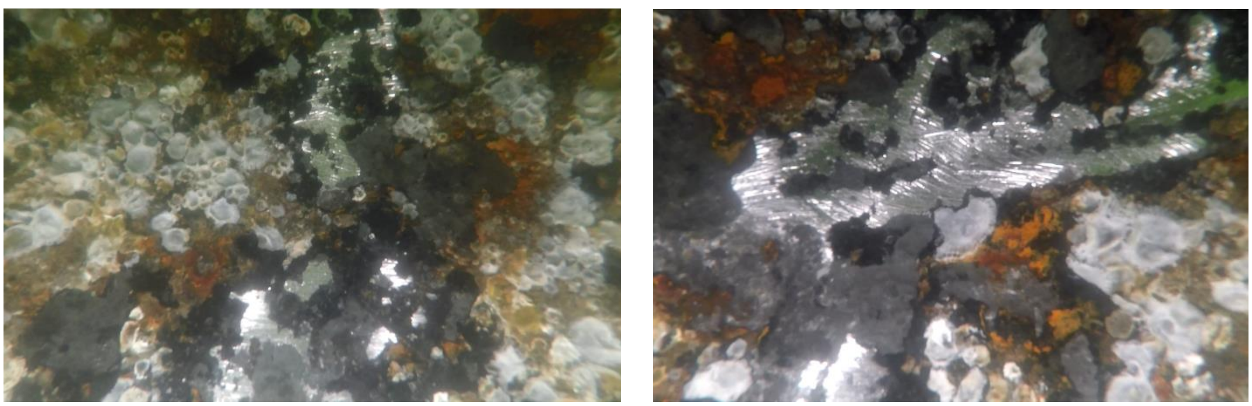


Figure 10.34 Shaft 94-4 after cleaning showing steel casing.

Pier 95 (4-ft diameter, single cage)

All shafts at Pier 95 are 4 ft in diameter and have a Type 1 reinforcement cage (Figure 10.19b). Two shafts, 95-1 and 95-3 on the north side, were chosen for cleaning and inspection on Pier 95 as the casing had previously been removed. Noted in the diver report, 95-1 had a void 12 in. below the footing/seal that is 32 in. H x 12 in. W x 2 in. D. Shaft 95-3 had no known/noted deficiencies.

Shaft 95-1 did have a void as noted, and it was noticeable even prior to cleaning (Figure 10.35). This void was substantial; Figure 10.36 shows the void before cleaning, which involved scraping barnacles, grinding and/or wire-brushing. A screwdriver was used for reference of void depth, which was about 2-3 in. (Figure 10.37). However, on the south (interior) side of the same shaft, the concrete was smooth (Figure 10.39). Figure 10.39 does demonstrate a crease, however this was noted to be from casing removal as it was a singular occurrence. The north and south sides of the shaft can be seen in Figure 10.38 and 10.39 respectively. It should be noted that while the sides of shafts appear to be concave this is from camera lens distortion.

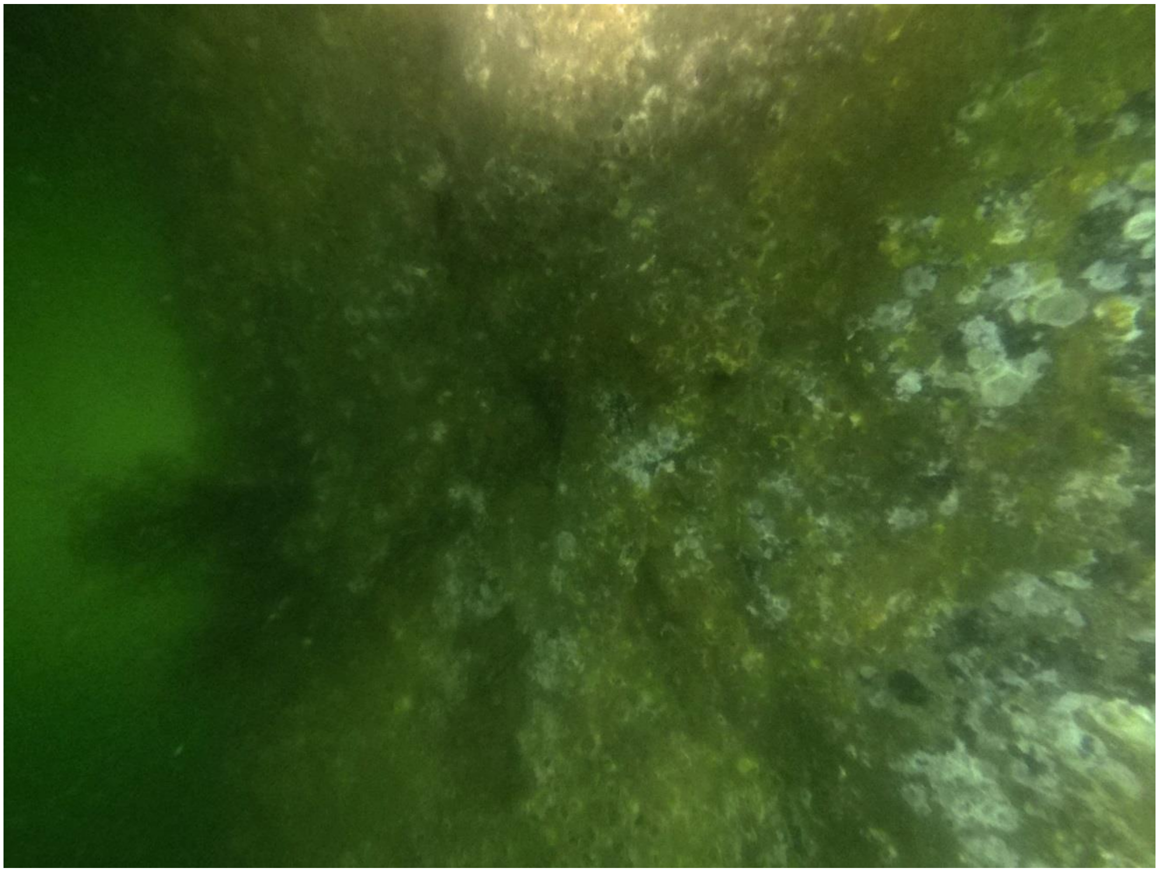


Figure 10.35 95-1 prior to cleaning.

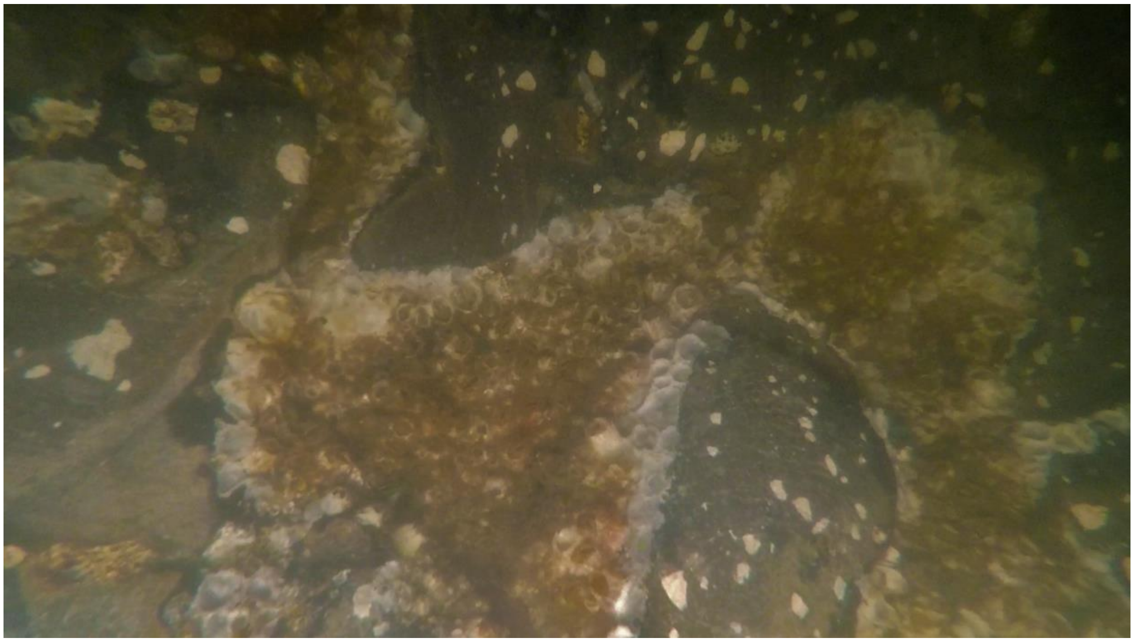


Figure 10.36 Void on 95-1, this was taken after grinding the shaft and before scraping out the barnacles residing in the voids.

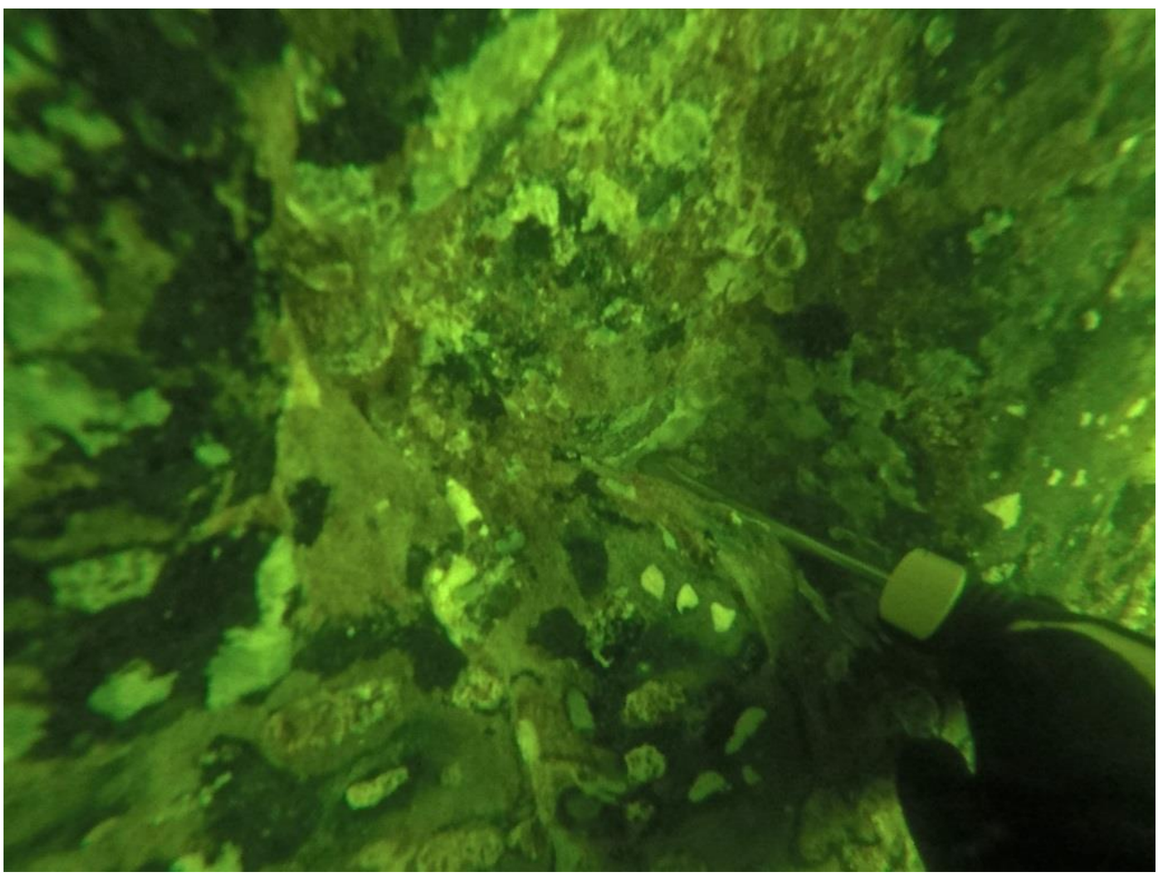
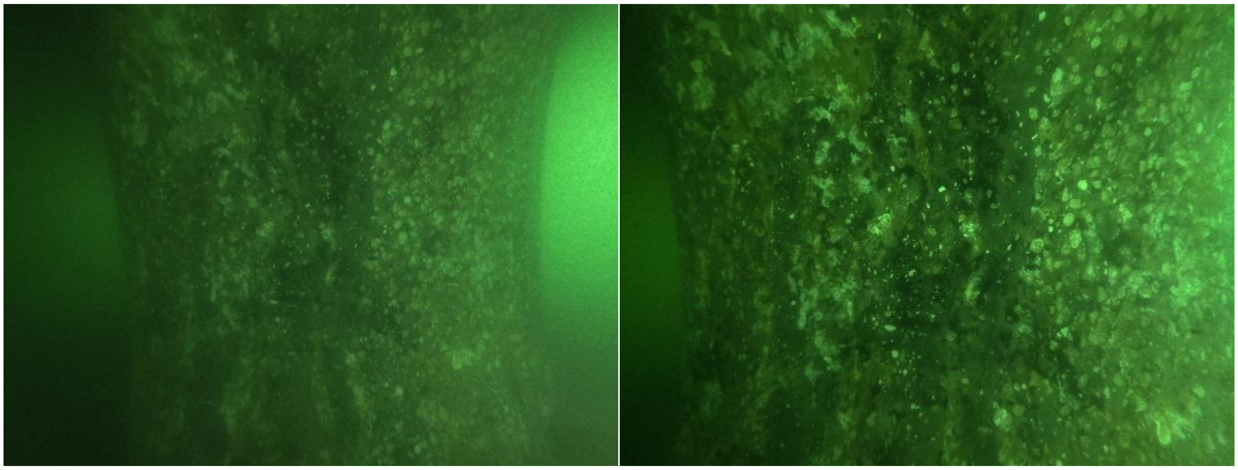


Figure 10.37 Two-to-three inch void in Shaft 95-1 post-barnacle scraping.



(a) (b)

Figure 10.38 (a) View of the north side of 95-1 post-cleaning, (b) zoomed in view of (a).

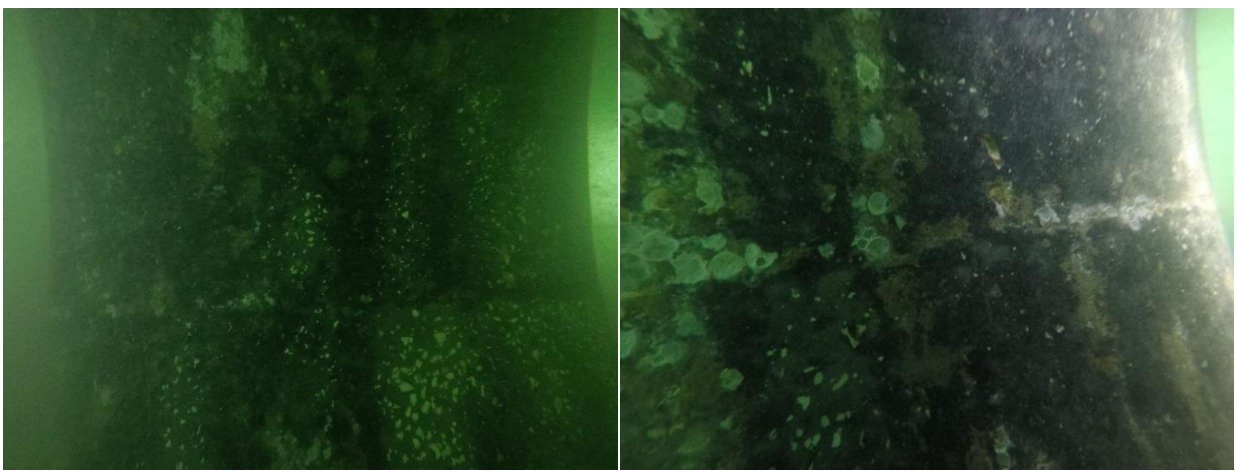


Figure 10.39 (a) South (interior) side of 95-1 post-cleaning, (b) zoomed in view of (a).

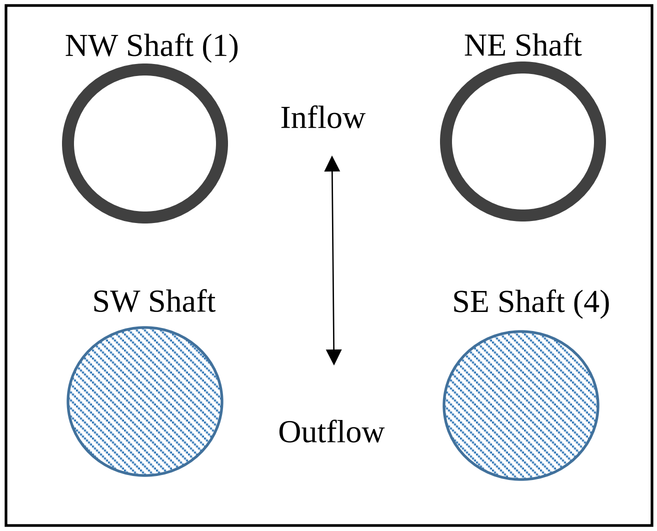
Shaft 95-3 was cleaned for comparison as a shaft with no observed problems. Figure 10.40 shows 95-3 prior to cleaning. Due to the lack of water clarity on the day of inspection it was difficult to take a picture of the entire shaft, so sections are presented instead of an overall image. Figure 10.41 demonstrates a cleaned portion of the shaft. This shaft did not have any noticeable voids like 95-1. As seen from the cleaned portions the concrete surface was smooth.

Figure 10.40 Shaft 95-3 pre-cleaning.

Figure 10.41 Shaft 95-3 sections post-cleaning.

Pier 60 (6-ft diameter, double cage)

Pier 60 contains 6-ft diameter Type 1 reinforced shafts (Figure 10.19(c)) and was selected to have two windows of casing removed. This pier was chosen as it had no markings of any prior casing removal from the dive reports and could serve as a comparison of fresh concrete to the inspections performed on Piers 84 and 85. A map indicating which shafts had casings removed is shown in Figure 10.42. While the shafts of Pier 60 have a double reinforcement cage, both windows of removed casing showed clean, smooth concrete (Figures 10.43 and 10.44). For comparison, this differs from what was noted with piers 84 and 85 after the concrete surface was cleaned.



= a window was cut and concrete was noted to be good by the diver
= a window was cut and concrete was noted to be bad by the diver (pictures included)
= no window was cut

Figure 10.42 Pier 60 casing window removal and concrete surface condition map.

Figure 10.43 Pier 60 SW shaft (60-2).

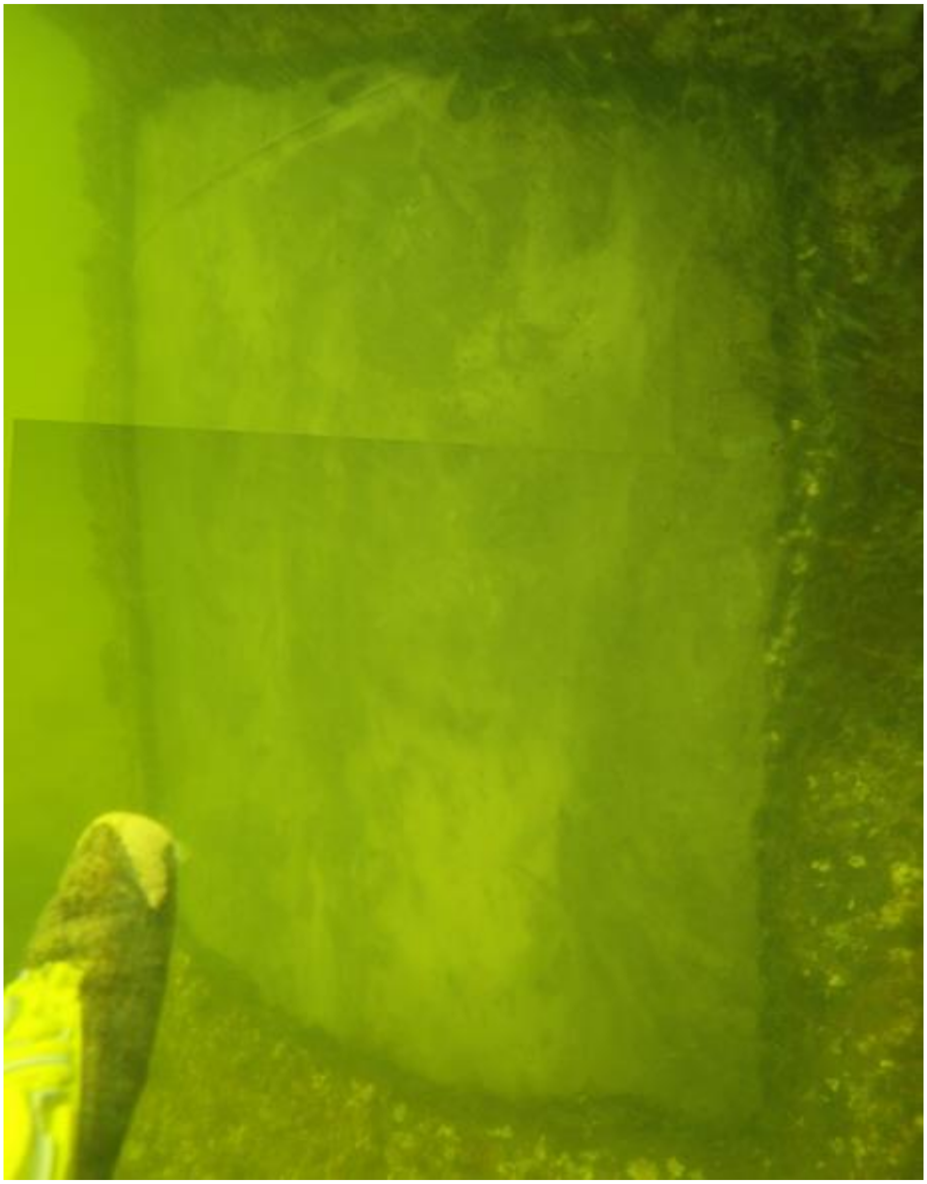
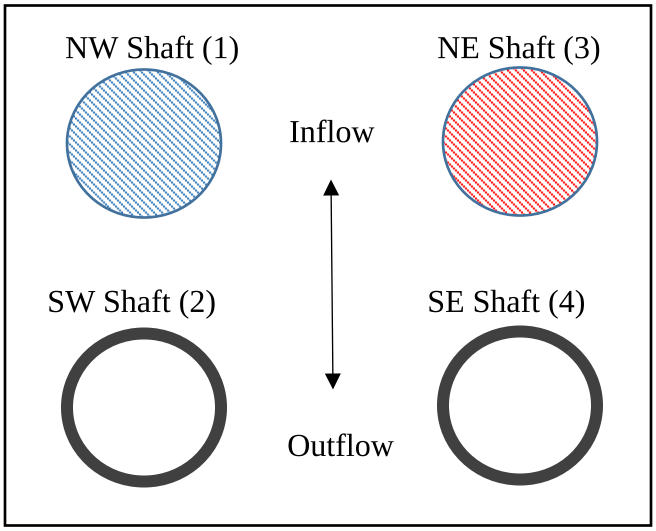


Figure 10.44 Pier 60 SE shaft (60-4).

Pier 70 (7-ft diameter, double cage)

For comparison to the shafts inspected on Pier 71, Pier 70 was selected for casing removal as it also has 7-ft diameter Type 1 shafts (Figure 10.19(d)), however, there was no evidence of previous casing removal. Thus, two casing windows were cut and removed on shafts 70-1 and 70-3. This revealed the fresh concrete surface, in comparison to one covered by sea life. A map similar to that shown for Pier 60 can be viewed in Figure 10.45. Note while shaft 70-1 (Figure 10.46) displayed smooth concrete, shaft 70-3 showed a rough surface with light creasing and visible aggregate (Figure 10.47).



= a window was cut and concrete was noted to be good by the diver
= a window was cut and concrete was noted to be bad by the diver (pictures included)
= no window was cut

Figure 10.45 Pier 70 casing window removal and concrete surface condition map.

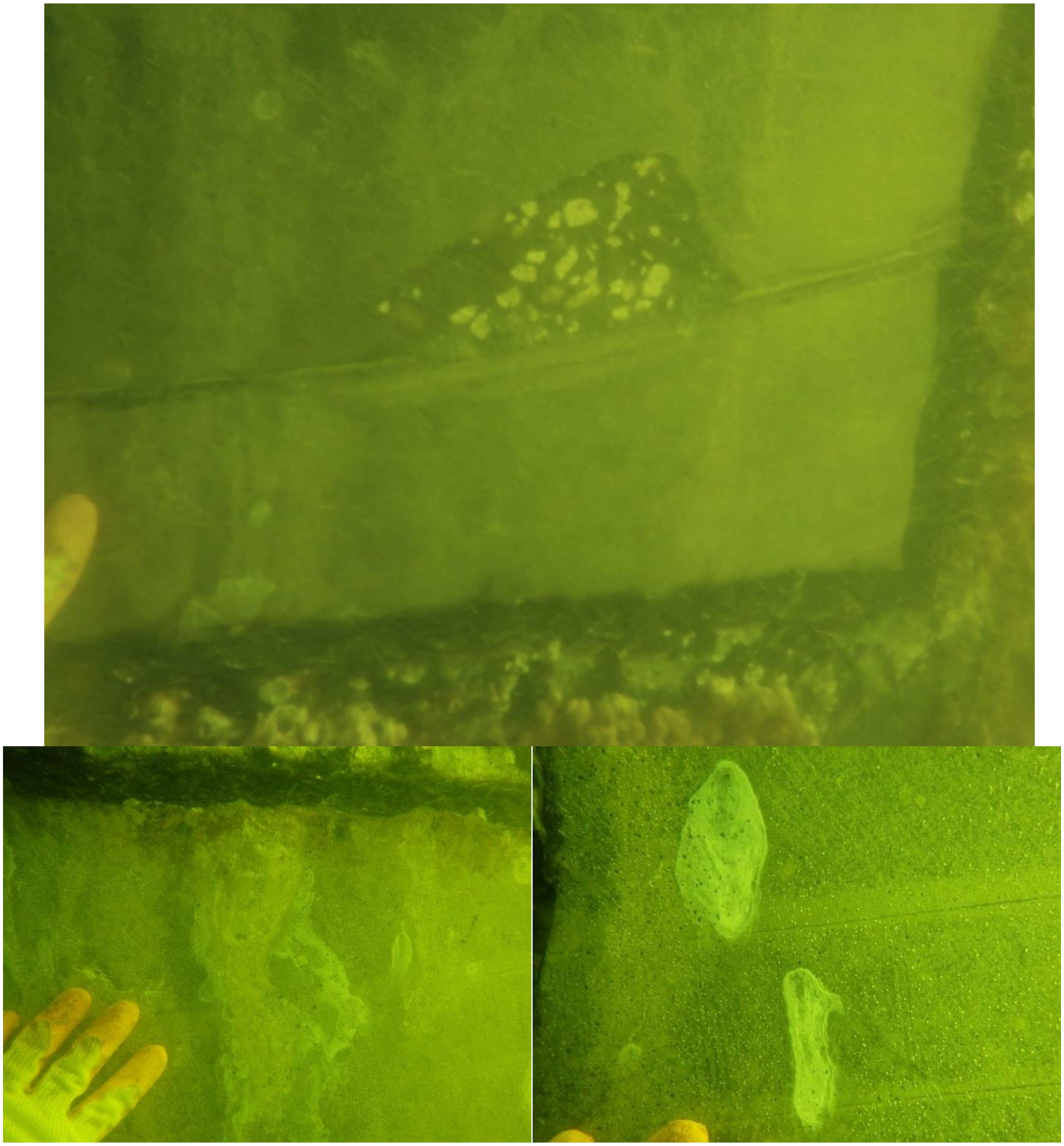


Figure 10.46 Shaft 70-1.

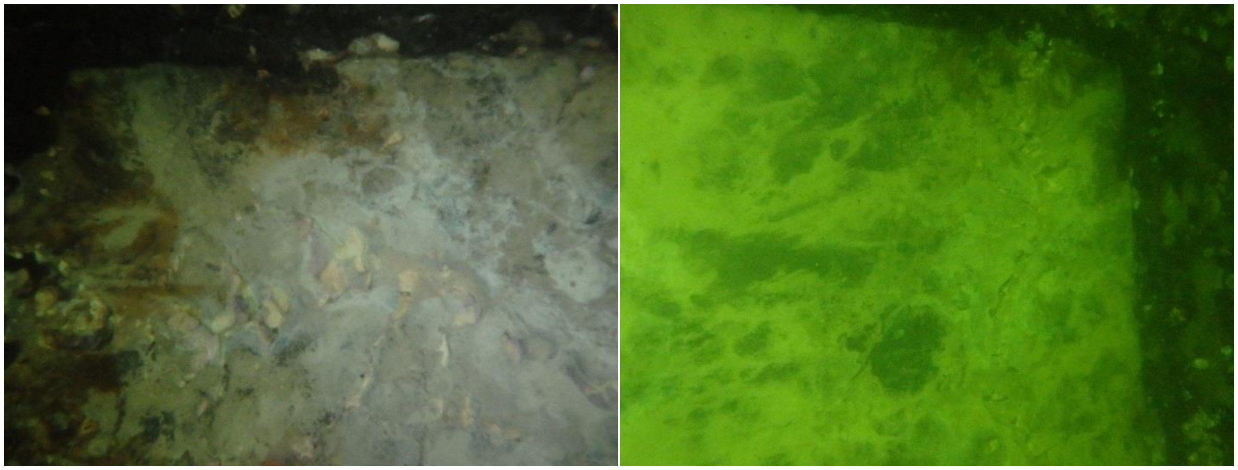
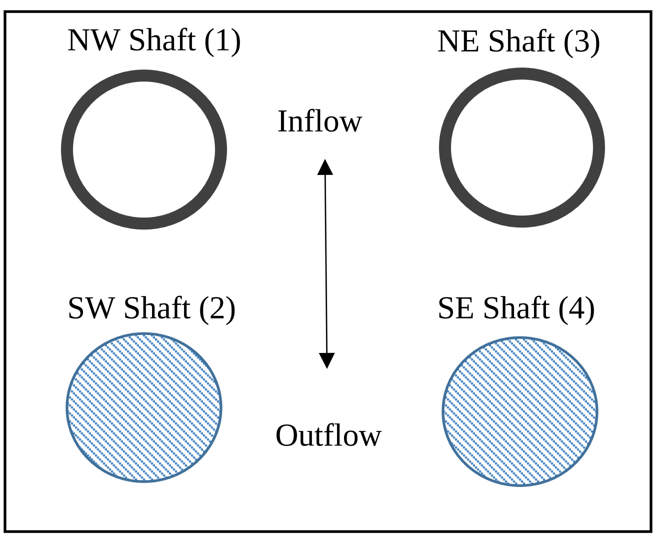


Figure 10.47 Shaft 70-3.

Pier 91 (4-ft diameter, single cage)

Pier 91 was chosen to have two windows of steel casing removed to inspect the fresh concrete surface as there were no notes of previous casing removal. Pier 91 has shafts 4 ft in diameter with a Type 1 reinforcement cage (Figure 10.19b). This can be compared with the cleanings and observations noted at Piers 94 and 95. Figure 10.48 displays the shaft formation as well as which shafts had casing removed and their surface condition upon removal. The legend is displayed at the base of Figure 10.48. The blue shading indicates both shafts exhibited ideal, smooth concrete surfaces. Figure 10.49 is an indicator of this as it shows a photograph of Shaft 91-2.



= a window was cut and concrete was noted to be good by the diver

= a window was cut and concrete was noted to be bad by the diver (pictures included)

= no window was cut

Figure 10.48 Pier 91 casing window removal and concrete surface condition map.

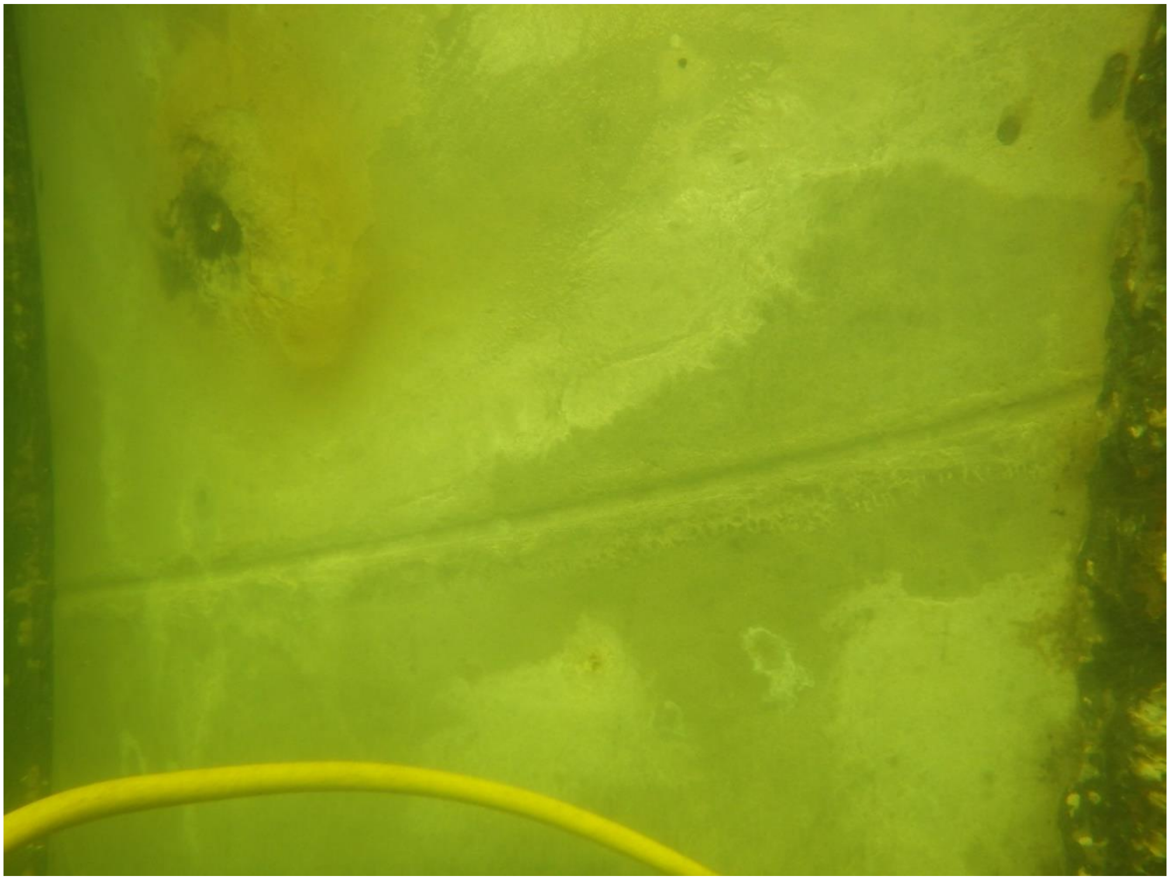


Figure 10.49 Shaft 91-2 (SW) immediately after casing window removal.

10.2.2 Santa Fe River Bridge 260112 (US-441 over the Santa Fe River)

The Santa Fe River Bridge has three intermediate piers (Figure 10.51), each on two 5-ft shafts (Figure 10.50). Two piers were in the river at the time of the on-site review (Figure 10.52). The shafts for this bridge were cast using bentonite slurry, making this bridge a good candidate for the study. According to the plans, shafts for this bridge terminate very close to the mudline and are mono-shaft column structures. However, considering river bottom depth changes after looking at the inspection report, it was decided that this bridge yielded potential for investigation. The piers were investigated first using a remote-operated vehicle (ROV) (Figure 10.53), then by divers (Figure 10.54). While it was expected that only the very top of the shaft would potentially be available, on-site inspection revealed depth changes due to scouring were not enough to expose the shafts. All four shaft/columns were investigated and only the smaller diameter 3-ft columns could be seen (Figure 10.55).

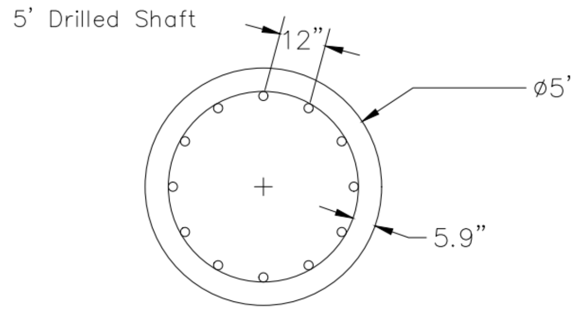


Figure 10.50 Intermediate drilled shaft layout

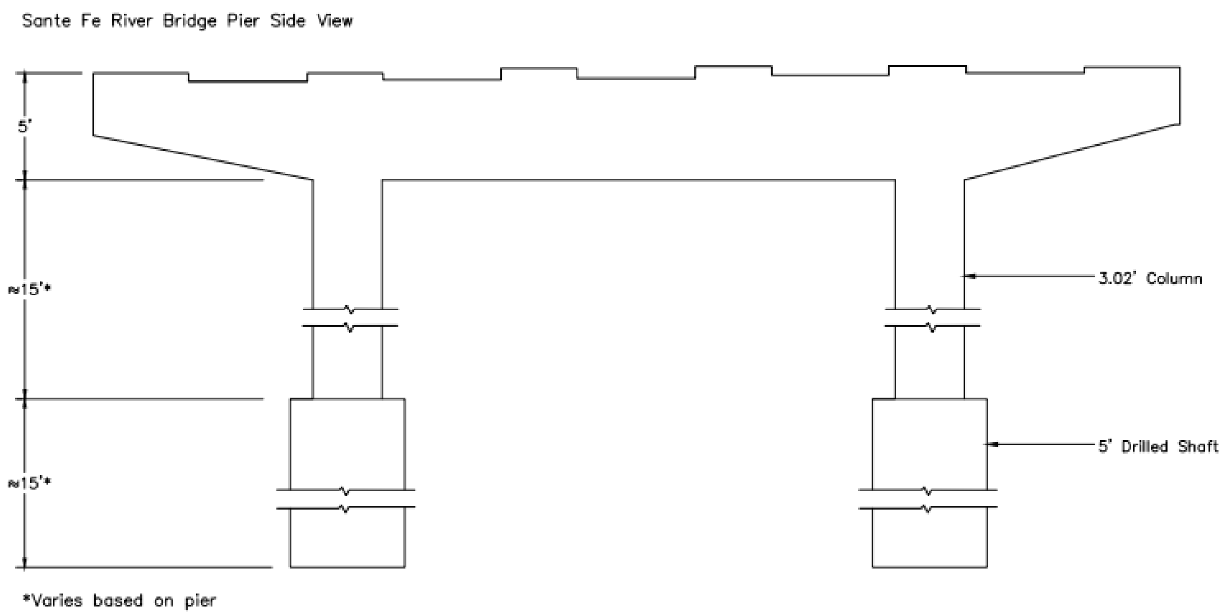


Figure 10.51 Layout for intermediate piers, height of column and shaft approximated as it varies by pier.

Figure 10.52 Four water column-to-shaft structures.

Figure 10.53 Remote operated vehicle system used to capture underwater images without divers.

Figure 10.54 Diver follow-up to further review column-to-shaft interface.

Figure 10.55 One of the columns at the mudline, no exposed shaft available for inspection.

10.2.3 **Blountstown Bridge 470052 (SR-20 over Apalachicola River)**

The SR-20 bridge over the Apalachicola River (Figure 10.56), bridge number 470052 built in 1998 (Figure 10.57), was deemed to be a good inspection candidate as the 9-ft shafts were cast with bentonite slurry and the plans noted the casing was to be burned off down to an elevation of 30 ft. That meant there should have been at least a portion of exposed shaft for investigation. Figure 10.58 shows the plan view of Piers 58 and 59 as seen in the plans. Figure 10.59 highlights the strut in the top view and Figure 10.60 best demonstrates the column going into the shaft through the section view, as well as the reinforcement layout and where the casing was noted to be burned off. At the time of these pictures, clearance was 61 ft and water elevation was 35 ft, making the water 17.5 ft deep.

Figure 10.56 Main span of the Blountstown Bridge.

(a)

(b)

Figure 10.57 (a) Date the bridge was built, (b) title of bridge and bridge number as seen on bridge.

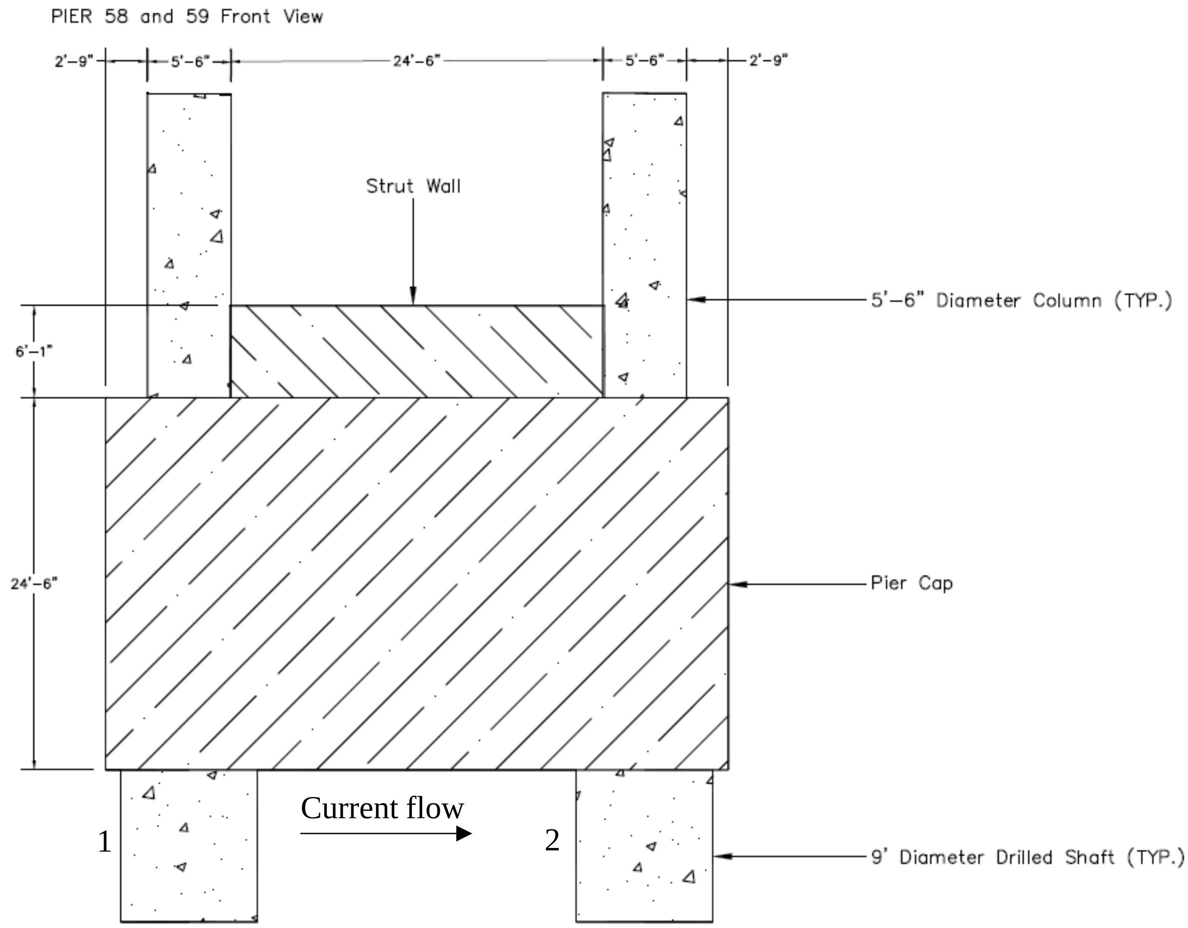


Figure 10.58 Elevation (front) view of Piers 58 and 59 looking east, river flow is to the right.

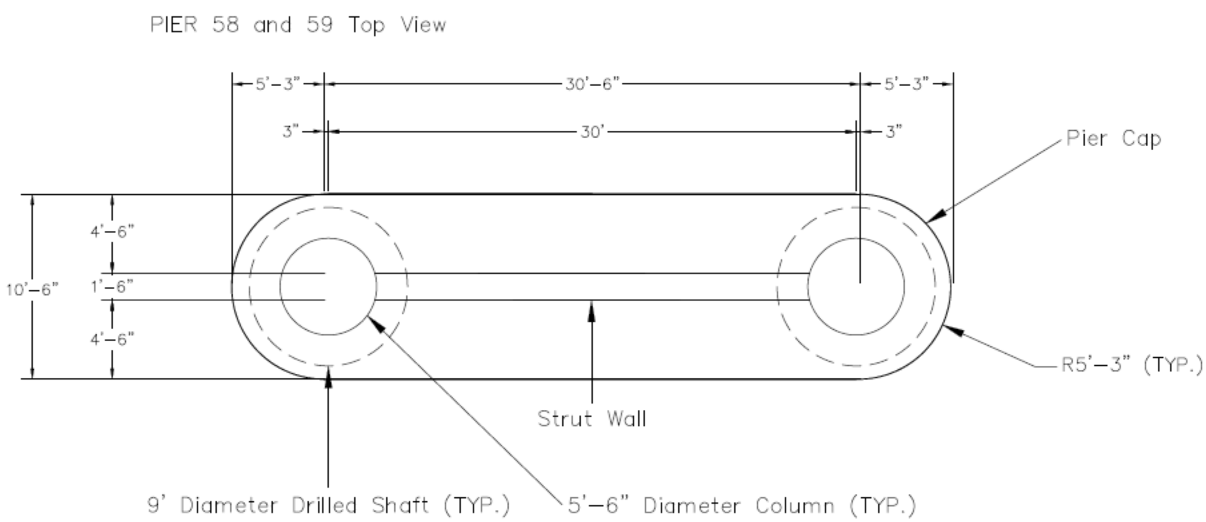


Figure 10.59 Top view of Piers 58 and 59.

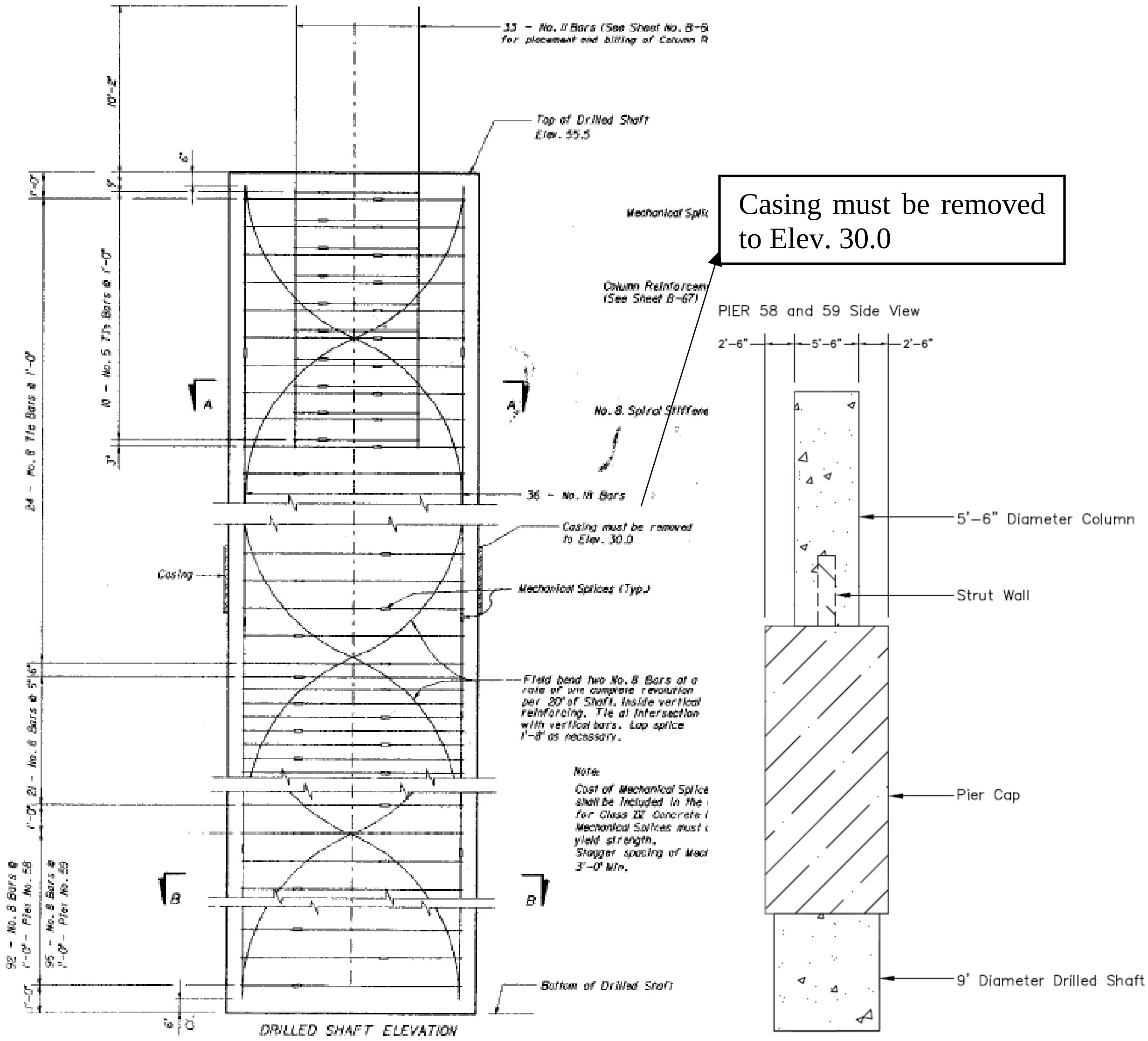


Figure 10.60 Section view of Piers 58 and 59.

Initial Inspection and Cleaning

Pier 58 was selected initially for cleaning and inspection. While it was expected that there would be exposed shaft, after inspection it was realized that the only portion exposed was from what looked like a missing seal slab support collar (Figure 10.61). Rubble on the bottom or river was likely to be remnants of the seal slab, and a steel collar was seen around shaft 2. Nevertheless, approximately 16-18 inches of uncased shaft was visible. Figure 10.62 is a picture of what the exposed portion of shaft looked like prior to cleaning shaft 58-1 (note shafts were labeled 1 and 2 based on river flow seen in Figure 10.58). Figure 10.63 shows the 58-1 shaft after cleaning. Figure 10.63 also shows four positions labeled 1-4 corresponding to close-up images taken at those locations. The water clarity was poor and made overview images unhelpful. In all four images of

Figure 10.63, vertical or horizontal creases were found; image (2) shows both vertical and horizontal creases (camera slightly tilted) along with a white patching compound in the center of the squares that was not concrete.

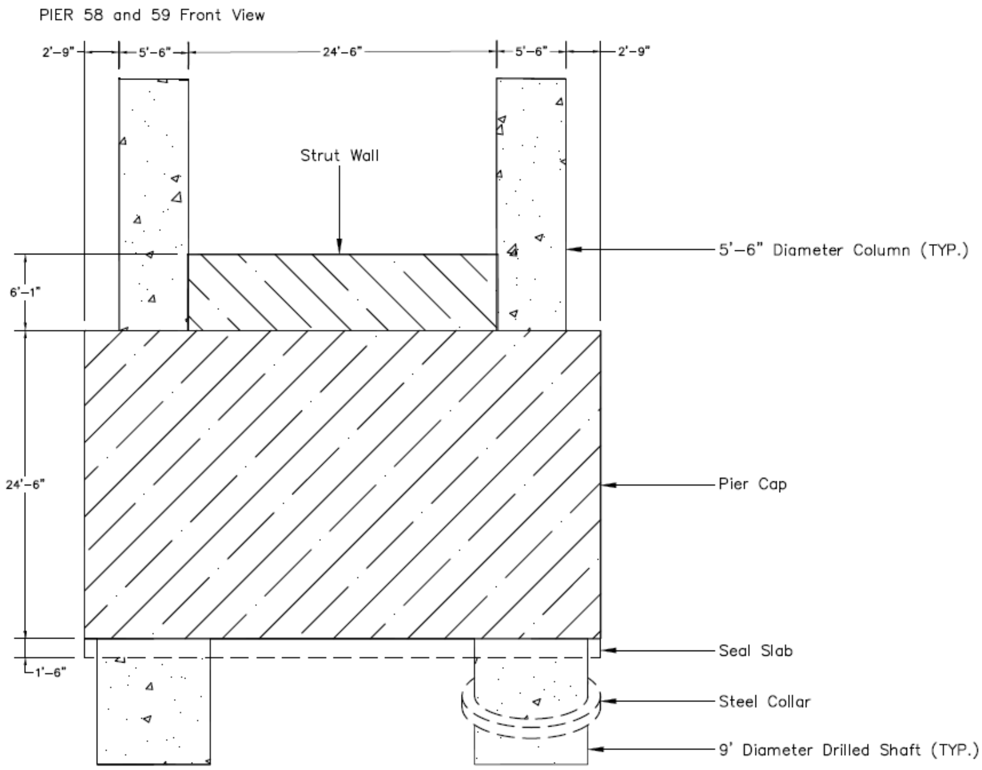


Figure 10.61 Front view of Pier 58 with seal slab and observed steel collar around shaft 2.

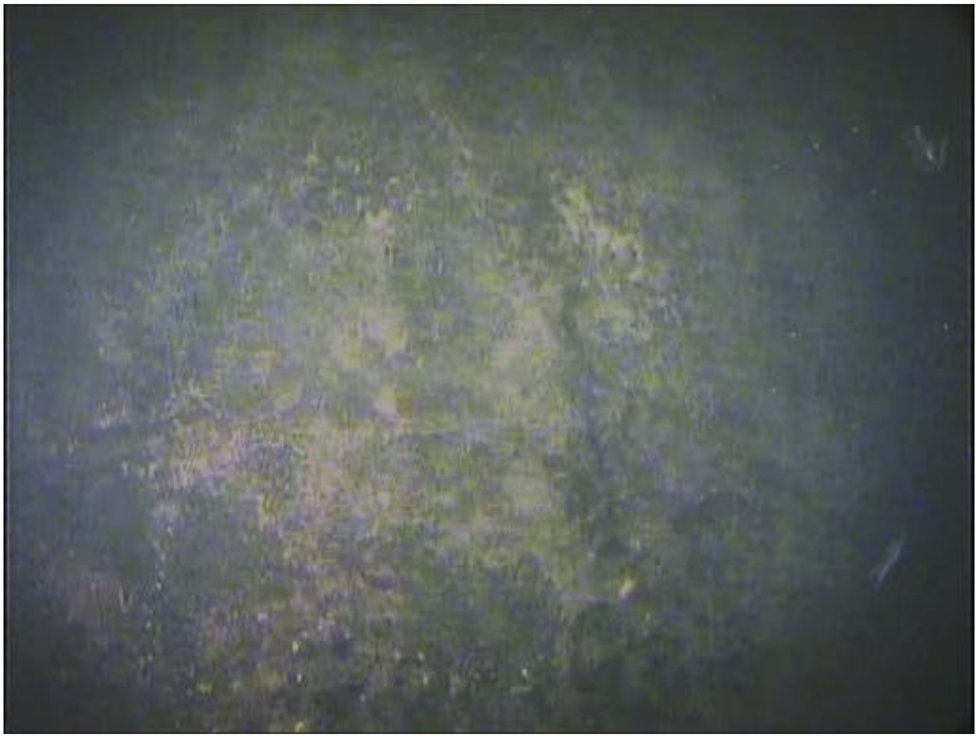


Figure 10.62 Shaft 58-1 prior to cleaning

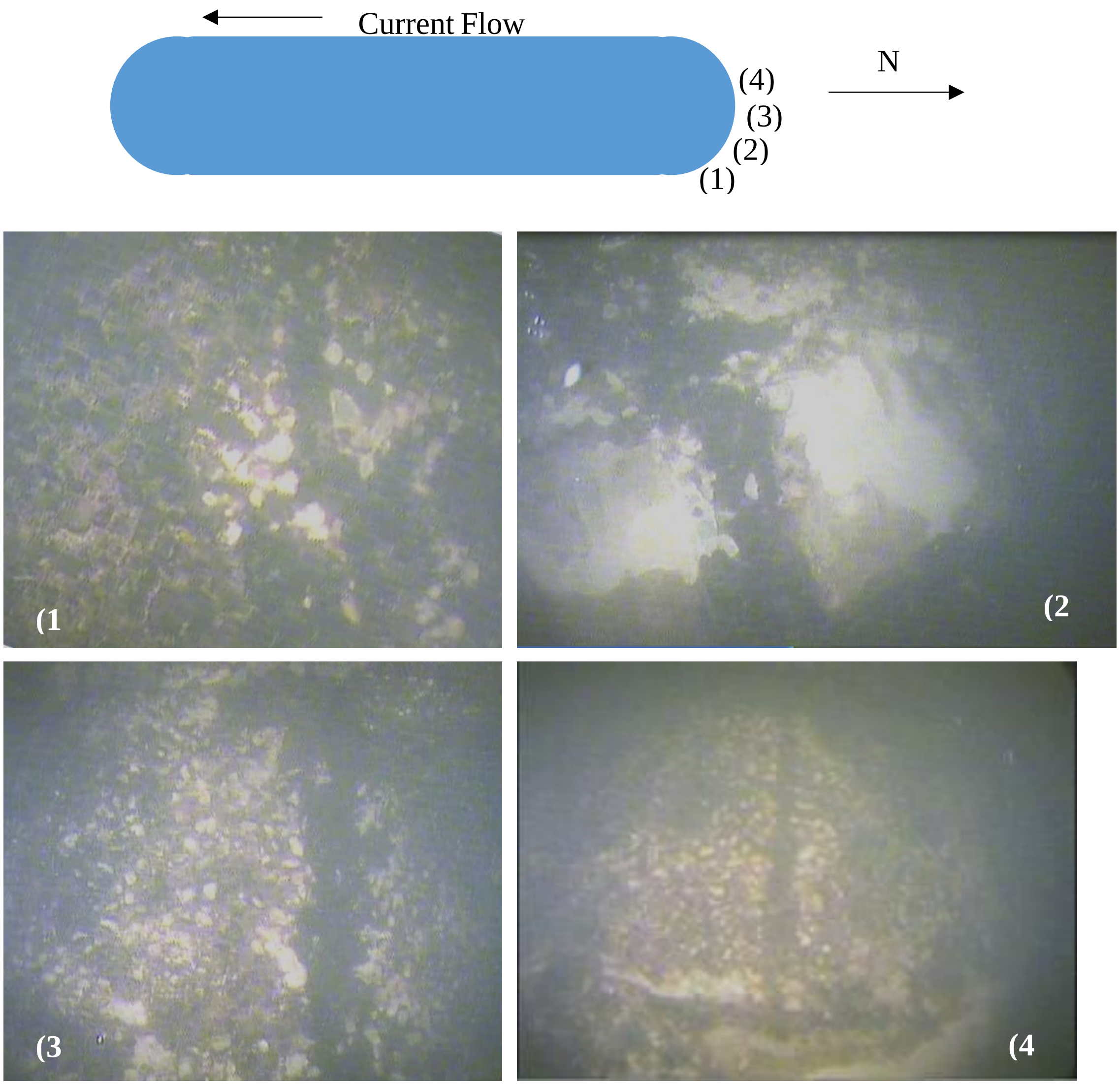


Figure 10.63 Images demonstrating quilting on Shaft 58-1. Images 1-4 correlate to positions 1-4 noted at the top of the figure.

Shaft 58-2 showed the same quilting as noted with shaft 58-1 prior to cleaning. This in part was based on experience from shaft 58-1. However, this shaft still had a portion of the collar (noted in the plans) partially dislodged and precariously leaning against the shaft on the south (downstream) side. The collar was seen as a safety hazard, and along with an increasing stream flow/current, conditions did not allow for a more thorough initial inspection.

Adjacent Pier 59, test shaft 8 (Figure 10.64) was also inspected. This is a 9-ft diameter out-of-position shaft south of pier 59 near the main channel. By visual inspection there was some light creasing along the vertical reinforcement (Figure 10.65). Figure 10.66 shows test shaft 8 before cleaning; Figure 10.67 shows after cleaning. Again, the creasing is light but it can be seen. Being a test shaft, construction sequencing is often not exactly the same as production shafts. This shaft had additional longitudinal instrumentation, which compounds concrete flow problems.

Figure 10.64 Test Pile adjacent (west) of Pier 59.

Figure 10.65 West side of test shaft 8, light creases from vertical reinforcement.

Figure 10.66 East side of test shaft 8 pre-cleaning.

Figure 10.67 East side of test shaft 8 post-cleaning.

Casing Removal

After initial observations, this bridge was selected for casing removal. The first window of casing was removed from shaft 58-2. This first attempt at casing removal (where 58-2 was removed) led to all day efforts just for the removal of one casing window as the river level then was 15.86, where anything above 17 is considered flooding. On this trip the high river levels proved to be an issue. USF's first visual inspection occurred at a river level of 6.56, where the current was swift, but the conditions were workable. Thus, after this the river levels were closely monitored for workable conditions (Figure 10.68), which took about 5 months to achieve.

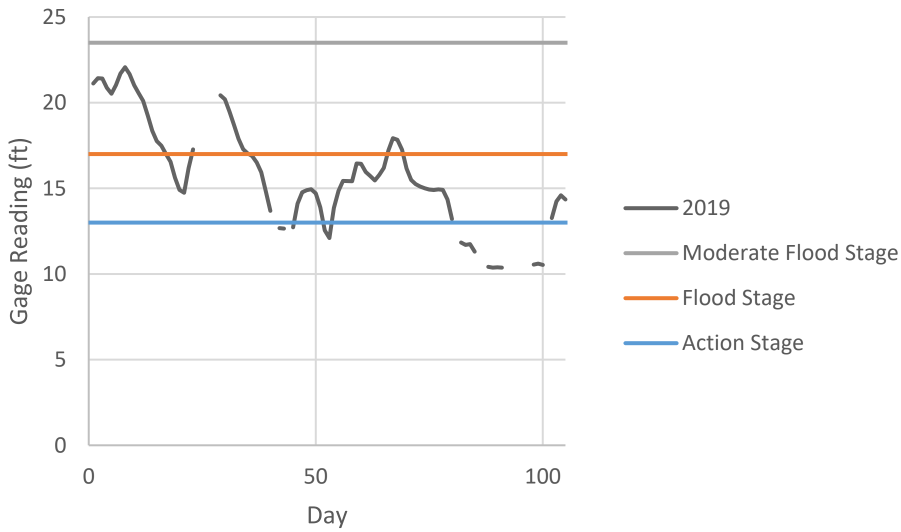


Figure 10.68 Blountstown river levels for 2019.

Once the river level decreased to a workable level, the dive team returned for casing removal. Casing windows were removed from each shaft from an upper and a lower portion of the shaft. The previous window cut from shaft 58-2 was counted as a top window. Figures 69 to 76 display the concrete conditions observed from each cutout window in the casings for shafts 58-1, 58-2, 59-1, and 59-2, top and bottom, respectively. Every shaft showed a highly degraded concrete surface. In most cases the images were taken from video that swept across the shaft at a very close distance due to water quality conditions. Figure 10.69 shows a compilation of many images in their approximate position estimated from the video speed and closeness.

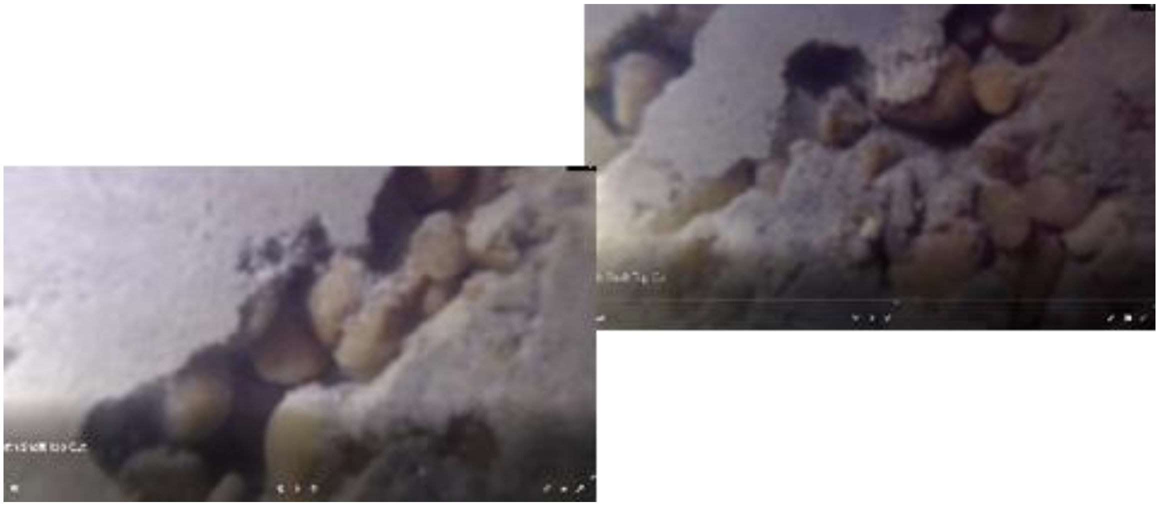


Figure 10.69 Shaft 58-1 casing removed from the upper portion of the shaft (close-up).

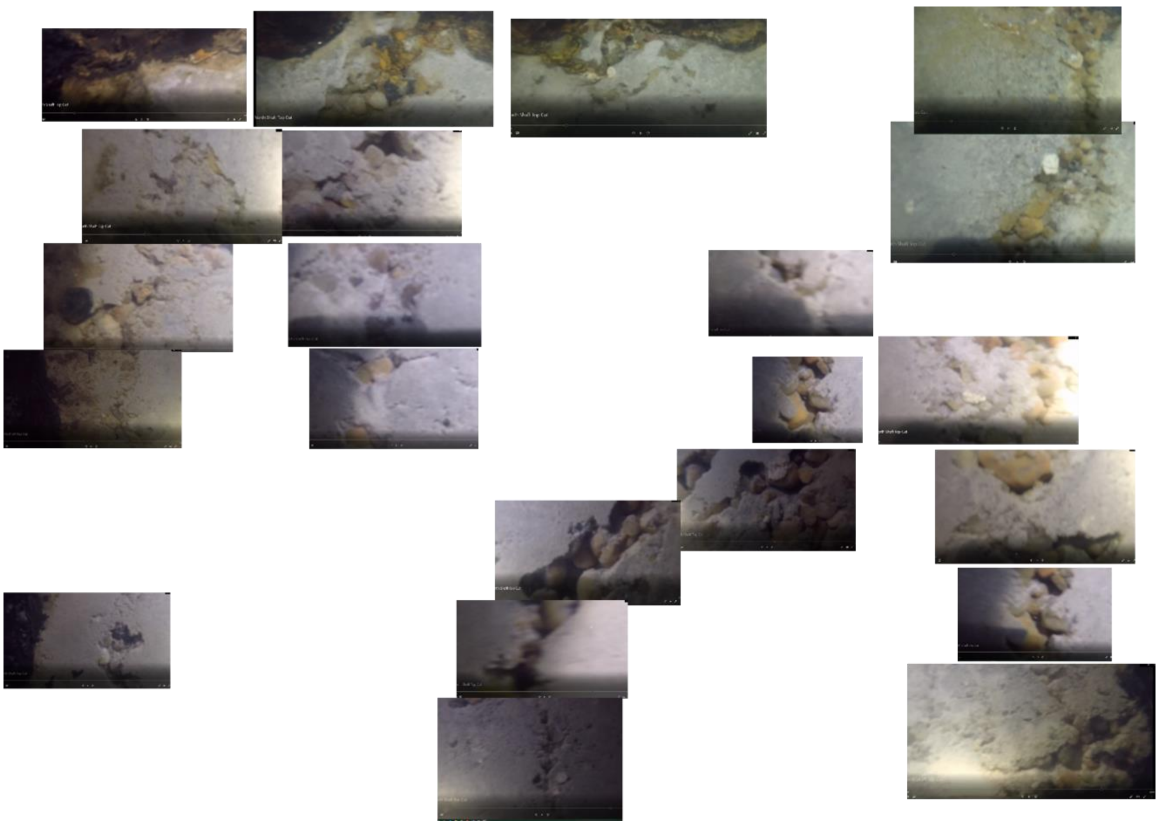


Figure 10.69 Shaft 58-1 stitched together images showing defects in upper portion of the shaft.

Figure 10.70 Shaft 58-1 casing removed from the lower portion of the shaft.

Figure 10.70 Shaft 58-1 casing removed from the lower portion of the shaft (continued).

Figure 10.70 Shaft 58-1 casing removed from the lower portion of the shaft (continued).

Figure 10.71 Images of Shaft 58-2 with the casing removed from the upper portion of the shaft demonstrating quilting and poor surface quality .

Figure 10.72 Shaft 58-2 with the casing removed from the lower portion of the shaft .

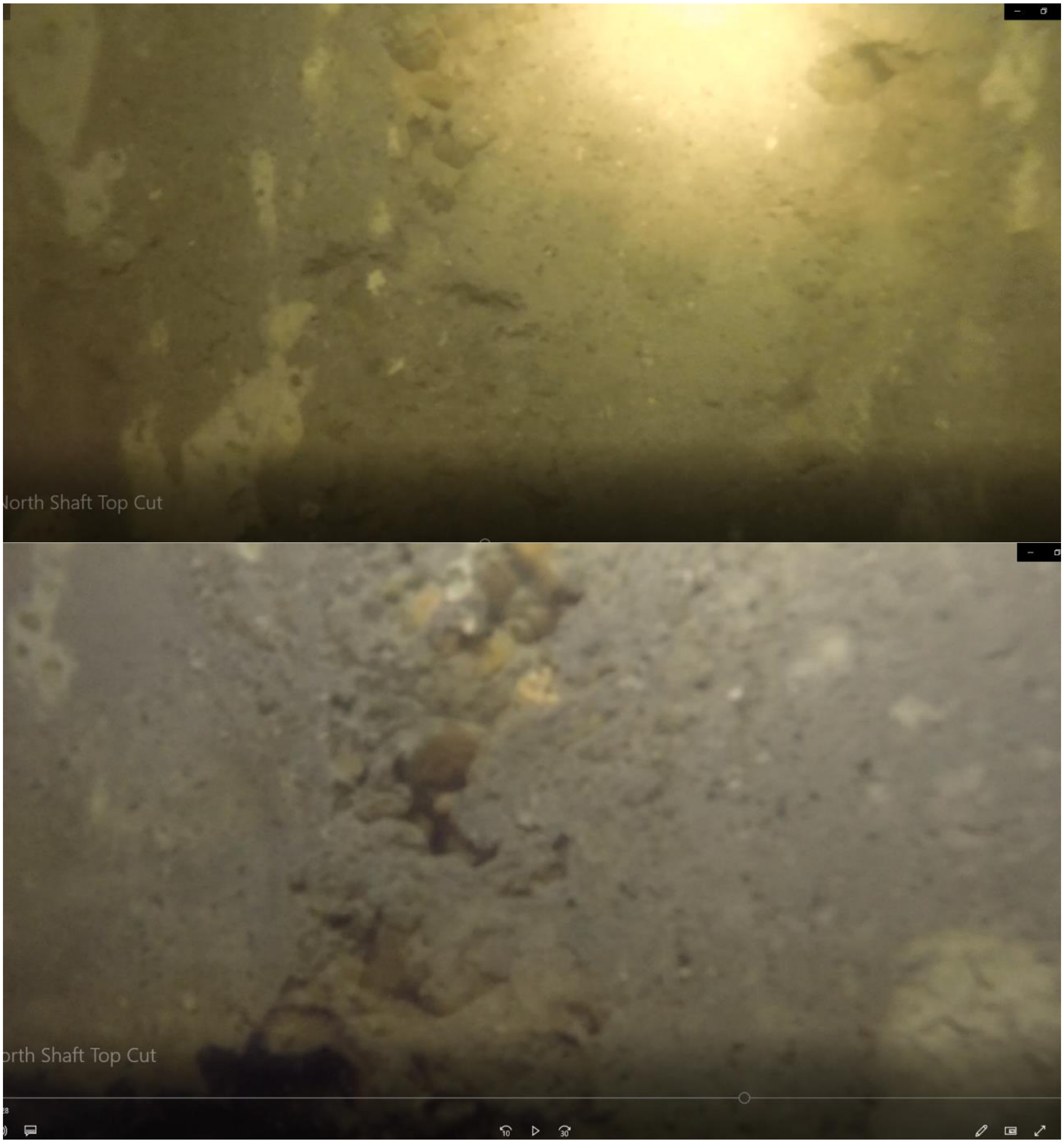


Figure 10.73 Shaft 59-1 with the casing removed from the upper portion of the shaft.

Figure 10.74 Shaft 59-1 with the casing removed from the lower portion of the shaft.

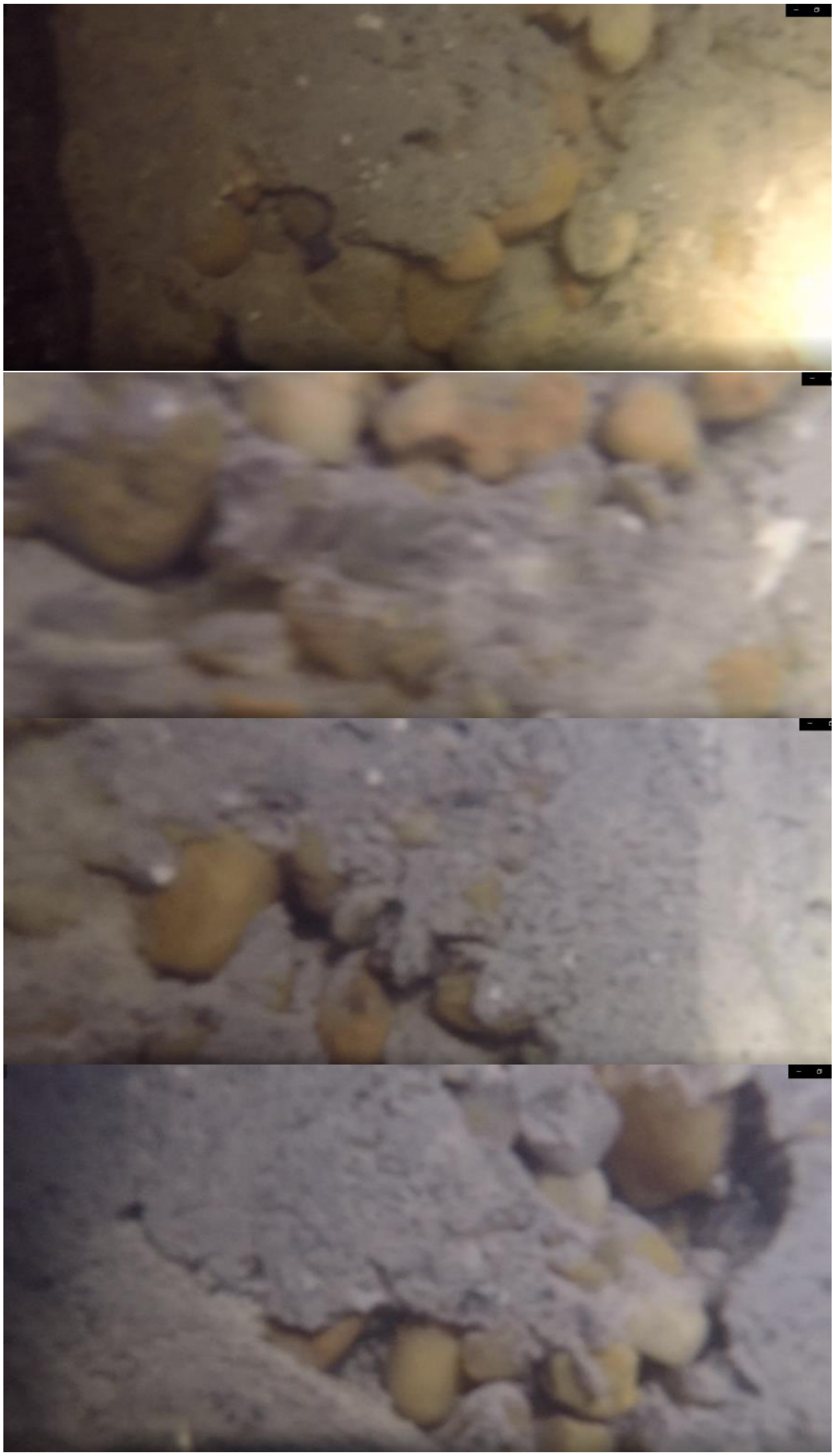


Figure 10.75 Shaft 59-2 with the casing removed from the upper portion of the shaft.

Figure 10.76 Shaft 59-2 with the casing removed from the lower portion of the shaft.

10.2.4 **Bridge of Lions 780074 (A-1-A over Matanzas River)**

When Bridge of Lions was rehabilitated, shafts were constructed using attapulgite slurry around the existing piers and a below-water footing was cast to tie the existing caissons to the new sister shafts (Figure 10.79 and 10.80). Figure 10.79 shows the 8-ft diameter casing at the corners of the west bascule pier. These shafts were those indicated as possible locations for investigation. Figure 10.80 shows four 8-ft casings around each of the neighboring piers 13 and 14.

Figure 10.79 West Bascule (Pier 15 implied via plans / Pier 11 via inspection report) during rehabilitation; image taken from SE side looking NW.

Figure 10.80 West Bascule (Pier 15; right), Piers 14 and 13 moving to the left with four 8-ft shaft casings shown from south looking north during rehabilitation.

Based on dive/inspection reports this bridge was deemed a good candidate for inspection and comparison. Inspection note:

> *"the SW and NW drilled shafts on Pier 10, NE and SE on Pier 11 have larger round steel casings extending up from the groundline to within 6 ft of the bottom of the footing"*

While the plan set never assigns a pier number to the bascule piers (only west or east bascule), the inspection report seemed to imply that the west bascule was pier 10 and the east was pier 11, but these inspection logs reference pier numbering from the east, which is opposite the plan set. Nevertheless, this comment gave confidence that there may be two shafts on the bascule piers without casing. Unfortunately, the initial on-site examination, combined with the comment and the plan and profile view of the bridge, indicated that the diver was commenting on the shaft that transitioned from an 8-ft diameter shaft to a 5-ft diameter column. Much of the confusion stemmed from pier numbering differences; this can be seen in Figures 10.81 and 10.82 which show the discrepancy between pier numbering from inspection reports and original plan set, respectively. Nothing but steel casing was found upon dive investigation of these piers. Therefore, the Bridge of Lions (Figure 10.81) also required that casing be removed to view the effects of attapulgite slurry.

Figure 10.81. Existing main span bascule piers of Bridge of Lions (left); approach spans Pier 14 closest decreasing in number into the distance (per plan numbering).

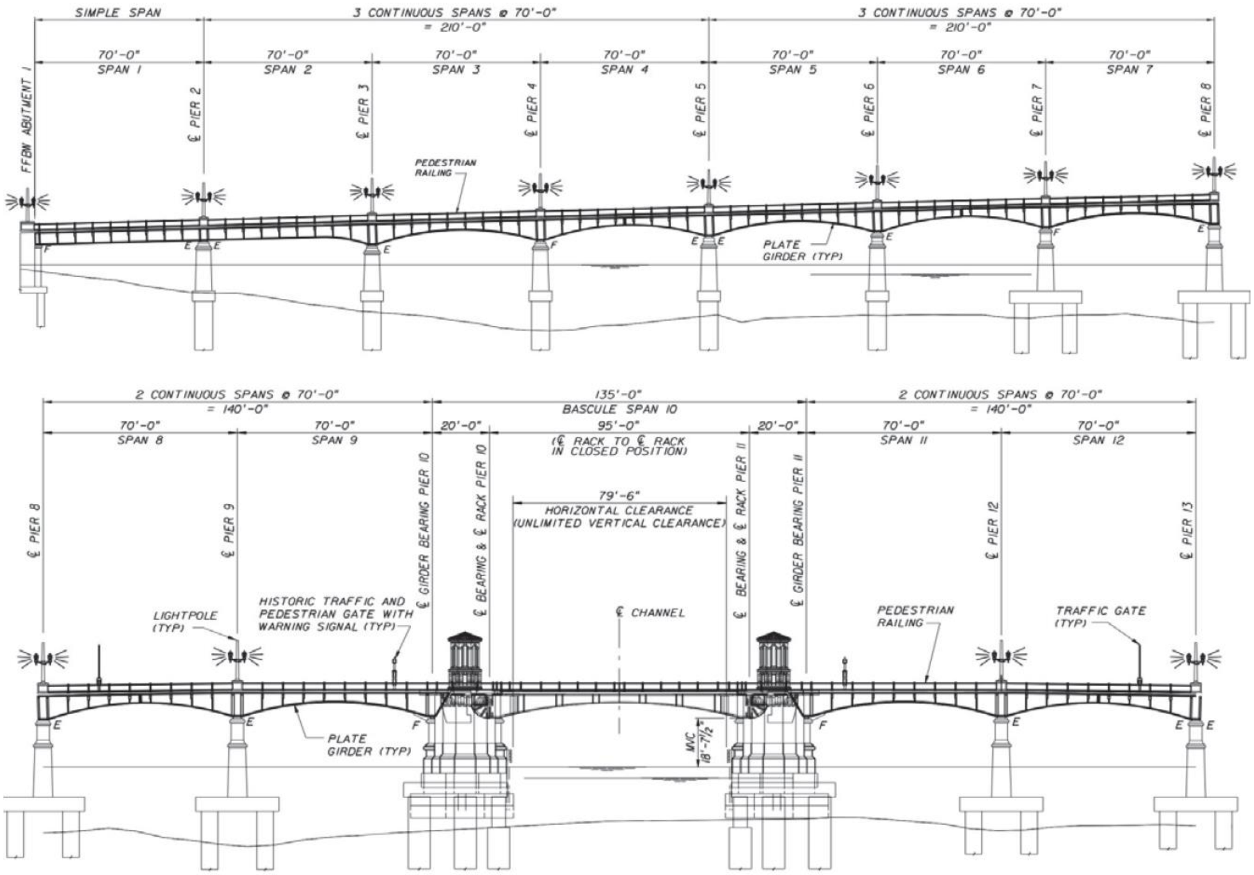


Figure 10.82 Plans from the inspection report noted as looking south which shows Pier 1 to be the east most end bent.

DRILLED SHAFT INSTALLATION TABLE										
INSTALLATION CRITERIA					DESIGN CRITERIA					
PIER OR BENT NO.	SHAFT SIZE ϕ (IN)	TIP ELEV. (FT)	MIN. TIP ELEV. (FT)	MIN. ROCK SOCKET LENGTH (FT)	FACTORED DESIGN LOAD (TONS)	DOWN DRAG (TONS)	LONG-TERM SCOUR ELEV. (FT)	100 YR. SCOUR ELEV. (FT)	ϕ	TOP OF SHAFT ELEV. (FT)
WEST ABUTMENT 1	36	-80	-80 ~~N/A~~	N/A	115	N/A	N/A	-41.2	0.55	-4.25
PIER 2	36	-88	-85 ~~N/A~~	N/A	163	N/A	N/A	-23.2	0.55	-7.75
PIER 3	36	-88	-85 ~~N/A~~	N/A	163	N/A	N/A	-25.0	0.55	-9.75
PIER 4	36	-88	-85 ~~N/A~~	AB N/A	163	N/A	N/A	-26.4	0.55	-9.75
PIER 5	36	-88	-85 ~~N/A~~	N/A	163	N/A	N/A	-26.9	0.55	-9.75
PIER 6	90	-101	-100	N/A	569	N/A	N/A	-34.4	0.55	-14.75
PIER 7	90	-108	-100	N/A	569	N/A	N/A	-37.3	0.55	-14.75
PIER 8	90	-100	-100	N/A	569	N/A	N/A	-36.8	0.55	-14.75
PIER 9	90	-100	-100	N/A	569	N/A	N/A	-37.9	0.55	-14.75
PIER 10	90	-109	-100	AB N/A	569	N/A	N/A	-39.2	0.55	-14.75
PIER 11	90	-110	-105 ~~-110~~	N/A	569	N/A	N/A	-43.9	0.55	-14.75
PIER 12	72	-109	-100 ~~-105~~	N/A	395	N/A	N/A	-48.5	0.55	-15.75
PIER 13	72	-105	-100 ~~-105~~	N/A	395	N/A	N/A	-47.8	0.55	-15.75
PIER 14	72	-106	-100 ~~-105~~	N/A	395	N/A	N/A	-50.3	0.55	-15.75
PIER 15 SHAFTS 1, 2, & 3	96	-135	-135	N/A	910	N/A	N/A	-46.1	0.55	-9.75
PIER 15 SHAFTS 4 & 5*	96	-135	-135	N/A	910	N/A	N/A	-46.1	0.55	-22.5
PIER 16 SHAFTS 1 & 2*	96	-135	-135	N/A	910	N/A	N/A	-50.3	0.55	-22.5
PIER 16 SHAFTS 3, 4, & 5	96	-135	-135	N/A	910	N/A	N/A	-50.3	0.55	-9.75
PIER 17	72	-105	-105	AB N/A	395	N/A	N/A	-56.9	0.55	-15.75
PIER 18	72	-105	-100 ~~-105~~	N/A	395	N/A	N/A	-50.5	0.55	-15.75
PIER 19	72	-105	-100 ~~-105~~	N/A	395	N/A	N/A	-48.3	0.55	-15.75
PIER 20	90	-110	-105 ~~-110~~	N/A	569	N/A	N/A	-45.2	0.55	-14.75
PIER 21	90	-110	-105 ~~-110~~	N/A	569	N/A	N/A	-44.8	0.55	-14.75
PIER 22	90	-110	-110	AB N/A	569	N/A	N/A	-51.6	0.55	-14.75
PIER 23	90	-110	-105 ~~-110~~	N/A	569	N/A	N/A	-42.0	0.55	-14.75
PIER 24	90	-103	-95 ~~-100~~	N/A	569	N/A	N/A	-32.3	0.55	-14.75
EAST ABUTMENT 25	36	-97	-97 ~~N/A~~	N/A	205	N/A	N/A	-37.8	0.55	-8.75
EAST SEAWALL	36	-75	-75 ~~N/A~~	N/A	135	N/A	N/A	-14.0	0.55	-8.75

Figure 10.83 List of piers as shown in plan set noting west-most abutment to be Pier 1.

Access to some drilling logs from the reconstruction was available, so these logs were compiled and inspected to choose the best shafts for casing removal (Table 10.18). Given there was no depth profile, the drilling logs were also used to determine the depth of the water for diving purposes. In addition to depth, the volume of concrete was also inspected to be aware of any abnormalities. It was stated that casing should not be removed from the main piers 15 and 16, thus these shafts were eliminated from consideration. Thus, based on depth profile and concrete volume, piers 17 to 19 were selected for casing window removal.

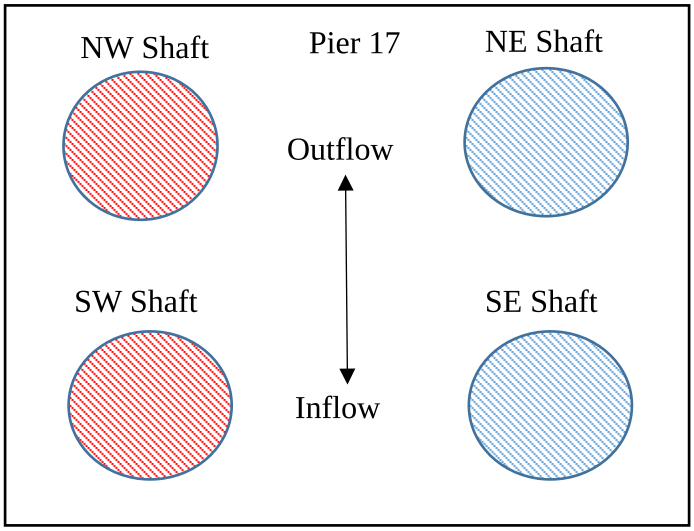
Table 10.18 Summary of shafts from drilling logs cast with Florigel (Attapulgite) during reconstruction.

Pier	Shafts
2	1, 3, 4
3	1, 2, 3, 4
4	1, 2, 3, 4
5	1, 2, 3, 4
12	2
13	2, 3, 4
15	4, 5
16	1, 2, 3, 4, 5
17	1, 2, 3, 4
18	1, 2, 3, 4
19	1, 2, 3, 4
20	1, 2
22	2
23	2
25	1, 2, 3, 3B, 4, 4B, 5, 6, 7, 8, 9, 10, 11, 12, 13, 15
SE Seawall	1, 3
NE Seawall	2
E Abutment	2, 14
SESW	4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 16, 18R

In total, 11 of the 12 shafts (indicated by green shading in Table 10.18) were able to have a window of casing removed. As it is not specified which shaft is 1 to 4 in correlation with direction, each shaft is referenced by geographical location (i.e. NW) from this point forward.

Pier 17 (6-ft diameter, single cage)

For Pier 17, all four shafts had a window of casing removed. On this pier, initial inspection showed two shafts with clean concrete, and two with issues. A summary of this and the corresponding visual inspection notes are seen in Figure 10.84. Figures 10.85 and 10.86 display photographs taken of the NW and SW shafts, respectively, showing images of the poor concrete quality.



= a window was cut and concrete was noted to be good by the diver

= a window was cut and concrete was noted to be bad by the diver (pictures included)

= no window was cut

Figure 10.84 Pier 17 casing window removal and concrete surface condition map.

Figure 10.85 Pier 17 NW shaft concrete quality.

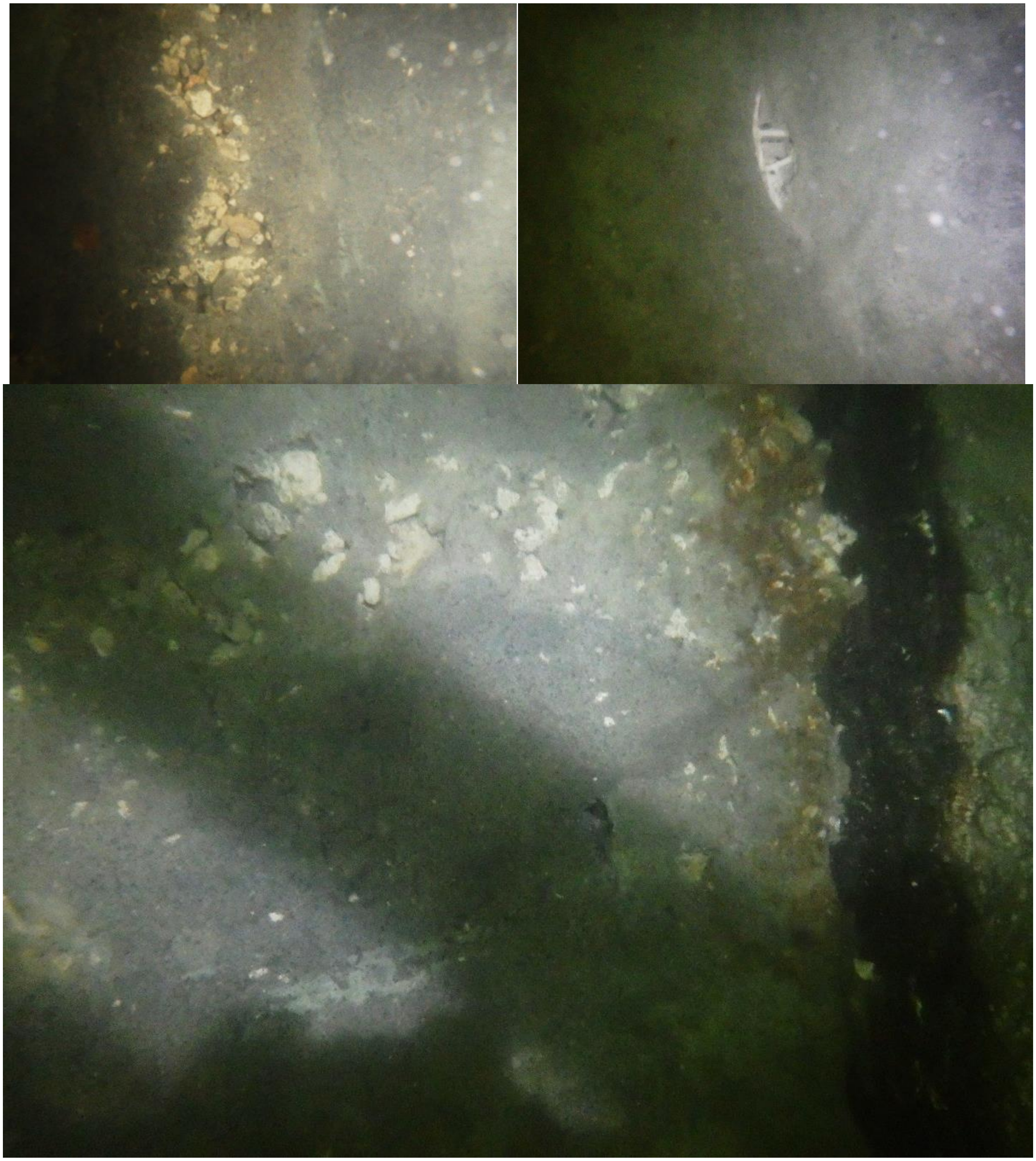
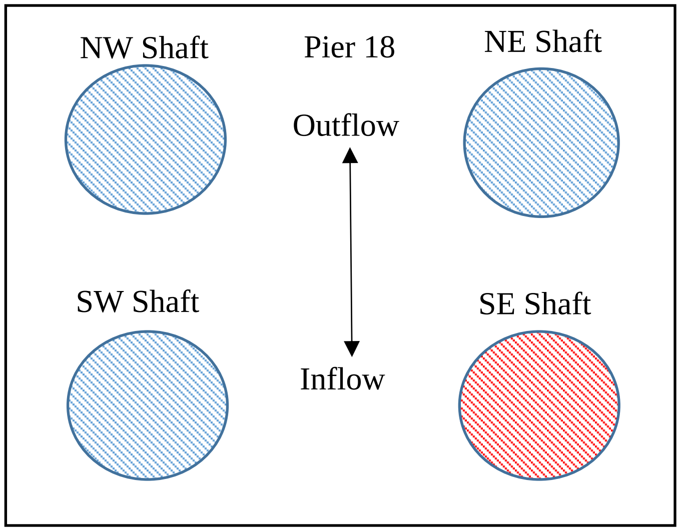


Figure 10.86 Pier 17 SW shaft concrete quality.

Pier 18 (6-ft diameter, single cage)

Pier 18 also had a window of casing removed per shaft. Only the SE shaft displayed poor concrete quality, while the other three shafts were noted to have clean concrete surfaces. A summary of this and the corresponding visual inspection notes is seen in Figure 10.87. Figure 10.88 displays photographs taken of the SE shaft.



= a window was cut and concrete was noted to be good by the diver
= a window was cut and concrete was noted to be bad by the diver (pictures included)
= no window was cut

Figure 10.87 Pier 18 casing window removal and concrete surface condition map.

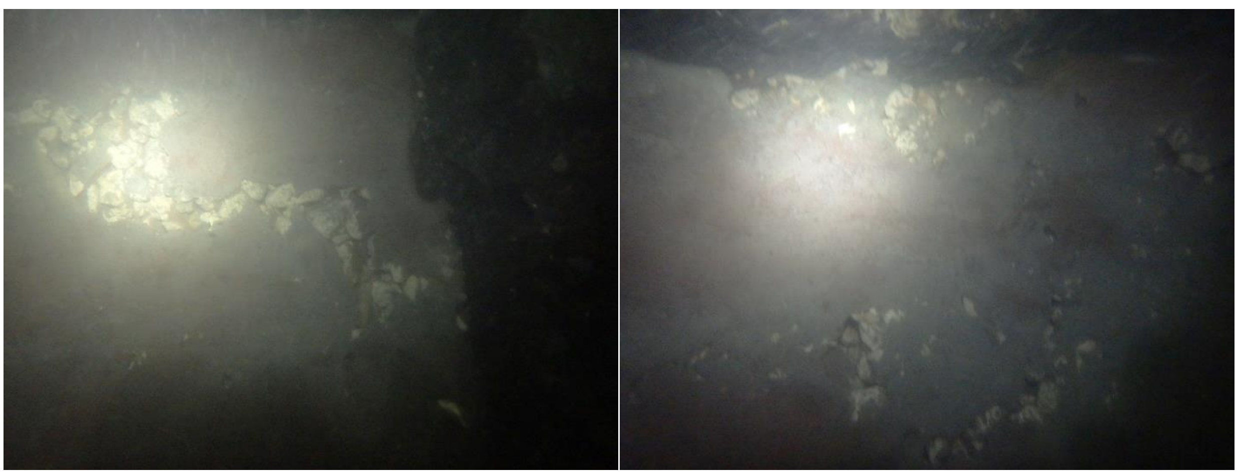
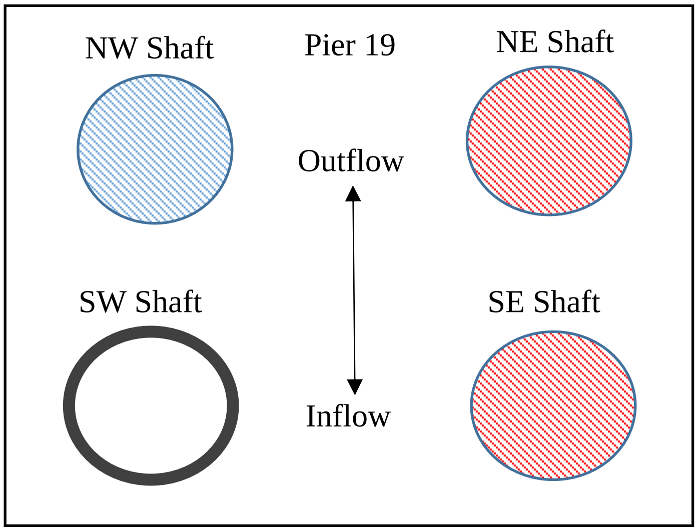


Figure 10.88 Pier 18 SE shaft concrete quality.

Pier 19 (6-ft diameter, single cage)

On Pier 19, only the NW, NE, and SE shafts had a window of casing removed. While the NW shaft showed smooth concrete, the NE and SE shafts showed poor concrete surface quality. A summary of this and the corresponding visual inspection notes is seen in Figure 10.89. Figures 10.90 and 10.91 display photographs taken of the NE and SE shafts, respectively.



= a window was cut and concrete was noted to be good by the diver

= a window was cut and concrete was noted to be bad by the diver (pictures included)

= no window was cut

Figure 10.89 Pier 19 casing window removal and concrete surface condition map.

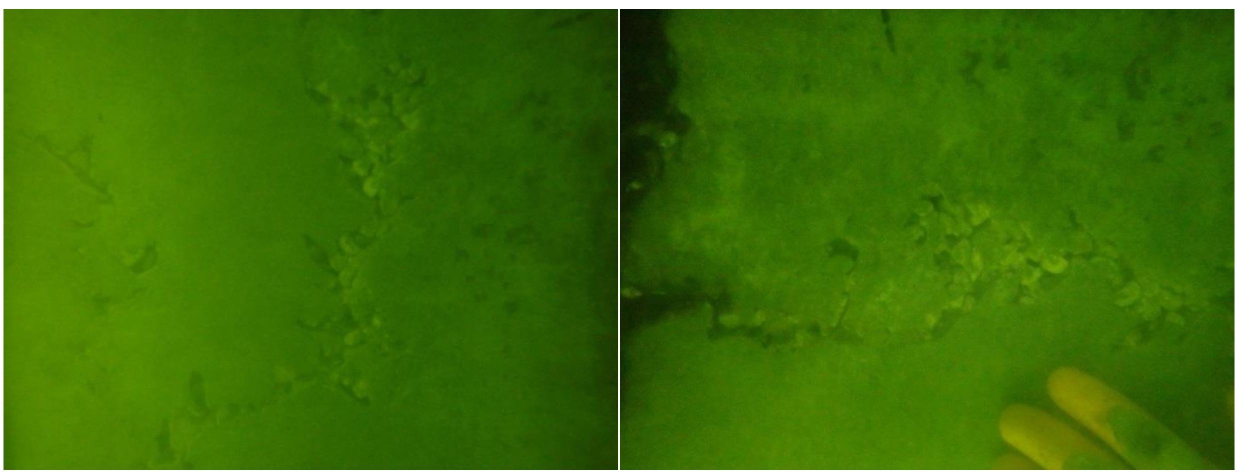


Figure 10.90 Pier 19 NE shaft concrete quality.

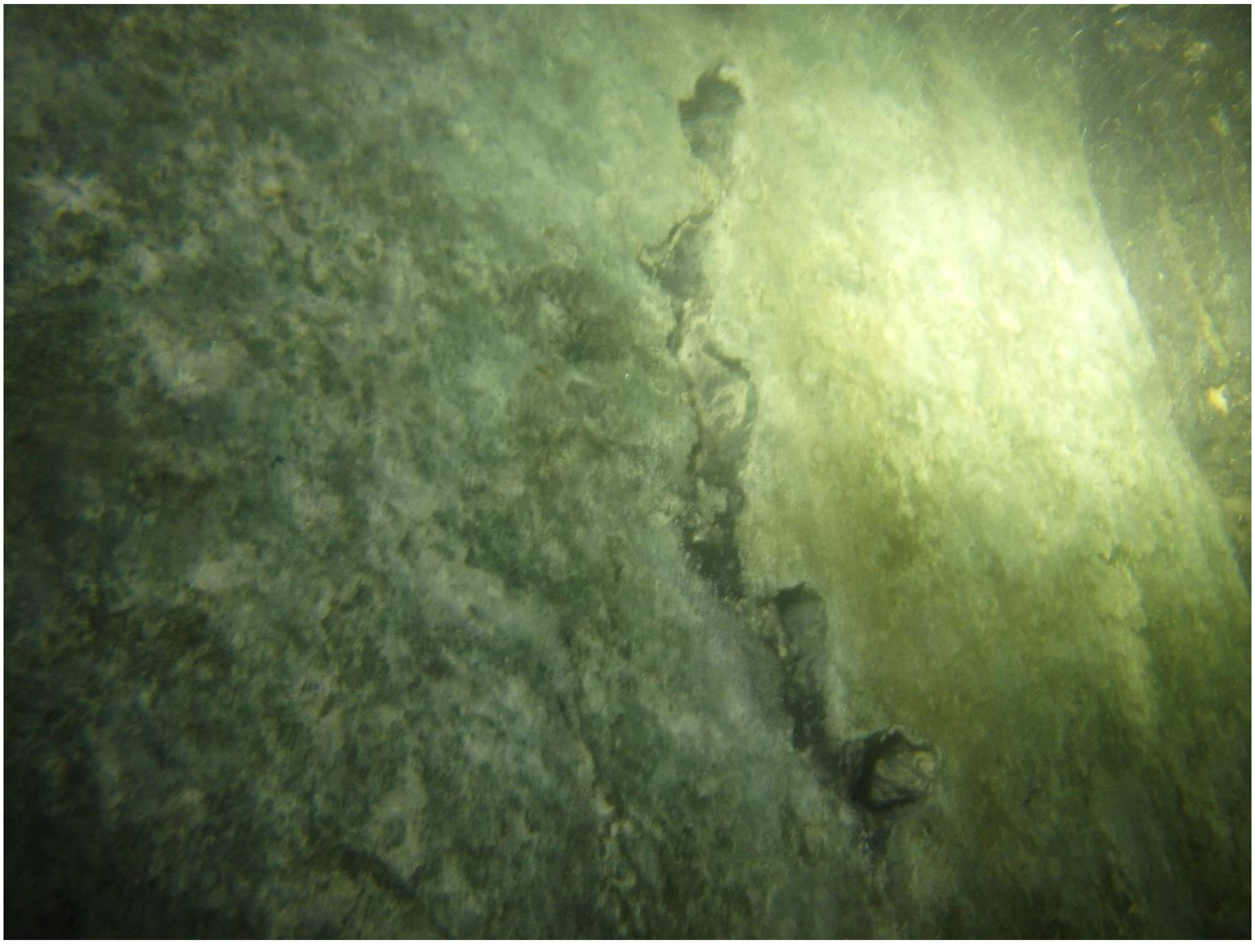


Figure 10.91 Pier 19 SE shaft concrete quality.

10.2.5 Caryville Bridge 520149 (SR-10/US-90 over Choctawhatchee River)

The Caryville bridge was not initially selected for inspection due to a comment under "drilled shaft notes" in the plans that stated there is permanent casing. The bridge was selected for casing removal later as it was cast using bentonite slurry. Figures 10.92 and 10.93 display the drilled shaft layout of the inspected piers as well as the drilled shaft reinforcement layout, respectively. All shafts inspected were 6 ft (1.83 m) in diameter.

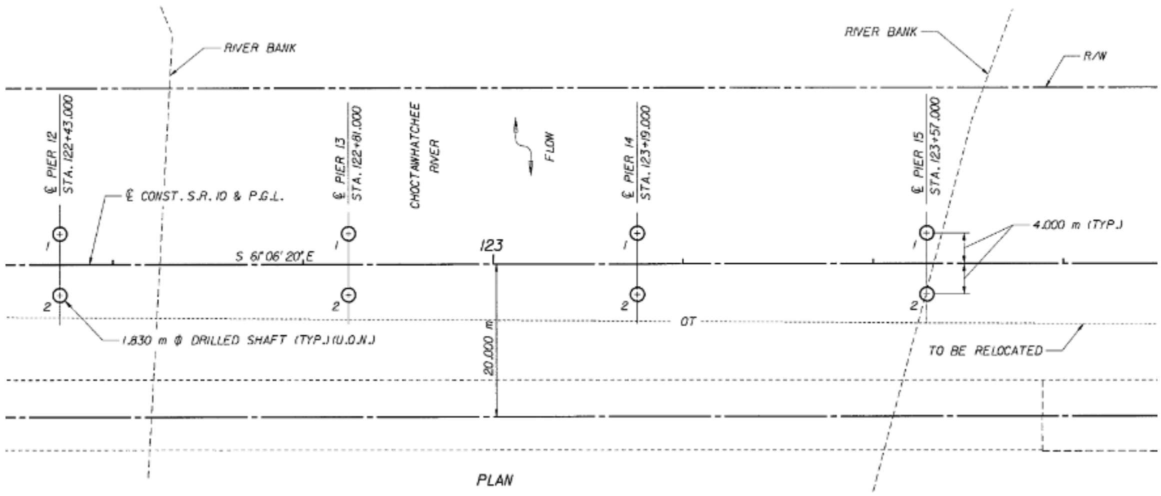


Figure 10.92 Caryville bridge drilled shaft layout for Piers 12 to 15.

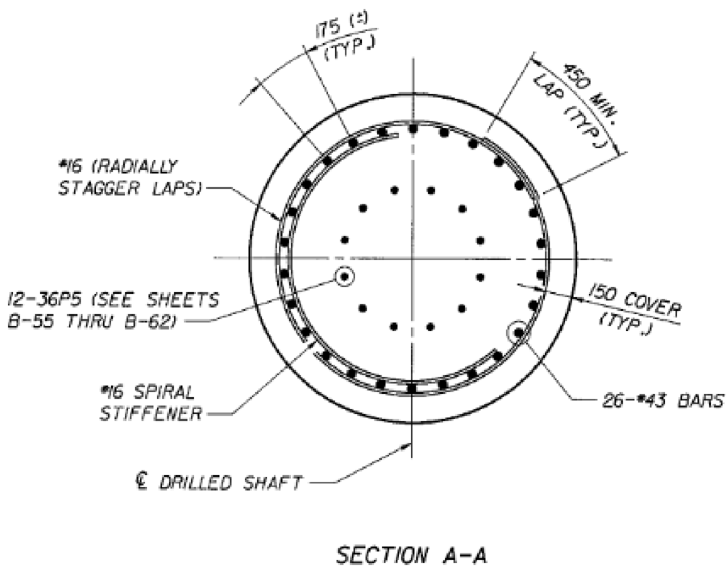


Figure 10.93 Drilled shaft reinforcement cage configuration, 6-ft (1.83m) diameter.

Upon initial inspection, mattressing could be seen prior to casing removal as some casing had previously been removed above water, Figures 10.94 and 10.95. Additionally, Pier 12 was on the river bank, so it was possible to do an above-ground inspection of shafts 12-1 and 12-2 of Pier 12. Both shafts 12-1 and 12-2 displayed creases and concrete deficiencies, Figures 10.96 and 10.97 respectively.

Figure 10.94 Overview of the Caryville bridge and drilled shafts; Pier 13 (left), 14 (center), 15 (right) .

Figure 10.95 Drilled shafts displaying mattressing creases.

Figure 10.96 Drilled shaft 12-1.

Figure 10.97 Drilled shaft 12-2.

Table 10.19 displays the locations of casing removal along with the initial concrete quality observed. Only one shaft (13-2) displayed clean concrete; however, on the opposing side, mattressing was seen, suggesting the possibility of poor construction. On the creased side of 13-2, the crease was noted to start above water and continue the entire length of the shaft. Figures 10.98 to 10.103 display images from the casing removal inspections of Piers 13, 14, and 15, for shafts 13-1, 13-2, 14-1, 14-2, 15-1, and 15-2, top and bottom, respectively.

Table 10.19. Summary of shafts inspected at Caryville Bridge.

Pier #	Shaft	Cut Location	Concrete Quality
13	North	Top - Downstream side	Poor
13	North	Bottom - Downstream side	Poor
13	South	Top - Downstream side	Clean
13	South	Top - East side	Poor
13	South	Bottom - East side	Poor
14	North	Top - Downstream side	Poor
14	North	Bottom - East side	Poor
14	South	Top - Downstream side	Poor
14	South	Bottom - East side	Poor
15	North	West side	Poor

Figure 10.98 Shaft 13-1 upper casing removed.

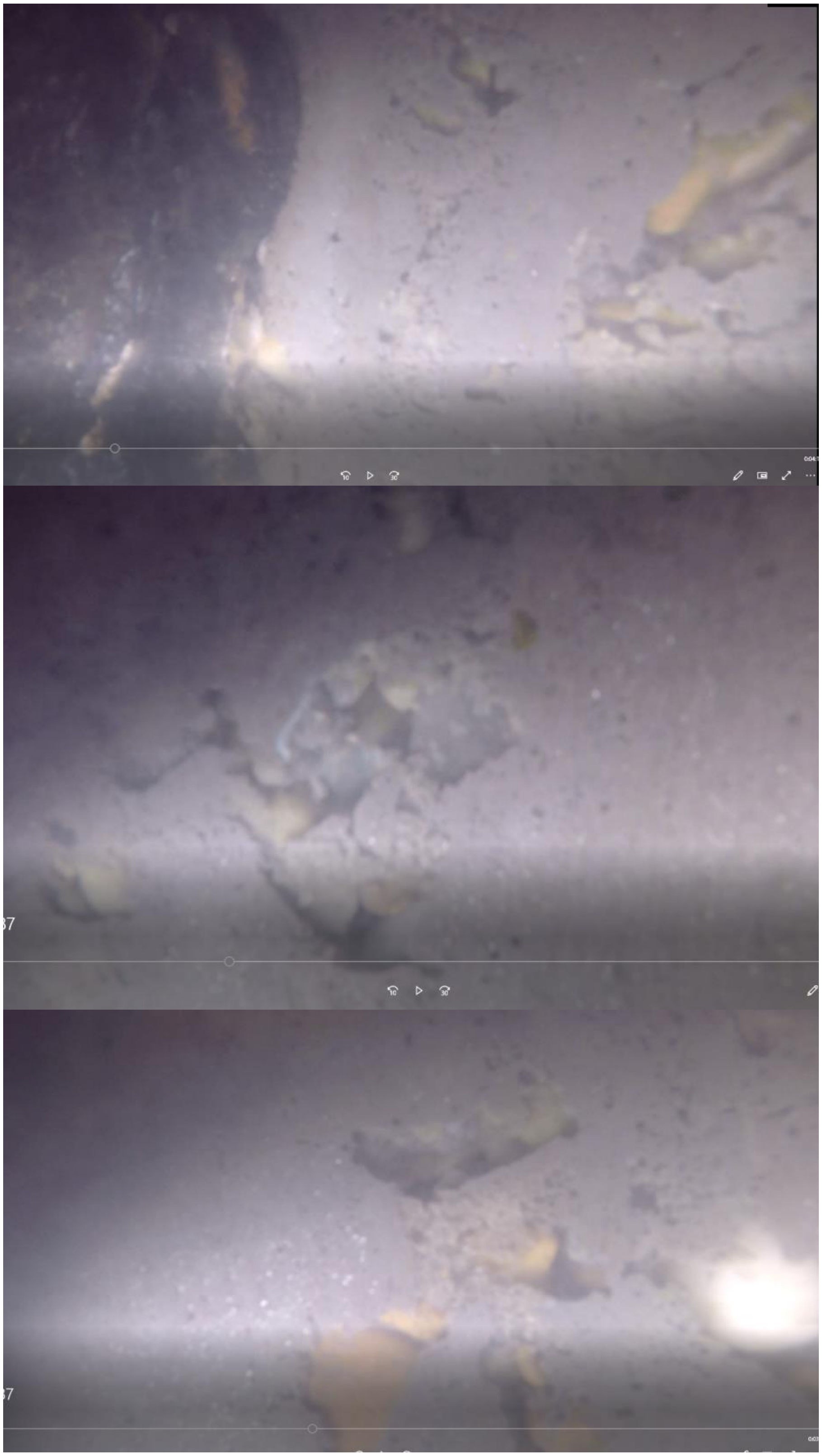


Figure 10.99 Shaft 13-2 lower casing removed.

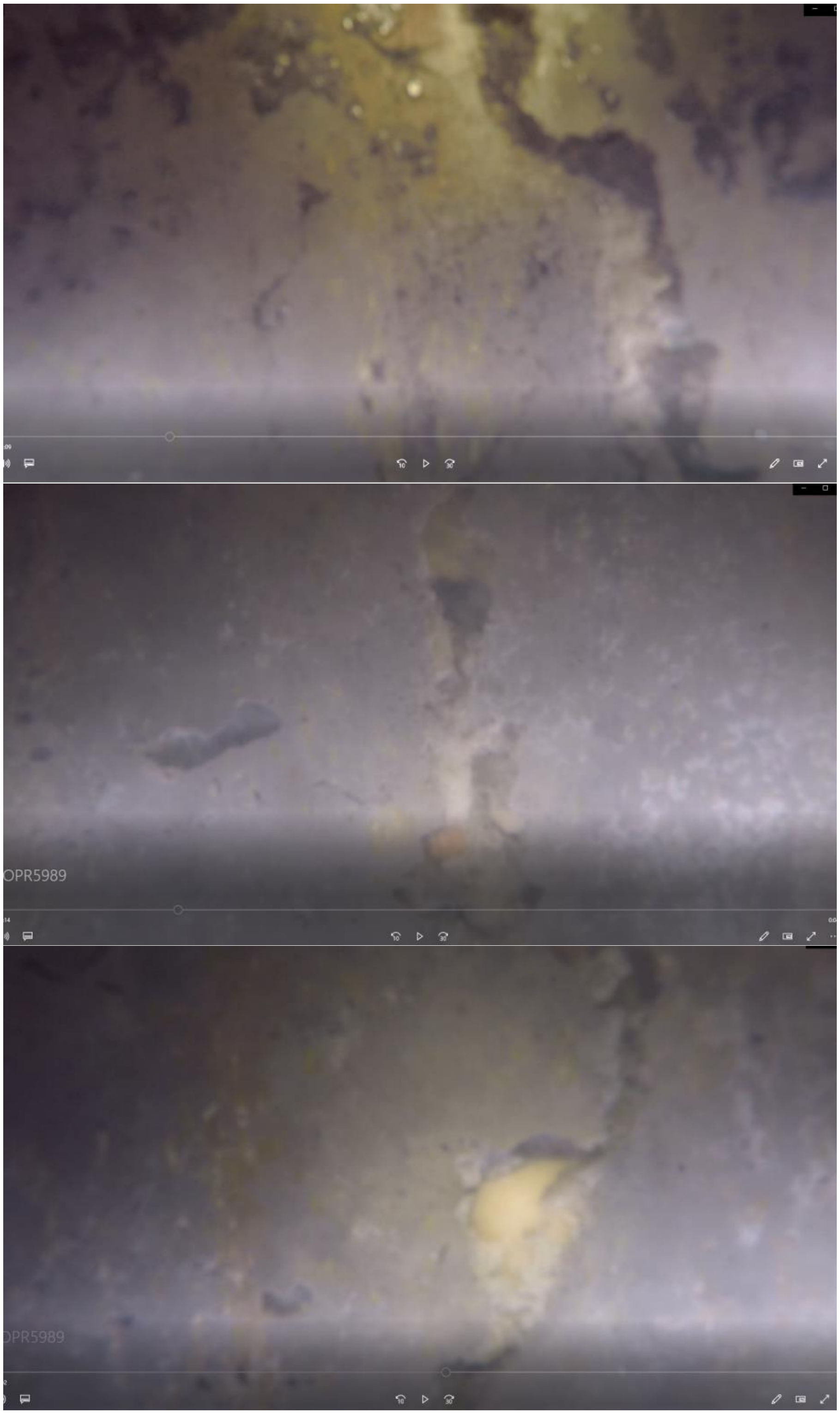


Figure 10.100 Shaft 14-1 upper casing removed.

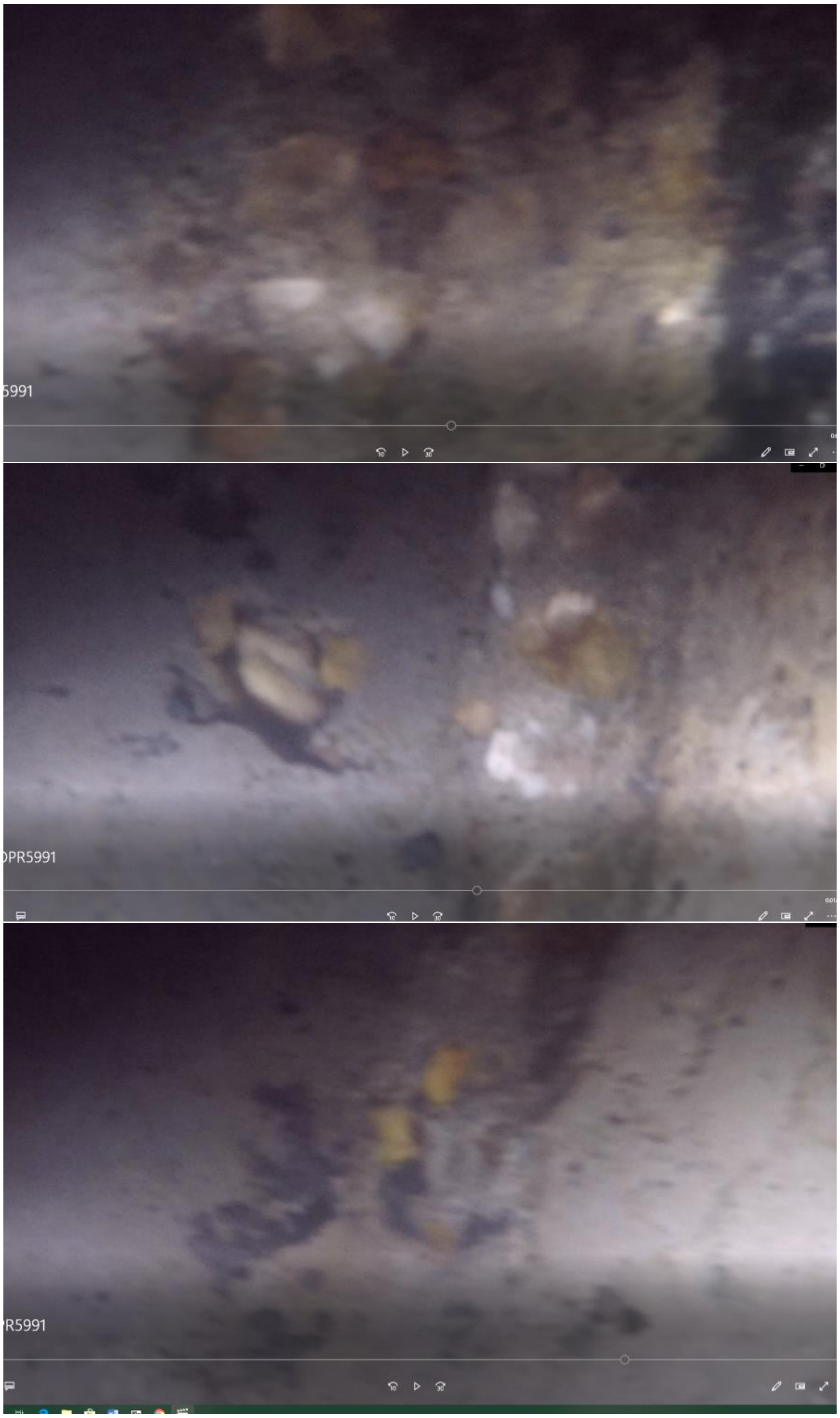


Figure 10.101 Shaft 14-2 lower casing removed.

Figure 10.102 Shaft 15-1 upper casing had already been removed.

Figure 10.103 Shaft 15-2 lower casing removed.

10.3 Chapter Summary

The goal of this Task was to identify the effects of slurry type on shaft surface roughness, void volume, and cover quality. The premise of the approach was that overwater bridges supported by shafts will have some portion of the shaft between the mudline and footing that is exposed and therefore could be an easy way to reveal and assess the shaft surface. This type of investigation was performed in lieu of excavations beside on-land shafts or footings to expose the shaft side surface. To be a viable method/approach, the exposed underwater portion of a shaft must be free of casing, which is always used to provide formwork in the water up to the footing elevation.

Review of plan sets from fourteen overwater bridges known to have been constructed on shafts gave rise to five candidate bridges that coincidentally also incorporated three different slurry types (natural, bentonite and attapulgite). On-site investigations of these five bridges were conducted using both a remote-operated vehicle that incorporated underwater video, and hands-on diving, which included surface cleaning and photography. In addition, investigations of four of the five bridges included return visits to cut out windows of casing and examine the surface of the fresh concrete.

The four bridges that had casing removed provided valuable information. The Gandy Bridge provided a baseline for shafts cast with natural slurry, which was shown to have no detrimental effects in laboratory samples (Chapter 3). Additional information was obtained that included the effect of varying shaft sizes and reinforcement cage configurations. The SR-20 Blountstown Bridge investigation provided information pertaining to the effects of bentonite slurry on tremie-cast shafts, which had, as shown in the photographs, heavily degraded shaft surfaces. Evaluation of the Bridge of Lions shafts indicated that the use of attapulgite slurry resulted in heavily degraded surfaces for about 45% of the shafts, which is similar to that noted in laboratory specimens (Chapter 3). No usable information was obtained from the US 441 Santa Fe River as the mudline elevation rose and the shafts could not be seen, nor could the casings be cut.

Gandy Bridge: Shafts constructed using a single-cage reinforcement with natural slurry exhibited pristine concrete surface conditions with virtually no surface voids. In isolated cases, irregularities were found that appeared to be in no way associated with quilting. Shafts examined that were constructed with concentric double-cage reinforcement yielded more frequent and pronounced voids, but again these voids seem to be more random and were not associated with the quilting pattern.

Blountstown Bridge: Five out of five shafts in the river portion of the bridge were examined and all showed evidence of quilting. These shafts were all cast with bentonite. The quilting pattern was so pronounced that the grinder was barely, if at all, able to make a smooth surface in any area.

Bridge of Lions: Five of the eleven casing windows removed revealed damaged concrete surfaces. All shafts inspected were cast with attapulgite slurry. Some of the concrete surfaces show a quilting pattern that is less pronounced than what is seen with bentonite.

Caryville Bridge: Six out of six in-river shafts showed signs of mattressing distress, typical for use of bentonite slurry found in this study. Other shafts investigated, in the flood plain but out of the water, all showed the same distress. All shafts showed concrete deficiencies.

A surprising discovery from diving both fresh and salt water bridges was that pitting in the steel casing was much worse in fresh water (Figure 10.104) when compared to salt water. The bridges compared, Gandy and Blountstown, were almost the same age, built in 1996 and 1998, respectively. Figure 10.104 shows pronounced pitting corrosion at Blountstown and very little visible pitting in the Gandy casings. When preparing the surface of the Gandy shafts, a thin, black, presumably organic layer was found that was difficult to remove (Figure 10.104 and 10.105). This layer was hypothesized to be a layer of anaerobic decay from the barnacles and sea-growth, which would be an oxygen barrier and may slow or prevent corrosion. Discussions with the USF Department of Integrative Biology confirmed similar observations of a black layer under oyster and barnacle growth. Some consideration will be given to promoting encrusted surfaces that would be beneficial to both concrete and steel surfaces under these conditions. While the Bridge of Lions casing was heavily encrusted, no data or observations were collected which could have provided insight into the effect of east coast marine conditions on corrosion.

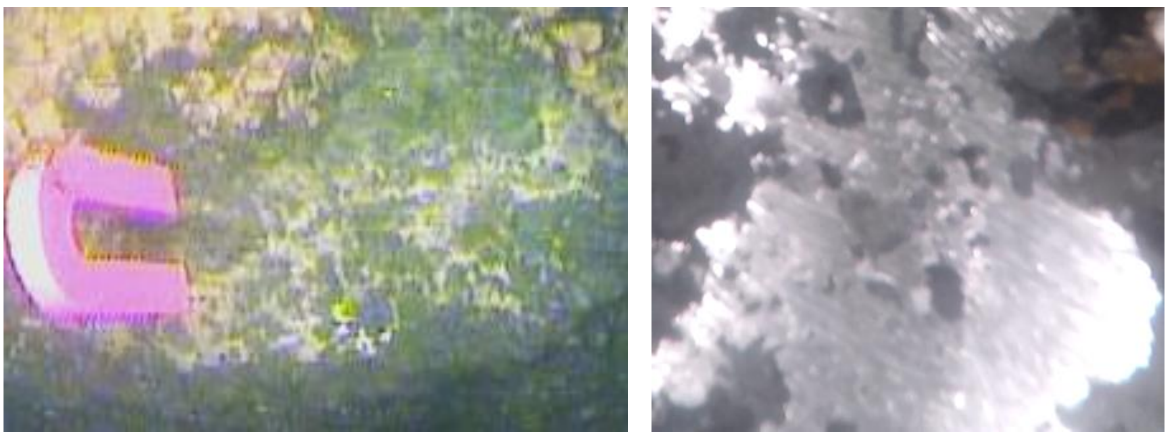


Figure 10.104. Pitting corrosion in freshwater (left); very little pitting in marine environment (right).

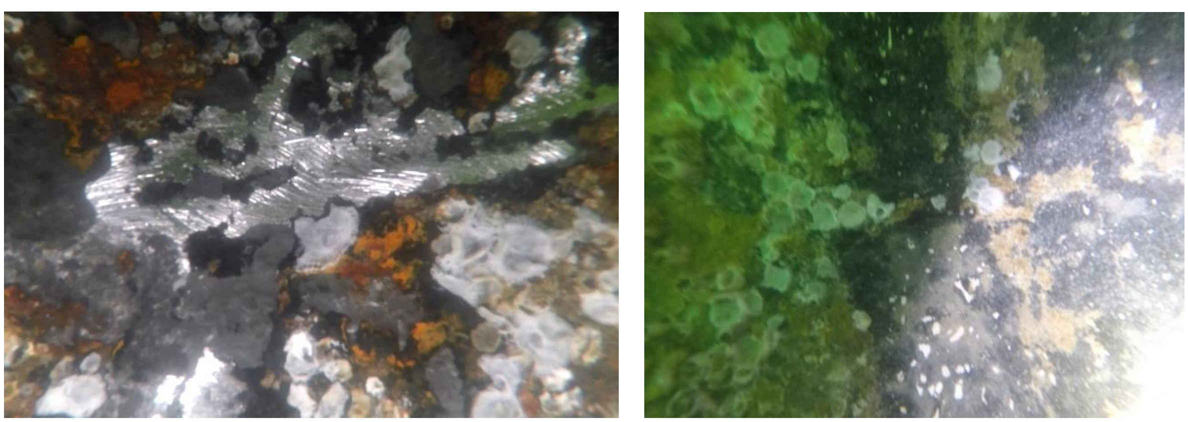


Figure 10.105 Black organic layer beneath barnacle growth: on steel casing (left), on concrete (right).

In summary, there are several observations that can be made:

- While it would have been ideal to have more inspections for comparison, quilting was found as expected in bentonite-constructed shafts.
- Shafts constructed with natural slurry (water) were largely in pristine condition.
- Single-cage reinforcement led to fewer voids and defects than double-cage reinforcement.
- Pitting in steel casing was worse in fresh water without encrusted organism growth.
- While casing does not appear to have an adverse effect on shaft performance, it does not eliminate the presence of quilting within the casing. These areas are just as vulnerable to corrosion as those without the casing.
- Drilled shaft concrete surface quality was evaluated by examination of overwater bridges, but very few examples were found without casing. However, shafts without casing are in abundance for land piers, where cover quality is highly likely to be compromised.

Underwater evaluations of shaft surface quality/roughness was performed using subjective visual methods.

Despite overwhelming support and assistance from FDOT district maintenance personnel, the simple approach to review shafts underwater (without excavation) revealed the following shortcomings:

(1) The state bridge inventory database does not note the foundation element type (shaft or pile), which made personal recollections the only first round screening tool,

(2) Diver inspector reports are scarce, and if included, are non-descriptive, inconsistent, and devoid of engineering terminology,

(3) Biennial inspection reports in today's era of databases and associated querying tools should be far more comprehensive and searchable. It is assumed that biennial inspections are privatized, performed by consultants that could change periodically. As a result, the continuity required to make decisions in-the-field pertaining to points of concern or interest is limited without a chronologically-organized inspection list for each element, not just the overall reports.

(4) Construction logs/records were understandably more poorly archived 20 years ago when the Gandy and Blountstown bridges were built, but more recently constructed bridges like the Bridge of Lions (completed 2010) should have very accurate and highly accessible information/records.

(5) All inspection reports should be in the same format statewide, and it would be implicit that the dive reports would be appended to each overall inspection report, especially when there is something found of concern. For instance, the Blountstown bridge during the October, 2011 inspection would have had a water level elevation around 28 ft (based on that date and the USACoE historical water level records); this means that the underwater issues found between 30 and 31 ft in this report would have been exposed and easily visible, including the dislodged steel collar on shaft 58-2. Perhaps this information exists somewhere, but it was not made available to the researchers.

APALACHICOLA
BLOUNTSTOWN
Gage Zero 27.0 Flood Stage 15
NOTE: These readings are taken at approximately 12AM
2017 WATER YEAR

	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP
1	.59	.44	.80	2.19		9.56	5.56	4.14	5.60	8.19		
2	.66	.45	.87	2.21		9.22	5.86	3.93	6.50	7.60		
3	.47	.47	.83	14.16		8.44	5.61	3.66	5.92	7.50		
4	.71	.48	.88	10.96		7.99	6.13	3.42	5.61	7.14		
5	.65	.51	.85	18.48		7.54	5.94	3.55	5.67	6.98		
6	1.04	.50	1.93	20.13		7.40	5.71	3.30	5.73			
7	.84	.48	8.42	20.37		7.39	11.23	3.17	5.88			
8	.67	.51	7.82	19.44	8.30	7.75	14.12	2.98	6.63			
9	.58	.47	8.19	18.08	10.93	8.29	15.23	2.83	8.68			
10	.50		7.41	17.24	12.30	7.94	14.81	2.75	9.26			
11	.46	.93	7.05	16.40	12.57	7.96	13.56	2.53	9.28			
12	.65	1.46	6.58	15.70	12.66		12.52	2.37	9.22			
13	.63	1.02	6.32	15.40	12.67	7.32	10.96	2.21	8.26			
14	.73	.53	5.96	14.94	11.20		9.71	1.81	7.94			
15	.66	.54	5.73	12.83	10.49	6.97	8.86	1.65	7.71			
16	.58	.49	5.37	11.14	9.60		7.76	1.33	7.56			
17	.49	.57	4.89	9.79	8.83		6.79	1.26	7.23			
18	.45	.56	4.70	8.96	8.42		6.70		7.02			
19	.42	.49	4.41	8.12	8.10		6.45		6.56			
20	.43	1.22	4.18	7.59			6.29	1.38	6.40			
21	.46	.85		7.63			6.29	1.35	7.30			
22	.52	.74	3.59	9.09	7.61	5.94	6.30	1.43	10.64			
23	.50	.73	3.26	-76.67	10.19	5.76	6.12	1.43	10.39			
24	.47	.72	2.97	18.95	13.65	5.89	6.08	7.47	9.46			
25	.46	.59	2.83	20.13	13.38	5.70	5.73	9.76	9.11			
26	.49	.52	2.82	19.73	12.86	5.63	5.35	8.87	9.71			
27	.52	.70	2.58	19.47	11.70	5.61	5.02	8.95	9.50			
28	.49	.84	2.57	18.72	10.78	5.56	4.74	8.15	9.34			
29	.51	.86	2.41	17.47		5.64	4.53	7.24	9.31			
30	.48	.84	2.25	16.34		5.62	4.38	7.20				
31	.44		2.22			5.51		6.26				
AVG	.57						7.81					
MAX	1.04						15.23					
MIN	.42						4.38					

Figure 10.106 Apalachicola River level at time of inspection (circled); historical values from October 2017 are boxed (http://water.sam.usace.army.mil/acfframe.htm).

Chapter Eleven: Newton Labs Underwater 3D Scanner

Chapter 9 outlined the use of the Artec Eva 3D scanner used to create detailed 3D models of the concrete surfaces of lab-cast specimens. Recall, the 3D data point clouds were reduced from several million data points to one or two million data points and then imported into AutoCAD Civil3D. A one square foot representative section was used in which the void volume per square foot was determined. This void volume was then used to draw conclusions about the concrete cover quality of these test shafts. The next step was to obtain cover quality information of drilled shafts in the field. For this, 3D models of the shaft surface were needed. However, since the sections of interest are underwater, a new method of scanning needed to be used. The Newton Labs M3200UW underwater laser scanner was used to create these 3D models.

Four bridges were selected to use void volume determination as a means to quantify concrete cover quality: (1) Gandy Bridge (100585), (2) Caryville Bridge, SR10/US90 over Choctawhatchee River (520149), (3) Blountstown Bridge, SR20 over the Apalachicola River (470052), and (4) the Bridge of Lions (780074). These bridges were selected based on their respective construction method and the information gained from previous dive inspections. Since the sections of interest are behind steel casings, each bridge needed to have a 3-foot by 3-foot section of steel casing burned off to expose the concrete surface of the drilled shafts. Digital scanning of these regions was the focus of the efforts discussed in this Chapter.

11.1 Scanner Description

The Newton Labs M3200UW is a completely submersible underwater scanner rated for a depth of 3200 meters. The underwater scanner has a laser and a high-resolution optical camera, seen in Figure 11.1, in order to create 3D surface models of the concrete surface.

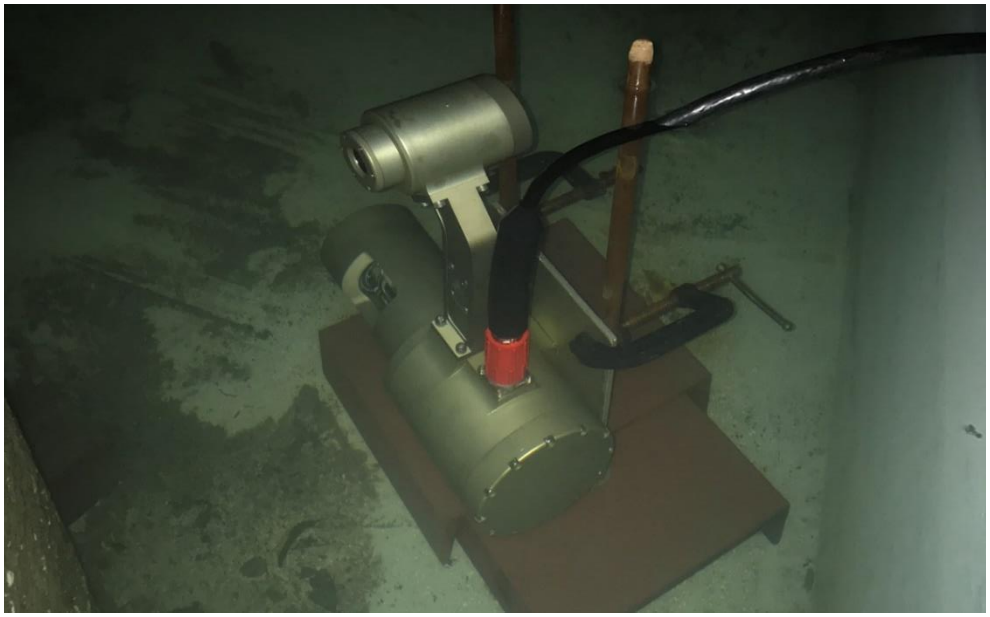


Figure 11.1 Newton M3200UW Underwater Scanner

The underwater scanner uses optical triangulation to construct 3D models. The laser creates a beam across the shaft surface, this beam is then reflected to a high-resolution digital camera. As the beam sweeps across the concrete shaft surface, the camera detects the location of the beam. Algorithms are then used to correlate this deformation to depth changes on the concrete surface. The scanner has a sub-millimeter accuracy, which is needed to accurately conduct void volume determination. Figure 11.2 shows the scanner line while in use.

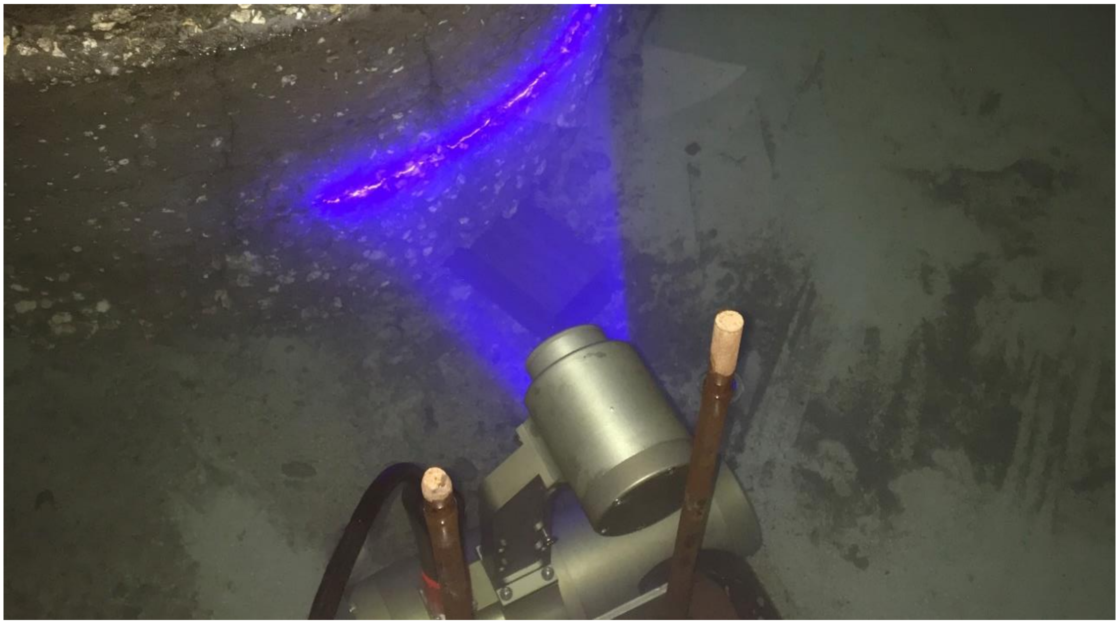


Figure 11.2 Newton M3200UW Laser Line

The scanner system has two main components, the underwater scanner and the surface computer. The underwater scanner contains the high-resolution optical camera and the laser. This device will be securely mounted to the underwater cutout of the steel casing, between 2 and 4 feet away from the concrete surface. The underwater scanner will be used at a depth of 3-5 meters.

The surface computer, seen in Figure 11.3, is a standard desktop computer in a protective case with a Linux operating system. The computer has specialized software to receive and interpret the data being sent from the underwater scanner. This software is provided by Newton Labs. The two components, the underwater scanner and surface computer, are connected by a 300-foot cable.

Figure 11.3 Newton Underwater Scanner Surface Computer

11.2 Scanner Usage

Using the Newton Labs M3200UW underwater scanner requires careful preparation to capture high-quality 3D scan data. When bringing the underwater scanner out into the field, two boats are securely tied up to the shaft being scanned. One boat contains all of the scanning equipment while the other boat carries all of the diving equipment.

Two divers clean the exposed concrete surface where the section of steel casing has been removed. The scanner, attached to the aluminum frame, is then lower into the water using a boat-mounted crane, seen in Figure 11.4. The divers secure the aluminum frame to the shaft by running two ratchet straps around the shaft and tying them to the sides of the aluminum frame.

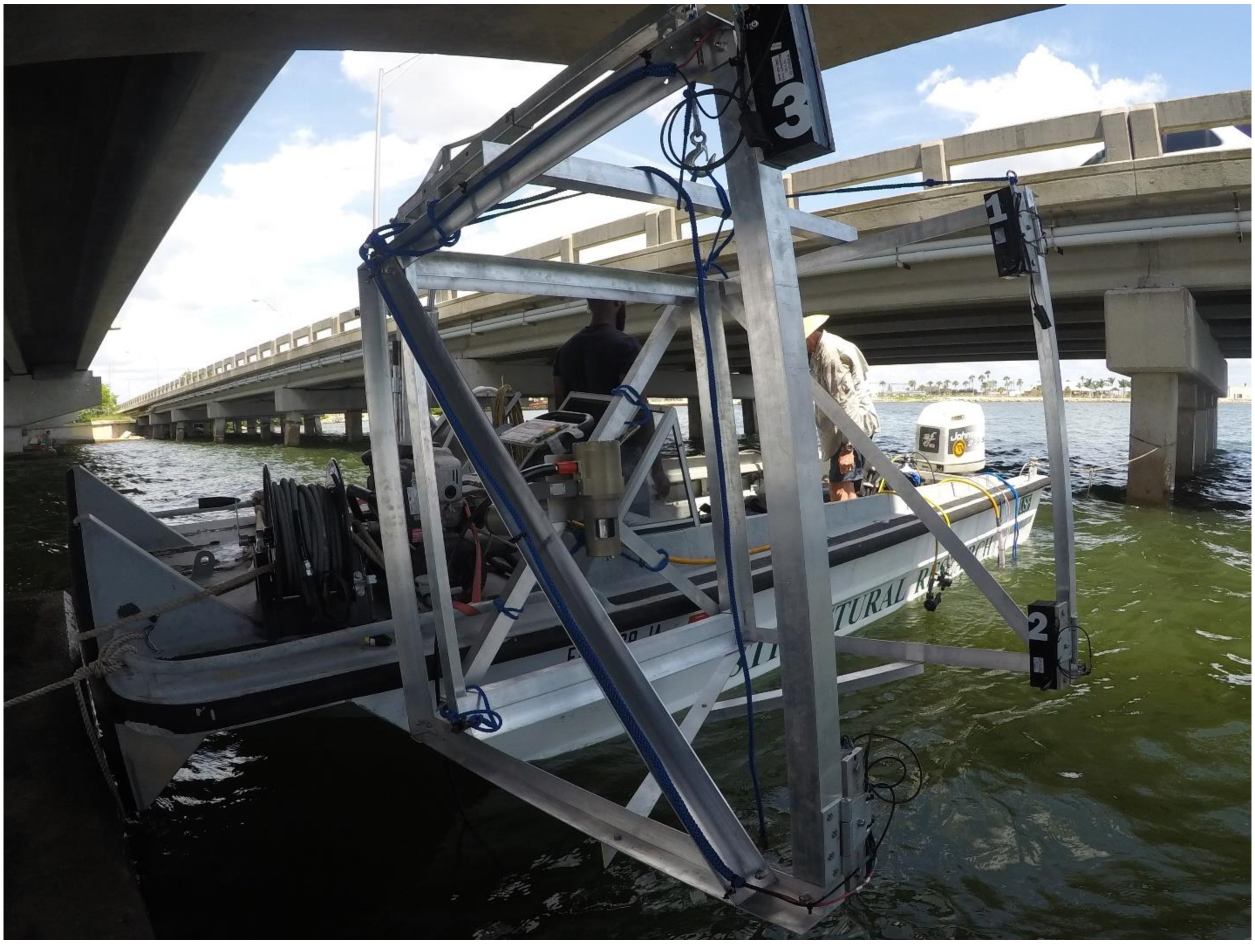


Figure 11.4 Underwater Scanner Mounted to Aluminum Frame

11.2.1 Concrete Surface Preparation

The first component of successful underwater scanning is proper concrete surface preparation. Since most of the concrete shafts being scanned are in saltwater environments, barnacles are prone to cover the target concrete surface. Before every scan, the surface needs to be thoroughly cleaned to remove all barnacles and expose the concrete. A scraper was used to remove the bulk of growth followed by a 3000 psi power washer, seen in Figure 11.5. This removed any loose particles, sediment, or other build up that might have filled the voids present in the concrete surface. Since void volume is being used to quantify concrete cover quality, all non-concrete material needs to be removed.

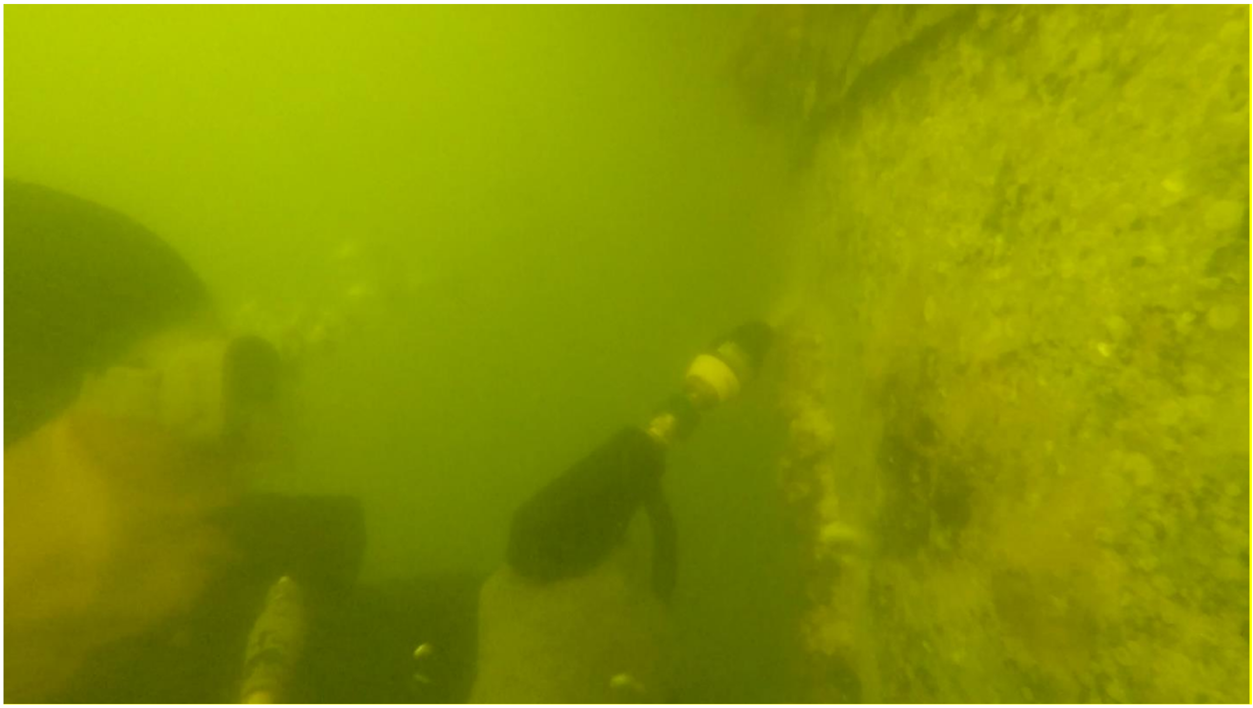


Figure 11.5 Pressure Washing Shaft Surface

11.2.2 Mounting the Scanner

It is extremely important that the underwater scanner remains completely still while scanning. Any vibration or movement will change the distance between the scanner and concrete shaft surface. This will correlate to a change in surface depth that does not exist. In order to mount the scanner firmly to the concrete shaft, an aluminum frame was constructed. The frame is 4 feet on every side, with a smaller inlaid frame to mount the underwater scanner, seen in Figure 11.6. This inlaid frame can slide forward and backward to adjust the distance between the shaft surface and the scanner. With murky water, the ability to change the distance between the scanner and surface is important. The laser light diffracts less with a shorter distance to travel.

The underwater laser scanner has four threaded inserts in a rectangular bolt pattern on the back. A rectangular aluminum plate is bolted into these threaded inserts. This plate is then bolted directly onto the inlaid frame. The aluminum frame is strapped to the drilled shaft using two large ratchet straps, also seen in Figure 11.6. One strap is placed at the top and the other at the bottom of the aluminum frame.

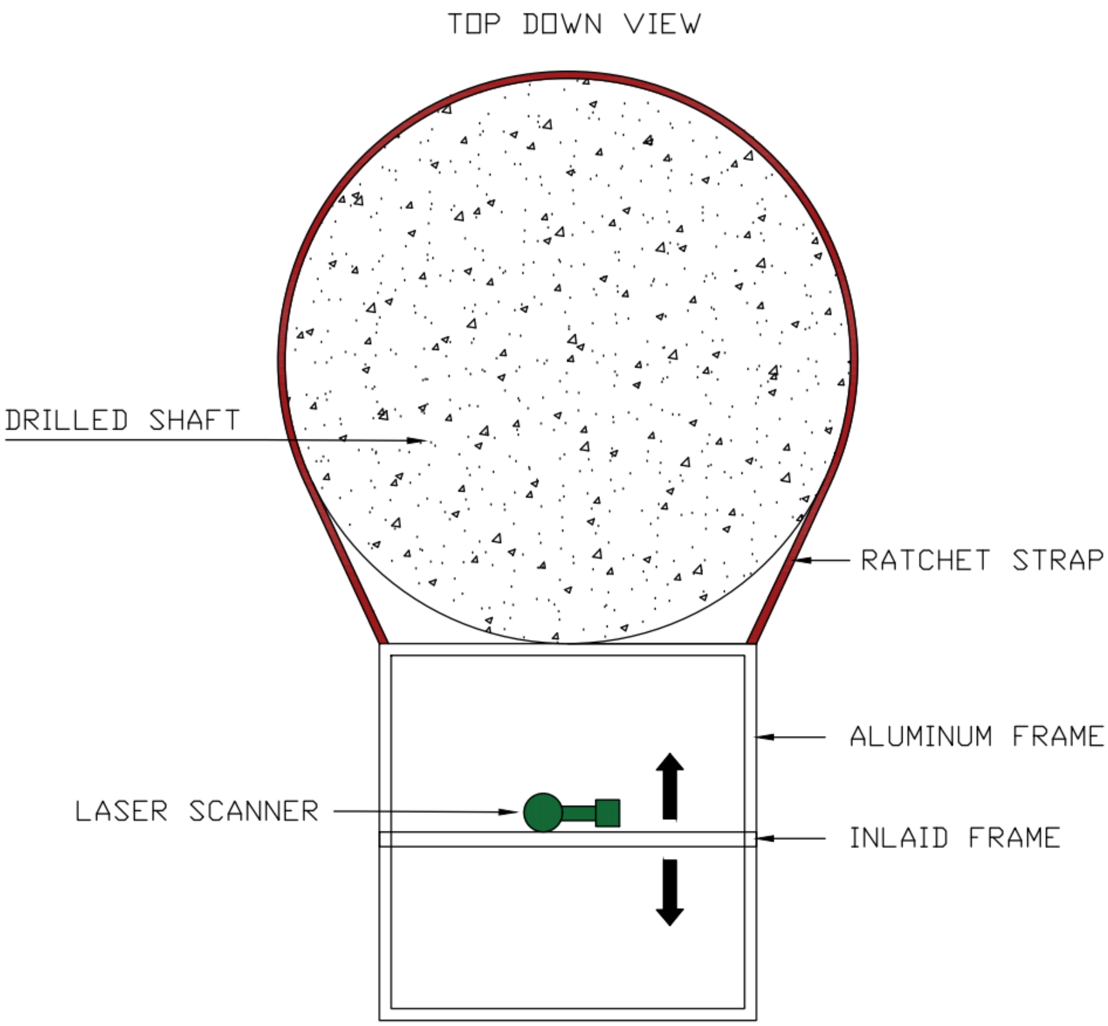


Figure 11.6 Top Down View of Aluminum Frame Mounted to Shaft Surface

11.2.3 **Water Filtration**

In order to achieve high quality 3D scans from the underwater scanner, the water between the scanner and the concrete surface needs to be as clean as possible. Any particulates in the water can diffract the laser light and create noise in the 3D model. This noise will appear as data points stretched away from the concrete surface or as bumps on the concrete surface. Thick plastic sheeting was wrapped around the scanner to create a seal against the concrete shaft surface. Once the aluminum frame was mounted to the shaft surface, this sealed volume of water was then filtered with a pond filter. The filter removed most particulates from the water within ten minutes.

11.2.4 **Ambient Light Reduction**

The underwater scanner uses a visible laser light to create the 3D model of the concrete surface. Any ambient sun light will wash out the light from the laser, reducing the quality of the data or making data collection impossible. The reduction of ambient sun light is imperative. The thick plastic sheeting that was used to create a seal against the shaft will also be painted black to block all ambient light from entering the aluminum frame.

11.3 Scanner Software

Newton Labs provides a software to work specifically with the Newton Labs M3200UW underwater scanner. The scanner software is run on the Linux-based surface computer. The software receives the data from the underwater scanner and creates a .xyz and .ply point cloud file. These point cloud files are then transformed into surfaces during post processing. The scanner software is used to adjust all scan settings that affect the quality and size of the point cloud files.

The scanner software has two main windows, the user interface and the Linux based console. The user interface will allow the user to adjust scan settings, view the live camera image, and export completed scan data. The console window shows the current commands being executed by the scan software. This console window will display specific error messages that may arise during scanning.

11.3.1 Connecting the Underwater Scanner

Connecting the Newton Labs M3200UW underwater scanner is the first step when using the scanning software. The 300-ft cable is used to connect the surface computer to the underwater scanner. Once properly connected, the head power switch on the surface computer turned on.

With power to the underwater scanner, the scan software can now be connected. Navigate to the *Settings* tab and using the drop-down menu, select *Newton Underwater M3200UW*. Once selected, hit the *Connect* button. The scanner and computer will go through a set up procedure. The underwater scanner will move the laser from the highest to the lowest angle in its scan window. It is important to allow the scanner to go through the setup procedure. Once the setup procedure is complete, the console window will provide a *Success* message.

11.3.2 User Interface Layout

It is important to understand the layout of the user interface. As seen in Figure 11.7, the user interface is divided into three main sections. The left-hand side is the *Live Camera View*. This is the image that the optical camera is currently seeing. The middle section contains *Scan Settings* used to adjust the quality of the scan. The right-hand side is the *Current Image Settings*, these will be used to create the ideal scan settings.

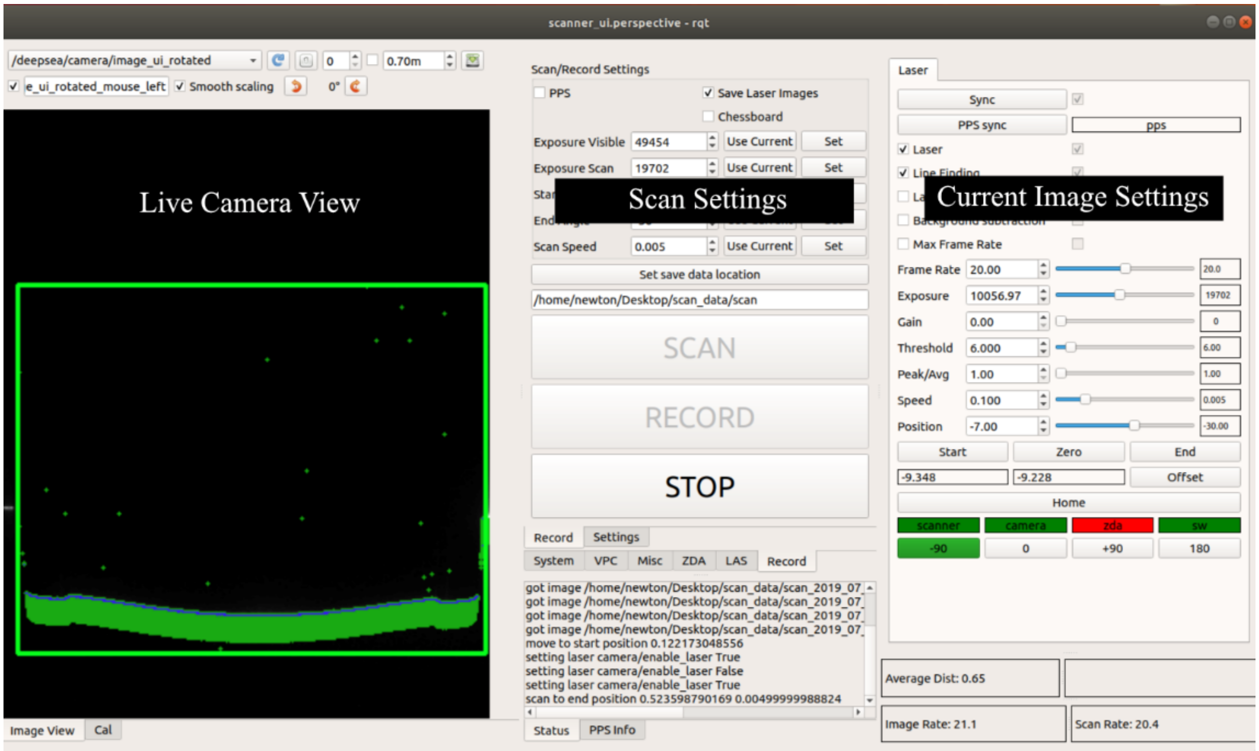


Figure 11.7 Scan Software User Interface

When operating the underwater laser scanner, the operator will start their adjustment in the *Current Image Settings* window. These settings will affect the quality of the image seen in the *Live Camera View*. The operator will adjust these settings until the scan quality is good. Ideal scan settings will be discussed in the following sections. Once the ideal settings have been selected, they need to be transferred to the *Scan Settings* window. They can either be entered manually or the operator can hit the *Use Current* button to transfer the corresponding settings. It is important to note that these settings will not transfer automatically.

11.3.3 **Adjusting Scan Settings**

There are many scan settings in the software that can be overwhelming. However, there are only a few that affect scan quality The main settings are exposure, gain, threshold, peak/average, and scan speed. These setting are under the *Scan Settings* section, seen in Figure 11.8.

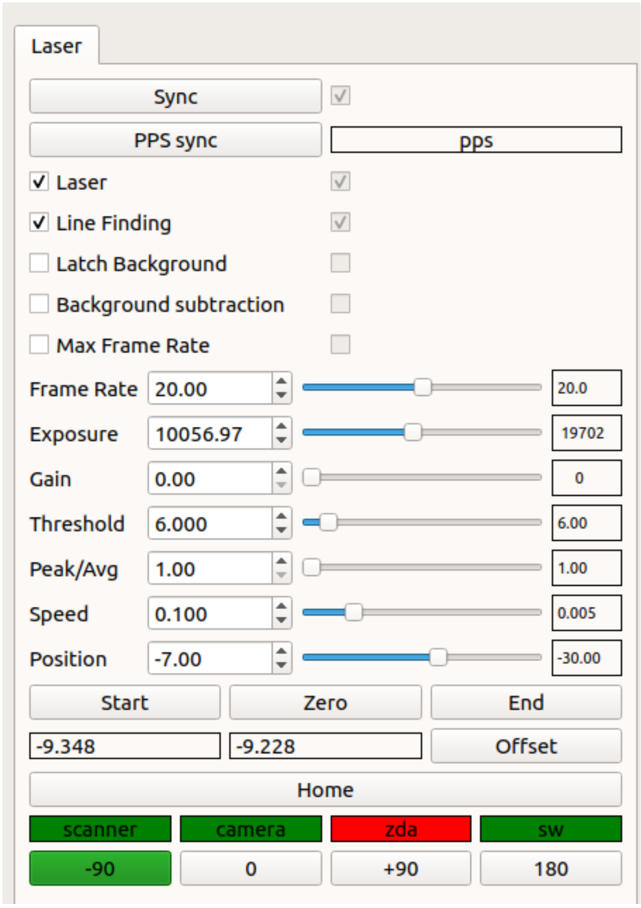


Figure 11.8 Main Scan Settings

The exposure is the amount of light the optical camera will take in when scanning. The threshold value describes how discriminatory the scanner is to collected data. Each data value seen by the optical camera will have a corresponding quality. A higher threshold value raises the minimum quality required for the scanner to establish a data point. Lastly, the gain parameter acts as a multiplier to the exposure and threshold values, leaving the gain at zero is standard.

In order to see how these settings affect scan quality, check the *Laser* and *Line Finding* boxes under the *Current Image Settings* section. In the *Live Camera View*, the laser will appear as a purple line. Adjusting these settings will make the line more or less clear. The line represents the amount of data that is collected at that particular laser position. The idea is to adjust these settings until the line is as straight and clear as possible. Once a clear line is found, the *Position* setting is used to sweep the laser line up and down on the target surface. Moving the laser line to various positions within the scanning range, adjust the scan settings accordingly to create a clear line over the entire scan area. These ideal scan settings are then transferred to the *Scan Settings* section of the user interface by using the *Use Current* button or manually entering them.

11.3.4 **Running a Scan**

The *Start Angle* and *End Angle* settings under the *Scan Settings* are used to specify the scan area. Below is the *Scan Speed* setting. A lower scan speed will yield more data points. The scan speed is set to 0.005, as seen in Figure 11.9.

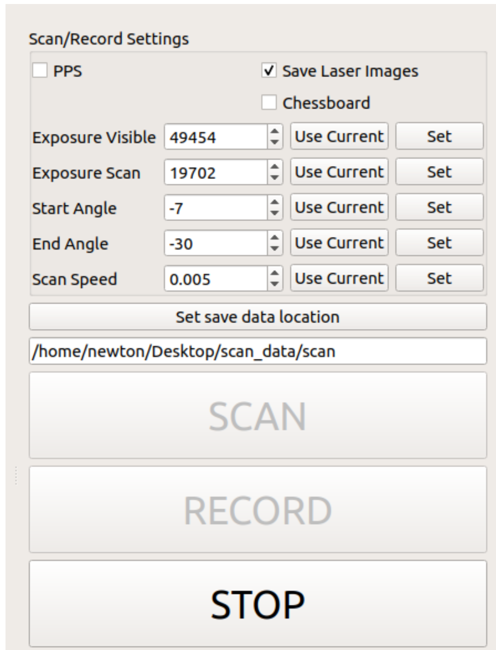


Figure 11.9 Scan Range and Speed

Once the scan range and speed are specified, the save directory is set, and the scan is started using the *Record* button, seen in Figure 11.9.

11.4 **Post Processing**

The .xyz and .ply point cloud files can be found in the directory specified in the scan settings. These point cloud files are then imported into MeshLab. MeshLab is a free software used to view and modify 3D triangular meshes. While MeshLab will not provide quantitative data of the scanned concrete surface, it is a good tool to quickly process data in the field. MeshLab provides a first impression of the data in the field to ensure the scan settings are producing a high-quality scan. The point cloud file is imported by going to *File – Import Mesh*, seen in Figure 11.10.

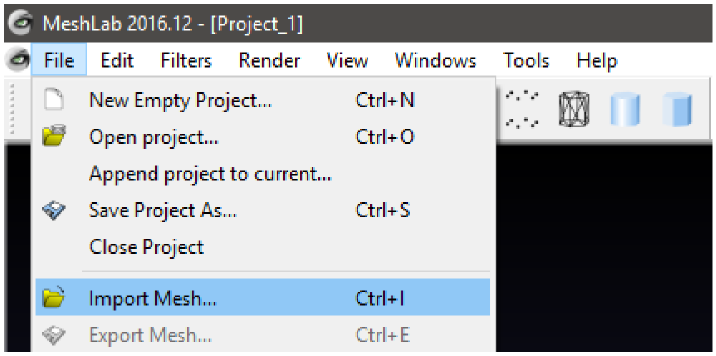


Figure 11.10 Importing a Point Cloud File into MeshLab

The imported point cloud resembles the shaft surface being scanned, as seen in Figure 11.11. The crease that strikes across the surface is a welding crease from the previously removed steel casing.

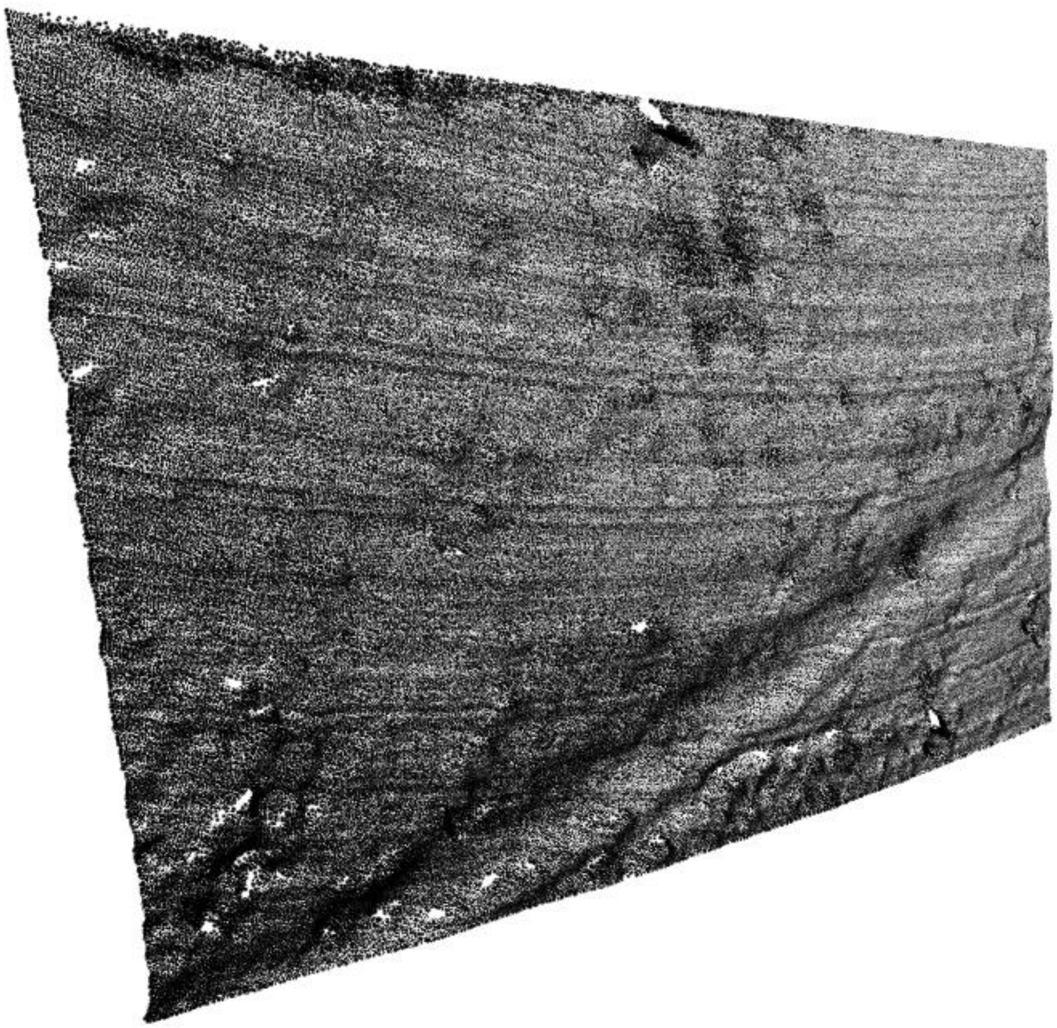


Figure 11.11 Raw Imported Point Cloud Data

The normals for the point cloud are then computed. The normals are vectors perpendicular to the predicted surface. MeshLab uses a specified number of surrounding points for each point to create that point's normal vector. These normal vectors will then be used to create a surface mesh. To compute the normals, go to *Filters – Point Set – Compute Normals for Point Sets*, as seen in Figure 11.12.

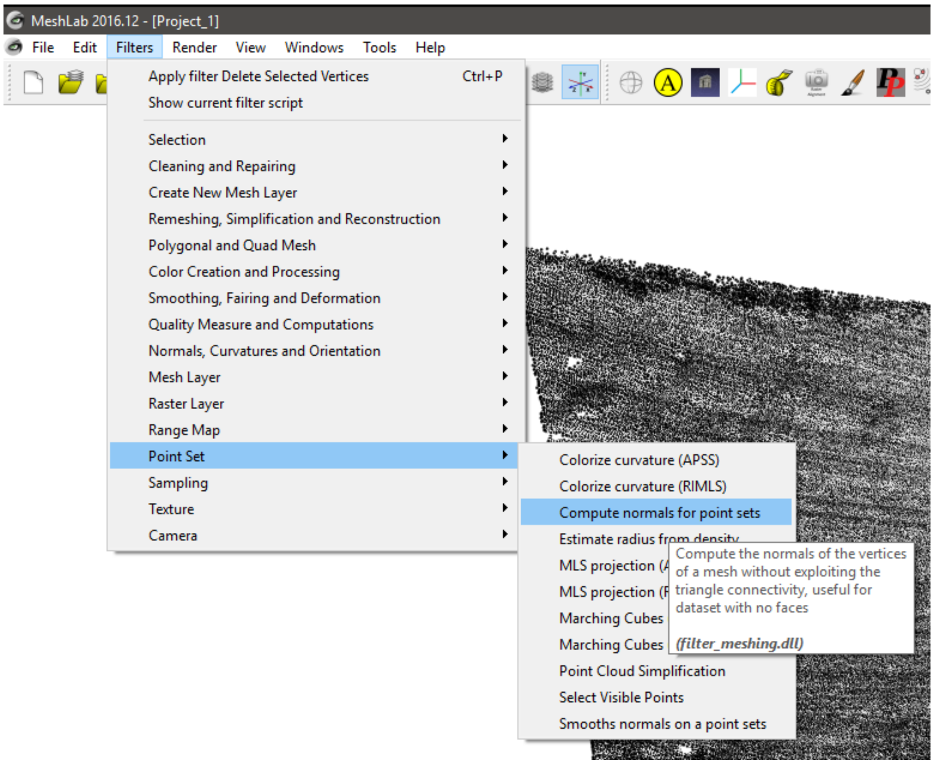


Figure 11.12 Computing Normals for the Point Set

In the *Compute Normals for Point Sets* window, the number of neighbors is selected as sixteen. This is the number of neighboring points used to create each perpendicular vector. The *Flip Normals w.r.t Viewpoint* option is also selected, seen in Figure 11.13. This will make the front of the surface face the user when the surface mesh is created.

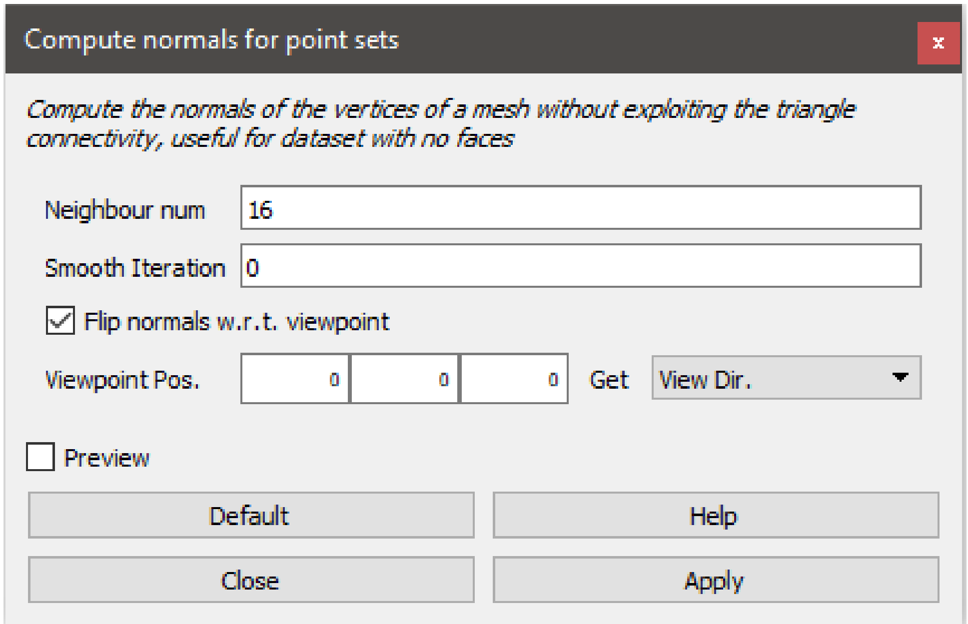


Figure 11.13 Computing Normals Settings

The surface is created using the *Screen Poisson Surface Reconstruction* algorithm. Navigate to *Filters – Remeshing, Simplification, and Reconstruction – Screen Poisson Surface Reconstruction*. The *Reconstruction Depth* is set to 12, the *Adaptive Octree Depth* to 8, and the *Scale Factor* to 1. The *Pre-Clean* option is also selected, seen in Figure 11.14.

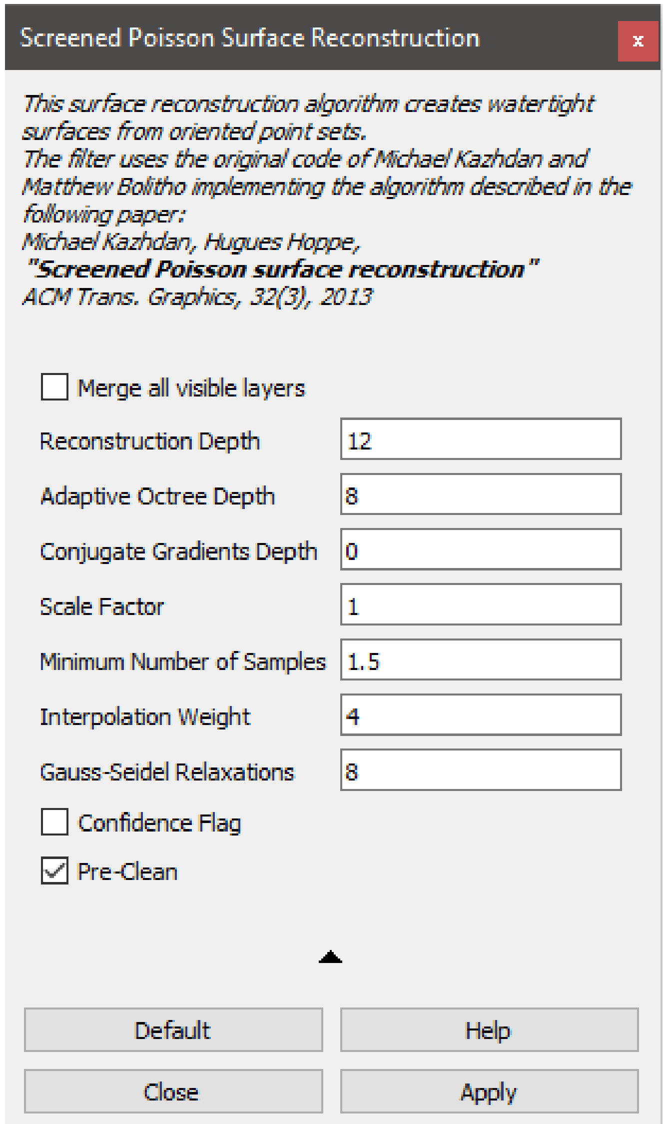


Figure 11.14 Screen Poisson Surface Reconstruction

Figure 11.15 shows the completed surface model generated in MeshLab. The shaft surface was relatively smooth with a large welding crease seen diagonally across the surface. This method of validating scanning settings and checking scan quality has proven quick and effective in the field.

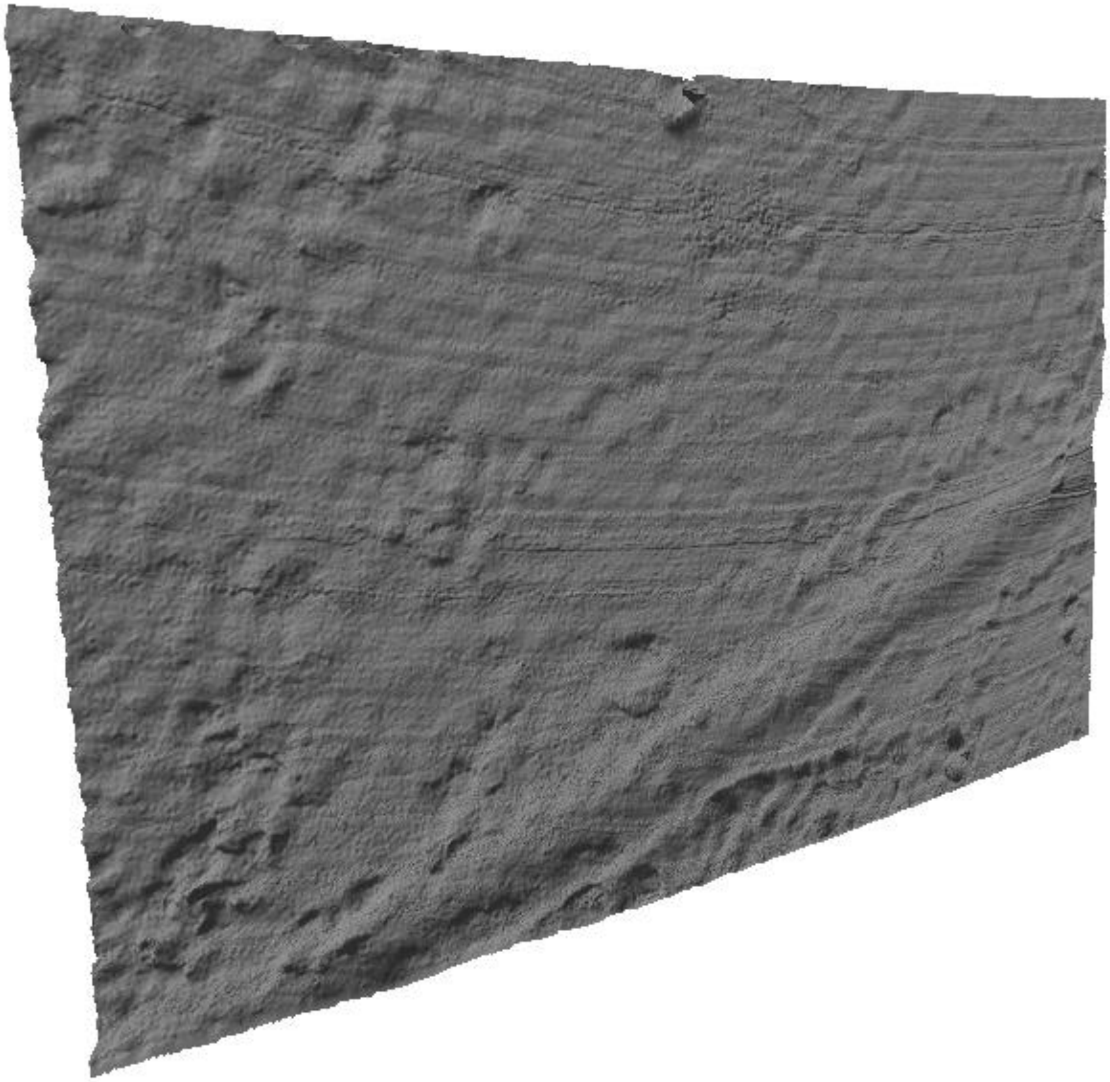


Figure 11.15 Completed Surface Model

The completed surface model is seen in Figure 11.15. This surface is visually compared to the physical surface being scanned to verify the scanner is correctly representing the concrete surface.

11.5 Sample Scans

The Newton underwater scanner has been tested extensively to better understand the best scan settings and the best environment for high quality scans. This section focuses on illustrating the capabilities of the underwater scanner. Before taking the scanner in the field, calibration scans, seen in Figures 11.16 and 11.17, were created to ensure the scanner was creating smooth 3D surface models. Figures 11.18-11.29 show scans of selected field and lab shaft surfaces.

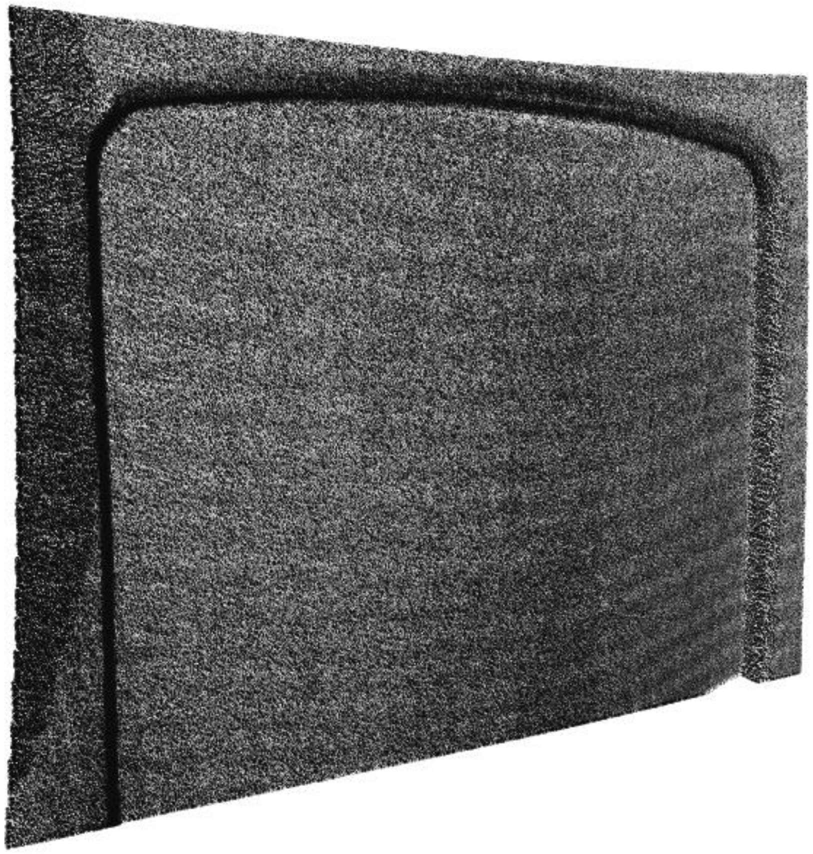


Figure 11.16 Smooth Surface Calibration Scan

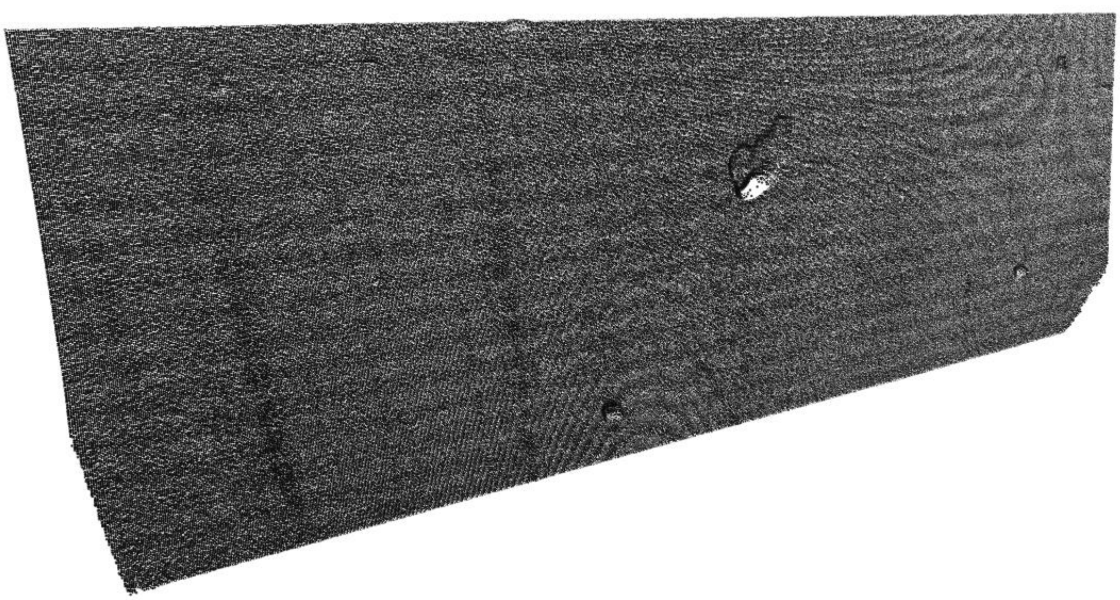


Figure 11.17 Smooth Surface with Minor Imperfections Calibration Scan

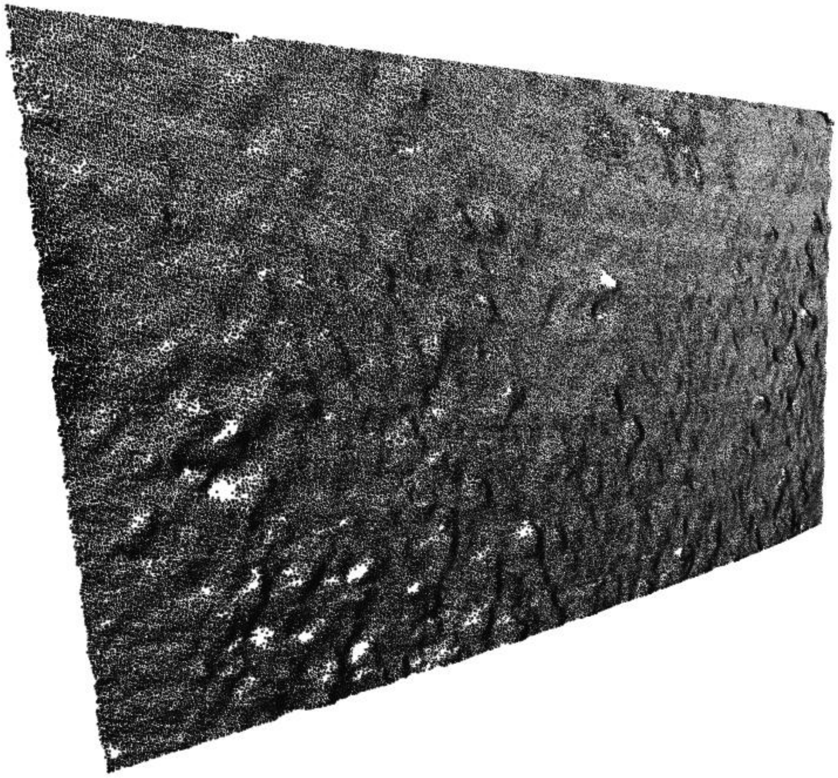


Figure 11.18 Gandy Pier 5 South Corner Pile Raw Scan Data 1

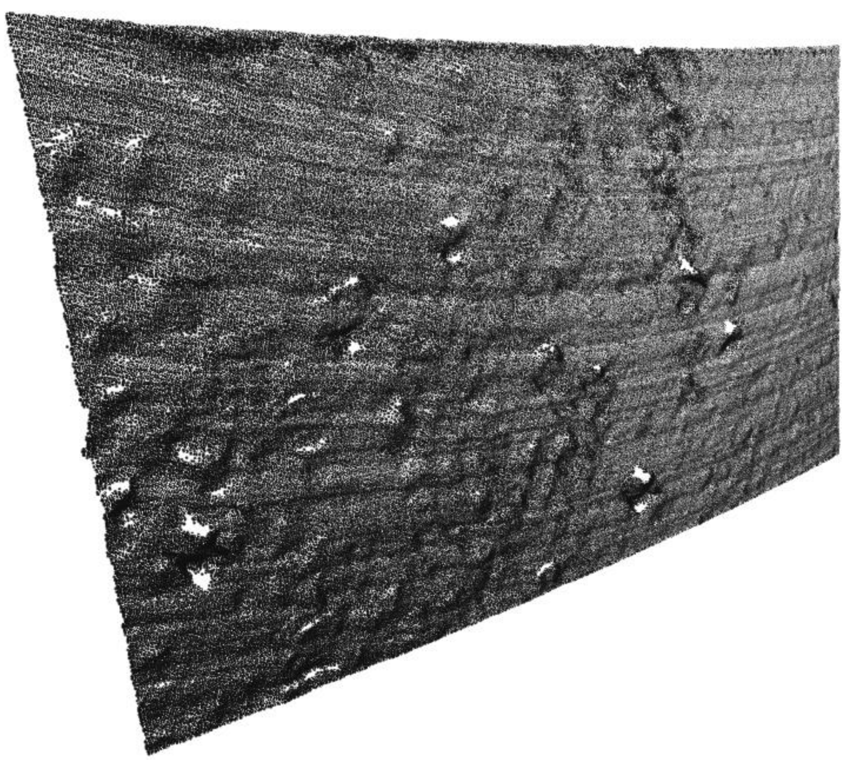


Figure 11.19 Gandy Pier 5 South Corner Pile Raw Scan Data 2

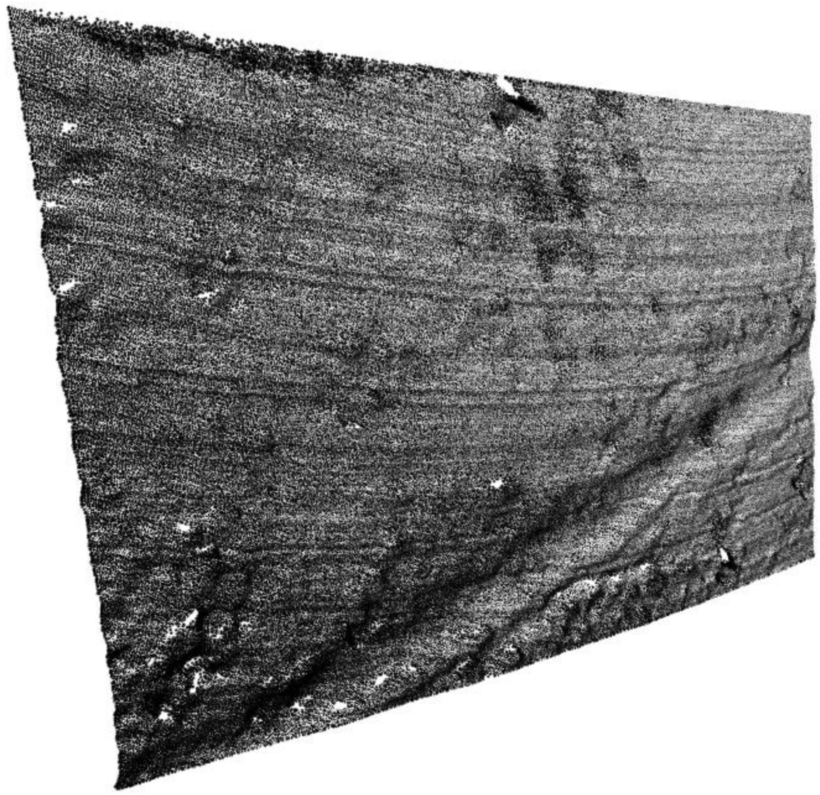


Figure 11.20 Gandy Pier 5 South Corner Pile Raw Scan Data 3

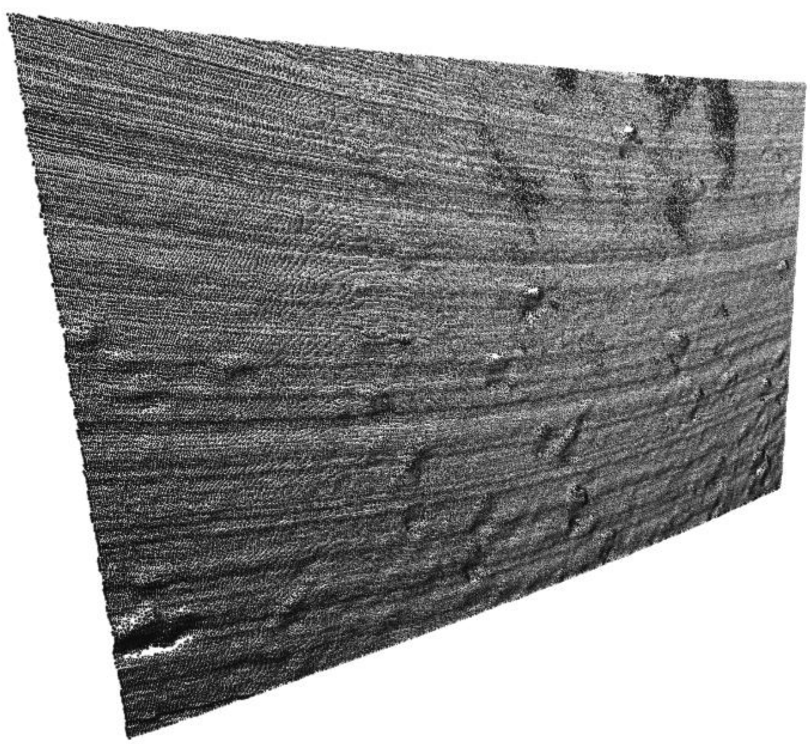


Figure 11.21 Gandy Pier 5 South Corner Pile Raw Scan Data 4

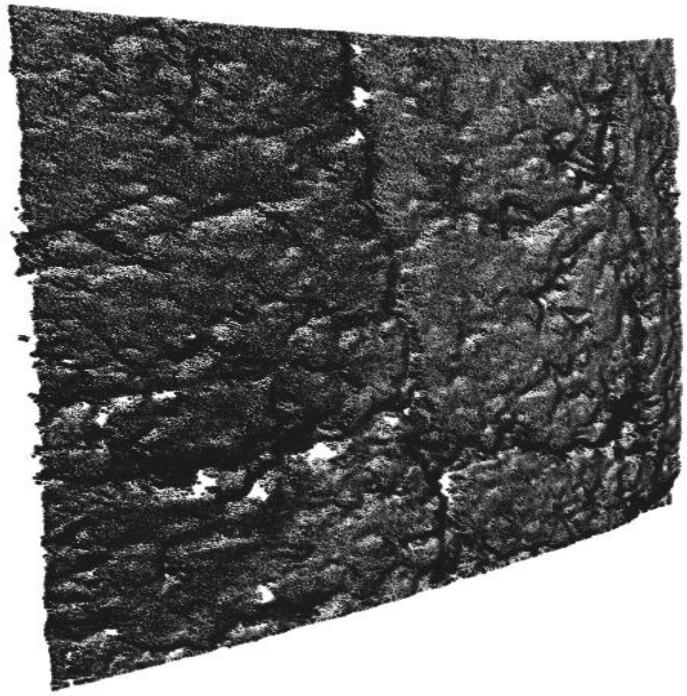


Figure 11.22 Bentonite-Cast Lab Shaft Raw Scan Data 1

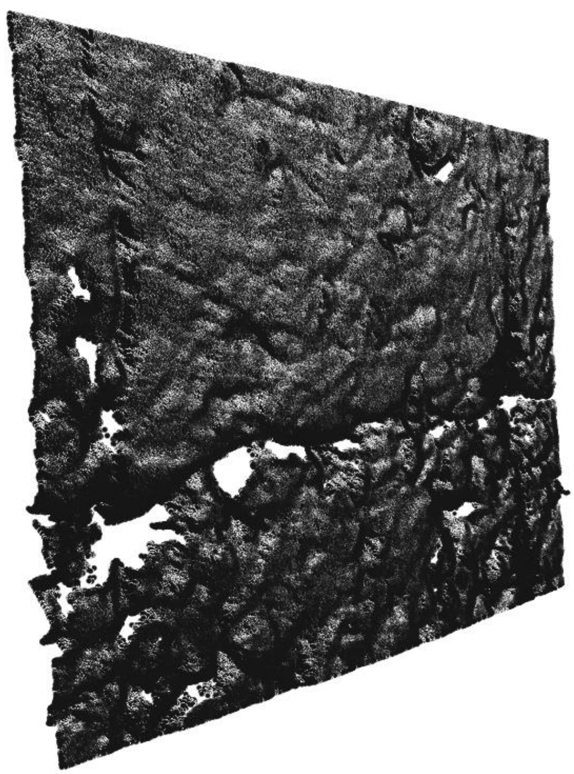


Figure 11.23 Bentonite-Cast Lab Shaft Raw Scan Data

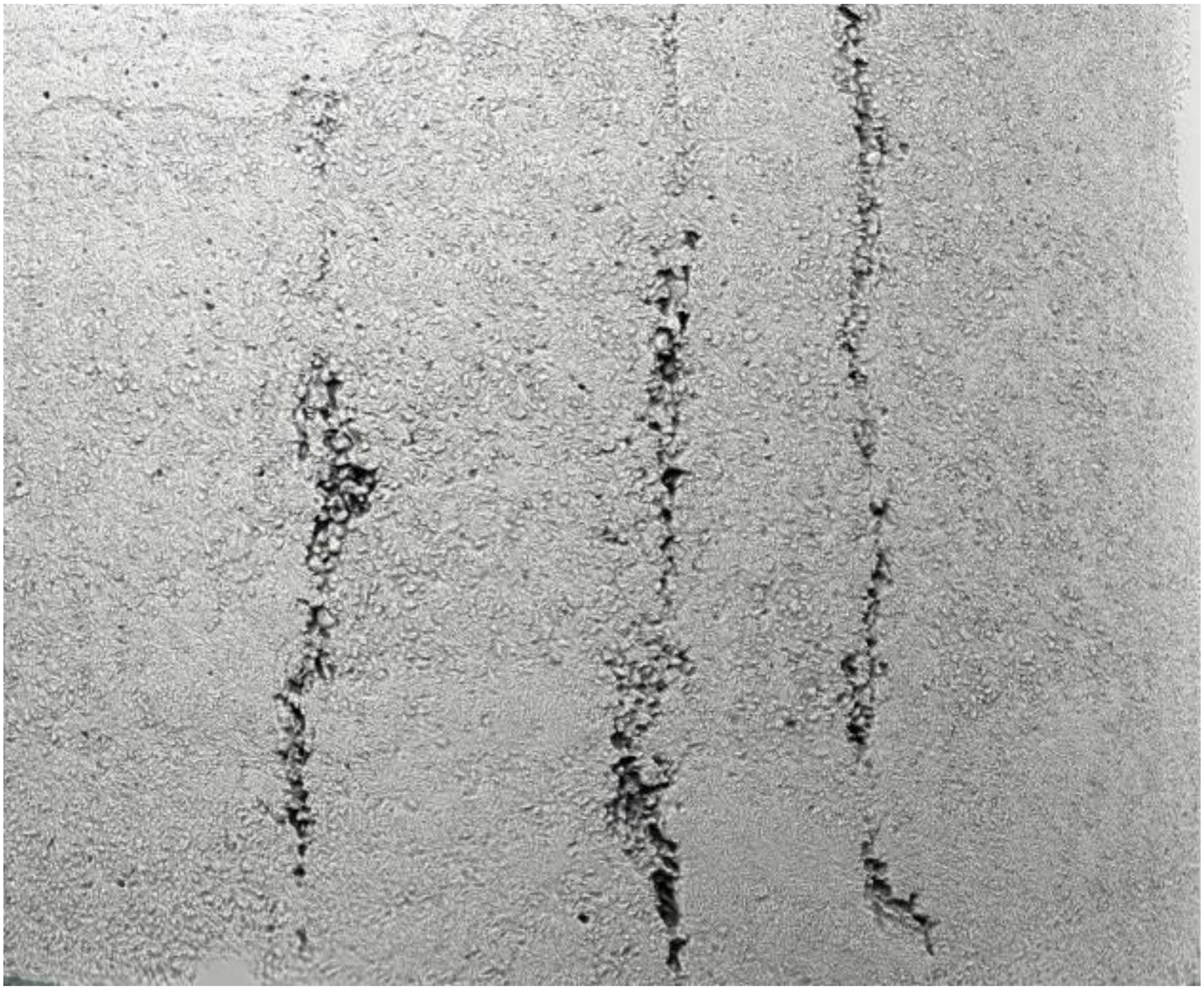


Figure 11.24 Digital scan of Pier 13, Shaft 1 (south) Caryville Bridge.

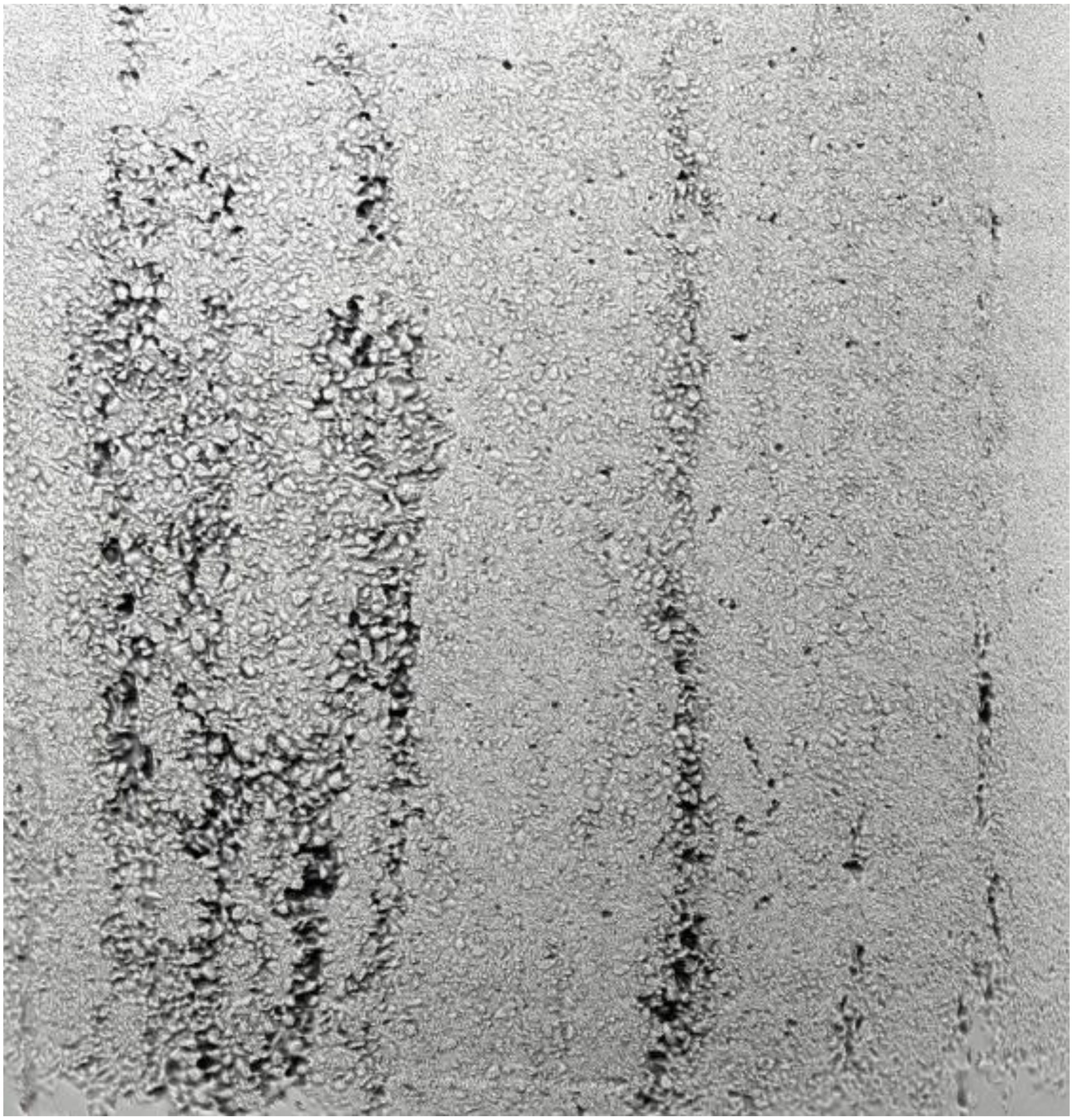


Figure 11.25 Digital scan of Pier 13, Shaft 2 (north) Caryville Bridge.

Figure 11.26 Digital scan of Pier 14, Shaft 1 Caryville Bridge.

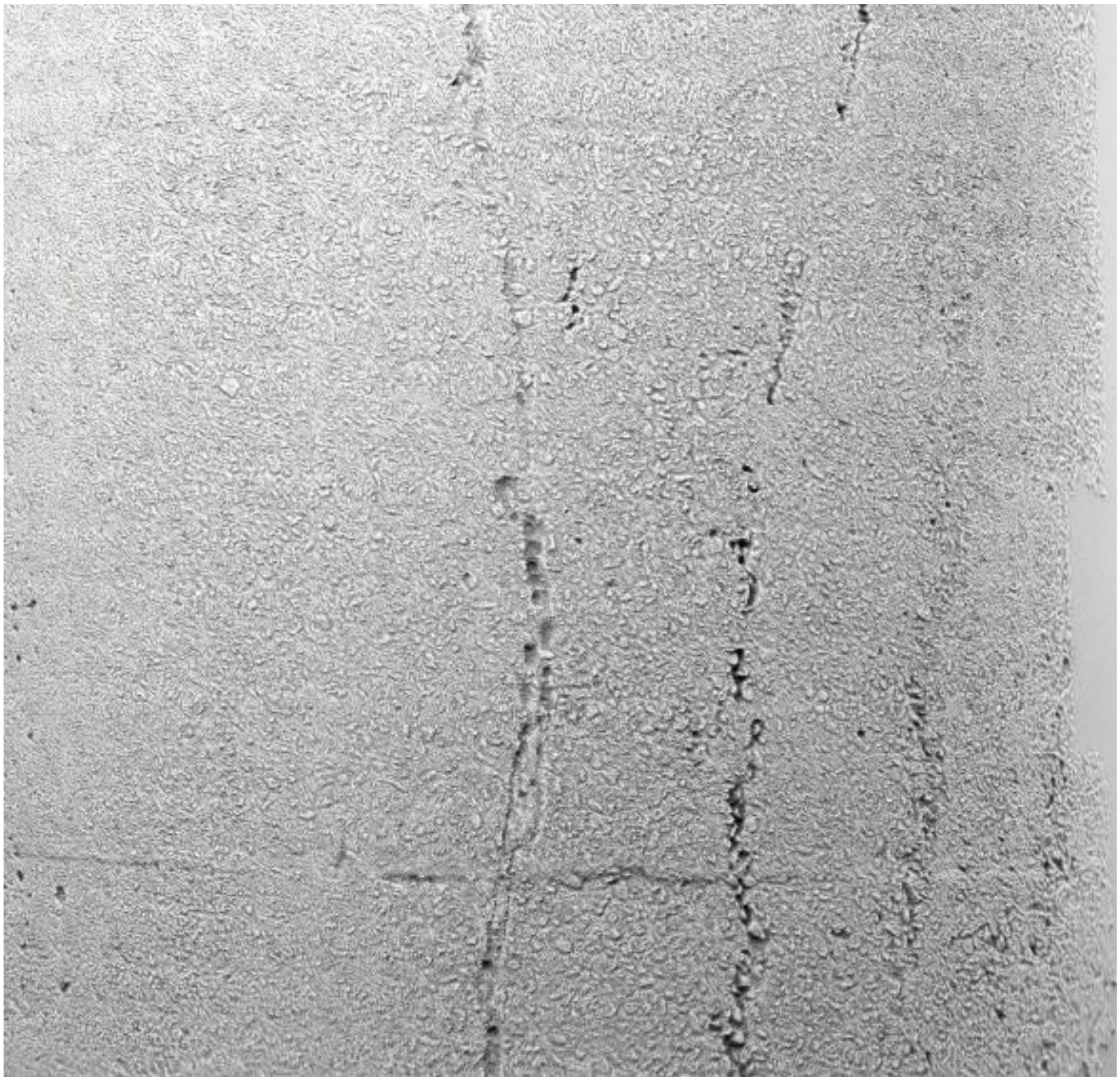


Figure 11.27 Digital scan of Pier 14, Shaft 2 Caryville Bridge.

Figure 11.28 Digital scan of Pier 59N, Blountstown Bridge.

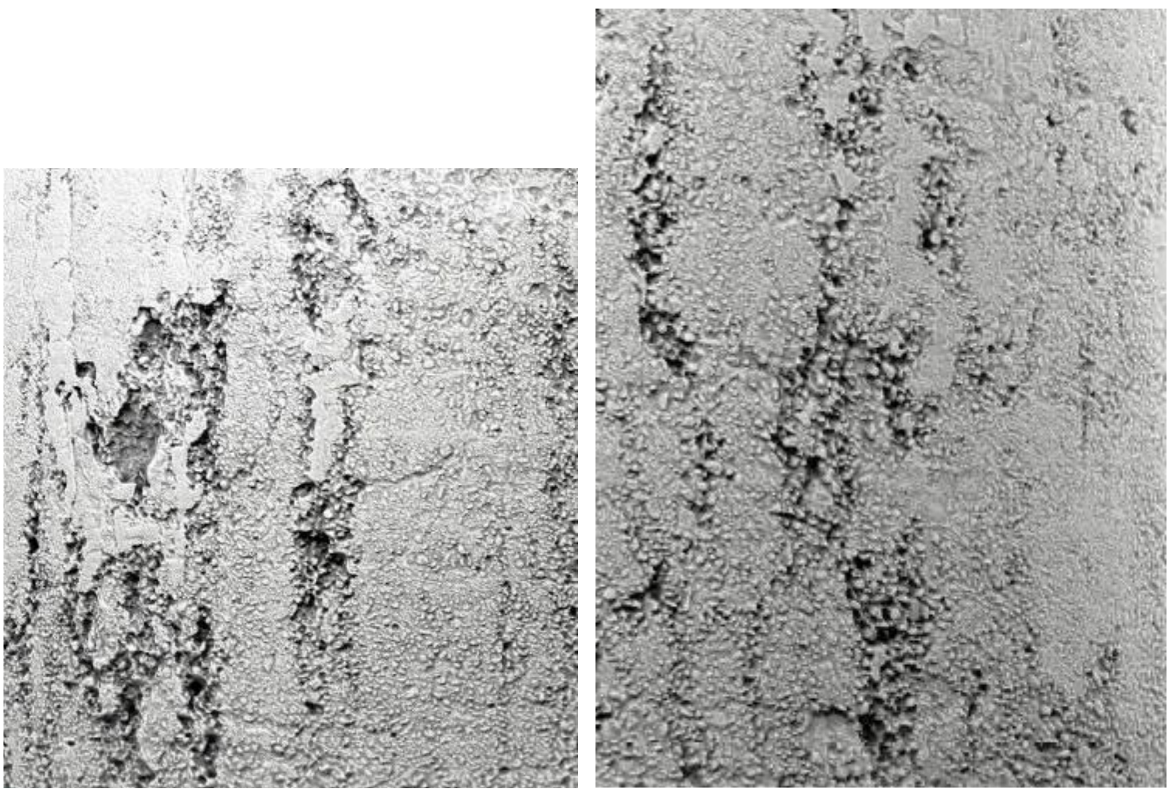


Figure 11.29 Digital scans of Pier 59S, Blountstown Bridge.

11.6 Chapter Summary

Understanding the connection between void volume and concrete cover quality is an important part in discussing drilled shaft construction methods. Using the Artec Eva 3D structured light scanner, the void volumes of lab specimens were determined. The Newton Labs underwater scanner allows the creation of 3D surface models from drilled shafts in real bridge foundations. These models were then processed to create 3D surfaces that illustrate the potential of this underwater scanner for collecting further void volume data.

Chapter Twelve: Discussion

The general school of thought in construction is that if all quality control and best management practices have been followed, the finished results will meet the design intention. In many instances this logic holds true; however, drilled shaft construction is primarily completed both underground and underwater which holds a certain level of uncertainty. The research presented herein emerged from the initial recognition that current construction methods and materials may be having negative effects on the durability and performance of drilled shafts that have heretofore gone unnoticed.

At the root of this issue, is a misconception as to how concrete flows in a tremie-placed, slurry displacing excavation. Recalling from Chapter one, Figure 12.1 demonstrates the comparison between the idealized concrete flow, rising in a single laminar lift and actual concrete flow that builds up inside the reinforcement cage before pushing horizontally into the cover region. As the

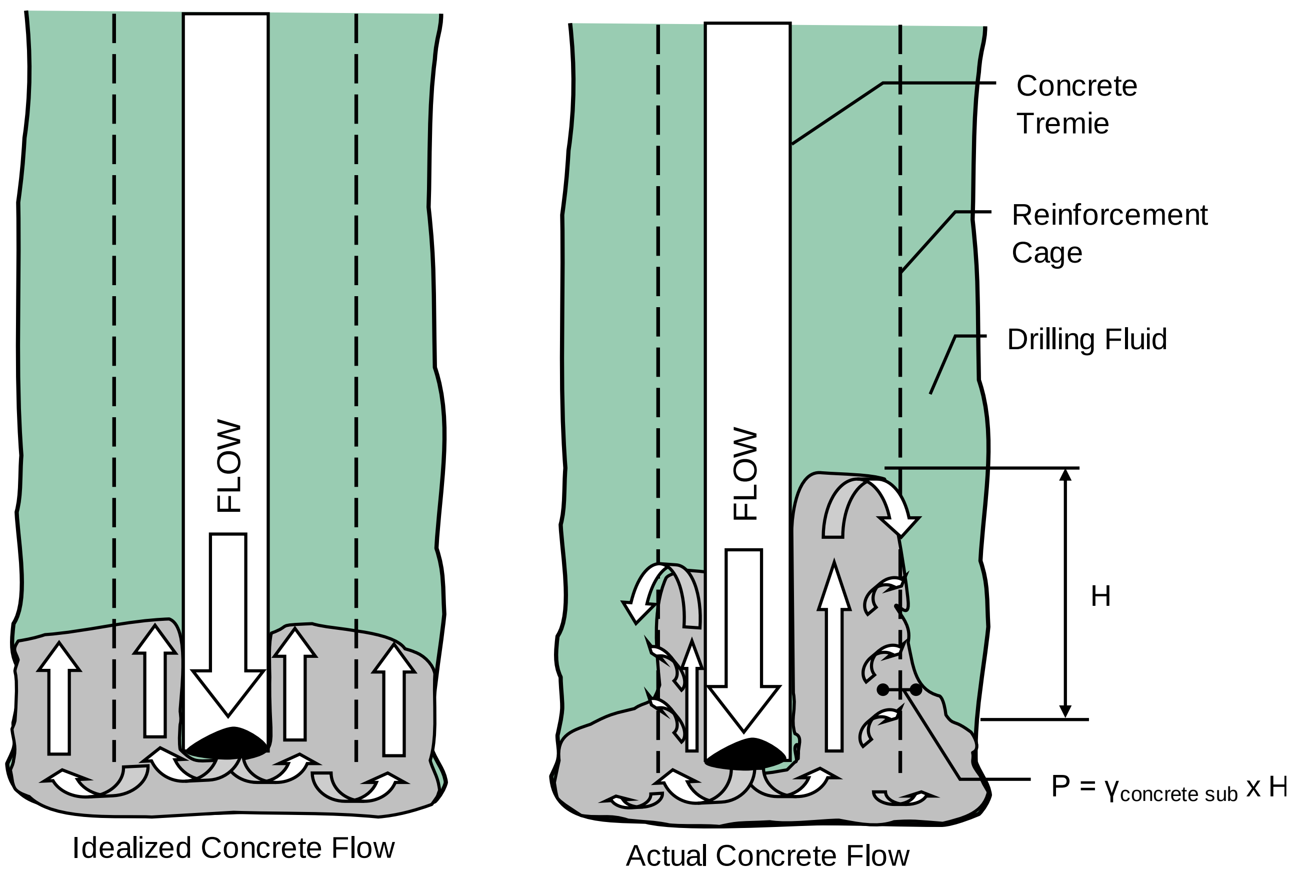


Figure 12.1 Comparison of idealized flow with observed (Mullins, 2005)

concrete travels through and around the reinforcement, support fluid becomes trapped as substantiated in Chapter three (Figure 12.2). This trapped fluid coats the reinforcement, compromises the cover region, and creates direct pathways for the transmission of environmental chlorides to the reinforcing steel. These pathways are reflected on the surface of the finished shaft and became the impetus for many of the experimental procedures detailed previously.

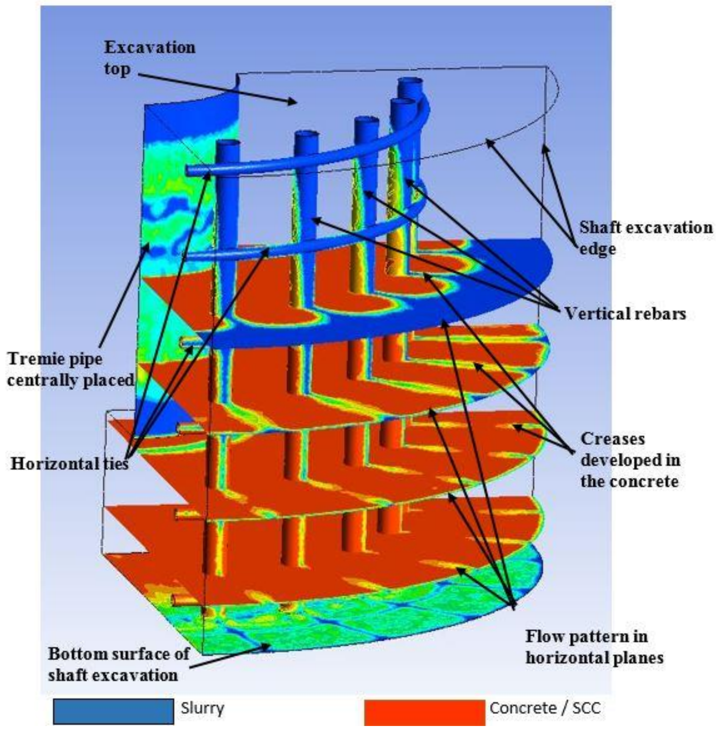


Figure 12.2 Concrete flow through reinforcing steel.

In order to encompass all of the impacts that arise throughout the concreting process, logic dictates beginning in the core of the shaft and working horizontally toward the finished shaft surface following the concrete flow.

12.1 Bond Strength

As the concrete leaves the tremie and pushes horizontally through the reinforcement, a layer of support fluid becomes trapped, coating the surface of the steel. The surface condition of the reinforcement bars is known to have an impact on pullout capacities as ACI 318-14, ACI 408R-03, and AASHTO equations for development length all provide modification factors for epoxy coated bars, ranging from 1.2 to 1.5, depending on the cage spacing and the cover dimensions. ACI 408R-03 explicitly discusses reinforcement surface condition, specifically bar cleanliness and epoxy coatings. Under bar cleanliness ACI 408R-03 states, "*To prevent a reduction in the bond strength, ACI 318 requires that reinforcement must be free of mud, oil, and other nonmetallic coatings that decrease bond strength*" (ACI Committee 408, 2003). However while this is stated, the only modification factors noted are for epoxy coating, with none stated for drilling fluid, which can leave a coating or residue on the reinforcement (Figure 12.3).

Figure 12.3 Bentonite coating on reinforcement and trapped around tie-wire.

Even under ideal concrete flow conditions, slurry becomes trapped along the topside of deformed rebar (Jones and Holt, 2004). However, in tremie-placed conditions the concrete flow is far from ideal as the concrete flows radially around the reinforcement, further encapsulating a layer of slurry on the bars and forming laitance creases. Ignoring the already accepted issues associated with concrete flow and cover, these laitance creases also cause a reduction in pullout capacity.

Given that a decrease in pullout capacity has been found for bentonite and polymer casting conditions when compared to water (control), this chapter discusses the recommendation of a slurry modification factor similar to that currently in place for epoxy-coated bars. Recommended slurry modification factors are proposed based on the results of all data collected, and not by viscosity (for bentonite) or manufacturer (for polymer), as seen in Chapter five.

12.1.1 Considerations for ACI 318-14

Based on the data analyzed, it is apparent that there is a decrease in reinforcement bond capacity and increase in data variability when slurry is present. There were two methodologies for ACI 318 that involved using or ignoring the $(c_b + K_{tr})/d_b$ term limitation. If the limitation was not used, resistance factors must be used to attain the desired reliability index of 3.5 for all casting conditions, Table 12.1. Recall that $\emptyset_d$ is used for development length and $\emptyset_b$ for bond strength as presented by Darwin et al. 1998.

Table 12.1 Effective strength reduction factor for all casting conditions for ACI 318-14 (no 2.5 limit).

Slurry Type	Water	Polymer	Bentonite	Dry
$\emptyset_d$	0.74	0.48	0.37	0.72

When incorporating the $(c_b + K_{tr})/d_b$ term limitation (of 2.5) and applying the same resistance factor concept, it was found that calculated values for water casting conditions (used as the control because data for dry conditions was unavailable) are exceedingly conservative, yielding a resistance factor of 1.795, negating the need for a resistance factor. Polymer slurries on average also proved to be more conservative than necessary, with a resistance factor of 1.171 and failure ratio of 1 in 24390. However, while water and polymer slurries were conservative, bentonite casting conditions were found to be below the acceptable level of reliability with a failure ratio of 1:1642 and resistance factor of 0.919. A visual depiction of the associated levels of conservatism is seen in Figure 12.4, which displays the Monte Carlo simulations for water, bentonite, and polymer; where anything below the 1:1 line dictates a failure.

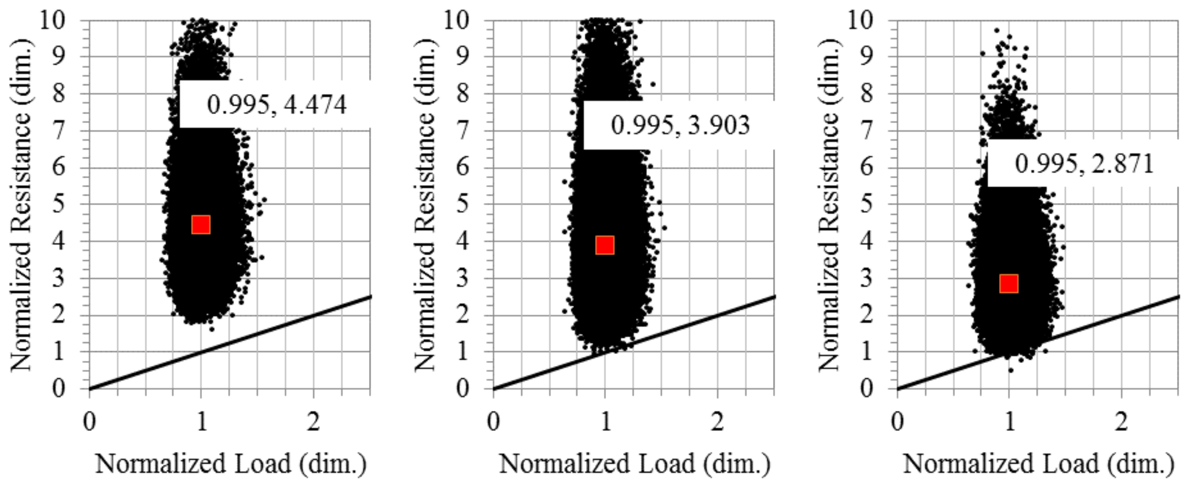


Figure 12.4 From left to right: water, polymer, and bentonite Monte Carlo simulations using the 2.5 limit.

Bentonite slurry is most concerning as it does not meet the desired level of reliability; however, there is a significant gap between the level of conservatism found for water conditions and that determined for polymer slurry. In order to produce the same level of conservatism, and thus reliability, for all casting conditions, development length modification factors (Ψ_{slurry}) were developed through the resistance factors determined from the level of reliability method, Monte Carlo simulations, and an alternate method using Equation 7, where $\Psi_{slurry} = l_{d\,slurry}/l_{d\,dry}$ (Equation 30) which then further simplifies to $\emptyset_{d\,dry}\,/\,\emptyset_{d\,slurry}$, shown below where x is all variables aside from $\emptyset_d$.

$$l_d = \left(\frac{3}{40} \frac{f_y}{\lambda \emptyset_d \sqrt{f_c'}} \frac{\Psi_t \Psi_e \Psi_s}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) d_b = x/\emptyset_d$$

Thus,

$$\frac{l_{d\,slurry}}{l_{d\,dry}} = \frac{x/\emptyset_{d\,slurry}}{x/\emptyset_{d\,dry}} = \frac{\emptyset_{d\,dry}}{\emptyset_{d\,slurry}} \qquad \text{(Eq. 12.1)}$$

While Equation 12.1 states for $\emptyset_{d\,dry}$ to be used as the control, natural slurry (water) conditions were used for the data generated utilizing the splitting failure limitation as no dry data was available. To make the level of conservatism equivalent, slurry modification factors were generated for both bentonite and polymer slurries of 1.95 (1.975/0.919) and 1.53 (1.975/1.171), respectively.

Without using the limitation, the initial resistance factors found in Table 4.4, produce slurry modification factors of 0.97, 1.90, and 1.49 for water, bentonite, and polymer, respectively, in relation to dry conditions. In order to achieve the desired level of reliability with no $(c_b + K_{tr})/d_b$ limitation, the slurry modification factor must be used in conjunction with the resistance factor for dry conditions. Otherwise, the modification factor can be disregarded and the resistance factor for the specific casting condition from Table 4.4 may be used.

Interestingly, using two different equation conditions yielded almost identical slurry modification factors. The small difference is most likely due to the change in control conditions. When the same resistance factor ratio ($\emptyset_{dry}$ / $\emptyset_{slurry}$) is used, and the dry factor is replaced by that for water, the slurry modification factors for bentonite and polymer become 1.95 and 1.53, respectively, identical to that found when utilizing the splitting failure limitation. Thus, if these values were to be compared using dry conditions instead of water, a subtle decrease in the modification value would be expected, based on this data.

It should be noted, that the resistance factors were calculated using the worst-case limitation value of 2.5. ACI 318-14 provides a table with a simplified equations (Table 5.1) where the $(c_b + K_{tr})/d_b$ term is set to either 1.5 or 1, depending on the spacing and cover conditions. When a value of 1.5 or 1 is used all casting conditions meet the failure ratio of 0:1,000,000 except for bentonite slurry, which results in a 1:1,000,000 failure ratio for a value of 1.5.

12.1.2 Considerations for ACI 408R-03

While analyzed similarly, there is quite a discrepancy between ACI 318-14 and ACI 408R-03 in how the limitation impacts reliability. While using the $(c_b + K_{tr})/d_b$ limitation for ACI 318-14 makes the reliability overly conservative, the same limitation for ACI 408R-03 does not have this effect. Only when resistance factors were calculated for bias values utilizing the 4.0 limitation and applied to the test data did the failure ratios meet the desired reliability index. Thus, when using or disregarding the limitation for splitting failure, for ACI 408R-03 a resistance factor must be used. Table 12.2 shows bond resistance factors that may be used for various values of this limitation, dry conditions have been excluded.

Table 12.2 Bond resistance factors ($\emptyset_b$) for varying casting conditions for various limitation values.

Limitation Applied	Water	Bentonite	Polymer
None	0.677	0.441	0.418
4.0	0.707	0.449	0.434
2.5	0.826	0.547	0.512
1.5	0.923	0.624	0.581
1.0	0.976	0.660	0.624

ACI 408R-03 recommends using a value of 0.82 for $\emptyset_d$ based on $\emptyset_b/0.9$ and applies this to Equation 5.20. While utilizing individual resistance factors as noted in Table 5.1 is one method of attaining the desired level of reliability, slurry modification factors could alternatively be used in conjunction with the 0.82 resistance factor (i.e. Equation 5.20). Due to the formulation of Equation 20, the simple resistance factor ratio used for ACI 318-14 is not applicable. Thus instead Table 12.3 was formed computing the development length multiplier ($l_{d\ slurry}/l_{d\ dry}$) for water, bentonite, and polymer slurries based on concrete compressive strength. This table was generated using resistance factors generated with no $(c_b + K_{tr})/d_b$ limitation; however, slurry factors stay the same regardless of the limitation used. Water conditions show a general multiplier of 1.1, which can be applied to all concrete strengths; however, bentonite and polymer slurries range from 1.9 to 2.0 and 2.0 to 2.1, respectively as concrete strength rises. The use of these proposed slurry modification factors will allow slurry-coated reinforcement to achieve the desired level of reliability.

Table 12.3 Development length multipliers for ACI 408R-03.

f'_c (psi)	Water	Bentonite	Polymer
3000	1.1	1.9	2.0
4000	1.1	1.9	2.0
5000	1.1	1.9	2.1
6000	1.1	1.9	2.1
8000	1.1	2.0	2.1
10,000	1.1	2.0	2.1

12.2 Electrochemical Evaluation

As the outward flowing concrete is cleaved by the reinforcement, it rejoins in the cover region leaving a crease and creating a direct pathway to the surface of steel, negating the protective qualities of the concrete cover and opening up the system to increased potential for corrosion. Two sets of half-cell potential readings were taken on all shaft specimens over a timeframe of 2 months to 6 years after casting. The average E_{50} potential values for all water, polymer, and bentonite specimens were -264, -280, and -365mV, respectively. This confirms corrosion protection for water-cast specimens but highlights the original concerns about the cover quality of bentonite shafts, and brings into question the consistency of a polymer-cast shaft. Seventy percent of all bentonite specimens exhibited E_{50} surface potentials below the -350mV corrosion threshold, and

84% of bentonite samples exhibited localized critical potential values (E_{min}) somewhere in the measured data field. Viscosity is commonly perceived as the controlled parameter for support fluids, with the intention that if a viscosity remains within a specified range, the material is suitable for use. In Figures 12.5 and 12.6, the E_{50} and E_{min} values for all subject shafts are plotted versus the viscosity of the support fluid. What becomes apparent is there is no "safe" viscosity range for either bentonite or polymer. Readings below the corrosion threshold can be found at all viscosities in both plots. This indicates support fluid viscosity is not a controlling parameter for corrosion protection, the mere presence of support fluid is enough to compromise durability.

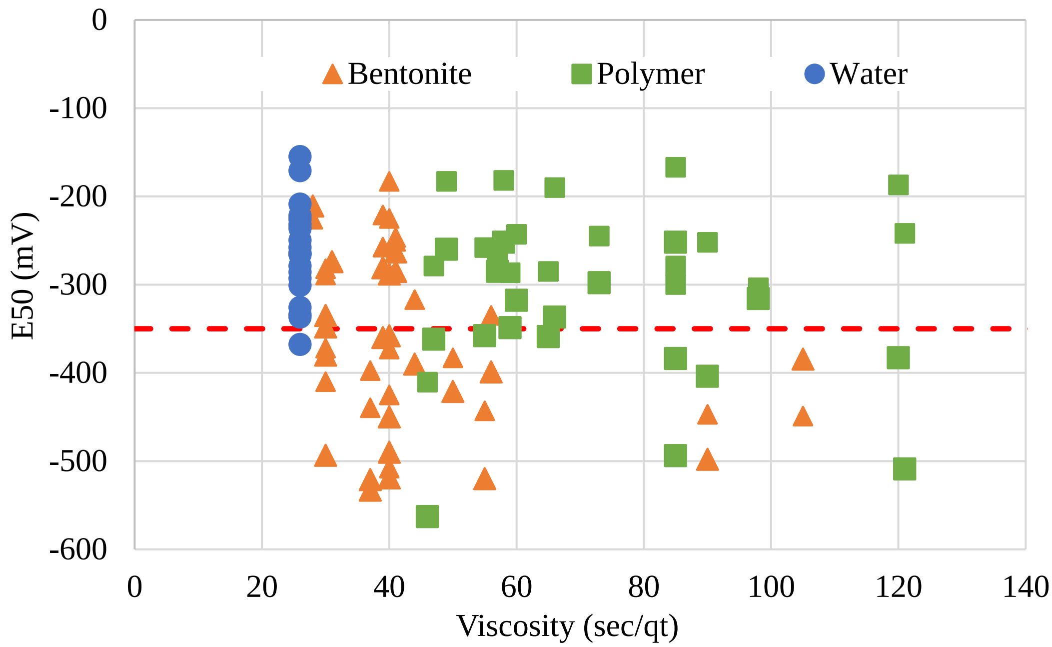


Figure 12.5 E_{50} versus viscosity

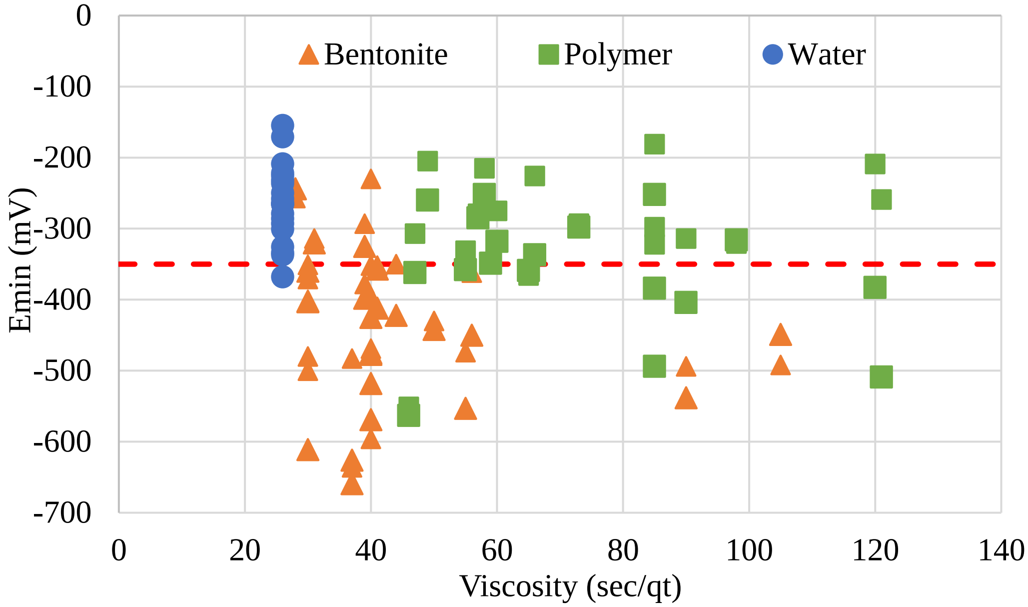


Figure 12.6 E_{min} versus viscosity

As noted, two sets of data were collected for each shaft specimen. Figures 12.7 – 12.9 show the E_{min} values plotted versus age of the specimen at the time of testing for bentonite, polymer, and water, respectively. Each fluid showed a worsening trend of -10, -17 and -3 mV/year, respectively. The polymer data was further separated by manufacturer but no substantial trend was noted. Corrosion is most often expressed in rates. Though this data is too scattered to confidently establish a corrosion rate specific to each support fluid, it does confirm corrosion is a function of shaft age and support-fluid-cast shafts are corroding at a faster rate than those cast in water.

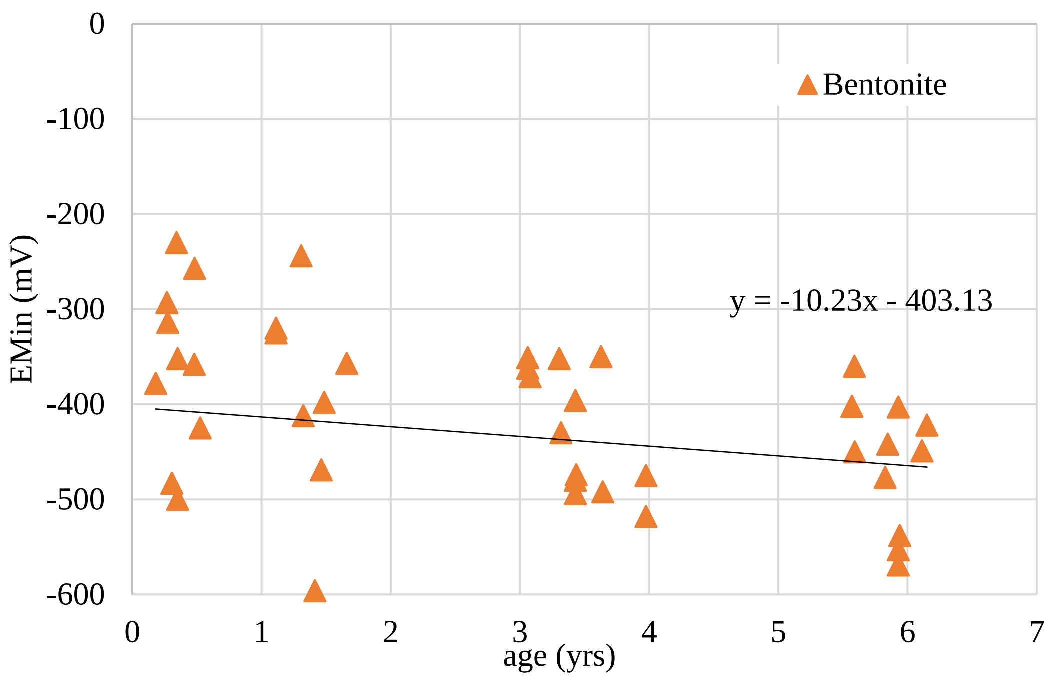


Figure 12.7 Minimum half-cell potential versus time

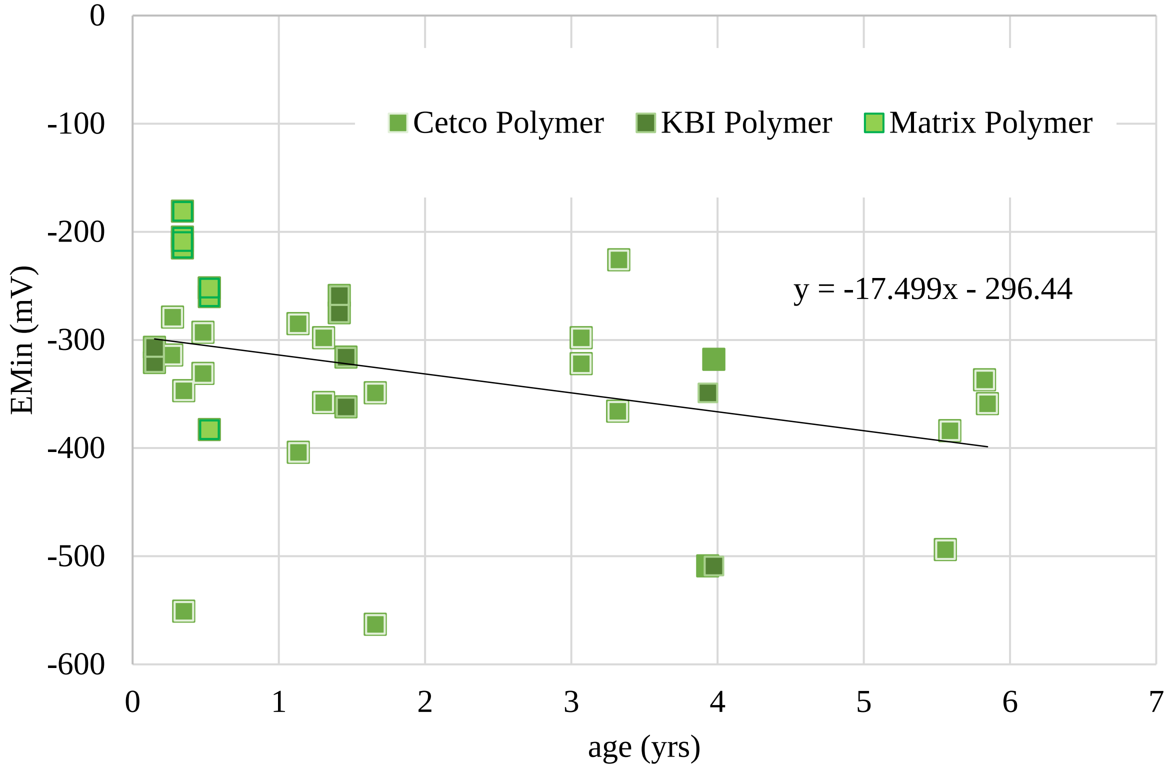


Figure 12.8 Minimum half-cell potential versus time

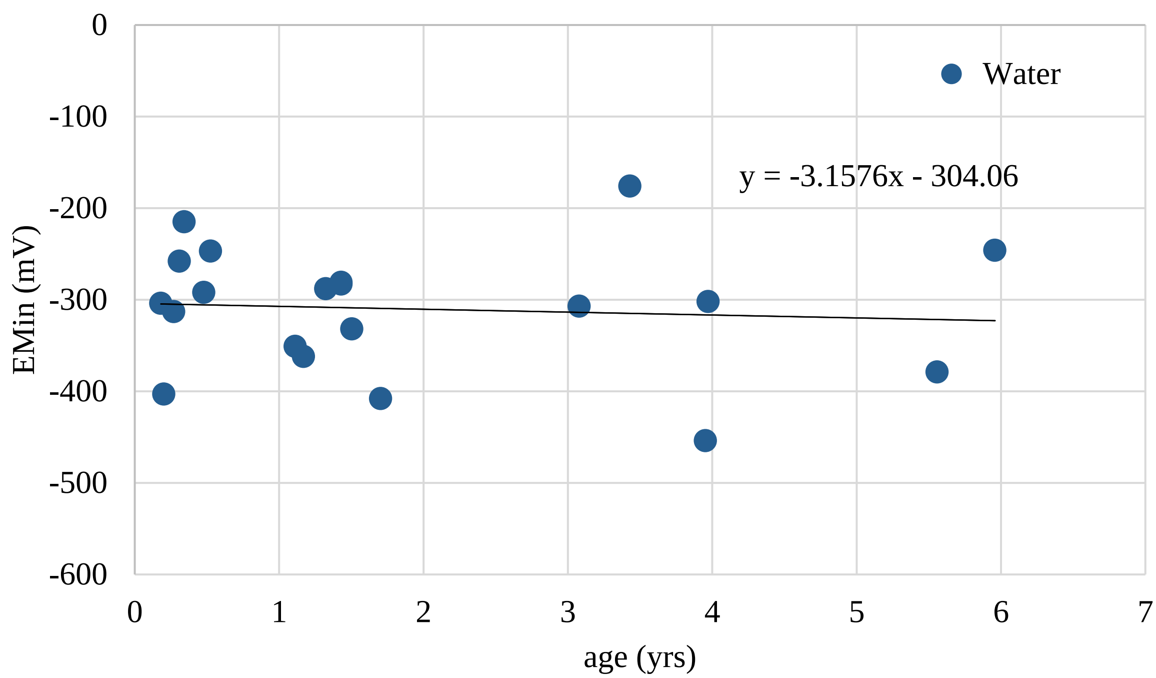


Figure 12.9 Minimum half-cell potential versus time

12.3 Strength Profiling

The strength of concrete is determined by several factors, but is it commonly accepted that compromised concrete displays lower strength properties than uncompromised concrete. Working under the presumption that concrete not only mixes with support fluid in the cover region during concreting but that the mixing constitutes a compromised material, a minimum of five strength profiles was taken from each of 30 subject shafts using an instrumented drill constructed for that purpose. Of those five, two profiles were taken inside the reinforcement cage, two were taken in the cover region and one was taken on a vertical crease. The average ratios of concrete strength inside the reinforcement cage and in the cover region for bentonite-, polymer-, and water-cast shafts were, 0.88, 0.97.5, and 0.99.5, respectively. This confirms the concrete cover quality of polymer- and water-cast shafts, but again highlights the effects of concrete mixing with bentonite support fluid as it goes through and around the reinforcement cage during casting. In Figure 12.10, the relative concrete cover strength is plotted versus the support fluid viscosity. The dotted red line represents the baseline strength, and apparent reductions in strength are noted for all support fluid types through the full range of viscosities tested.

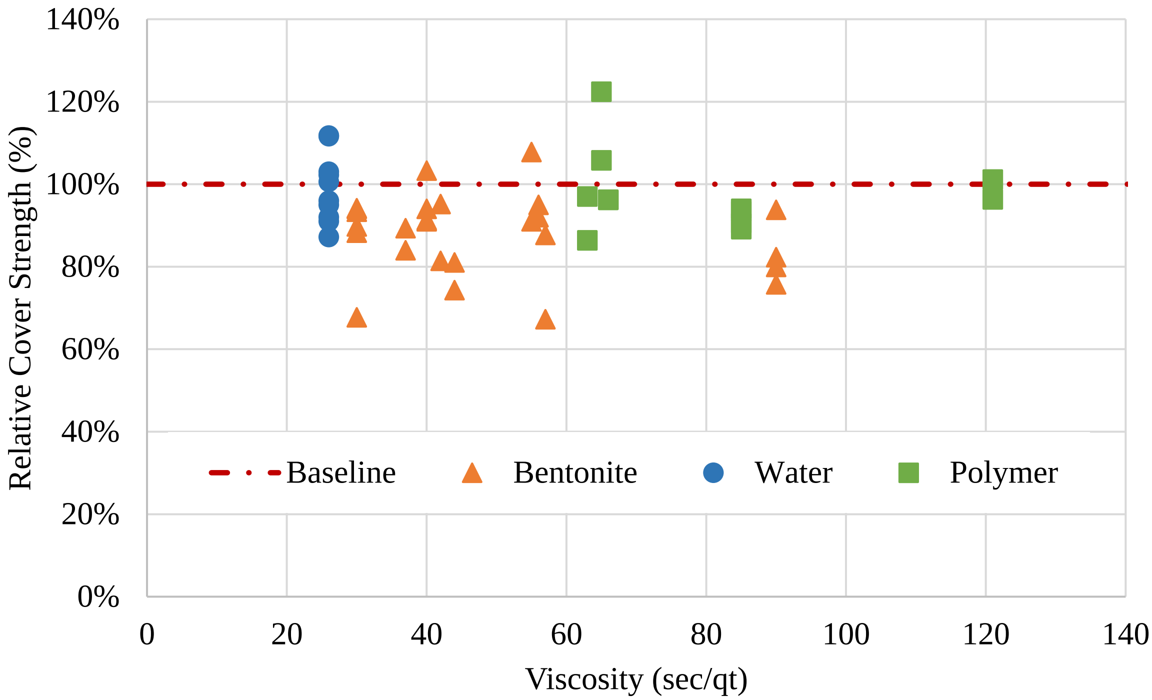


Figure 12.10 Relative cover strength versus viscosity

For each shaft tested, a strength profile was taken on a vertical crease in the cover region. This crease should represent the worst-case strength for each shaft. The average ratio of concrete strength inside the reinforcement cage and along a vertical crease for bentonite-, polymer- and water-cast shafts were, 0.82, 0.86 and 0.92, respectively. Where the reduction in strength in the cover region could be attributed to mixed or compromised concrete materials, the strength reduction at the crease can be taken as the result of missing concrete material. The relationship between measured strength and lost concrete area is linear, so a prorated strength can be assigned assuming zero strength from the crease region and full-strength concrete outside the crease (Figure 12.11 and Equation 12.2). Using that logic, a relationship between the lost area below the core bit and the crease width was developed.

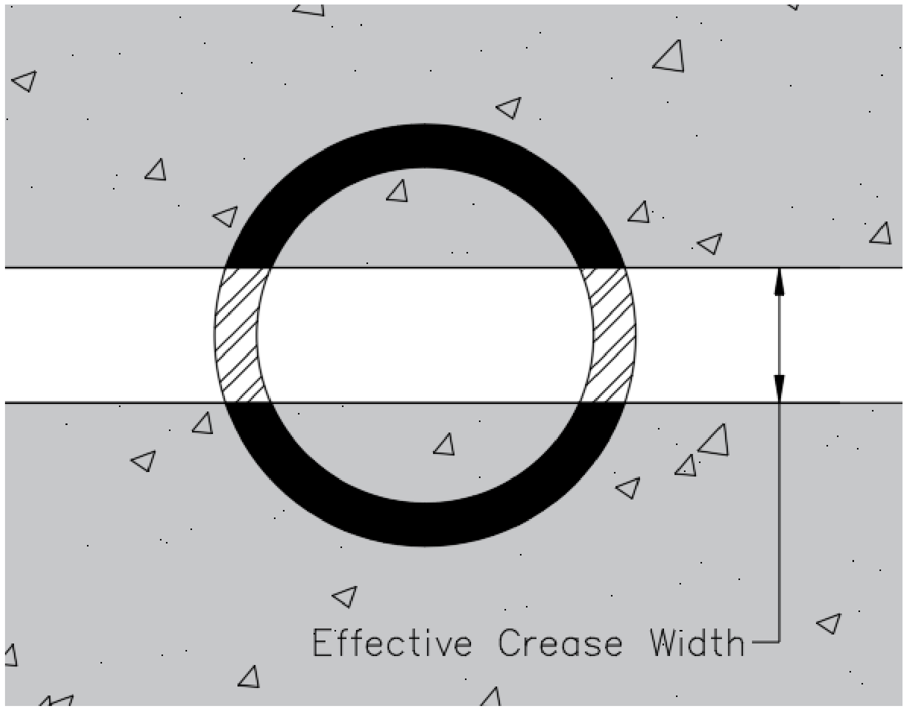


Figure 12.11 Effective crease width

$$f'c_{measured} = \frac{A_{crack}(f'c=0)+A_{remainder}f'c_{core}}{A_{total}} \tag{12.2}$$

Figure 12.11 shows the cross section of a core bit over a crease. AutoCAD area tools were used to compute the area of the core bit over the crease as the crease width increased. The crease widths ranged from 0 inches to 1.25 inches in order to accommodate all possible area values. The results of those calculations were plotted and fit with a second order polynomial (Figure 12.12). The equation of best-fit polynomial was used to calculate the average effective crease width for each crease strength profile (Tables 12.4-12.6).

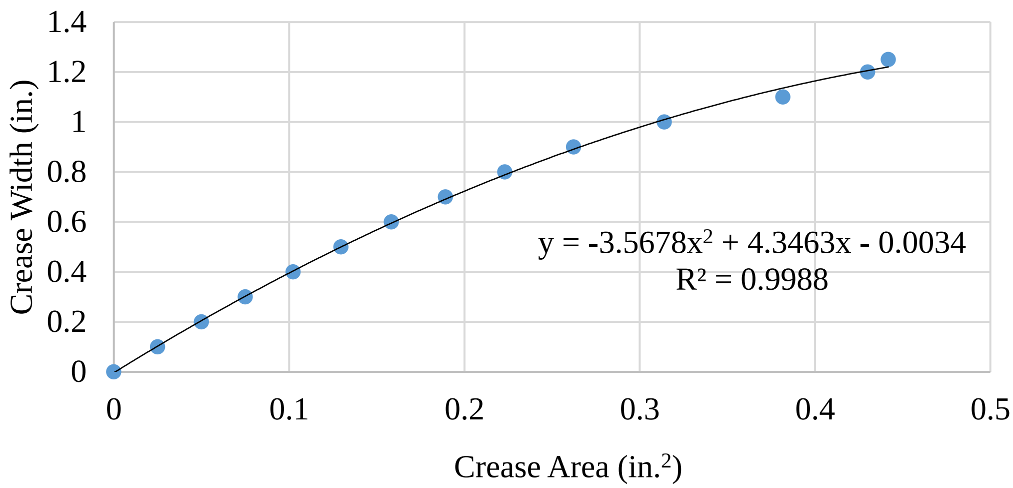


Figure 12.12 Effective crease width, effective crease area relationship.

Table 12.4 Bentonite effective crease width

Bentonite				
Shaft #	Viscosity	Baseline	Crease Strength Ratio	Effective Crease Width
	(s)	(psi)		(in.)
1	44	4687.9	0.82	0.31
3	40	4296	0.86	0.24
4	55	4268	1.06	-0.08
5	90	5022.7	0.61	0.66
7	30	4303.5	0.67	0.56
8	40	4532.3	0.88	0.20
9	57	5627	0.74	0.46
10	90	5142	0.77	0.40
13	30	3796.115	0.93	0.11
14	30	4511.9	0.68	0.54
15	56	4742	1.05	-0.07
21	42	5092.17	0.73	0.47
45	37	4847.1	0.83	0.29
			Average	**0.31**

Table 12.5 Polymer effective crease width

Polymer				
Shaft #	Viscosity	Baseline Strength	Crease Strength Ratio	Effective Crease Width
	(s)	(psi)		(in.)
11	65	4220	1.04	-0.05
12	66	5976	0.77	0.40
16	85	4045	0.87	0.22
17	85	4345	0.91	0.15
19	63	5739	0.84	0.28
20	121	4720	0.76	0.42
			Average	**0.24**

Table 12.6 Water effective crease width

Water				
Shaft #	Viscosity	Baseline Strength	Crease Strength Ratio	Effective Crease Width
	(s)	(psi)		(in.)
6	26	4752	0.85	0.26
18	26	3957.3	0.84	0.28
22	26	4597.6	0.88	0.21
32	26	4956.7	1.15	-0.19
46	26	5341.3	0.86	0.25
			Average	**0.16**

The average effective crease width for the bentonite-, polymer-, and water-cast shafts were 0.31 inches, 0.24 inches, and 0.16 inches, respectively. The coresponding maximum effective crease widths were 0.66 inches, 0.40 inches, and 0.28 inches, respectively. Again, the shafts cast in bentonite support fluid showed the most dramatic effects and visual inspection of the surface of the bentonite shafts corroborated the calculated widths. What is surprising about these results is the surface of the polymer and water cast shafts showed very little, if any, evidence of creasing. What this means is although the concrete is visibly perfect, the material had been compromised by a poorly-bonded interface due to entrapped laitance and perhaps a region of higher w/c ratio.

The average strength of each shaft has been used as a means of simplifying a large data set into comparable discrete points. The reality of these strength issues is they vary not only radially (inside to outside of the reinforcement) but also with depth. Figure 12.13 shows a sample effective crease area profile (grey) along with the crease strength profile (orange) and the baseline core strength profile (blue).

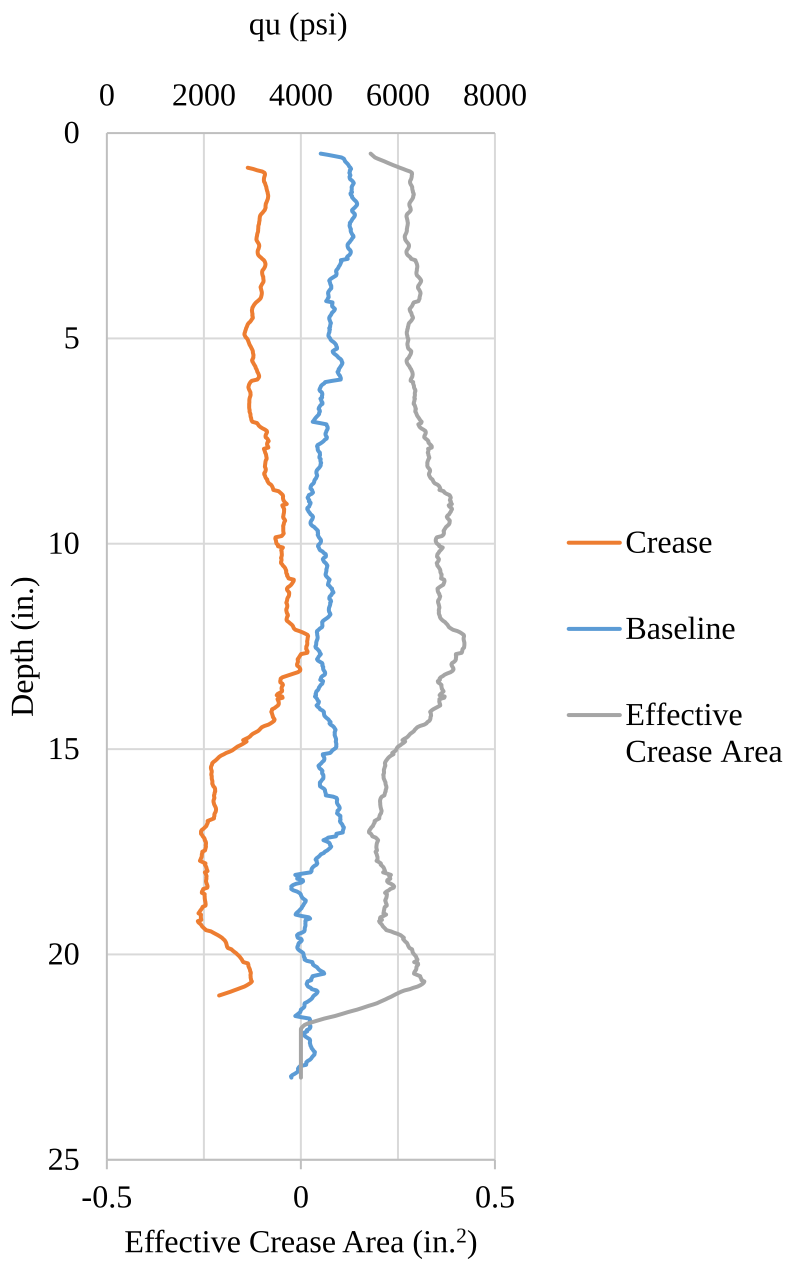


Figure 12.13 Effective crease area profile.

The tabulated crease width and strength ratios in Tables 12.4 – 12.6 were based on average values from the strength profiles. However, from the Figure 12.13 profile, large variations in crease area can be seen that were most often compromised in the bottom half of the shaft.

12.4 **Surface Roughness**

The pattern formed by the trapped laitance creases is reflected to varying degrees on the surface of the support-fluid-cast shafts. A quantification of this surface pattern, or roughness, was determined by two methods, physical and digital. The results from both sets of tests being comparable, the digital method has been used herein for the purpose of simplification. The average surface roughness values for all bentonite-, polymer-, and water-cast shafts were 10.86 in.^{3}/sf, 3.57 in.^{3}/sf, and 2.36. in.^{3}/sf, respectively. Using this system, the larger the roughness value, the more deteriorated the surface. The bentonite average is three times that of the polymer or water. This confirms the visual assessments for each shaft. Plotting these values versus the viscosity of each support fluid again reveals the noted trends appear to be uninfluenced by viscosity (Figure 12.14). In the case of bentonite, high surface roughness values were obtained from across the full range of viscosities tested, the worst of which were within normal construction limits of 30 to 50 sec/qt.

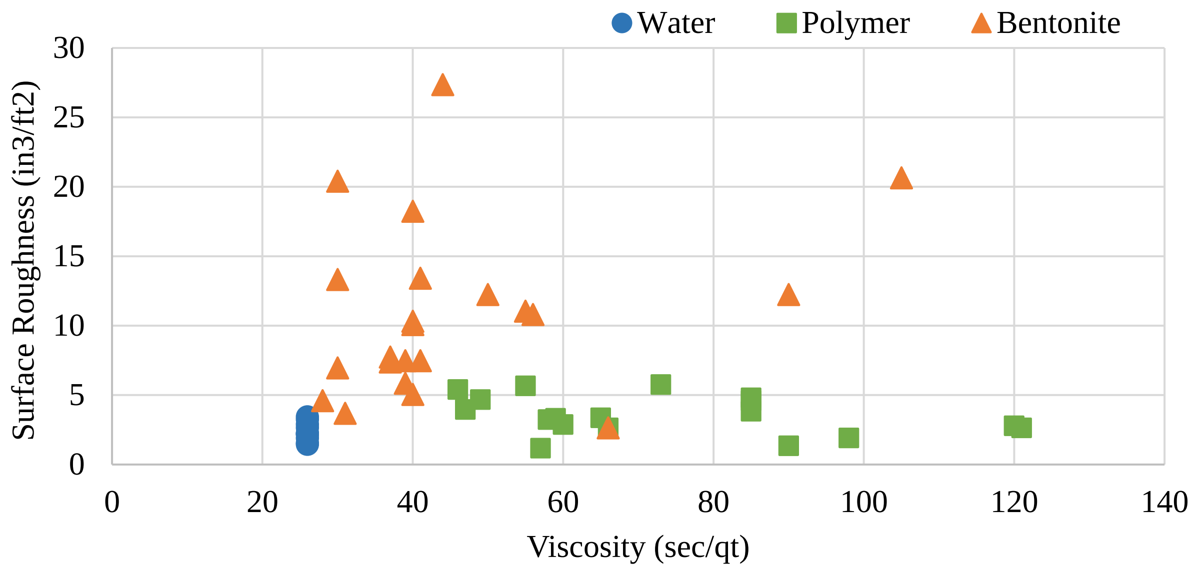


Figure 12.14 Surface roughness versus viscosity

The primary focus of this study began as exploratory with the intent of gaining the ability to quantify field observations and make confident correlations between surface roughness and structural health. As electrochemical measurements are difficult, if not impossible, to take underwater, defining a correlation between surface quality / roughness and half-cell potential became of paramount importance. Figure 12.15 shows one such correlation where the E_{50} values for both tests on each shaft were graphed against the digital surface roughness.

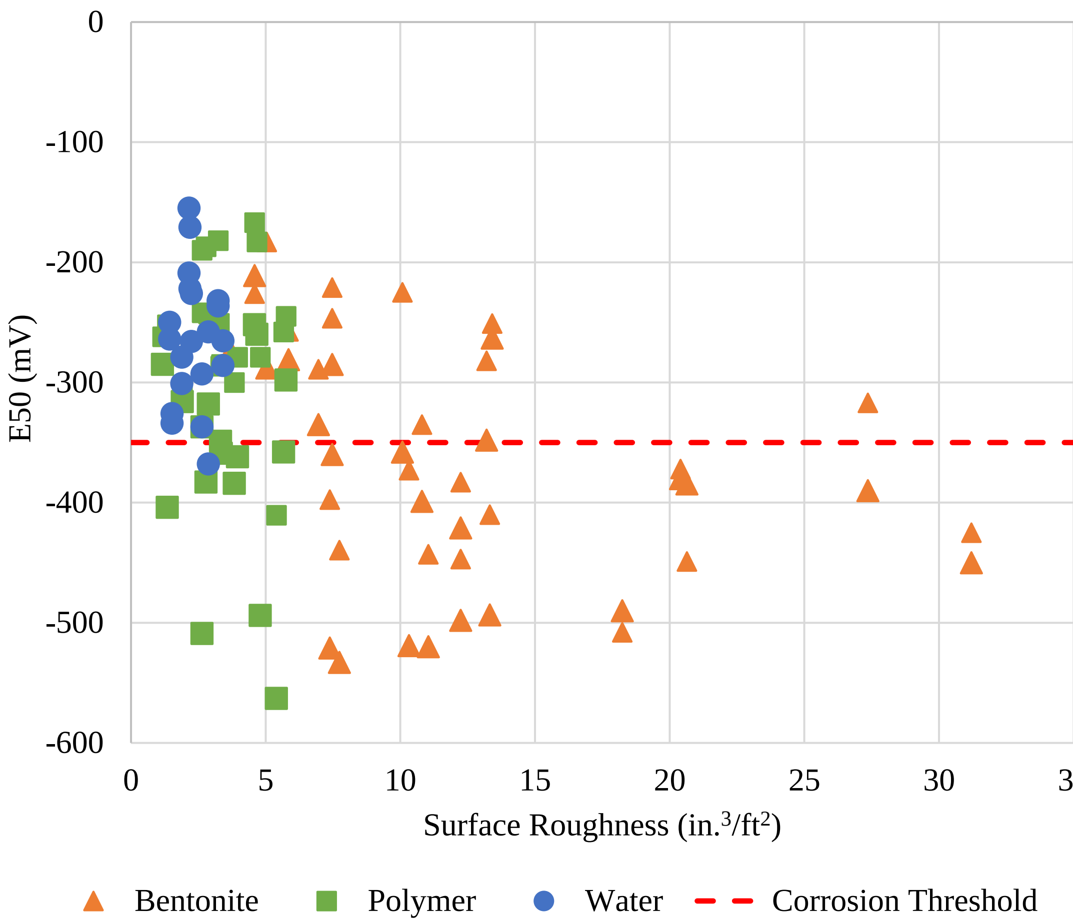


Figure 12.15 E_{50} versus surface roughness

In Figure 12.15 the data sets are separated by support fluid type. The blue circles are water-cast shafts and all but one is above the corrosion threshold and all have a surface roughness less than 5 in.^{3}/sf. The polymer-cast shafts are represented by green squares and are scattered almost evenly above and below the corrosion threshold but maintain a smooth surface with surface roughness values below 7 in.$^{3/}$sf. The orange triangles are bentonite-cast shafts. The majority of the bentonite data points are below the corrosion threshold and all but two have surface roughness values above 5 in.^{3}/sf with some surpassing 30 in.^{3}/sf. When the E_{min} value is plotted against the surface roughness the trends are even more apparent (Figure 12.16).

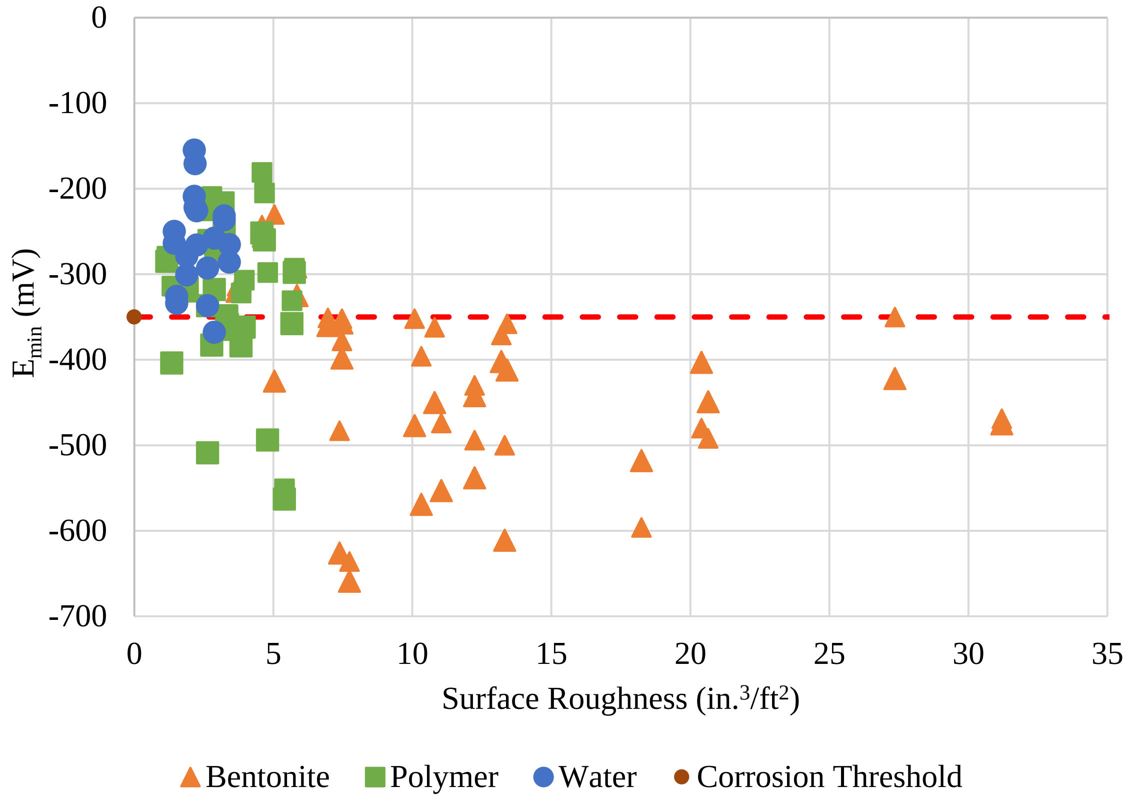


Figure 12.16 E_{50} versus surface roughness

The data in Figure 12.16 paints an even more startling picture, but one with patterns repeated through the entirety of testing. A high surface roughness value (over 5 in.^{3}/sf) indicates a strong probability of corrosion, and therefore a compromised cover region. As predicted, the cover region in bentonite-cast shafts are universally compromised and the surface is rough, where water (or fully-cased) shafts have a smooth surface and show virtually no signs of a compromised cover region. Polymer-cast shafts have a surface nearly as smooth as water but exhibit unpredictable quality traits. This was initially attributed to the lack of consistency between suppliers, but when the data points are separated by manufacturer, the individual sets remain inconsistent; meaning a conservative design would consider the cover region compromised for all support-fluid-cast shafts.

Field inspection and subsequent surface roughness evaluation showed the bentonite cast shafts at Caryville and Blountstown could be expected to be actively corroding. Figure 12.17 shows the results of the field evaluations superimposed onto the lab-cast specimens data.

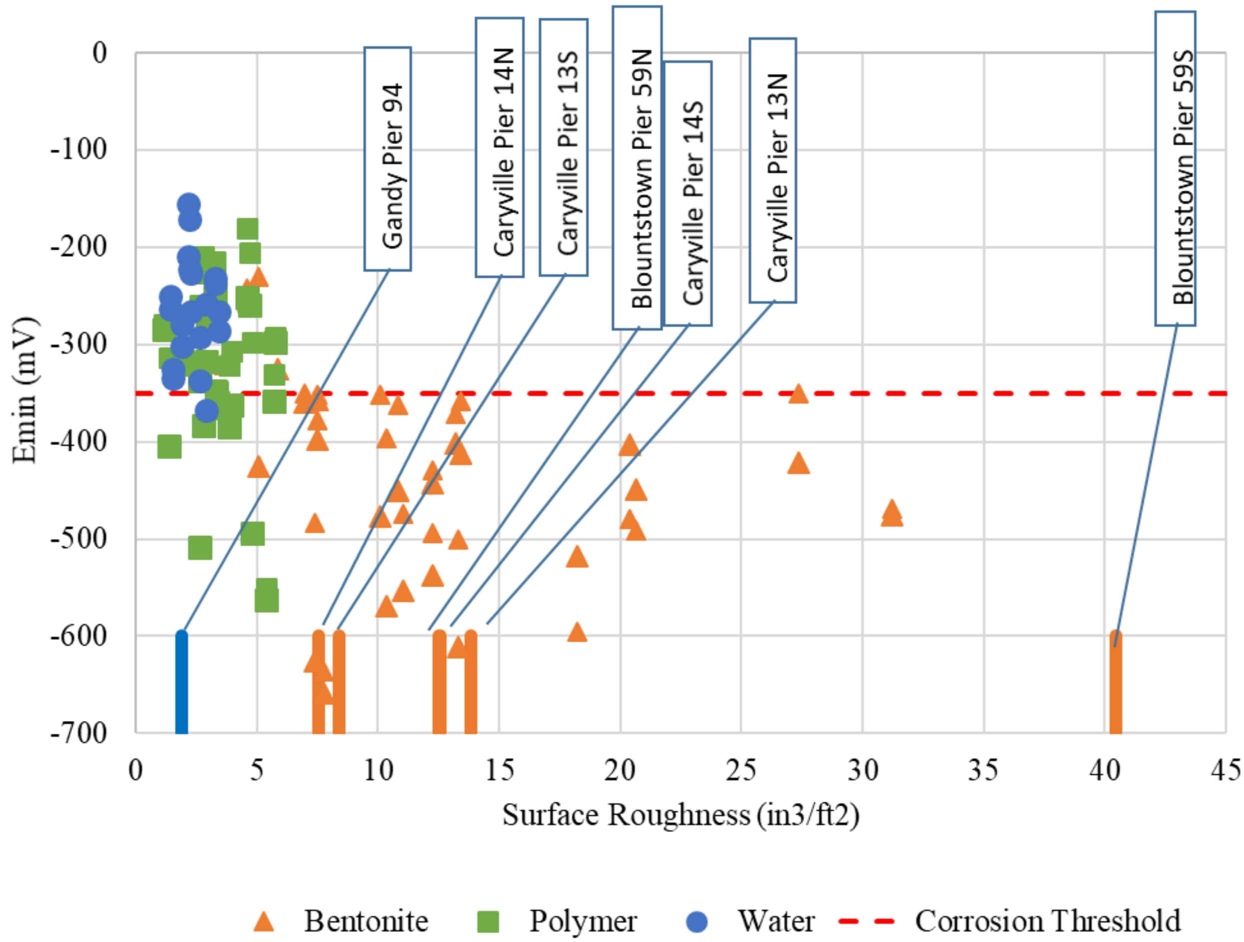


Figure 12.17 All field data showed bentonite surface roughness to exceed surface roughness criterion; water cast shafts were well below and again similar to lab cast specimens.

12.5 Chapter Summary

Through a series of experimental programs, the effects of support fluid on rebar bond, corrosion potential, concrete integrity, and surface degradation are discussed. The overarching consensus being that the presence of support fluid of any type during the concreting process leads to a compromised final product.

Chapter Thirteen: Conclusions and Recommendations

Slurry support fluid is a vitally important component of the wet shaft construction process and bentonite is the most reliable support fluid for maintaining sidewall stability. Unfortunately, this study showed bentonite slurry to cause the greatest number of problems. This research examined the methods and materials used in tremie-placed, slurry displacing concreting during drilled shaft construction. A total of 58 laboratory drilled shaft specimens were constructed using bentonite, polymer, or natural slurry to identify correlations between slurry type and shaft integrity. The overarching conclusion being any shaft cast using bentonite or polymer slurry has a high probability of being compromised either in rebar bond or corrosion resistance. Visual inspection of 30 shafts from four bridges cast in water, bentonite, and attapulgite slurry confirmed the laboratory findings. No polymer shafts were evaluated in the field as no over-water bridges are presently in the FDOT inventory; polymer slurry has only recently been approved for bridge structures.

To address the findings of this study the following observations and recommendations are provided:

(1) Use full-length temporary casing where possible for overwater bridges exposed to harsh environments (like Gandy Bridge). A permanent upper casing would still be used.

(2) For overwater construction in less harsh conditions and where permanent casing is already left in place, there is the potential to perform normal tremie placement of concrete (support-fluid-displaced) up to and within the casing portion, and then pump down and remove the remaining support fluid prior to completing the last portion of the concreting process. This would remove the potential for anomaly formation in the high moment regions just below the cap and increase corrosion resistance where it is most needed from the splash zone or more highly oxygenated portions of the shaft. Concrete head pressure at the bottom of the casing, when pumped down, must equal or exceed the hydrostatic water table pressure.

(3) Permanent casing left on bridges is not assumed to contribute to the strength, but does provide some additional capacity. However, it does not stop corrosion from occurring and hinders the ability for biennial inspections to assess the actual condition. Further, present inspection records are poorly archived and not database-searchable for details concerning the foundation element types, etc.

(4) Support fluid is often cleaned or exchanged prior to concreting. For bentonite fluids, the process is time-consuming and therefore is often exchanged rather than insitu recirculation and cleaning (desanding). It is conceivable to exchange bentonite with polymer fluids just prior to concreting such that the bentonite would have done the work of stabilizing the excavation but would not be needed at the time of concreting, where the polymer fluid could suffice. While polymer has shown to cause some problems, the severity would be lessened.

(5) The testing of rebar pullout / development length in this study was all conducted on specimens cast in clean slurry with no suspended soil cuttings. Unfortunately, this is the best case and not necessarily representative of actual field conditions. Therefore, it is recommended that the worst case development length multiplier of 2.0 be used for all polymer or mineral support fluid-displaced concreting applications. This is necessary to provide the same level of reliability commonly accepted for dry applications. The most common application for this multiplier will be stirrup overlap lengths. For main bars,

mechanical slices would be preferred to minimize cage congestion and the associated concrete flow restrictions.

(6) Polymer support fluid products are currently being approved on the basis of comparison to bentonite, where they must perform equal to or better than bentonite. This is a flawed criterion for comparison. If polymer is to be approved, the process should use water (natural support fluid) as a comparator and a minimum of one year of electrochemical evaluation should be taken into account to serve as a validation of an intact cover region. Corrosion resistance is presently not considered in the approval of new support fluid products.

(7) There is a large inventory of bridges like those investigated that are most likely in the same state of disrepair.

(8) Surface roughness and mattressing documented in the field investigations warrant further evaluation using electrochemical/corrosion mapping techniques.

13.1 Recommended FDOT Specification Changes Based on this Research

Presently, bentonite, attapulgite and polymer slurry are permitted for use for bridge foundation elements primarily selected on the basis of contractor experience, soil type, and/or the groundwater quality (fresh or salt). The study findings suggest bentonite slurry is unacceptable for all applications subjected to harsh environments (chlorides) and may also lead to poor structural performance when considering rebar development lengths. This is a drastic departure from previously accepted construction practices but which should be reevaluated when addressing newer specifications. Water displaced concreting showed no adverse effects; polymer displaced concreting had mixed performance where not all polymer products performed to the same level.

Past approval of polymer slurry products has hinged on results from soil/concrete bond, concrete/rebar bond, and demonstrating no adverse effects on concrete per the 455 specifications. The basis for comparison was whether a shaft cast with a given product performed equal to or better than bentonite (the preferred slurry product prior to this study). The study findings suggest polymer products should not be compared to bentonite but rather water or dry casting conditions; therefore, present acceptance of polymer products should be revisited to ensure truly adequate performance. With this in mind, only one of the three presently approved products performed to the level of water casting conditions.

13.2 Suggestions for Future Research

In the eighteen months prior to the completion of this study two large bridges collapsed elsewhere in the world with ages of 20 and 40 years; initial evaluations indicate durability issues. Durability of concrete structures now receives attention, but perhaps not all it requires. In this study, corrosion was active in 86% of all bentonite, lab-cast shafts and 100% when surface roughness was greater than 5 in.^{3}/ft^2. Of the bentonite cast shafts inspected in the field, all showed telltale signs of corrosion susceptibility with surface roughness greater than the 5 in.^{3}/ft^2 lab-determined threshold. It is strongly suggested that future studies evaluate the FDOT bridge inventory for all bridges founded on drilled shafts that were constructed in this manner. Further, those bridges identified in this study with high potential for active corrosion should be more rigorously evaluated using surface potential or similarly indicative electrochemical means.

The existing 59 test shafts (in part cast for this study) continue to mature and represent specimens between 1.5 and 7.5 years of age at the time of this reporting. The latest data indicated the best of the polymer specimens were also beginning to show trends toward active corrosion and yet, none are in harsh environmental conditions. These specimens as well as presently constructed polymer shafts should be evaluated to determine if these too are prone to poor corrosion protection.

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APPENDIX A. CONCRETE POUR DETAILS

Table A.1 Pour one summary

Pour Number	1
Pour Date	2/20/2013
Mix	4KDS
Average Concrete Compressive Strength (psi)	6150
Shafts Poured	1,2

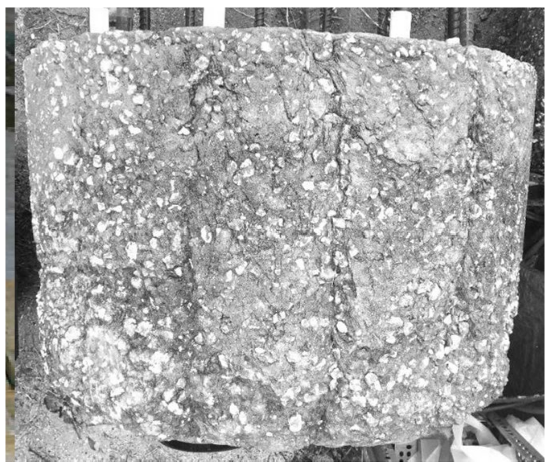


Figure A.1 Shaft 1

Figure A.2 Shaft 2

Table A.2 Pour two summary.

Pour Number	2
Pour Date	5/8/2013
Mix	4KDS
Average Concrete Compressive Strength (psi)	4358
Shafts Poured	3-6

Figure A.3 Shaft 3

Figure A.4 Shaft 4

Figure A.5 Shaft 5

Figure A.6 Shaft 6

Table A.3 Pour three summary.

Pour Number	3
Pour Date	5/8/2013
Mix	4KDS
Average Concrete Compressive Strength (psi)	4358
Shafts Poured	7-12

Figure A.7 Shaft 7

Figure A.8 Shaft 8

Figure A.9 Shaft 9

Figure A.10 Shaft 10

Figure A.11 Shaft 11

Figure A.12 Shaft 12

Table A.4 Pour four summary.

Pour Number	4
Pour Date	6/18/2013
Mix	4KDS
Average Concrete Compressive Strength (psi)	4358
Shafts Poured	13-18

Figure A.13 Shaft 13

Figure A.14 Shaft 14

Figure A.15 Shaft 15

Figure A.16 Shaft 16

Figure A.17 Shaft 17

Figure A.18 Shaft 18

Table A.5 Pour five summary.

Pour Number	5
Pour Date	9/20/2013
Mix	4KDS
Average Concrete Compressive Strength (psi)	4753
Shafts Poured	19-22

Figure A.19 Shaft 19

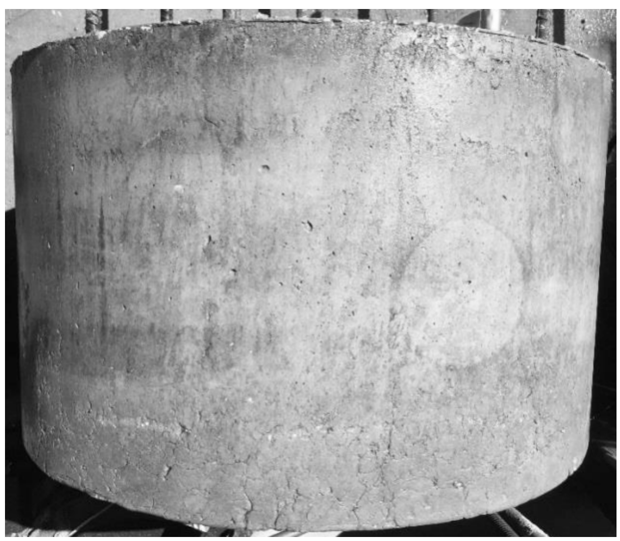


Figure A.20 Shaft 20.

Figure A.21 Shaft 21

Figure A.22 Shaft 22

Table A.6 Pour six summary.

Pour Number	6
Pour Date	5/3/2015
Mix	SCC
Average Concrete Compressive Strength (psi)	4100
Shafts Poured	23,24

Figure A.23 Shaft 23

Figure A.24 Shaft 24

Table A.7 Pour seven summary.

Pour Number	7
Pour Date	7/25/17
Mix	4KDS
Average Concrete Compressive Strength (psi)	5210
Shafts Poured	25-30

Figure A.25 Shaft 25

Figure A.26 Shaft 26

Figure A.27 Shaft 27

Figure A.28 Shaft 28

Figure A.29 Shaft 29

Figure A.30 Shaft 30

Table A.8 Pour eight summary.

Pour Number	8
Pour Date	10/6/2017
Mix	4KDS
Average Concrete Compressive Strength (psi)	6150
Shafts Poured	31-36

Figure A.31 Shaft 31

Figure A.32 Shaft 32

Figure A.33 Shaft 33

Figure A.34 Shaft 34

Figure A.35 Shaft 35

Figure A.36 Shaft 36

Table A.9 Pour nine summary.

Pour Number	9
Pour Date	12/6/2017
Mix	4KDS
Average Concrete Compressive Strength (psi)	N/A
Shafts Poured	37-42

Figure A.37 Shaft 37

Figure A.38 Shaft 38

Figure A.39 Shaft 39

Figure A.40 Shaft 40

Figure A.41 Shaft 41

Figure A.42 Shaft 42

Table A.10 Pour ten summary.

Pour Number	10
Pour Date	2/7/2018
Mix	4KDS
Average Concrete Compressive Strength (psi)	N/A
Shafts Poured	43-46

Figure A.43 Shaft 43

Figure A.44 Shaft 45

Figure A.45 Shaft 46

Table A.11 Pour eleven summary.

Pour Number	11
Pour Date	2/21/2018
Mix	4KDS
Average Concrete Compressive Strength (psi)	9128
Shafts Poured	47-52

Figure A.46 Shaft 47

Figure A.47 Shaft 48

Figure A.48 Shaft 49

Figure A.49 Shaft 50

Figure A.50 Shaft 51

Figure A.51 Shaft 52

Table A.12 Pour twelve summary.

Pour Number	12
Pour Date	9/25/2018
Mix	4KDS
Average Concrete Compressive Strength (psi)	5940
Shafts Poured	53-58

Figure A.52 Shaft 53

Figure A.53 Shaft 54

Figure A.54 Shaft 55

Figure A.55 Shaft 56

Figure A.56 Shaft 57

Figure A.57 Shaft 58

APPENDIX B: RHEOLOGICAL MODELING

The flow patterns were obtained from the parametric study performed for γ, with values 0.5, 0.1, 0.05, and 0.01 m/sec. From this study a γ value of 0.05 m /sec was selected. For this γ value of 0.05 m /sec, the flow pattern was better than the other γ values. Other values of γ led to generation of unphysical pockets of slurry within the concrete.

For σ, flow patterns were studied for the values 0.05, 0.1 and 0.50 N/m and a value of 0.1 N/m was selected which showed a realistic flow behavior of the concrete in the excavation compared to other values. For other values, the flow patterns along the excavation wall and tremie surface were showing some inclination from the anticipated vertical direction.

For ε, flow patterns were studied for the values of 0.005, 0.007, 0.010 and 0.015m. There was not much appreciable difference in the flow patterns obtained from these values and based on close observation, a value of 0.007m was considered.

The flow patterns obtained from the parametric studies for γ, σ, and ε, are provided herein.

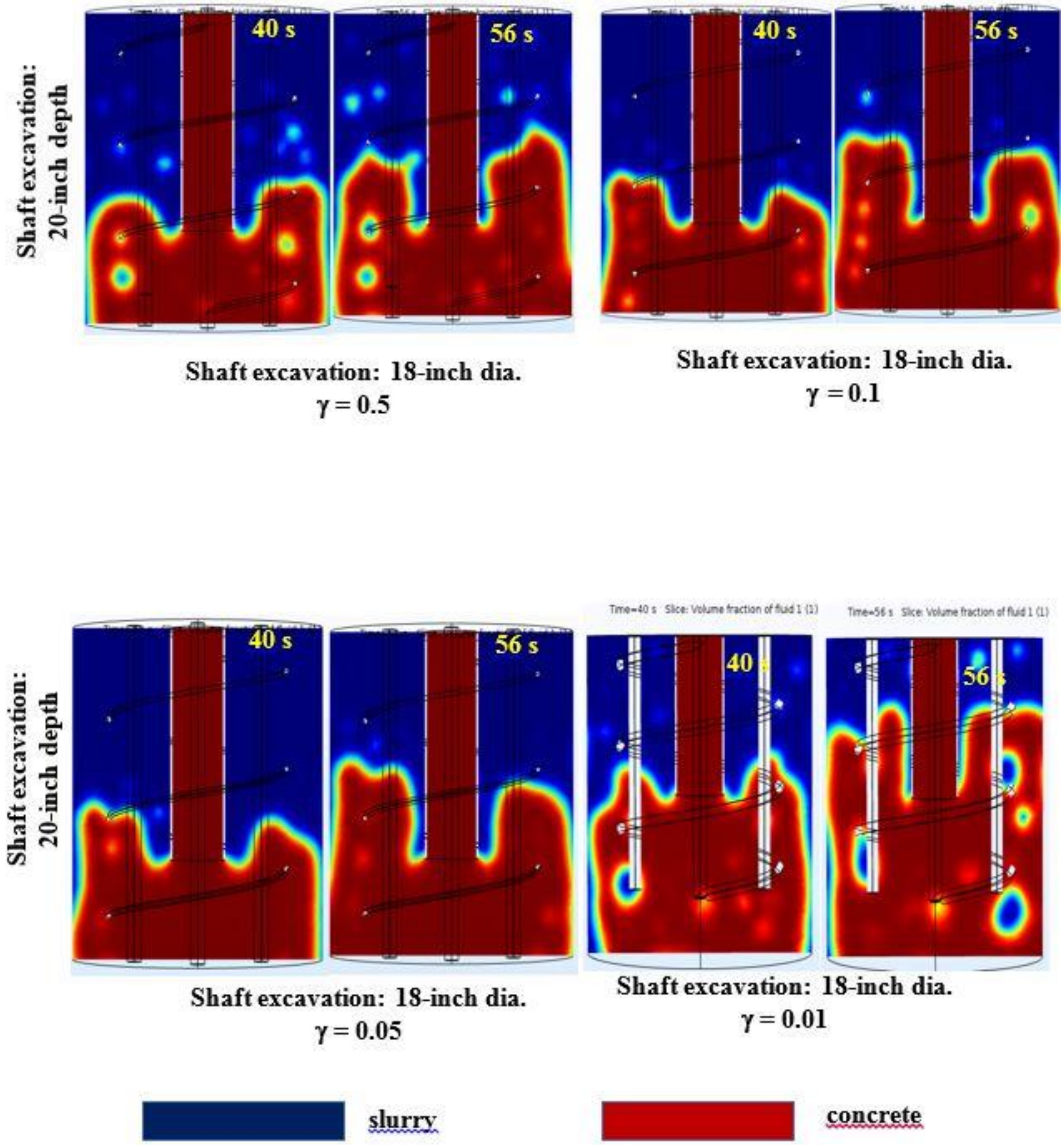


Figure B.1 3-D Model in COMSOL: Parametric Study of Re-initialization Intensity Parameter γ– Flow Pattern of Concrete in 18-inch Dia. 20-inch Depth Shaft Model

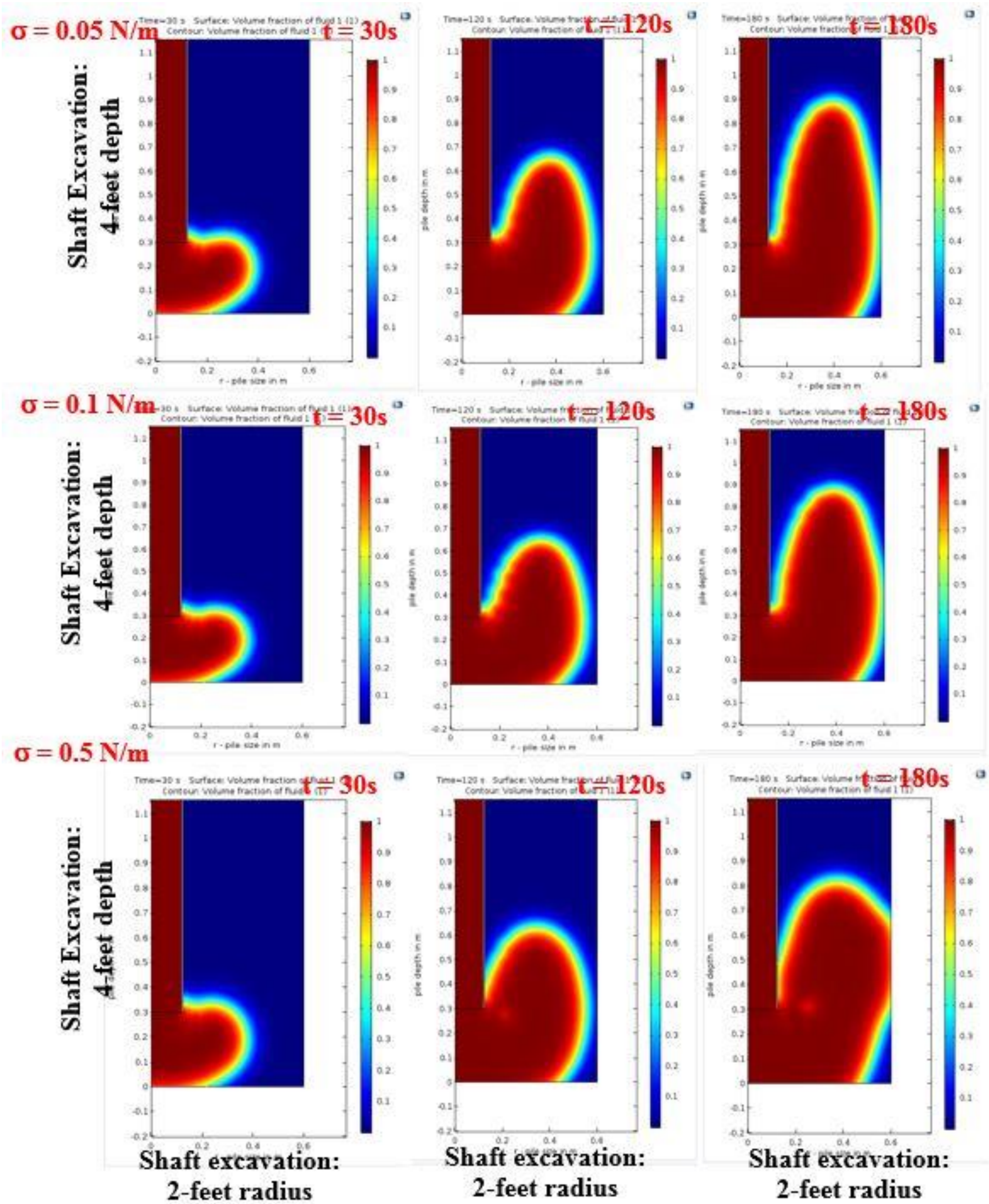


Figure B.2 2-D Model in COMSOL: Parametric Study on σ, Surface Tension-Flow Pattern of Concrete in 2-feet Radius and 4-feet Depth Shaft Model

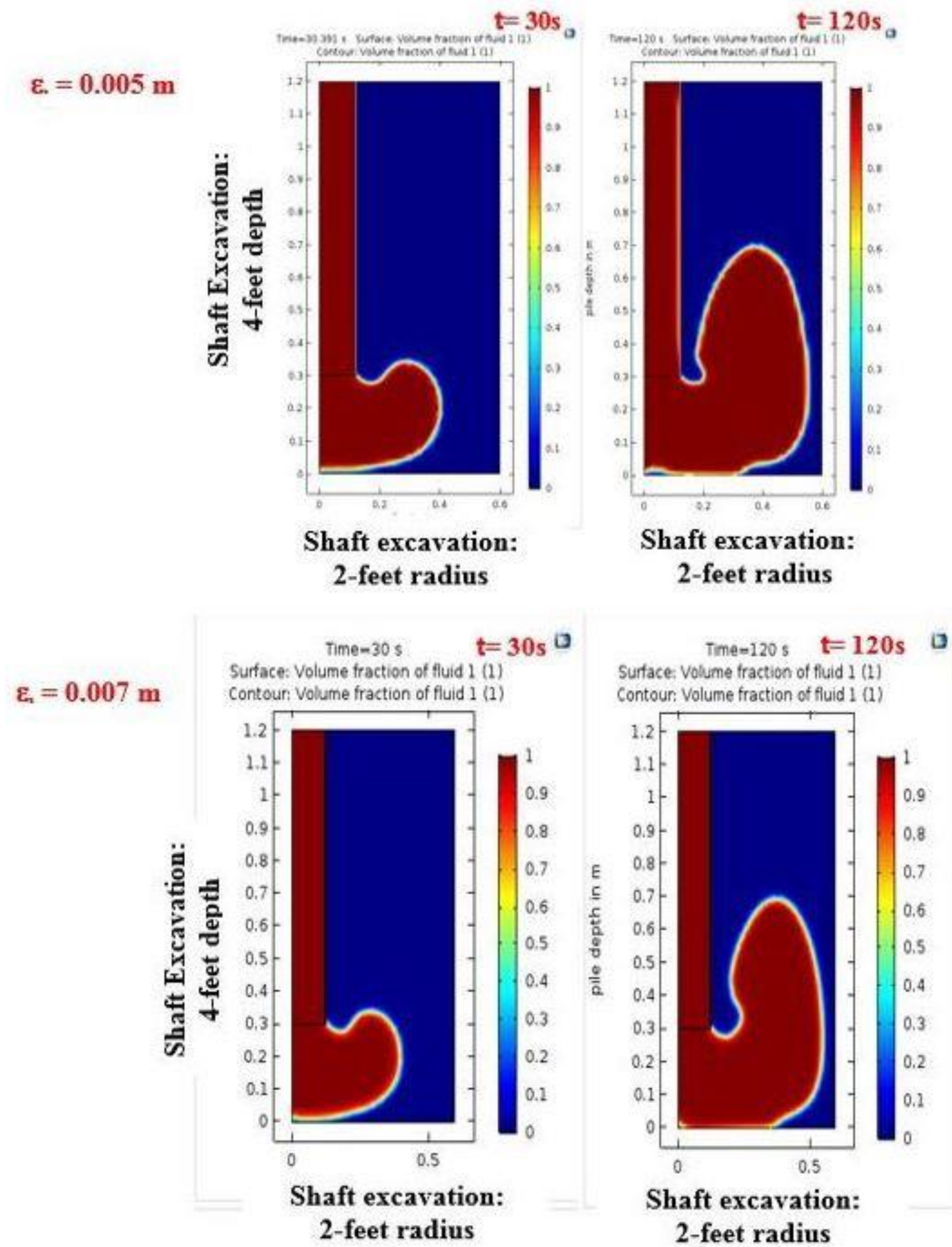


Figure B.3a 2-D Model in COMSOL: Parametric Study on ε, Interface Thickness Parameter-Flow Pattern of Concrete in 2-feet Radius and 4-feet Depth Shaft Model,
ε = 0.005 m   ε = 0.007 m

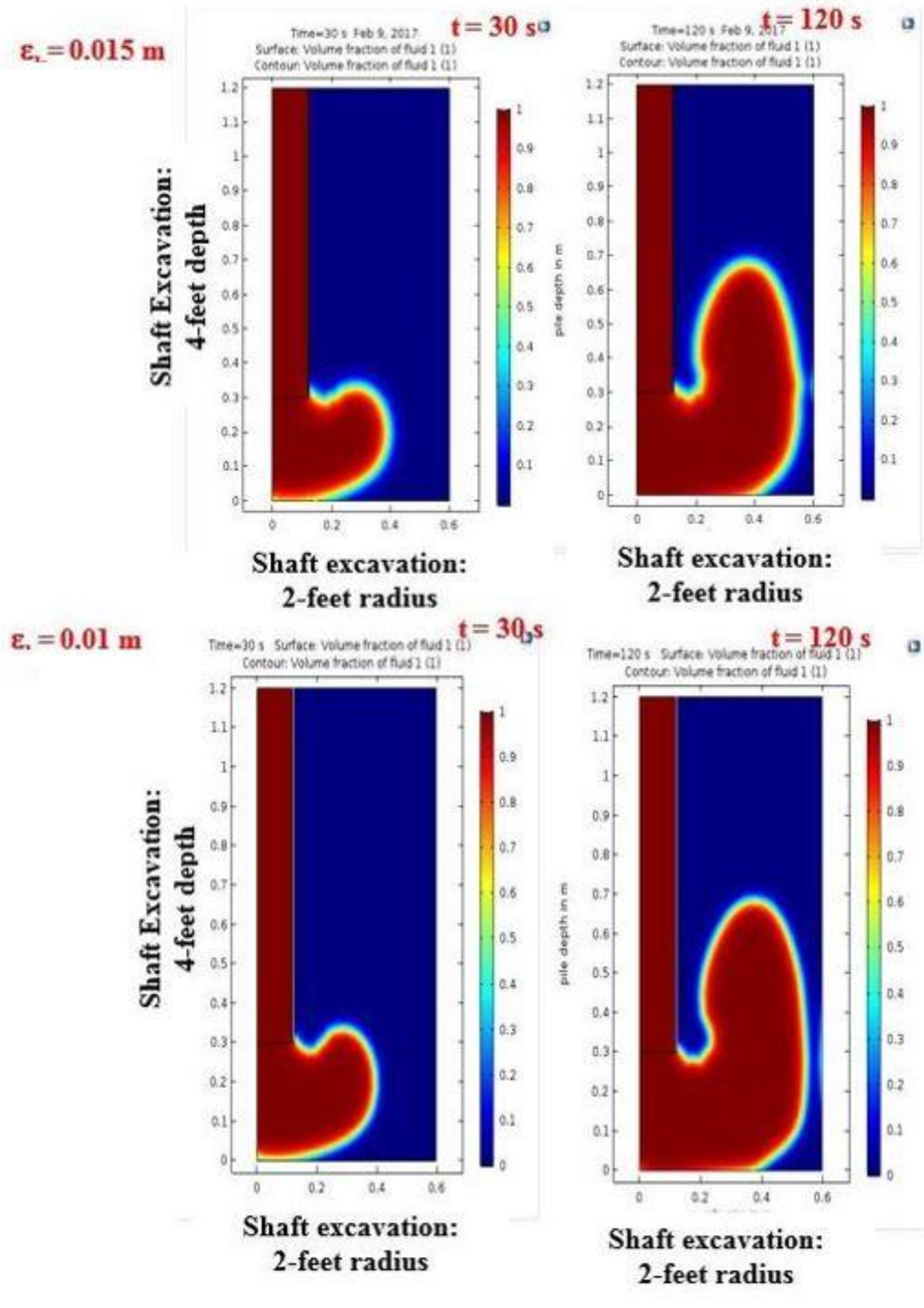


Figure B.3b 2-D Model in COMSOL: Parametric Study on ε, Interface Thickness Parameter-Flow Pattern of Concrete in 2-feet Radius and 4-feet Depth Shaft Model,
$\varepsilon = 0.015$ m $\varepsilon = 0.010$ m

APPENDIX C: MONTE CARLO

An example calculation of the Monte Carlo analysis performed is displayed here. Using the calculations for water (ACI 318-14 prediction, Table C.1) the example is as follows.

Table C.1 Mean bias, coefficient of variation (CoV), and calculated resistance factor for example.

Water	
Mean Bias ($\bar{r}$)	1.21
CoV (V_r)	0.25
Calculated ∅	0.61

$$\bar{Q} = 1, input\ parameter$$

$$\sigma_Q = \bar{Q} * V_Q = 1 * 0.102 = 0.102$$

$$\bar{R} = \frac{\overline{r(LF * Q)}}{\emptyset} = \frac{1.21(1.33 * 1)}{0.61} = 2.64$$

$$\sigma_R = \bar{R} * V_r = 2.64 * 0.25 = 0.66$$

The mean load and resistance along with the respective standard deviations are then converted to lognormal.

Example calculation for load, Q:

$$\sigma_{lnQ}^2 = ln\left[1 + \left(\frac{\sigma_Q}{\mu_Q}\right)^2\right] = \ln\left[1 + \left(\frac{0.102}{1}\right)^2\right] = 0.10$$

$$\mu_{lnQ} = ln\mu_Q - \frac{1}{2}\sigma_{lnQ}^2 = ln(1) - \frac{1}{2}(0.10)^2 = -0.005$$

Example calculation for resistance, R:

$$\sigma_{lnR}^2 = ln\left[1+\left(\frac{\sigma_R}{\mu_R}\right)^2\right] = \ln\left[1+\left(\frac{0.66}{2.64}\right)^2\right] = 0.25$$

$$\mu_{lnQ} = ln\mu_R - \frac{1}{2}\sigma_{lnR}^2 = ln(2.64) - \frac{1}{2}(0.25)^2 = 0.94$$

Tables C.2 and C.3 depict the inputs used for Monte Carlo simulations.

Table C.2 Example of spreadsheet for Monte Carlo.

Simulation	X	Y	Log-normal Load	Log-normal Resistance
1	=norminv (rand(),0,1)	=norminv (rand(),0,1)	=μ_{lnQ}+(σ_{lnQ}*X)	=μ_{lnR}+(σ_{lnR}*Y)

Table C.3 Continuation of Monte Carlo spreadsheet, connecting to Table C.2.

Normal Load (Q)	Normal Resistance (R)	Failures
=EXP(μ_{lnQ}+(σ_{lnQ}*X))	=EXP(μ_{lnR}+(σ_{lnR}*Y))	=IF(Q>R,1,0)

Table C.4 displays the generation of the failure ratio.

Table C.4 Determining failure ratio in Microsoft Excel.

Total Failures	A	Failure Ratio		
=sum(failures)	=sum(failures)/1000000	1 in	=1/A	failures

APPENDIX D – SAMPLE DATA COLLECTION SHEET

Shaft No: ______________

Date: ______________

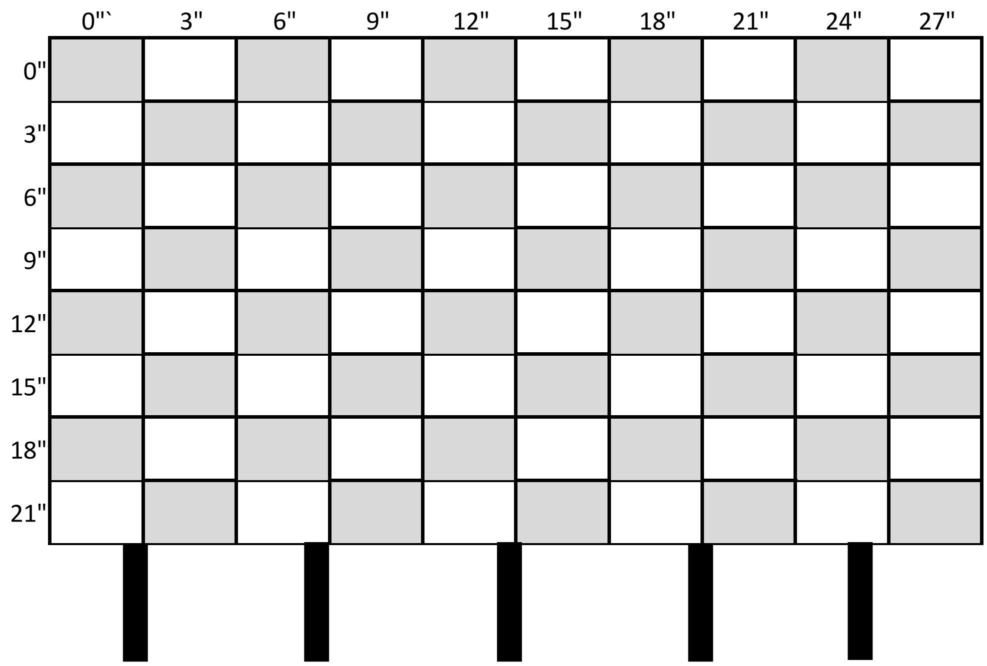


Figure D.1 Sample half-cell potential data collection sheet.

APPENDIX E – HALF-CELL POTENTIAL SUMMARY SHEETS

Table E.1 Shaft 1, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-266	-289	-333	-337	-313	-298	-296	-289
3"	-279	-292	-316	-317	-311	-301	-300	-293
6"	-290	-302	-312	-312	-308	-306	-307	-303
9"	-290	-308	-315	-312	-309	-310	-308	-307
12"	-307	-316	-315	-318	-317	-314	-311	-302
15"	-309	-317	-319	-325	-324	-322	-320	-311
18"	-314	-323	-324	-332	-330	-328	-327	-320
21"	-323	-330	-330	-338	-338	-342	-331	-322
24"	-327	-330	-338	-344	-345	-348	-337	-329
27"	-328	-333	-338	-344	-347	-350	-338	-337

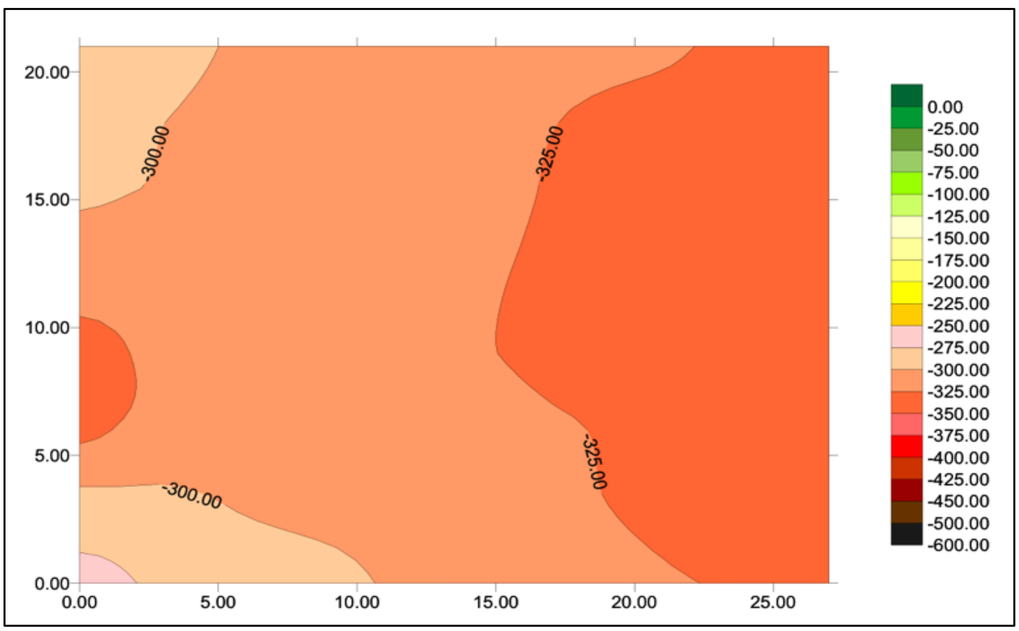


Figure E.1 Shaft 1, test 1 data map

Table E.2 Shaft 1, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-395	-404	-401	-396	-390	-388	-383	-383
3"	-404	-420	-417	-399	-391	-388	-383	-384
6"	-410	-422	-417	-404	-394	-390	-385	-382
9"	-408	-412	-407	-397	-390	-386	-381	-375
12"	-403	-406	-399	-392	-387	-380	-376	-374
15"	-406	-406	-400	-391	-386	-377	-369	-365
18"	-405	-402	-394	-385	-378	-370	-368	-364
21"	-409	-399	-391	-379	-374	-367	-364	-363
24"	-416	-401	-390	-380	-370	-366	-364	-362
27"	-412	-401	-391	-382	-368	-364	-360	-360

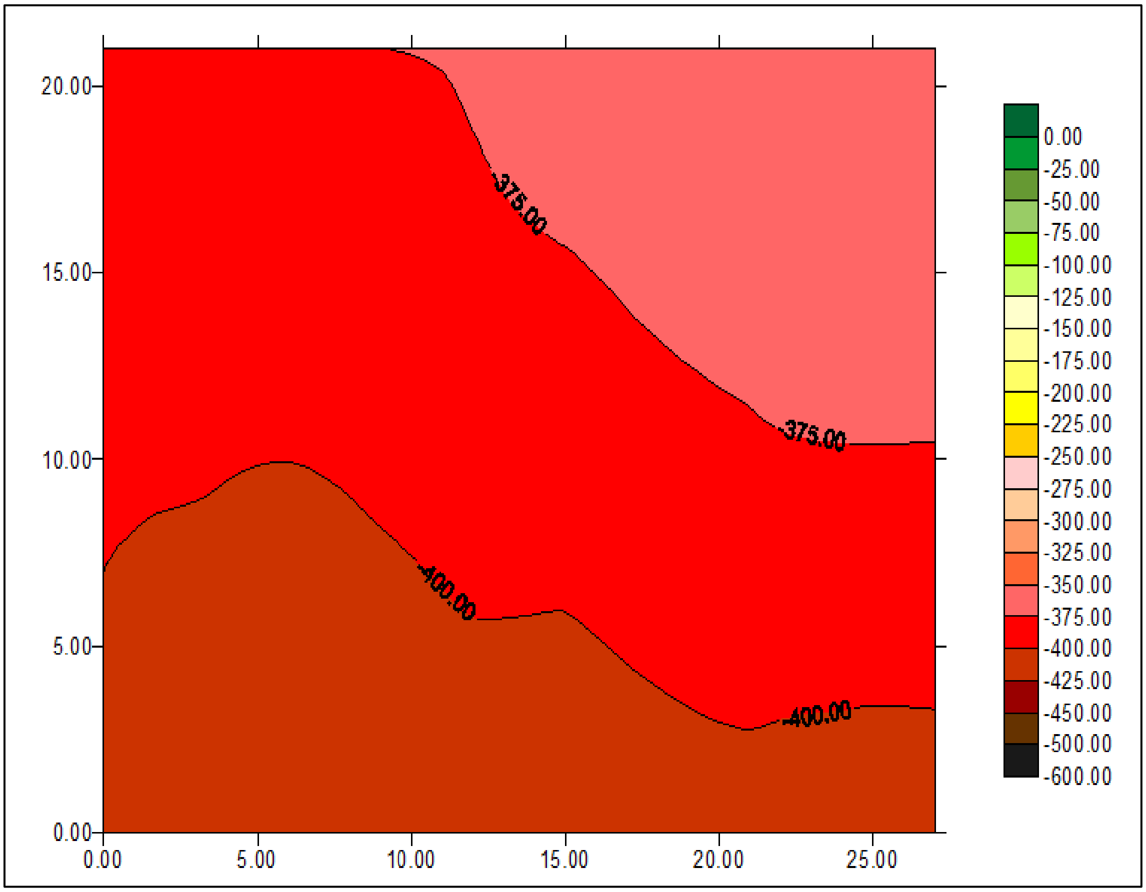


Figure E.2 Shaft 1, test 2 data map

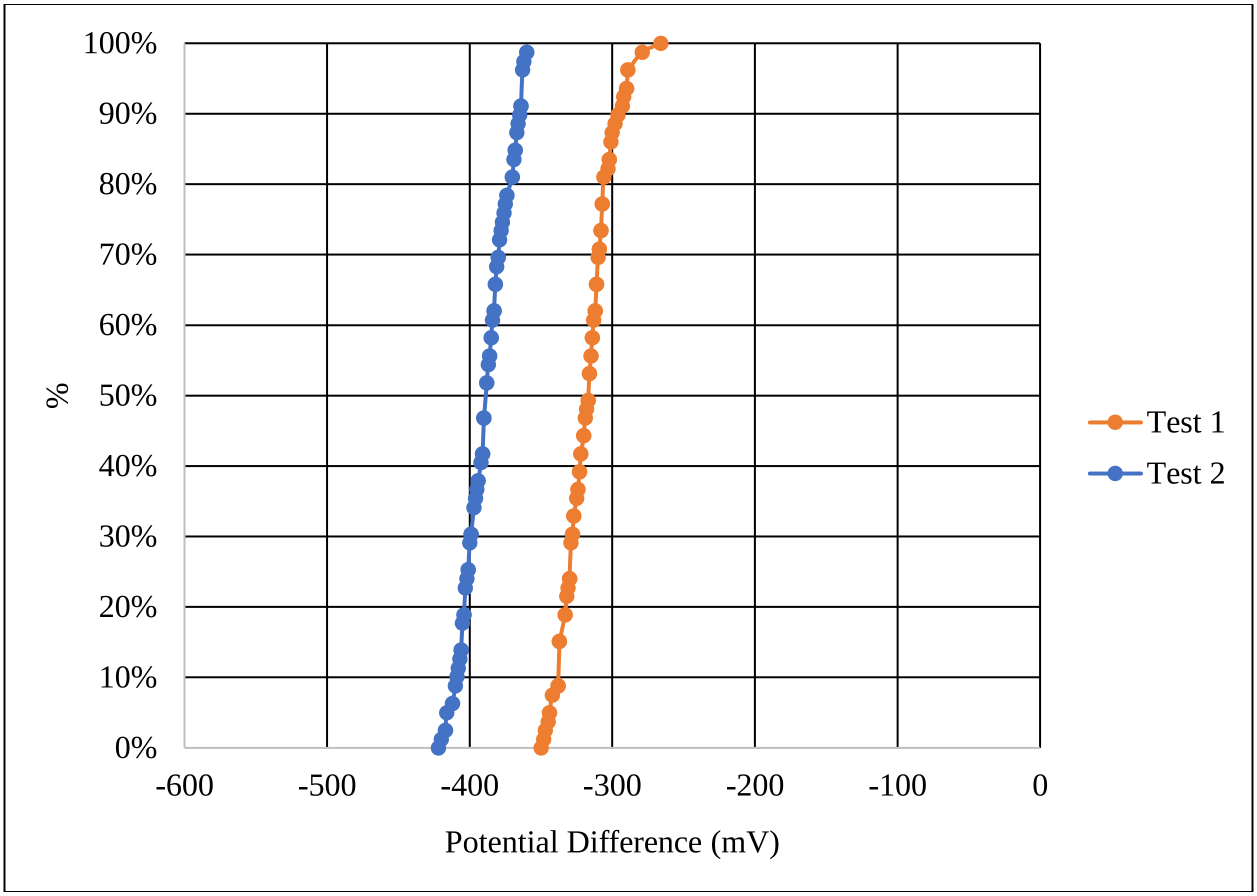


Figure E.3 Shaft 1, percentile distributions

Table E.3 Shaft 2, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-384	-387	-406	-421	-434	-438	-431	-411
3"	-402	-398	-416	-431	-454	-456	-446	-434
6"	-400	-403	-420	-442	-460	-464	-461	-454
9"	-392	-408	-430	-452	-466	-474	-472	-465
12"	-402	-417	-447	-463	-476	-470	-462	-459
15"	-402	-416	-445	-463	-474	-478	-463	-462
18"	-395	-410	-460	-468	-482	-477	-458	-449
21"	-396	-404	-466	-483	-492	-485	-469	-449
24"	-389	-400	-444	-470	-481	-472	-461	-440
27"	-390	-398	-428	-467	-472	-472	-461	-447

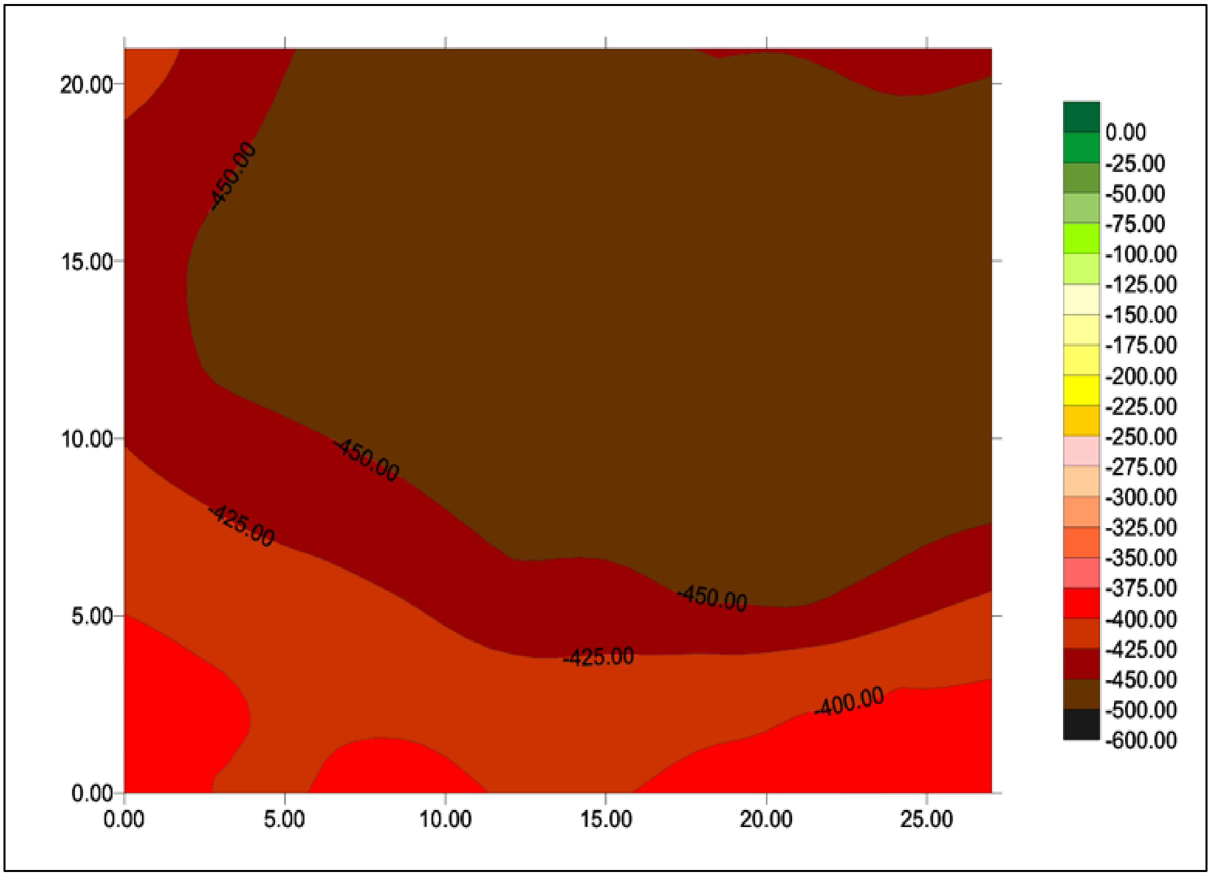


Figure E.4 Shaft 2, test 1 data map

Table E.4 Shaft 2, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-347	-364	-395	-419	-437	-449	-444	-438
3"	-345	-364	-392	-416	-436	-445	-437	-431
6"	-347	-357	-384	-402	-420	-430	-428	-425
9"	-347	-357	-381	-396	-421	-434	-428	-429
12"	-346	-355	-375	-390	-410	-422	-425	-419
15"	-347	-351	-362	-377	-395	-401	-400	-402
18"		-352	-362	-374	-386	-402	-402	-395
21"		-350	-362	-372	-381	-395	-393	-393
24"		-349	-358	-374	-375	-392	-385	-382
27"		-344	-356	-368	-374	-380	-377	-377

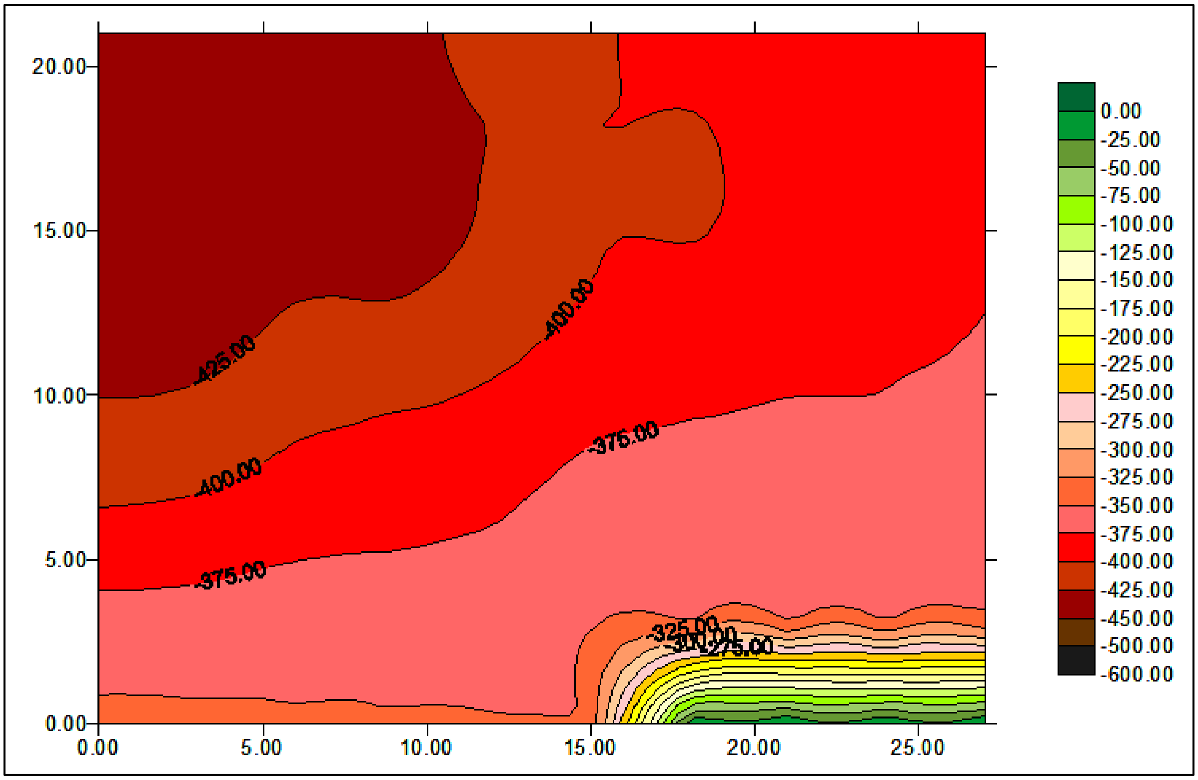


Figure E.5 Shaft 2, test 2 data map

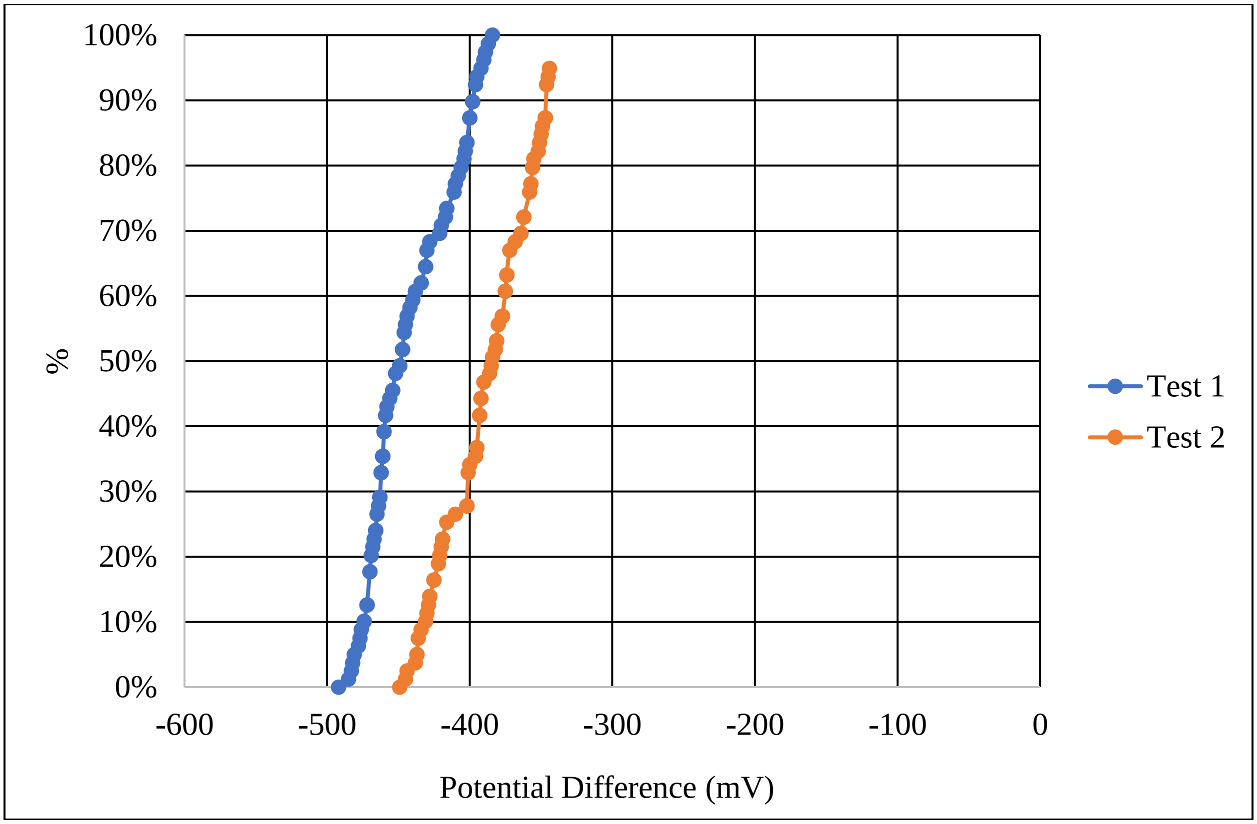


Figure E.6 Shaft 2 percentile distributions

Table E.5 Shaft 3, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-372	-382	-389	-390	-396	-396	-384	-379
3"	-373	-378	-381	-382	-393	-386	-381	-378
6"	-374	-374	-377	-377	-379	-380	-373	-379
9"	-376	-368	-372	-359	-361	-360	-363	-379
12"	-370	-364	-369	-352	-351	-353	-351	-369
15"	-374	-366	-360	-350	-349	-353	-346	-361
18"	-377	-360	-364	-352	-356	-355	-355	-354
21"	-379	-361	-380	-359	-364	-367	-370	-378
24"	-379	-367	-378	-368	-368	-380	-382	-383
27"	-373	-367	-374	-376	-376	-378	-381	-388

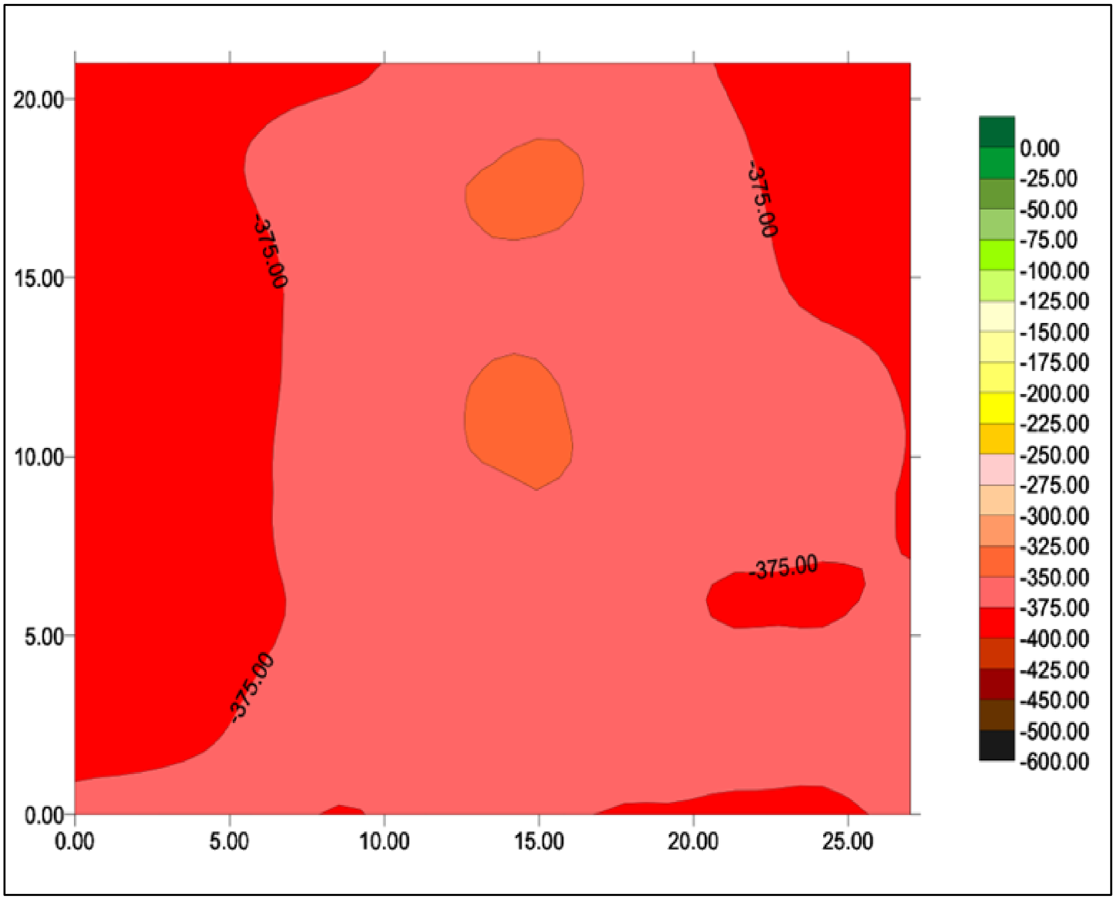


Figure E.7 Shaft 3, test 1 data map

Table E.6 Shaft 3, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-518	-529	-529	-542	-521	-510	-477	-488
3"	-554	-546	-545	-546	-515	-505	-468	-473
6"	-561	-540	-536	-542	-513	-505	-468	-461
9"	-569	-552	-535	-540	-497	-499	-456	-440
12"	-551	-545	-533	-530	-521	-514	-497	-450
15"	-538	-538	-527	-542	-517	-520	-471	-468
18"	-526	-529	-526	-521	-505	-488	-463	-461
21"	-528	-535	-530	-527	-496	-516	-472	-470
24"	-530	-551	-534	-544	-501	-514	-476	-484
27"	-519	-529	-519	-513	-488	-493	-473	-467

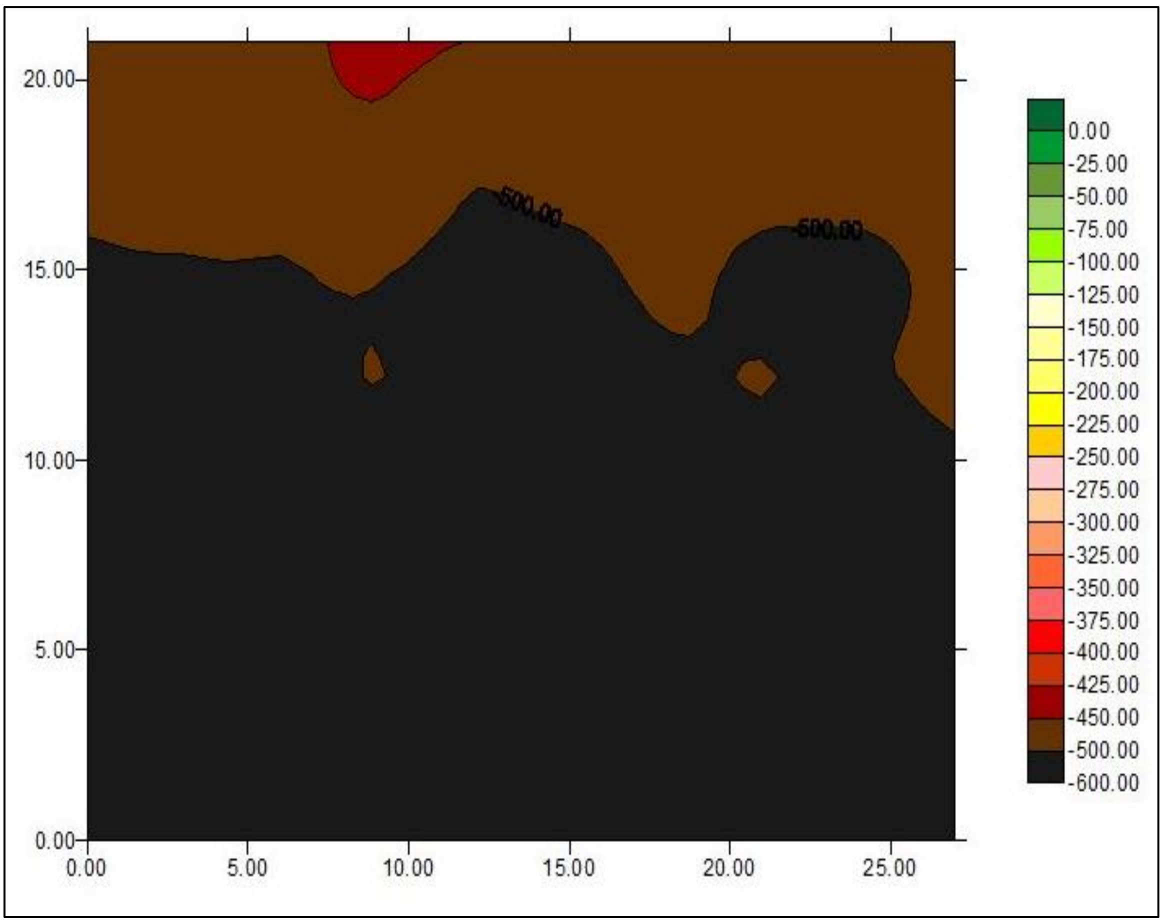


Figure E.8 Shaft 3, test 2 data map

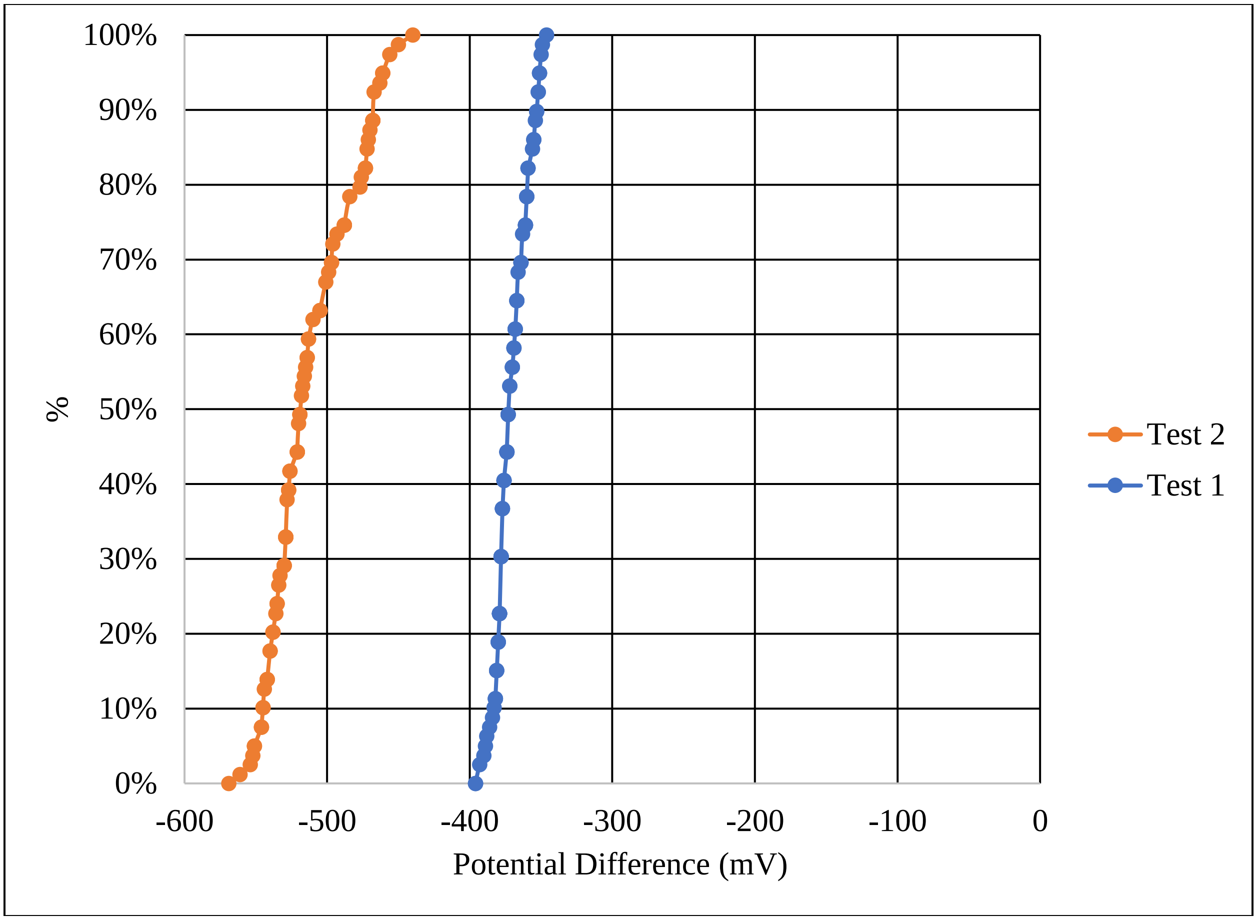


Figure E.9 Shaft 3 percentile distributions

Table E.7 Shaft 4, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-411	-441	-447	-462	-472	-454	-437	-425
3"		-443	-453	-463	-472	-453	-433	-423
6"	-442	-444	-447	-467	-473	-459	-434	-419
9"	-429	-434	-439	-463	-474	-458	-427	-415
12"		-427	-435	-458	-470	-458	-433	-409
15"		-429	-435	-454	-461	-450	-437	-414
18"	-425	-426	-444	-458	-473	-459	-445	-433
21"	-413	-425	-437	-470	-471	-461	-451	-439
24"	-420	-422	-430	-452	-466	-458	-449	-439
27"	-418	-417	-423	-449	-459	-463	-451	-443

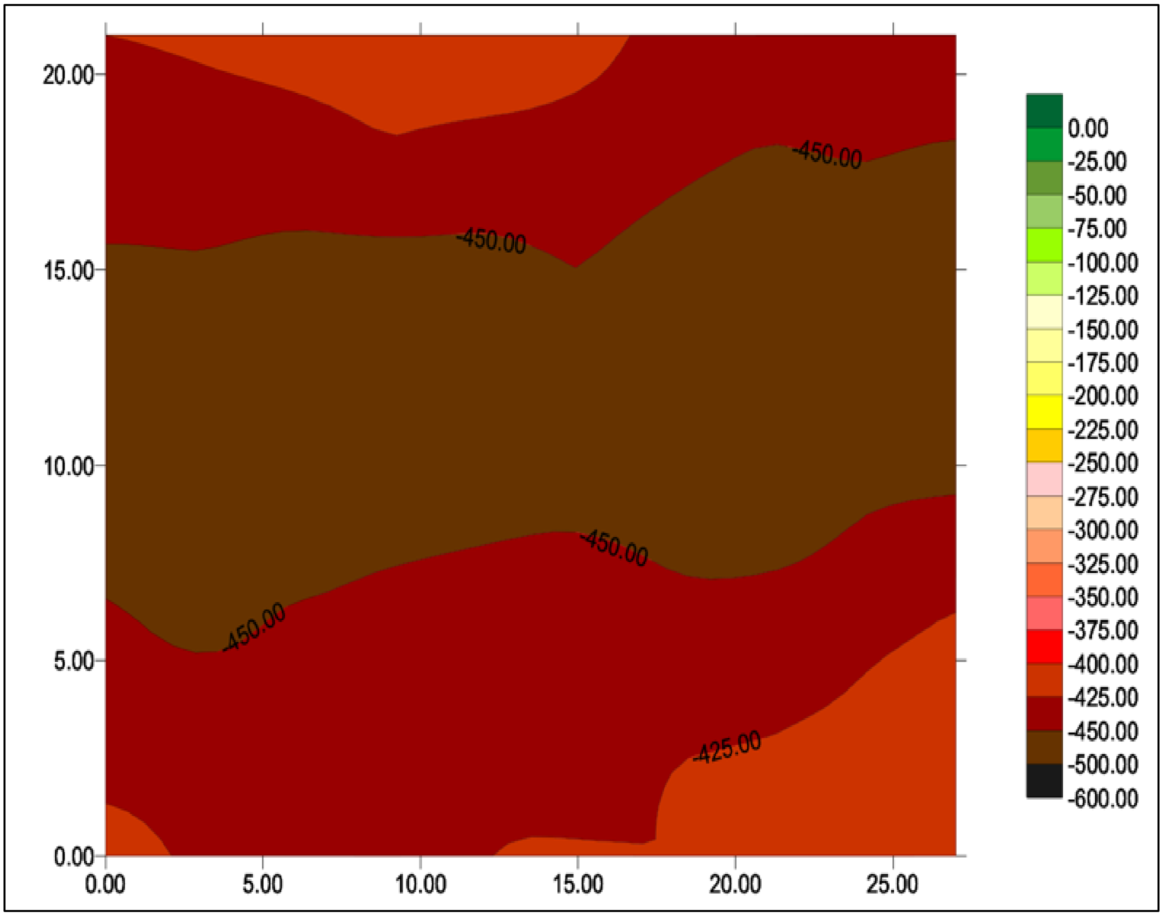


Figure E.10 Shaft 4, test 1 data map

Table E.8 Shaft 4, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-520	-526	-535	-547	-553	-545	-536	-528
3"	-513	-520	-531	-542	-547	-549	-539	-526
6"	-508	-517	-529	-541	-545	-538	-523	-512
9"	-496	-512	-533	-540	-535	-520	-494	-484
12"	-487	-507	-530	-533	-520	-510	-500	-478
15"	-497	-510	-524	-530	-515	-518	-520	-507
18"	-501	-508	-517	-522	-516	-526	-537	-523
21"	-505	-507	-512	-523	-528	-530	-523	-517
24"	-506	-508	-510	-527	-521	-512	-524	-543
27"	-504	-507	-508	-525	-525	-509	-507	-519

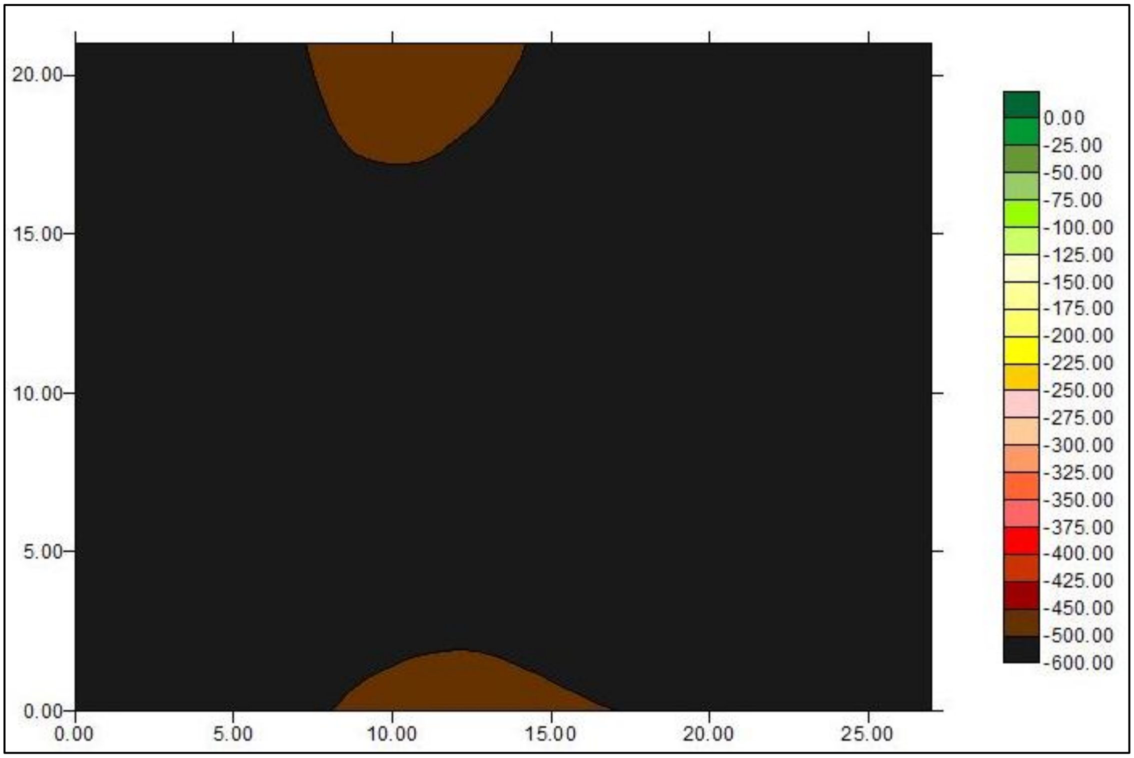


Figure E.11 Shaft 4, test 2 data map

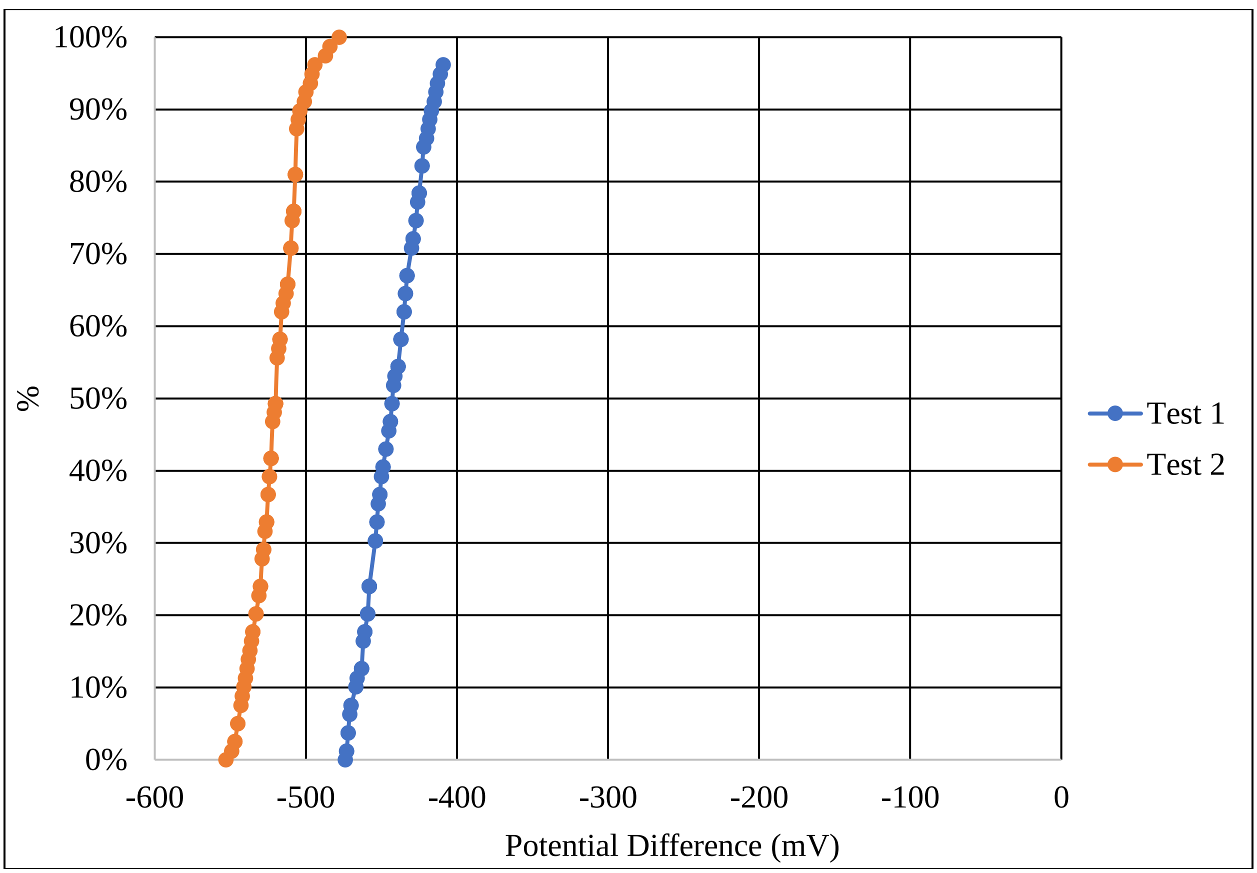


Figure E.12 Shaft 4 percentile distributions

Table E.9 Shaft 5, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-400	-408	-425	-447	-469	-482	-484	-475
3"	-408	-413	-428	-456	-476	-489	-494	-489
6"	-413	-416	-432	-460	-473	-482	-492	-485
9"	-413	-415	-433	-457	-466	-467	-470	-464
12"	-407	-402	-422	-450	-459	-465	-463	-455
15"	-390	-402	-416	-440	-441	-445	-459	-449
18"	-394	-402	-413	-433	-433	-435	-437	-426
21"	-392	-407	-416	-441	-448	-450	-457	-450
24"	-401	-416	-435	-452	-457	-469	-469	-469
27"	-412	-420	-432	-450	-460	-465	-469	-457

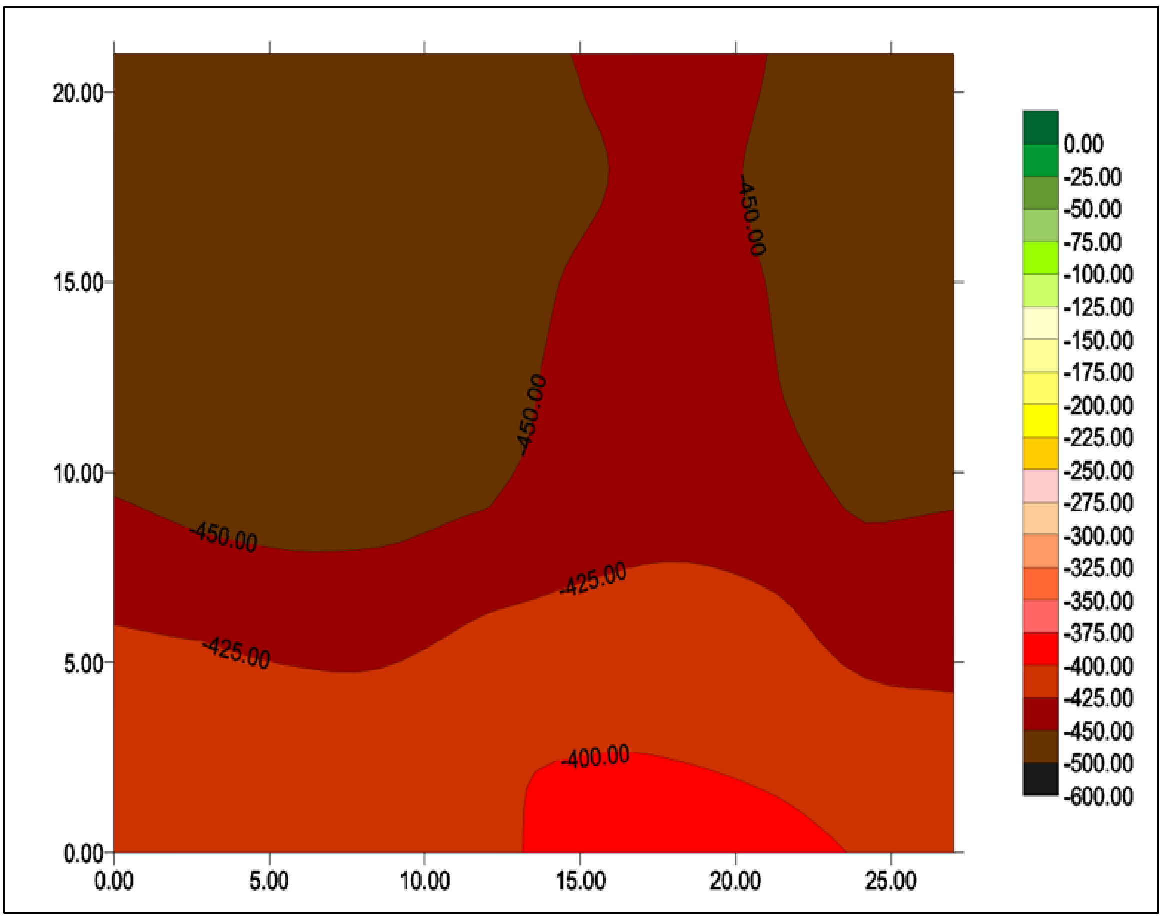


Figure E.13 Shaft 5, test 1 data map

Table E.10 Shaft 5, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-494	-498	-512	-523	-533	-538	-537	-528
3"	-492	-498	-508	-521	-532	-535	-537	-520
6"	-492	-495	-503	-515	-525	-529	-530	-505
9"	-488	-487	-497	-506	-517	-526	-512	-494
12"	-480	-483	-498	-504	-512	-521	-505	-496
15"	-471	-479	-486	-494	-496	-501	-494	-488
18"	-472	-473	-480	-493	-495	-516	-510	-488
21"	-475	-478	-488	-491	-499	-509	-527	-501
24"	-477	-476	-477	-484	-492	-507	-520	-511
27"	-483	-482	-476	-485	-491	-507	-522	-512

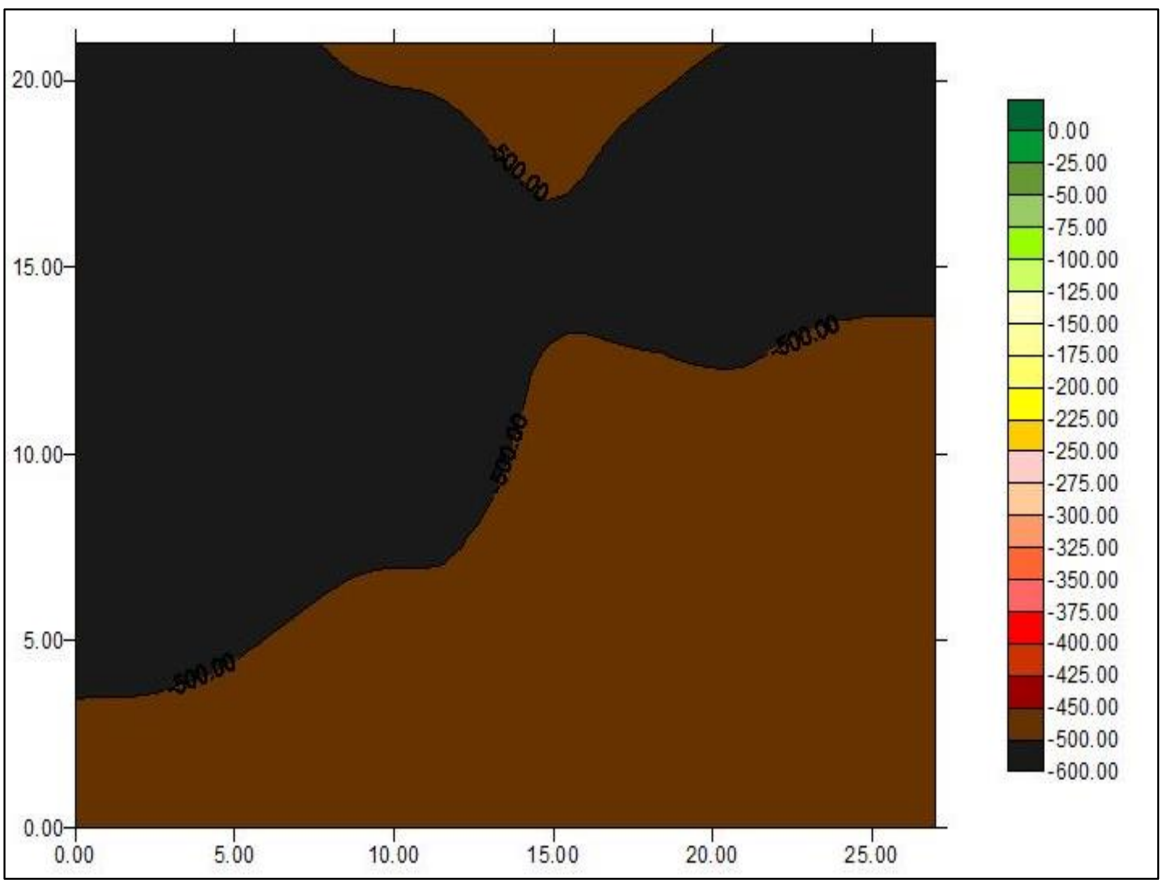


Figure E.14 Shaft 5, test 2 data map

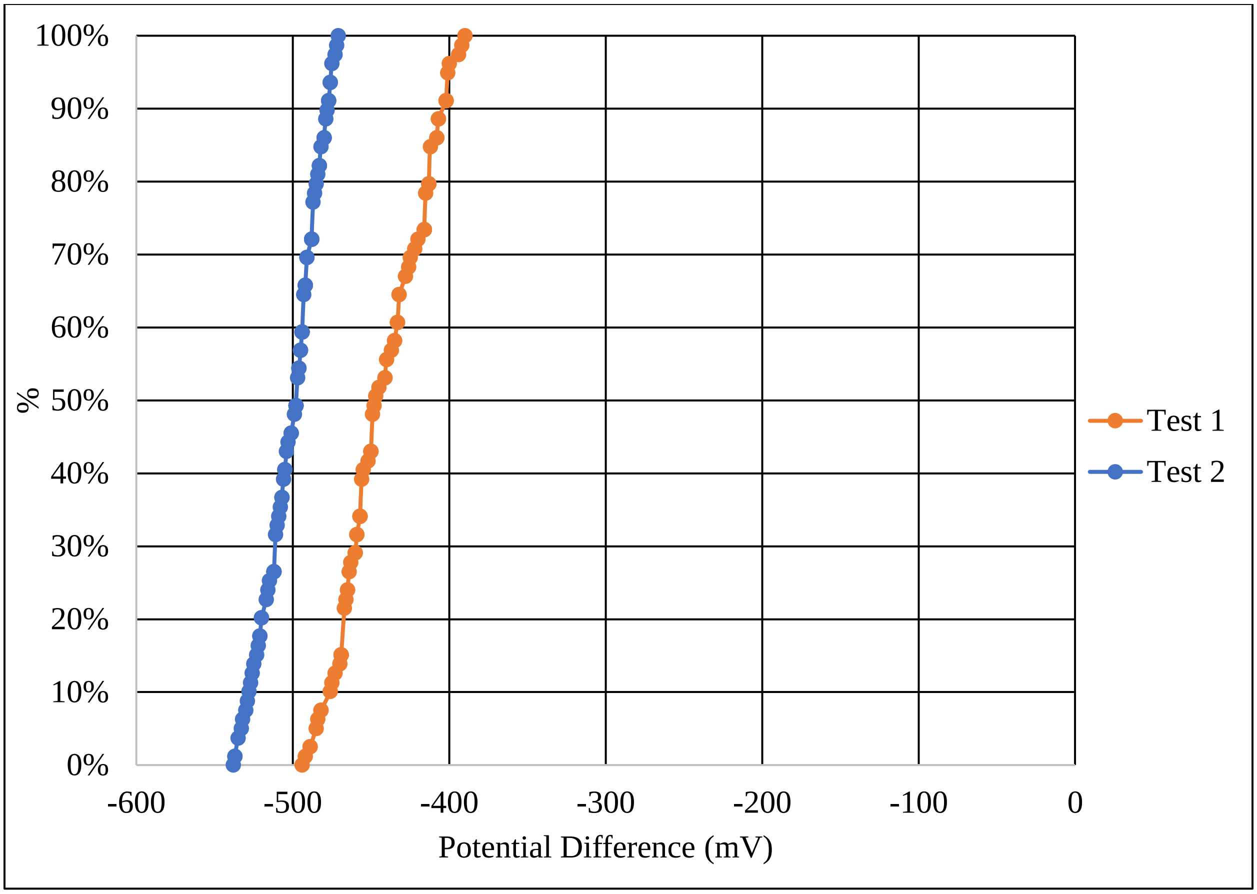


Figure E.15 Shaft 5 percentile distributions

Table E.11 Shaft 6, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-160	-148	-144	-139	-129	-131	-141	-155
3"	-168	-157	-143	-135	-130	-131	-141	-153
6"	-170	-163	-158	-147	-136	-138	-138	-142
9"	-168	-161	-153	-140	-128	-127	-140	-144
12"	-169	-161	-155	-145	-134	-129	-141	-146
15"	-170	-164	-157	-130	-142	-139	-143	-152
18"	-172	-167	-163	-136	-148	-149	-150	-152
21"	-174	-172	-166	-159	-155	-156	-156	-153
24"	-174	-174	-167	-168	-156	-159	-159	-160
27"	-176	-174	-161	-165	-159	-161	-161	-157

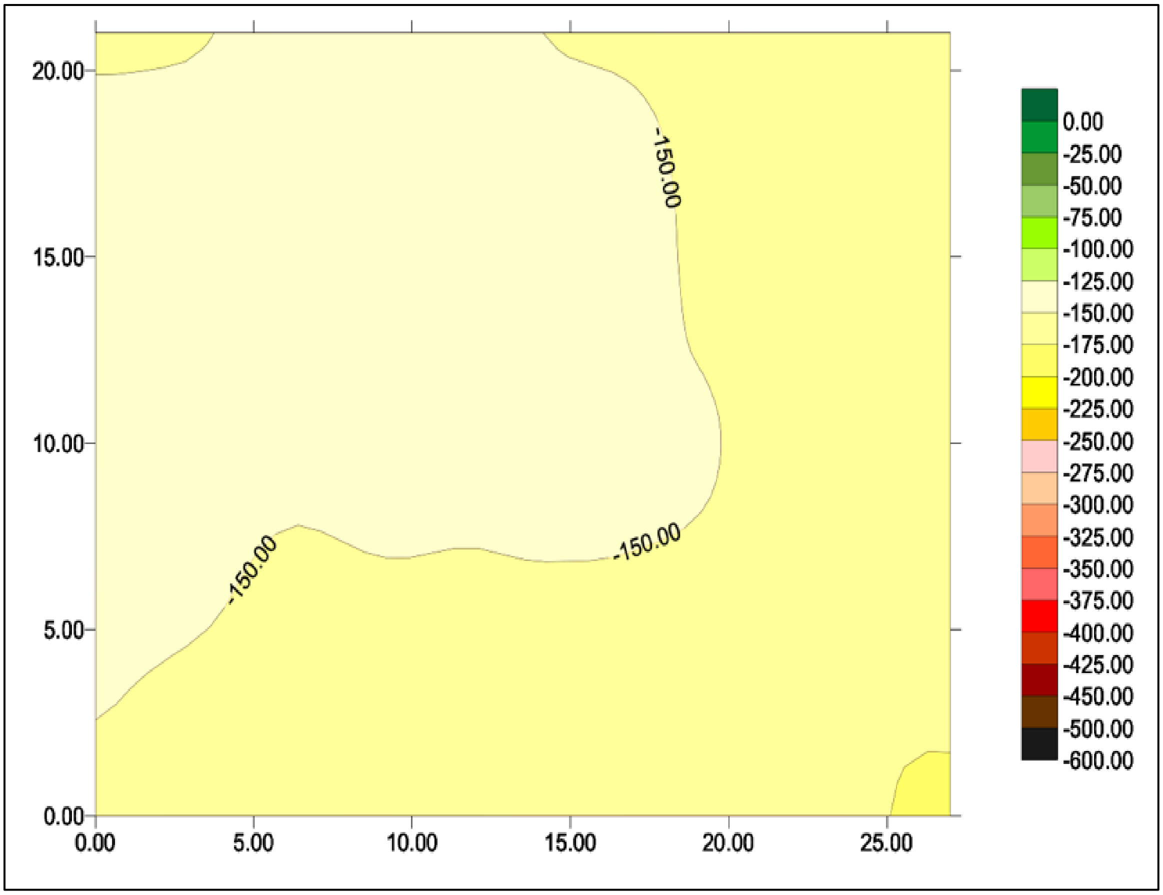


Figure E.16 Shaft 6, test 1 data map

Table E.12 Shaft 6, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-202	-212	-213	-198	-193	-183	-182	-171
3"	-210	-226	-212	-201	-192	-193	-178	-177
6"	-208	-219	-223	-216	-196	-191	-188	-179
9"	-217	-220	-218	-209	-199	-197	-192	-188
12"	-230	-232	-226	-217	-208	-206	-202	-194
15"	-237	-241	-242	-232	-217	-215	-210	-202
18"	-238	-246	-244	-227	-223	-215	-214	-209
21"	-229	-237	-236	-222	-207	-210	-205	-190
24"	-219	-225	-221	-207	-196	-198	-196	-177
27"	-209	-213	-209	-202	-183	-186	-185	-175

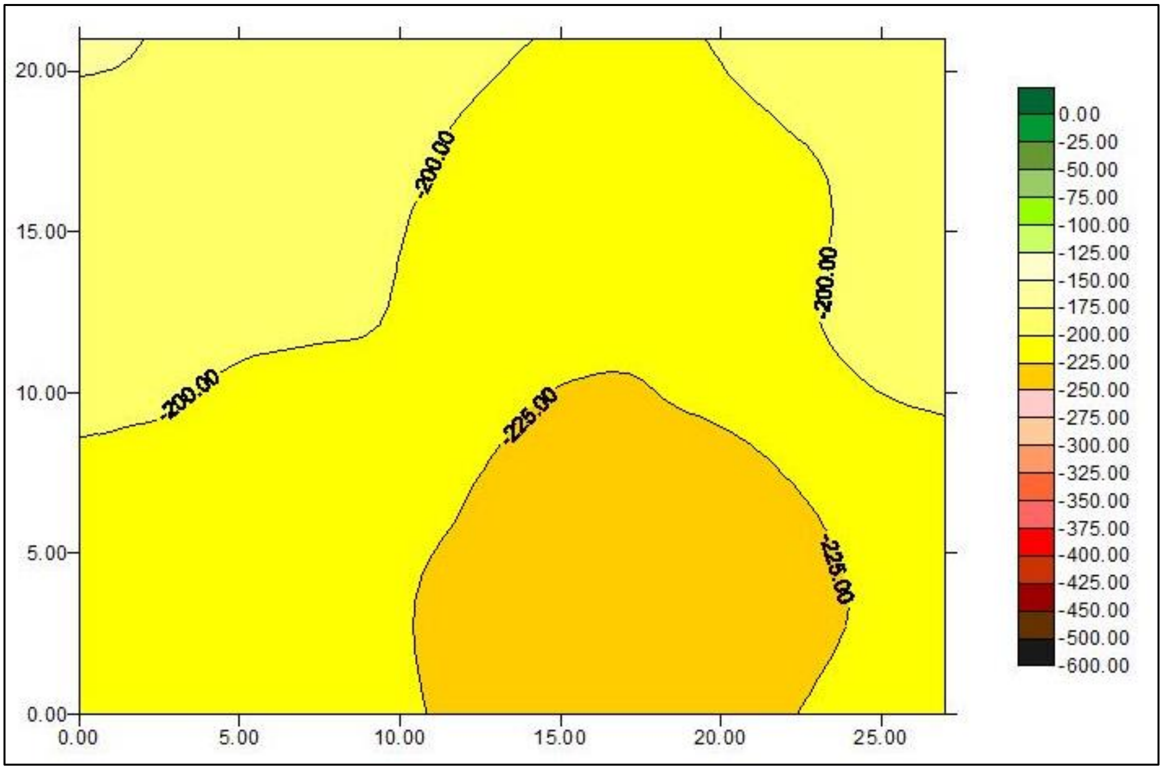


Figure E.17 Shaft 6, test 2 data map

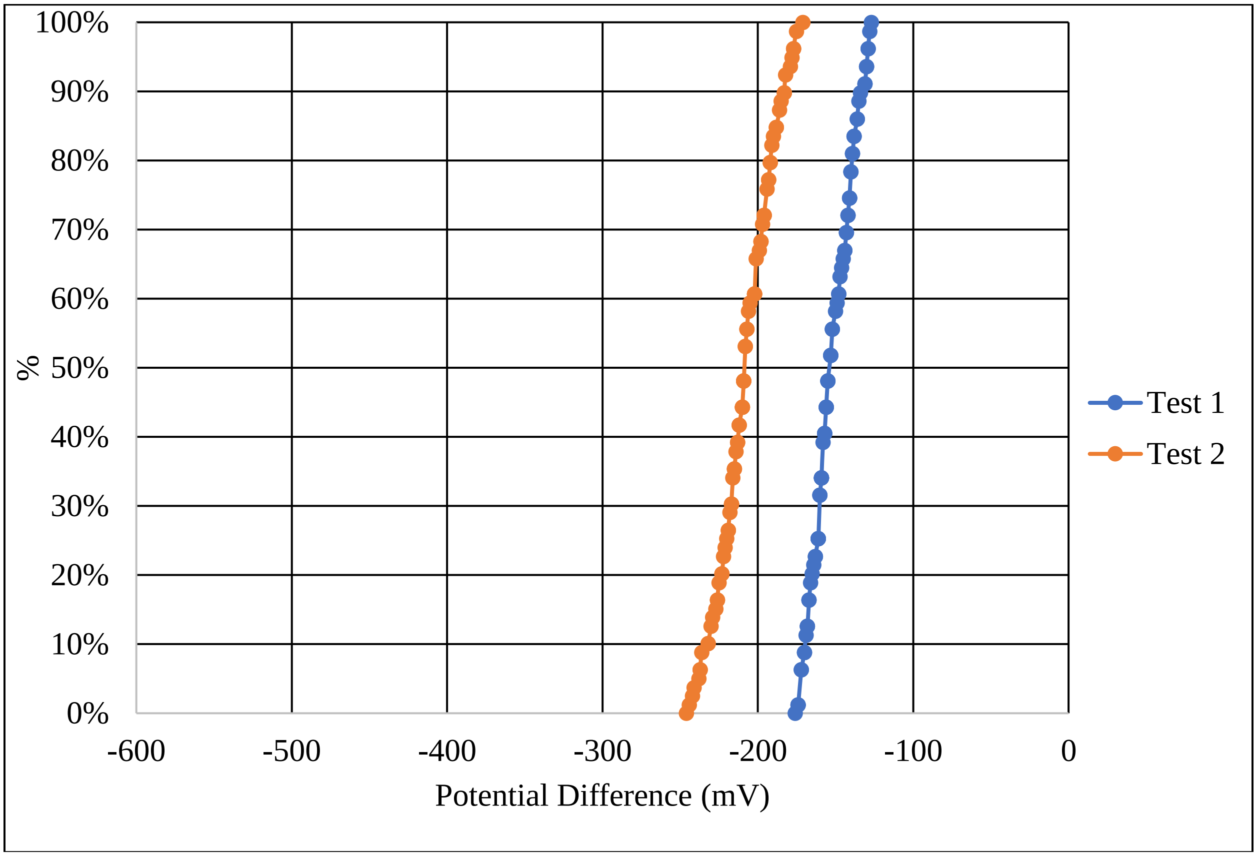


Figure E.18 Shaft 6 percentile distributions

Table E.13 Shaft 7, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-331	-331	-329	-338	-344	-350	-361	-361
3"	-339	-340	-339	-343	-345	-354	-372	-382
6"	-340	-346	-345	-348	-351	-366	-386	-402
9"	-352	-352	-354	-356	-361	-375	-398	-409
12"	-357	-358	-362	-364	-380	-398	-419	-422
15"	-354	-363	-373	-376	-398	-421	-444	-437
18"	-365	-364	-374	-383	-399	-430	-468	-454
21"	-364	-368	-377	-393	-400	-436	-462	-475
24"	-366	-369	-378	-395	-416	-436	-459	-466
27"	-364	-363	-378	-393	-415	-442	-458	-480

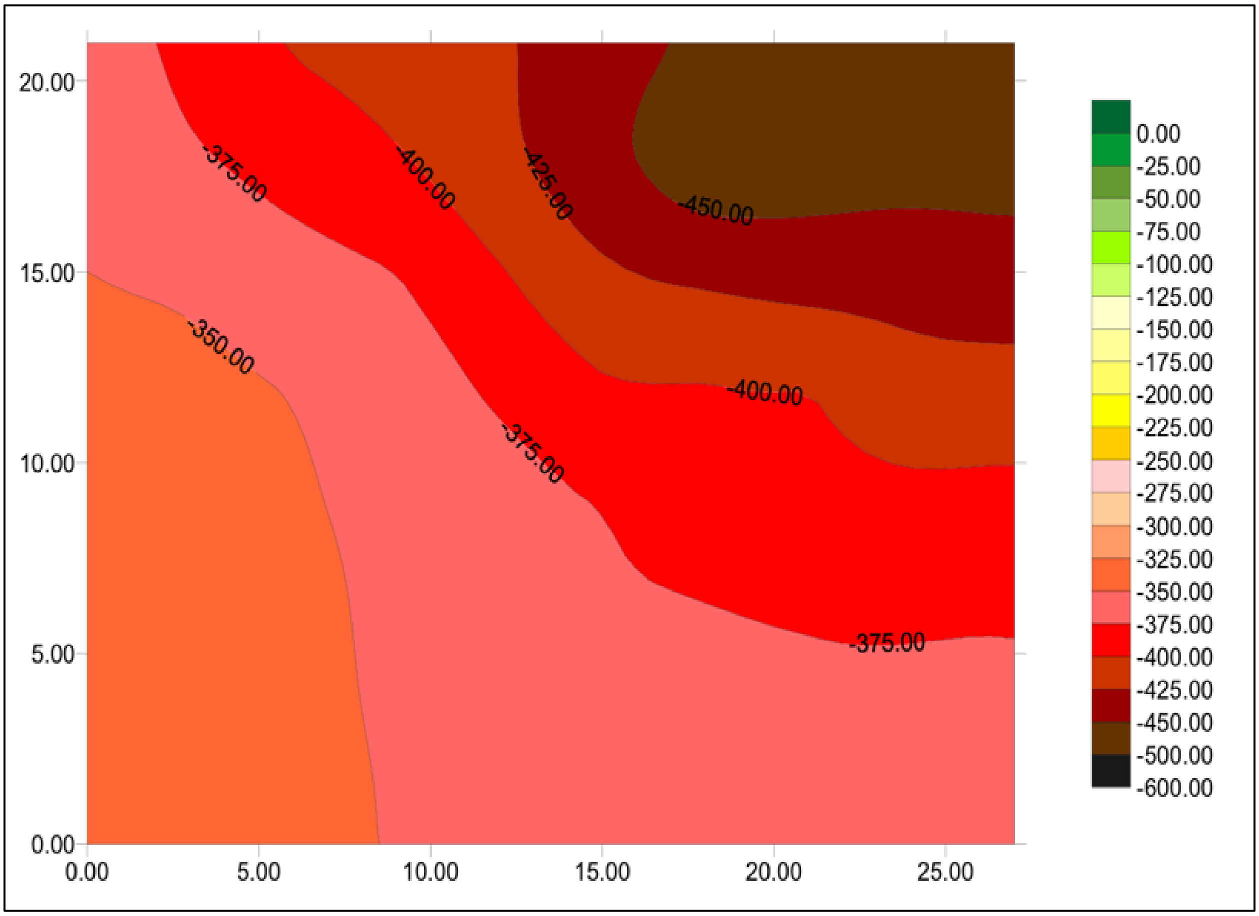


Figure E.19 Shaft 7, test 1 data map

Table E.14 Shaft 7, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-372	-372	-383	-384	-385	-379	-370	-380
3"	-373	-378	-377	-378	-381	-376	-370	-382
6"	-373	-393	-380	-381	-379	-376	-372	-370
9"	-385	-395	-403	-393	-386	-386	-384	-395
12"	-397	-383	-392	-391	-393	-391	-400	-395
15"	-386	-383	-382	-384	-382	-390	-394	-396
18"	-386	-383	-381	-382	-386	-379	-385	-391
21"	-373	-376	-371	-370	-371	-369	-371	-383
24"	-368	-363	-365	-363	-359	-362	-362	-371
27"	-357	-360	-358	-357	-352	-348	-349	-352

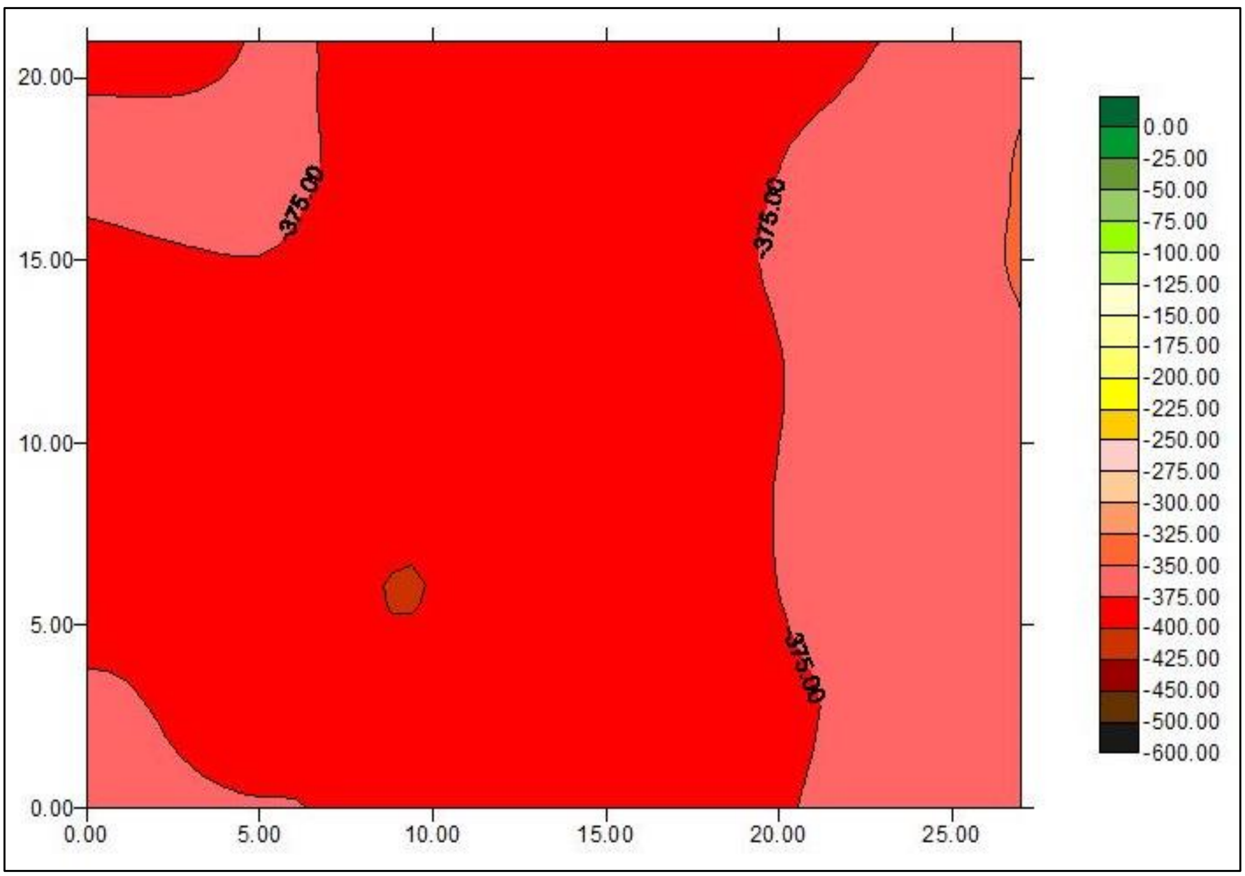


Figure E.20 Shaft 7, test 2 data map

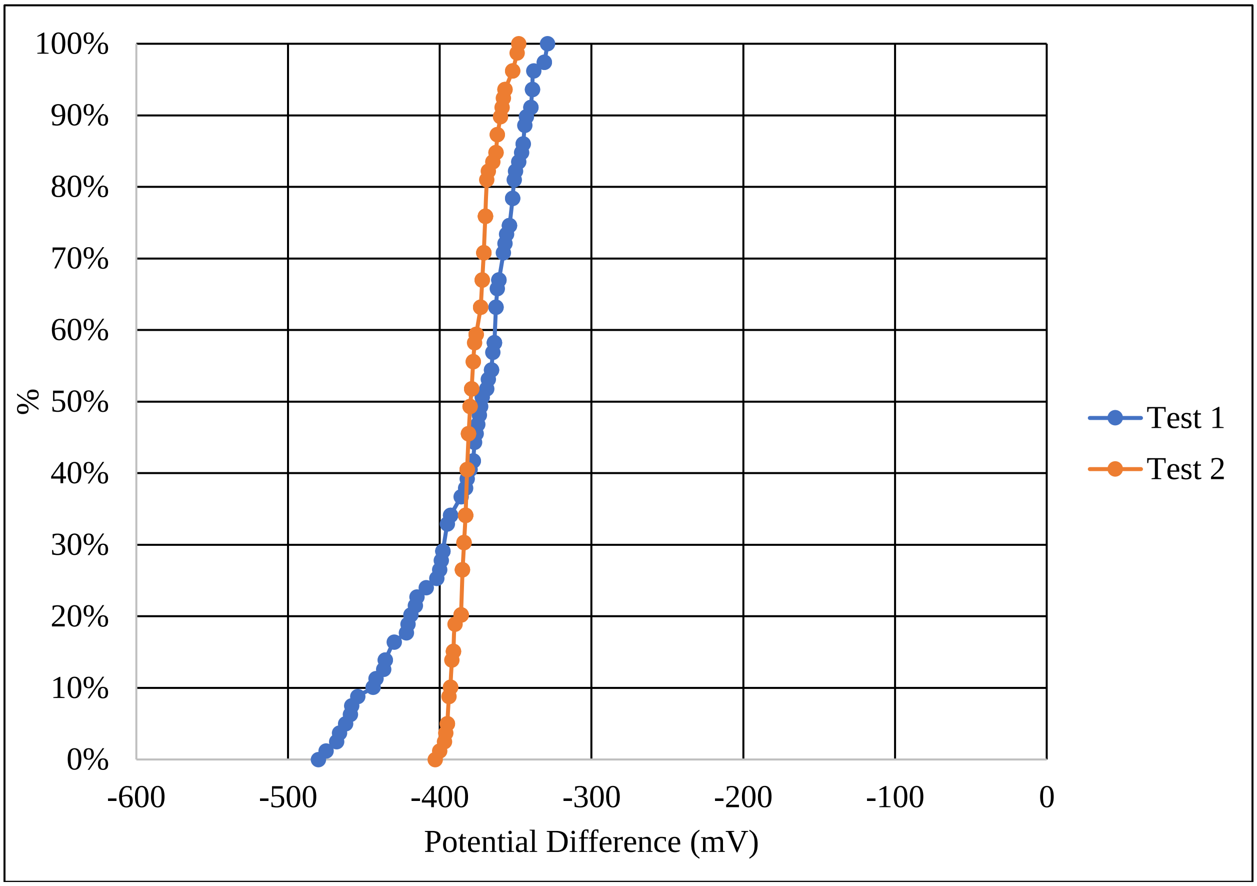


Figure E.21 Shaft 7 percentile distributions

Table E.15 Shaft 8, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-239	-243	-234	-233	-215	-211	-210	-215
3"	-233	-241	-237	-236	-218	-213	-220	-216
6"	-223	-221	-237	-235	-222	-213	-216	-209
9"	-326	-323	-235	-225	-213	-209	-208	-209
12"	-343	-235	-231	-227	-218	-217	-214	-212
15"	-352	-247	-239	-234	-226	-227	-229	-225
18"	-351	-238	-232	-227	-223	-212	-227	-219
21"	-345	-242	-236	-233	-223	-216	-215	-216
24"	-243	-238	-234	-230	-215	-214	-218	-223
27"	-240	-239	-233	-224	-219	-211	-209	-216

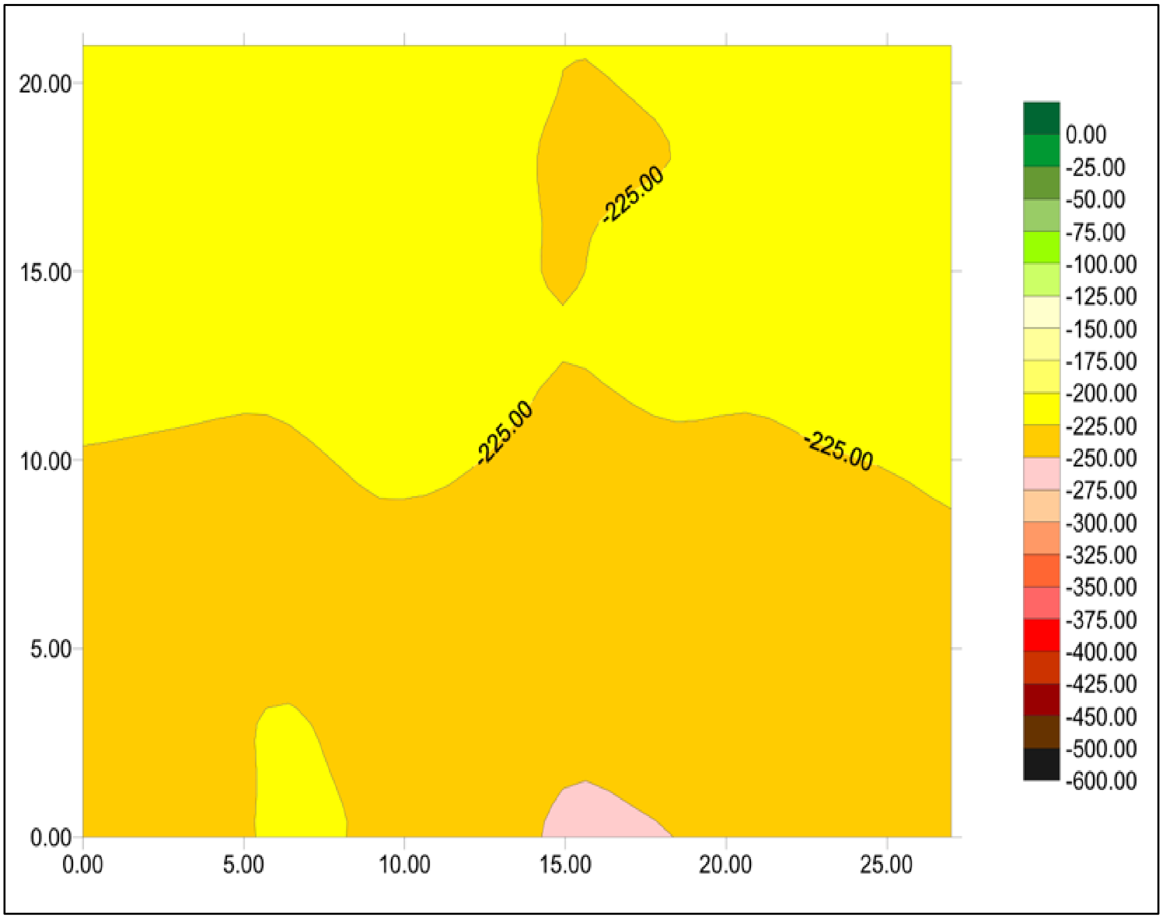


Figure E.22 Shaft 8, test 1 data map

Table E.16 Shaft 8, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-398	-397	-382	-358	-335	-322	-319	-315
3"	-427	-420	-401	-375	-348	-335	-324	-318
6"	-249	-420	-399	-372	-349	-339	-327	-325
9"	-436	-421	-399	-377	-352	-340	-324	-320
12"	-429	-418	-398	-376	-345	-338	-325	-327
15"	-427	-416	-399	-377	-349	-335	-326	-326
18"	-423	-415	-400	-379	-358	-338	-330	-327
21"	-435	-413	-403	-383	-356	-345	-345	-337
24"	-449	-432	-407	-387	-359	-352	-352	-342
27"	-477	-448	-329	-378	-359	-353	-350	-340

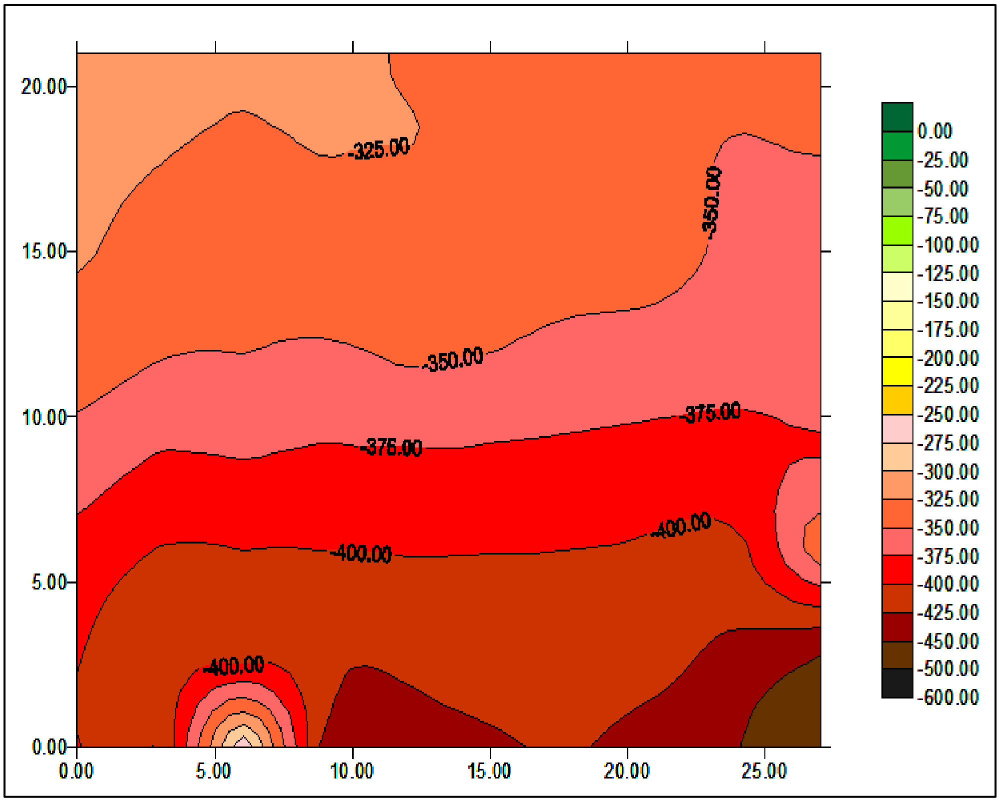


Figure E.23 Shaft 8, test 2 data map

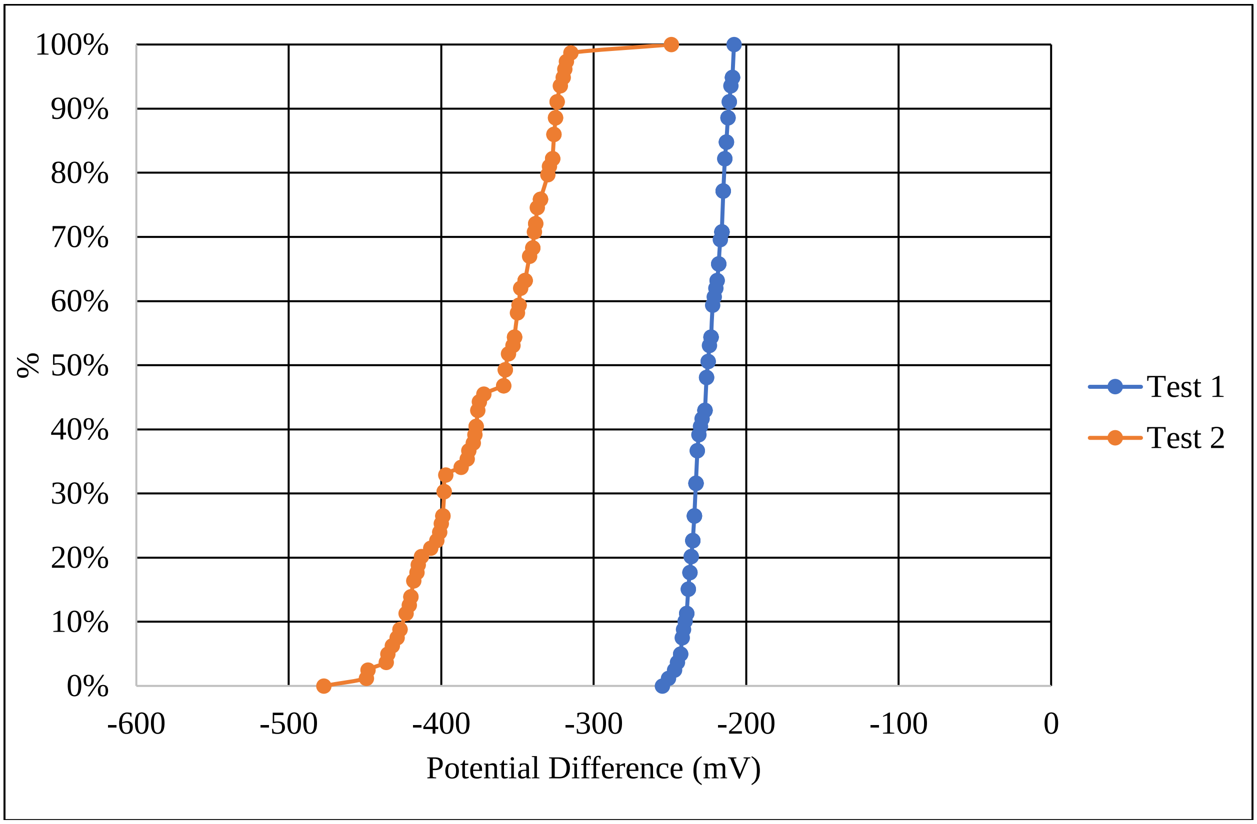


Figure E.24 Shaft 8 percentile distributions

Table E.17 Shaft 9, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-362	-364	-372	-381	-391	-410	-423	-424
3"	-356	-364	-368	-373	-385	-402	-420	-418
6"	-359	-359	-367	-383	-393	-413	-427	-430
9"	-362	-367	-370	-384	-397	-416	-427	-423
12"	-363	-366	-373	-382	-400	-415	-425	-419
15"	-372	-371	-375	-385	-399	-416	-421	-417
18"	-371	-372	-375	-379	-392	-403	-409	-410
21"	-365	-366	-367	-373	-392	-392	-396	-404
24"	-363	-365	-368	-375	-387	-387	-390	-396
27"	-357	-356	-366	-369	-383	-383	-387	-389

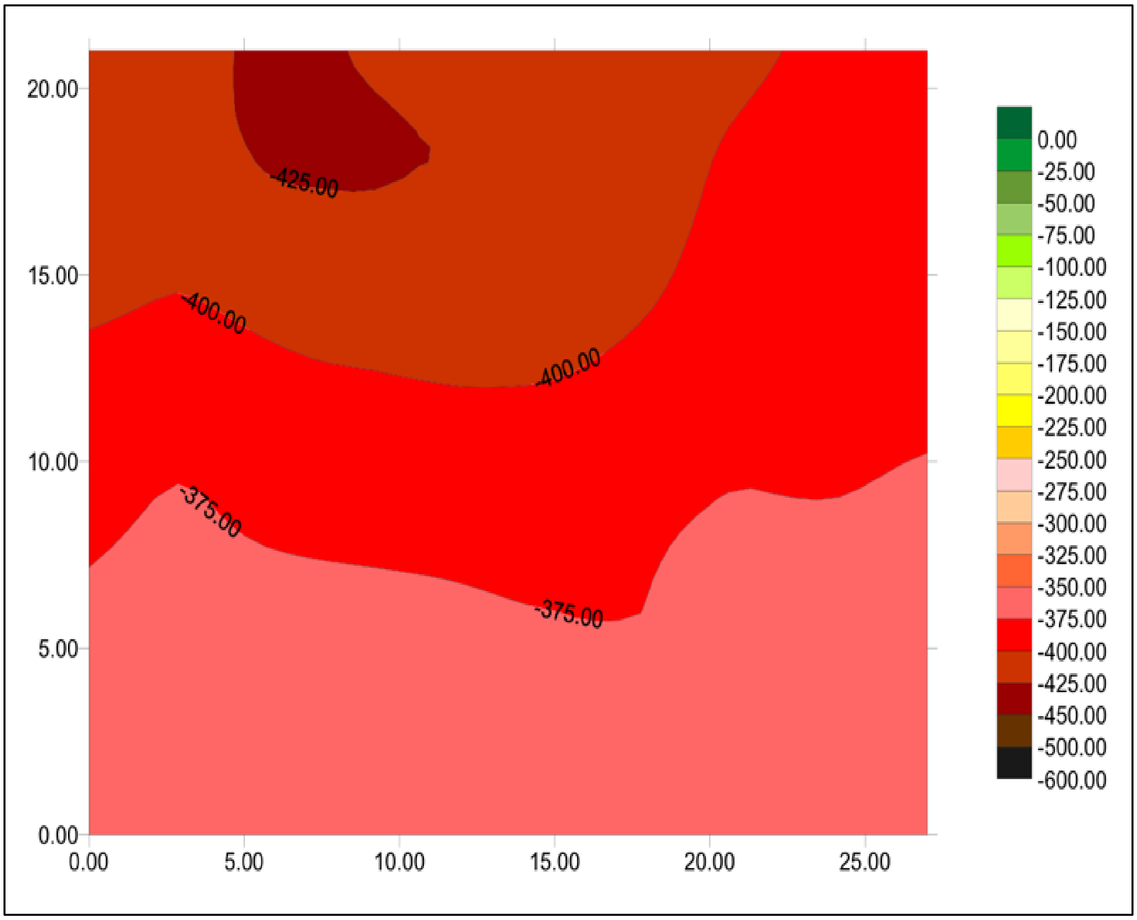


Figure E.25 Shaft 9, test 1 data map

Table E.18 Shaft 9, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-401	-413	-421	-430	-439	-442	-433	-416
3"	-407	-418	-424	-431	-439	-441	-436	-424
6"	-411	-419	-424	-430	-440	-437	-438	-431
9"	-414	-419	-421	-423	-430	-430	-431	-429
12"	-415	-418	-416	-419	-423	-430	-430	-433
15"	-421	-415	-415	-417	-417	-425	-425	-428
18"	-417	-412	-413	-418	-418	-425	-424	-422
21"	-417	-416	-414	-412	-414	-418	-423	-421
24"	-415	-417	-417	-420	-417	-425	-428	-427
27"	-420	-418	-420	-423	-420	-422	-430	-430

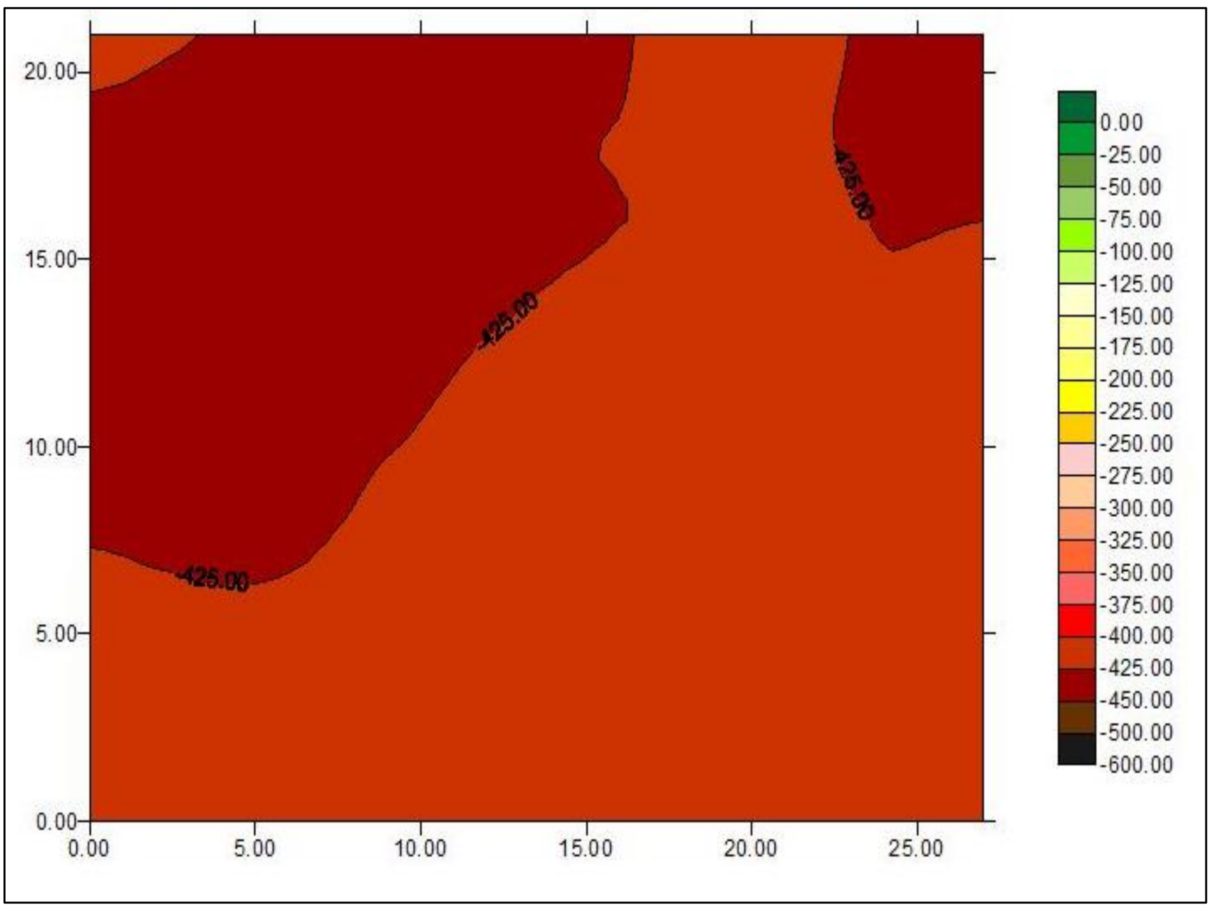


Figure E.26 Shaft 9, test 2 data map

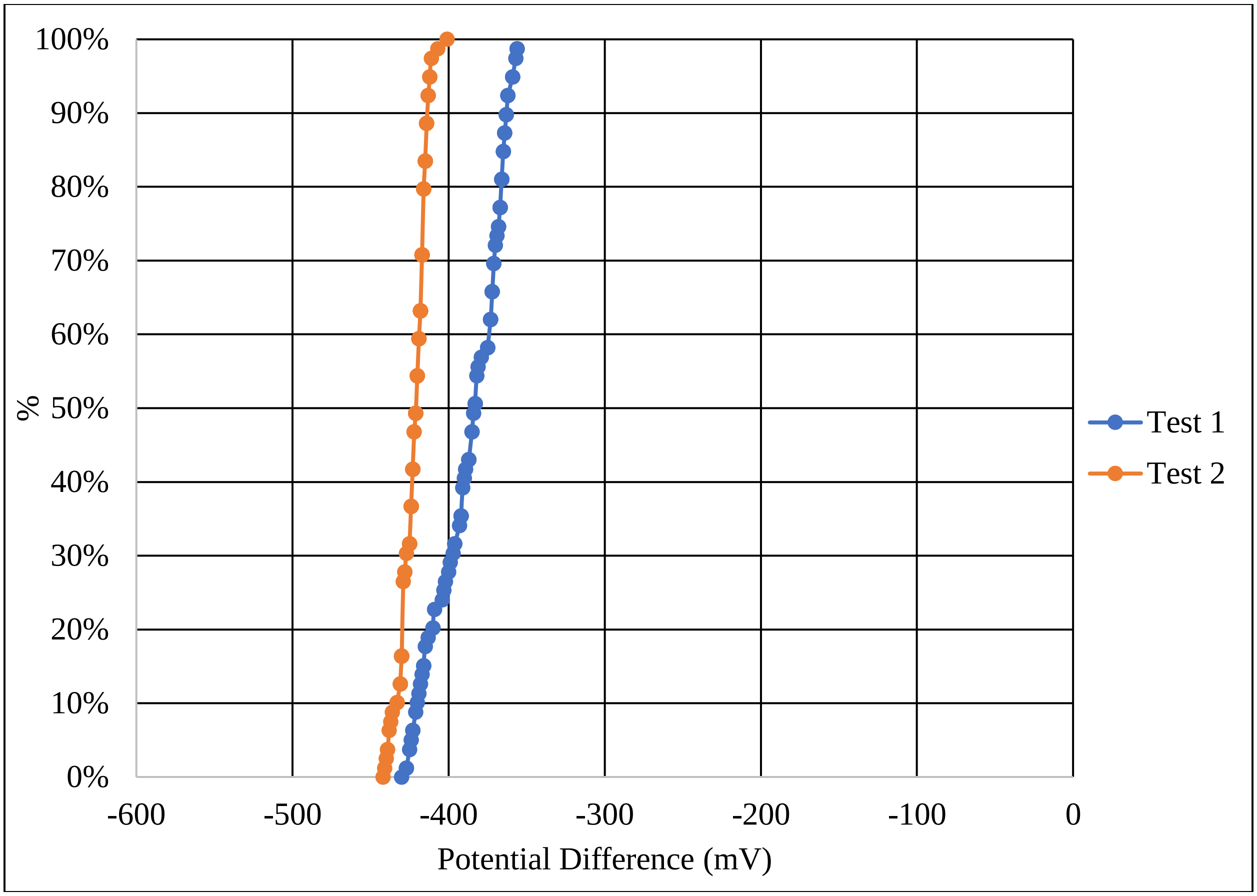


Figure E.27 Shaft 9 percentile distributions

Table E.19 Shaft 11, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-263	-271	-268	-267	-266	-265	-266	-258
3"	-264	-273	-272	-274	-272	-274	-271	-260
6"	-269	-278	-280	-281	-280	-287	-283	-266
9"	-272	-282	-283	-287	-287	-291	-282	-269
12"	-273	-283	-287	-288	-287	-285	-280	-273
15"	-285	-295	-292	-295	-290	-290	-293	-274
18"	-289	-299	-298	-298	-293	-303	-293	-271
21"	-297	-299	-296	-290	-287	-286	-287	-268
24"	-299	-312	-310	-348	-296	-297	-285	-278
27"	-305	-318	-342	-366	-321	-297	-289	-278

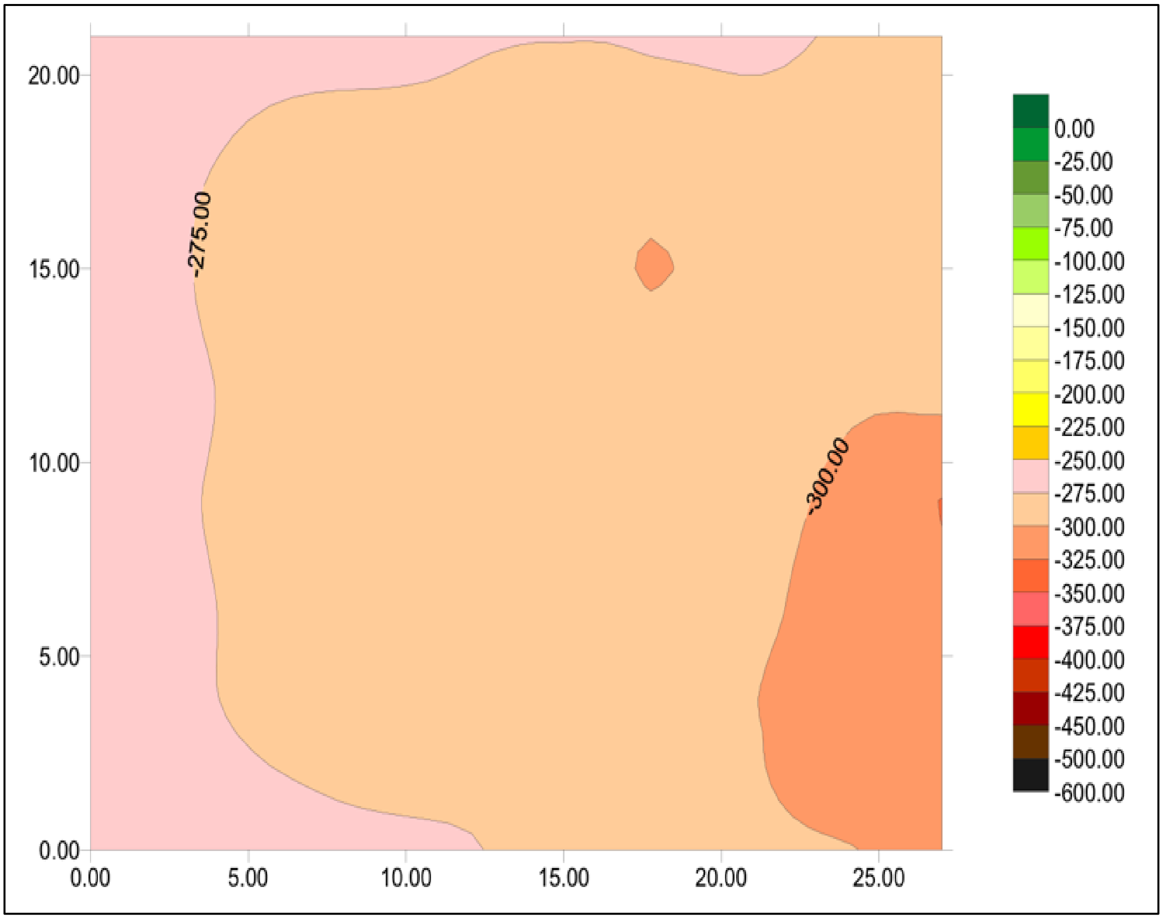


Figure E.28 Shaft 11, test 1 data map

Table E.20 Shaft 11, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-303	-311	-309	-307	-308	-290	-281	-270
3"	-309	-318	-316	-329	-316	-300	-290	-270
6"	-311	-318	-314	-314	-310	-300	-285	-273
9"	-323	-324	-321	-317	-314	-303	-288	-277
12"	-336	-340	-334	-334	-324	-315	-304	-288
15"	-346	-353	-347	-343	-323	-313	-311	-296
18"	-353	-356	-350	-341	-323	-313	-306	-291
21"	-357	-350	-349	-347	-329	-333	-317	-292
24"	-355	-354	-343	-340	-333	-330	-310	-297
27"	-352	-359	-342	-329	-331	-323	-310	-287

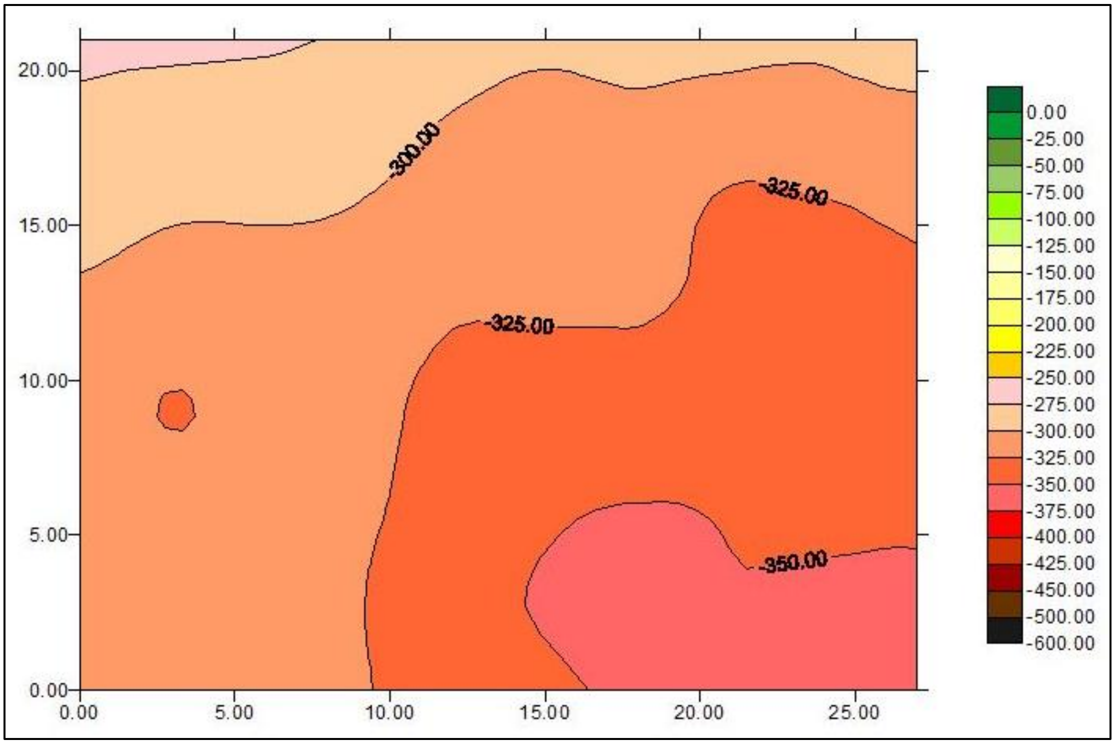


Figure E.29 Shaft 11, test 2 data map

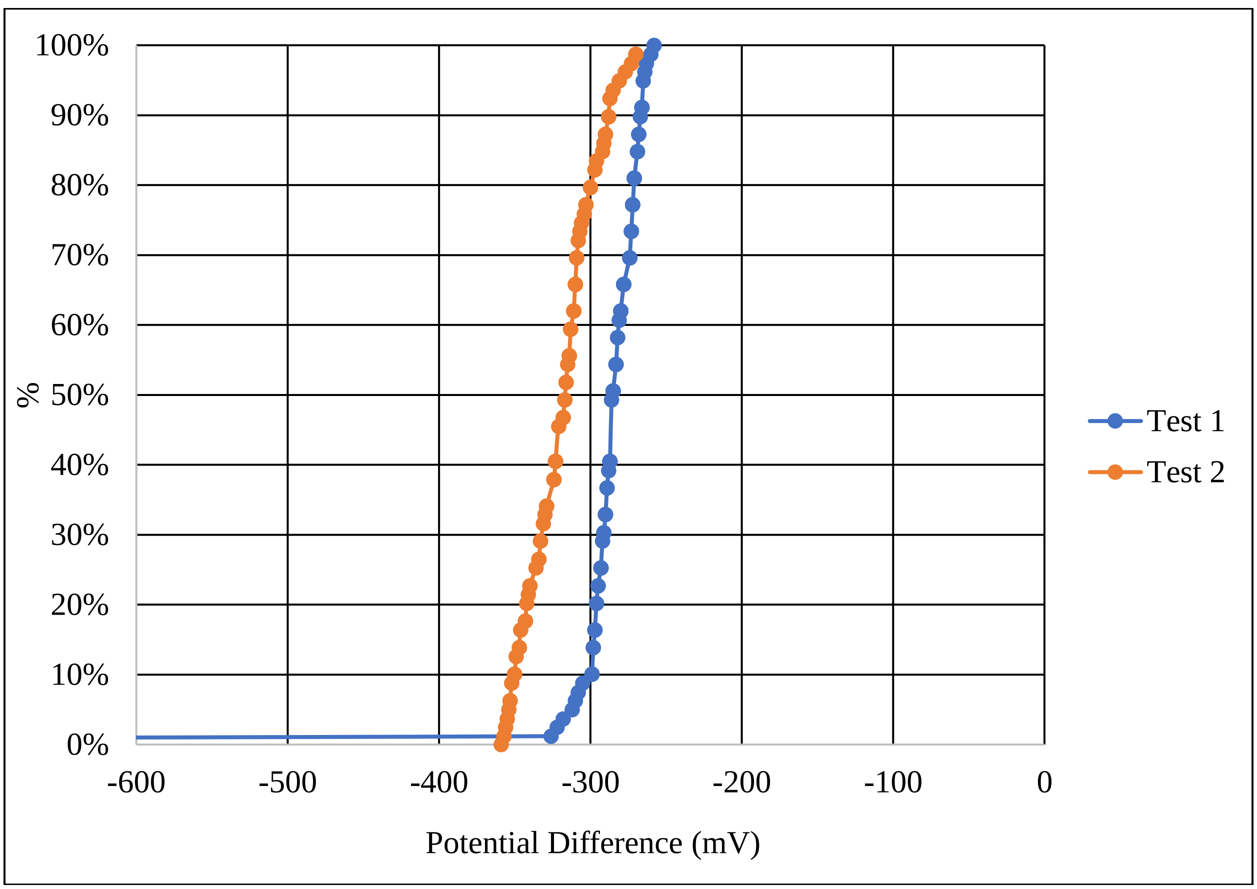


Figure E.30 Shaft 11 percentile distributions

Table E.21 Shaft 12, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-184	-188	-181	-176	-173	-174	-175	-170
3"	-195	-201	-189	-186	-181	-176	-175	-173
6"	-196	-201	-197	-191	-179	-172	-171	-168
9"	-204	-210	-200	-190	-179	-174	-170	-170
12"	-225	-217	-201	-193	-185	-179	-171	-170
15"	-220	-220	-206	-193	-191	-178	-176	-172
18"	-226	-222	-210	-206	-199	-187	-183	-187
21"	-220	-220	-213	-205	-195	-189	-185	-185
24"	-219	-221	-212	-204	-206	-186	-188	-181
27"	-218	-221	-212	-206	-199	-192	-186	-187

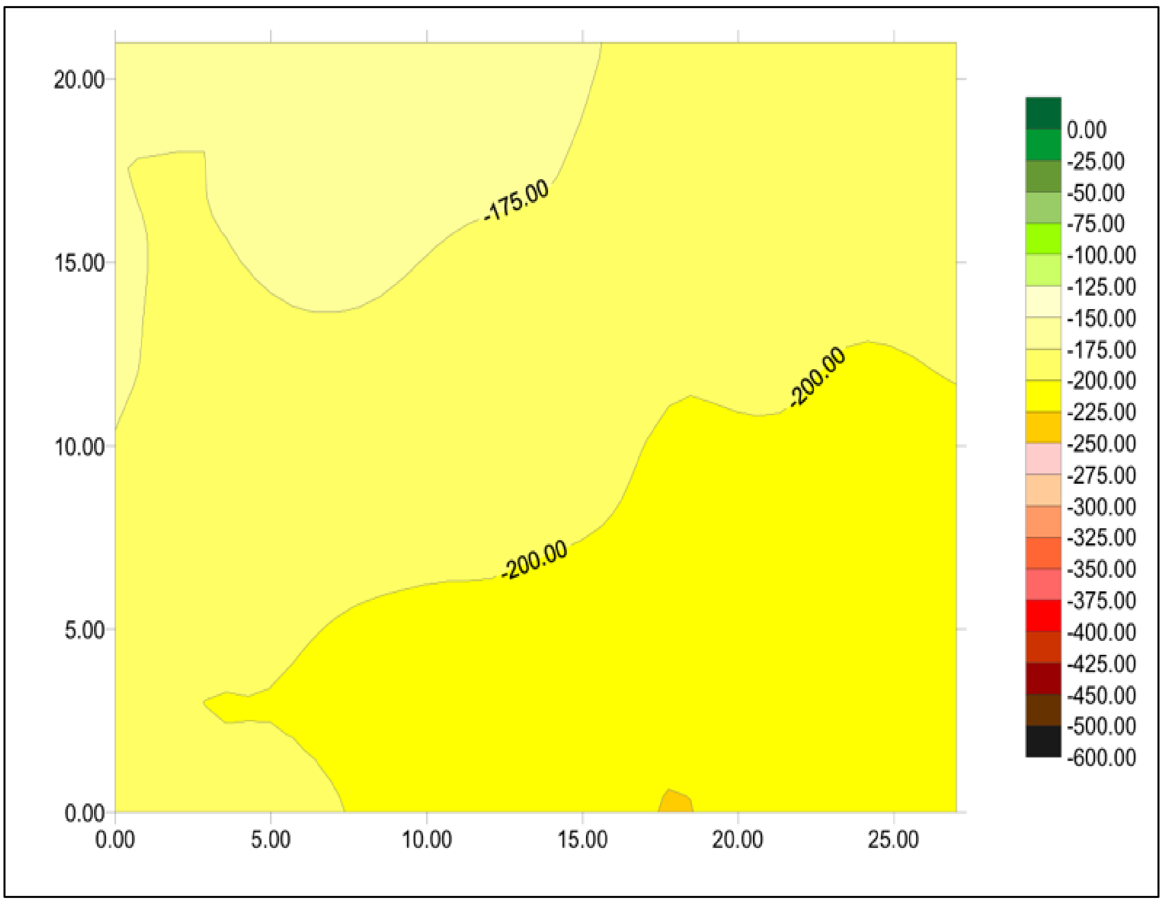


Figure E.31 Shaft 12, test 1 data map

Table E.22 Shaft 12, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-234	-241	-236	-233	-229	-228	-225	-221
3"	-240	-244	-238	-236	-231	-231	-227	-221
6"	-245	-247	-238	-236	-232	-229	-226	-223
9"	-253	-254	-242	-236	-231	-229	-223	-218
12"	-258	-260	-249	-239	-231	-231	-210	-224
15"	-270	-267	-258	-248	-231	-239	-223	-232
18"	-277	-280	-259	-251	-254	-251	-245	-244
21"	-294	-296	-286	-278	-272	-263	-259	-257
24"	-320	-315	-305	-302	-287	-279	-278	-272
27"	-337	-333	-318	-308	-295	-291	-283	-279

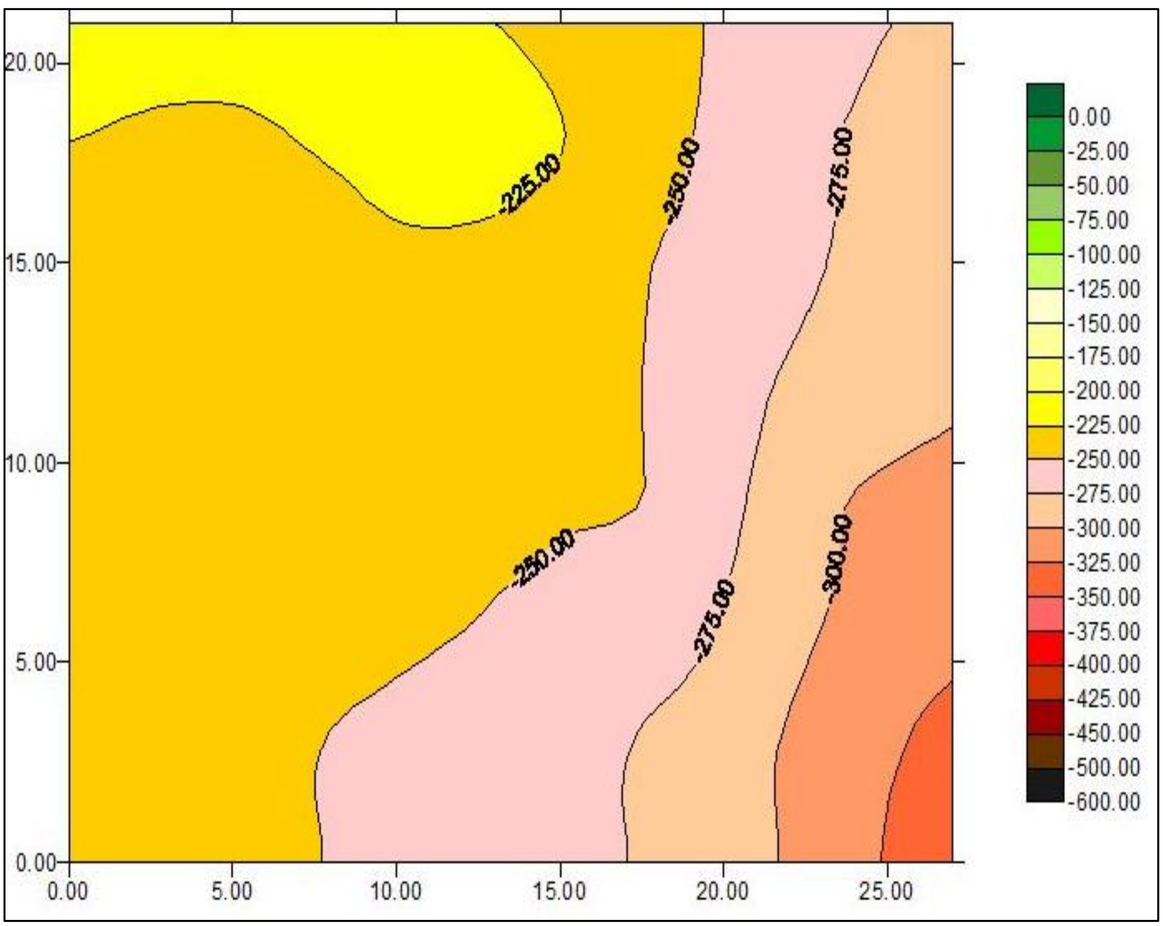


Figure E.32 Shaft 12, test 2 data map

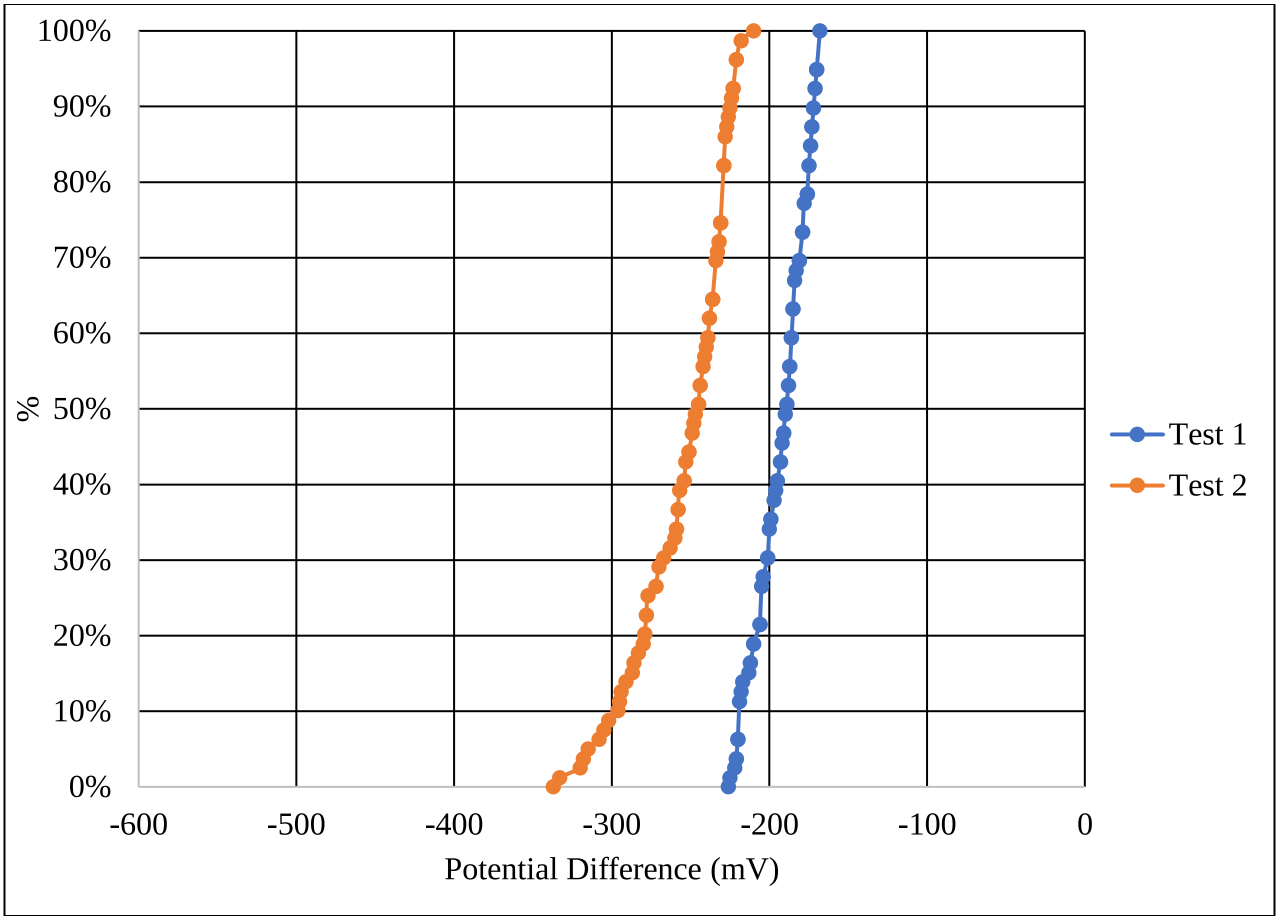


Figure E.33 Shaft 12 percentile distributions

Table E.23 Shaft 13, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-306	-300	-291	-288	-286	-285	-285	-289
3"	-305	-298	-291	-291	-290	-285	-286	-286
6"	-305	-299	-294	-289	-286	-284	-287	-285
9"	-303	-297	-293	-290	-285	-286	-285	-284
12"	-304	-299	-294	-291	-286	-285	-282	-282
15"	-307	-300	-296	-291	-285	-281	-280	-278
18"	-303	-302	-294	-290	-286	-283	-278	-278
21"	-297	-300	-300	-293	-288	-286	-283	-277
24"	-345	-302	-302	-293	-290	-288	-284	-280
27"	-351	-336	-301	-293	-289	-285	-283	-278

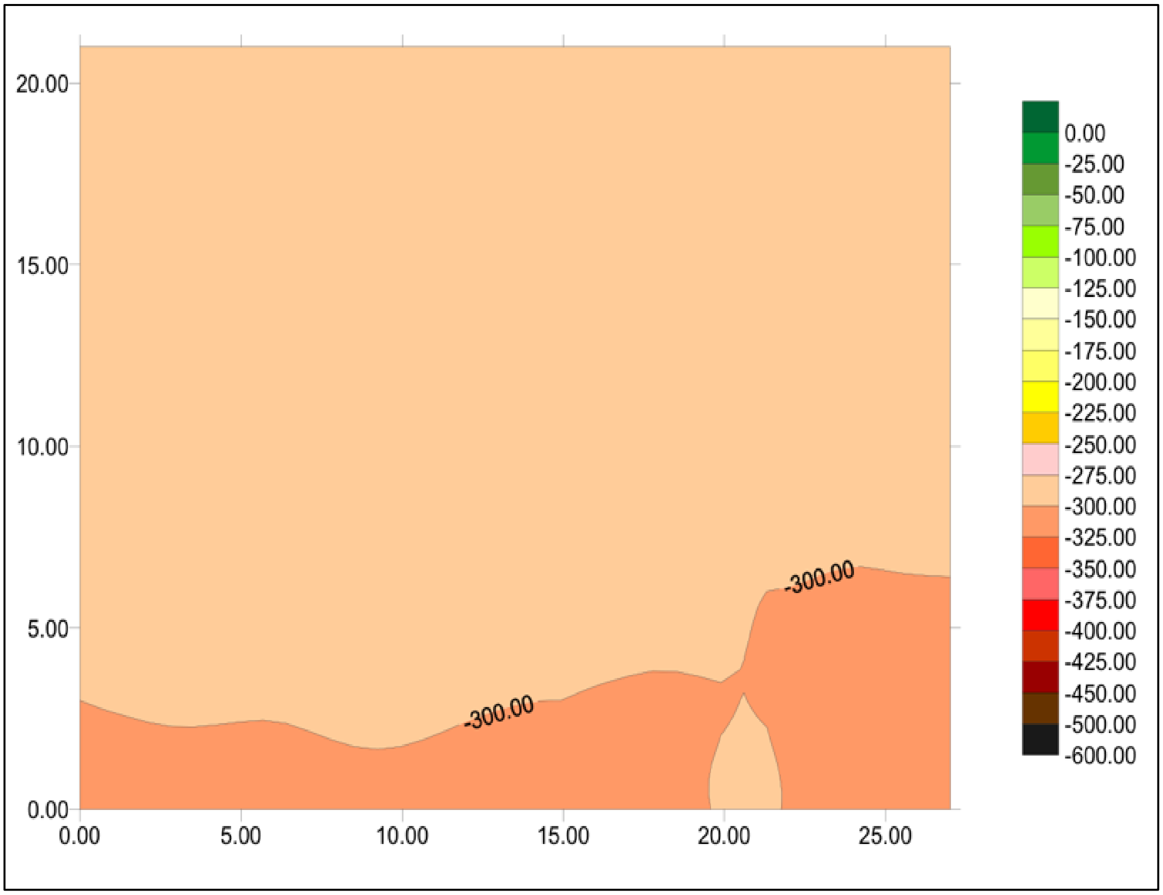


Figure E.34 Shaft 13, test 1 data map

Table E.24 Shaft 13, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-342	-337	-330	-325	-324	-321	-316	-308
3"	-345	-339	-336	-332	-327	-322	-315	-309
6"	-352	-345	-341	-334	-332	-329	-321	-311
9"	-356	-349	-341	-334	-333	-328	-321	-311
12"	-351	-347	-343	-337	-334	-326	-321	-313
15"	-351	-347	-342	-337	-333	-328	-321	-317
18"	-352	-348	-341	-335	-332	-329	-322	-321
21"	-351	-347	-343	-340	-338	-334	-323	-315
24"	-357	-351	-346	-343	-337	-337	-331	-322
27"	-360	-355	-350	-345	-341	-339	-335	-323

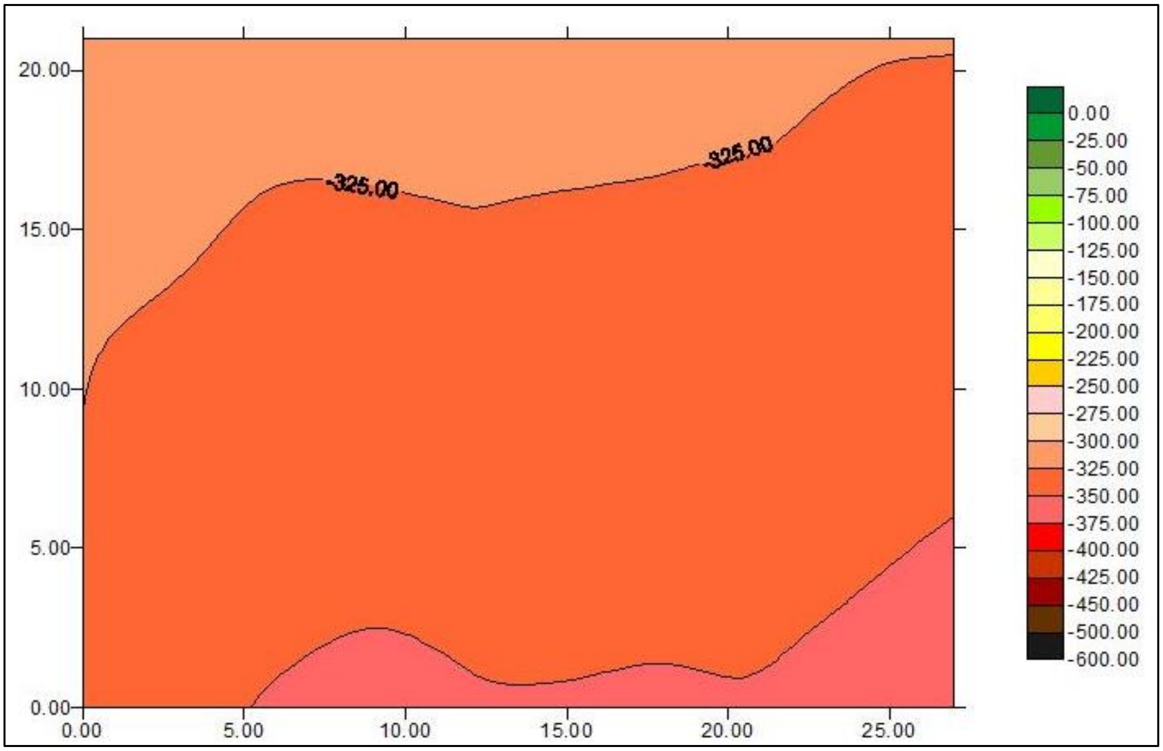


Figure E.35 Shaft 13, test 2 data map

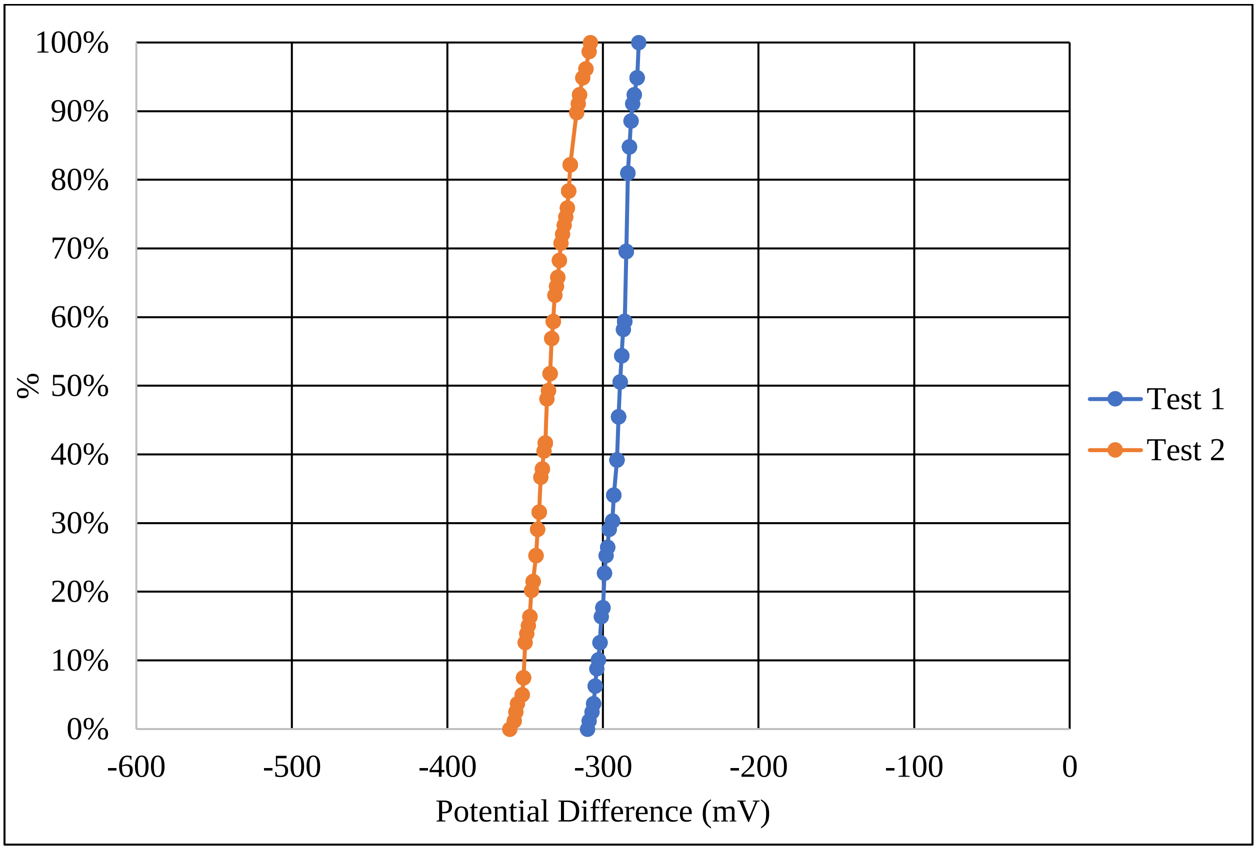


Figure E.36 Shaft 12 percentile distributions

Table E.25 Shaft 14, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-371	-300	-291	-282	-273	-248	-255	-263
3"	-337	-296	-292	-284	-235	-214	-230	-251
6"	-298	-299	-294	-289	-280	-243	-245	-268
9"	-300	-294	-291	-285	-280	-276	-253	-281
12"	-298	-293	-288	-285	-285	-280	-270	-268
15"	-302	-295	-288	-284	-281	-281	-275	-279
18"	-297	-293	-289	-284	-279	-277	-276	-278
21"	-300	-296	-291	-284	-278	-276	-276	-277
24"	-301	-297	-291	-283	-274	-262	-262	-270
27"	-304	-298	-294	-286	-277	-270	-267	-273

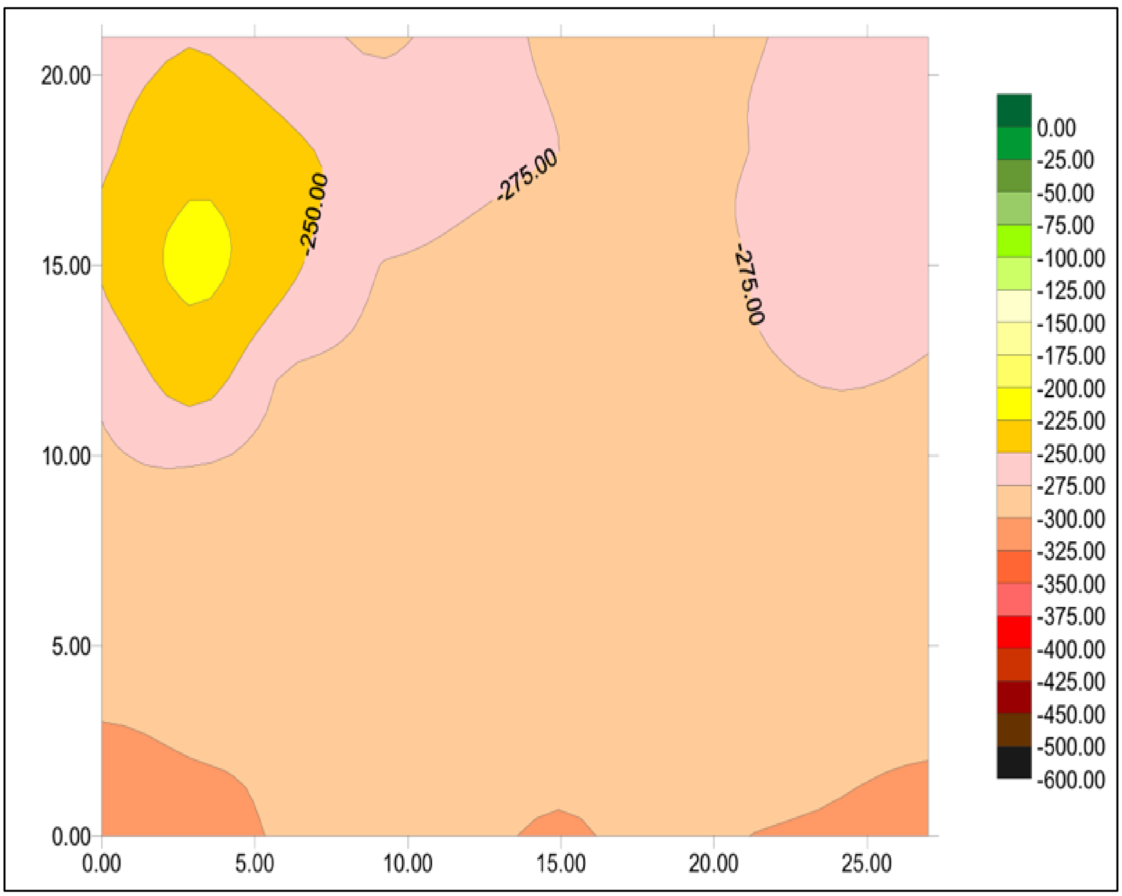


Figure E.37 Shaft 14, test 1 data map

Table E.26 Shaft 14, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-353	-352	-351	-348	-342	-341	-339	-330
3"	-358	-357	-352	-344	-340	-339	-341	-336
6"	-365	-360	-353	-344	-339	-336	-339	-340
9"	-375	-368	-357	-348	-342	-339	-338	-341
12"	-376	-369	-358	-349	-342	-340	-337	-336
15"	-376	-369	-360	-352	-346	-340	-335	-331
18"	-377	-372	-363	-353	-347	-338	-334	-331
21"	-385	-375	-367	-355	-346	-339	-334	-333
24"	-398	-384	-372	-359	-348	-342	-340	-336
27"	-402	-385	-369	-360	-351	-348	-345	-345

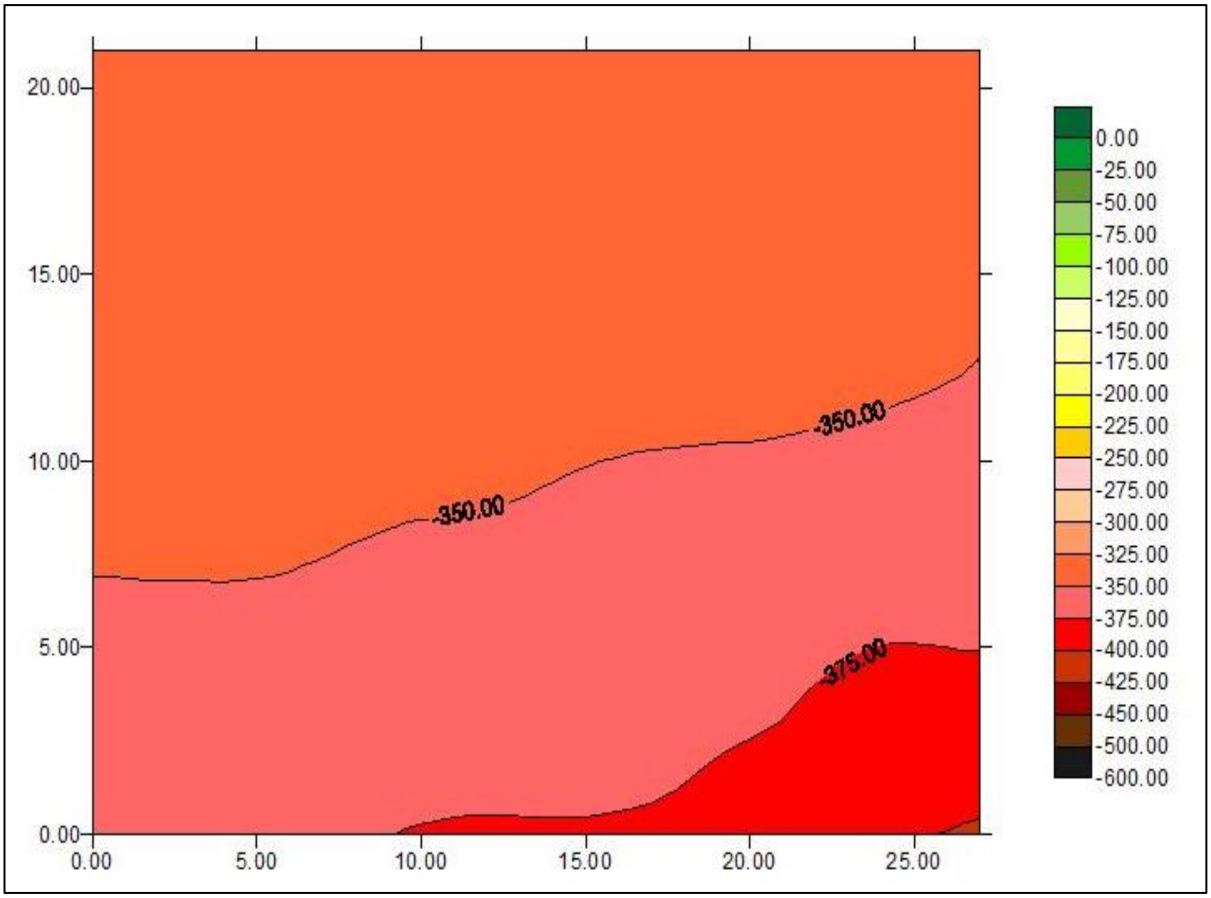


Figure E.38 Shaft 14, test 2 data map

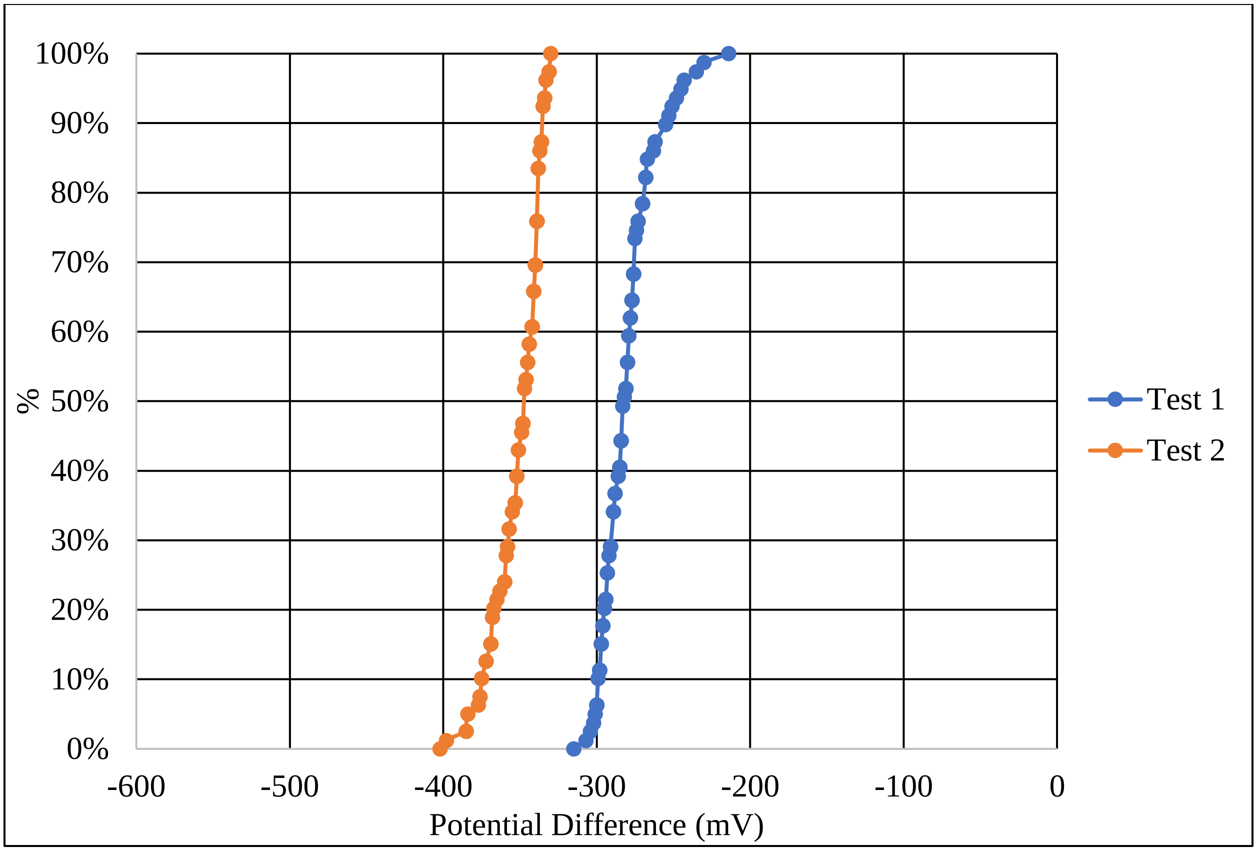


Figure E.39 Shaft 14 percentile distributions

Table E.27 Shaft 15, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"			-349	-350	-351	-340	-339	-347
3"			-339	-345	-340	-338	-338	-341
6"		-337	-336	-338	-340	-330	-335	-336
9"		-335	-334	-333	-327	-328	-331	-334
12"			-330	-330	-327	-327	-329	-334
15"		-332	-337	-331	-328	-330	-332	-336
18"		-337	-336	-335	-332	-331	-334	-337
21"		-344	-336	-335	-328	-327	-333	-339
24"		-349	-348	-341	-338	-332	-337	-341
27"	-362	-357	-344	-341	-337	-333	-336	-337

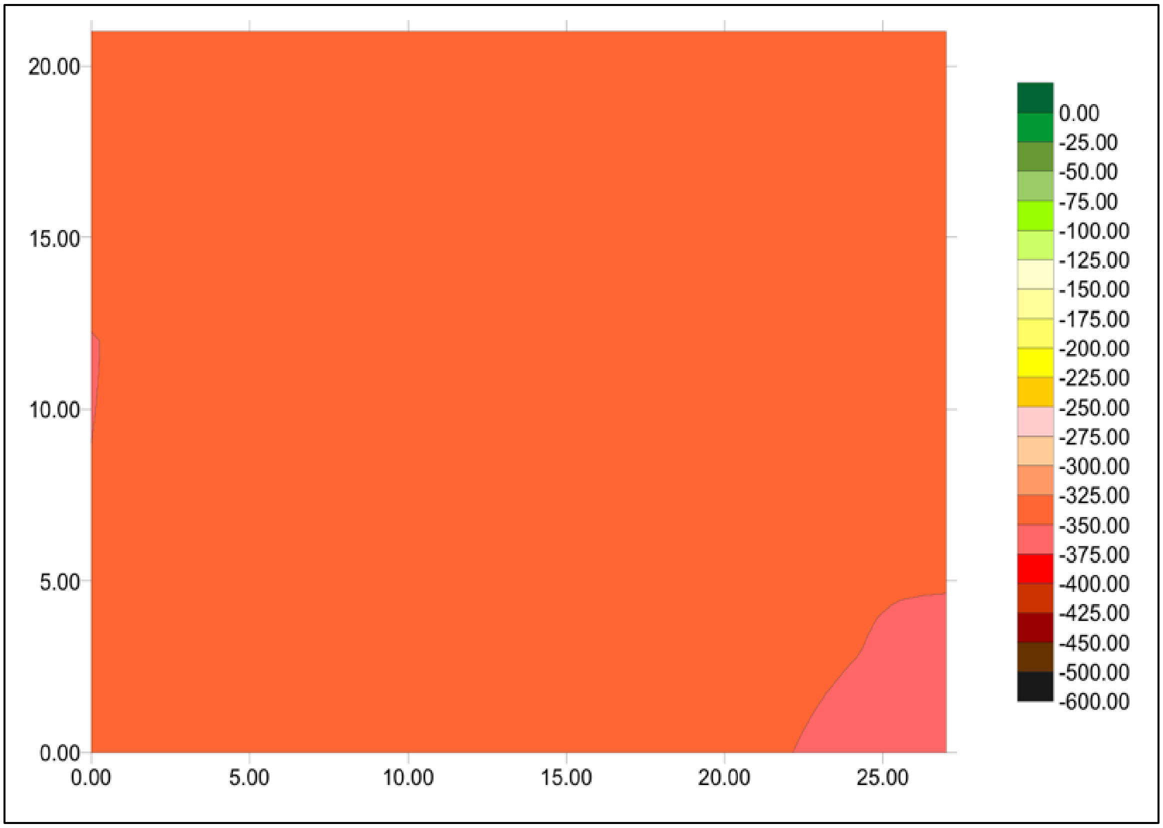


Figure E.40 Shaft 15, test 1 data map

Table E.28 Shaft 15, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-435	-419	-412	-404	-387	-365	-368	-359
3"	-430	-418	-414	-399	-385	-362	-370	-357
6"	-431	-410	-419	-399	-390	-336	-364	-353
9"	-416	-413	-410	-403	-375	-369	-369	-357
12"	-422	-418	-412	-408	-391	-378	-374	-362
15"	-426	-416	-415	-408	-395	-377	-374	-363
18"	-431	-414	-415	-411	-399	-383	-374	-368
21"	-436	-440	-421	-413	-399	-386	-376	-371
24"	-450	-449	-428	-419	-395	-384	-381	-369
27"	-439	-436	-434	-420	-403	-388	-375	-366

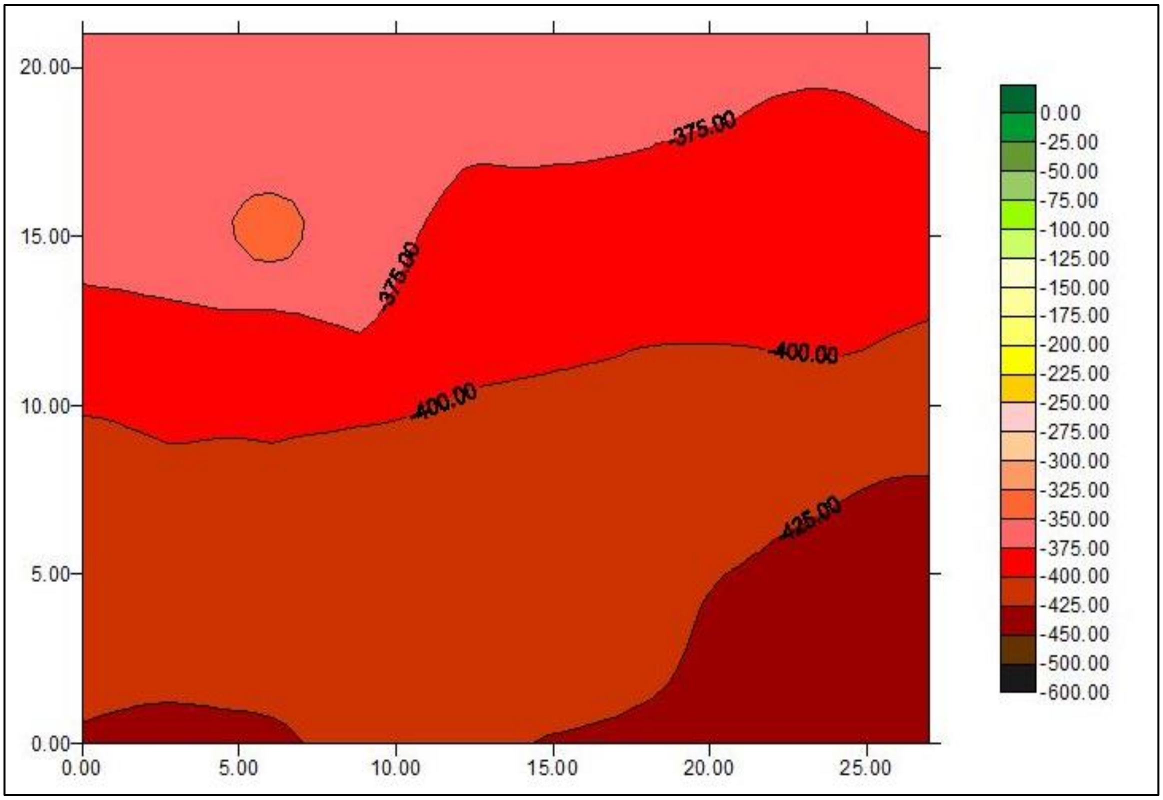


Figure E.41 Shaft 15, test 2 data map

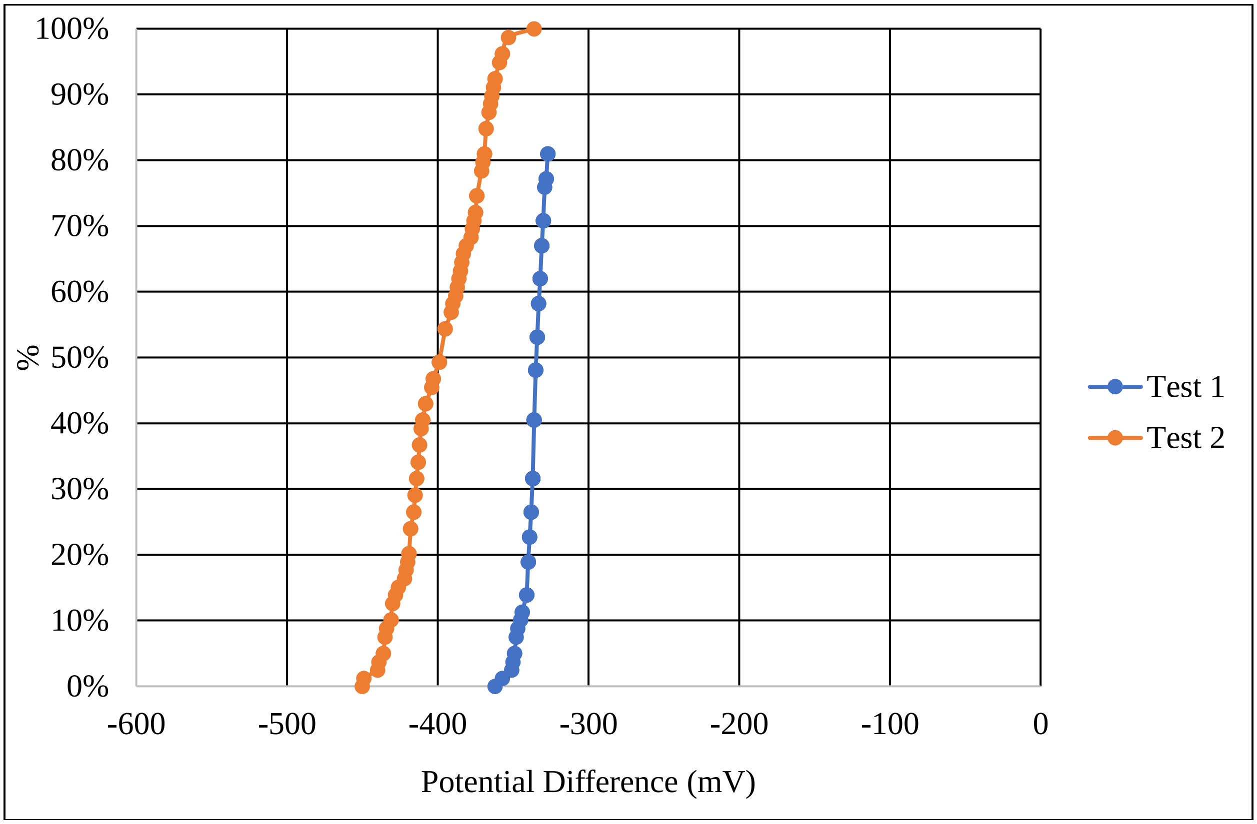


Figure E.42 Shaft 15 percentile distributions

Table E.29 Shaft 16, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-291	-278	-271	-266	-259	-256	-261	-251
3"	-298	-287	-279	-275	-269	-266	-271	-262
6"	-298	-294	-286	-278	-272	-271	-270	-264
9"	-292	-290	-283	-276	-277	-275	-271	-265
12"	-293	-290	-284	-274	-270	-273	-275	-269
15"	-292	-291	-289	-279	-278	-280	-278	-271
18"	-293	-288	-284	-278	-275	-278	-282	-275
21"	-292	-290	-285	-283	-279	-279	-284	-279
24"	-293	-290	-289	-283	-280	-283	-284	-283
27"	-289	-288	-286	-282	-280	-281	-286	-286

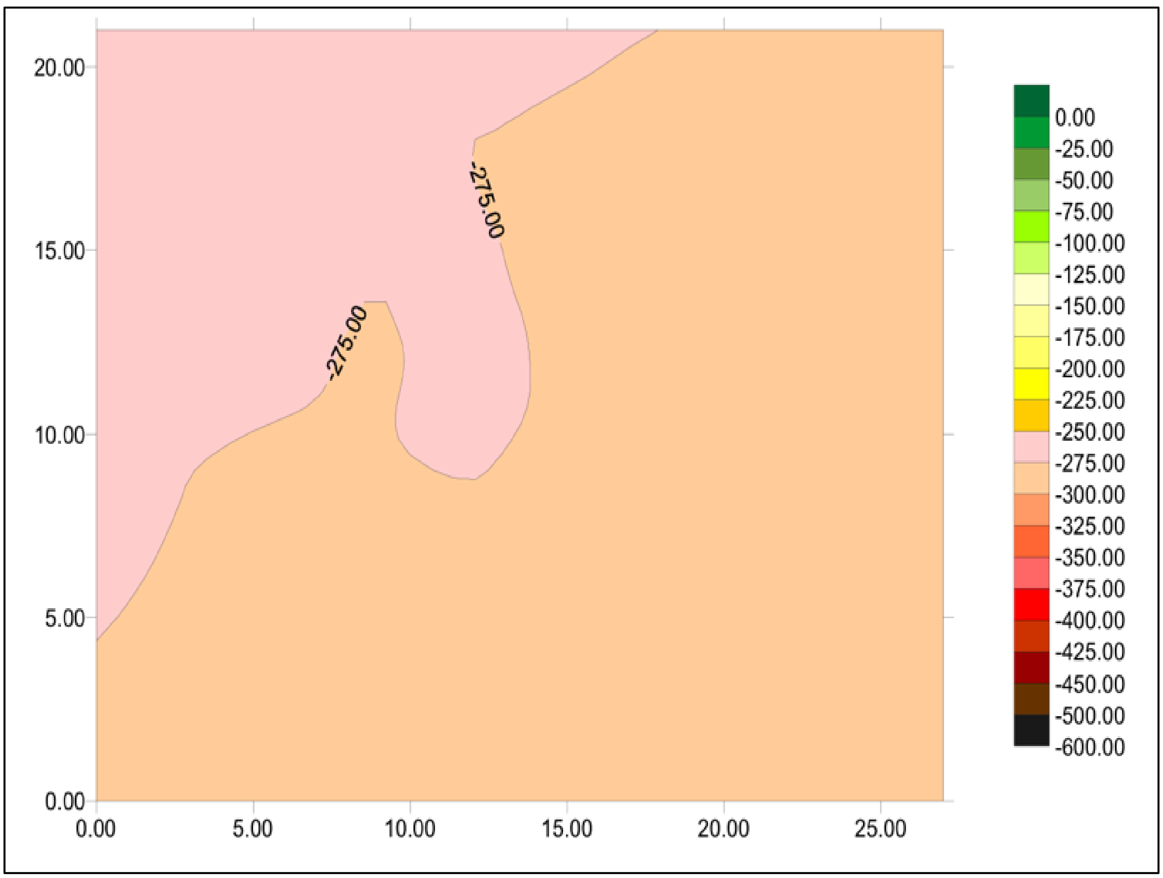


Figure E.43 Shaft 16, test 1 data map

Table E.30 Shaft 16, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-474	-463	-458	-452	-451	-453	-456	-452
3"	-472	-467	-458	-453	-452	-456	-462	-455
6"	-470	-466	-458	-453	-451	-454	-458	-448
9"	-472	-468	-460	-456	-459	-460	-463	-453
12"	-475	-472	-463	-460	-464	-462	-461	-449
15"	-478	-471	-463	-459	-461	-458	-459	-450
18"	-487	-478	-469	-466	-461	-464	-466	-458
21"	-494	-491	-479	-476	-469	-473	-470	-465
24"	-492	-490	-483	-481	-475	-474	-475	-466
27"	-488	-486	-484	-478	-472	-472	-475	-474

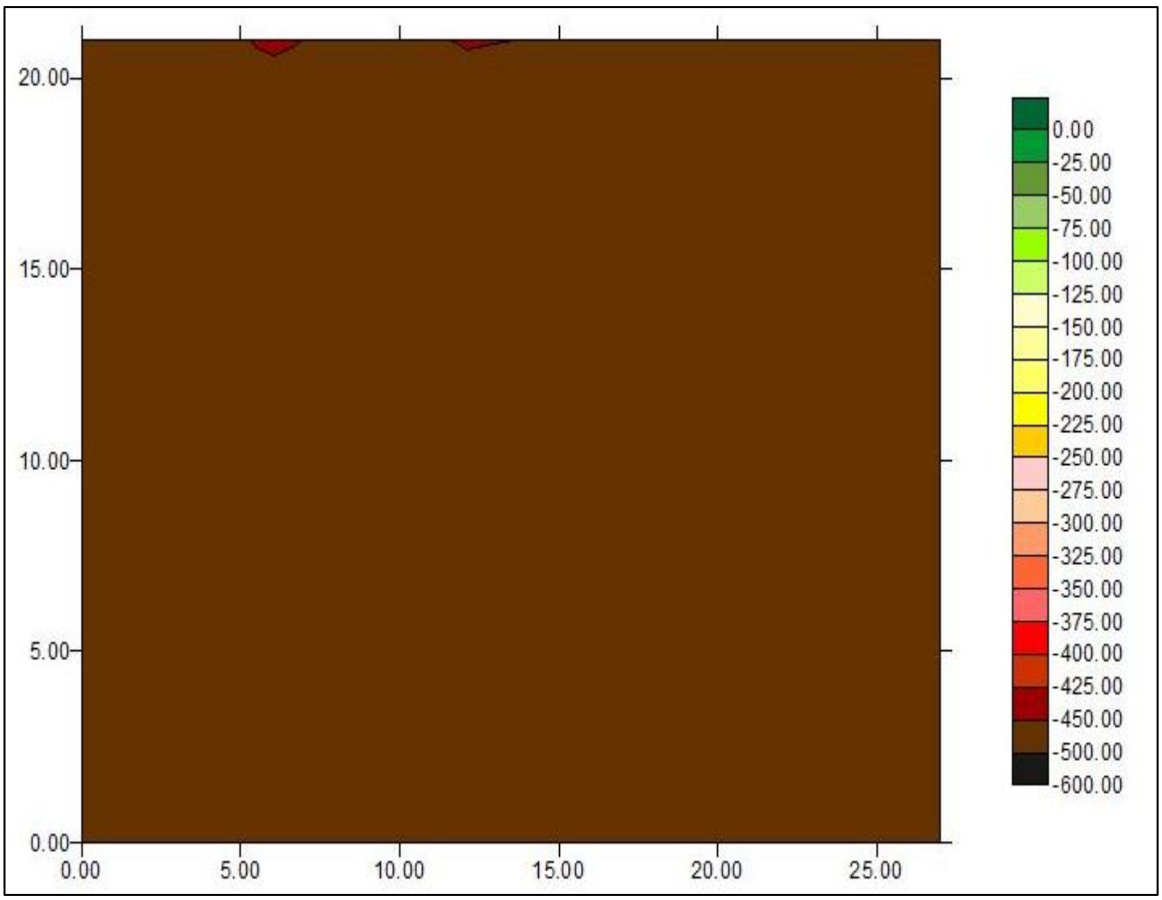


Figure E.44 Shaft 16, test 2 data map

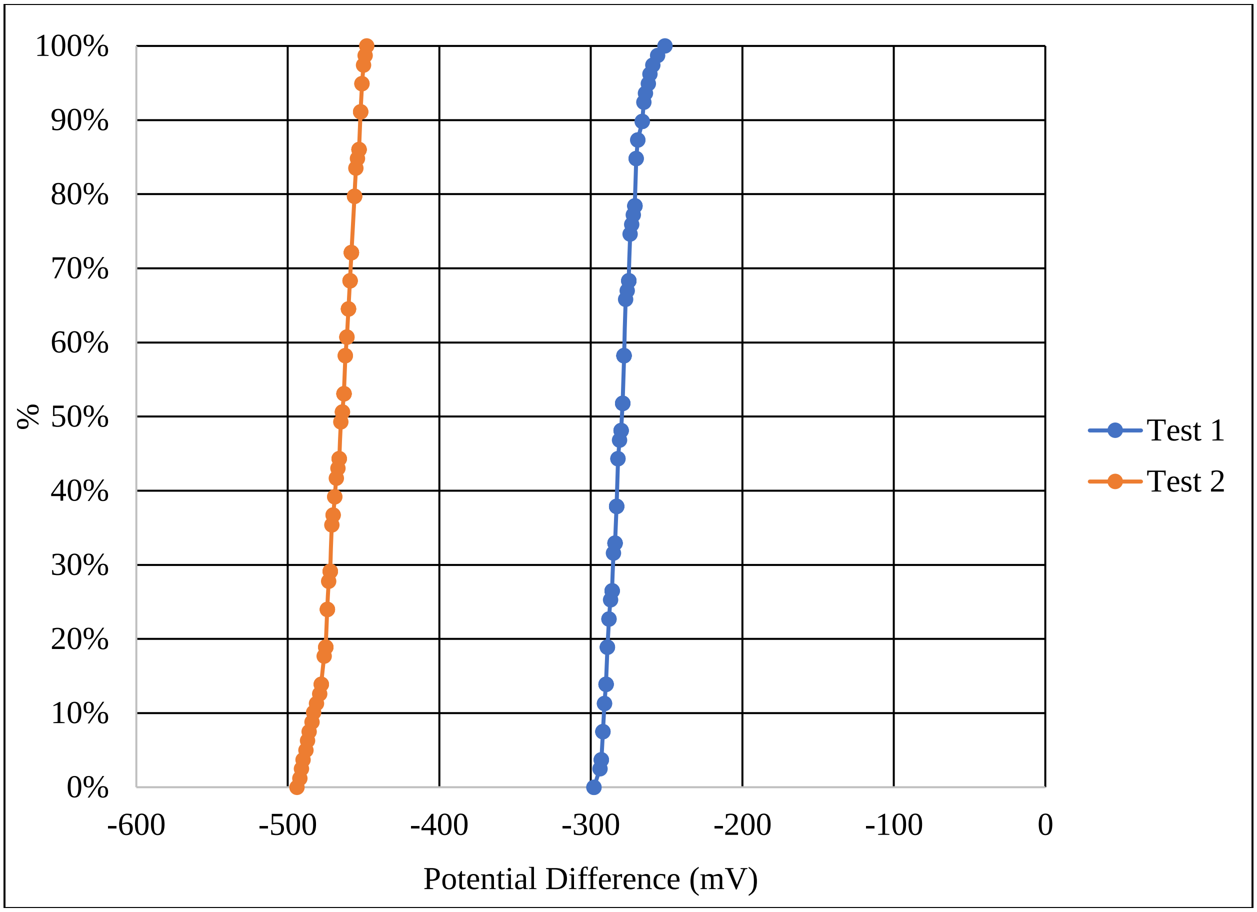


Figure E.45 Shaft 16 percentile distributions

Table E.31 Shaft 17, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-309	-303	-301	-301	-299	-305	-304	-304
3"	-306	-306	-303	-303	-302	-299	-303	-301
6"	-309	-307	-303	-308	-304	-299	-298	-300
9"	-306	-305	-302	-299	-293	-291	-294	-293
12"	-312	-310	-304	-298	-298	-292	-296	-290
15"	-319	-312	-304	-303	-298	-295	-295	-288
18"	-322	-311	-302	-296	-293	-290	-291	-288
21"	-319	-310	-301	-300	-292	-288	-287	-285
24"	-311	-310	-302	-297	-291	-287	-285	-286
27"	-305	-306	-297	-295	-290	-286	-285	-280

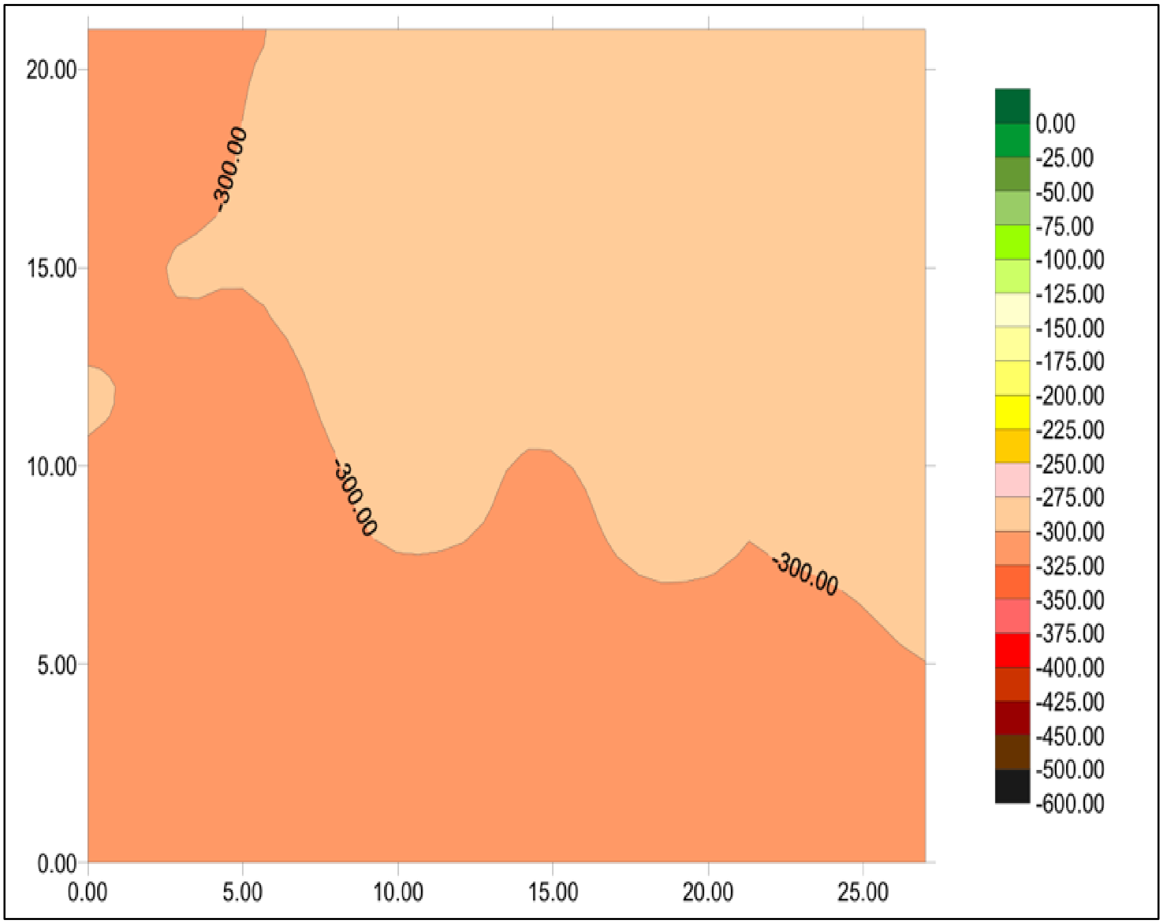


Figure E.46 Shaft 17, test 1 data map

Table E.32 Shaft 17, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-365	-362	-351	-345	-340	-327	-320	-315
3"	-363	-363	-354	-362	-332	-327	-318	-318
6"	-371	-366	-359	-348	-338	-331	-324	-321
9"	-374	-371	-362	-356	-347	-342	-330	-323
12"	-384	-378	-364	-358	-351	-342	-335	-330
15"	-380	-375	-365	-357	-347	-343	-337	-336
18"	-377	-371	-362	-355	-349	-345	-341	-334
21"	-368	-366	-357	-347	-342	-336	-333	-328
24"	-360	-358	-350	-339	-331	-325	-321	-315
27"	-358	-357	-347	-338	-335	-325	-322	-314

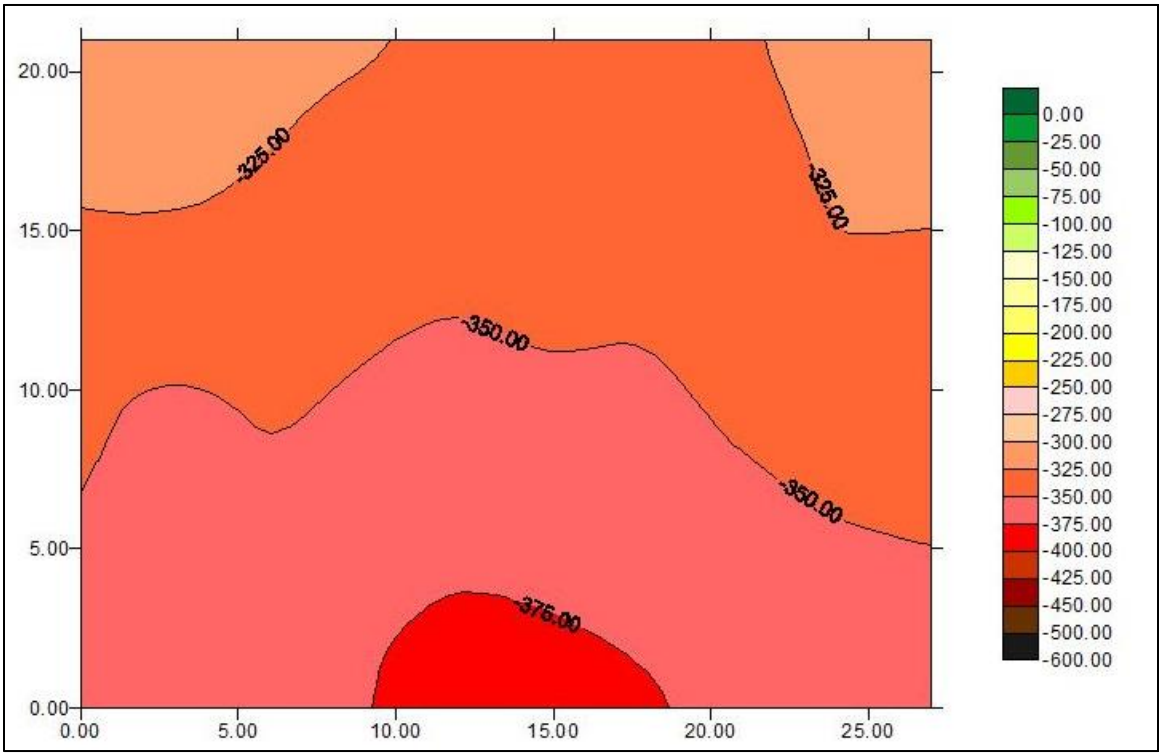


Figure E.47 Shaft 17, test 2 data map

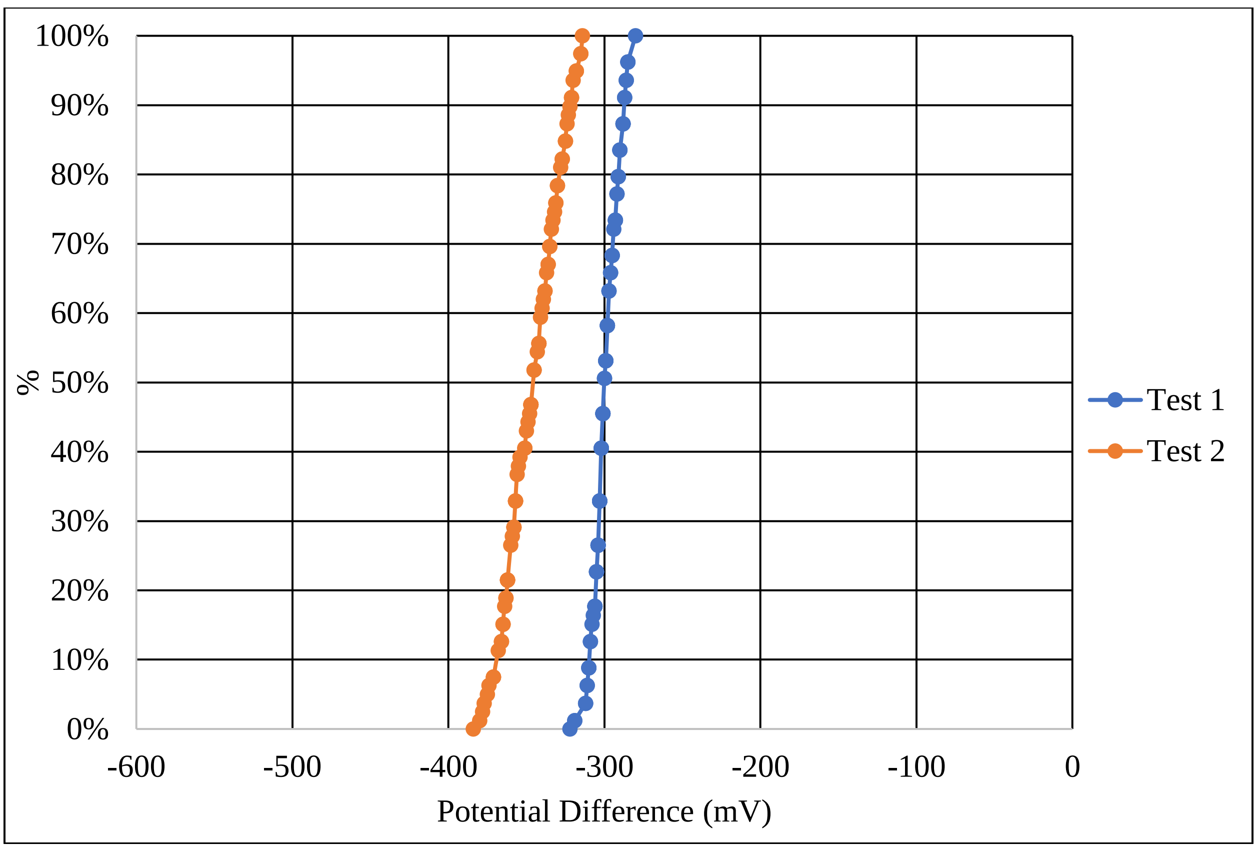


Figure E.48 Shaft 17 percentile distributions

Table E.33 Shaft 18, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-301	-304	-291	-291	-284	-295	-300	-302
3"	-300	-302	-290	-285	-282	-290	-299	-307
6"	-299	-303	-291	-284	-282	-292	-299	-306
9"	-294	-297	-290	-286	-284	-293	-299	-304
12"	-295	-293	-285	-281	-283	-290	-293	-300
15"	-296	-298	-285	-282	-285	-291	-291	-298
18"	-296	-296	-288	-283	-292	-293	-295	-292
21"	-301	-297	-291	-288	-294	-293	-297	-303
24"	-302	-298	-291	-290	-293	-287	-294	-297
27"	-299	-297	-291	-297	-302	-295	-297	-301

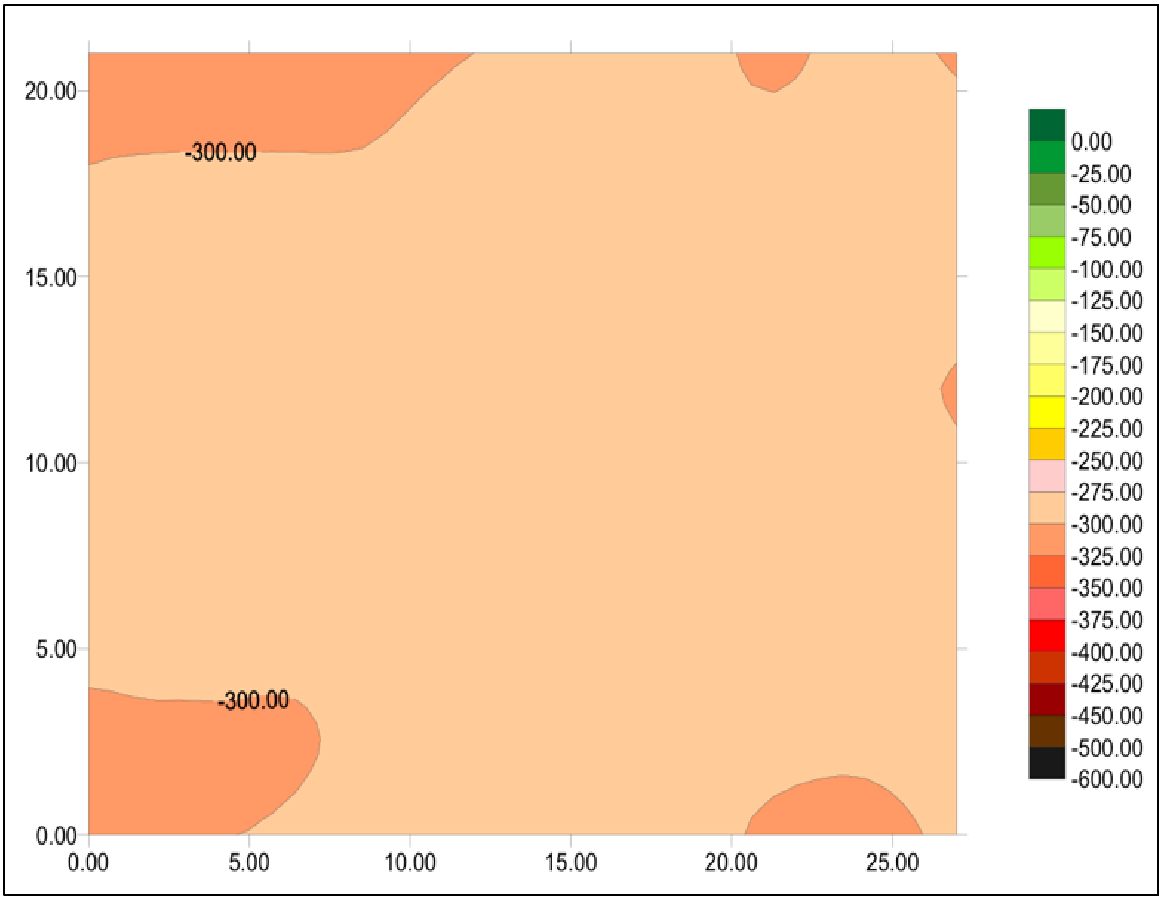


Figure E.49 Shaft 18, test 1 data map

Table E.34 Shaft 18, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-337	-337	-339	-335	-329	-318	-308	-289
3"	-336	-337	-341	-339	-327	-319	-312	-287
6"	-338	-341	-338	-339	-329	-326	-318	-294
9"	-343	-342	-346	-342	-334	-332	-323	-295
12"	-356	-348	-348	-337	-339	-330	-323	-300
15"	-352	-350	-339	-339	-333	-325	-316	-292
18"	-358	-354	-342	-342	-340	-332	-320	-297
21"	-365	-352	-349	-349	-338	-332	-320	-301
24"	-376	-368	-361	-361	-345	-331	-316	-293
27"	-379	-379	-375	-357	-344	-337	-321	-295

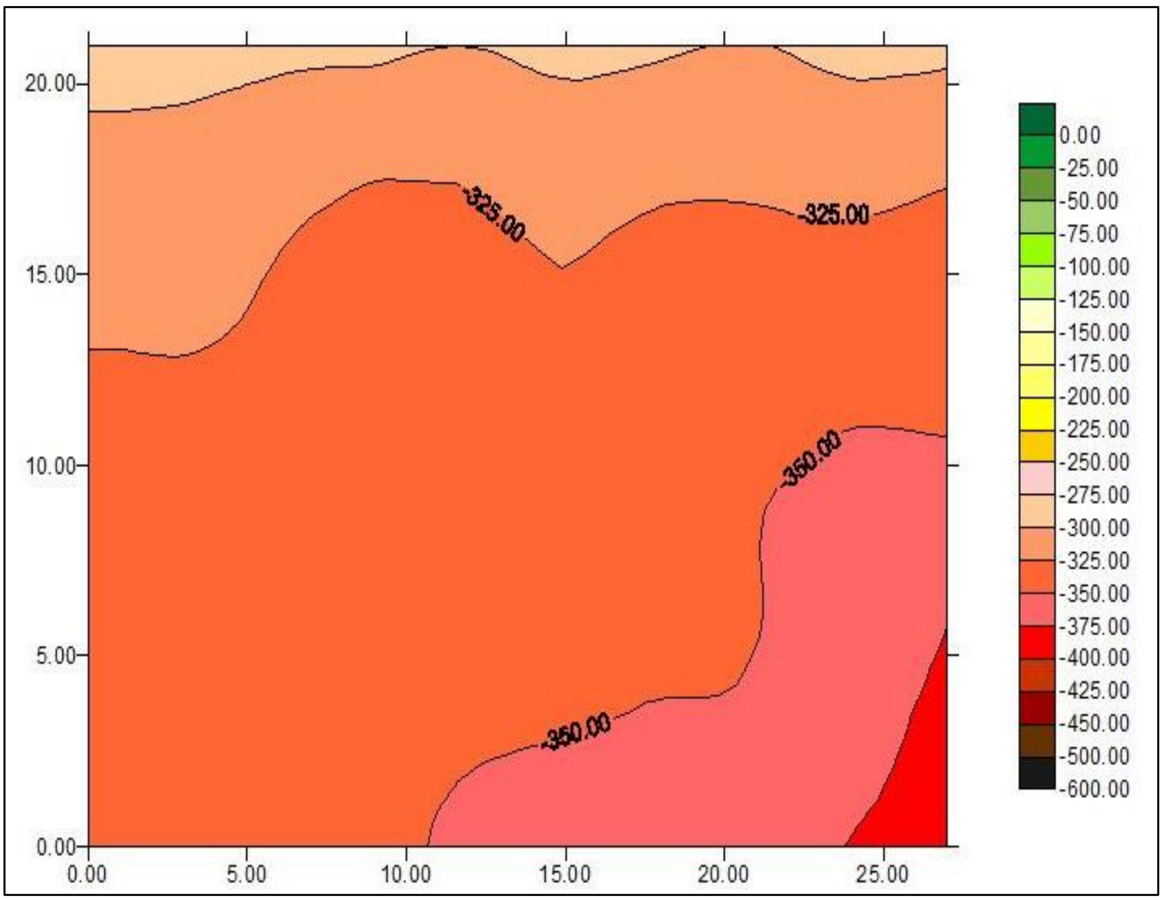


Figure E.50 Shaft 18, test 2 data map

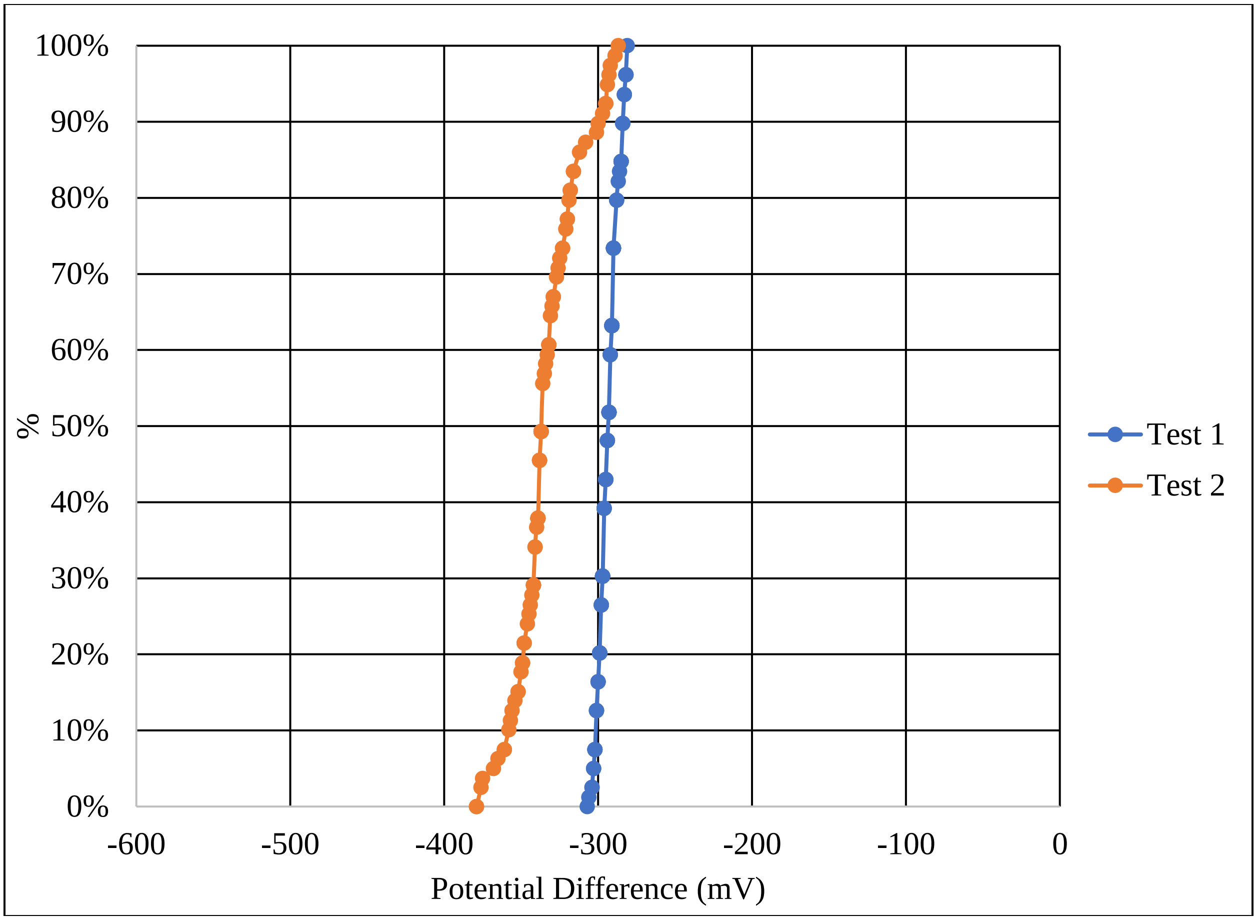


Figure E.51 Shaft 18 percentile distributions

Table E.35 Shaft 19, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-243	-238	-234	-231	-230	-225	-229	-234
3"	-242	-242	-238	-234	-235	-230	-234	-240
6"	-246	-246	-238	-235	-231	-235	-241	-247
9"	-256	-250	-242	-234	-235	-238	-243	-255
12"	-258	-251	-241	-232	-230	-235	-248	-265
15"	-265	-259	-249	-243	-238	-240	-248	-259
18"	-263	-258	-247	-243	-240	-241	-245	-249
21"	-260	-258	-250	-243	-244	-243	-246	-245
24"	-268	-263	-255	-247	-244	-245	-248	-249
27"	-275	-263	-250	-242	-240	-239	-246	-252

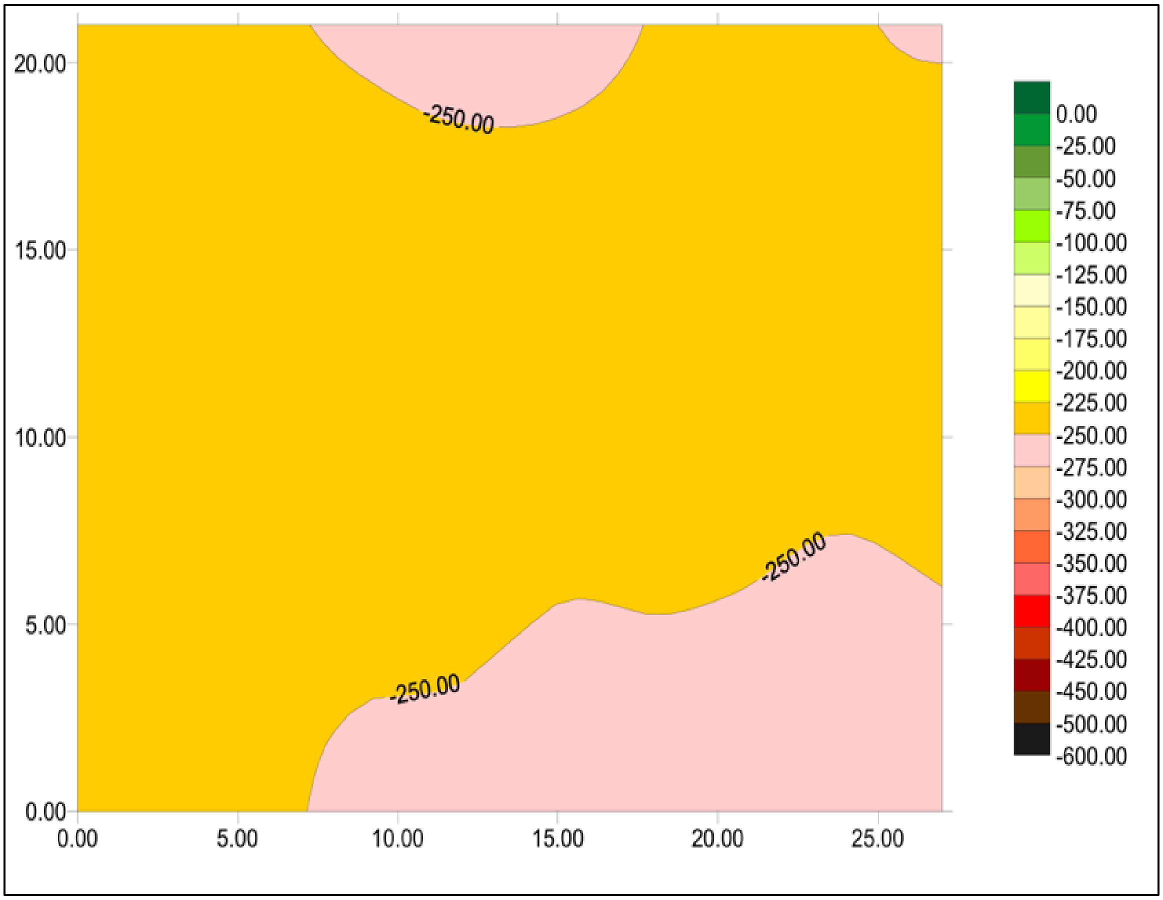


Figure E.52 Shaft 19, test 1 data map

Table E.36 Shaft 19, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-318	-302	-285	-278	-269	-263	-258	-258
3"	-310	-298	-285	-275	-268	-263	-259	-252
6"	-297	-292	-282	-274	-270	-264	-256	-249
9"	-291	-290	-284	-272	-269	-262	-256	-244
12"	-290	-292	-286	-278	-275	-269	-260	-244
15"	-290	-287	-282	-275	-267	-261	-256	-242
18"	-287	-283	-276	-272	-262	-253	-253	-236
21"	-281	-285	-288	-270	-262	-255	-260	-238
24"	-275	-270	-275	-273	-263	-257	-248	-235
27"	-272	-267	-260	-255	-244	-238	-235	-228

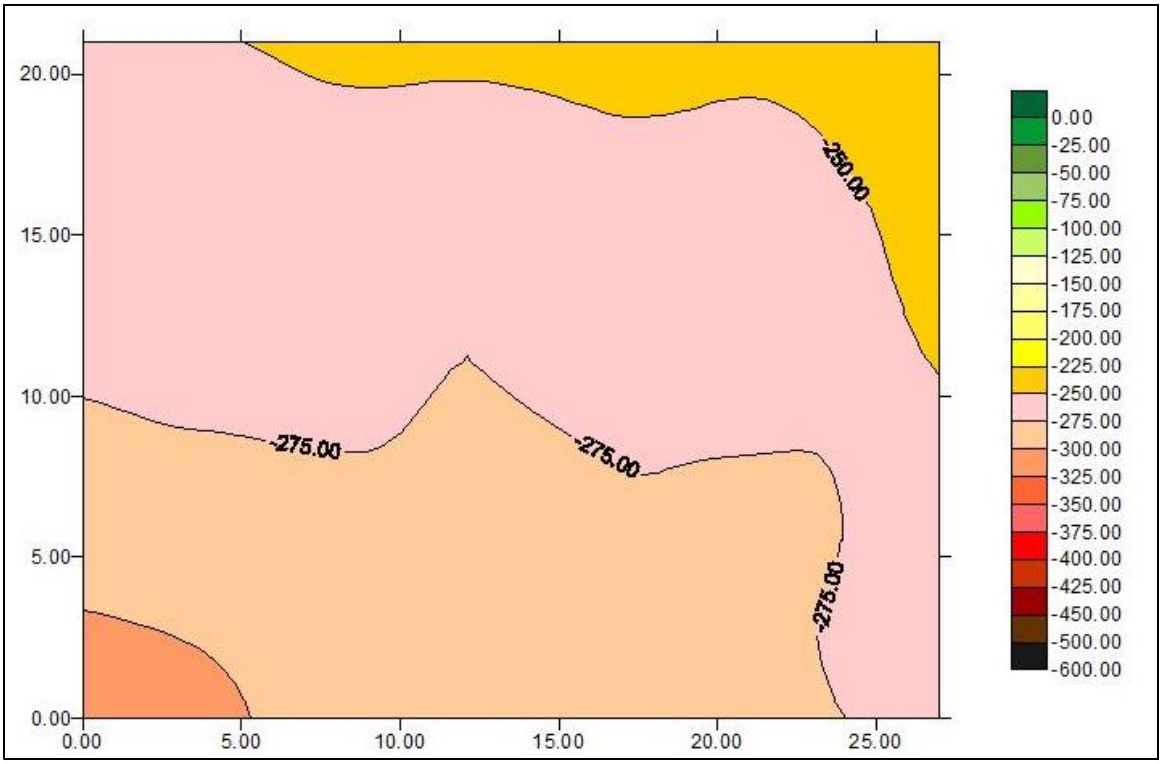


Figure E.53 Shaft 19, test 2 data map

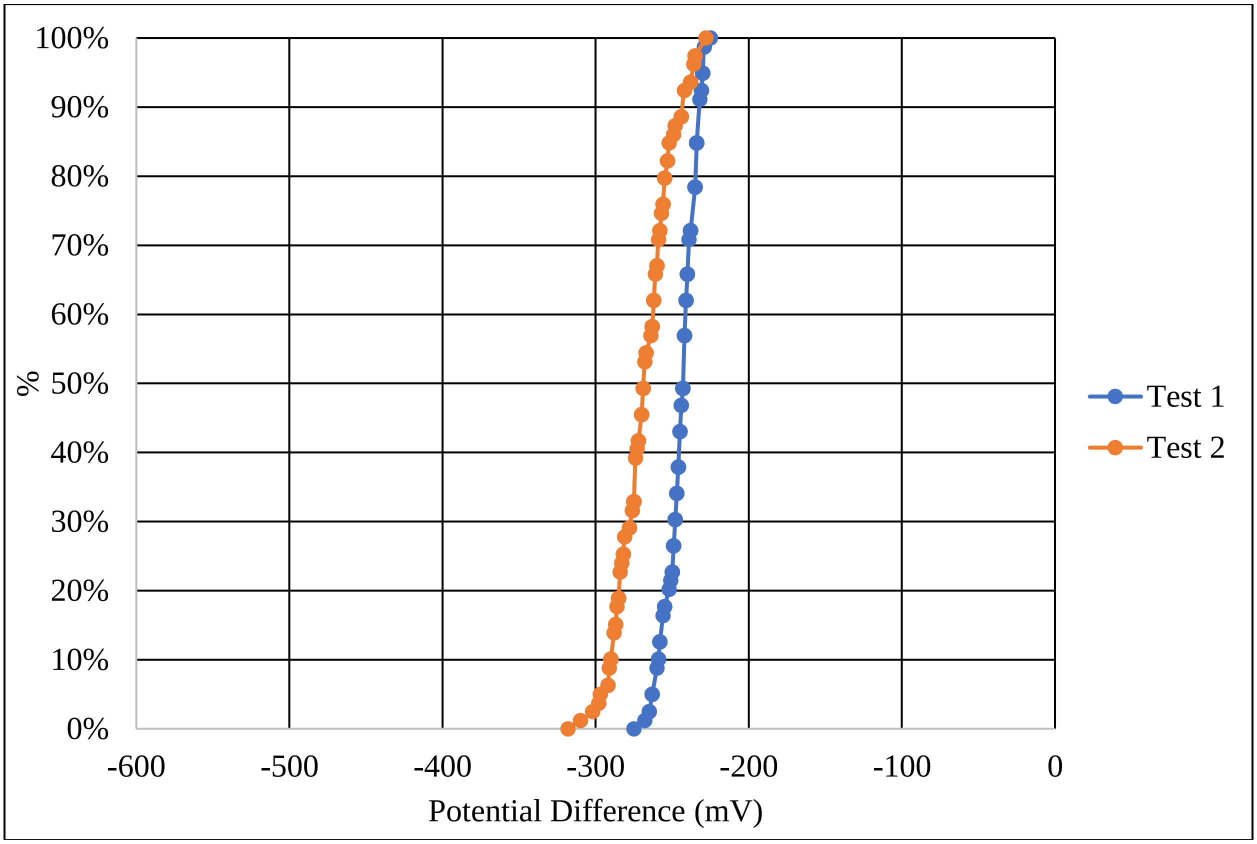


Figure E.54 Shaft 19 percentile distributions

Table E.37 Shaft 20, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-239	-234	-225	-231	-235	-238	-243	-235
3"	-222	-231	-231	-231	-236	-236	-244	-247
6"	-239	-236	-235	-238	-239	-239	-241	-237
9"	-233	-233	-240	-241	-242	-242	-238	-242
12"	-241	-242	-241	-241	-242	-242	-242	-245
15"	-248	-244	-248	-245	-239	-239	-242	-243
18"	-253	-250	-251	-245	-245	-245	-242	-241
21"	-253	-252	-251	-246	-246	-246	-254	-245
24"	-251	-253	-250	-247	-247	-247	-250	-250
27"	-259	-259	-254	-253	-255	-255	-251	-249

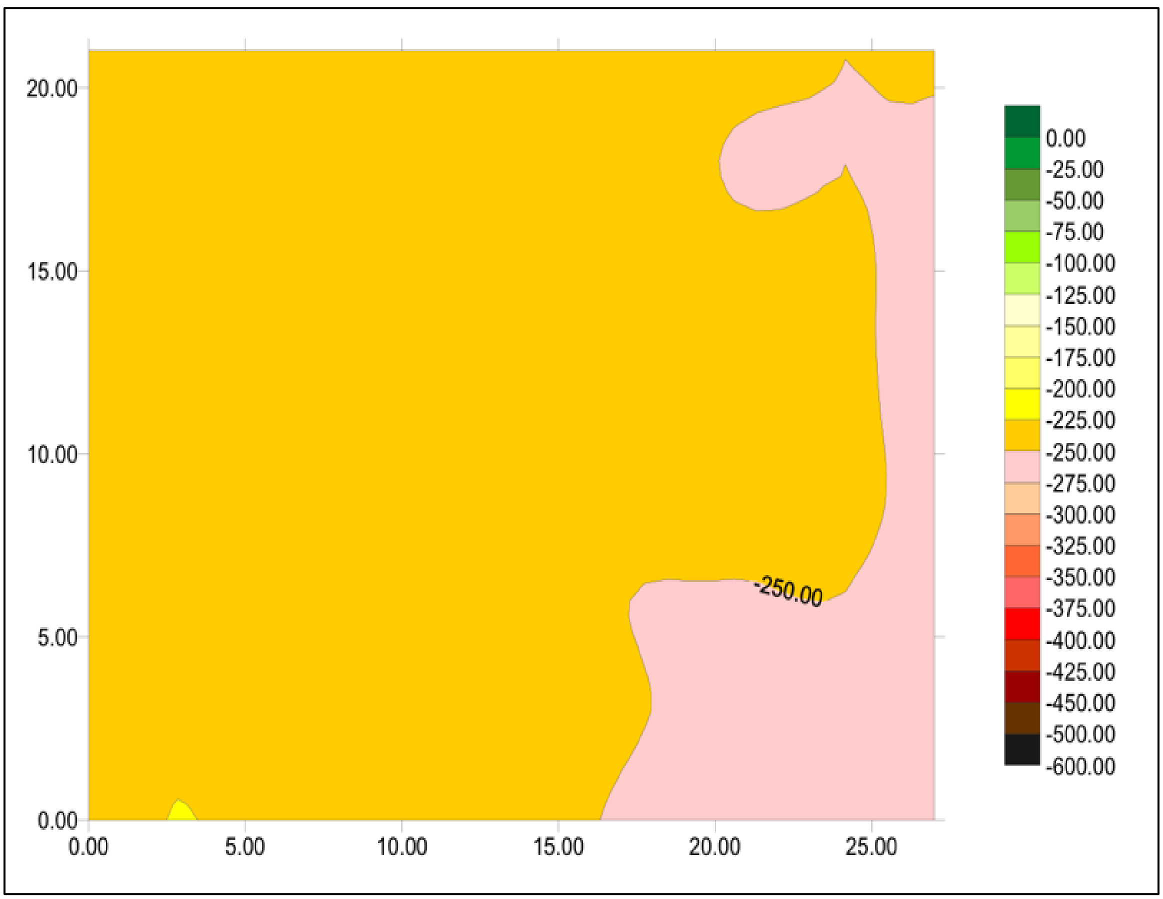


Figure E.55 Shaft 20, test 1 data map

Table E.38 Shaft 20, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-338	-433	-439	-448	-450	-451	-452	-439
3"	-470	-465	-456	-462	-466	-469	-469	-458
6"	-482	-476	-478	-476	-482	-484	-492	-485
9"	-484	-479	-476	-478	-483	-483	-507	-503
12"	-496	-482	-476	-476	-482	-475	-509	-501
15"	-491	-479	-475	-478	-479	-478	-498	-499
18"	-481	-469	-470	-468	-476	-480	-488	-485
21"	-466	-463	-462	-462	-461	-470	-473	-466
24"	-452	-449	-446	-451	-454	-467	-463	-457
27"	-435	-440	-441	-443	-454	-459	-465	-457

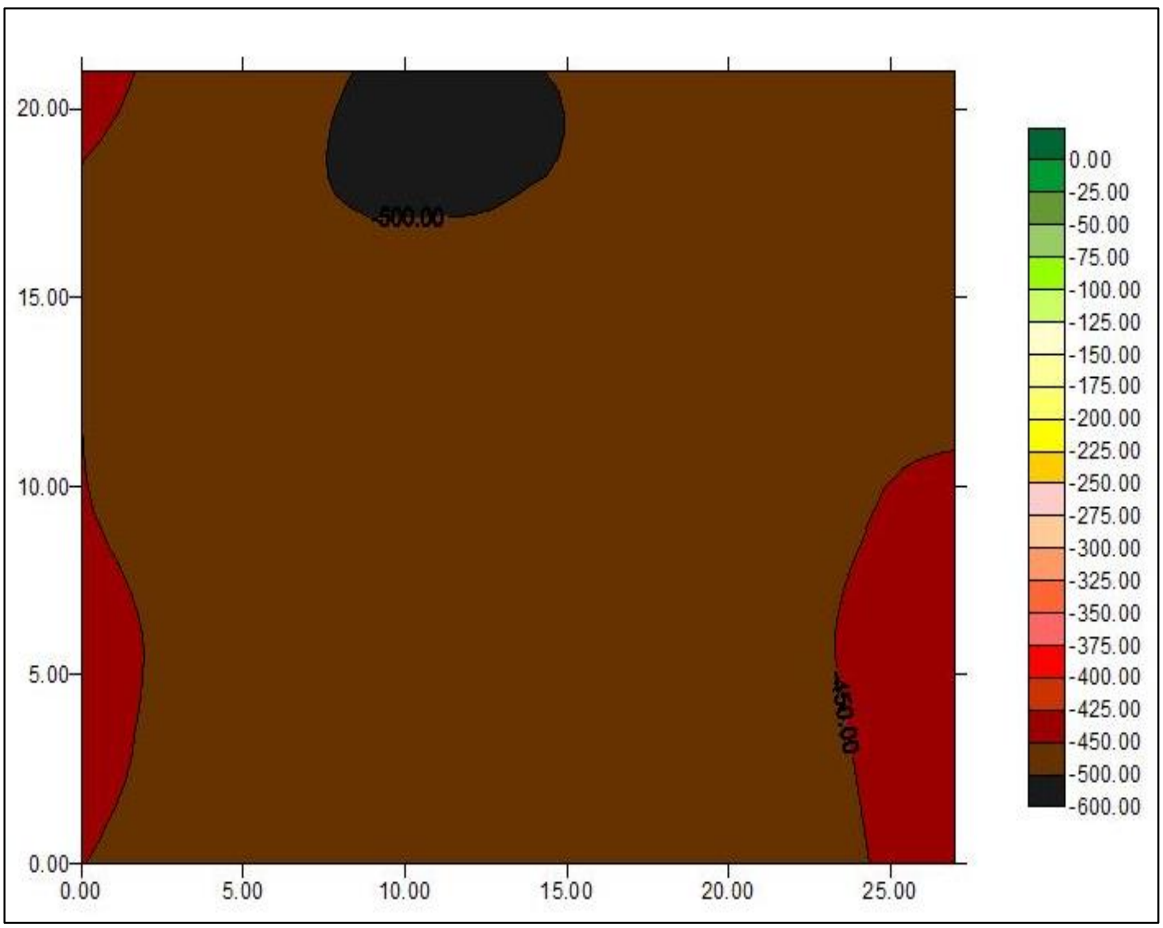


Figure E.56 Shaft 20, test 2 data map

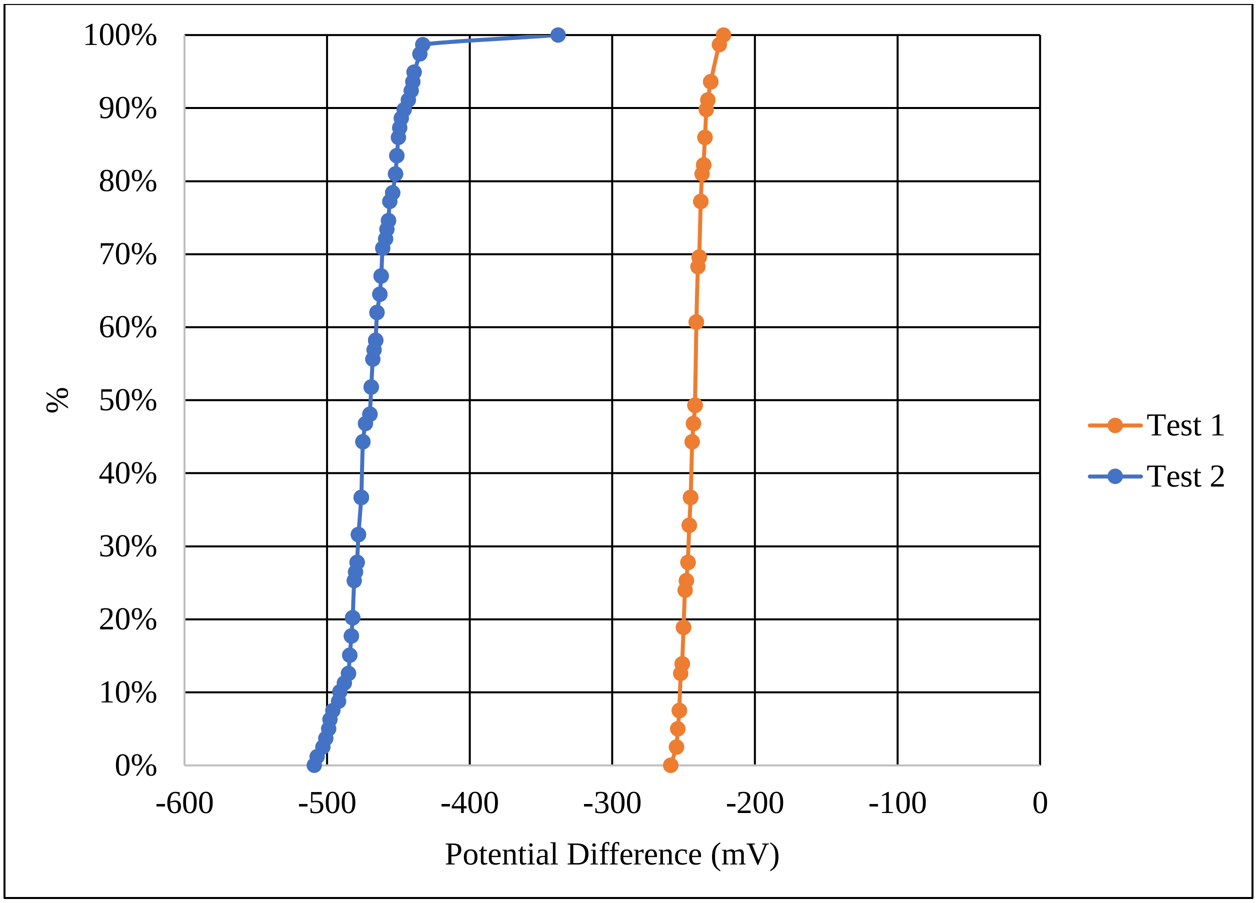


Figure E.57 Shaft 20 percentile distributions

Table E.39 Shaft 21, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-491	-492	-495	-518	-552	-534	-548	-596
3"	-482	-489	-489	-493	-533	-524	-527	-500
6"	-480	-481	-489	-484	-508	-493	-509	-495
9"	-491	-489	-488	-497	-522	-493	-516	-502
12"	-485	-464	-488	-486	-511	-501	-522	-506
15"	-495	-494	-488	-492	-496	-494	-502	-511
18"	-500	-497	-508	-513	-536	-524	-529	-524
21"	-504	-505	-516	-528	-560	-538	-539	-516
24"	-508	-503	-508	-522	-540	-547	-556	-520
27"	-510	-510	-536	-538	-573	-558	-564	-542

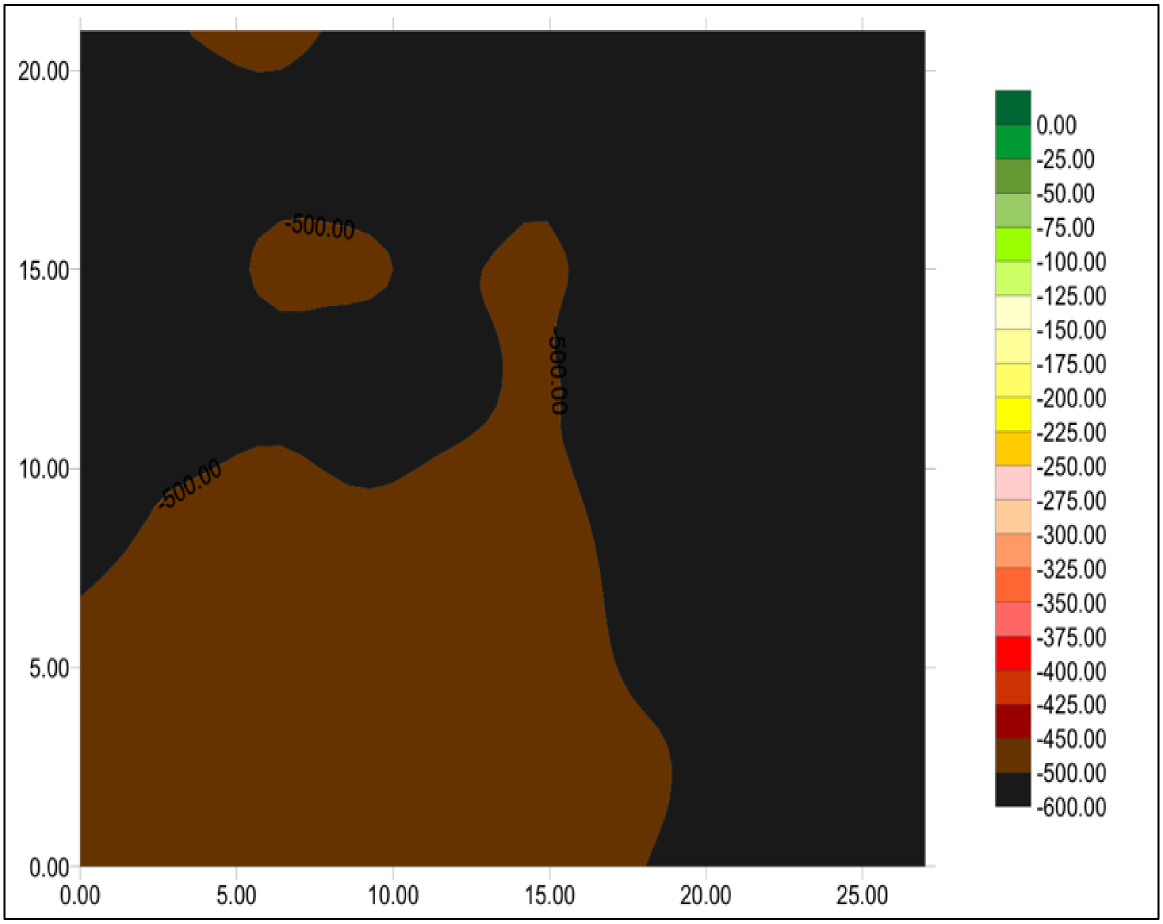


Figure E.58 Shaft 21, test 1 data map

Table E.40 Shaft 21, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-464	-478	-483	-483	-494	-491	-499	-491
3"	-478	-477	-480	-483	-478	-478	-483	-494
6"	-488	-493	-494	-477	-490	-482	-505	-486
9"	-497	-499	-514	-487	-497	-481	-505	-494
12"	-514	-513	-505	-495	-492	-478	-479	-466
15"	-512	-512	-507	-500	-513	-490	-504	-478
18"	-489	-491	-498	-501	-518	-499	-501	-490
21"	-467	-473	-487	-487	-493	-492	-495	-481
24"	-460	-465	-469	-477	-518	-486	-482	-466
27"	-432	-447	-459	-476	-498	-485	-493	-469

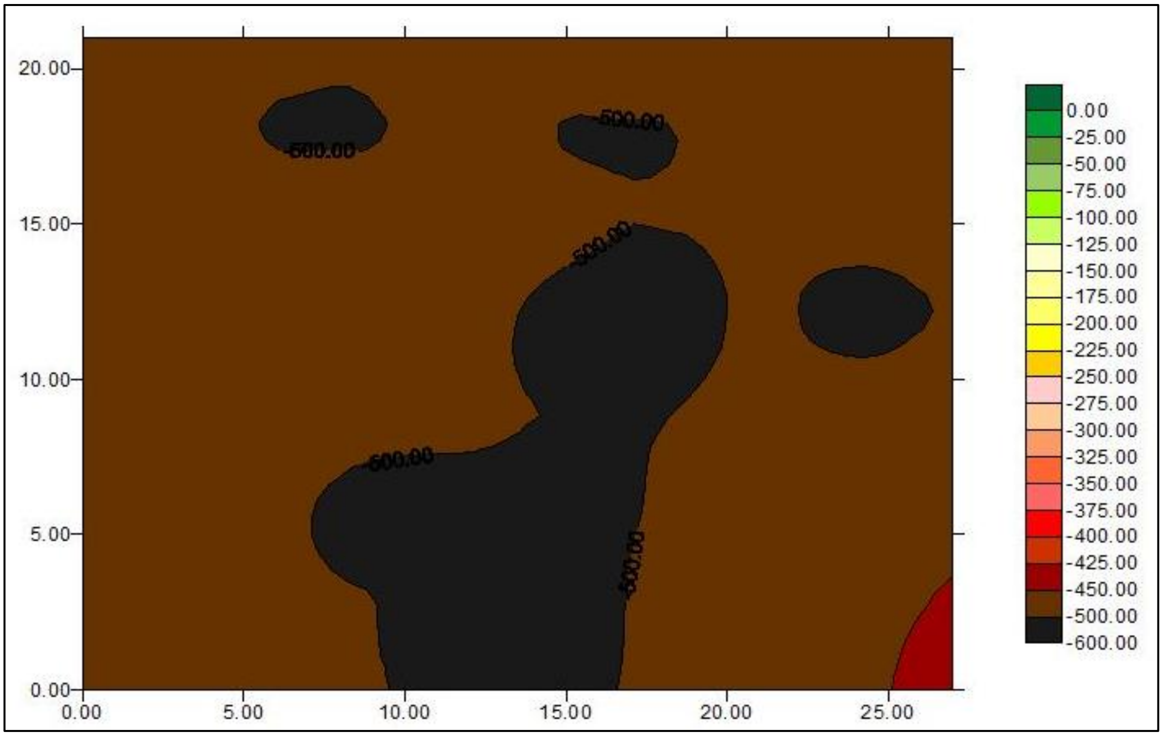


Figure E.59 Shaft 21, test 2 data map

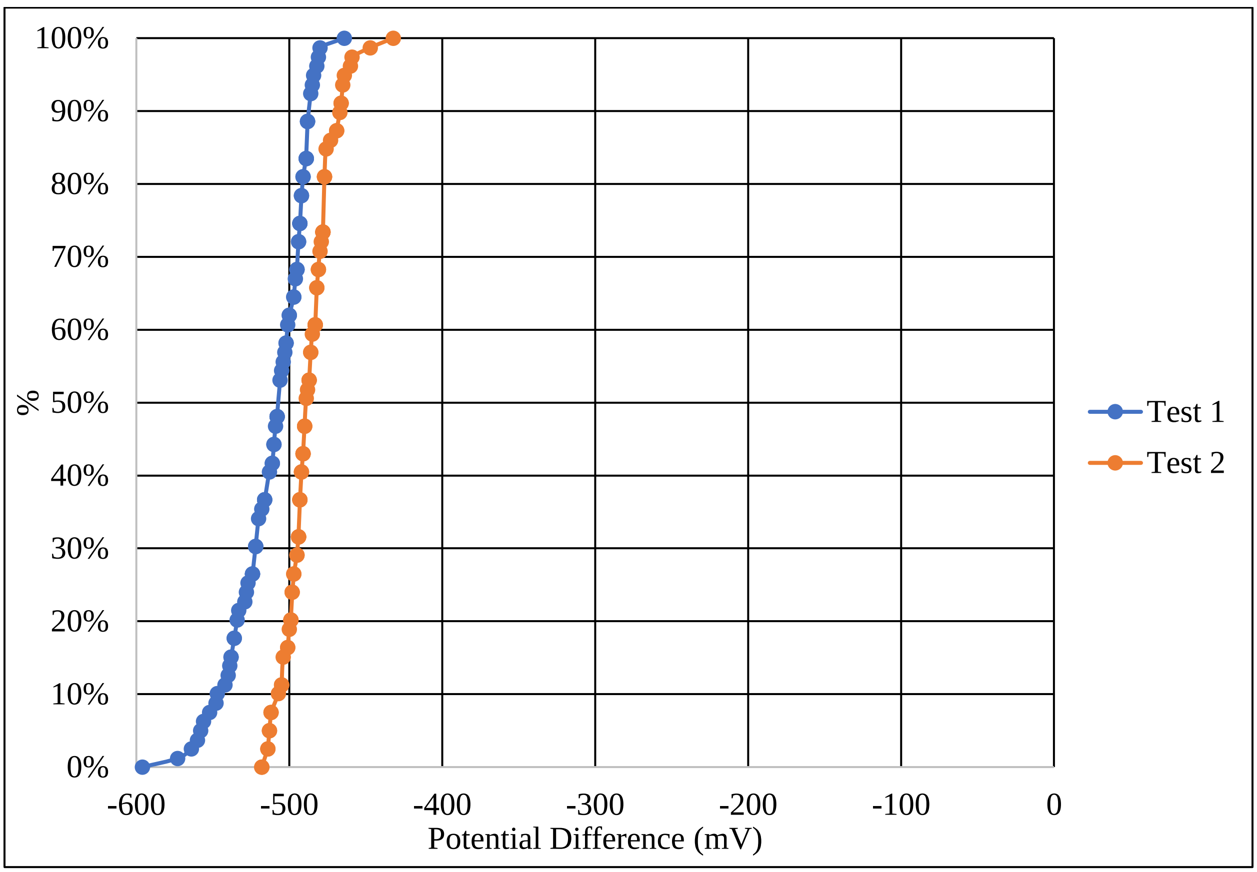


Figure E.60 Shaft 21 percentile distributions

Table E.41 Shaft 22, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-268	-264	-248	-244	-243	-236	-230	-243
3"	-265	-273	-255	-247	-237	-237	-241	-248
6"	-281	-277	-268	-254	-244	-240	-248	-251
9"	-272	-281	-276	-261	-255	-252	-248	-250
12"	-263	-272	-261	-246	-238	-238	-237	-239
15"	-257	-259	-248	-240	-236	-234	-241	-246
18"	-261	-260	-247	-237	-237	-239	-244	-252
21"	-277	-278	-262	-249	-245	-247	-256	-269
24"	-279	-279	-266	-249	-245	-251	-259	-277
27"	-280	-279	-265	-250	246	-249	-256	-268

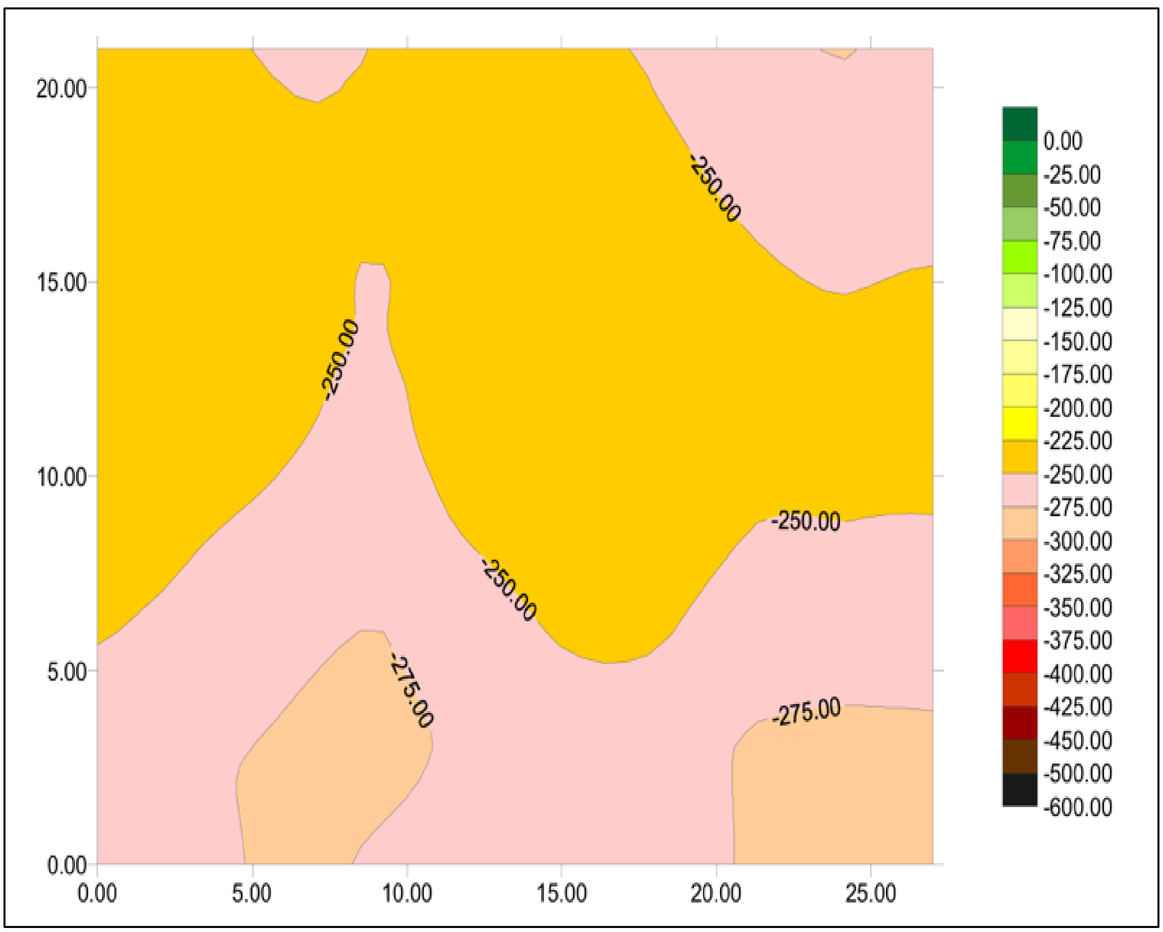


Figure E.61 Shaft 22, test 1 data map

Table E.42 Shaft 22, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-275	-277	-273	-251	-244	-239	-234	-227
3"	-278	-277	-268	-247	-243	-235	-232	-220
6"	-281	-283	-279	-258	-247	-245	-238	-230
9"	-285	-287	-281	-264	-254	-240	-237	-229
12"	-276	-284	-275	-269	-259	-246	-238	-229
15"	-273	-280	-296	-281	-267	-260	-246	-235
18"	-283	-285	-291	-286	-264	-264	-254	-243
21"	-291	-299	-299	-285	-271	-264	-260	-240
24"	-296	-301	-292	-280	-269	-259	-250	-240
27"	-297	-302	-287	-267	-258	-250	-248	-242

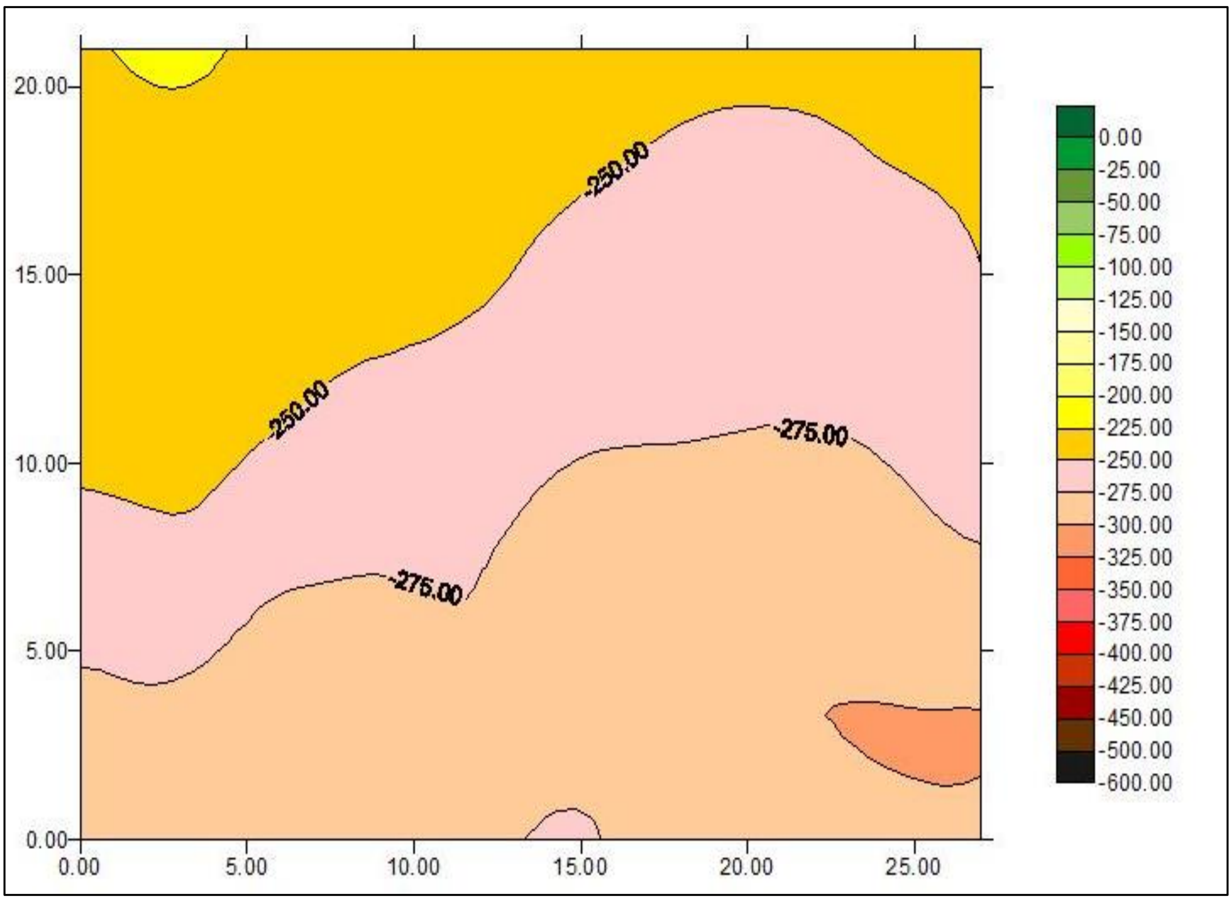


Figure E.62 Shaft 22, test 2 data map

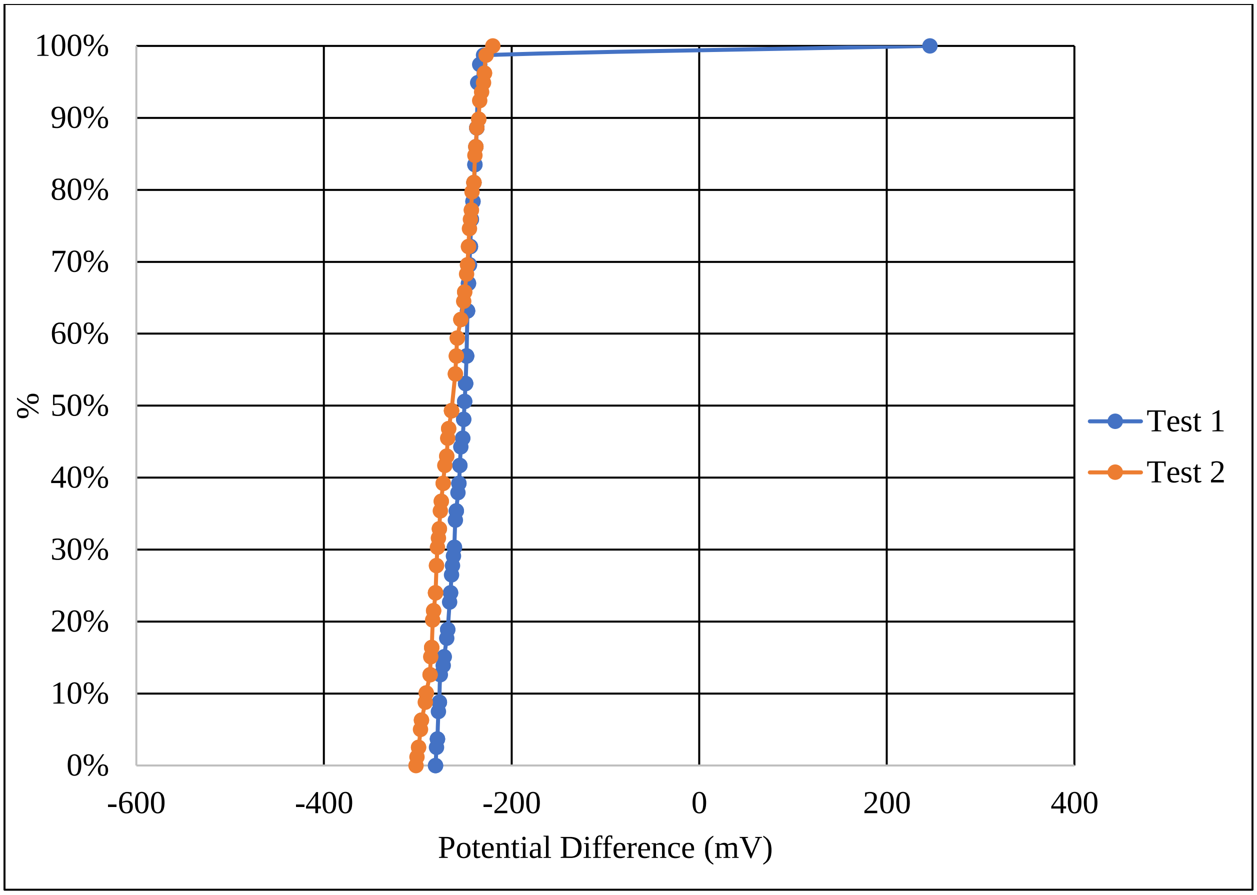


Figure E.63 Shaft 22 percentile distributions

Table E.43 Shaft 23, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-273	-265	-264	-260	-258	-262	-263	-269
3"	-276	-263	-263	-260	-258	-262	-268	-273
6"	-272	-262	-263	-257	-258	-263	-270	-260
9"	-271	-260	-261	-255	-255	-282	-269	-261
12"	-271	-259	-258	-254	-253	-283	-271	-260
15"	-266	-256	-251	-248	-252	-255	-264	-277
18"	-265	-255	-253	-247	-249	-250	-258	-269
21"	-261	-254	-246	-243	-242	-246	-250	-257
24"	-261	-252	-246	-241	-236	-241	-245	-254
27"	-261	-254	-246	-237	-233	-238	-243	-250

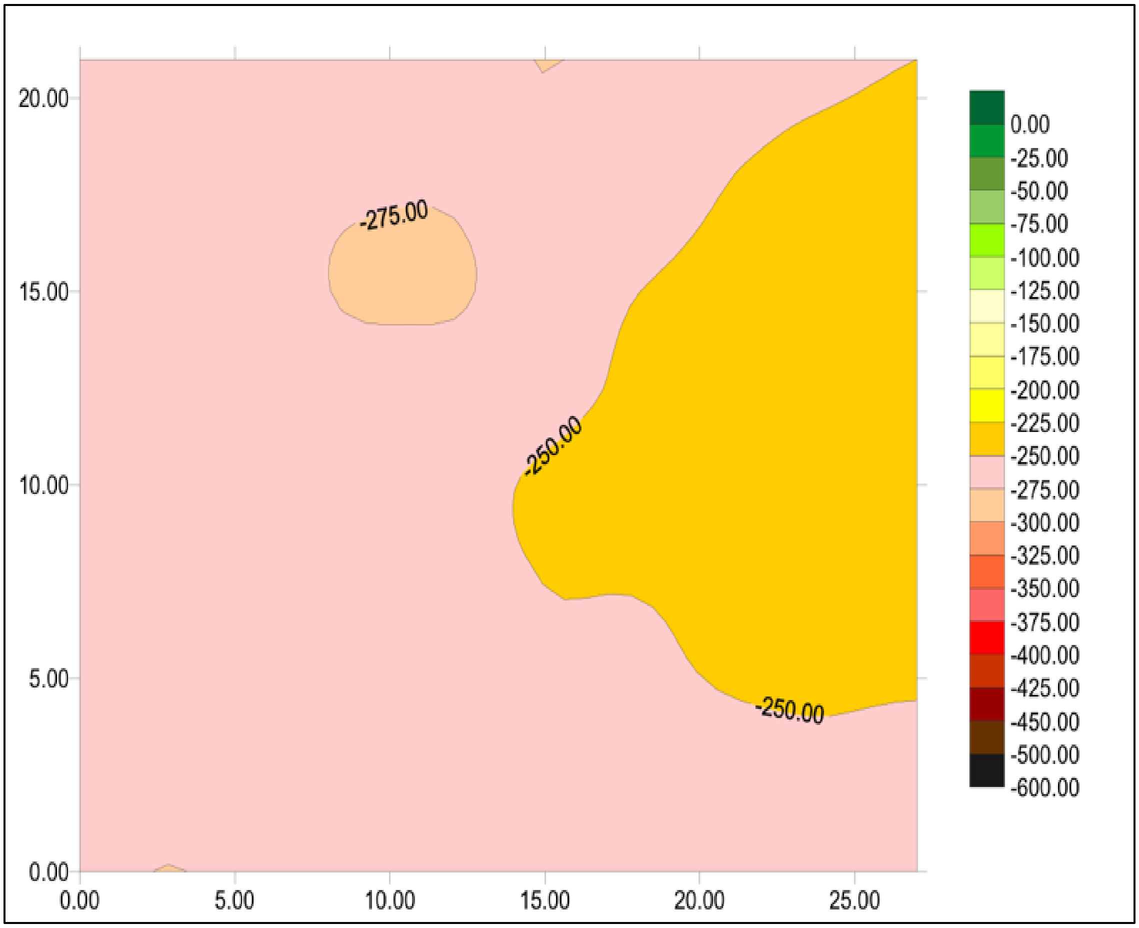


Figure E.64 Shaft 23, test 1 data map

Table E.44 Shaft 23, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-451	-416	-384	-368	-353	-349	-350	-360
3"	-448	-417	-391	-374	-358	-350	-348	-354
6"	-440	-414	-392	-374	-360	-353	-348	-349
9"	-438	-413	-392	-376	-361	-354	-348	-345
12"	-438	-417	-396	-378	-361	-352	-347	-346
15"	-449	-422	-403	-381	-366	-355	-348	-351
18"	-454	-424	-400	-382	-365	-353	-348	-355
21"	-452	-424	-398	-379	-362	-350	-349	-352
24"	-439	-419	-395	-377	-360	-350	-346	-347
27"	-430	-412	-392	-374	-357	-349	-344	-342

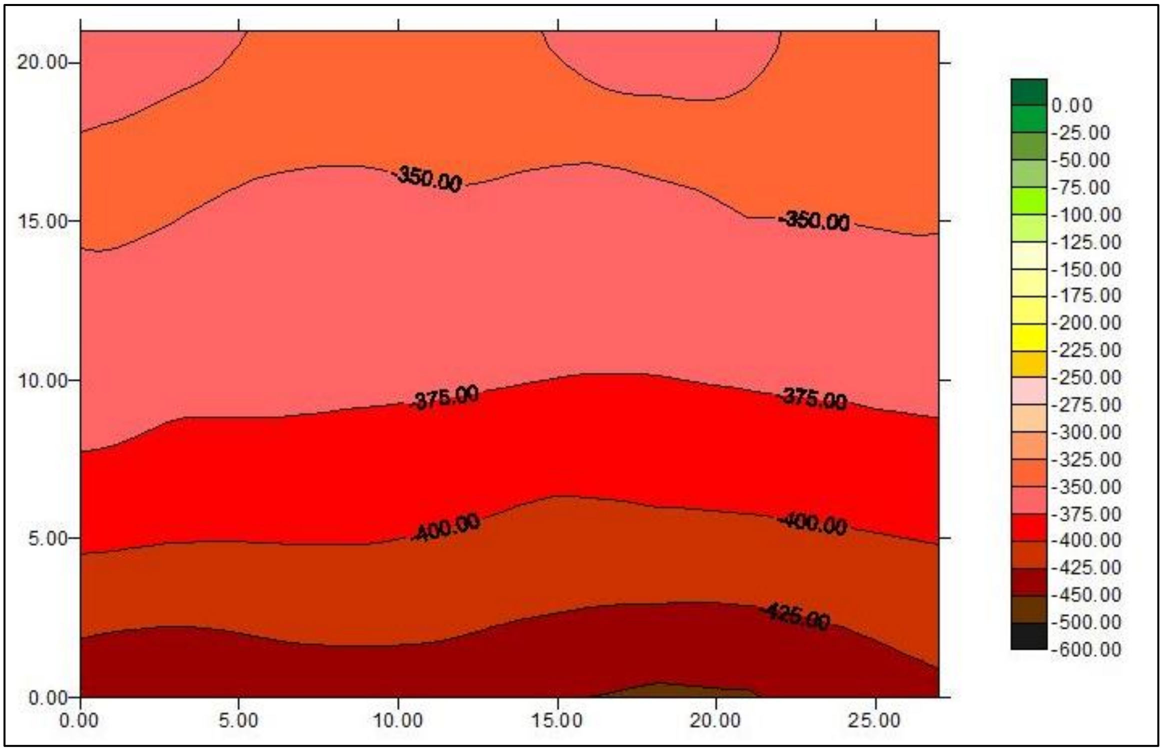


Figure E.65 Shaft 23, test 2 data map

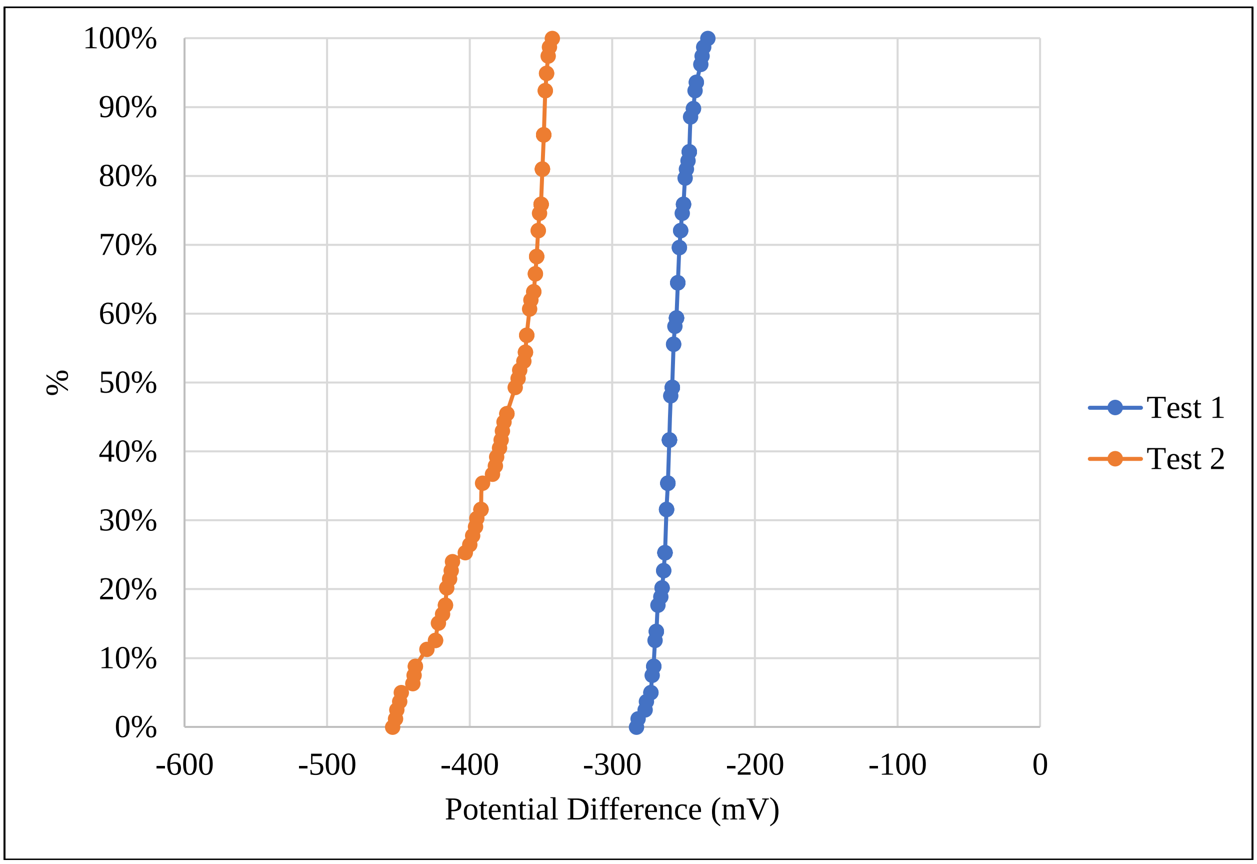


Figure E.66 Shaft 23 percentile distributions

Table E.45 Shaft 24, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-450	-453	-449	-452	-464	-446	-413	-393
3"	-439	-447	-443	-445	-456	-441	-410	-394
6"	-436	-443	-441	-430	-464	-443	-416	-400
9"	-433	-441	-442	-442	-469	-455	-425	-404
12"	-429	-436	-438	-444	-464	-454	-423	-404
15"	-423	-429	-437	-444	-465	-456	-424	-407
18"	-414	-421	-425	-434	-454	-433	-414	-403
21"	-411	-412	-421	-425	-437	-422	-398	-394
24"	-410	-408	-408	-408	-412	-401	-390	-389
27"	-404	-399	-395	-405	-403	-395	-389	-384

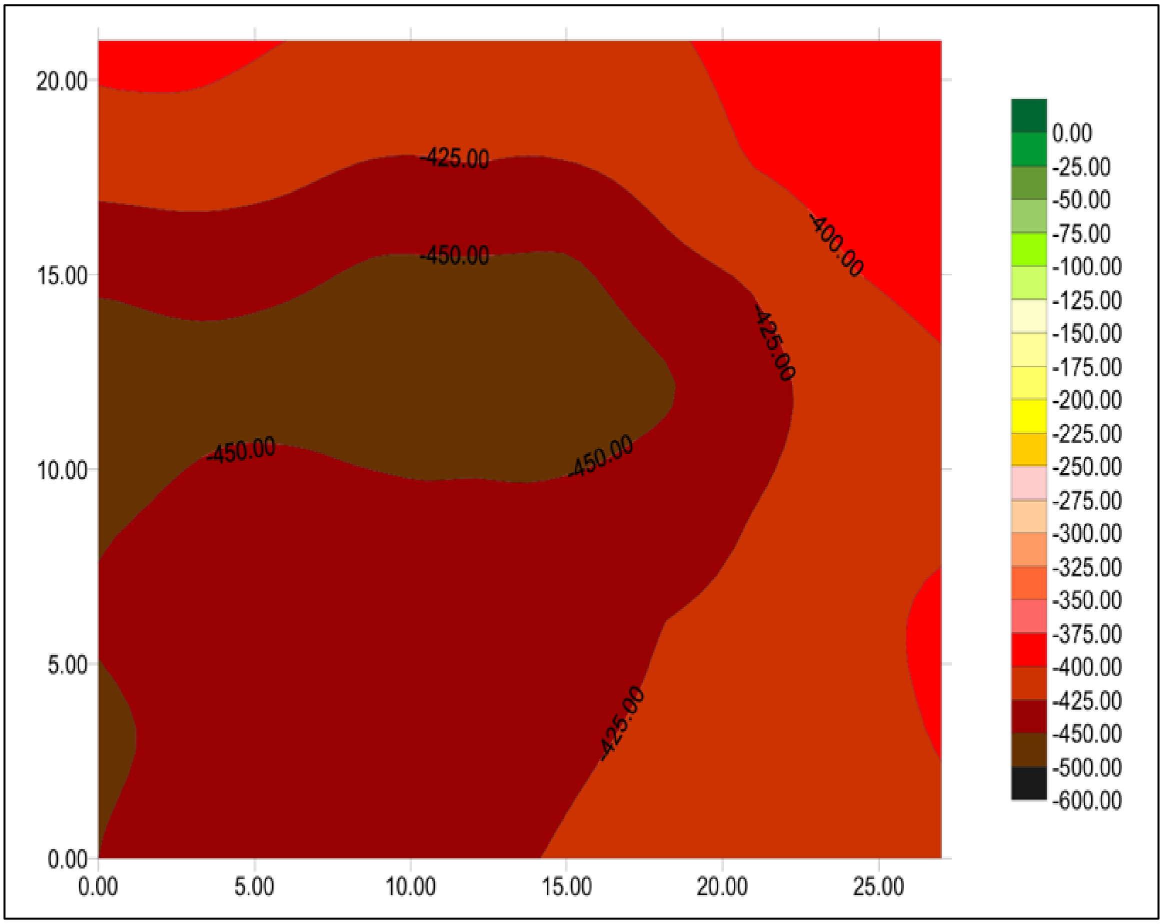


Figure E.67 Shaft 24, test 1 data map

Table E.46 Shaft 24, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-433	-444	-459	-469	-475	-452	-414	-383
3"	-437	-446	-451	-468	-473	-450	-412	-383
6"	-438	-448	-458	-464	-464	-450	-414	-388
9"	-443	-452	-457	-459	-457	-439	-408	-382
12"	-444	-449	-452	-459	-451	-430	-398	-371
15"	-451	-457	-460	-450	-428	-393	-391	-361
18"	-445	-451	-459	-459	-451	-425	-382	-354
21"	-449	-453	-462	-462	-453	-428	-386	-335
24"	-450	-454	-463	-463	-454	-429	-389	-360
27"	-454	-455	-463	-460	-453	-427	-389	-365

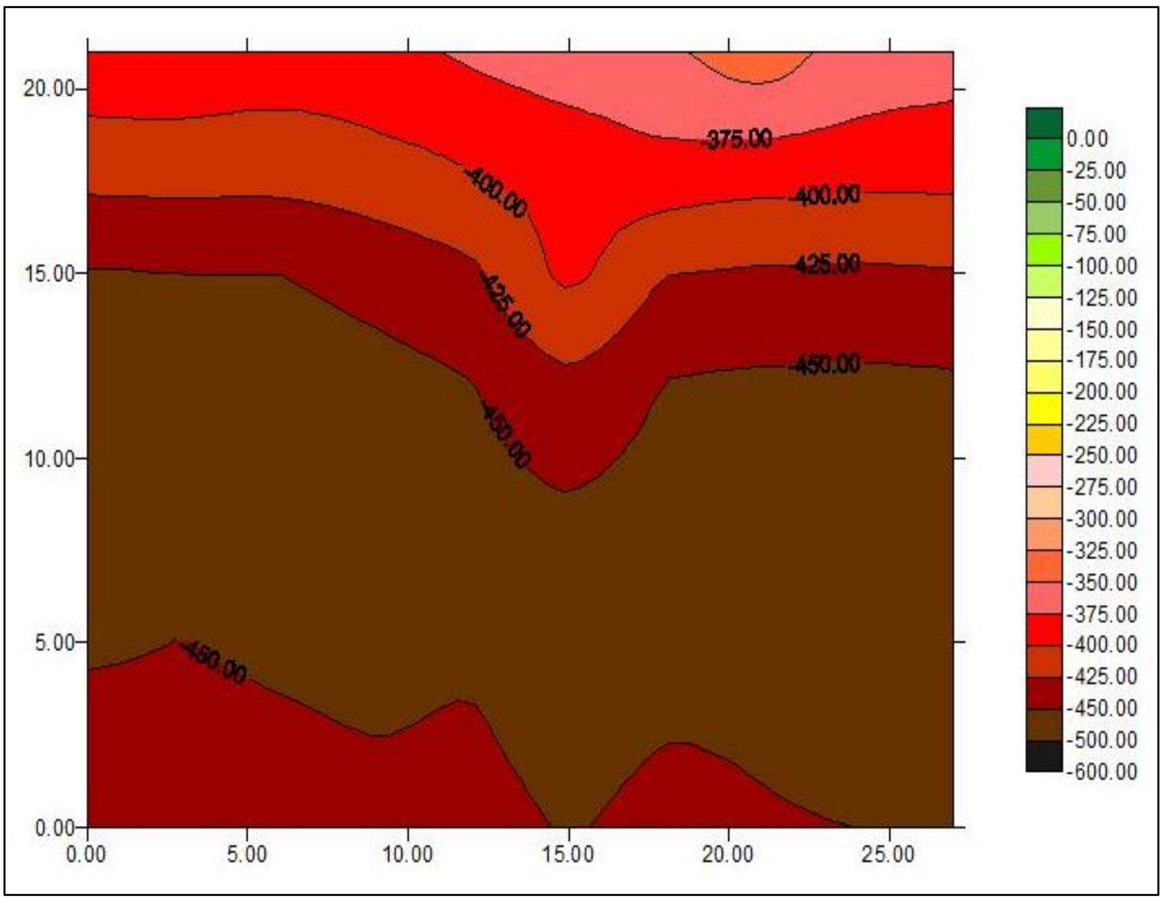


Figure E.68 Shaft 24, test 2 data map

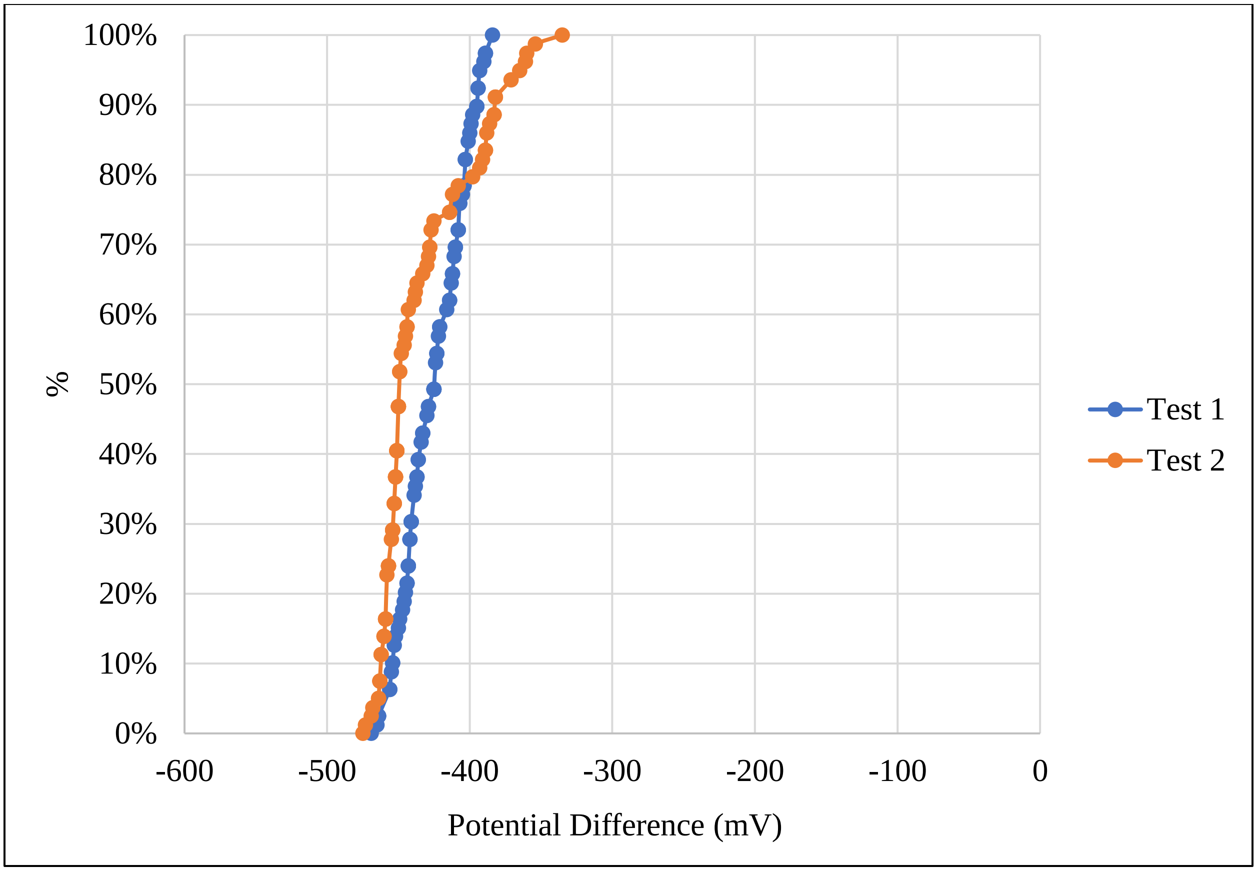


Figure E.69 Shaft 24 percentile distributions

Table E.47 Shaft 25, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-398	-408	-409	-422	-421	-416	-412	-406
3"	-403	-409	-407	-422	-419	-421	-410	-410
6"	-413	-408	-406	-422	-418	-419	-410	-412
9"	-420	-410	-403	-423	-417	-419	-407	-410
12"	-431	-421	-415	-414	-416	-424	-411	-408
15"	-443	-430	-419	-414	-414	-418	-412	-406
18"	-455	-449	-435	-406	-415	-414	-418	-402
21"	-463	-451	-438	-407	-417	-419	-415	-404
24"	-472	-458	-438	-409	-418	-419	-413	-404
27"	-472	-456	-438	-411	-416	-415	-407	-409

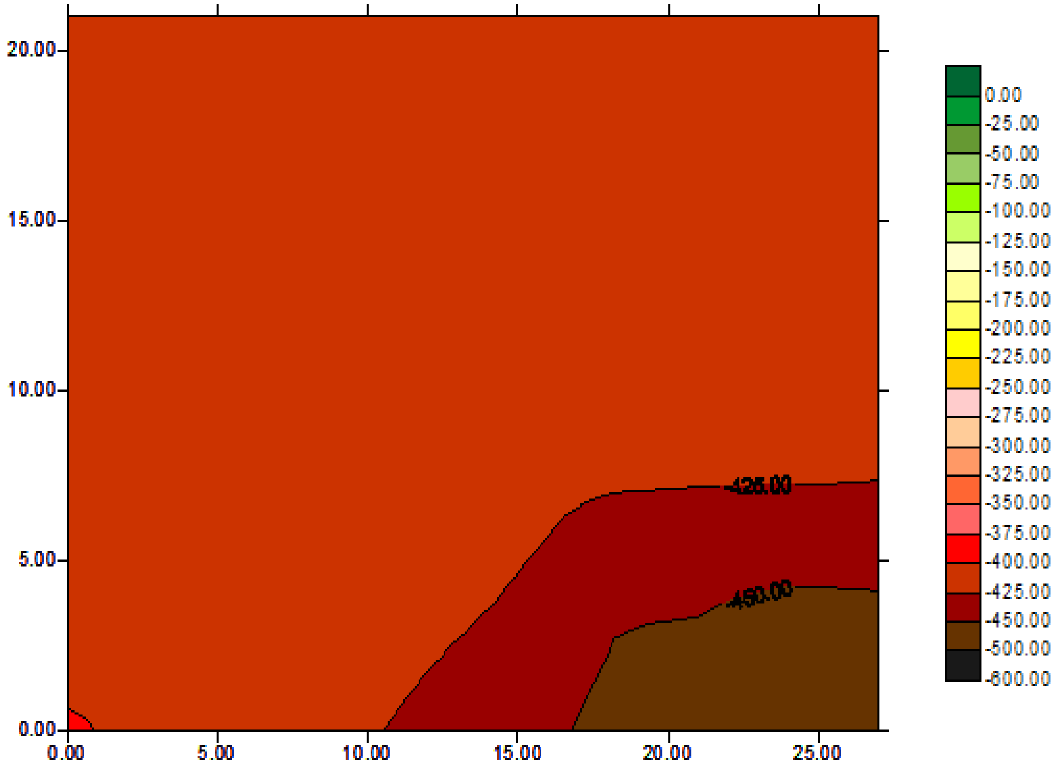


Figure E.70 Shaft 25, test 1 data map

Table E.48 Shaft 25, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-357	-358	-362	-370	-377	-386	-397	-407
3"	-357	-360	-366	-373	-379	-388	-399	-412
6"	-359	-364	-369	-374	-379	-390	-405	-418
9"	-363	-365	-371	-376	-389	-392	-409	-425
12"	-371	-372	-380	-389	-393	-405	-417	-433
15"	-381	-381	-388	-393	-400	-413	-425	-443
18"	-385	-386	-390	-397	-411	-423	-440	-449
21"	-390	-391	-396	-405	-420	-434	-445	-461
24"	-392	-390	-401	-411	-426	-445	-457	-469
27"	-392	-396	-404	-415	-430	-446	-461	-476

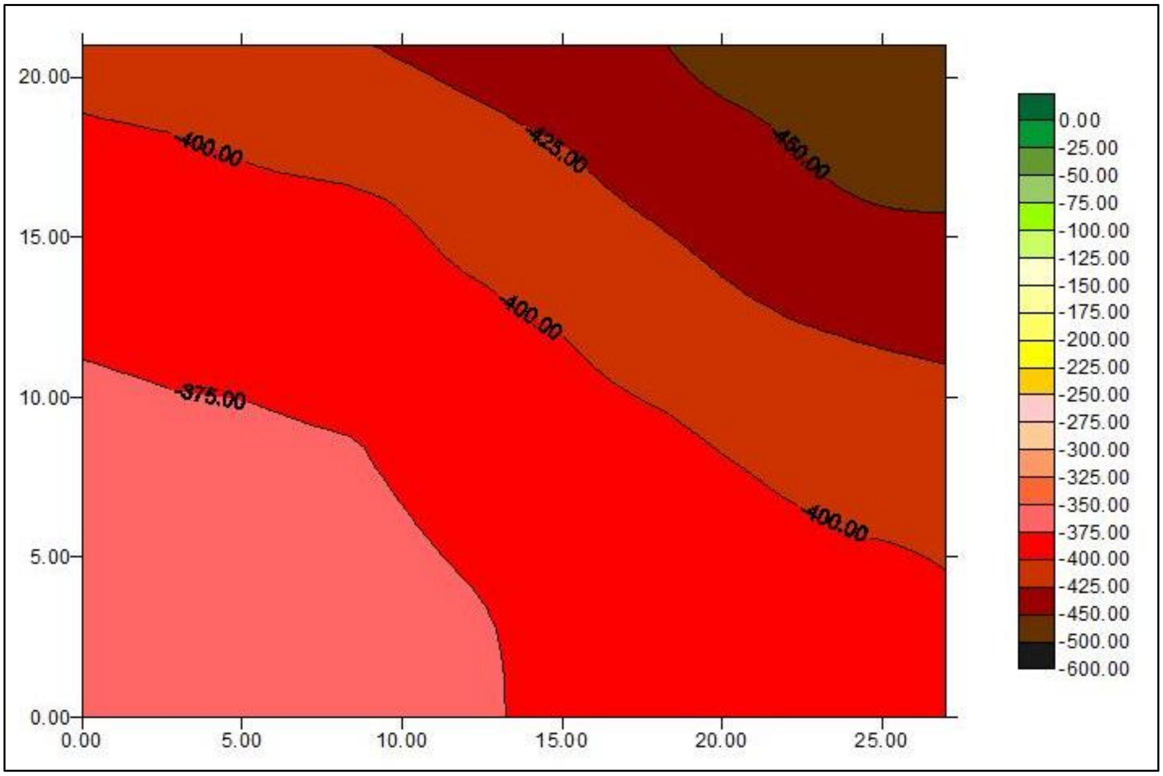


Figure E.71 Shaft 25, test 2 data map

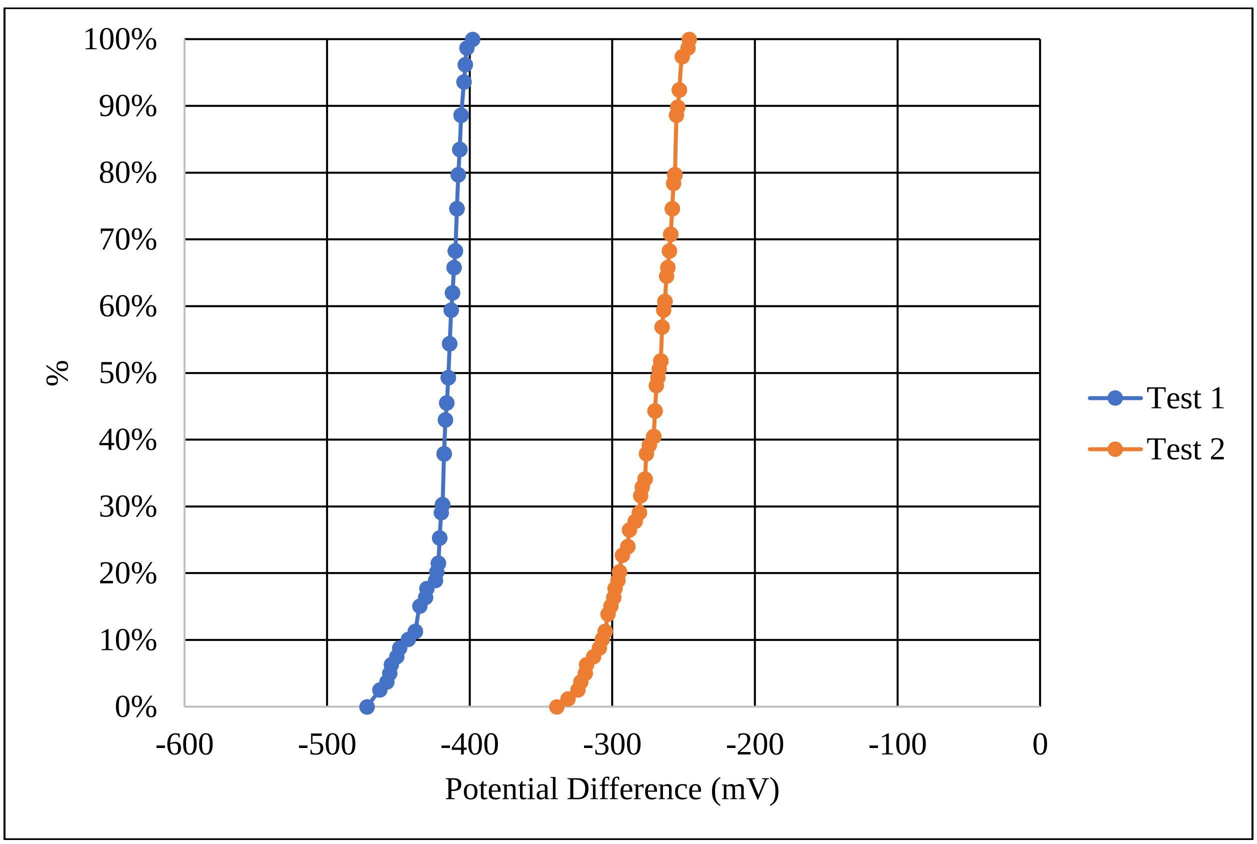


Figure E.72 Shaft 25 percentile distributions

Table E.49 Shaft 26, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-341	-328	-308	-293	-292	-304	-335	-367
3"	-339	-326	-305	-291	-290	-304	-329	-359
6"	-344	-325	-305	-291	-288	-299	-325	-350
9"	-344	-330	-310	-291	-289	-299	-321	-349
12"	-351	-334	-314	-298	-292	-300	-322	-349
15"	-363	-346	-321	-307	-301	-308	-325	-353
18"	-373	-353	-335	-313	-305	-309	-329	-360
21"	-383	-367	-342	-321	-310	-318	-335	-363
24"	-397	-379	-350	-336	-314	-319	-337	-369
27"	-403	-382	-355	-327	-315	-318	-341	-368

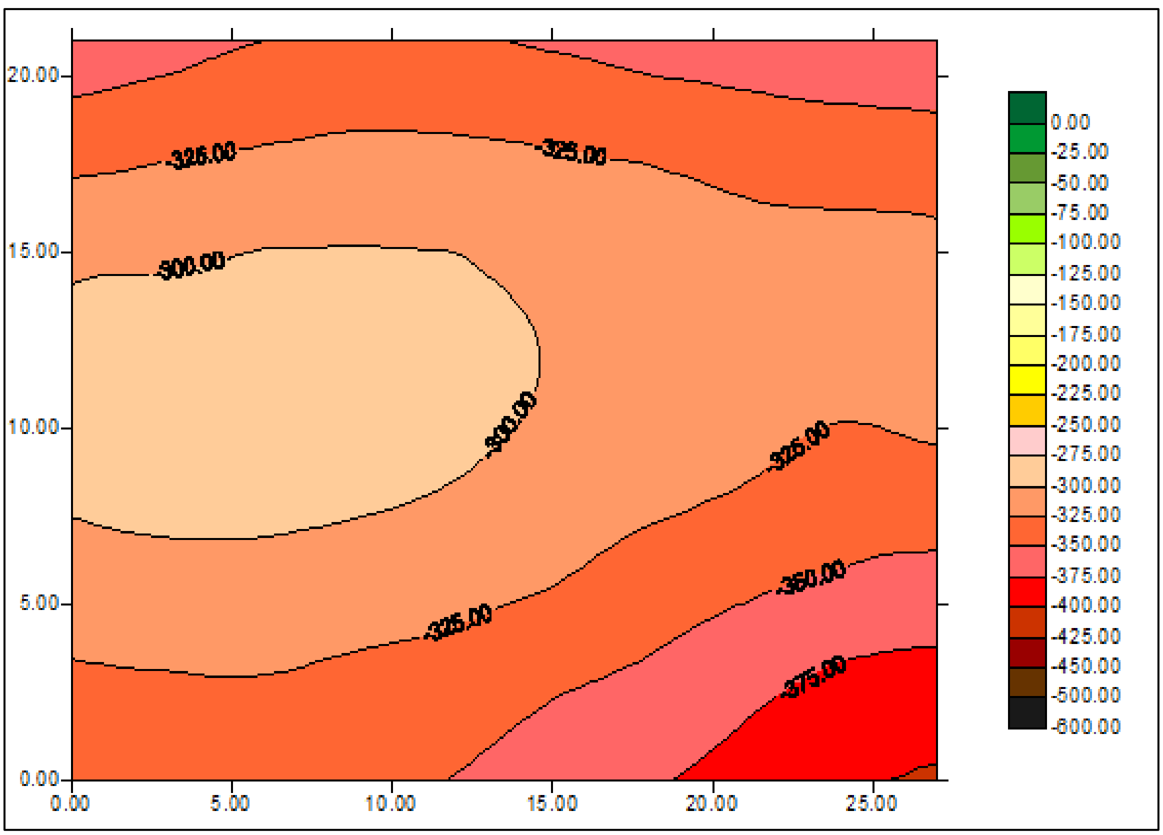


Figure E.73 Shaft 26, test 1 data map

Table E.50 Shaft 26, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-344	-331	-313	-307	-308	-320	-333	-347
3"	-348	-333	-316	-310	-309	-320	-332	-344
6"	-357	-338	-319	-311	-310	-319	-333	-345
9"	-367	-347	-327	-321	-303	-329	-337	-344
12"	-386	-359	-341	-326	-323	-332	-344	-347
15"	-401	-368	-342	-332	-327	-333	-344	-348
18"	-408	-376	-347	-333	-328	-333	-345	-350
21"	-401	-373	-345	-332	-327	-334	-344	-346
24"	-386	-361	-338	-325	-325	-333	-342	-345
27"	-363	-348	-327	-317	-316	-323	-333	-338

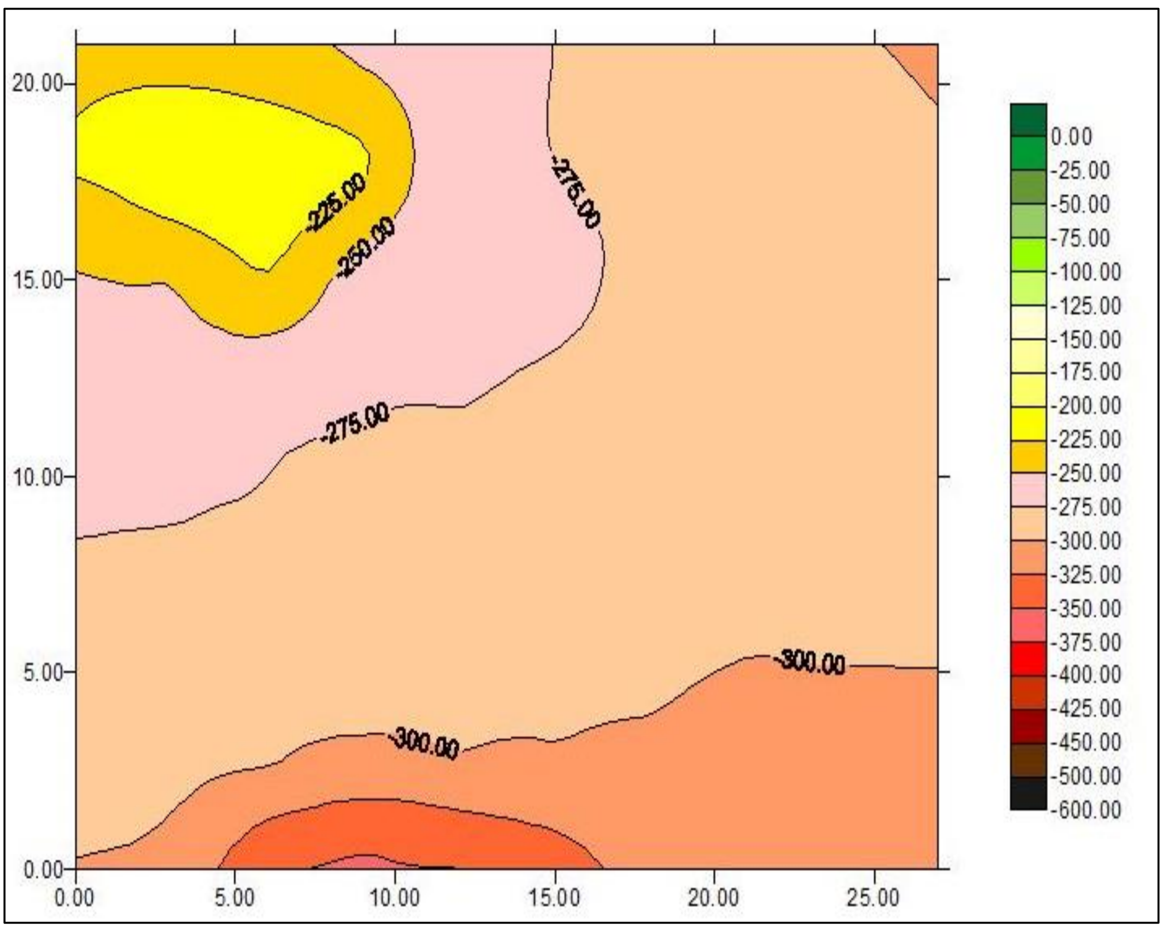


Figure E.74 Shaft 26, test 2 data map

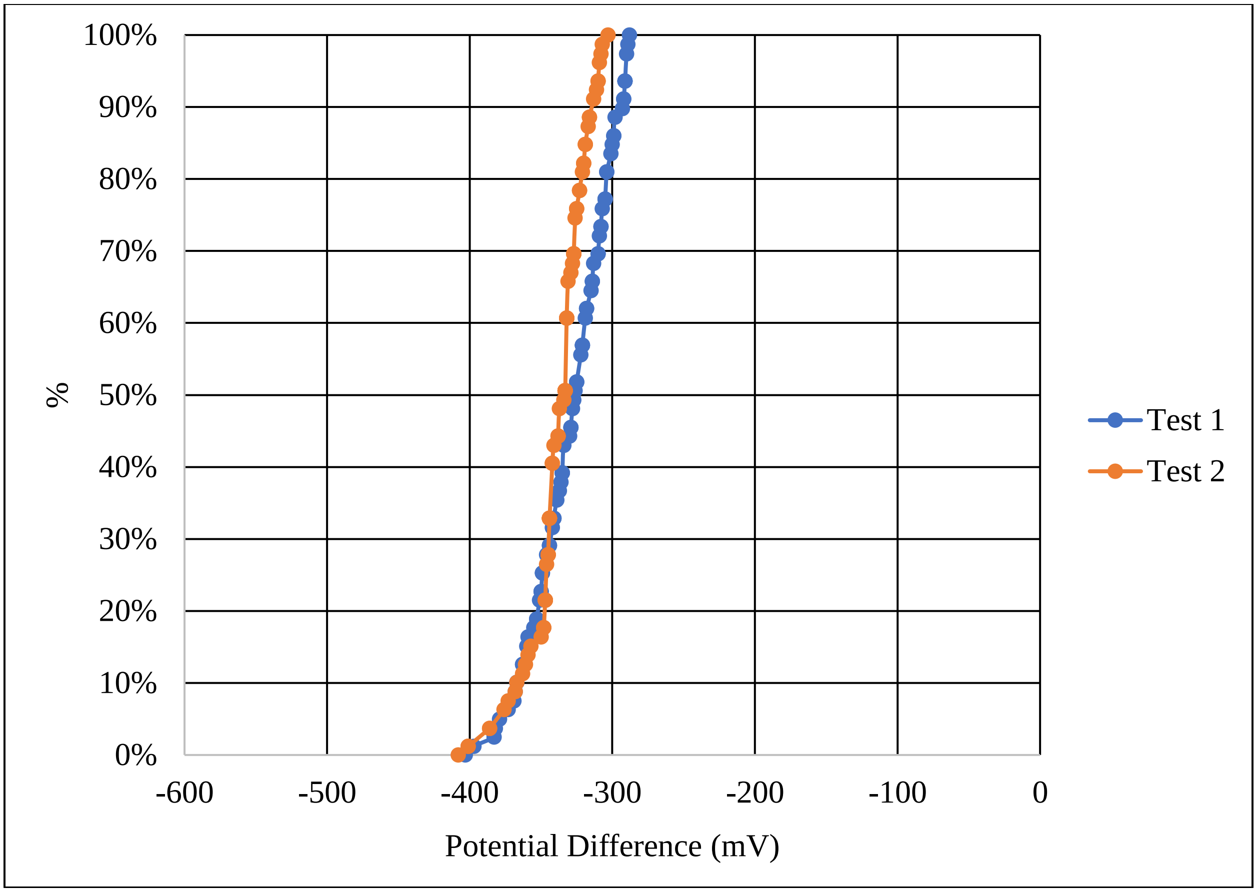


Figure E.75 Shaft 26 percentile distributions

Table E.51 Shaft 27, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-222	-232	-237	-244	-254	-259	-265	-292
3"	-224	-232	-231	-241	-255	-263	-267	-295
6"	-225	-232	-231	-240	-255	-263	-267	-332
9"	-220	-229	-232	-242	-260	-265	-347	-352
12"	-218	-225	-232	-244	-257	-269	-269	-332
15"	-220	-223	-225	-238	-254	-273	-281	-288
18"	-216	-220	-224	-235	-250	-269	-283	-293
21"	-219	-228	-225	-237	-249	-263	-285	-302
24"	-216	-226	-235	-237	-250	-265	-290	-291
27"	-223	-232	-230	-239	-252	-264	-292	-295

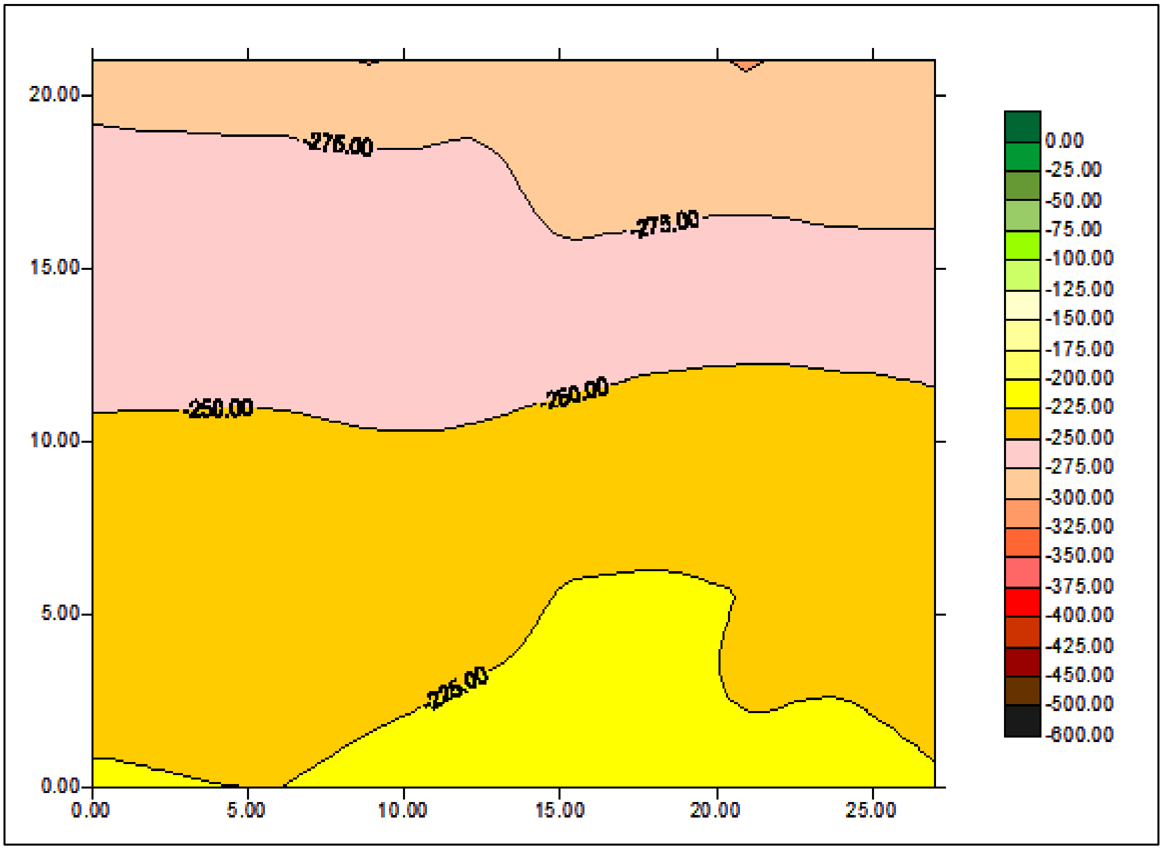


Figure E.76 Shaft 27, test 1 data map

Table E.52 Shaft 27, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-309	-290	-283	-273	-264	-253	-222	-234
3"	-306	-291	-285	-274	-265	-250	-207	-237
6"	-349	-293	-289	-276	-271	-225	-209	-244
9"	-357	-304	-283	-283	-273	-266	-218	-258
12"	-351	-298	-290	-283	-274	-271	-271	-268
15"	-337	-301	-294	-285	-278	-270	-275	-275
18"	-316	-302	-297	-289	-284	-279	-279	-284
21"	-309	-305	-299	-292	-286	-279	-286	-293
24"	-314	-308	-297	-292	-289	-283	-284	-299
27"	-314	-310	-296	-292	-290	-284	-298	-303

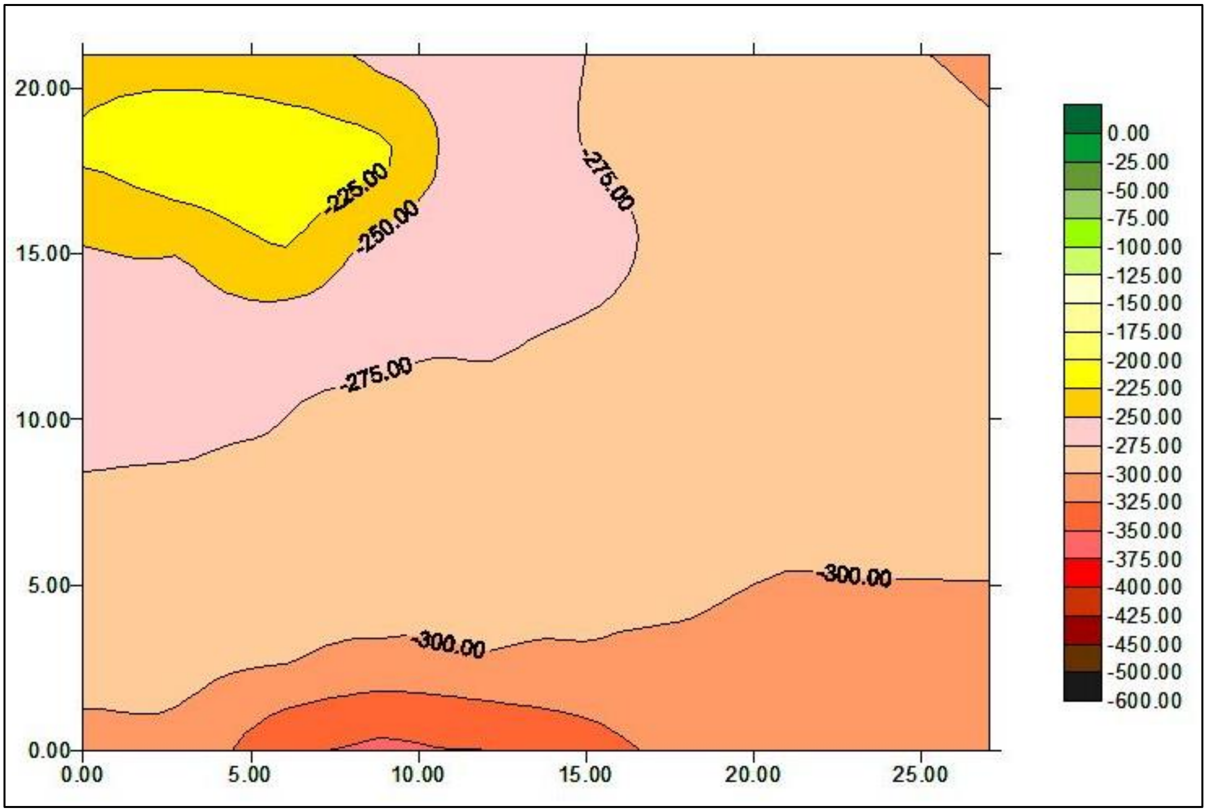


Figure E.77 Shaft 27, test 2 data map

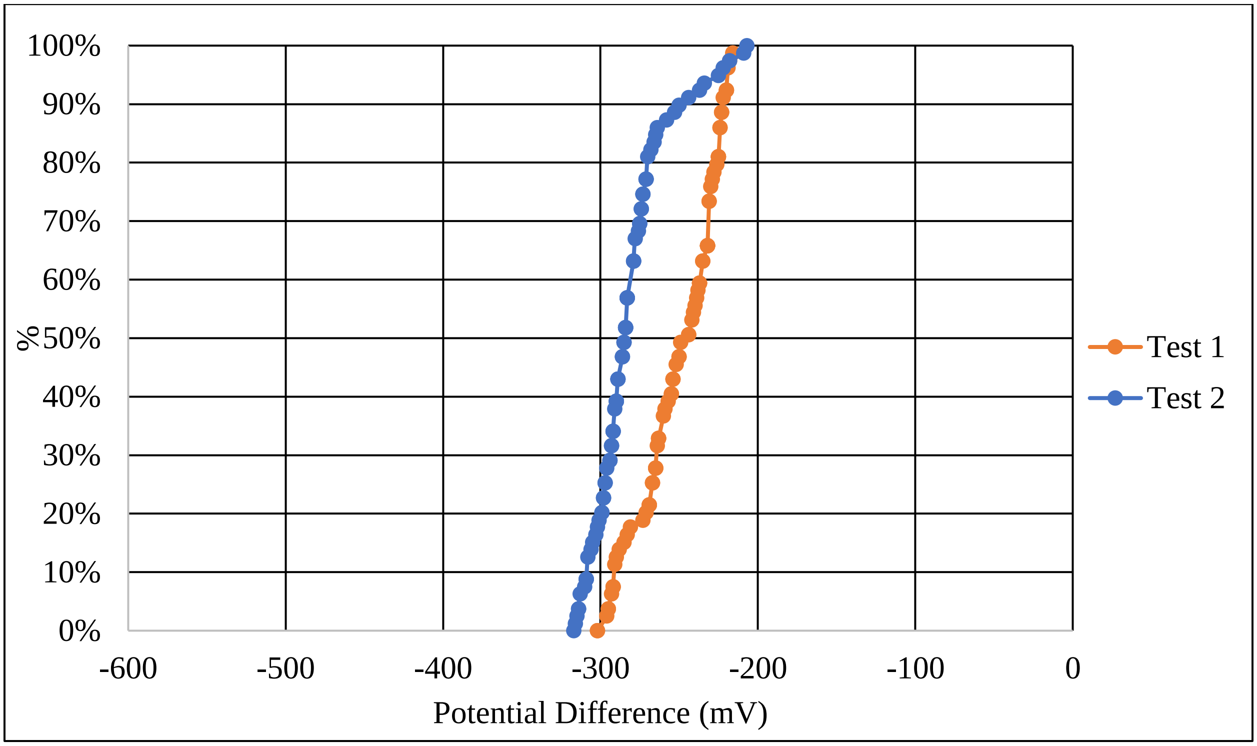


Figure E.78 Shaft 27 percentile distributions

Table E.53 Shaft 28, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-209	-243	-245	-257	-270	-295	-318	-329
3"	-215	-248	-252	-262	-272	-296	-317	330
6"	-242	-248	-254	-268	-279	-298	-319	-330
9"	-241	-247	-258	-268	-280	-290	-325	-334
12"	-242	-249	-258	-268	-281	-303	-326	-339
15"	-238	-246	-257	-268	-283	-303	-329	-347
18"	-238	-250	-257	-268	-284	-300	-325	-345
21"	-241	-252	-256	-269	-287	-304	-324	-341
24"	-240	-249	-251	-268	-283	-300	-324	-340
27"	-240	-246	-253	-261	-278	-295	-323	-344

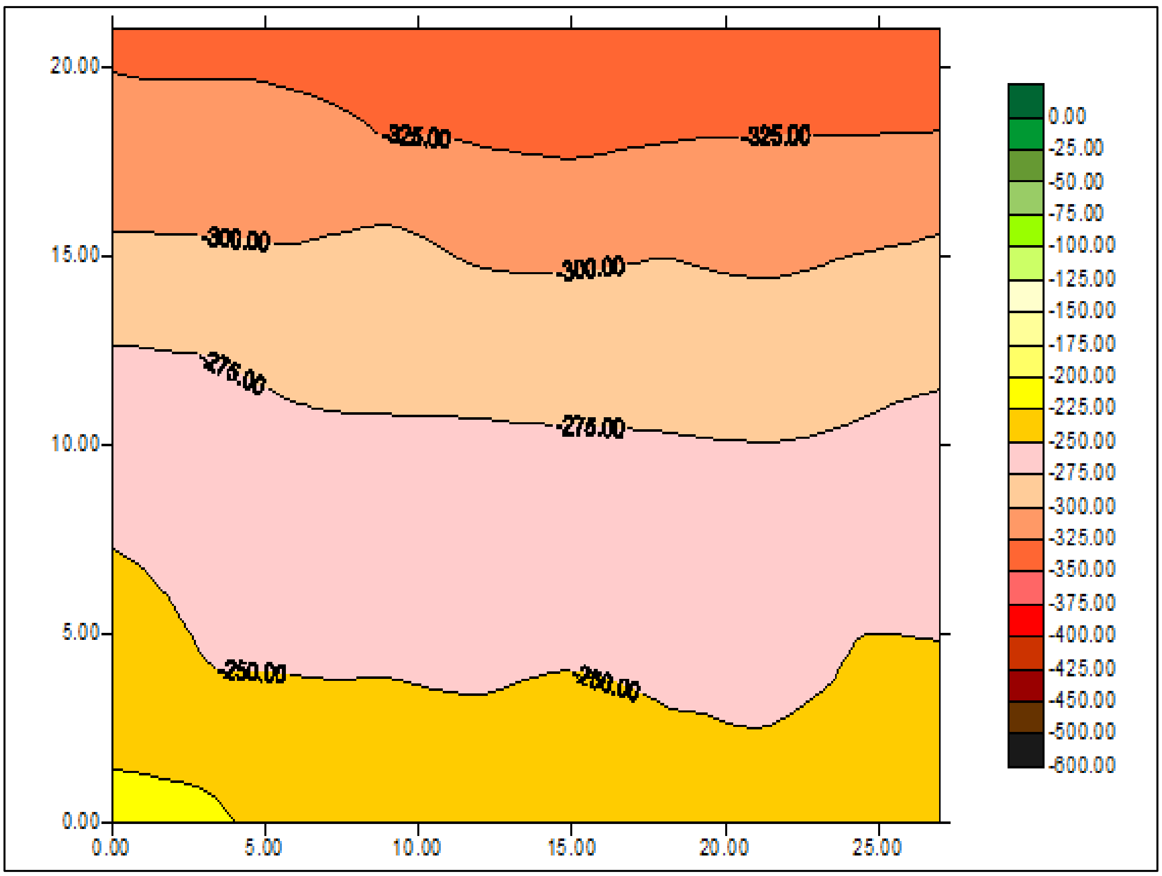


Figure E.79 Shaft 28, test 1 data map

Table E.54 Shaft 28, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-349	-324	-298	-277	-269	-259	-257	-246
3"	-331	-319	-270	-277	-268	-260	-258	-247
6"	-322	-309	-289	-270	-266	-260	-256	-253
9"	-318	-305	-284	-270	-263	-258	-261	-256
12"	-313	-301	-281	-266	-261	-259	-264	-267
15"	-307	-295	-276	-262	-256	-258	-263	-274
18"	-305	-295	-271	-259	-253	-254	-265	-277
21"	-303	-293	-271	-256	-253	-256	-266	-279
24"	-299	-288	-271	-256	-251	-254	-265	-280
27"	-296	-289	-266	-256	-253	-255	-263	-281

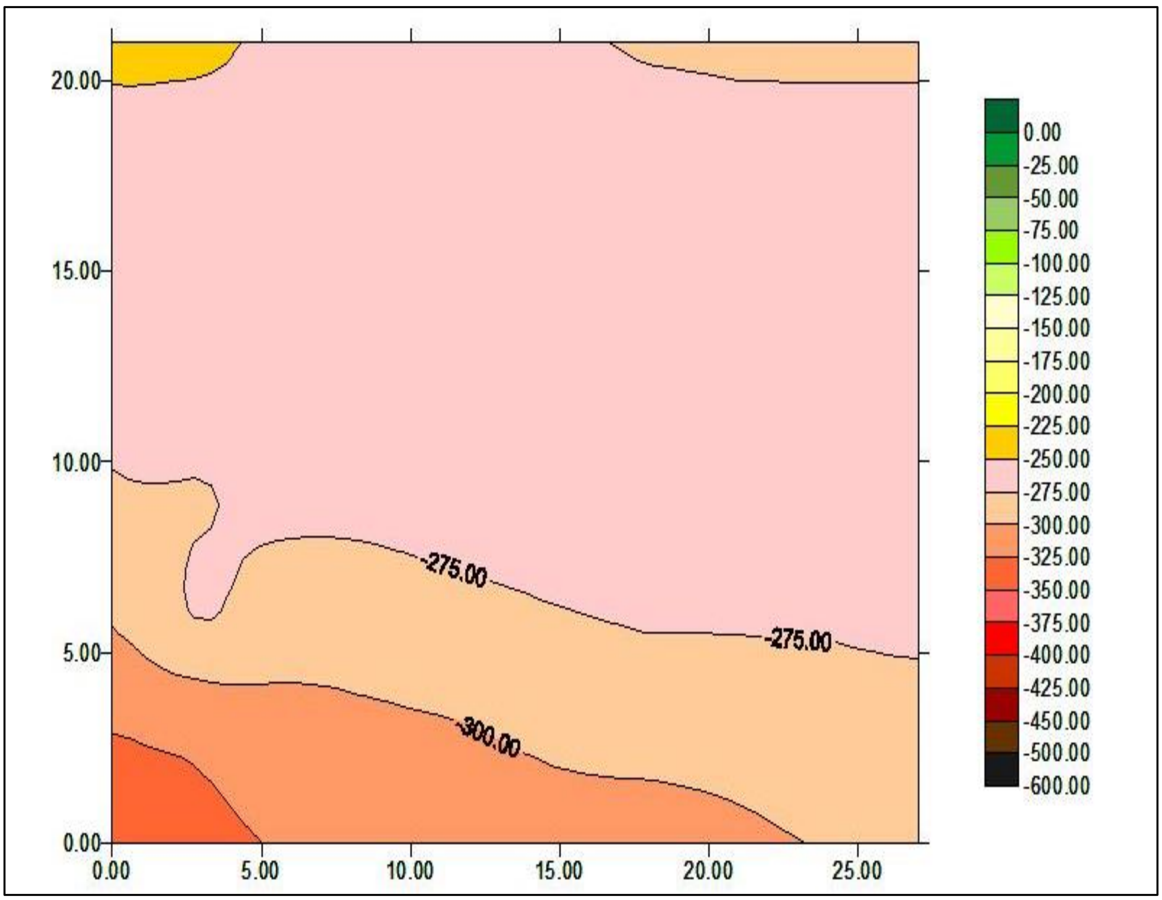


Figure E.80 Shaft 28, test 2 data map

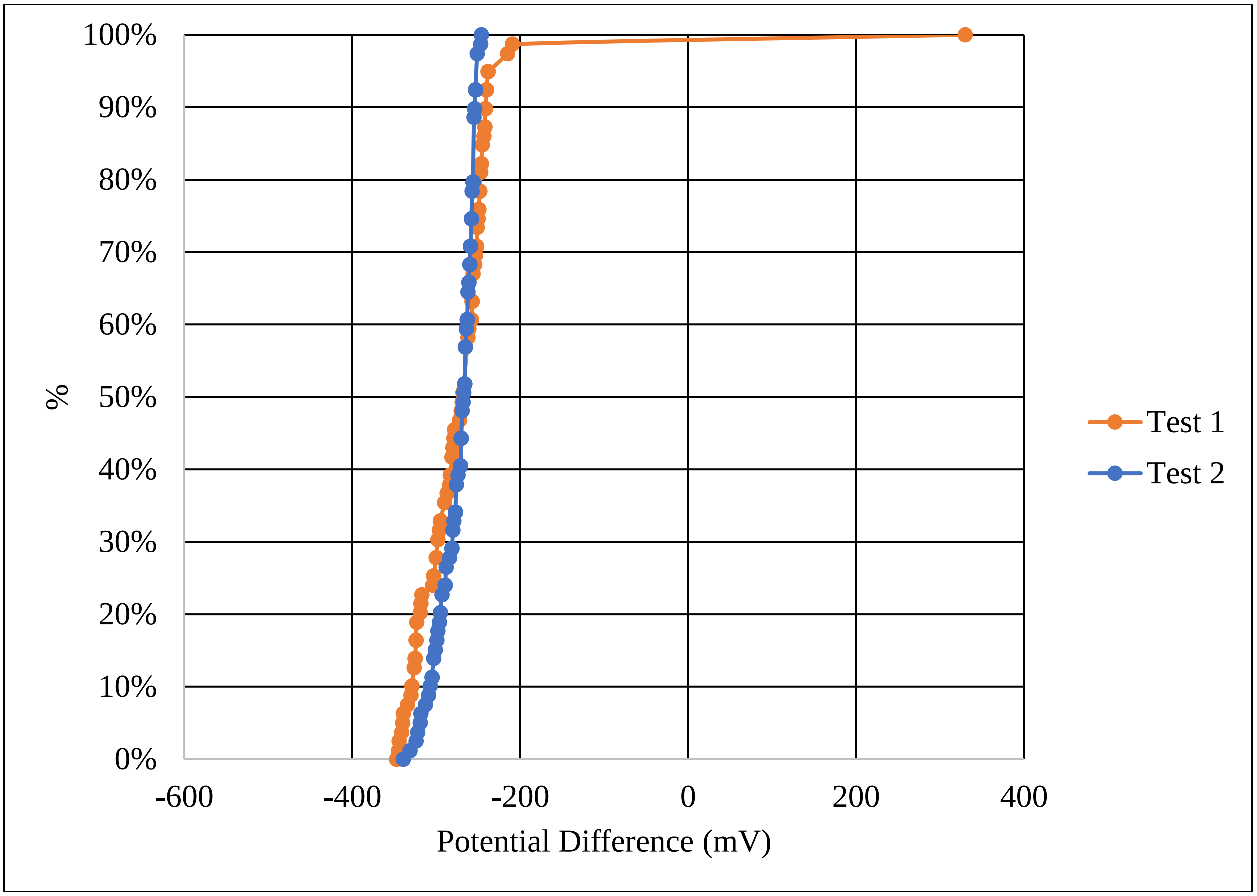


Figure E.81 Shaft 28 percentile distributions

Table E.55 Shaft 29, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-403	-440	-461	-510	-533	-549	-546	-551
3"	-395	-425	-440	-485	-517	-522	-505	-518
6"	-394	-416	-435	-462	-482	-486	-475	-480
9"	-389	-411	-428	-445	-450	-457	-455	-456
12"	-393	-408	-419	-432	-429	-436	-434	-430
15"	-390	-403	-410	-415	-415	-414	-417	-424
18"	-394	-404	-403	-402	-399	-401	-404	-411
21"	-400	-408	-403	-399	-397	-399	-404	-405
24"	-400	-406	-401	-395	-393	-397	-401	-406
27"	-403	-406	-400	-394	-385	-392	-396	-399

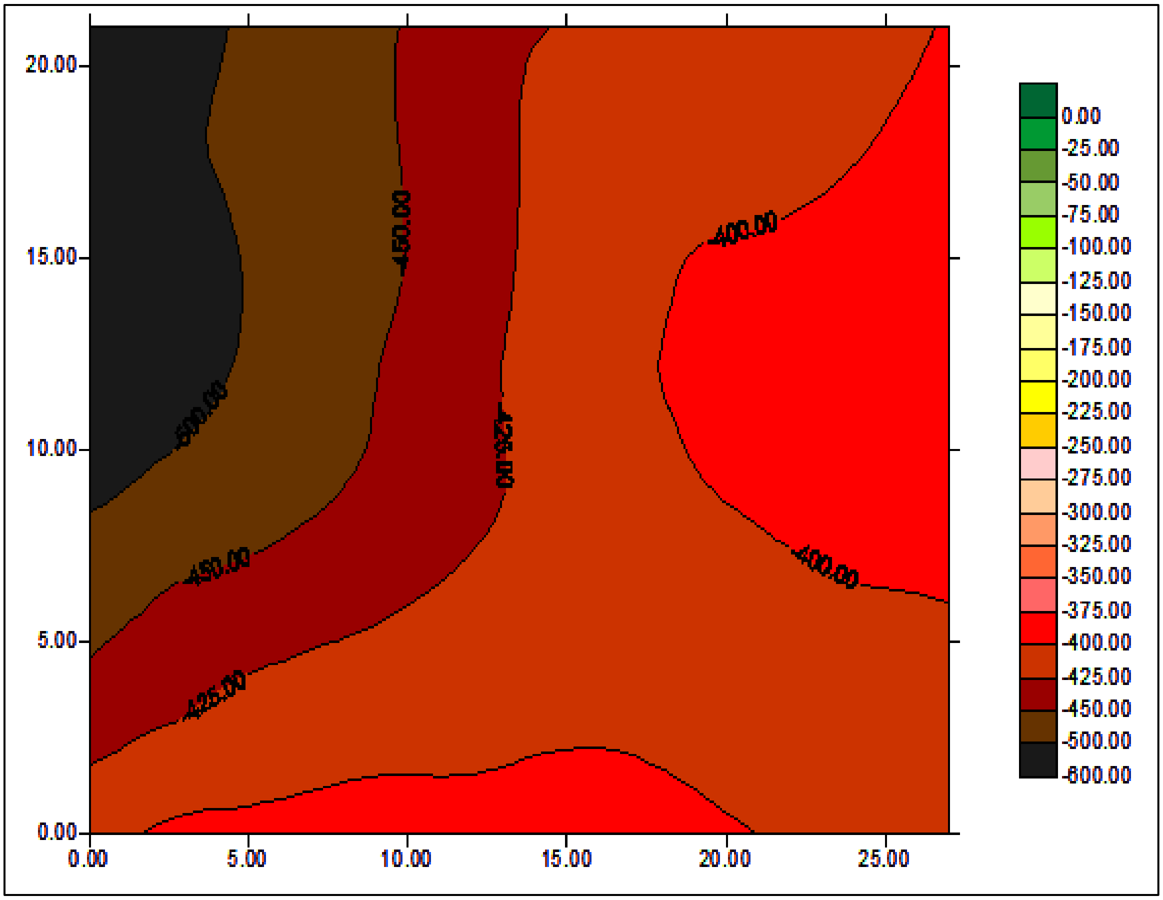


Figure E.82 Shaft 29, test 1 data map

Table E.56 Shaft 29, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-454	-447	-440	-430	-425	-425	-430	-434
3"	-438	-435	-430	-424	-420	-421	-423	-431
6"	-430	-424	-424	-420	-417	-417	-424	-429
9"	-424	-423	-425	-419	-417	-415	-422	-417
12"	-430	-427	-426	-417	-426	-420	-429	-429
15"	-438	-438	-441	-437	-440	-435	-436	-433
18"	-452	-453	-431	-469	-469	-458	-447	-440
21"	-471	-468	-484	-499	-495	-479	-463	-449
24"	-499	-494	-512	-528	-518	-491	-469	-449
27"	-525	-545	-563	-543	-524	-495	-472	-449

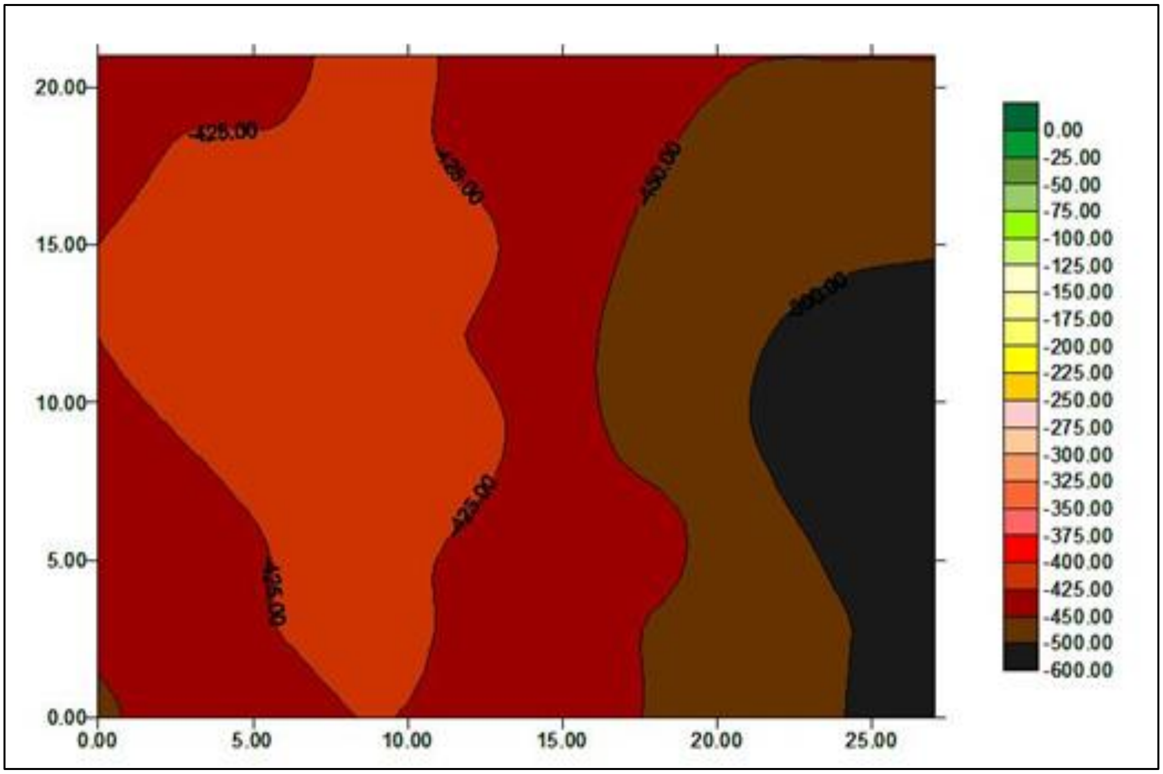


Figure E.83 Shaft 29, test 2 data map

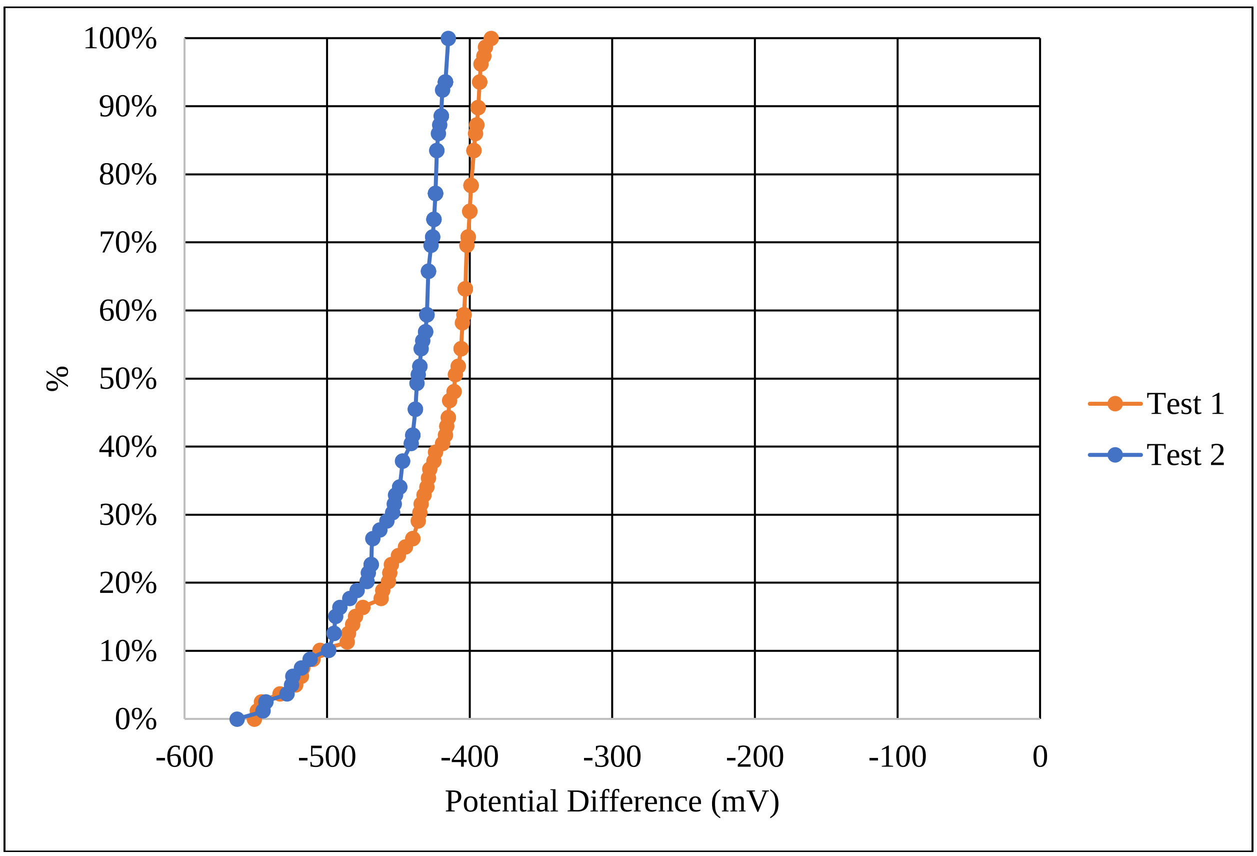


Figure E.84 Shaft 29 percentile distributions

Table E.57 Shaft 30, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-433	-422	-407	-385	-377	-388	-399	-427
3"	-445	-424	-416	-392	-380	-390	-406	-420
6"	-460	-444	-419	-399	-385	-387	-398	-424
9"	-472	-455	-429	-411	-392	-387	-396	-420
12"	-483	-468	-447	-414	-396	-388	-394	-404
15"	-490	-483	-465	-430	-398	-386	-388	-398
18"	-496	-490	-467	-431	-400	-388	-387	-397
21"	-499	-494	-468	-433	-402	-389	-390	-397
24"	-500	-485	-462	-432	-401	-391	-390	-399
27"	-499	-488	-475	-441	-409	-397	-403	-407

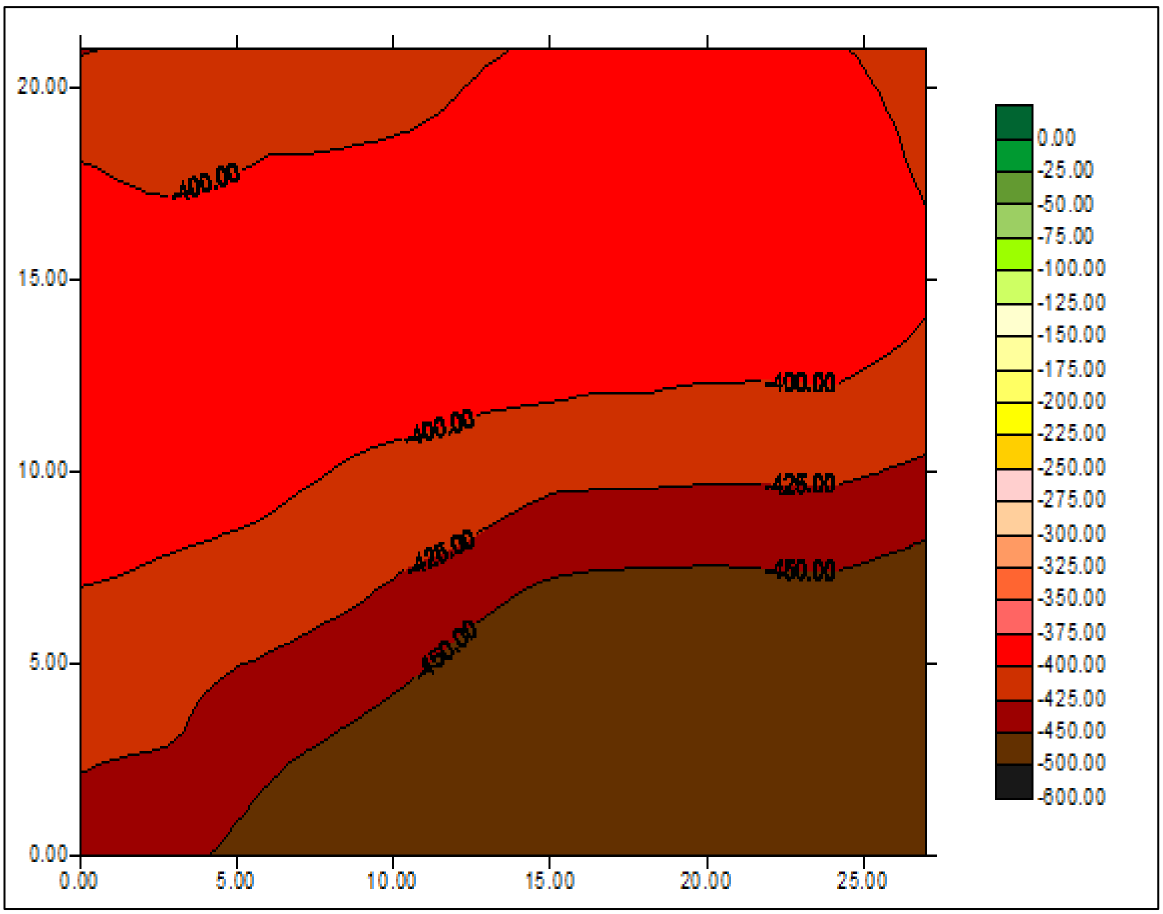


Figure E.85 Shaft 30, test 1 data map

Table E.58 Shaft 30, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-469	-462	-455	-468	-499	-534	-573	-596
3"	-470	-457	-457	-469	-505	-535	-585	-600
6"	-466	-457	-455	-474	-512	-544	-589	-609
9"	-450	-457	-453	-472	-510	-556	-601	-611
12"	-441	-438	-451	-470	-519	-563	-593	-611
15"	-436	-434	-445	-470	-510	-561	-597	-607
18"	-444	-439	-444	-462	-502	-558	-585	-607
21"	-452	-443	-451	-472	-507	-552	-579	-602
24"	-461	-460	-463	-479	-523	-556	-582	-604
27"	-474	-468	-471	-488	-524	-563	-586	-604

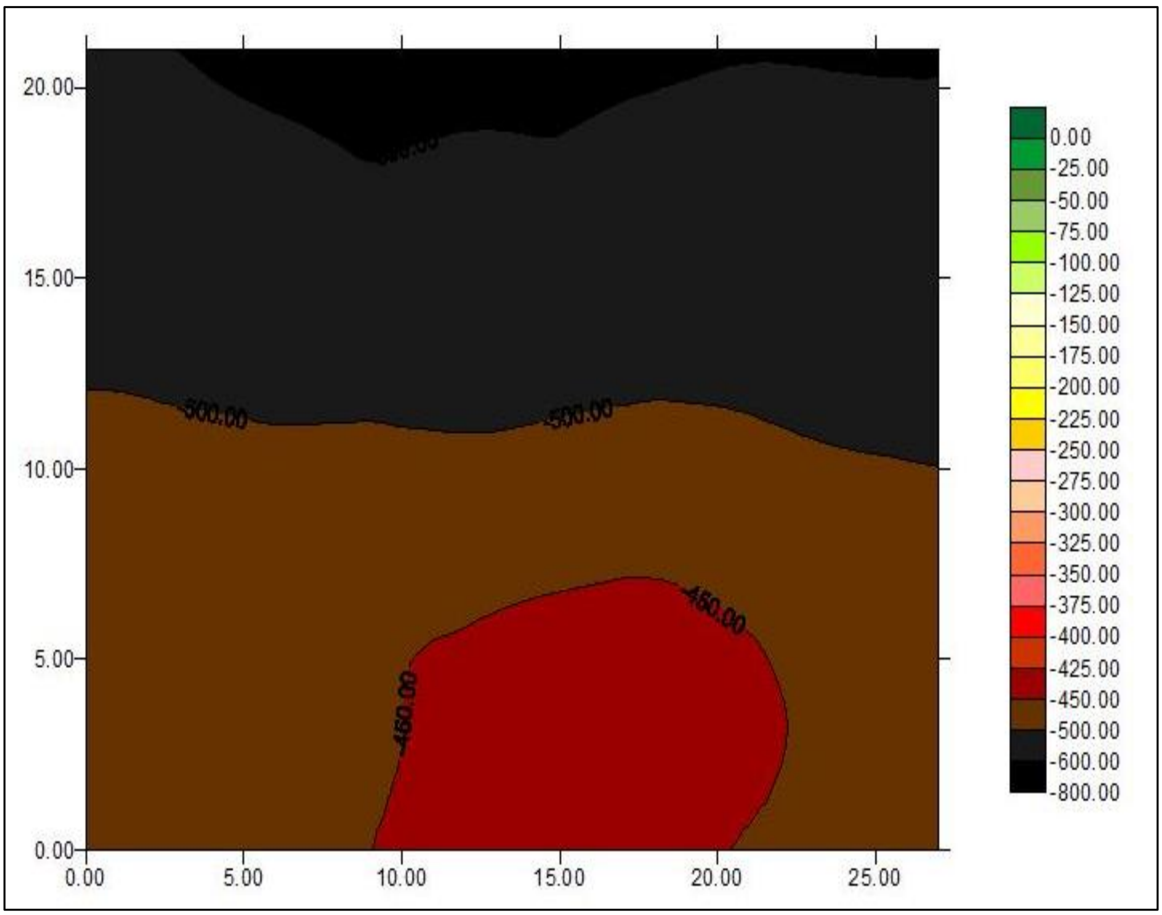


Figure E.86 Shaft 30, test 2 data map

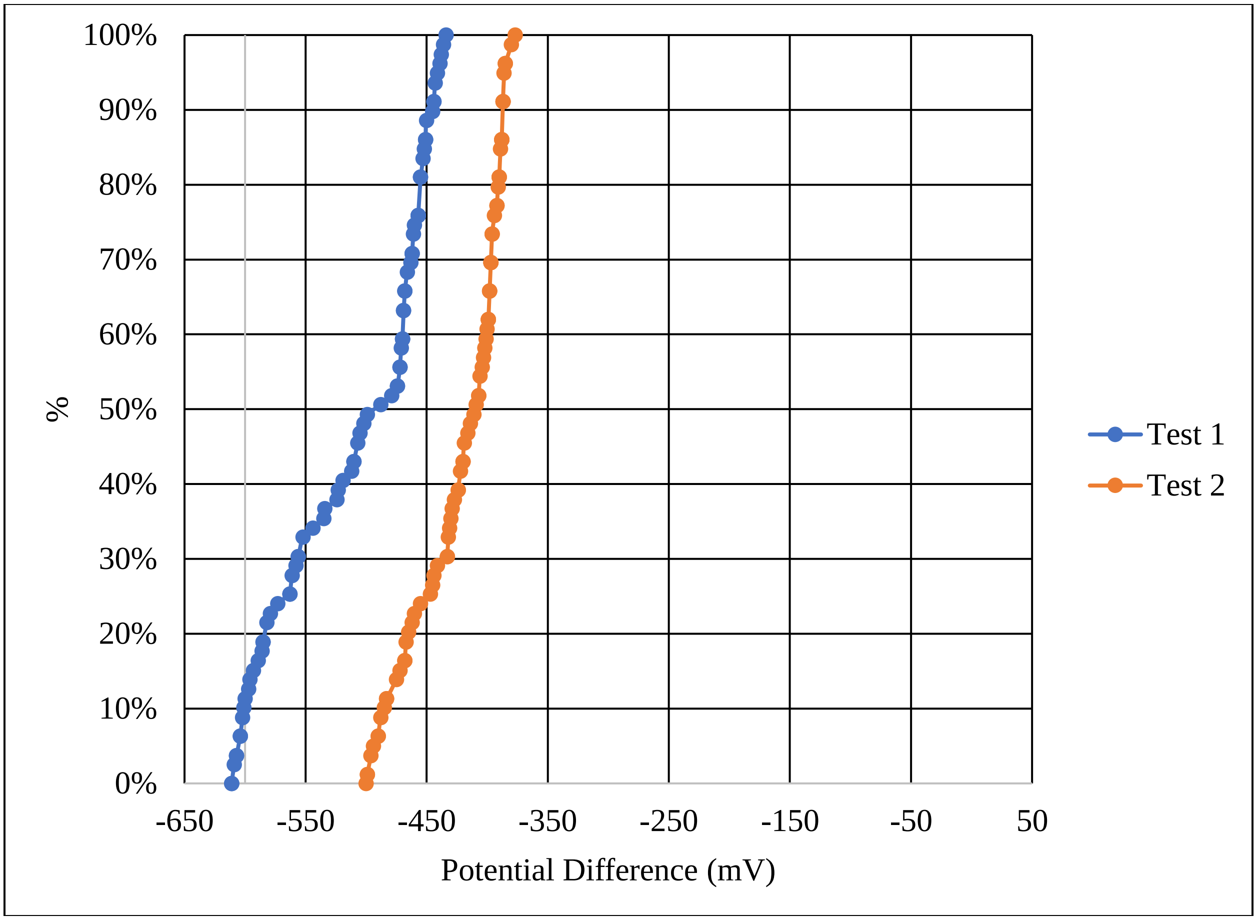


Figure E.87 Shaft 30 percentile distributions

Table E.59 Shaft 31, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-302	-297	-296	-306	-307	-315	-317	-311
3"	-297	-291	-292	-304	-311	-317	-321	-315
6"	-291	-292	-293	-306	-306	-311	-317	-308
9"	-303	-291	-290	-298	-301	-313	-314	-304
12"	-307	-297	-305	-296	-308	-312	-314	-308
15"	-298	-302	-300	-299	-303	-309	-309	-318
18"	-298	-301	-303	-298	-300	-305	-309	-304
21"	-302	-305	-303	-294	-305	-303	-301	-303
24"	-306	-295	-300	-300	-307	-301	-300	-301
27"	-312	-300	-307	-307	-309	-307	-299	-304

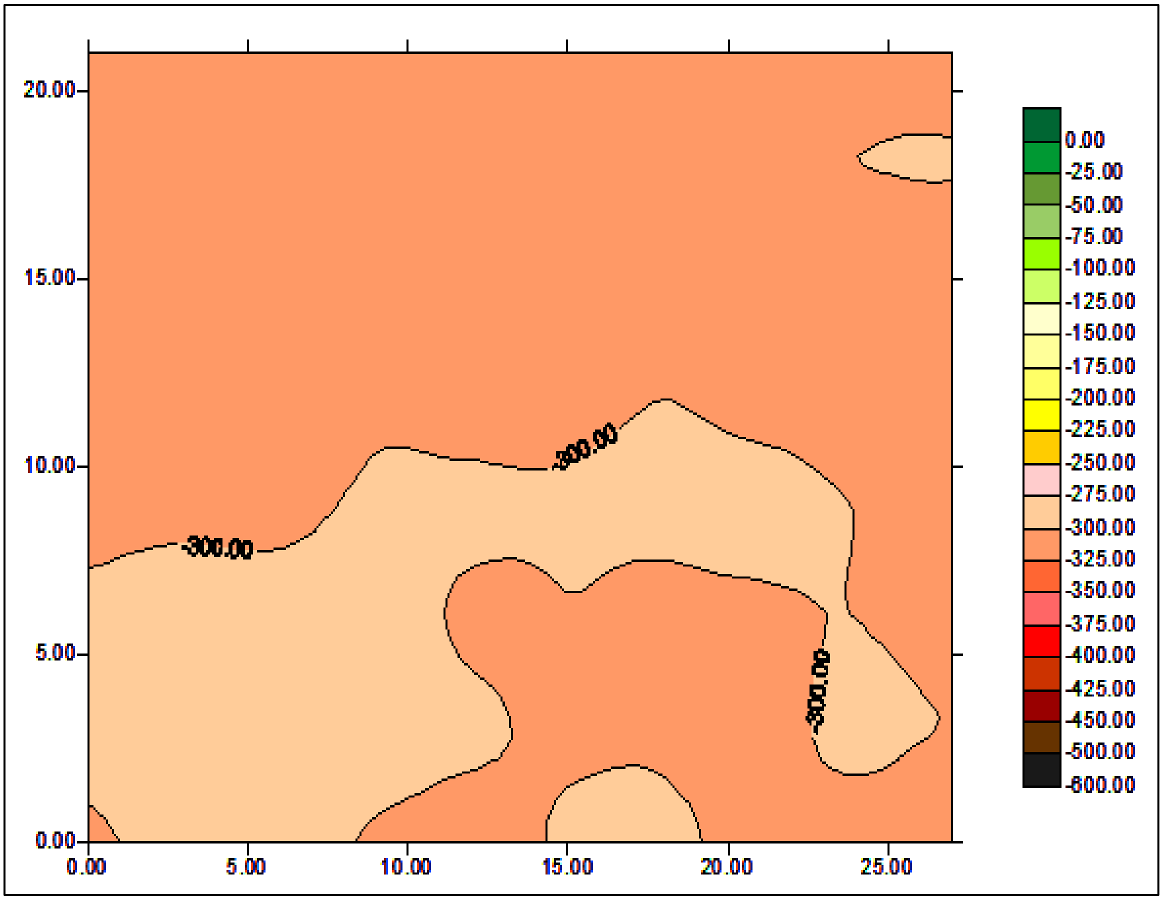


Figure E.88 Shaft 31, test 1 data map

Table E.60 Shaft 31, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-275	-278	-279	-283	-287	-290	-285	-288
3"	-278	-277	-283	-285	-285	-292	-291	-290
6"	-281	-282	-286	-289	-284	-293	-295	-291
9"	-281	-283	-289	-289	-289	-296	-298	-298
12"	-282	-284	-291	-292	-292	-301	-302	-303
15"	-290	-289	-285	-293	-294	-306	-302	-306
18"	-292	-290	-284	-292	-294	-299	-304	-306
21"	-289	-291	-289	-295	-297	-303	-308	-310
24"	-294	-293	-301	-303	-301	-303	-315	-314
27"	-293	-296	-297	-299	-302	-308	-311	-316

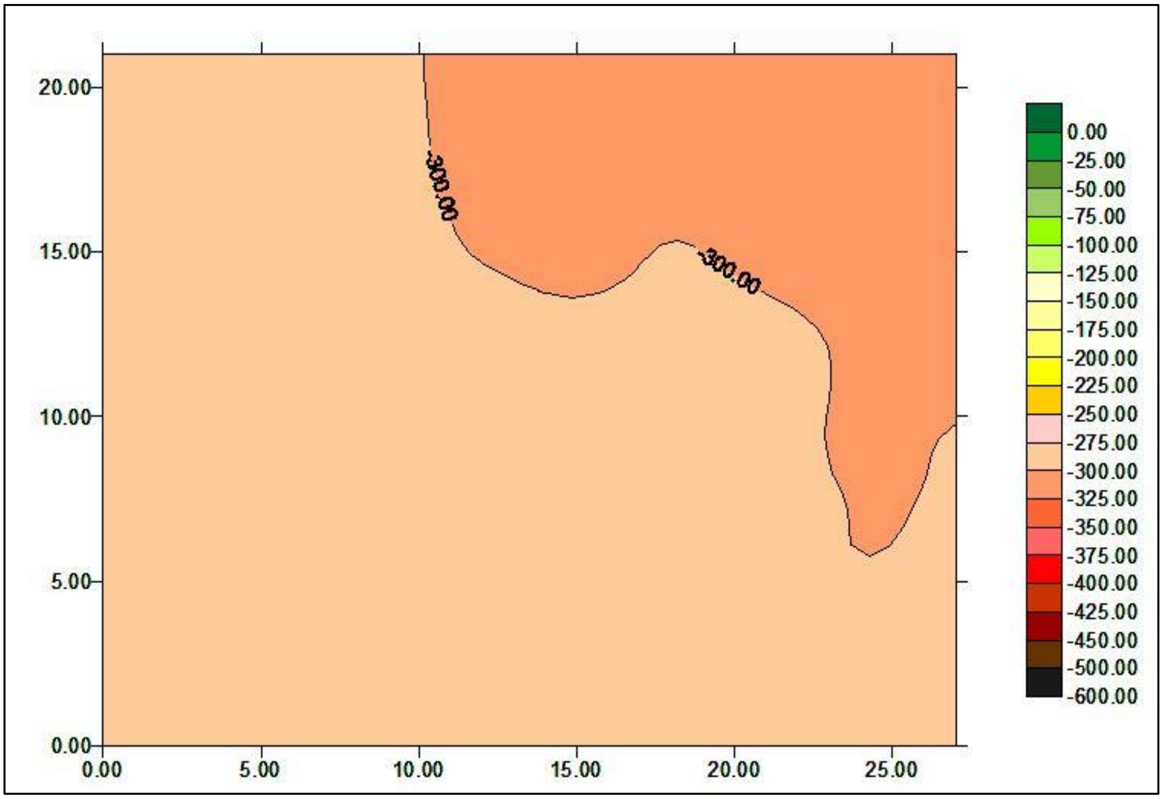


Figure E.89 Shaft 31, test 2 data map

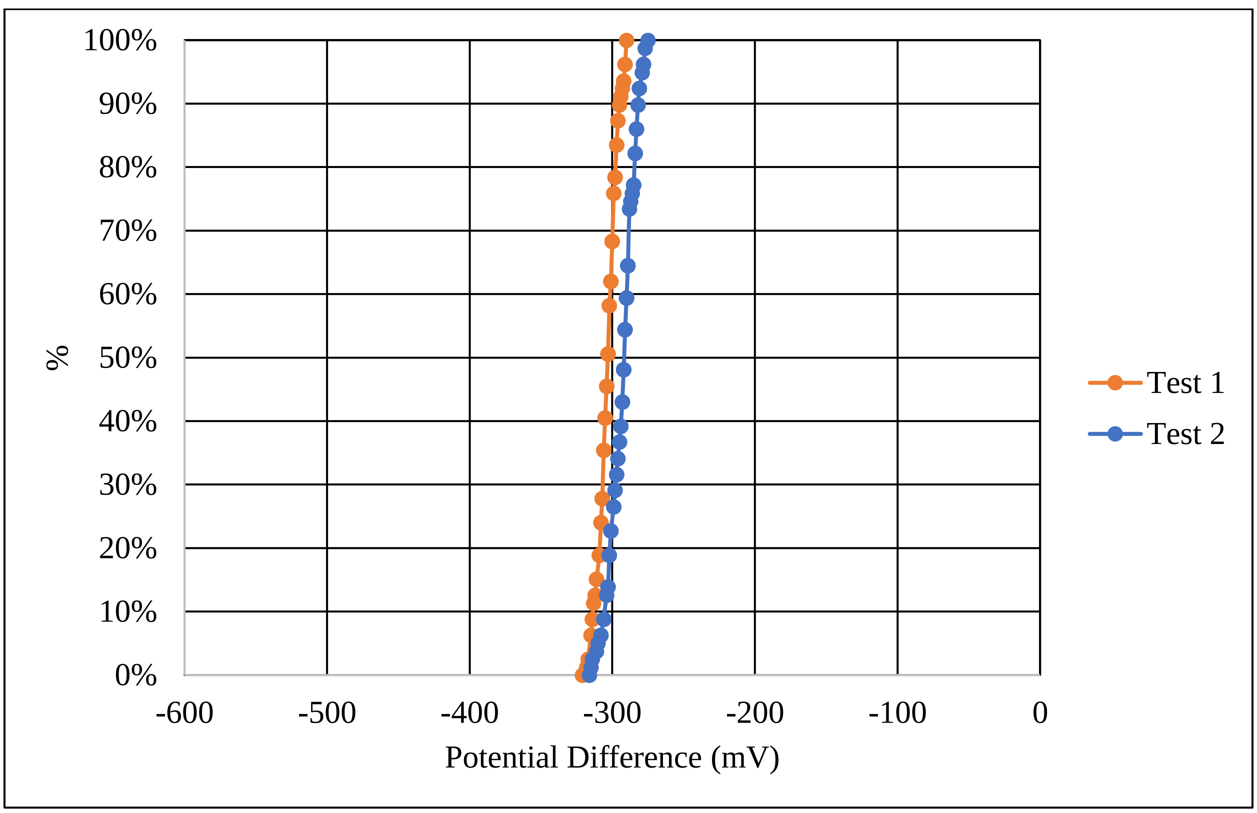


Figure E.90 Shaft 31 percentile distributions

Table E.61 Shaft 32, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-291	-290	-281	-284	-282	-281	-277	-276
3"	-291	-290	-283	-282	-285	-278	-279	-276
6"	-282	-290	-282	-282	-278	-278	-278	-279
9"	-284	-286	-279	-277	-278	-276	-272	-279
12"	-289	-299	-252	-248	-294	-280	-274	-279
15"	-285	-304	-248	-235	-283	-275	-275	-277
18"	-286	-293	-290	-257	-290	-280	-277	-274
21"	-279	-282	-276	-277	-277	-279	-275	-275
24"	-279	-283	-275	-280	-277	-277	-275	-286
27"	-279	-276	-271	-273	-273	-276	-283	-278

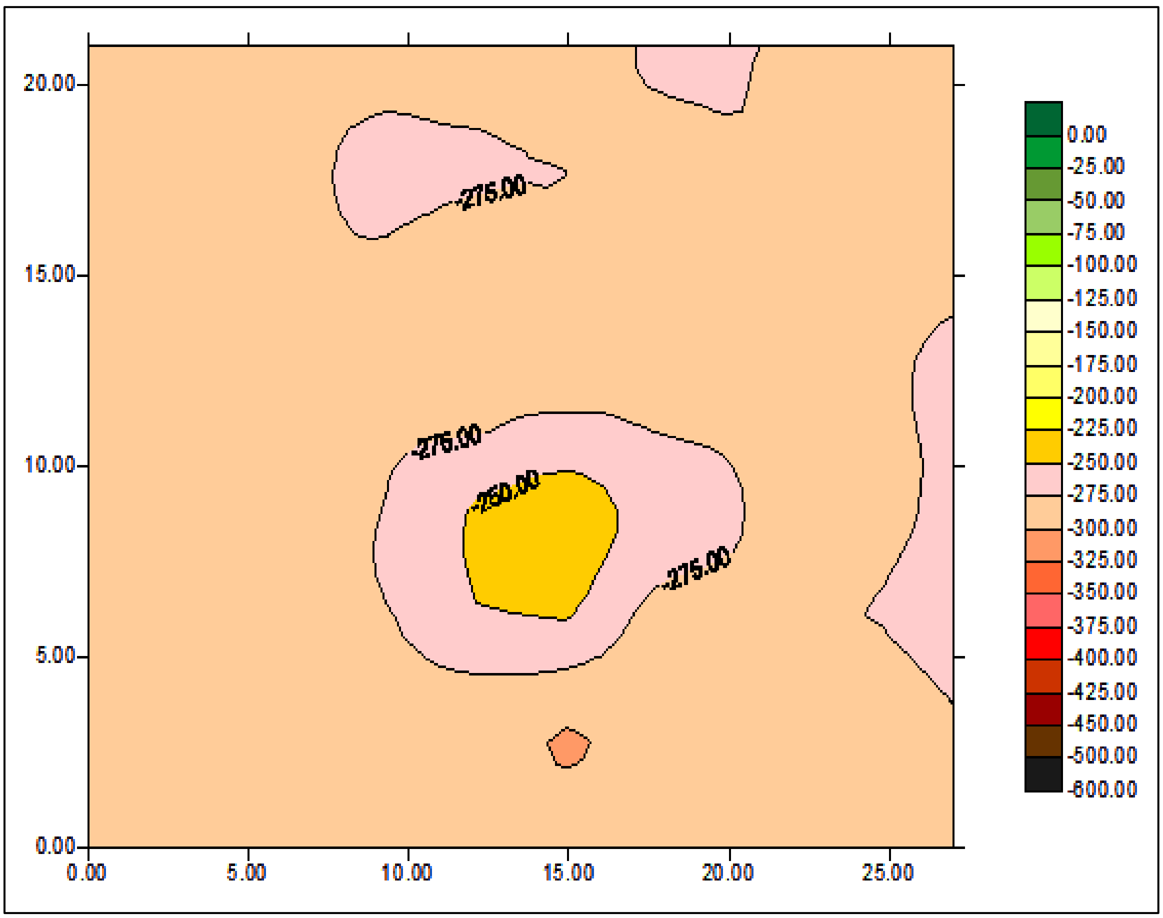


Figure E.91 Shaft 32, test 1 data map

Table E.62 Shaft 32, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-308	-302	-301	-297	-299	-301	-315	-314
3"	-309	-305	-300	-299	-301	-297	-310	-317
6"	-318	-295	-296	-298	-297	-304	-310	-316
9"	-321	-317	-303	-304	-304	-304	-306	-306
12"	-325	-317	-309	-302	-304	-304	-298	-299
15"	-309	-302	-295	-293	-292	-292	-285	-291
18"	-300	-295	-291	-285	-279	-279	-280	-288
21"	-312	-301	-294	-291	-278	-278	-280	-283
24"	-323	-315	-304	-296	-288	-289	-288	-285
27"	-332	-320	-310	-306	-306	-300	-291	-289

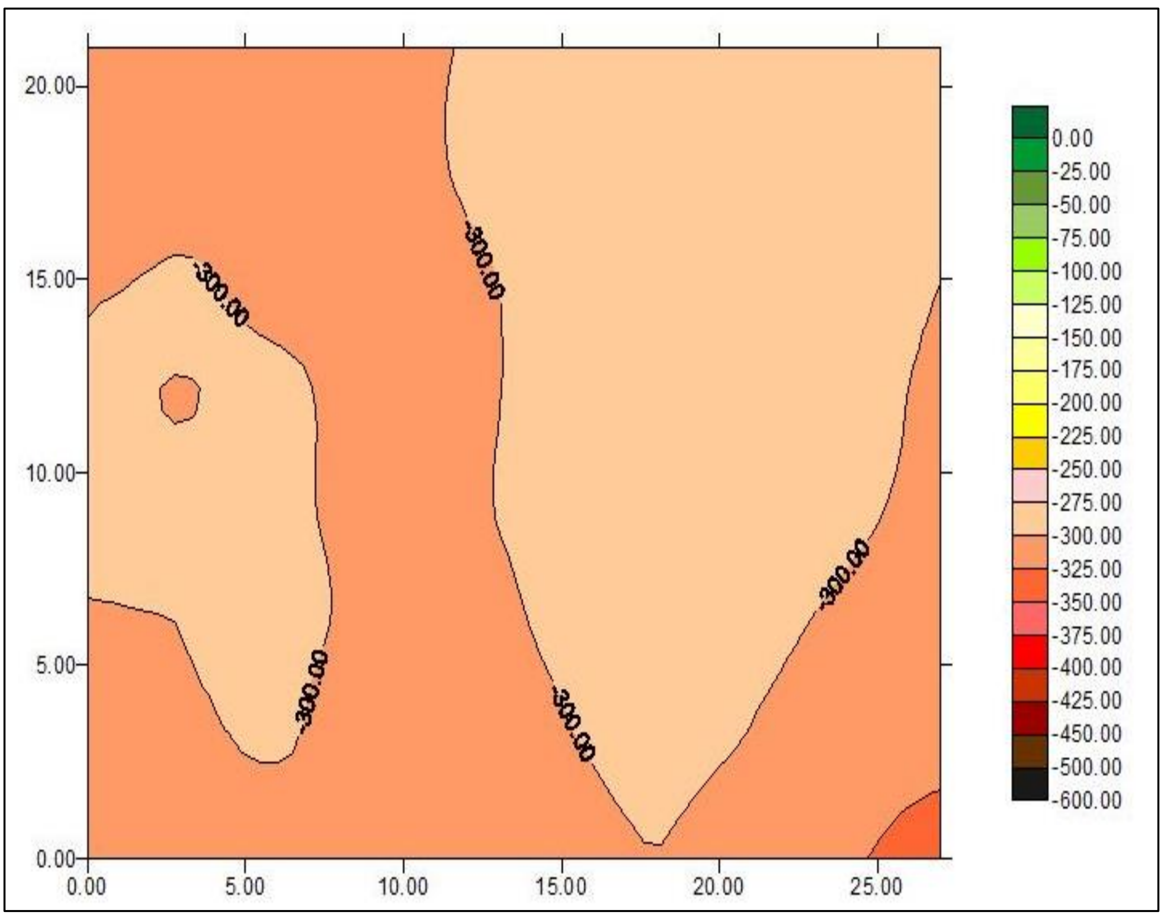


Figure E.92 Shaft 32, test 2 data map

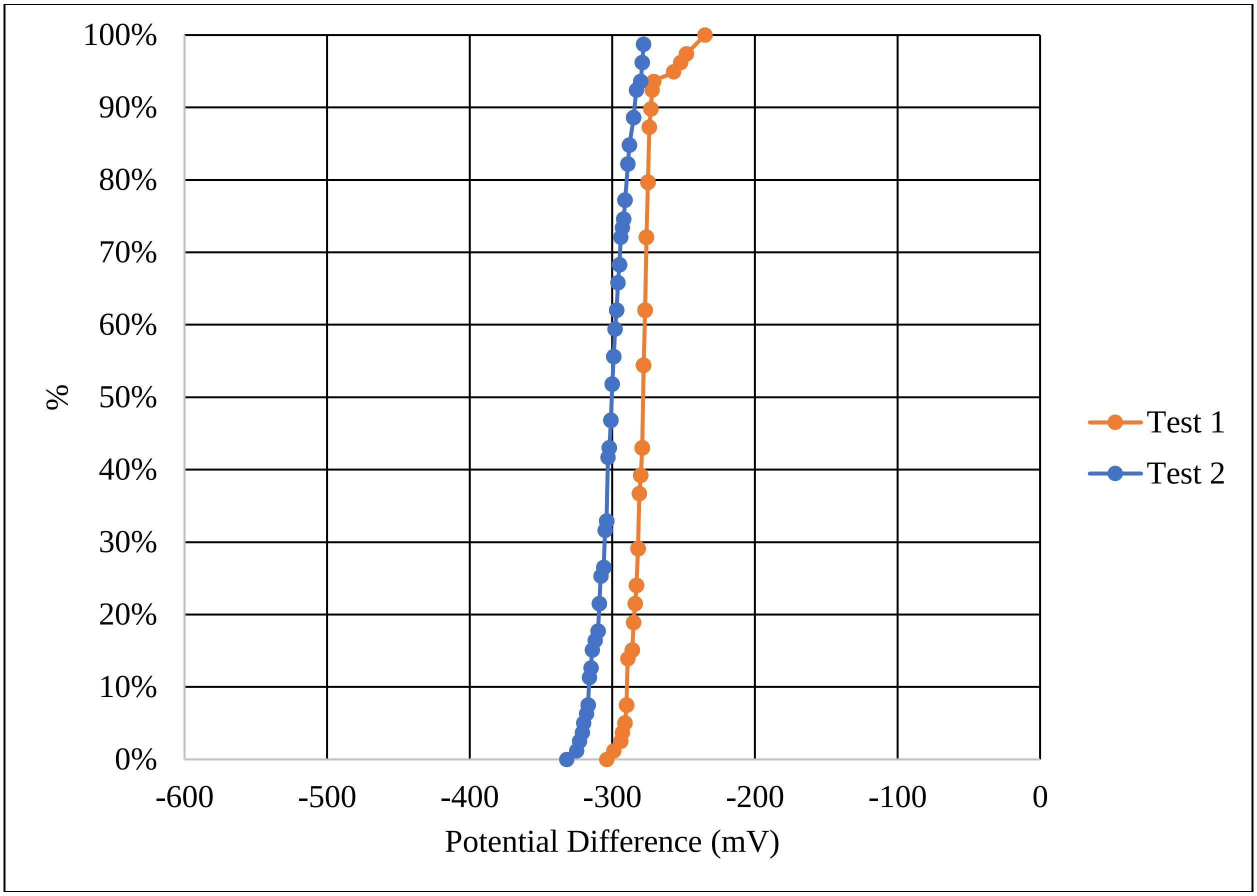


Figure E.93 Shaft 32 percentile distributions

Table E.63 Shaft 33, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-212	-222	-223	-232	-225	-229	-231	-239
3"	-207	-213	-229	-226	-230	-221	-223	-240
6"	-207	-212	-216	-219	-26	-223	-230	-223
9"	-206	-210	-225	-221	-218	-218	-221	-211
12"	-205	-218	-220	-231	-228	-224	-225	-218
15"	-213	-216	-218	-227	-225	-220	-212	-210
18"	-209	-214	-215	-225	-227	-223	-218	-211
21"	-202	-213	-221	-228	-227	-223	-220	-205
24"	-202	-220	-227	-231	-230	-222	-351	-366
27"	-203	-219	-224	-233	-226	-365	-378	-311

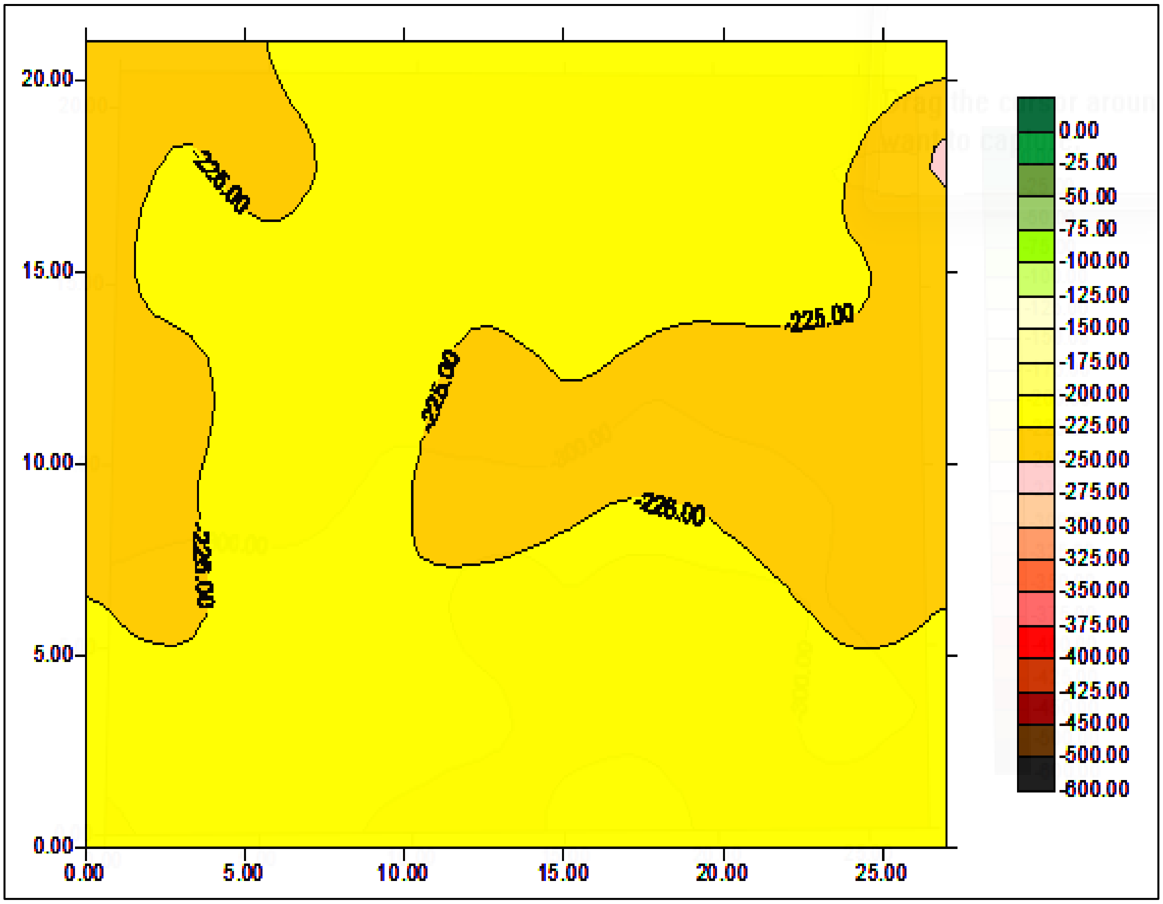


Figure E.94 Shaft 33, test 1 data map

Table E.64 Shaft 33, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-394	-390	-375	-364	-351	-353	-345	-342
3"	-394	-395	-373	-367	-353	-349	-348	-339
6"	-396	-384	-378	-368	-355	-353	-345	-343
9"	-396	-395	-379	-372	-356	-353	-348	-340
12"	-394	-392	-380	-371	-356	-353	-345	-337
15"	-397	-390	-377	-370	-360	-350	-346	-339
18"	-398	-386	-379	-368	-359	-348	-342	-346
21"	-392	-385	-374	-371	-357	-350	-246	-349
24"	-393	-396	-376	-367	-360	-352	-353	-350
27"	-395	-389	-381	-364	-357	-352	-344	-343

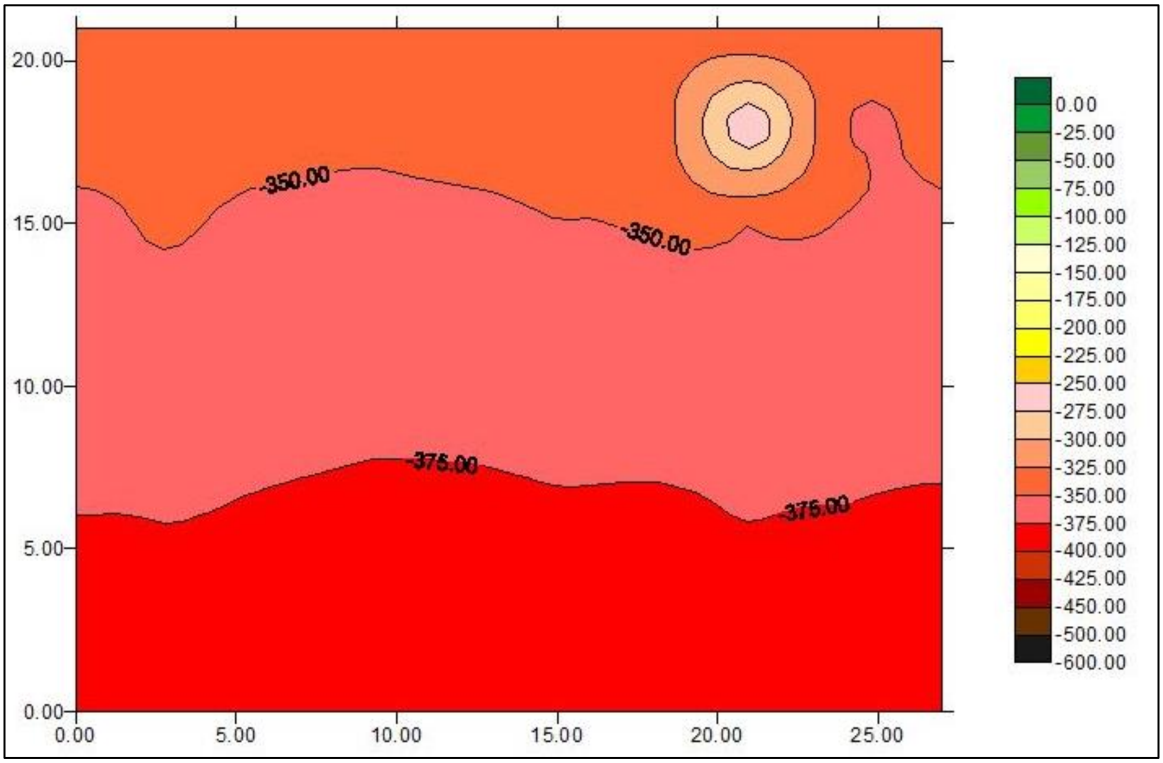


Figure E.95 Shaft 33, test 2 data map

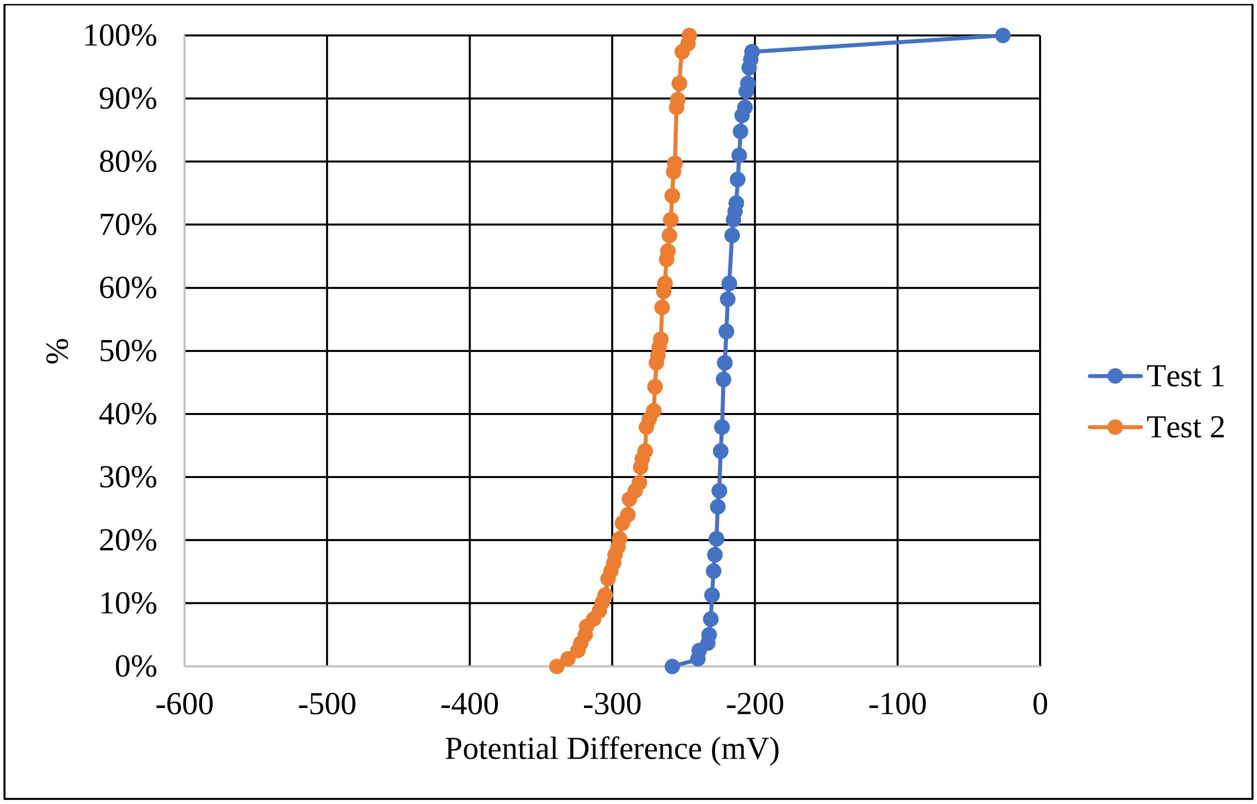


Figure E.96 Shaft 33 percentile distributions

Table E.65 Shaft 34, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-259	-264	-264	-260	-259	-253	-261	-266
3"	-262	-261	-273	-265	-263	-252	-262	-265
6"	-268	-268	-268	-257	-265	-257	-261	-261
9"	-271	-267	-274	-266	-269	-263	-258	-257
12"	-268	-270	-279	-272	-259	-259	-264	-260
15"	-271	-278	-291	-276	-264	-260	-263	-264
18"	-283	-290	-294	-284	-275	-268	-270	-262
21"	-298	-295	-289	-281	-274	-281	-276	-258
24"	-311	-304	-293	-288	-272	-271	-267	-256
27"	-318	-310	-296	-284	-274	-279	-261	-254

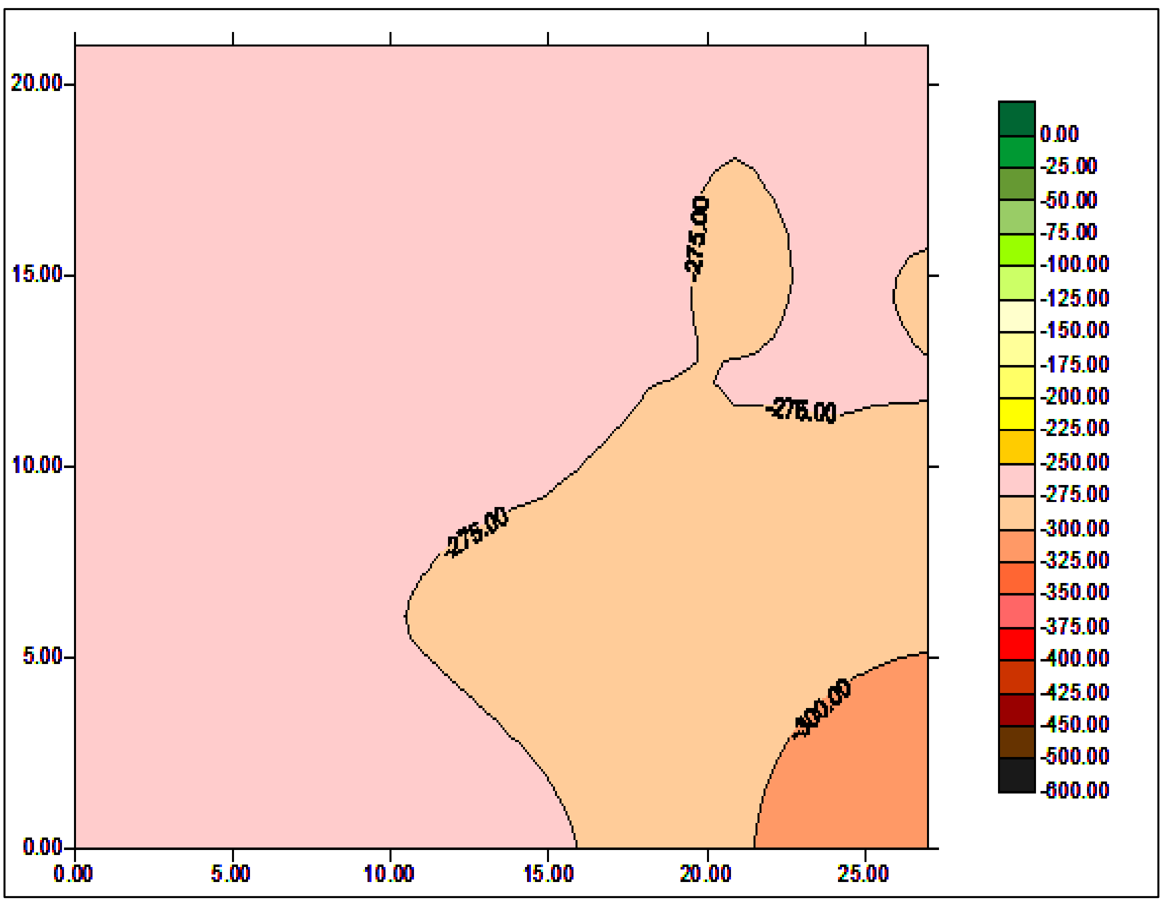


Figure E.97 Shaft 34, test 1 data map

Table E.66 Shaft 34, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-317	-325	-323	-331	-328	-337	-341	-355
3"	-324	-320	-324	-326	-325	-331	-342	-345
6"	-321	-323	-329	-326	-327	-331	-337	-347
9"	-330	-340	-333	-335	-332	-336	-345	-351
12"	-340	-343	-339	-344	-346	-352	-356	-365
15"	-351	-355	-353	-357	-361	-371	-372	-373
18"	-351	-353	-375	-364	-367	-380	-394	-409
21"	-356	-355	-367	-368	-375	-390	-408	-433
24"	-349	-358	-374	-373	-385	-399	-425	-447
27"	-353	-368	-375	-379	-389	-410	-436	-461

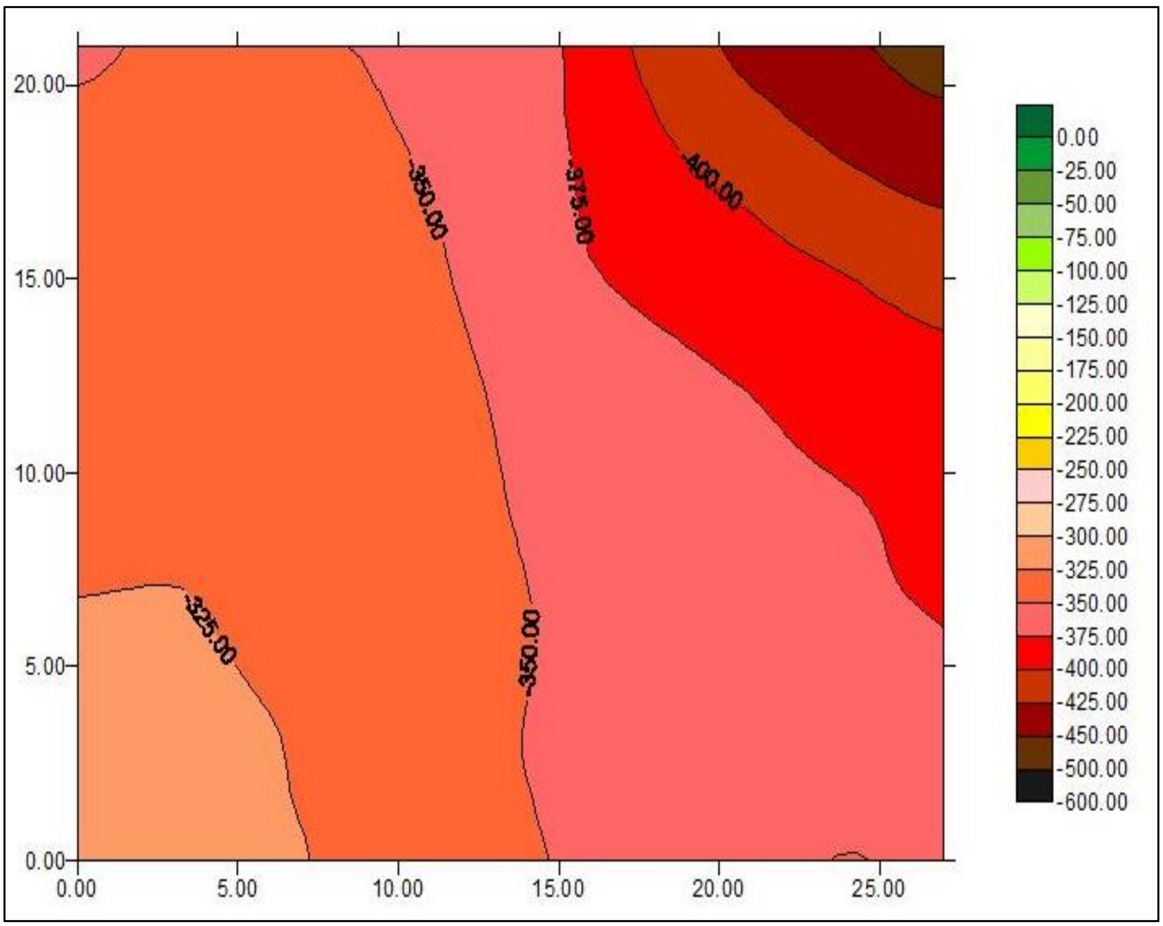


Figure E.98 Shaft 34, test 2 data map

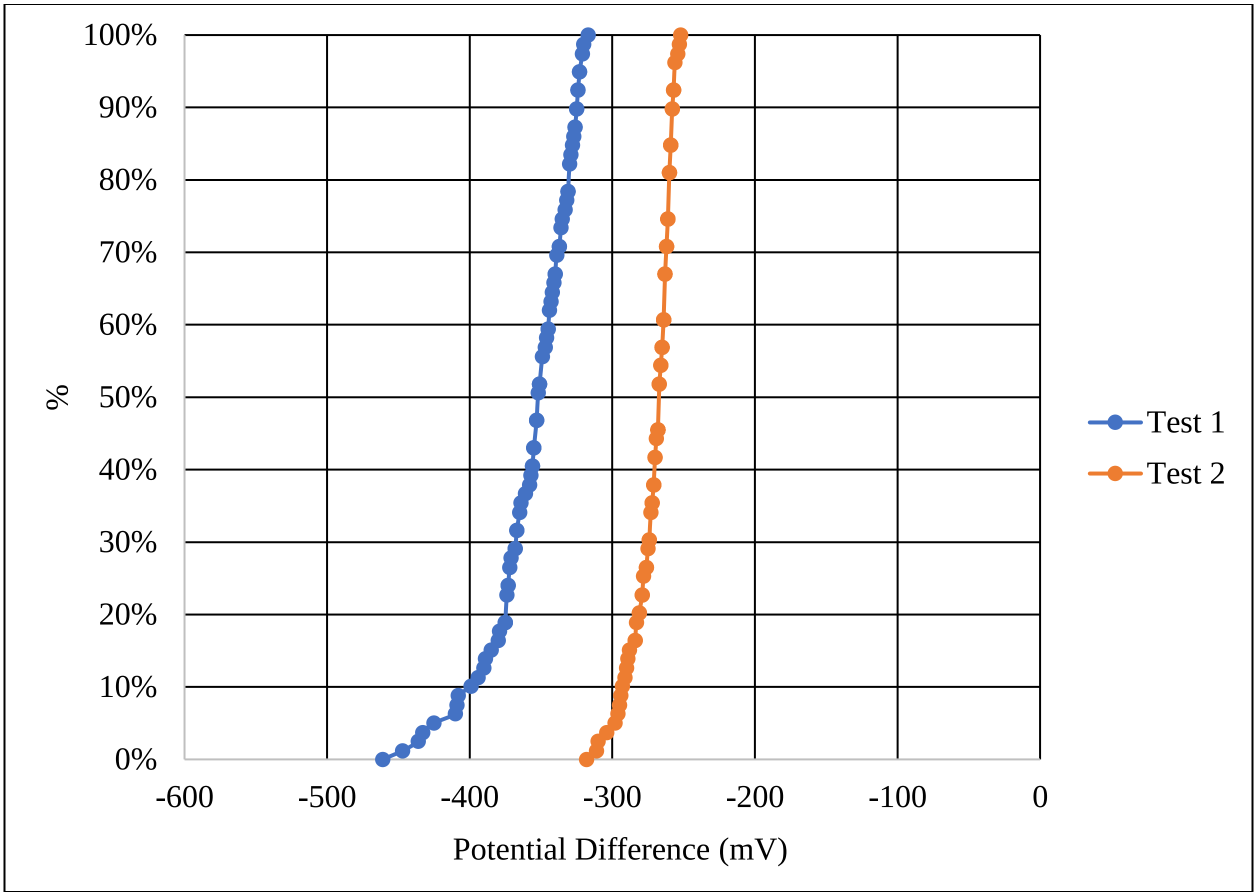


Figure E.99 Shaft 34 percentile distributions

Table E.67 Shaft 35, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-582	-560	-545	-530	-513	-496	-591	-483
3"	-578	-557	-539	-537	-519	-509	-513	-497
6"	-572	-567	-560	-551	-530	-530	-517	-498
9"	-577	-578	-582	-572	-553	-539	-521	
12"	-610	-590	-591	-598	-585	-571	-535	-524
15"	-623	-588	-598	-603	-591	-586	-548	-544
18"	-610	-575	-578	-587	-579	-571	-561	-558
21"	-588	-567	-569	-573	-574	-586	-571	
24"	-600	-570	-563	-594	-587	-578	-570	-576
27"	-603	-577	-564	-574	-592	-585	-586	-601

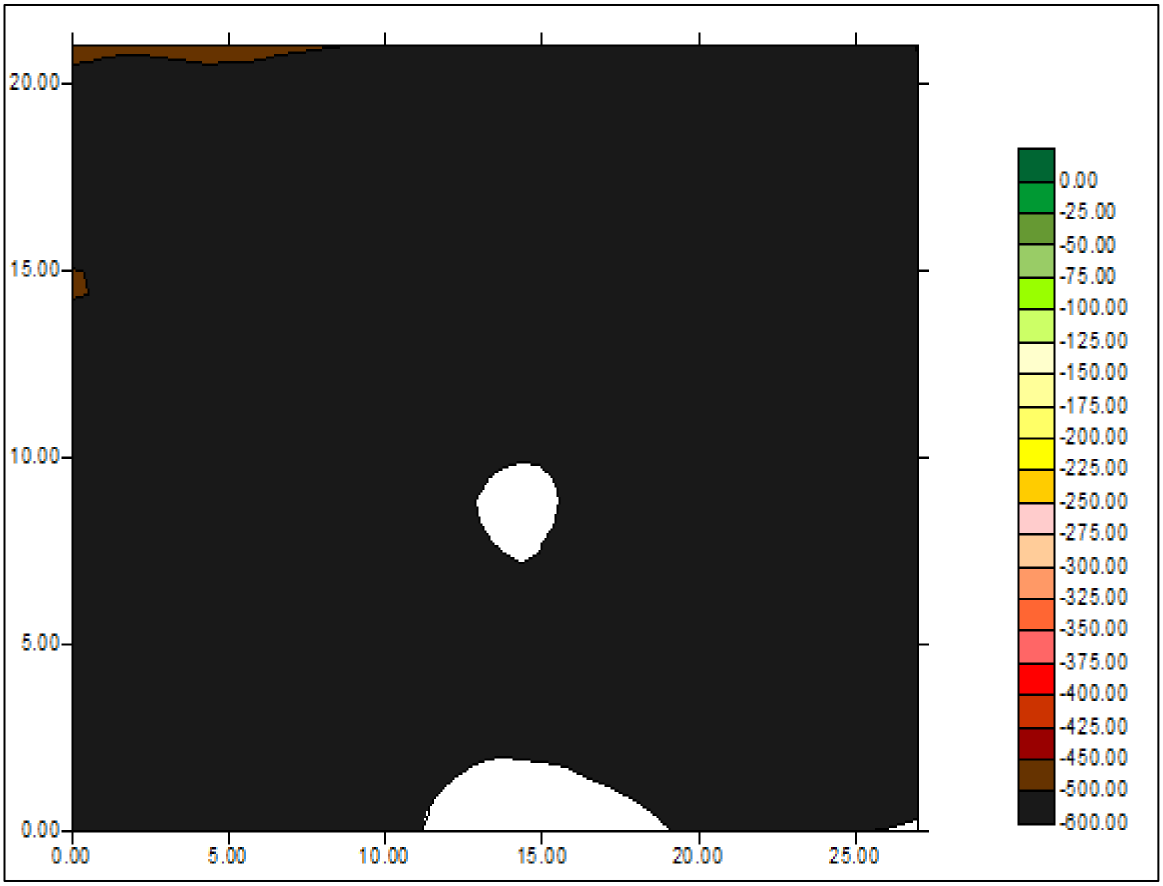


Figure E.100 Shaft 35, test 1 data map

Table E.68 Shaft 35, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-498	-531	-568	-591	-590	-593	-595	-589
3"		-511	-535	-561	-567	-573	-572	-592
6"	-523	-540	-550	-553	-571	-570	-584	-587
9"	-525	-540	-552	-550	-570	-559	-586	-597
12"	-519	-531	-556	-553	-560	-561	-577	-598
15"	-534	-560	-561	-574	-585	-593	-602	-611
18"		-558	-570	-576	-593	-593	-619	-631
21"	-584	-587	-582	-593	-592	-593	-610	-628
24"		-598	-590	-598	-590	-604	-607	-616
27"	-550	-571	-582	-597	-585	-598	-595	-603

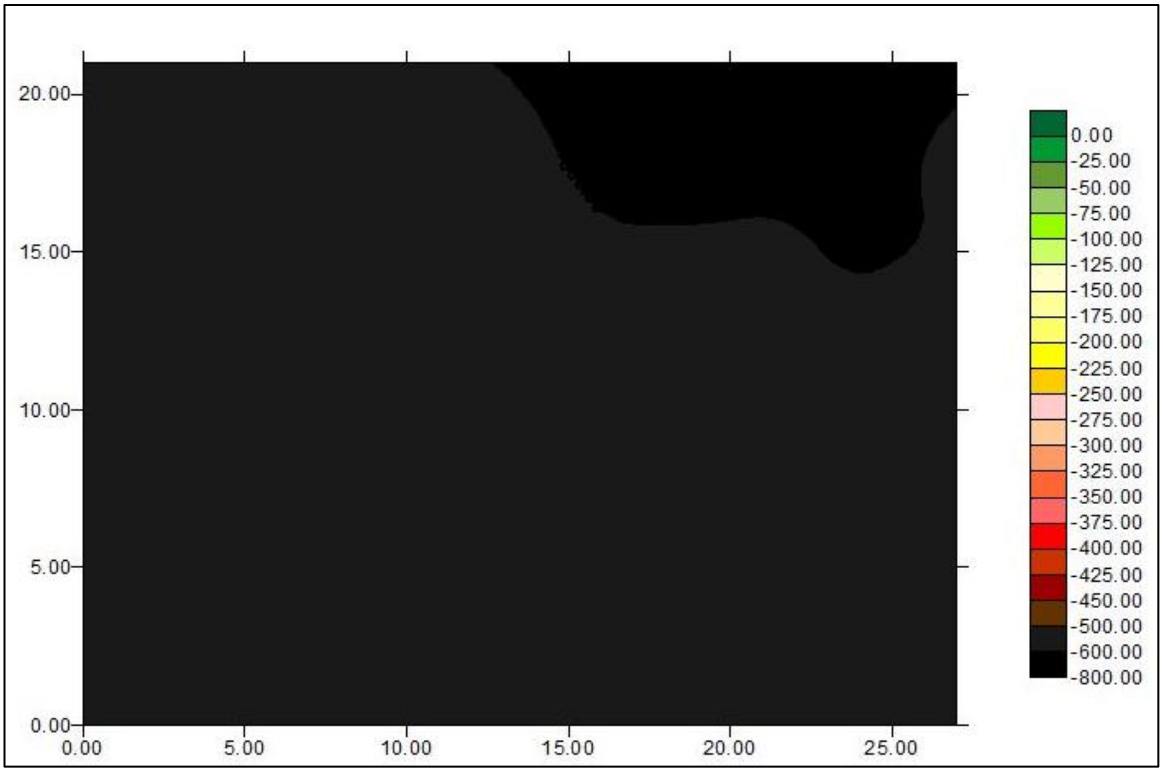


Figure E.101 Shaft 35, test 2 data map

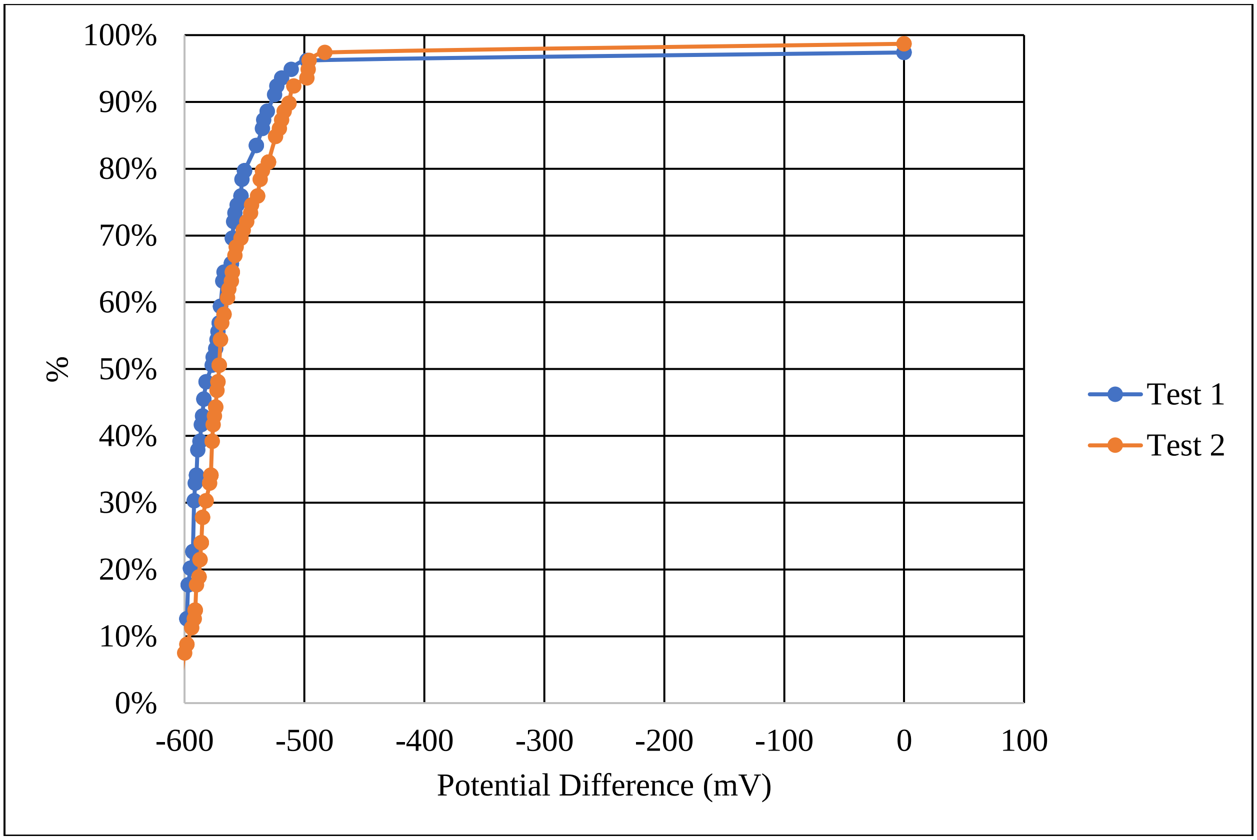


Figure E.102 Shaft 35 percentile distributions

Table E.69 Shaft 36, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-283	-284	-295	-288	-284	-286	-283	-284
3"	-278	-286	-289	-283	-285	-281	-277	-280
6"	-290	-269	-273	-280	-300	-285	-283	-289
9"	-289	-286	-284	-288	-307	-291	-280	-281
12"	-276	-266	-277	-290	-293	-284	-282	-283
15"	-287	-266	-284	-278	-278	-271	-280	-272
18"	-278	-263	-287	-285	-271	-276	-278	-271
21"	-283	-256	-271	-288	-272	-273	-261	-271
24"	-258	-247	-262	-256	-251	-254	-254	-266
27"	-257	-250	-261	-252	-248	-259	-253	-260

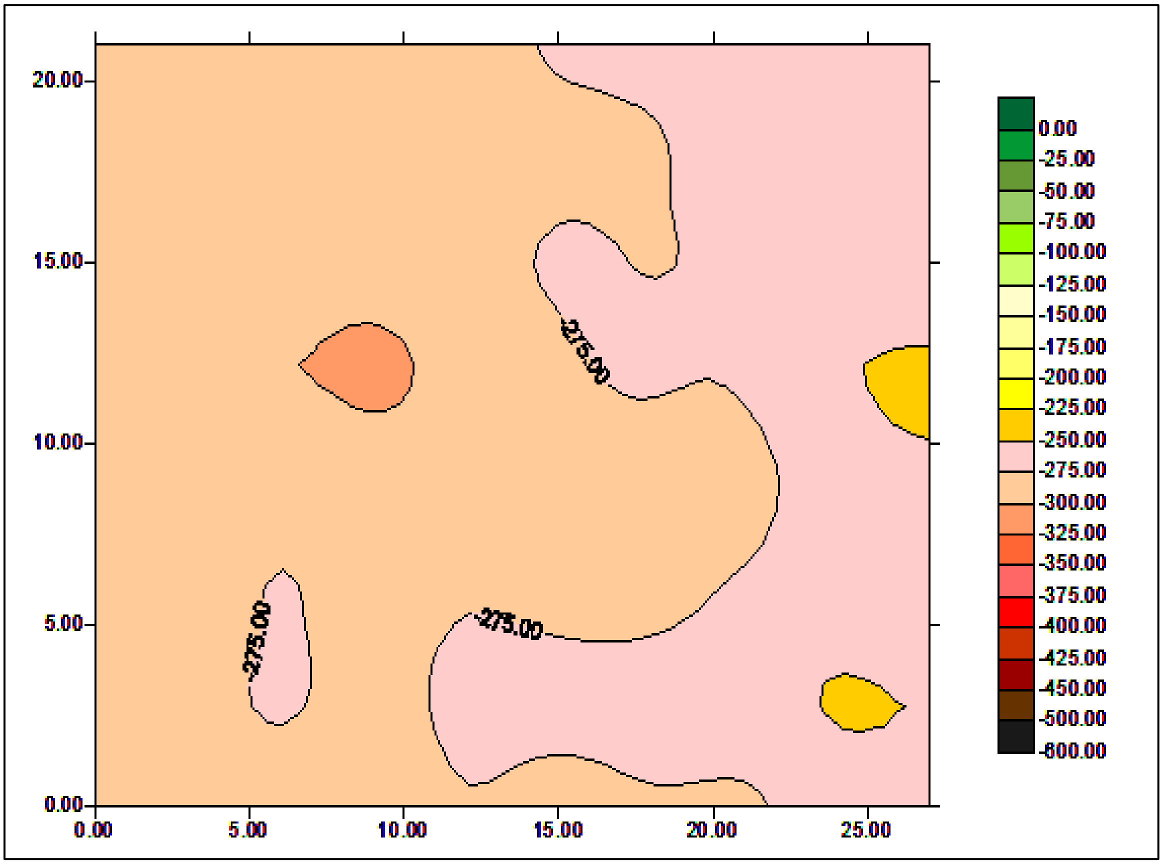


Figure E.103 Shaft 36, test 1 data map

Table E.70 Shaft 36, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-345	-341	-342	-334	-336	-328	-321	-329
3"	-349	-347	-341	-332	-326	-330	-328	-332
6"	-354	-345	-335	-331	-327	-327	-331	-333
9"	-357	-348	-332	-328	-331	-330	-327	-333
12"	-362	-347	-333	-330	-331	-329	-327	-325
15"	-358	-347	-336	-335	-317	-324	-332	-331
18"	-350	-353	-341	-330	-332	-327	-324	-333
21"	-346	-340	-337	-334	-327	-331	-330	-328
24"	-339	-334	-331	-332	-331	-332	-325	-325
27"	-329	-334	-329	-332	-331	-326	-311	-303

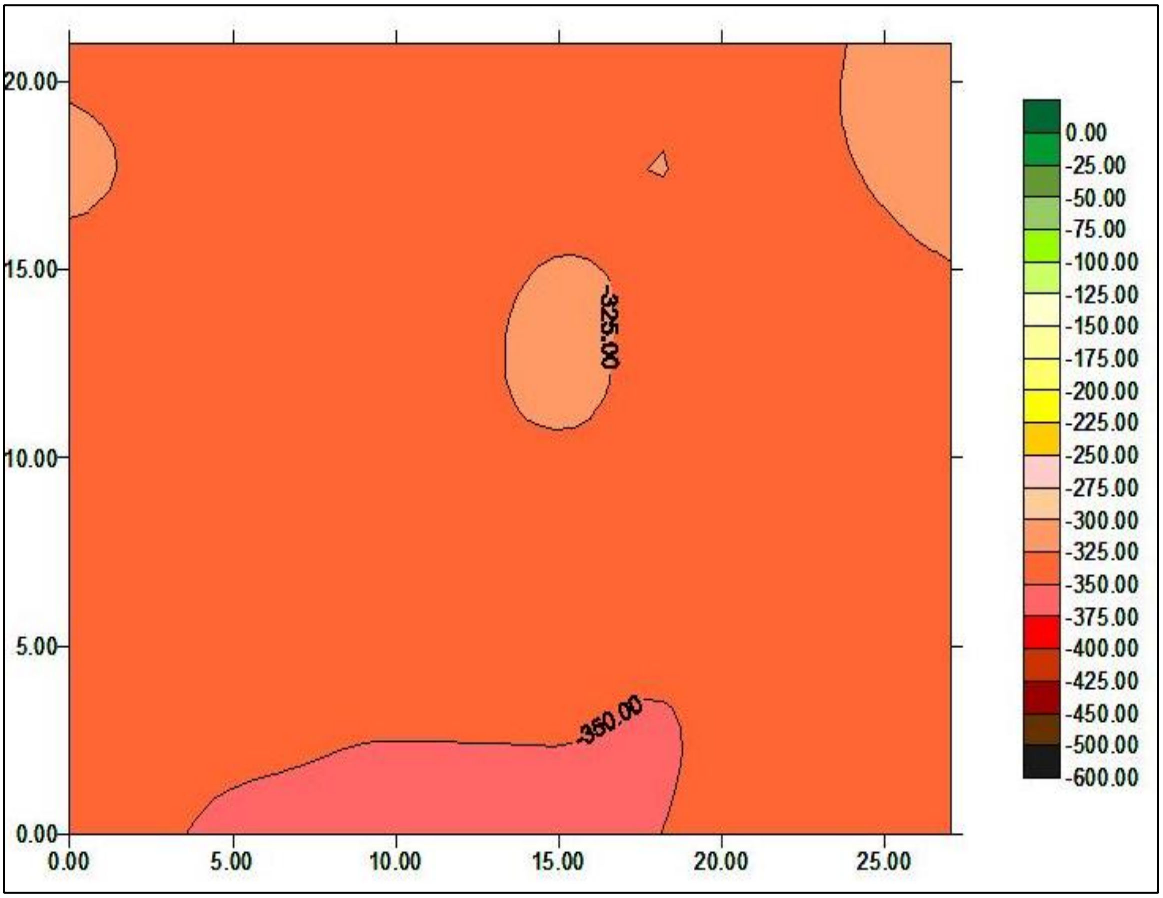


Figure E.104 Shaft 36, test 2 data map

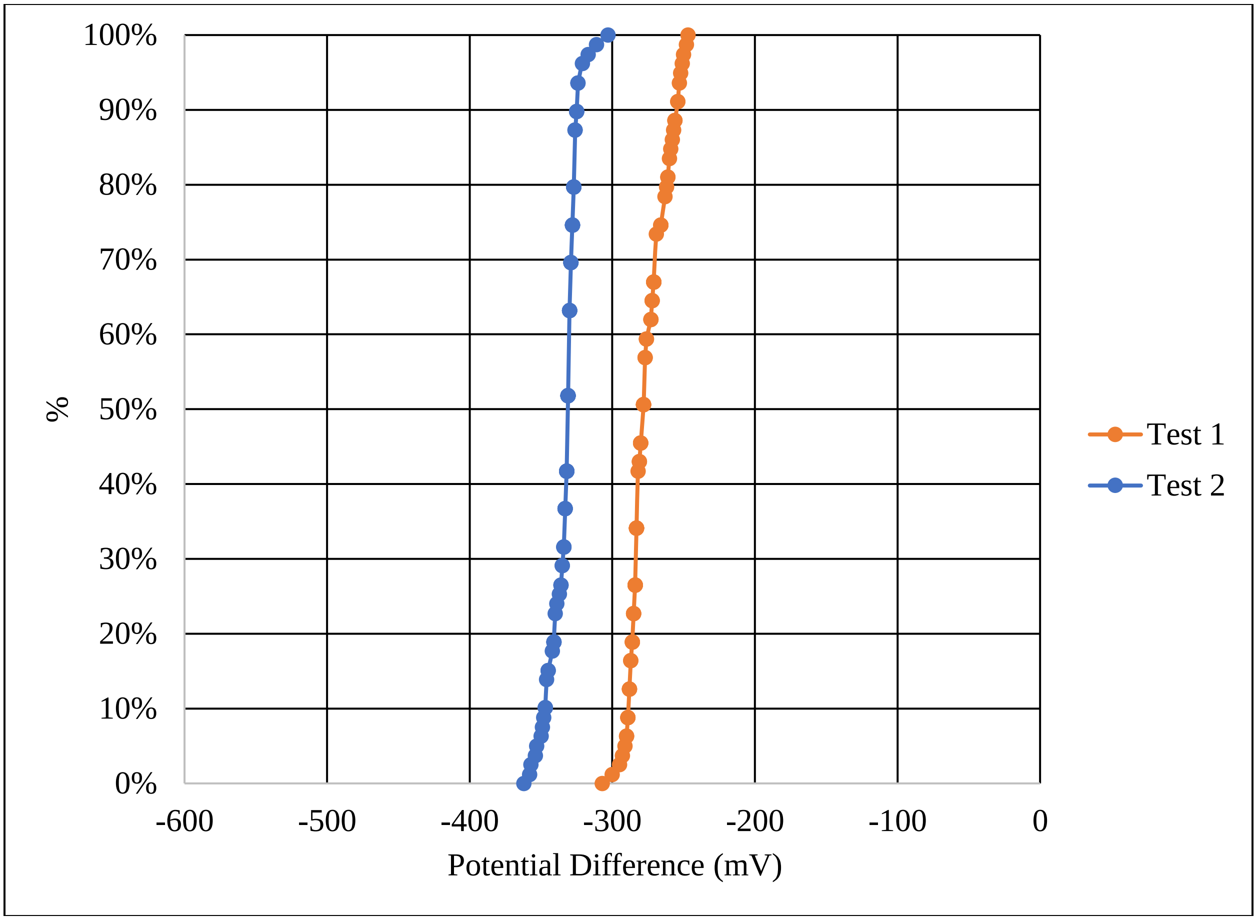


Figure E.105 Shaft 36 percentile distributions

Table E.71 Shaft 37, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-215	-233	-229	-235	-249	-259	-253	-243
3"	-217	-232	-241	-248	-250	-255	-256	-271
6"	-211	-221	-224	-230	-242	-250	-273	-268
9"	-215	-223	-226	-234	-245	-257	-252	-280
12"	-209	-230	-228	-243	-247	-258	-260	-278
15"	-209	-218	-221	-233	-239	-248	-261	-264
18"	-208	-214	-216	-231	-237	-248	-261	-268
21"	-213	-202	-205	-230	-236	-254	-265	-278
24"	-201	-203	-217	-224	-230	-252	-281	-278
27"	-207	-203	-218	-225	-230	-246	-292	-273

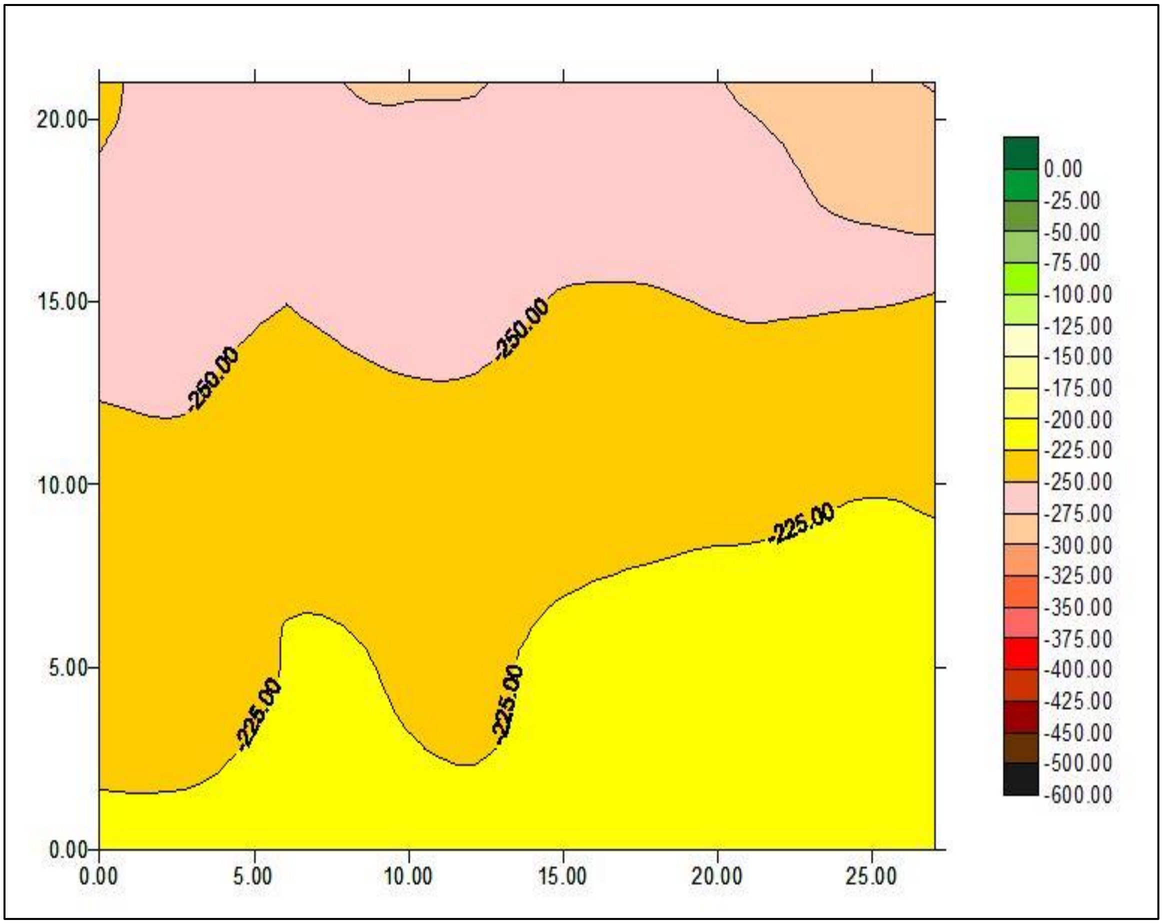


Figure E.106 Shaft 37, test 1 data map

Table E.72 Shaft 37, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-281	-283	-256	-222	-217	-210	-214	-217
3"	-274	-288	-264	-224	-220	-209	-210	-209
6"	-269	-277	-262	-221	-219	-221	-233	-202
9"	-273	-267	-249	-226	-221	-216	-230	-206
12"	-285	-274	-269	-240	-234	-221	-231	-215
15"	-267	-266	-275	-236	-232	-212	-226	-191
18"	-256	-278	-270	-236	-224	-219	-219	-201
21"	-260	-277	-256	-235	-224	-218	-218	-209
24"	-267	-268	-273	-249	-241	-226	-234	-210
27"	-241	-273	-266	-240	-227	-218	-214	-206

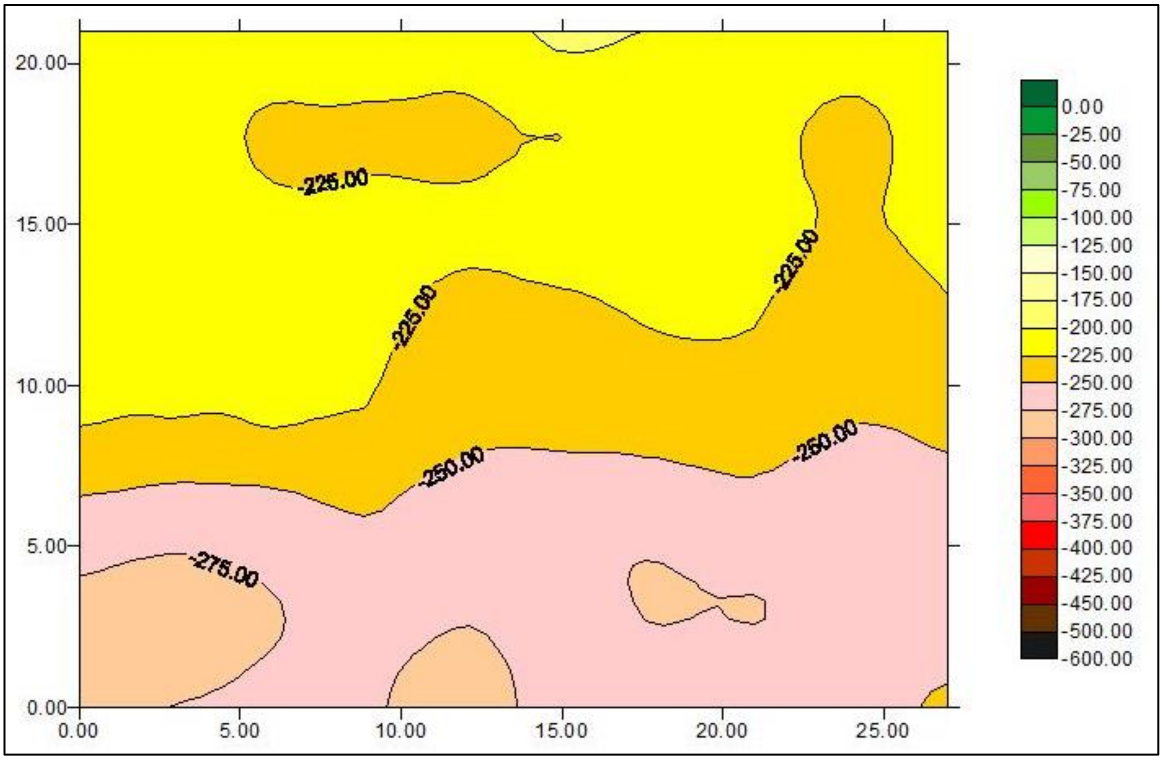


Figure E.107 Shaft 37, test 2 data map

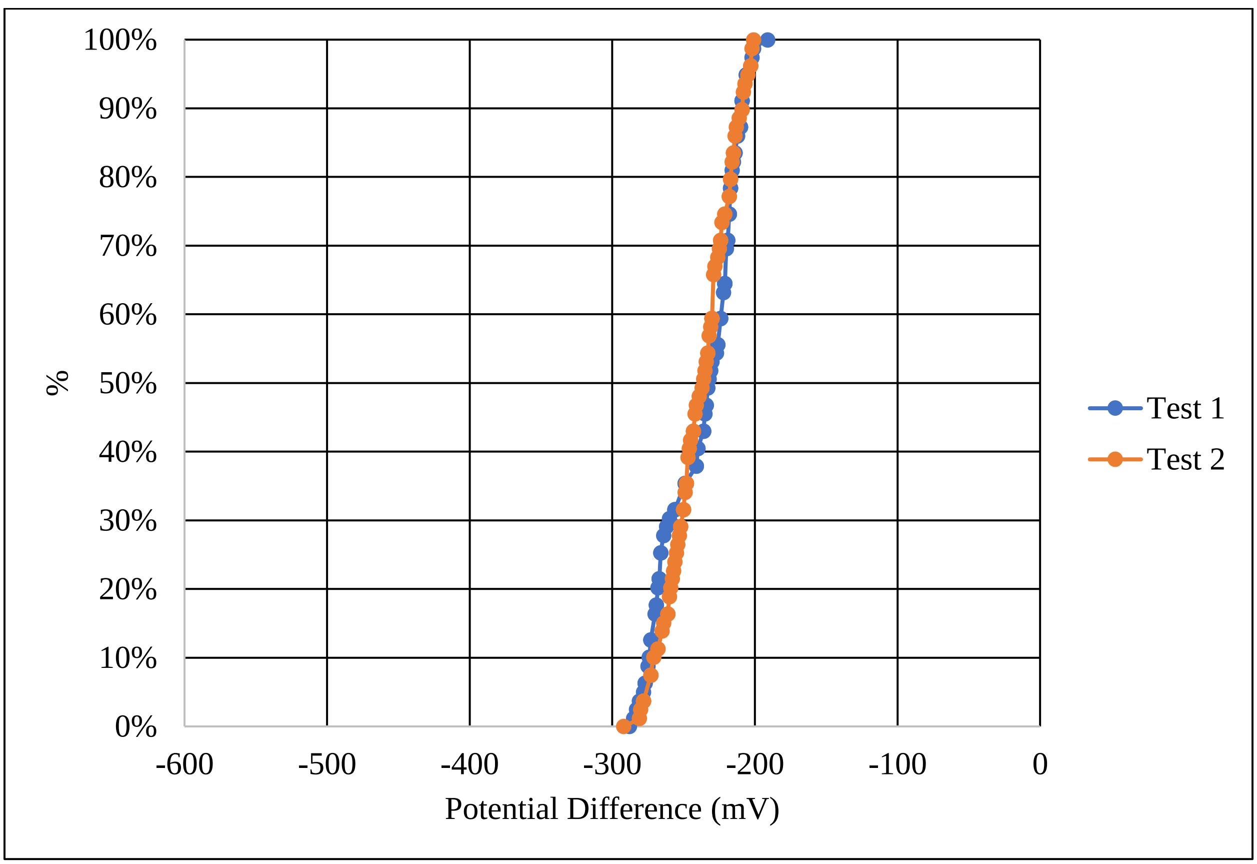


Figure E.108 Shaft 37 percentile distributions

Table E.73 Shaft 38, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-241	-239	-232	-240	-253	-250	-255	-253
3"	-232	-251	-268	-257	-243	-264	-265	-275
6"	-250	-267	-268	-248	-250	-242	-268	-249
9"	-243	-247	-266	-250	-250	-246	-167	-249
12"	-246	-254	-270	-254	-263	-259	-268	-267
15"	-250	-264	-270	-243	-250	-251	-307	-266
18"	-252	-255	-286	-258	-261	-261	-241	-260
21"	-247	-256	-259	-268	-274	-253	-294	-285
24"	-248	-261	-278	-274	-270	-268	-331	-290
27"	-243	-258	-310	-262	-286	-266	-322	-284

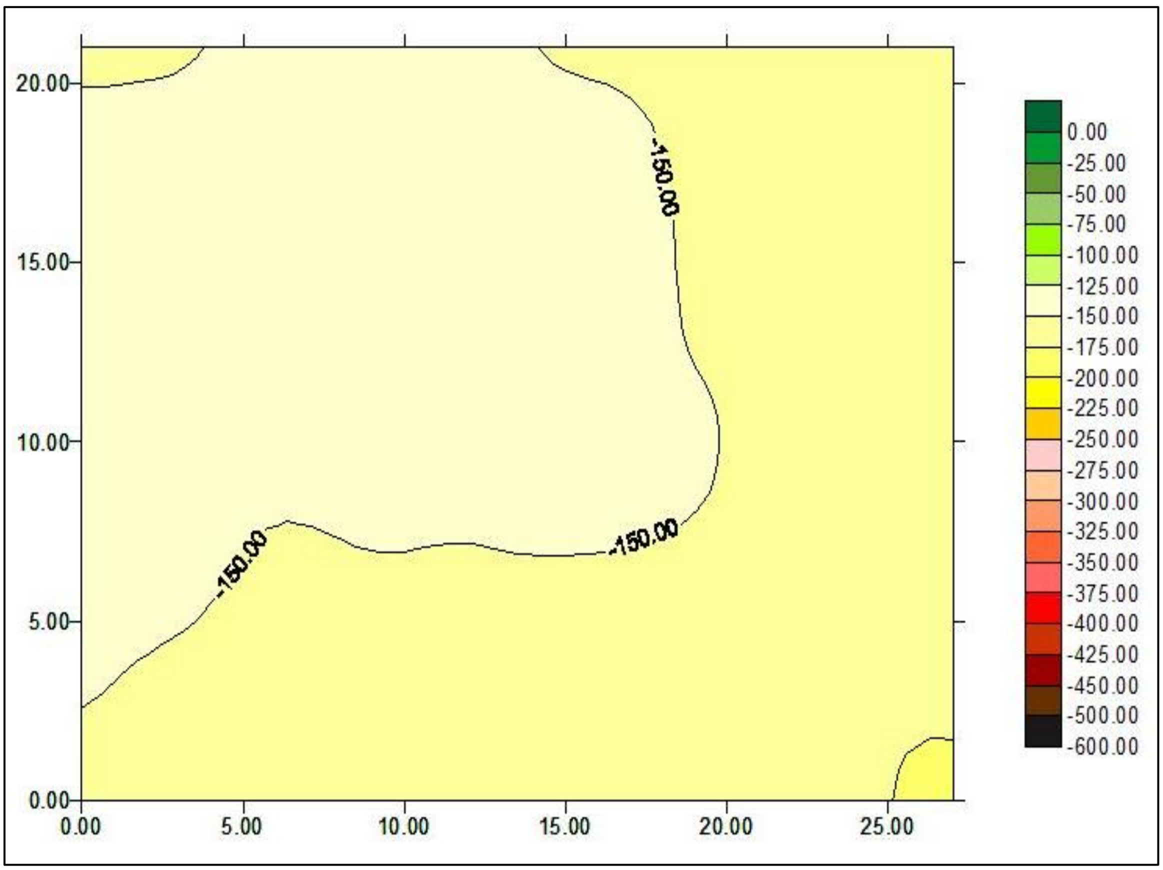


Figure E.109 Shaft 38, test 1 data map

Table E.74 Shaft 38, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-282	-268	-244	-243	-232	-241	-221	-204
3"	-342	-301	-264	-278	-265	-320	-232	-212
6"	-316	-358	-258	-285	-254	-257	-244	-227
9"	-307	-266	-257	-277	-264	-255	-242	-230
12"	-248	-233	-249	-268	-269	-255	-256	-258
15"	-296	-277	-244	-251	-276	-290	-264	-247
18"	-287	-303	-236	-254	-282	-290	-244	-223
21"	-287	-263	-238	-247	-261	-227	-237	-227
24"	-315	-288	-209	-235	-251	-238	-249	-238
27"	-262	-225	-225	-237	-251	-311	-239	-219

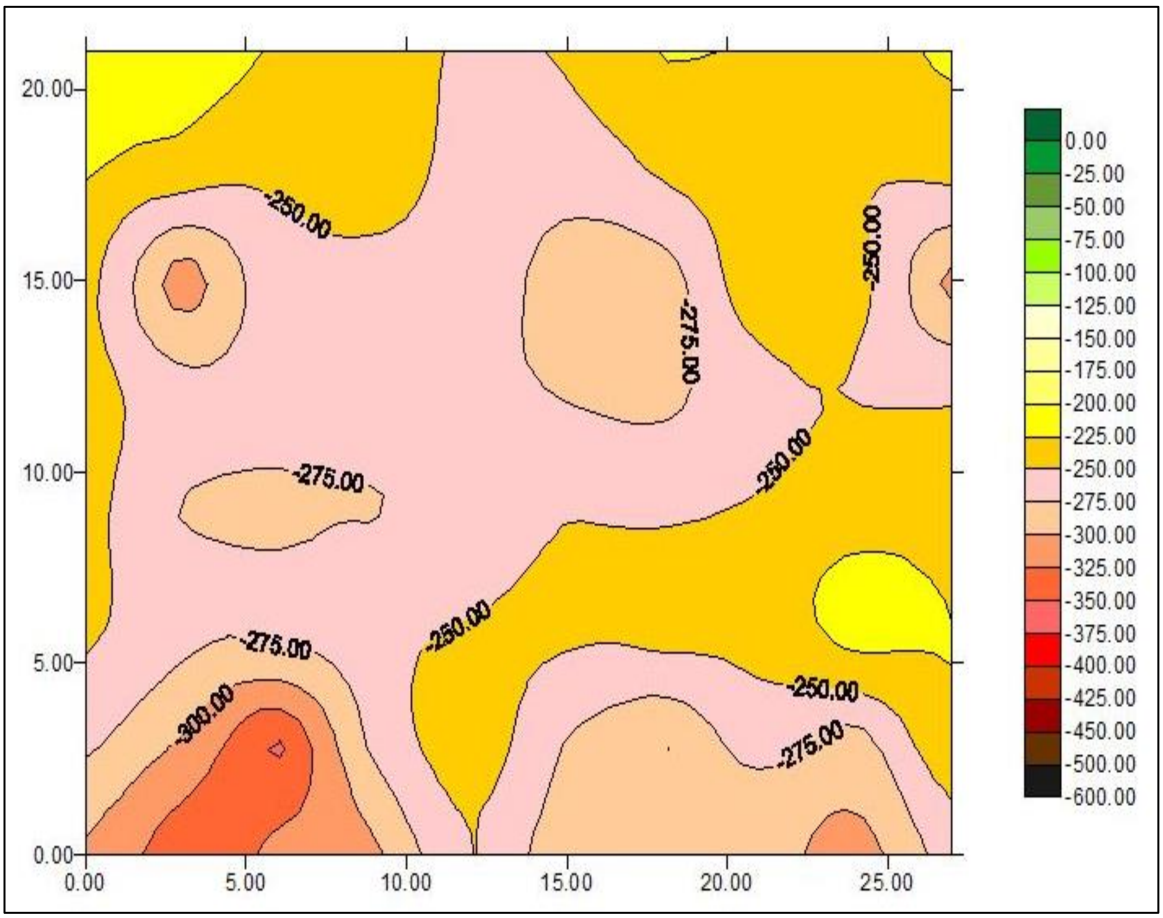


Figure E.110 Shaft 38, test 2 data map

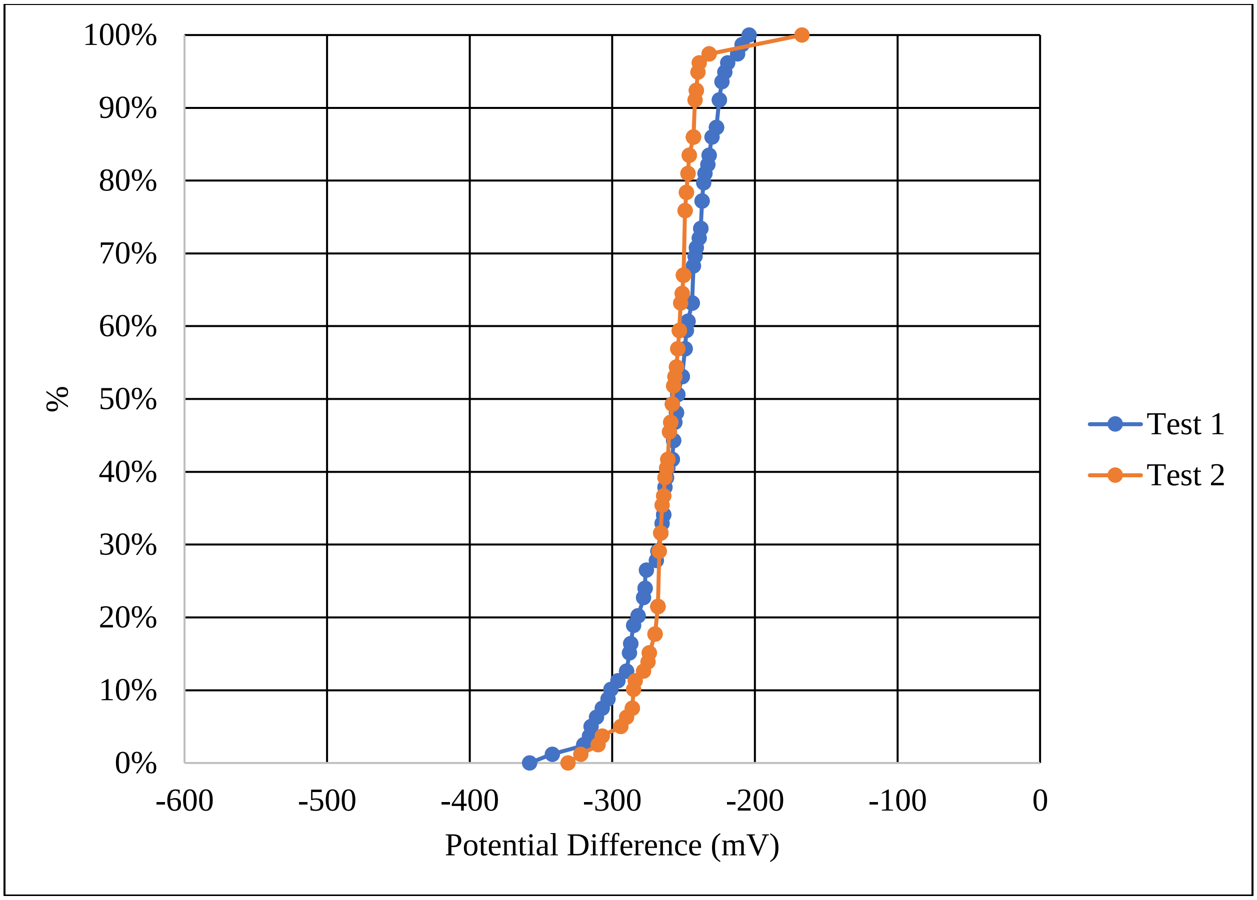


Figure E.111 Shaft 38 percentile distributions

Table E.75 Shaft 39, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-219	-246	-255	-253	-255	-250	-267	-246
3"	-233	-246	-252	-244	-239	-242	-270	-261
6"	-220	-244	-235	-252	-244	-251	-283	-231
9"	-233	-243	-241	-243	-256	-253	-284	-253
12"	-218	-233	-245	-244	-280	-252	-244	-255
15"	-222	-232	-277	-257	-254	-216	-245	-260
18"	-225	-243	-245	-293	-244	-239	-245	-259
21"	-233	-239	-263	-261	-232	-242	-245	-251
24"	-241	-237	-245	-245	-244	-254	-247	-268
27"	-231	-237	-243	-245	-247	-249	-252	-260

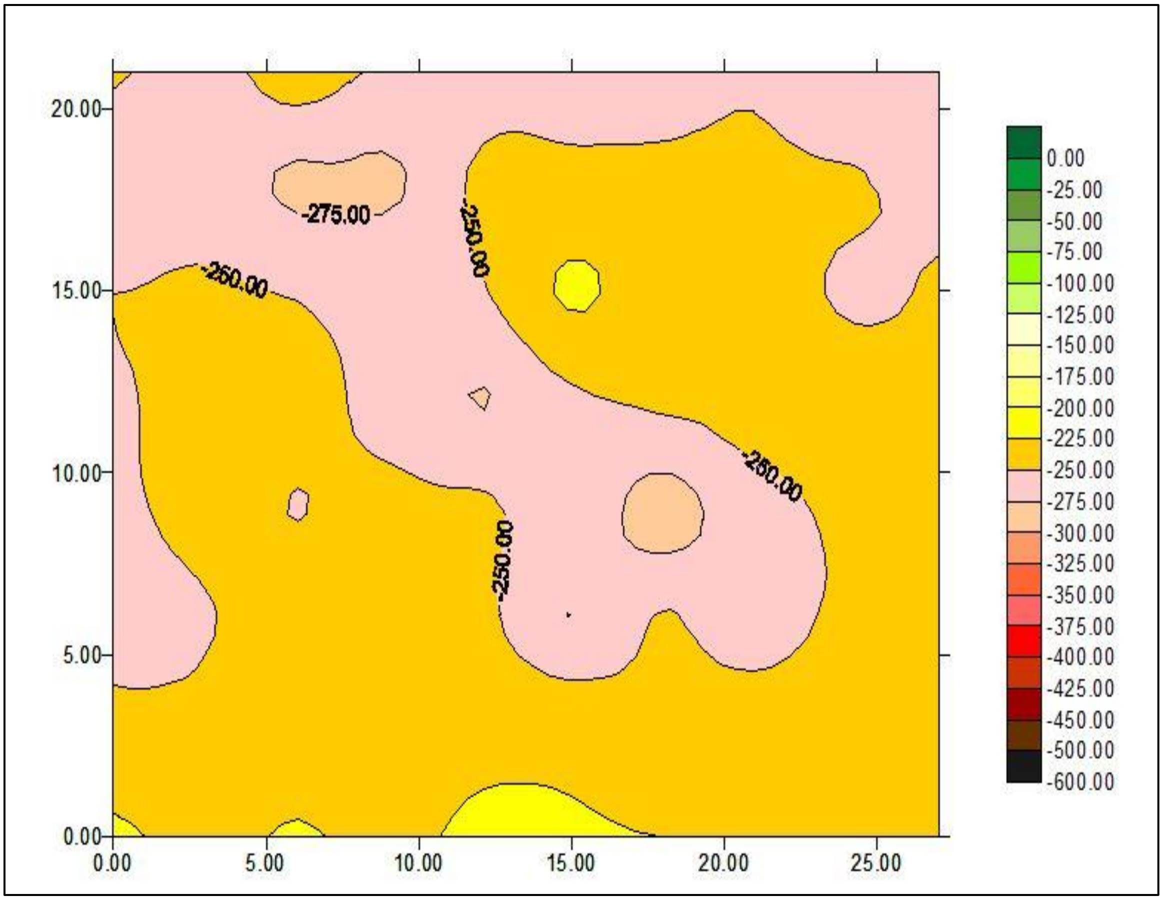


Figure E.112 Shaft 39, test 1 data map

Table E.76 Shaft 39, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-245	-257	-234	-230	-224	-222	-211	-204
3"	-260	-243	-240	-230	-226	-225	-214	-229
6"	-253	-244	-237	-221	-234	-246	-230	-215
9"	-277	-254	-232	-238	-295	-298	-222	-199
12"	-280	-248	-229	-243	-246	-240	-215	-193
15"	-264	-239	-297	-238	-229	-227	-229	-190
18"	-253	-274	-276	-264	-239	-240	-256	-205
21"	-242	-290	-248	-255	-250	-245	-256	-202
24"	-274	-266	-233	-229	-236	-240	-256	-211
27"	-252	-251	-258	-248	-251	-262	-235	-188

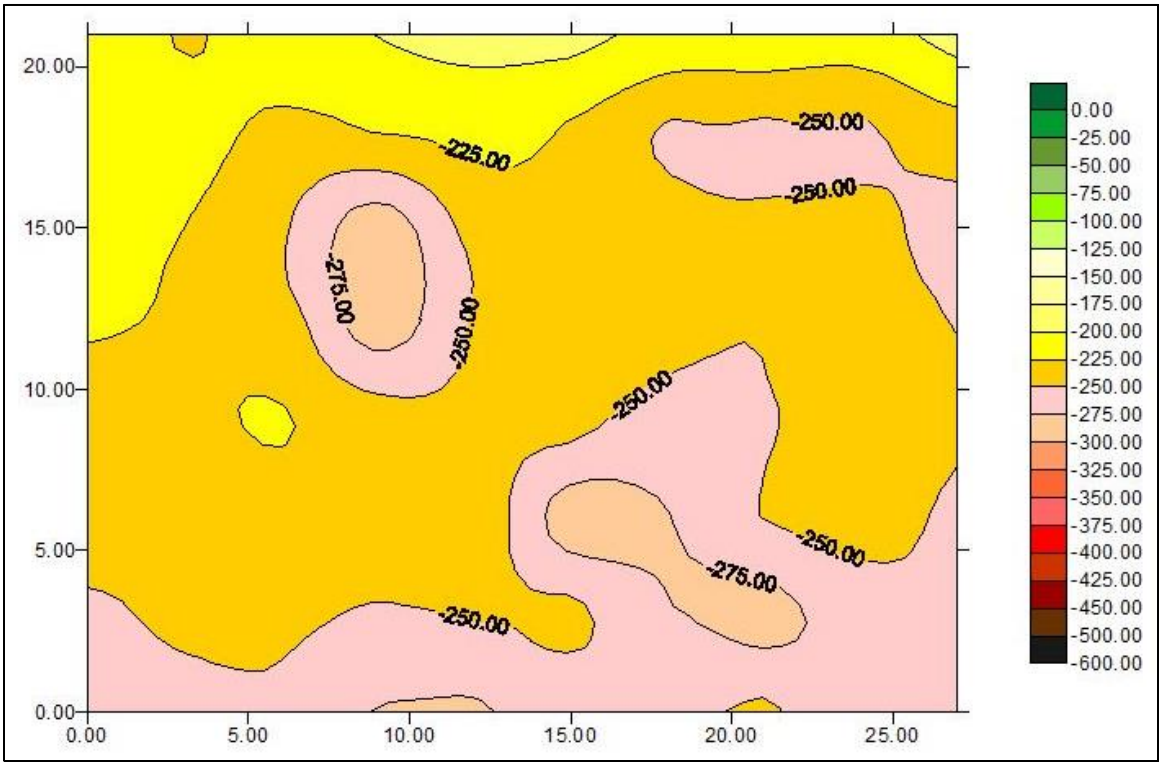


Figure E.113 Shaft 39, test 2 data map

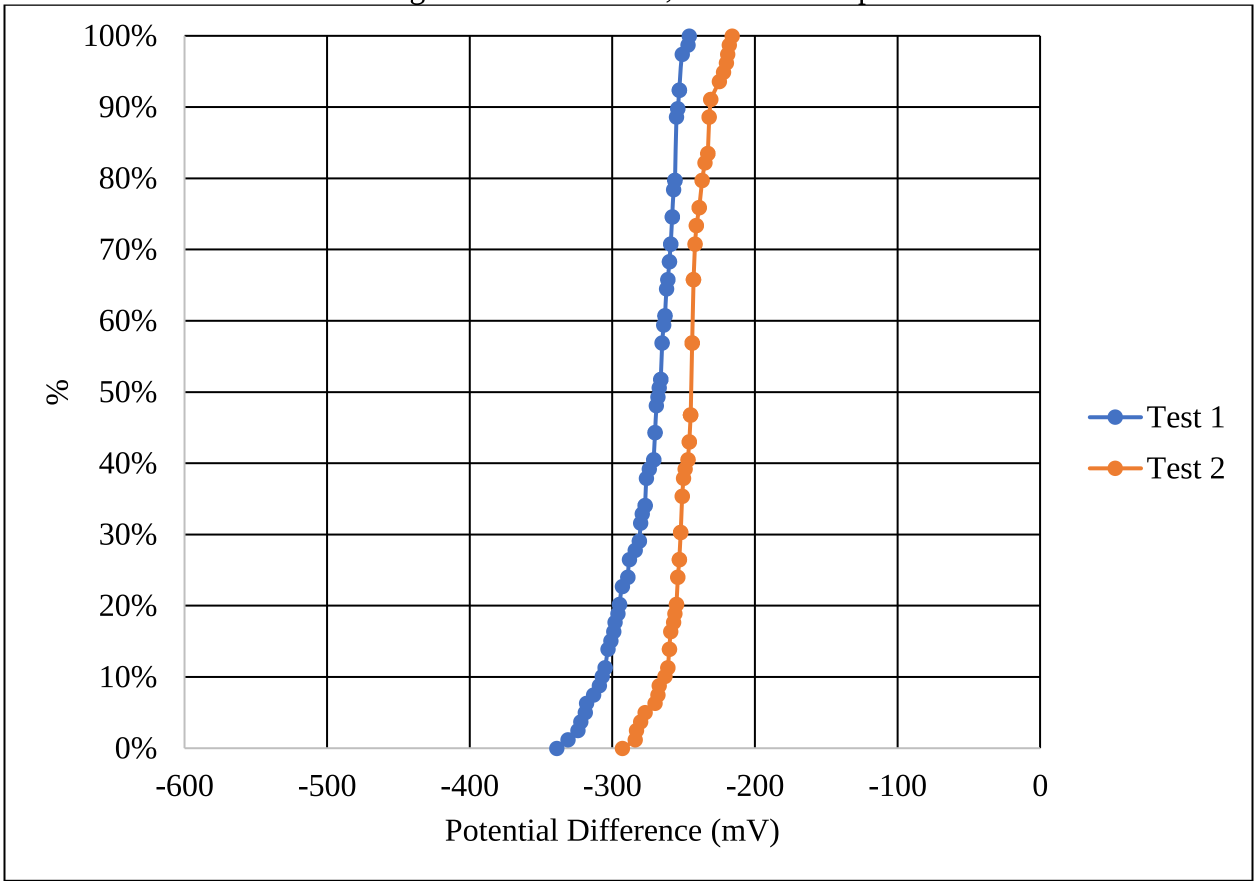


Figure E.114 Shaft 39 percentile distributions

Table E.77 Shaft 40, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-358	-343	-332	-334	-325	-321	-319	-319
3"	-355	-342	-329	-335	-316	-326	-326	-321
6"	-342	-336	-329	-326	-319	-327	-320	-321
9"	-350	-328	-336	-316	-317	-329	-318	-316
12"	-353	-346	-333	-328	-328	-332	-331	-309
15"	-352	-338	-341	-321	-333	-335	-327	-314
18"	-341	-332	-326	-306	-324	-316	-330	-307
21"	-341	-333	-324	-325	-324	-313	-329	-309
24"	-333	-333	-322	-306	-315	-306	-318	-305
27"	-327	-326	-313	-310	-309	-300	-308	-309

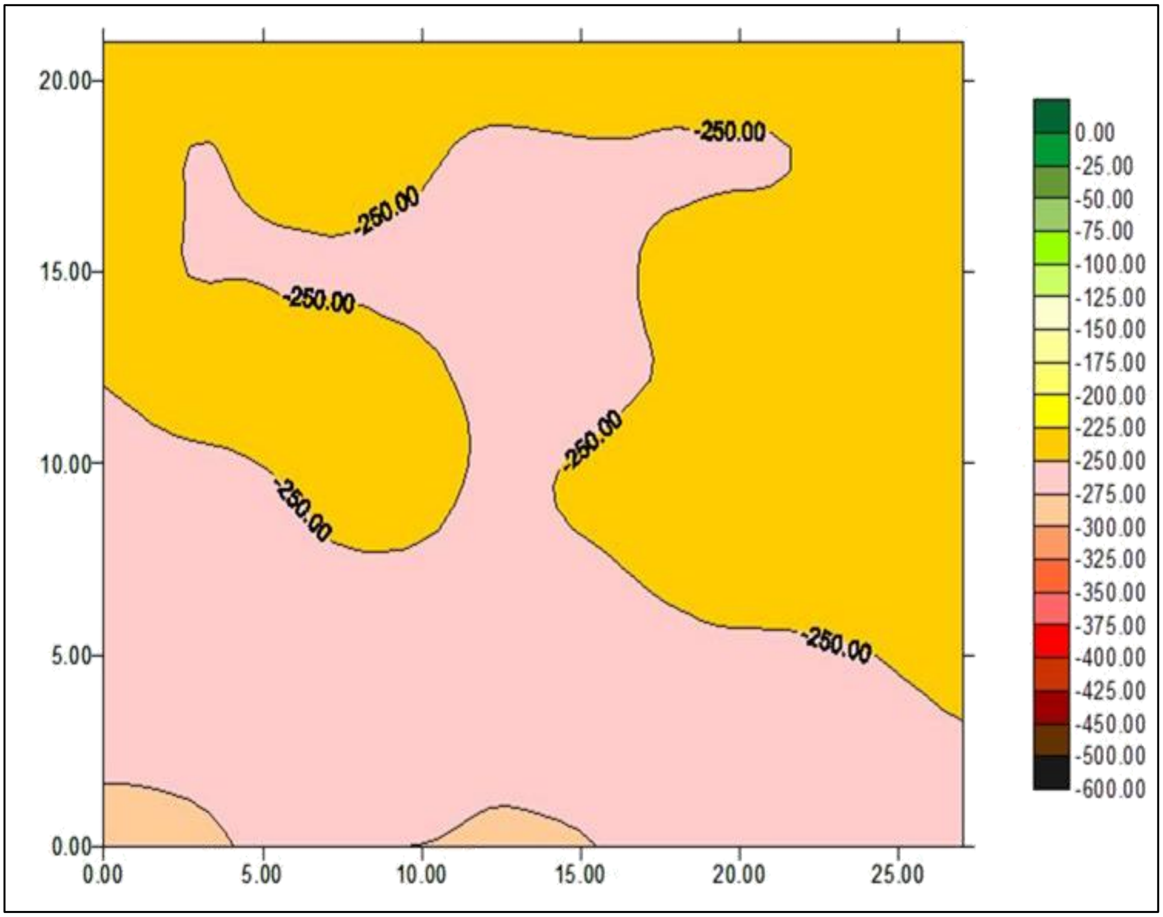


Figure E.115 Shaft 40, test 1 data map

Table E.78 Shaft 40, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-296	-310	-312	-323	-326	-333	-346	-347
3"	-294	-319	-322	-322	-326	-342	-358	-357
6"	-302	-329	-332	-332	-344	-346	-356	-368
9"	-302	-335	-338	-338	-325	-345	-357	-334
12"	-303	-343	-338	-338	-337	-348	-358	-343
15"	-288	-331	-340	-336	-353	-359	-365	-359
18"	-307	-317	-328	-336	-337	-360	-351	-368
21"	-314	-335	-341	-337	-350	-357	-371	-383
24"	-315	-335	-345	-340	-360	-355	-382	-392
27"	-320	-334	-343	-359	-368	-383	-400	-412

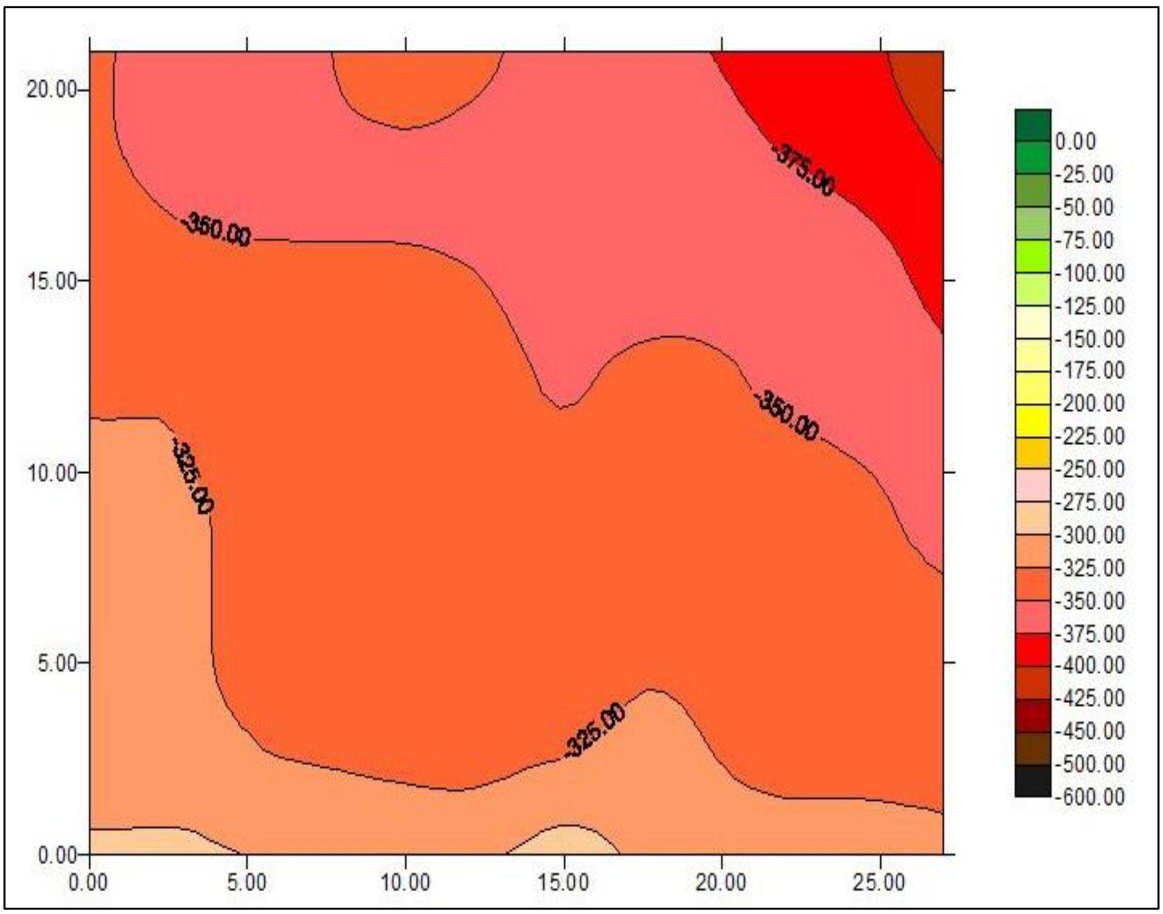


Figure E.116 Shaft 40, test 2 data map

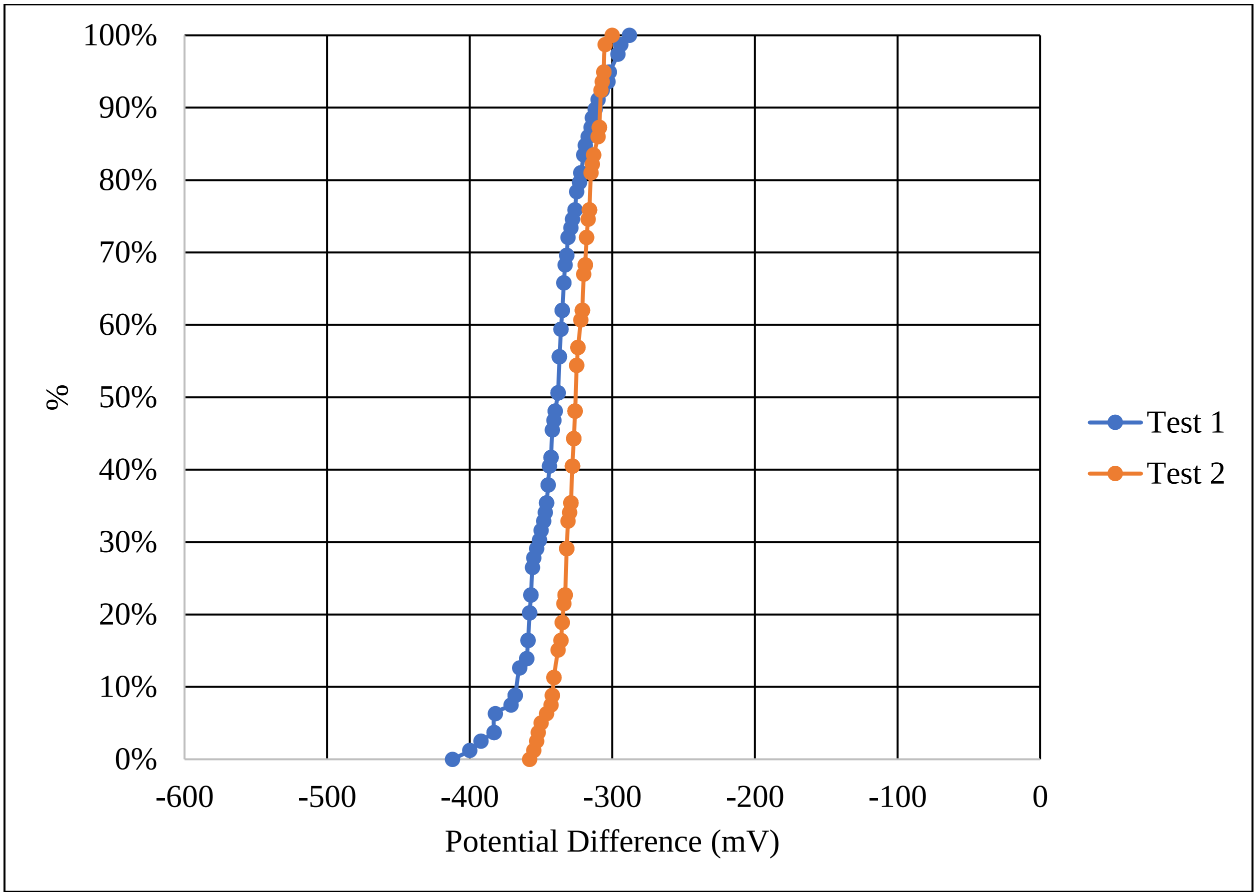


Figure E.117 Shaft 40 percentile distributions

Table E.79 Shaft 41, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-209	-212	-227	-212	-229	-241	-248	-226
3"	-215	-208	-227	-220	-231	-245	-246	-232
6"	-223	-214	-222	-217	-224	-240	-240	-241
9"	-222	-217	-223	-230	-229	-256	-252	-234
12"	-221	-215	-220	-218	-218	-234	-234	-238
15"	-221	-220	-223	-220	-221	-231	-240	-236
18"	-224	-221	-223	-220	-224	-239	-246	-237
21"	-226	-222	-222	-221	-236	-247	-242	-245
24"	-216	-216	-222	-218	-227	-236	-251	-255
27"	-219	-220	-227	-220	-234	-243	-256	-257

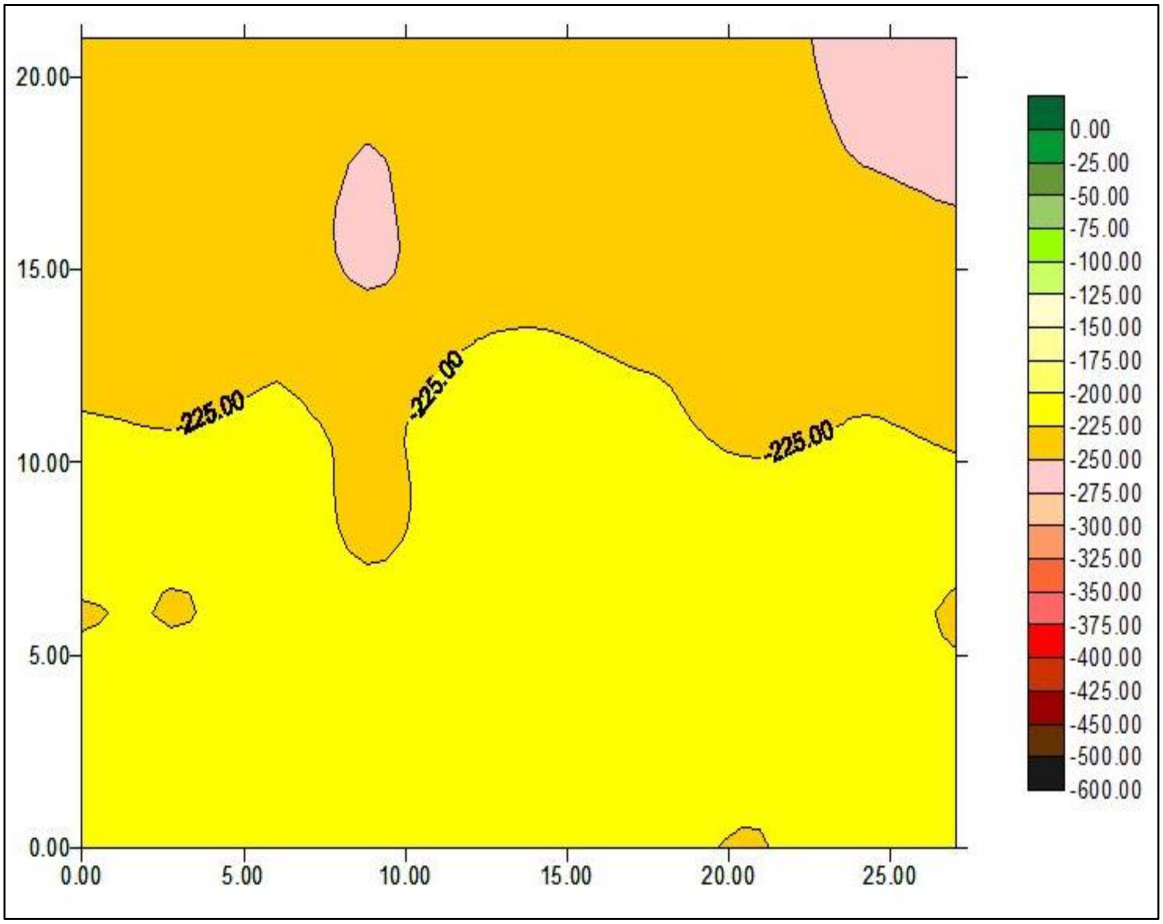


Figure E.118 Shaft 41, test 1 data map

Table E.80 Shaft 41, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-223	-225	-218	-210	-202	-210	-204	-203
3"	-236	-226	-226	-211	-199	-203	-194	-185
6"	-234	-217	-210	-201	-195	-198	-192	-190
9"	-244	-221	-223	-211	-196	-202	-199	-200
12"	-215	-232	-220	-207	-201	-201	-200	-203
15"	-215	-216	-210	-205	-199	-203	-203	-199
18"	-208	-218	-215	-207	-200	-205	-200	-211
21"	-214	-230	-243	-215	-213	-215	-213	-220
24"	-231	-227	-242	-207	-203	-214	-219	-223
27"	-220	-239	-235	-217	-207	-218	-226	-214

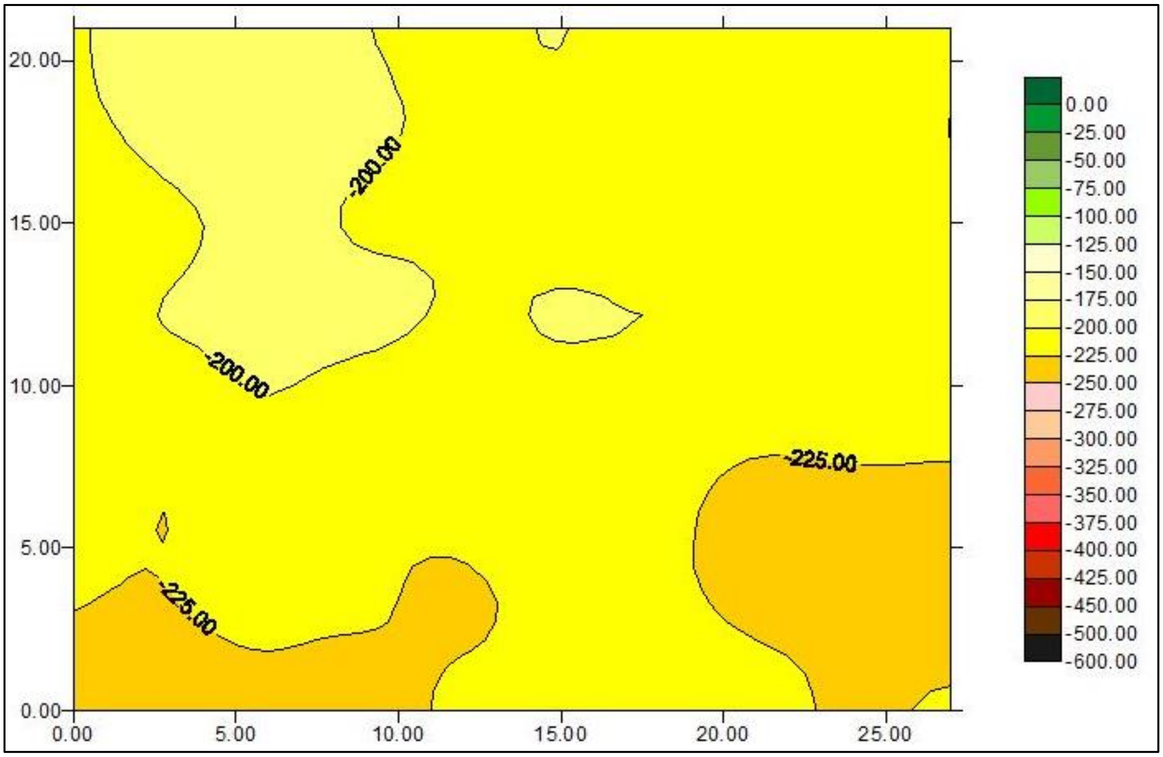


Figure E.119 Shaft 41, test 2 data map

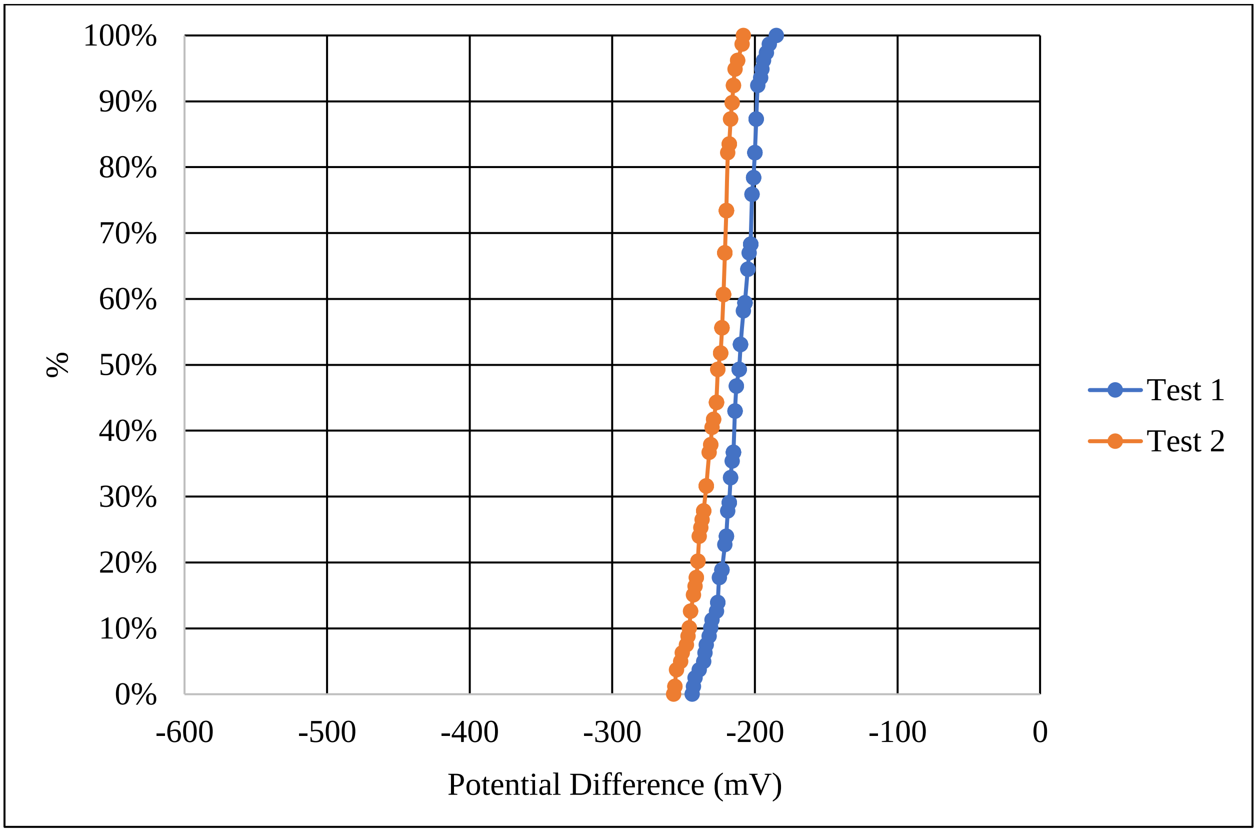


Figure E.120 Shaft 41 percentile distributions

Table E.81 Shaft 42, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-190	-190	-206	-204	-212	-214	-214	-195
3"	-191	-193	-204	-209	-211	-213	-200	-210
6"	-187	-193	-200	-201	-206	-202	-226	-204
9"	-192	-194	-199	-197	-205	-203	-219	-218
12"	-191	-203	-205	-218	-216	-210	-225	-215
15"	-192	-191	-197	-195	-204	-202	-212	-222
18"	-181	-189	-196	-200	-201	-200	-214	-223
21"	-183	-191	-194	-196	-202	-204	-226	-219
24"	-180	-186	-190	-192	-210	-211	-221	-208
27"	-179	-189	-192	-195	-201	-203	-212	-208

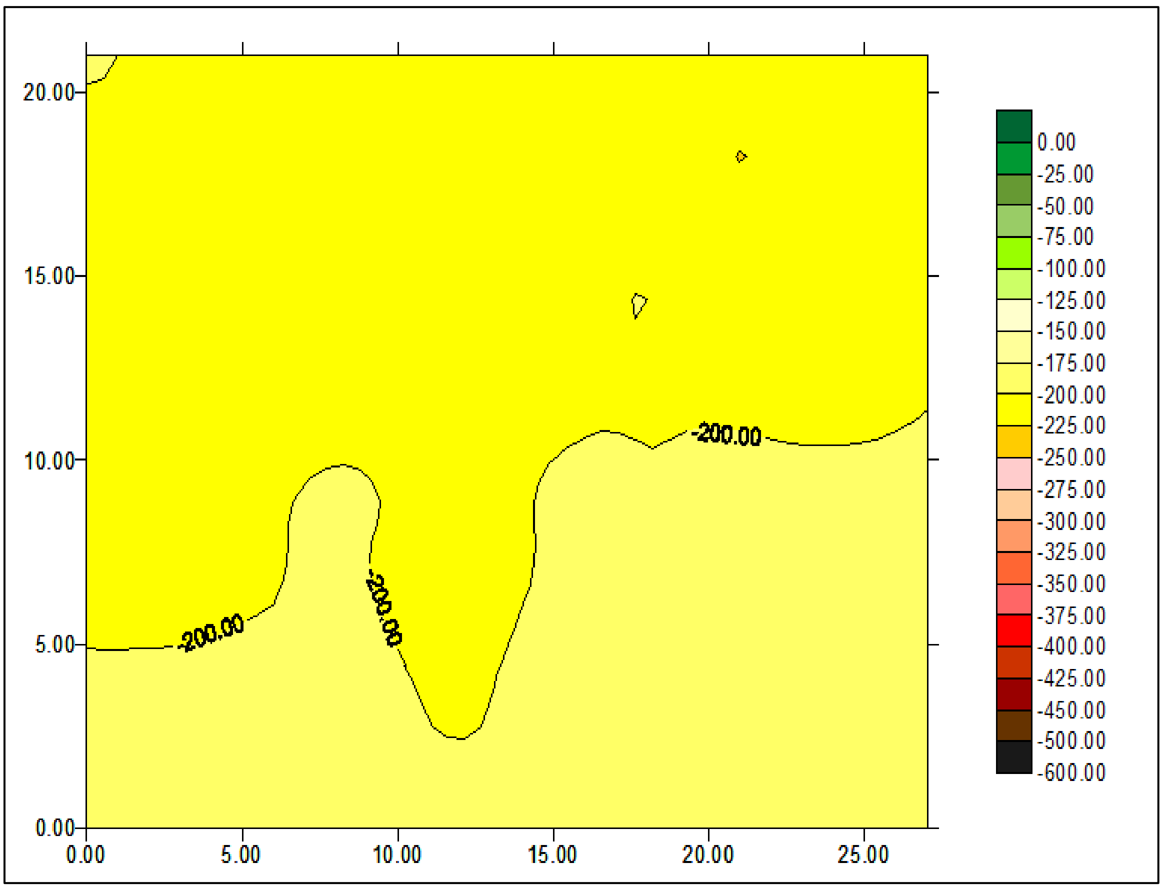


Figure E.121 Shaft 42, test 1 data map

Table E.82 Shaft 42, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-179	-186	-176	-182	-175	-178	-180	-166
3"	-199	-194	-183	-182	-175	-180	-183	-170
6"	-191	-207	-196	-199	-193	-189	-195	-187
9"	-205	-203	-187	-189	-184	-187	-198	-179
12"	-200	-198	-183	-190	-188	-193	-192	-187
15"	-203	-206	-191	-199	-193	-199	-200	-199
18"	-195	-213	-205	-209	-205	-197	-204	-205
21"	-201	-209	-185	-191	-198	-196	-195	-200
24"	-190	-215	-193	-193	-196	-210	-194	-187
27"	-199	-188	-209	-204	-210	-207	-208	-216

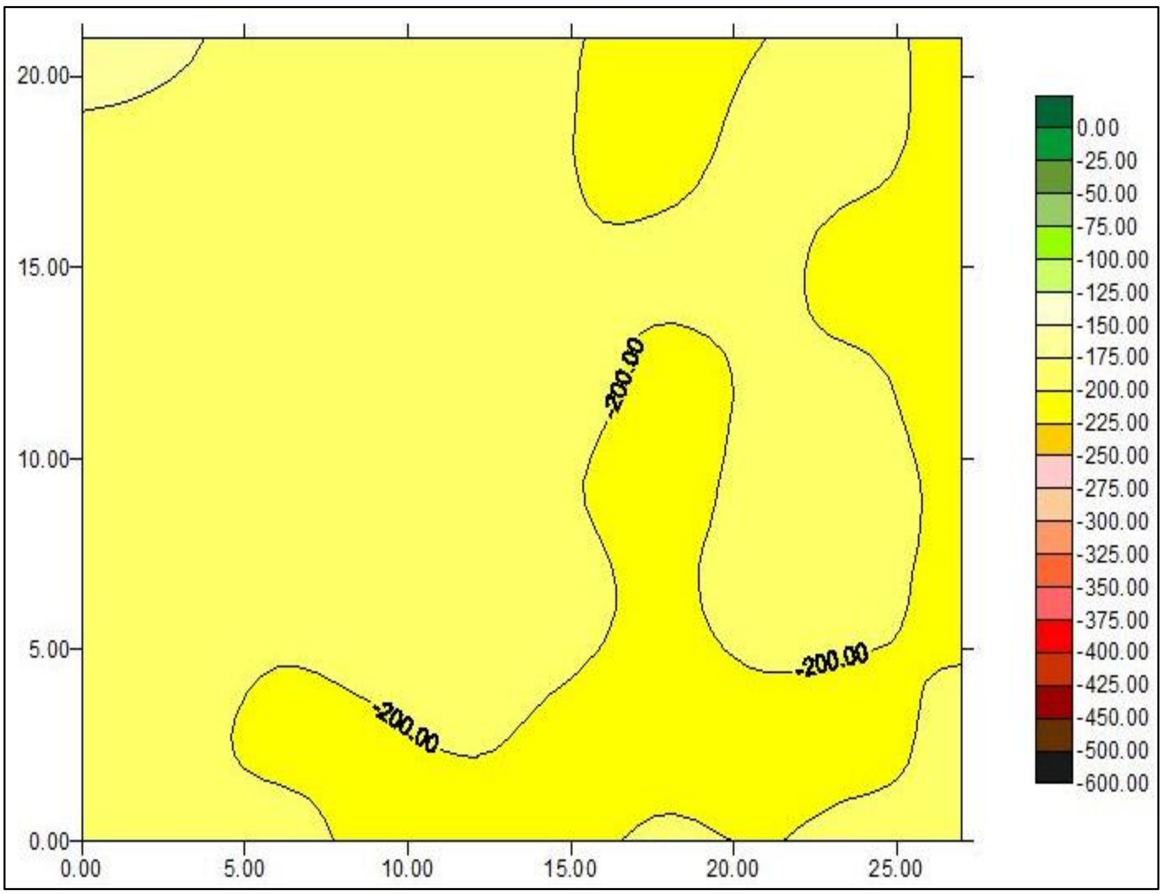


Figure E.122 Shaft 42, test 2 data map

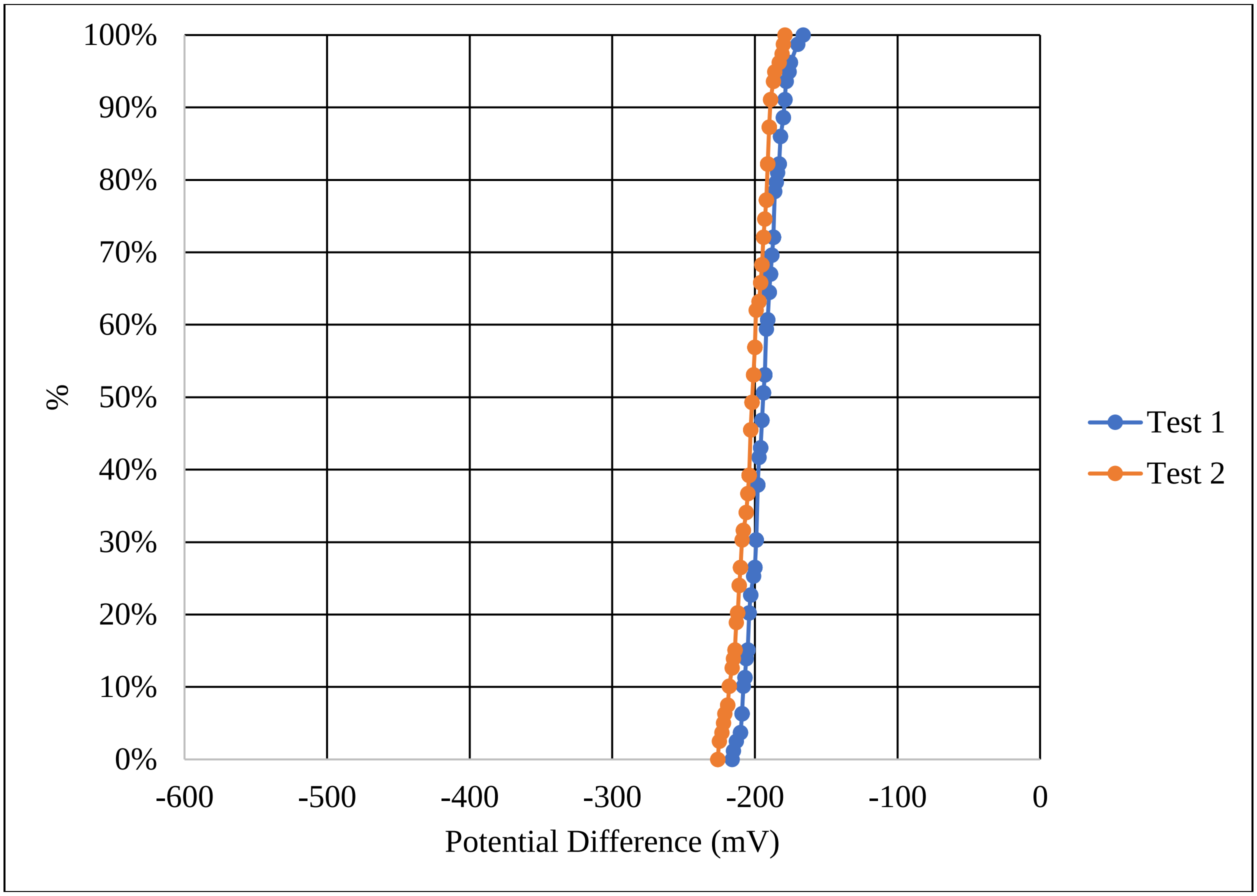


Figure E.123 Shaft 42 percentile distributions

Table E.83 Shaft 43, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-443	-423	-395	-396	-366	-358	-380	-376
3"	-483	-417	-400	-391	-379	-379	-381	-391
6"	-424	-413	-415	-385	-380	-382	-383	-389
9"	-433	-435	-419	-413	-404	-398	-410	-403
12"	-434	-451	-426	-424	-405	-392	-396	-397
15"	-451	-446	-441	-415	-402	-392	-394	-371
18"	-460	-457	-443	-410	-397	-393	-397	-391
21"	-466	-443	-415	-398	-404	-377	-389	-384
24"	-444	-433	-408	-390	-385	-381	-380	-389
27"	-434	-409	-389	-377	-358	-352	-364	-377

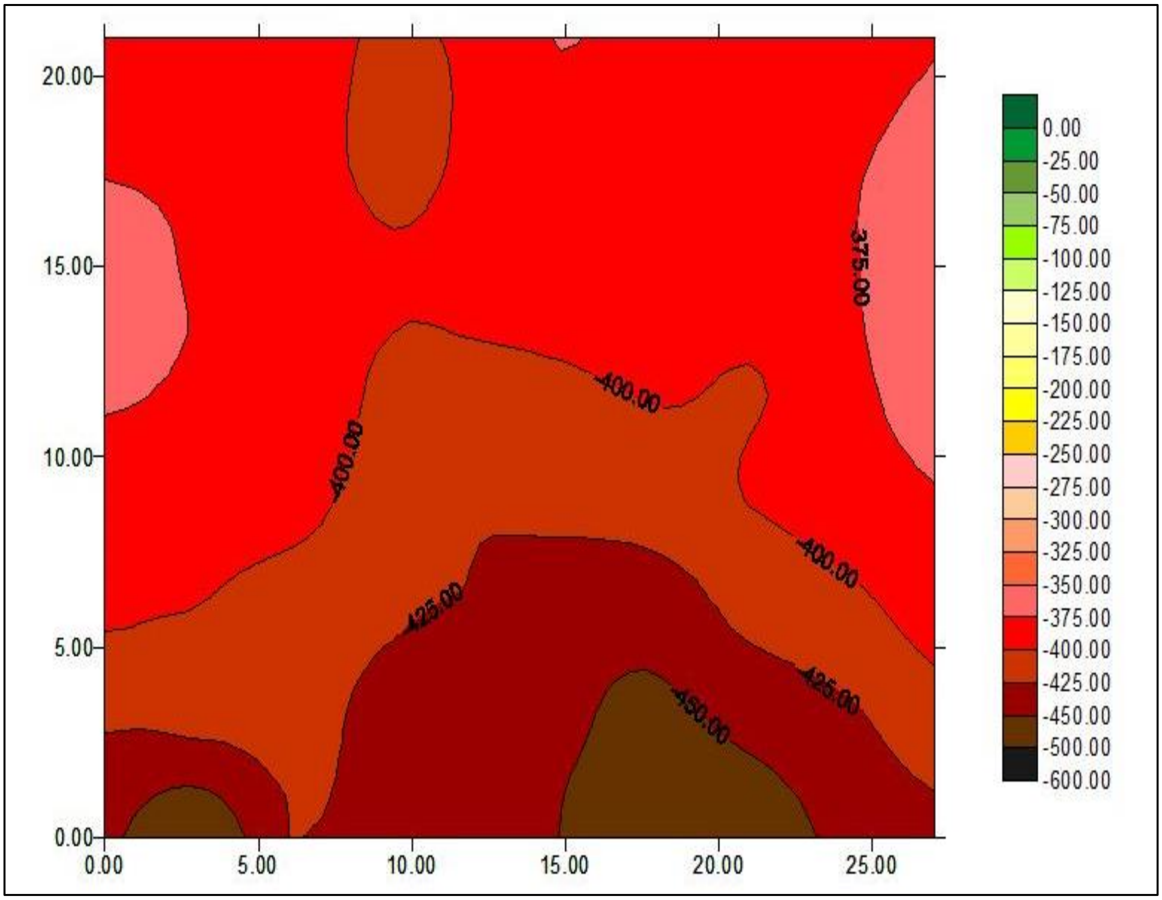


Figure E.124 Shaft 43, test 1 data map

Table E.84 Shaft 43, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-518	-528	-537	-530	-562	-585	-611	-626
3"	-504	-516	-522	-519	-538	-562	-589	-612
6"	-505	-509	-505	-516	-532	-545	-578	-598
9"	-503	-500	-509	-519	-523	-533	-561	-580
12"	-500	-496	-494	-500	-508	-529	-538	-557
15"	-505	-505	-503	-512	-515	-531	-532	-546
18"	-506	-499	-494	-511	-521	-519	-514	-524
21"	-515	-517	-501	-520	-522	-527	-526	-531
24"	-510	-514	-504	-514	-531	-534	-535	-536
27"	-521	-516	-511	-520	-533	-537	-538	-541

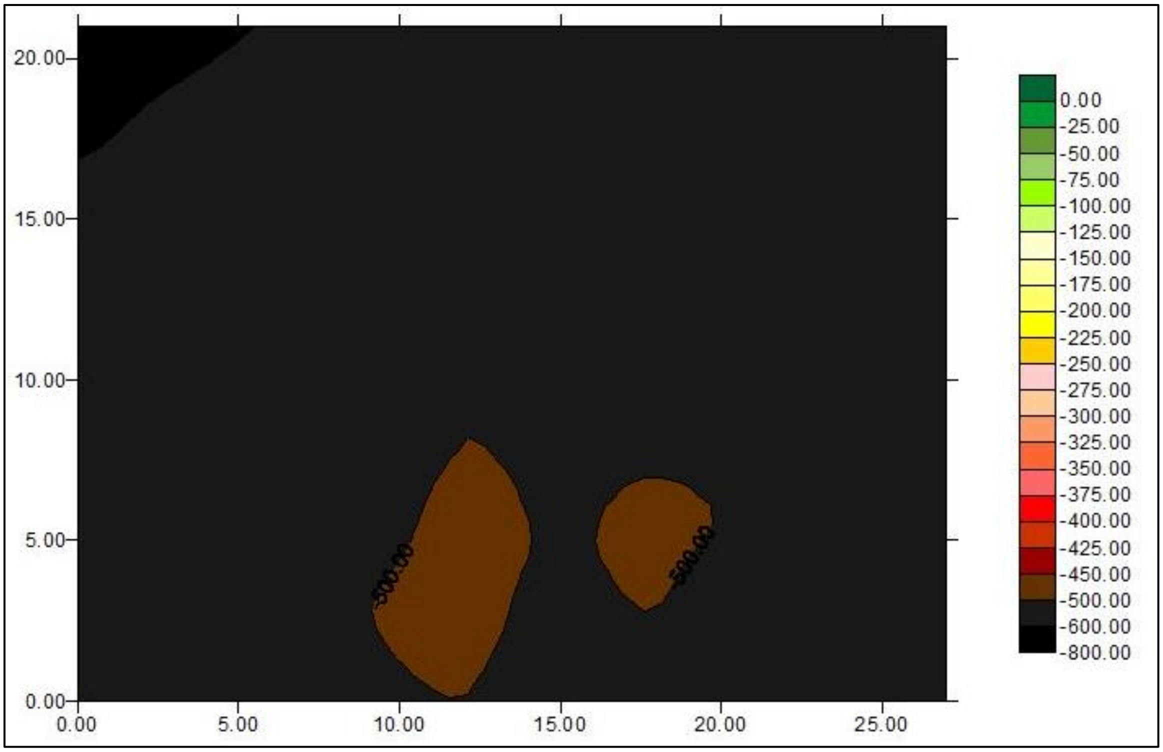


Figure E.125 Shaft 43, test 2 data map

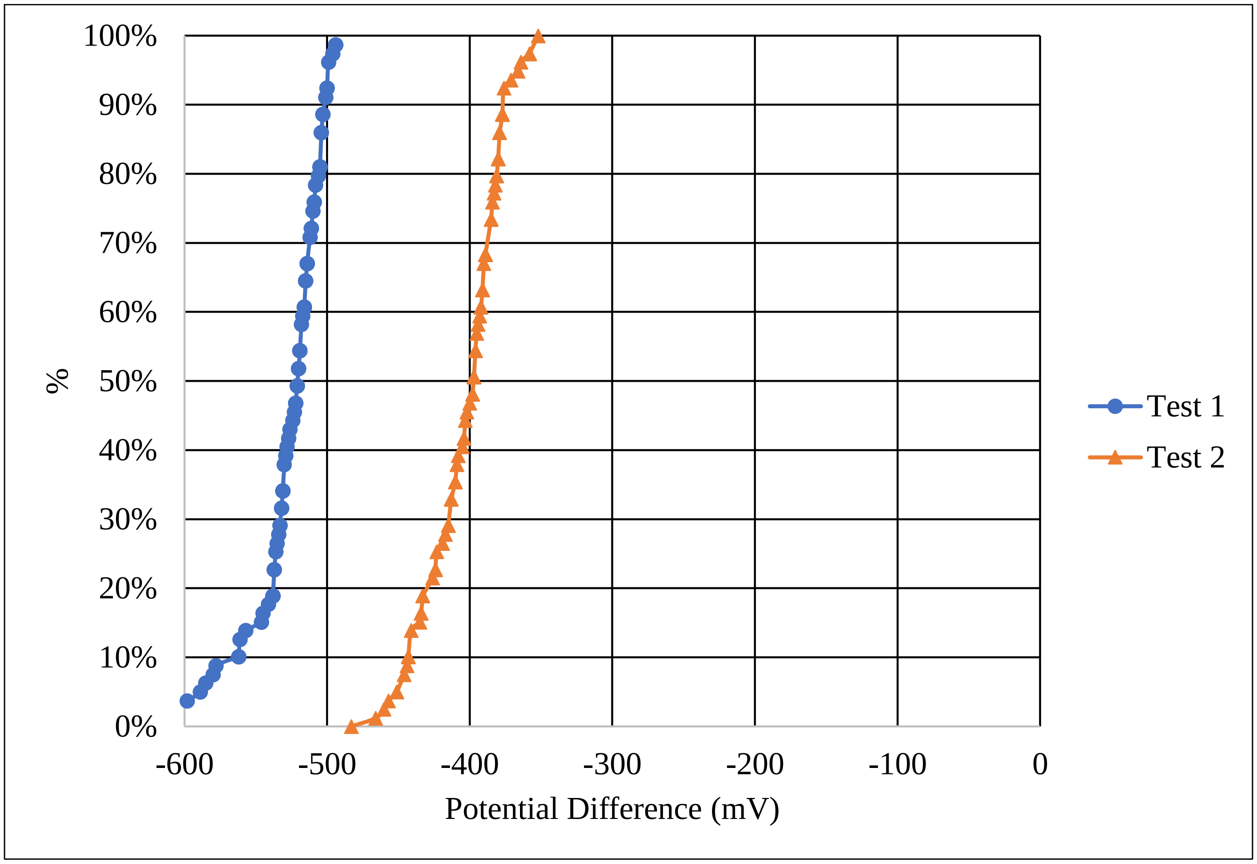


Figure E.126 Shaft 43 percentile distributions

Table E.85 Shaft 45, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-625	-562	-575	-498	-415	-432	-472	-483
3"	-636	-584	-570	-486	-406	-427	-429	-445
6"	-622	-585	-545	-455	-418	-381	-388	-417
9"	-620	-598	-550	-471	-401	-363	-381	-388
12"	-599	-568	-453	-457	-399	-363	-378	-390
15"	-585	-567	-532	-440	-385	-352	-329	-366
18"	-569	-559	-523	-420	-377	-332	-326	-342
21"	-551	-555	-506	-505	-419	-379	-328	-341
24"	-548	-562	-512	-430	-387	-330	-327	-328
27"	-561	-563	-530	-439	-381	-343	-341	-338

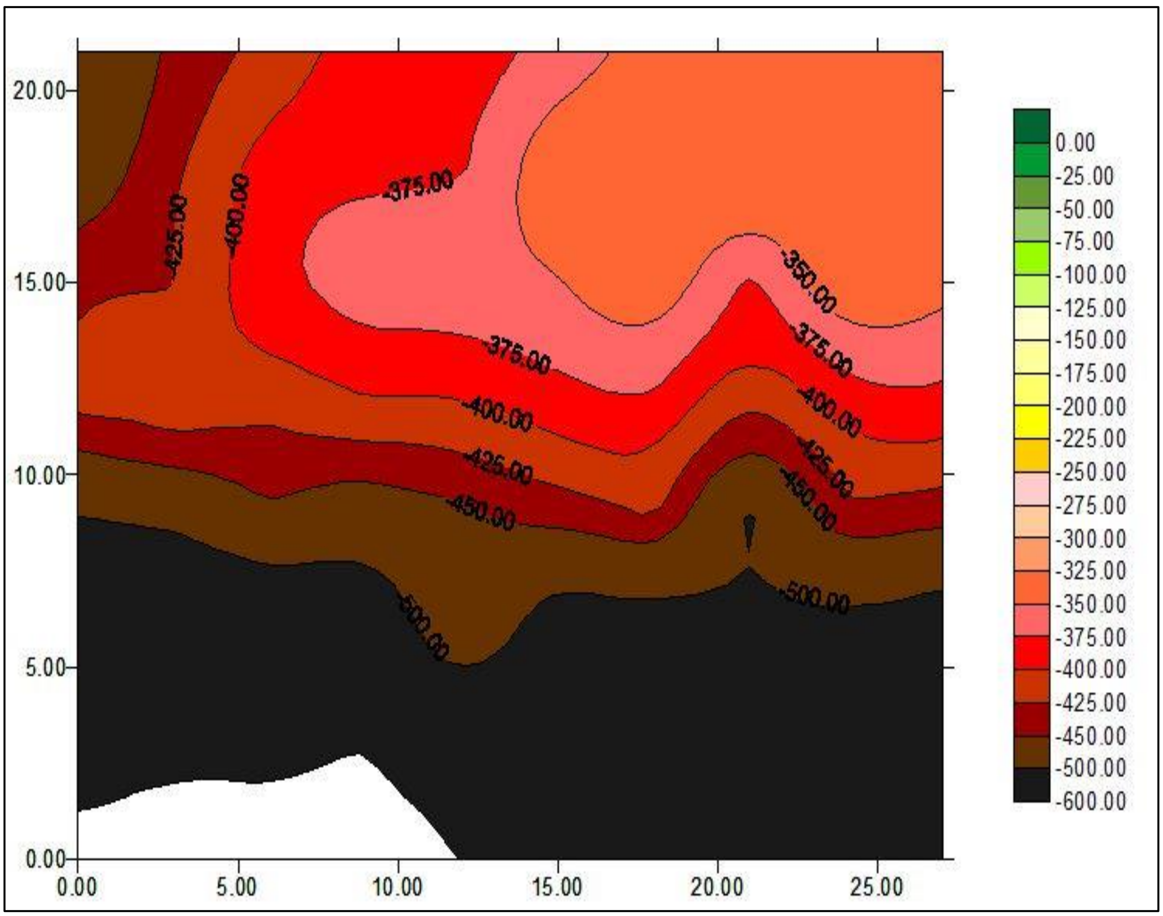


Figure E.127 Shaft 45, test 1 data map

Table E.86 Shaft 45, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-377	-379	-395	-420	-504	-576	-616	-617
3"	-417	-400	-412	-448	-517	-599	-633	-641
6"	-411	-395	-403	-440	-490	-585	-623	-639
9"	-409	-400	-411	-451	-519	-597	-631	-646
12"	-427	-399	-432	-484	-528	-615	-647	-644
15"	-451	-417	-428	-471	-576	-599	-630	-645
18"	-477	-443	-459	-518	-586	-625	-650	-659
21"	-533	-477	-476	-525	-586	-623	-640	-656
24"	-560	-519	-531	-577	-612	-648	-649	-650
27"	-412	-542	-544	-566	-609	-651	-648	-641

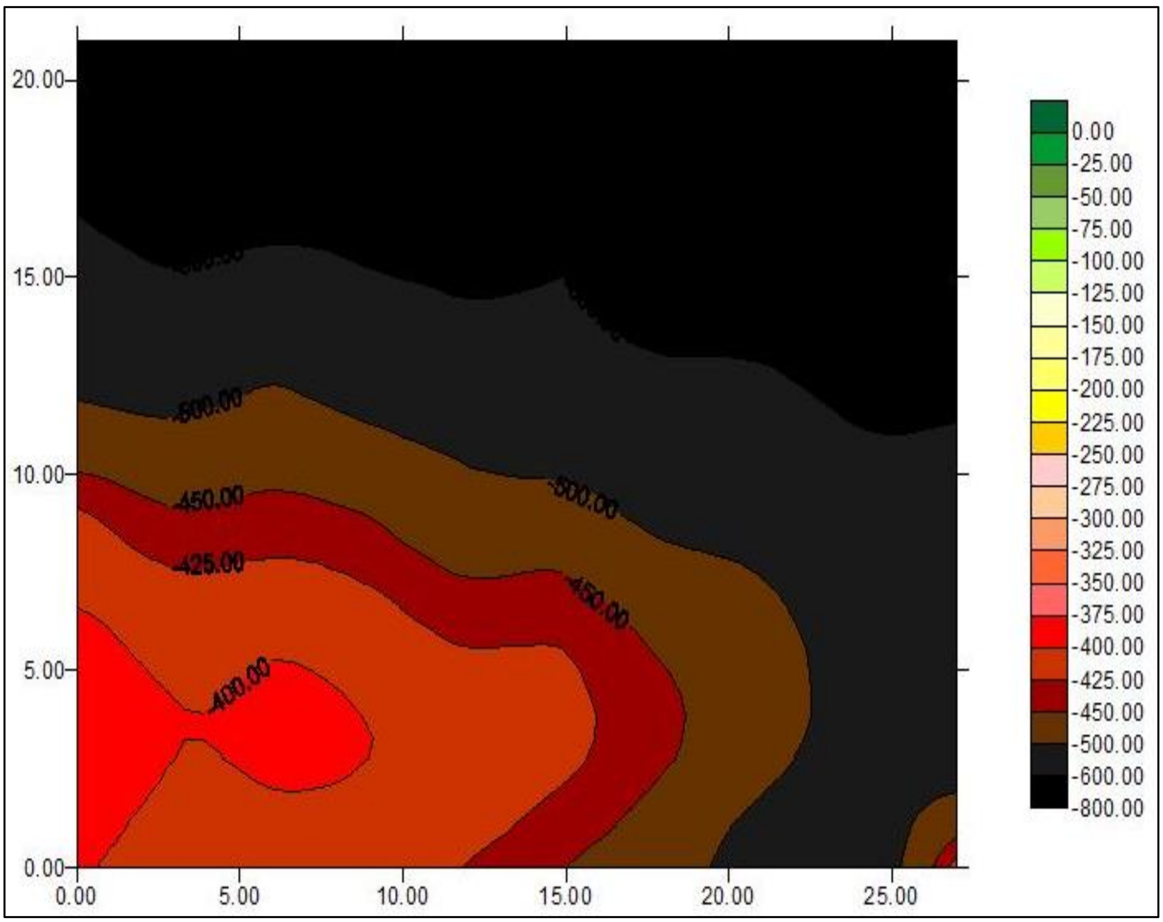


Figure E.128 Shaft 45 test 2 data map

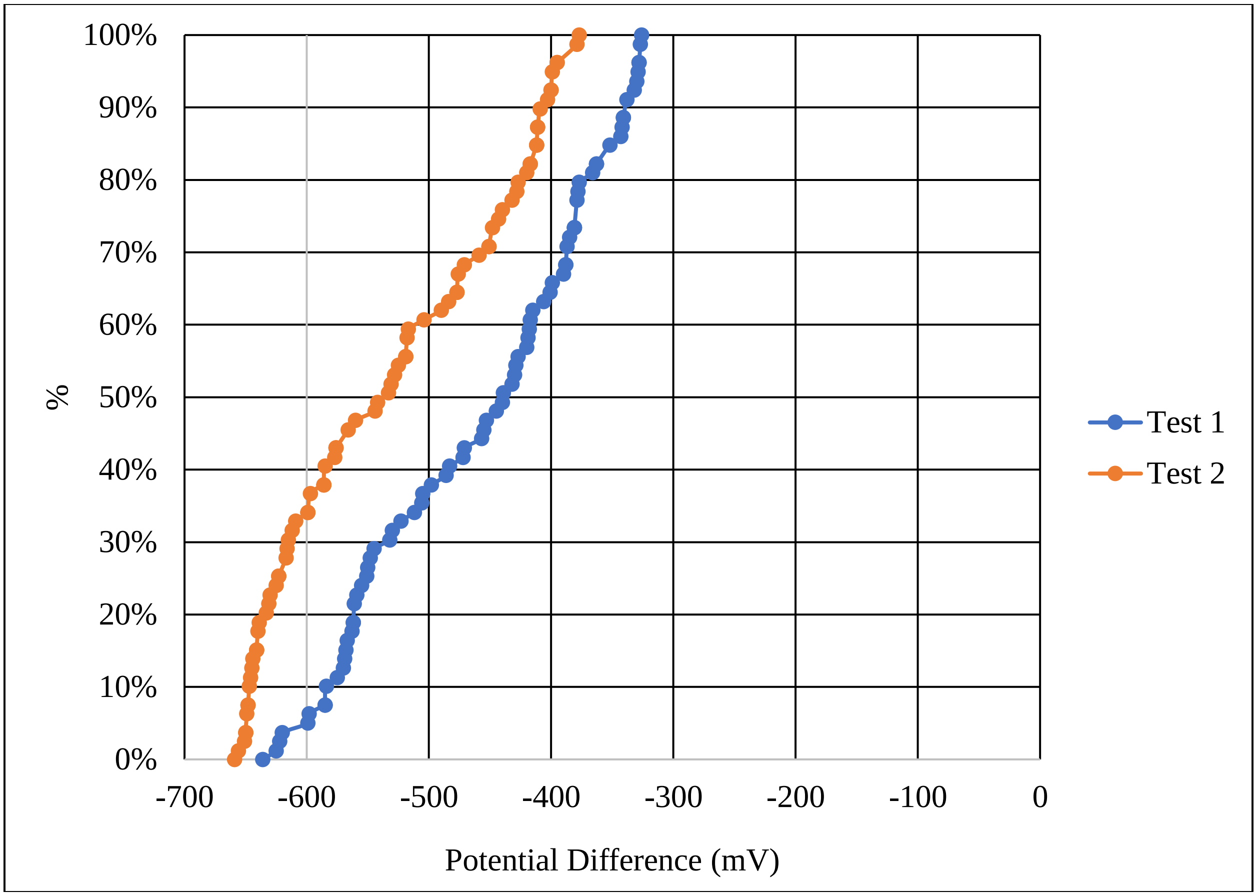


Figure E.129 Shaft 45 percentile distributions

Table E.87 Shaft 46, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-237	-230	-226	-223	-222	-222	-216	-214
3"	-232	-235	-224	-218	-221	-219	-227	-215
6"	-235	-232	-222	-224	-221	-226	-217	-213
9"	-238	-232	-226	-224	-223	-224	-223	-218
12"	-242	-235	-229	-225	-222	-225	-226	-222
15"	-242	-234	-225	-220	-220	-221	-223	-224
18"	-242	-236	-229	-222	-222	-226	-226	-237
21"	-245	-237	-231	-225	-222	-224	-229	-235
24"	-255	-249	-235	-225	-226	-235	-229	-243
27"	-258	-248	-237	-222	-222	-222	-228	-239

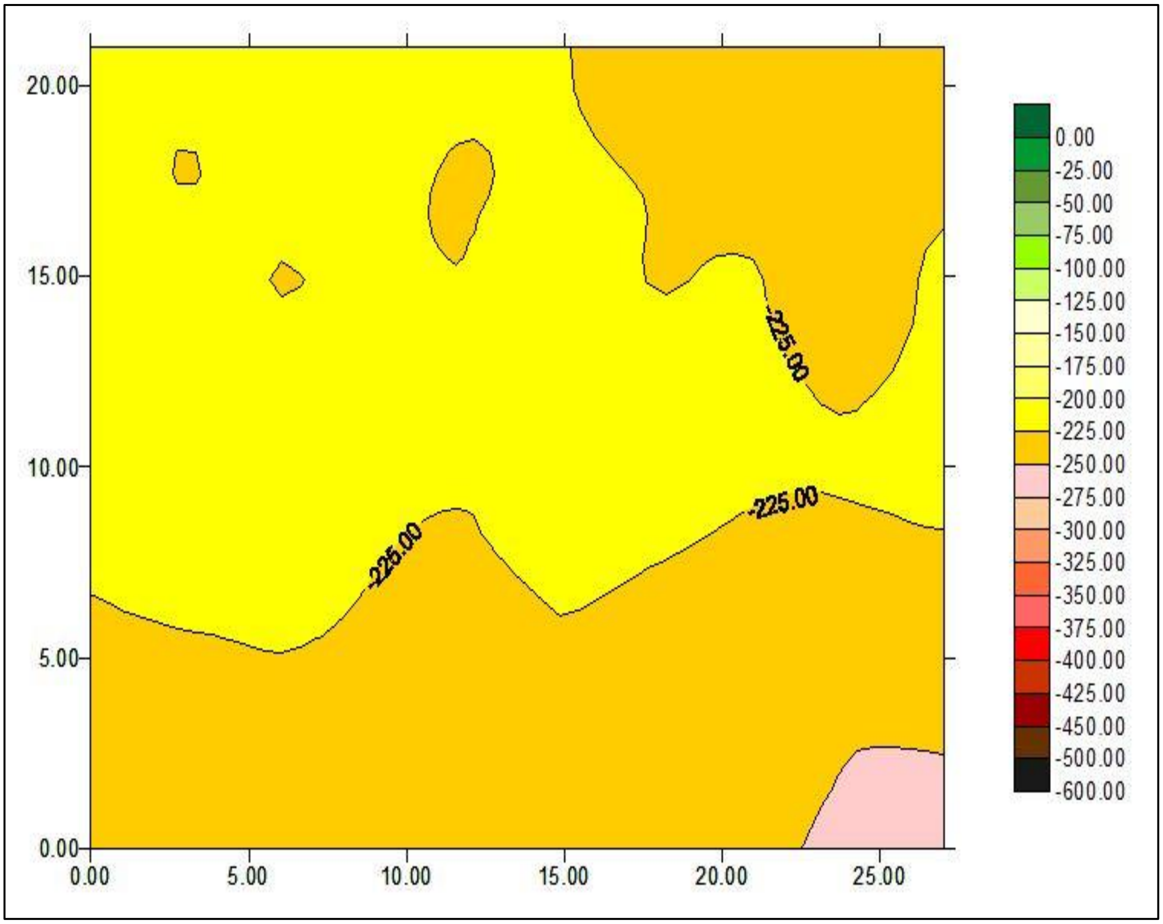


Figure E.130 Shaft 46, test 1 data map

Table E.88 Shaft 46, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-295	-277	-245	-238	-239	-239	-248	-268
3"	-299	-273	-243	-241	-241	-241	-262	-274
6"	-293	-275	-253	-249	-249	-249	-256	-275
9"	-287	-269	-248	-243	-243	-237	-261	-284
12"	-293	-274	-253	-242	-242	-241	-263	-278
15"	-303	-284	-259	-248	-248	-242	-262	-297
18"	-318	-296	-270	-254	-254	-247	-267	-307
21"	-326	-297	-263	-262	-262	-256	-274	-304
24"	-343	-304	-275	-266	-266	-257	-281	-306
27"	-362	-337	-290	-278	-278	-262	-284	-315

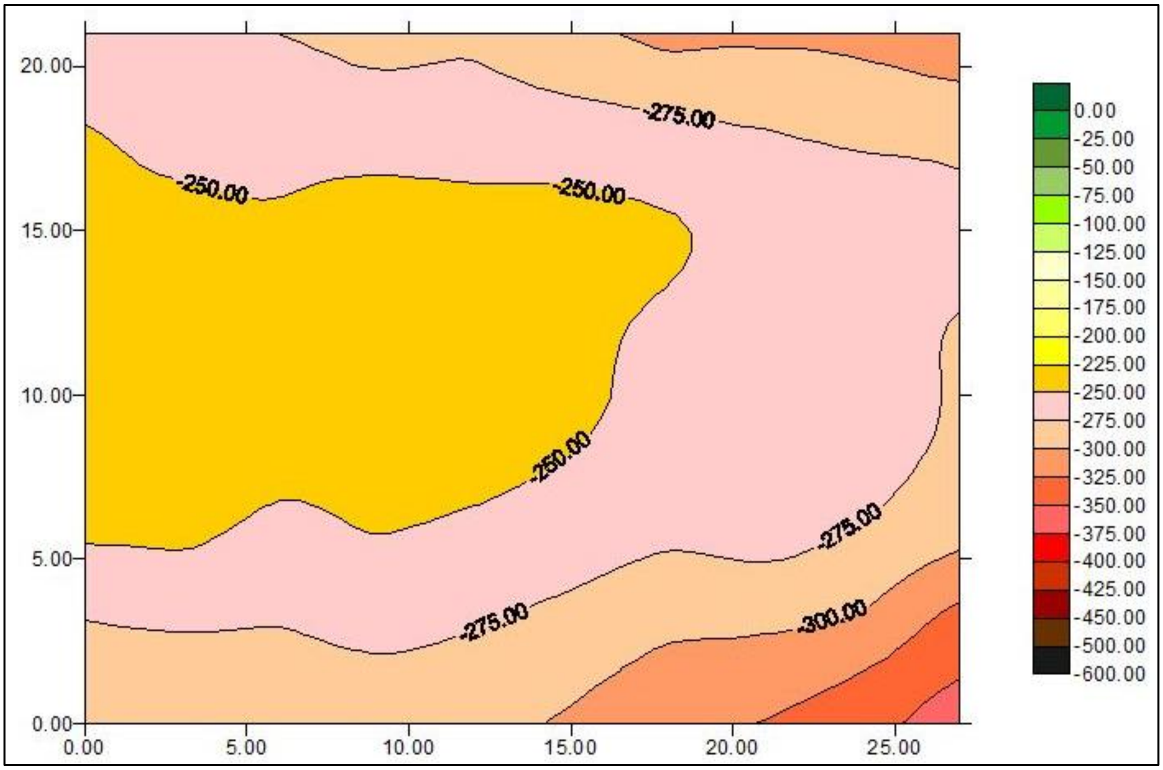


Figure E.131 Shaft 46 test 2 data map

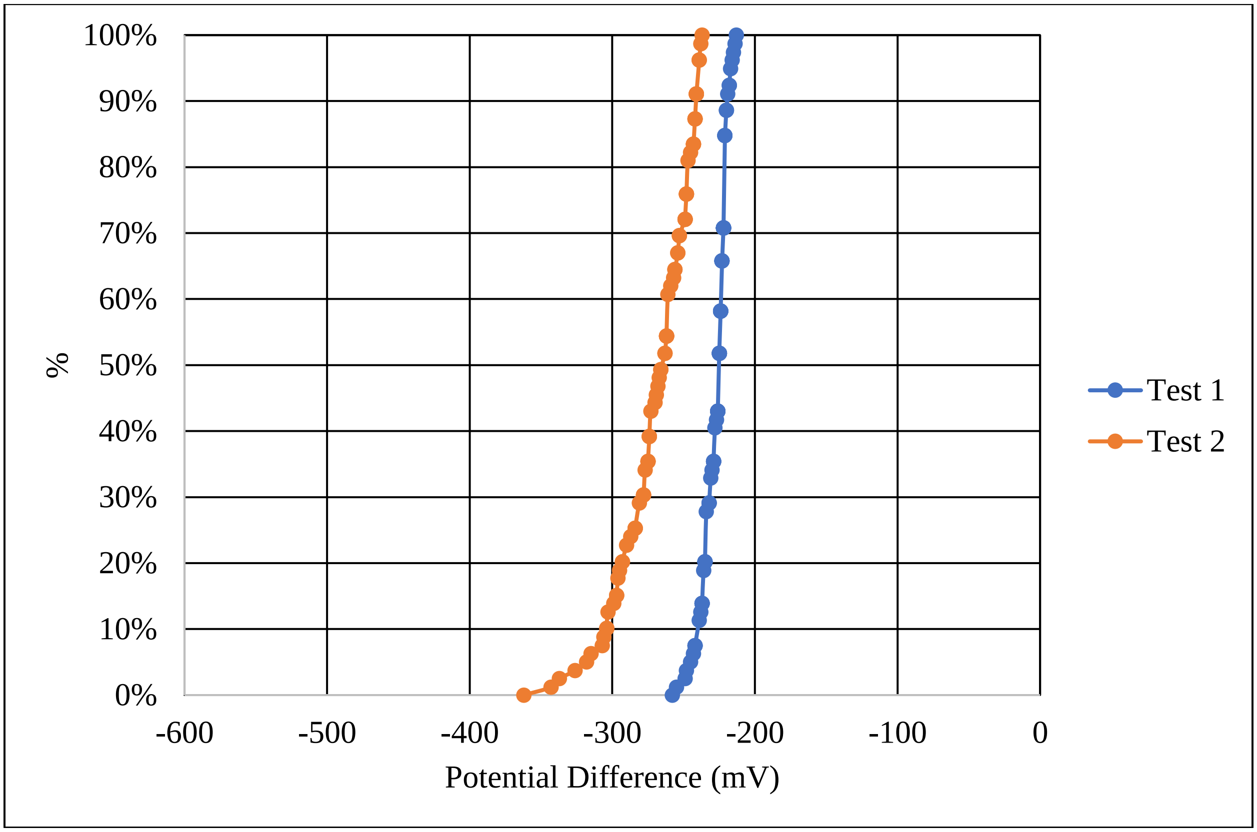


Figure E.132 Shaft 46 percentile distributions

Table E.89 Shaft 47, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-287	-279	-265	-256	-252	-244	-251	-292
3"	-279	-273	-261	-256	-255	-244	-256	-294
6"	-280	-272	-263	-258	-250	-244	-257	-293
9"	-283	-273	-264	-256	-250	-245	-257	-296
12"	-289	-278	-267	-255	-251	-244	-259	-291
15"	-295	-281	-267	-258	-253	-247	-265	-286
18"	-296	-279	-266	-258	-253	-252	-273	-296
21"	-294	-280	-268	-258	-253	-255	-273	-303
24"	-293	-282	-267	-261	-253	-256	-272	-313
27"	-290	-282	-270	-262	-260	-260	-281	-313

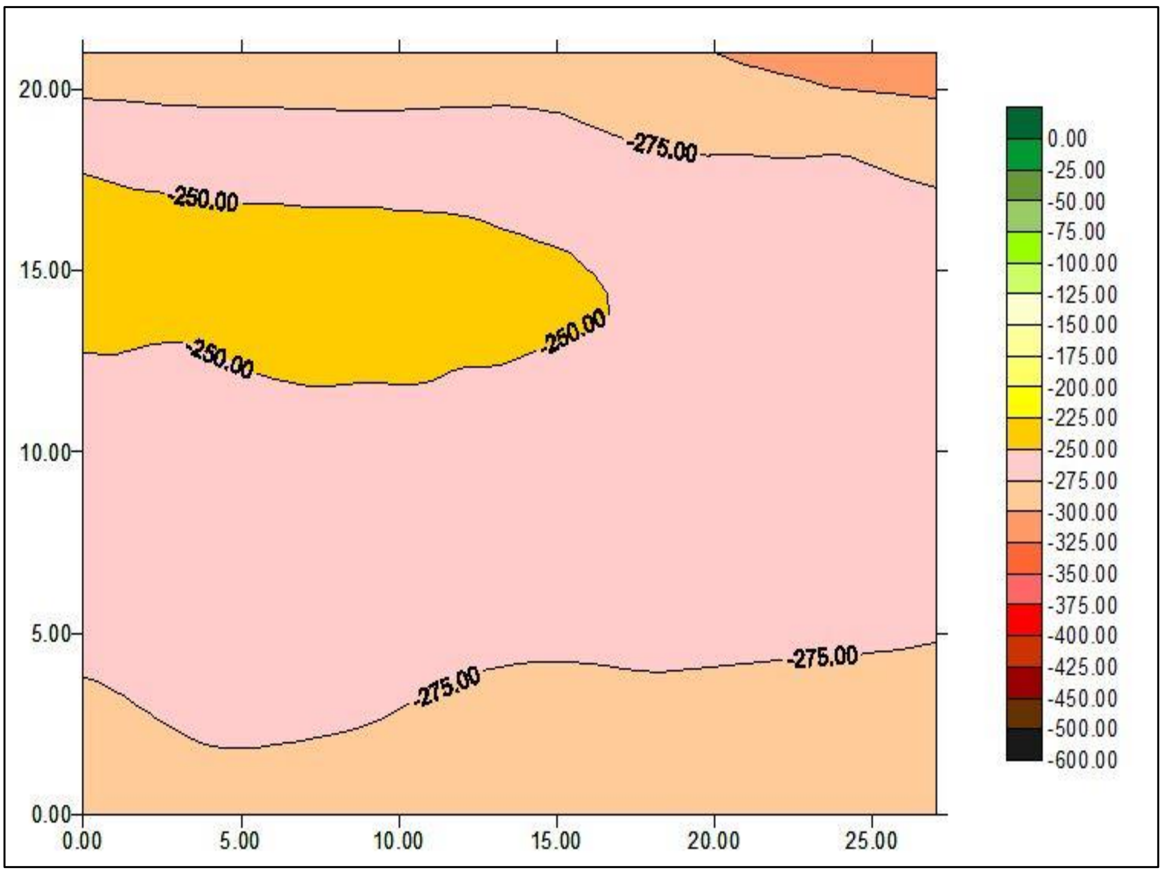


Figure E.133 Shaft 47, test 1 data map

Table E.90 Shaft 47, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-319	-309	-301	-277	-274	-274	-277	-290
3"	-333	-313	-306	-281	-275	-274	-283	-297
6"	-336	-320	-309	-279	-274	-271	-283	-299
9"	-349	-336	-314	-281	-274	-274	-283	-300
12"	-351	-343	-317	-285	-275	-274	-285	-300
15"	-343	-332	-314	-284	-275	-273	-286	-300
18"	-338	-320	-306	-280	-275	-274	-283	-296
21"	-335	-314	-302	-282	-277	-275	-282	-290
24"	-339	-311	-300	-280	-277	-274	-281	-290
27"	-341	-315	-300	-279	-273	-276	-282	-290

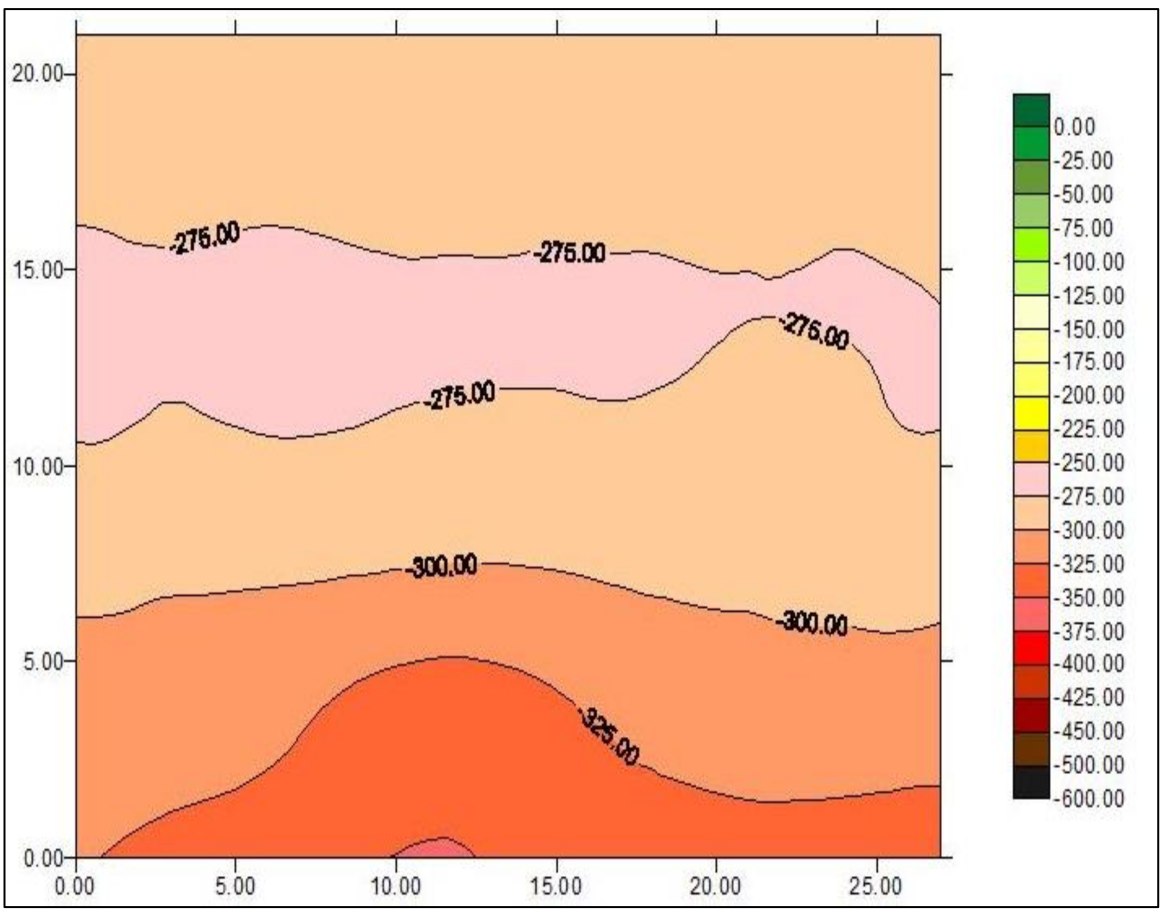


Figure E.134 Shaft 47 test 2 data map

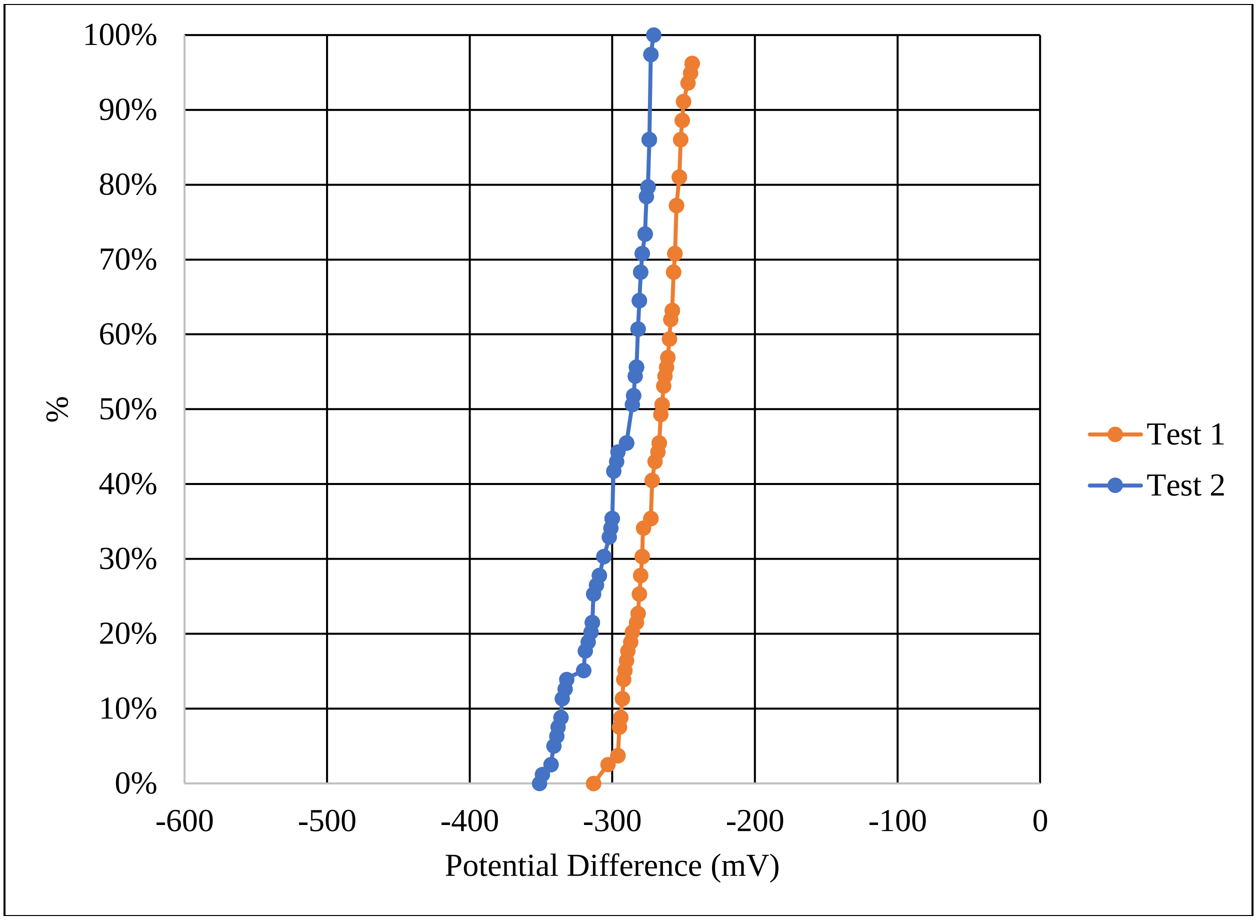


Figure E.135 Shaft 47 percentile distributions

Table E.91 Shaft 48, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-280	-271	-262	-259	-256	-258	-262	-252
3"	-286	-272	-260	-256	-253	-256	-261	-264
6"	-286	-272	-257	-247	-249	-253	-262	-275
9"	-289	-270	-256	-249	-246	-250	-262	-283
12"	-290	-275	-257	-251	-248	-249	-264	-286
15"	-287	-272	-256	-250	-246	-250	-267	-293
18"	-286	-271	-255	-248	-245	-248	-259	-288
21"	-281	-267	-254	-247	-244	-246	-255	-280
24"	-277	-264	-254	-246	-243	-246	-252	-275
27"	-273	-262	-253	-246	-243	-241	-245	-274

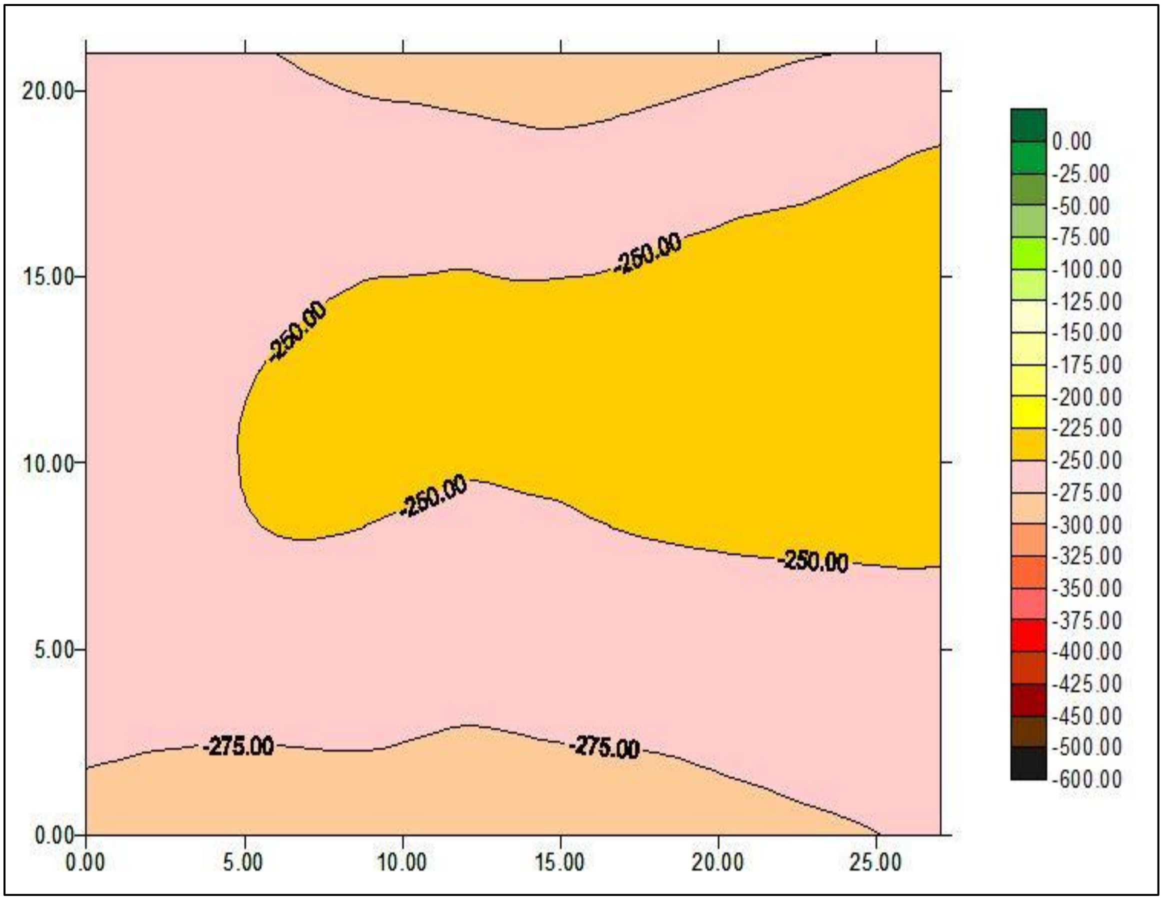


Figure E.136 Shaft 48, test 1 data map

Table E.92 Shaft 48, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-258	-251	-249	-253	-258	-266	-280	-294
3"	-278	-266	-259	-260	-267	-275	-291	-308
6"	-286	-273	-264	-265	-268	-280	-298	-321
9"	-294	-280	-269	-268	-272	-281	-300	-319
12"	-297	-277	-270	-270	-275	-285	-304	-325
15"	-297	-279	-272	-271	-275	-285	-304	-324
18"	-293	-280	-274	-274	-280	-287	-302	-321
21"	-293	-282	-277	-278	-283	-290	-302	-318
24"	-295	-284	-279	-280	-284	-292	-304	-317
27"	-298	-287	-281	-281	-286	-294	-307	-320

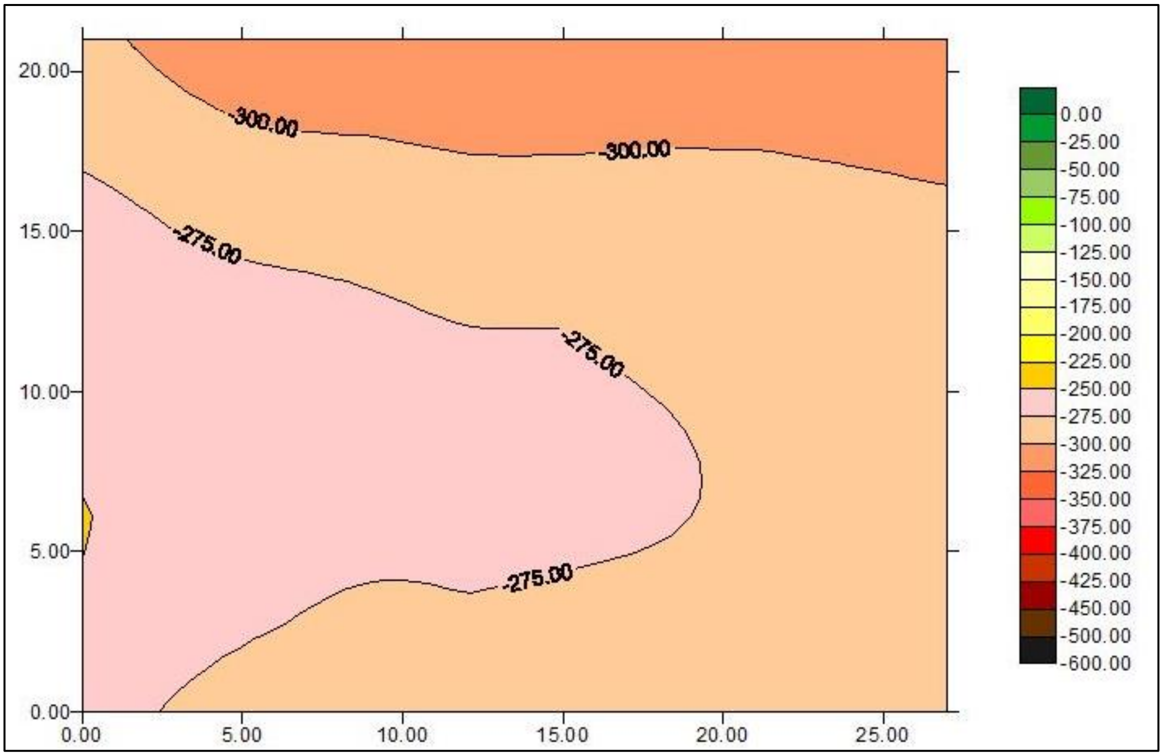


Figure E.137 Shaft 48 test 2 data map

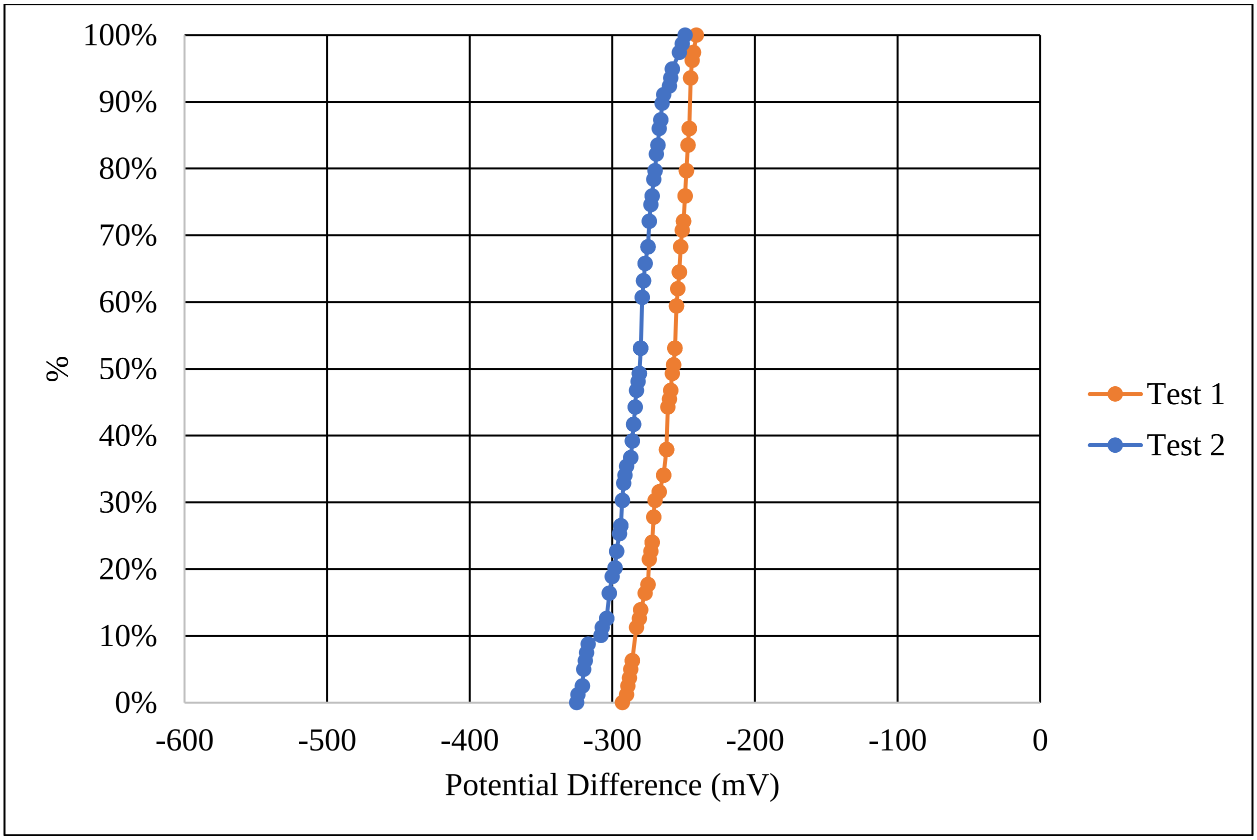


Figure E.138 Shaft 48 percentile distributions

Table E.93 Shaft 49, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-267	-261	-257	-255	-252	-252	-259	-266
3"	-279	-269	-261	-260	-255	-258	-265	-274
6"	-284	-274	-264	-260	-255	-262	-270	-281
9"	-292	-275	-264	-260	-254	-261	-272	-289
12"	-302	-282	-269	-260	-258	-262	-274	-297
15"	-306	-288	-271	-262	-259	-264	-275	-297
18"	-311	-293	-276	-275	-263	-265	-275	-294
21"	-314	-295	-279	-272	-266	-268	-276	-293
24"	-311	-296	-284	-275	-271	-272	-275	-288
27"	-305	-298	-289	-280	-277	-276	-277	-285

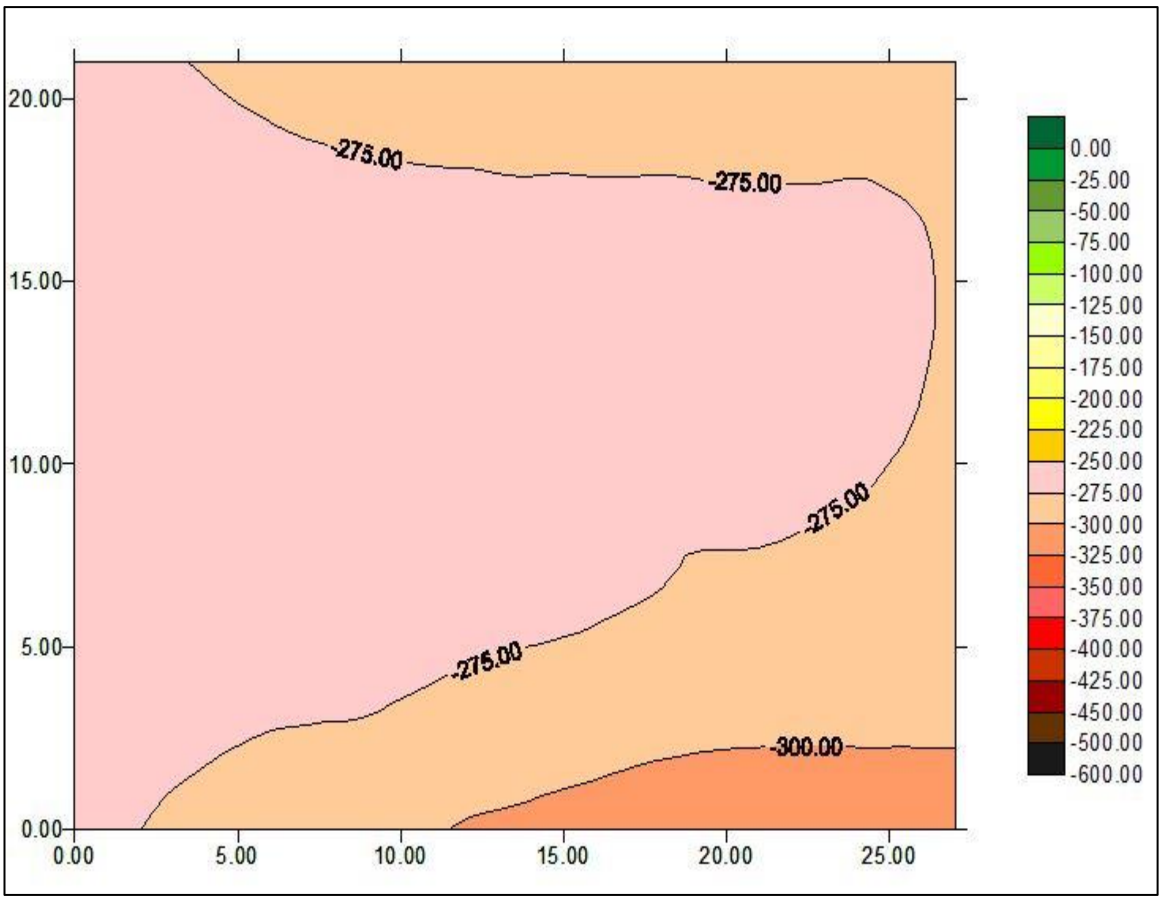


Figure E.139 Shaft 49, test 1 data map

Table E.94 Shaft 49, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-283	-274	-268	-266	-274	-283	-291	-310
3"	-292	-277	-265	-264	-271	-283	-299	-315
6"	-297	-279	-268	-263	-267	-278	-297	-320
9"	-305	-283	-269	-262	-264	-275	-295	-317
12"	-315	-287	-271	-262	-263	-272	-291	-308
15"	-319	-287	-271	-262	-262	-263	-286	-305
18"	-310	-285	-269	-261	-263	-268	-279	-295
21"	-300	-281	-267	-259	-260	-264	-275	-288
24"	-285	-275	-262	-257	-258	-263	-274	-286
27"	-277	-266	-256	-255	-255	-261	-268	-281

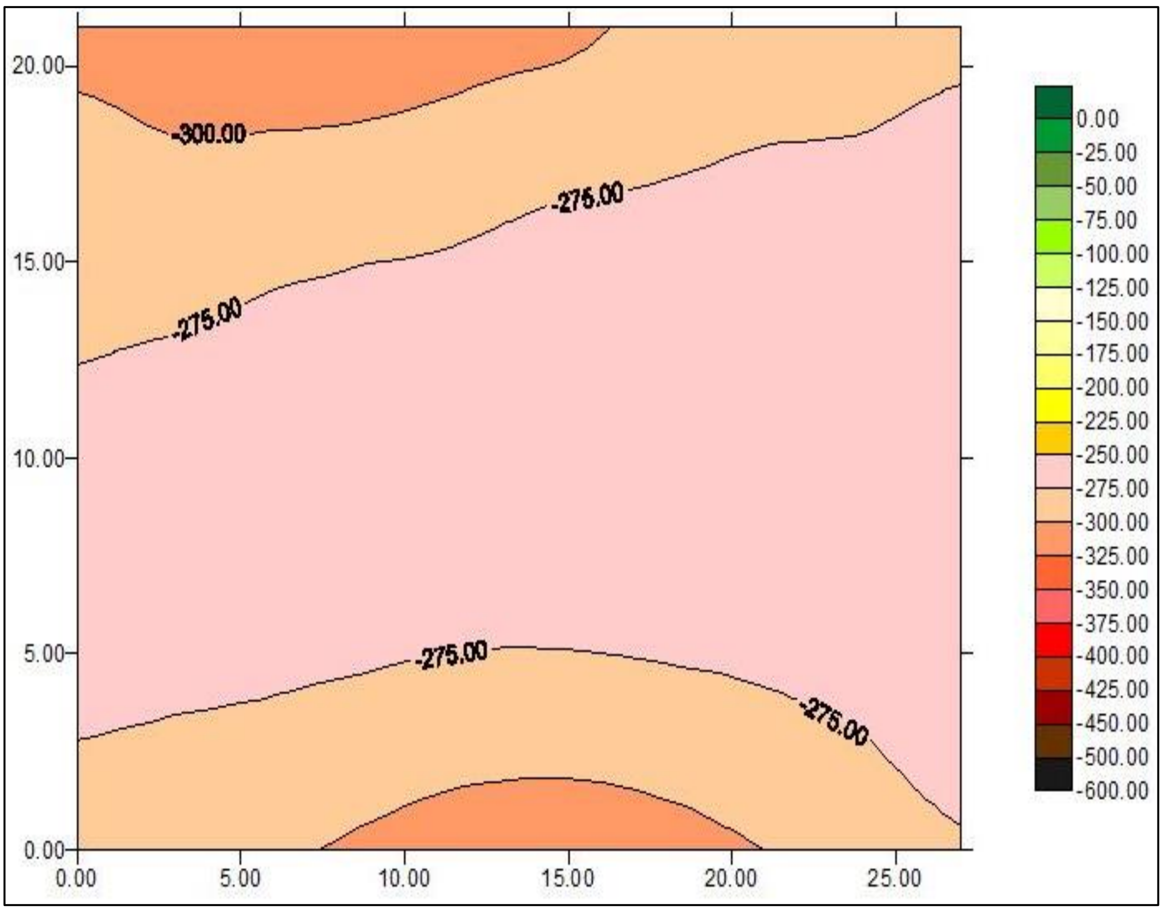


Figure E.140 Shaft 49 test 2 data map

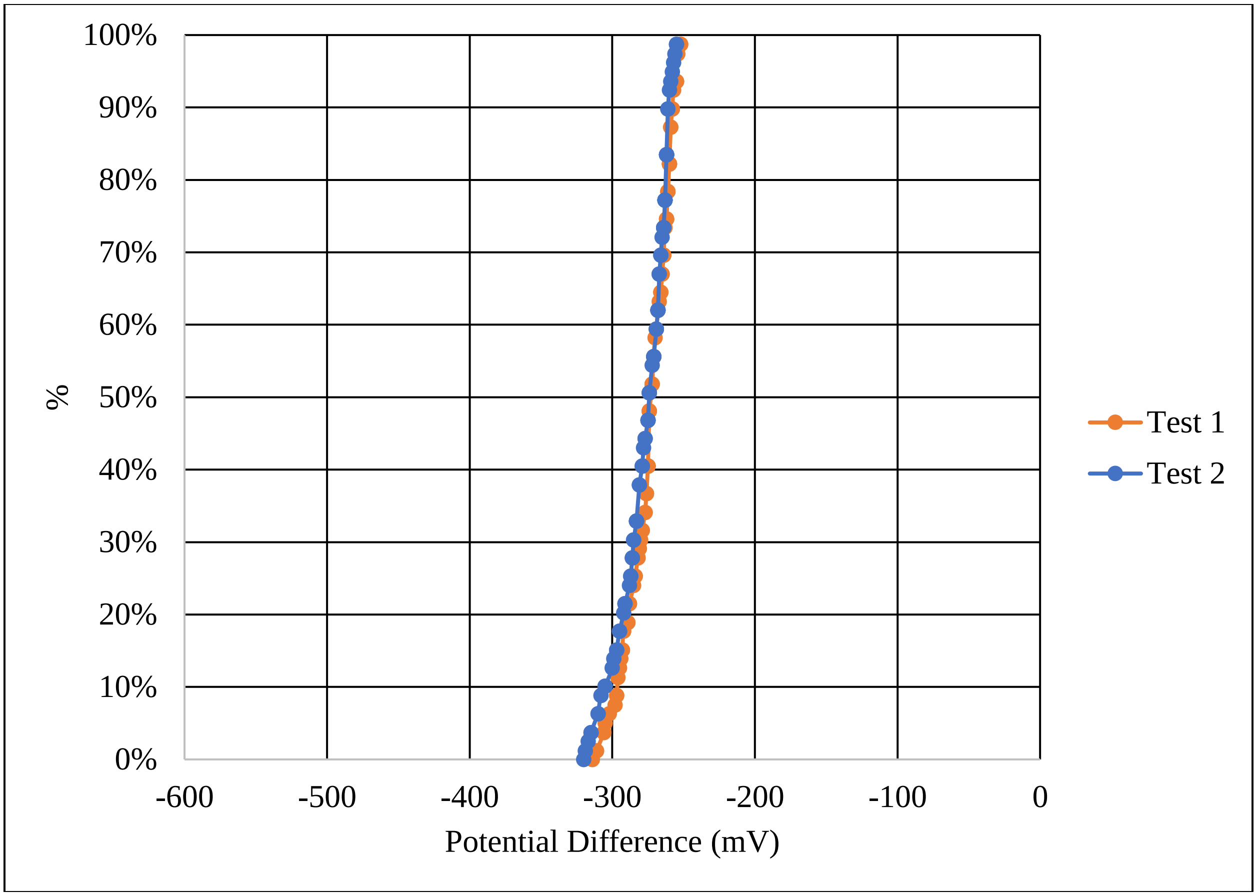


Figure E.141 Shaft 49 percentile distributions

Table E.95 Shaft 50, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-272	-262	-253	-252	-254	-261	-268	-273
3"	-265	-254	-252	-252	-255	-261	-267	-273
6"	-260	-251	-255	-258	-256	-262	-268	-274
9"	-258	-255	-257	-257	-258	-264	-267	-273
12"	-259	-257	-258	-258	-260	-260	-267	-275
15"	-265	-258	-258	-260	-261	-264	-267	-276
18"	-269	-265	-263	-261	-260	-260	-266	-276
21"	-274	-269	-268	-263	-258	-262	-265	-274
24"	-275	-274	-266	-262	-261	-263	-266	-270
27"	-279	-271	-266	-264	-260	-258	-262	-266

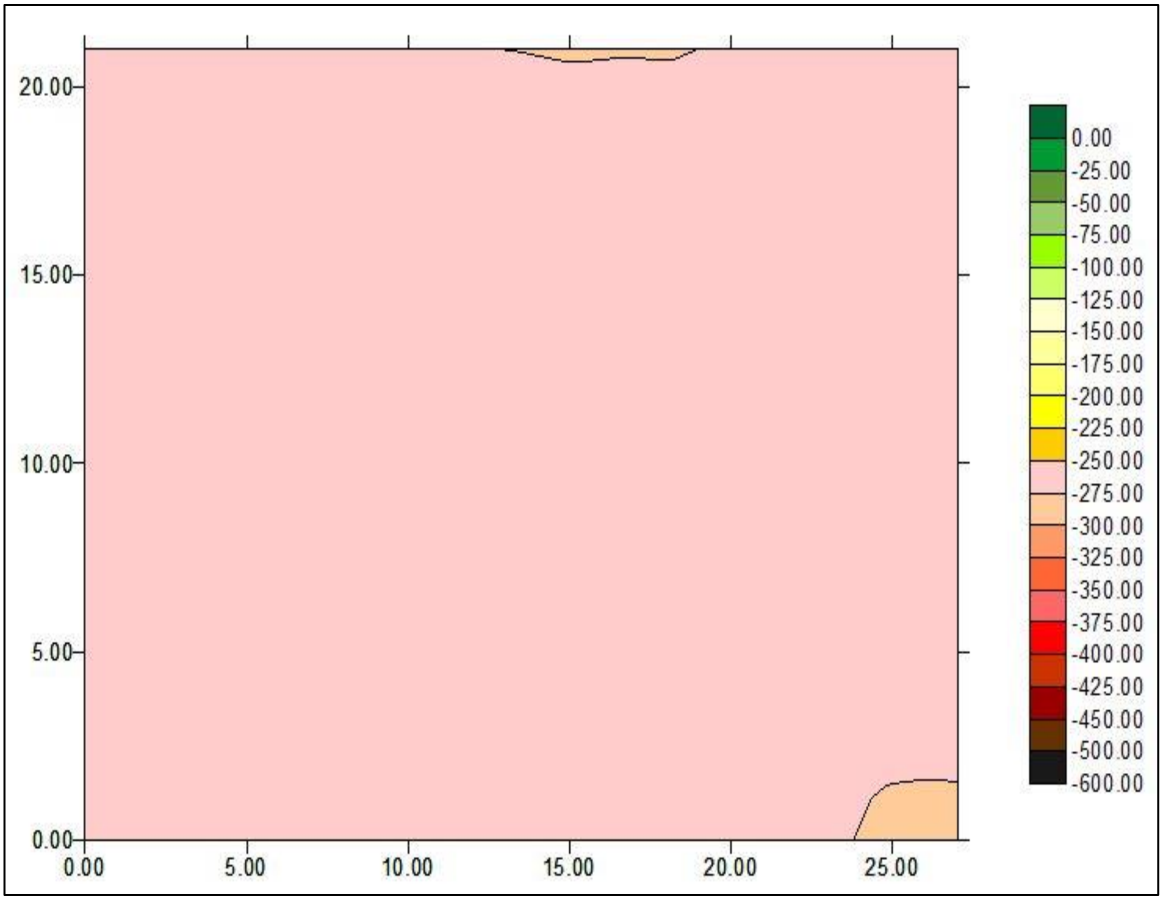


Figure E.142 Shaft 50, test 1 data map

Table E.96 Shaft 50, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-239	-240	-241	-241	-248	-234	-250	-260
3"	-241	-244	-247	-246	-249	-249	-256	-262
6"	-246	-250	-247	-249	-259	-250	-258	-271
9"	-251	-252	-250	-249	-248	-257	-264	-271
12"	-266	-262	-252	-252	-260	-265	-268	-273
15"	-274	-269	-263	-261	-260	-262	-268	-273
18"	-285	-282	-273	-265	-263	-261	-266	-273
21"	-281	-275	-272	-267	-262	-262	-259	-270
24"	-278	-276	-274	-268	-266	-263	-263	-273
27"	-278	-266	-262	-255	-256	-260	-264	-275

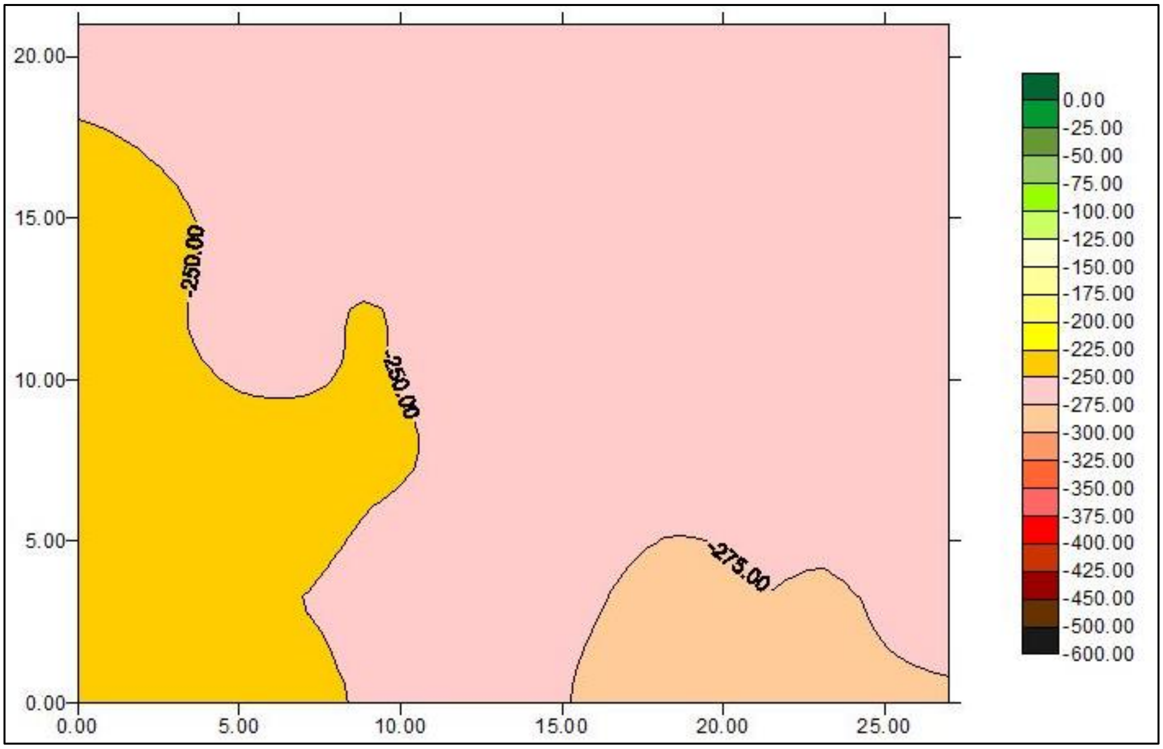


Figure E.143 Shaft 50 test 2 data map

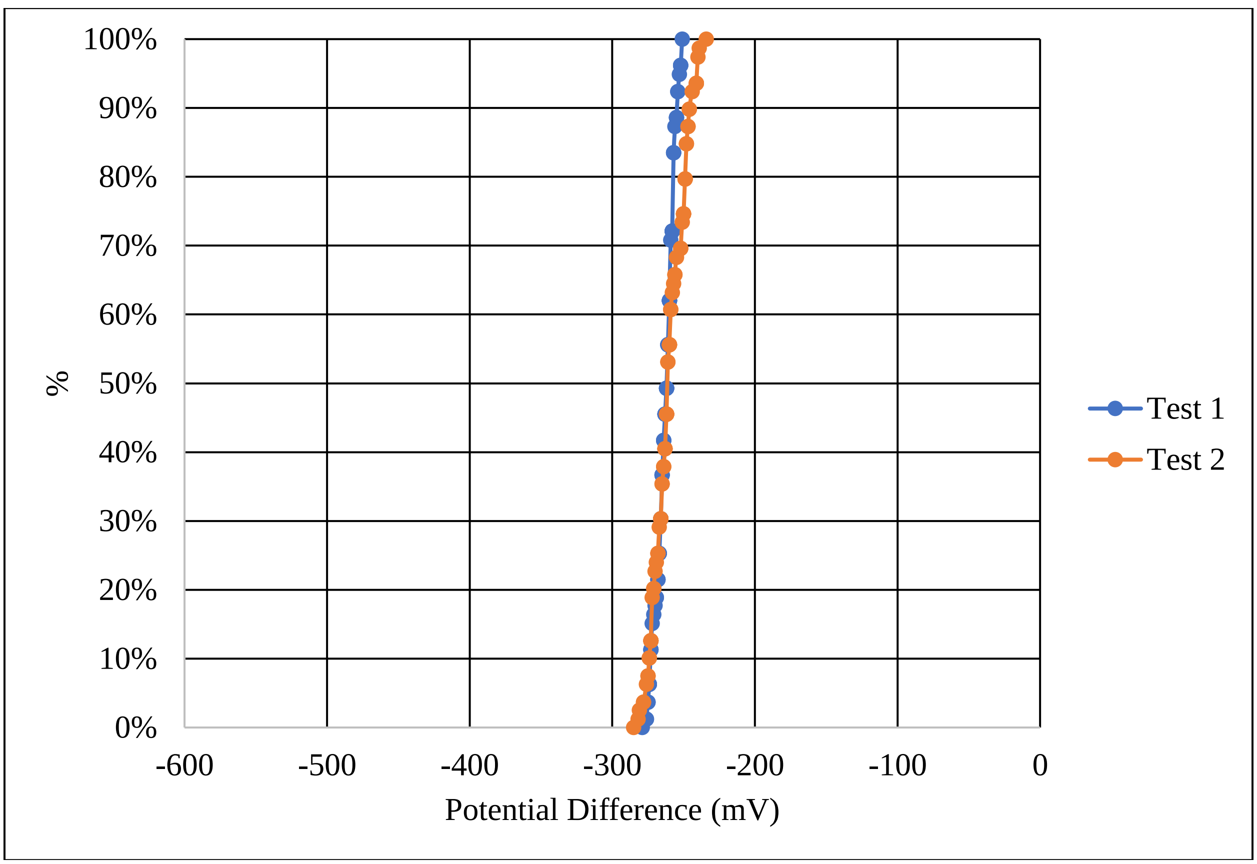


Figure E.144 Shaft 50 percentile distributions

Table E.97 Shaft 51, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-299	-278	-261	-251	-248	-252	-254	-258
3"	-306	-282	-262	-252	-245	-255	-259	-265
6"	-314	-285	-265	-252	-248	-251	-251	-261
9"	-312	-283	-262	-250	-248	-250	-251	-255
12"	-308	-285	-261	-254	-248	-246	-248	-252
15"	-306	-282	-263	-250	-246	-245	-244	-248
18"	-304	-280	-259	-254	-242	-241	-240	-244
21"	-305	-281	-258	-250	-239	-238	-240	-242
24"	-309	-284	-261	-248	-245	-240	-241	-242
27"	-311	-283	-259	-246	-239	-239	-243	-248

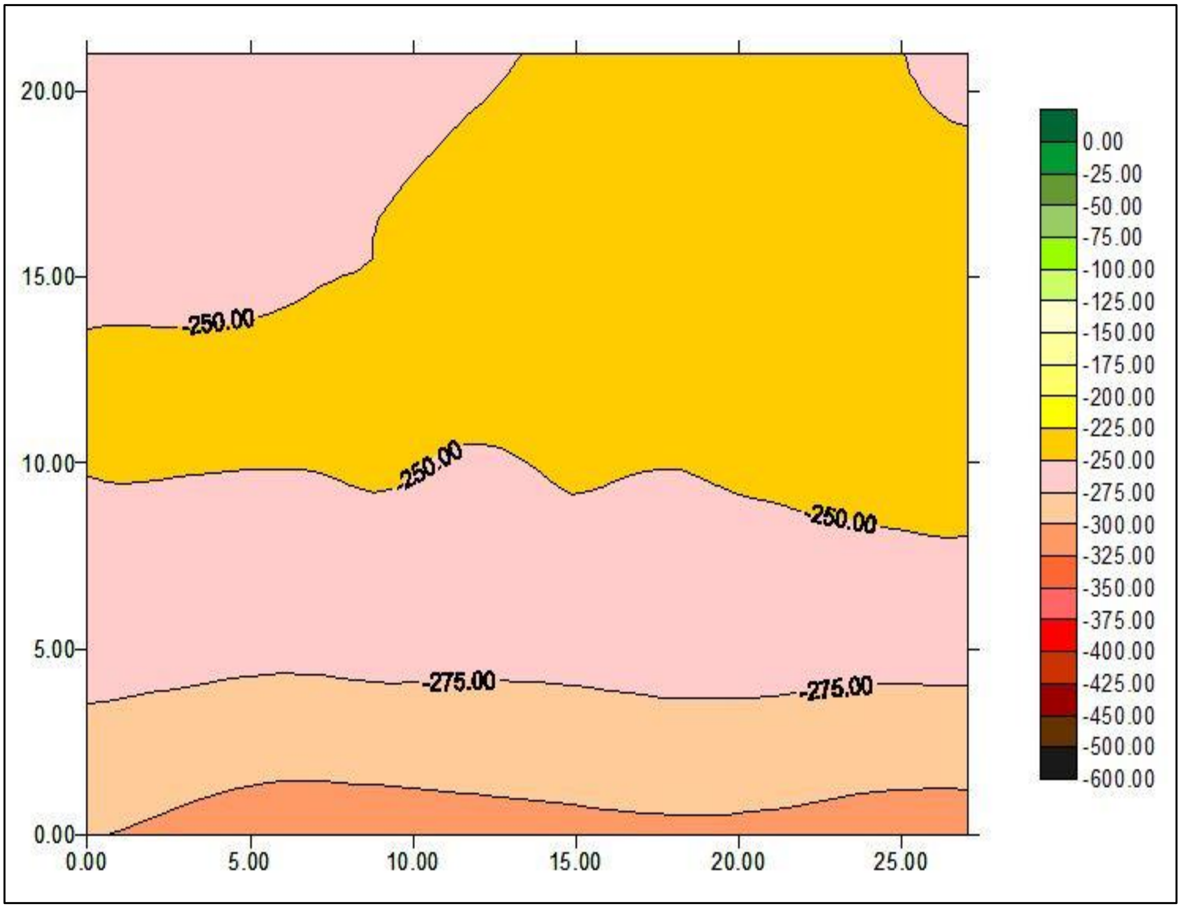


Figure E.145 Shaft 51, test 1 data map

Table E.98 Shaft 51, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-335	-323	-307	-306	-312	-329	-354	-366
3"	-343	-328	-314	-316	-323	-342	-361	-376
6"	-346	-329	-325	-328	-338	-349	-363	-377
9"	-349	-338	-332	-331	-338	-348	-364	-376
12"	-350	-344	-338	-334	-339	-355	-368	-381
15"	-348	-344	-338	-338	-350	-362	-376	-386
18"	-346	-341	-336	-337	-345	-361	-373	-390
21"	-342	-338	-333	-336	-347	-361	-377	-396
24"	-335	-334	-330	-332	-341	-357	-380	-400
27"	-337	-334	-329	-328	-356	-354	-379	-404

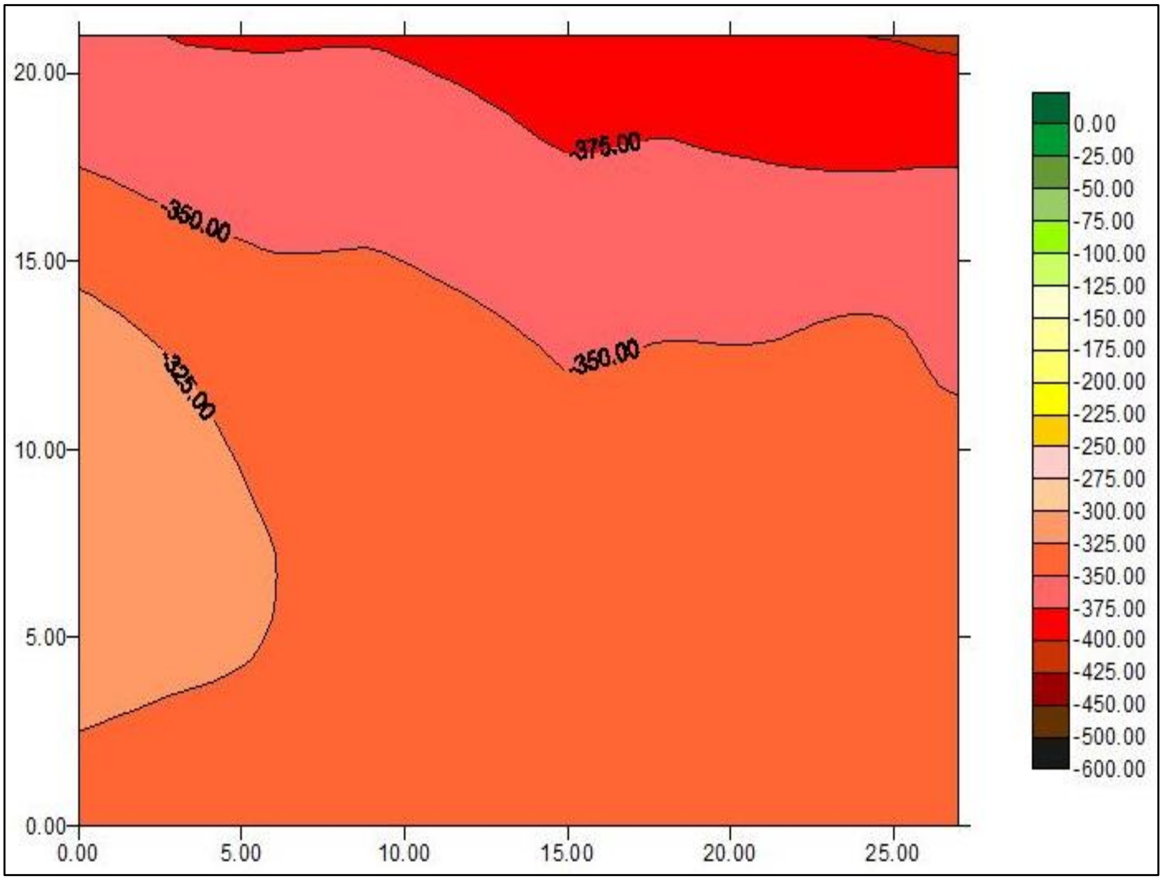


Figure E.146 Shaft 51 test 2 data map

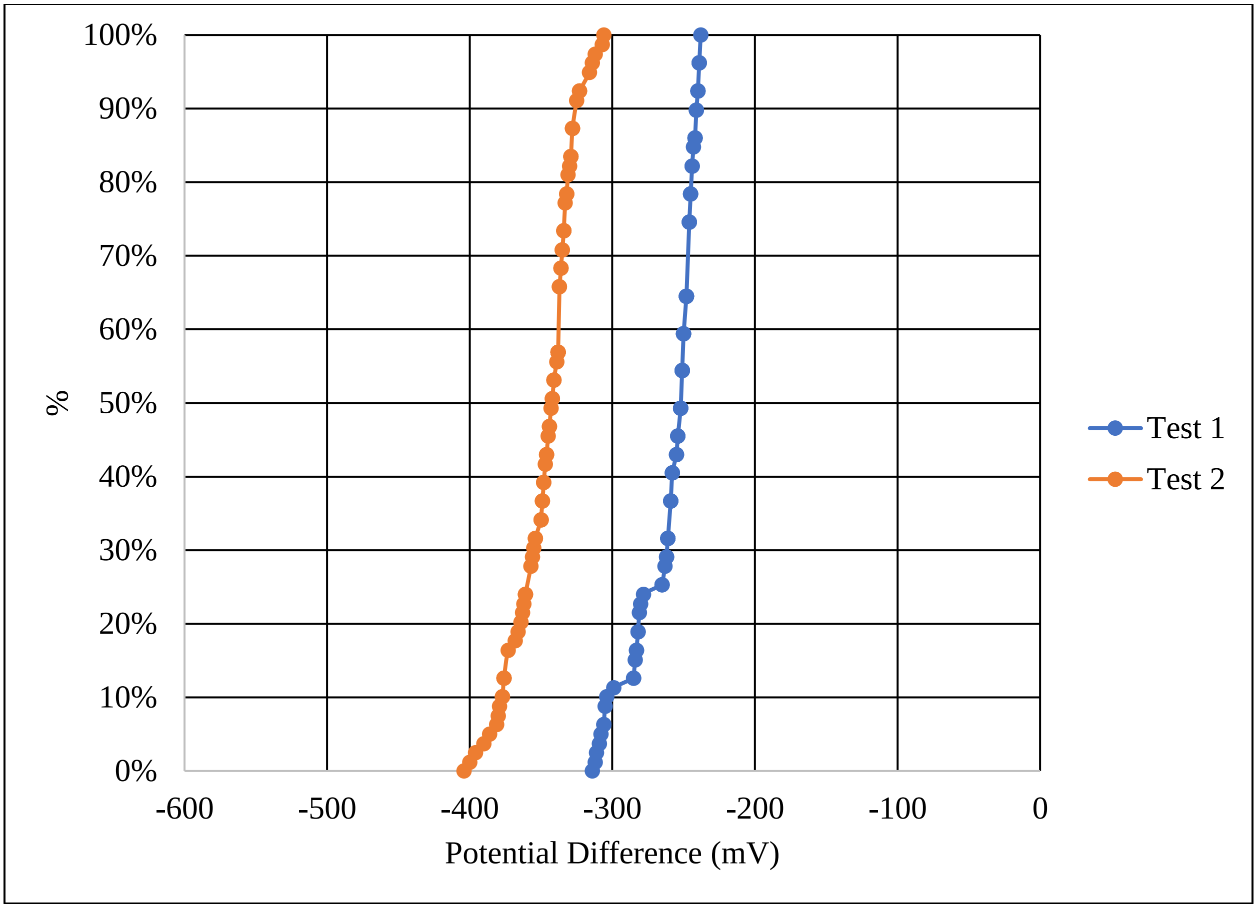


Figure E.147 Shaft 51 percentile distributions

Table E.99 Shaft 52, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-305	-283	-262	-255	-255	-261	-274	-292
3"	-309	-281	-260	-254	-252	-258	-271	-289
6"	-310	-282	-258	-255	-251	-256	-269	-288
9"	-309	-285	-257	-256	-251	-255	-269	-285
12"	-310	-284	-258	-253	-252	-256	-269	-285
15"	-316	-284	-260	-255	-251	-256	-270	-285
18"	-314	-286	-262	-253	-250	-257	-271	-289
21"	-308	-280	-262	-252	-250	-259	-271	-288
24"	-305	-279	-261	-252	-250	-258	-270	-286
27"	-305	-279	-262	-251	-249	-256	-268	-286

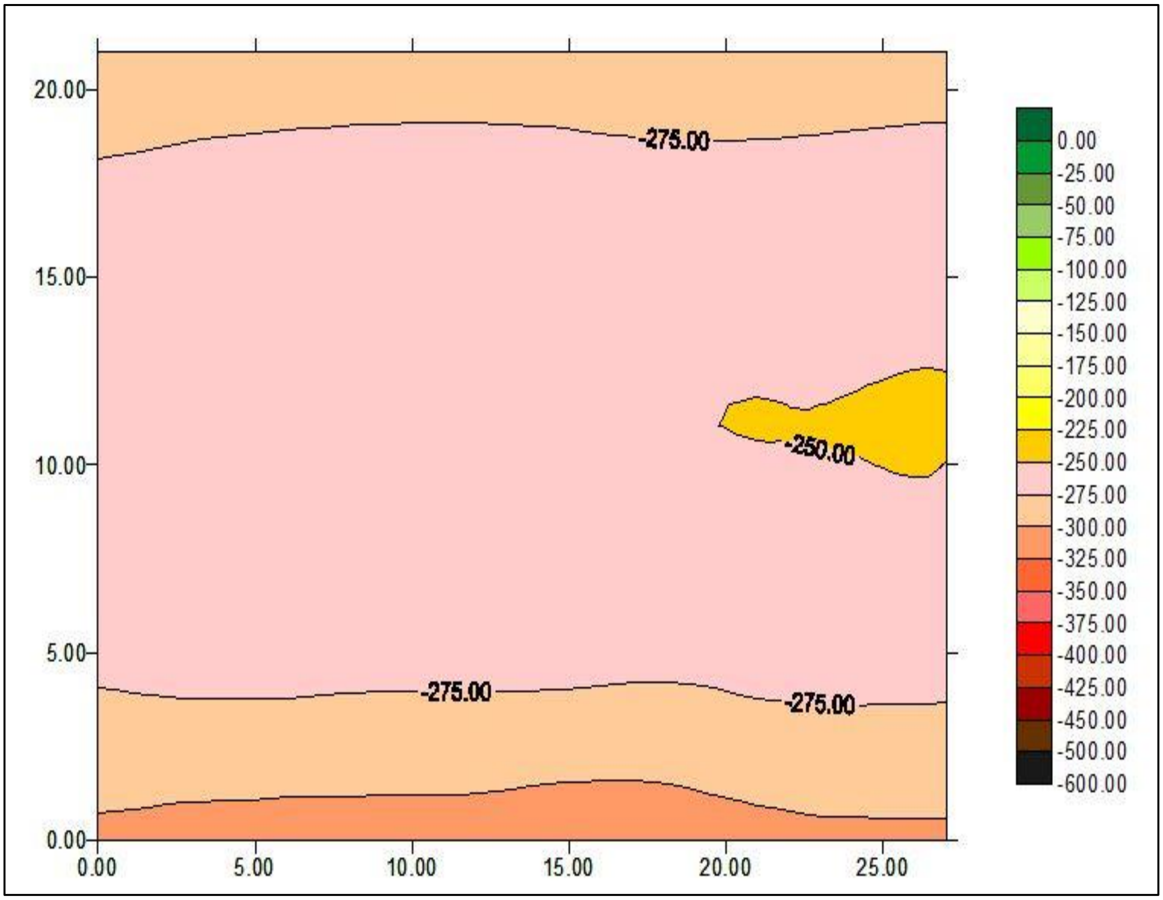


Figure E.148 Shaft 52, test 1 data map

Table E.100 Shaft 52, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-279	-260	-243	-240	-244	-257	-287	-319
3"	-282	-262	-246	-240	-247	-261	-289	-320
6"	-283	-260	-247	-240	-246	-261	-291	-321
9"	-280	-262	-247	-242	-246	-261	-293	-322
12"	-278	-263	-248	-242	-249	-262	-290	-321
15"	-283	-263	-249	-242	-251	-263	-293	-319
18"	-286	-265	-249	-245	-253	-268	-296	-321
21"	-286	-264	-252	-246	-255	-273	-303	-328
24"	-294	-272	-259	-255	-261	-274	-301	-330
27"	-298	-275	-260	-259	-263	-280	-307	-335

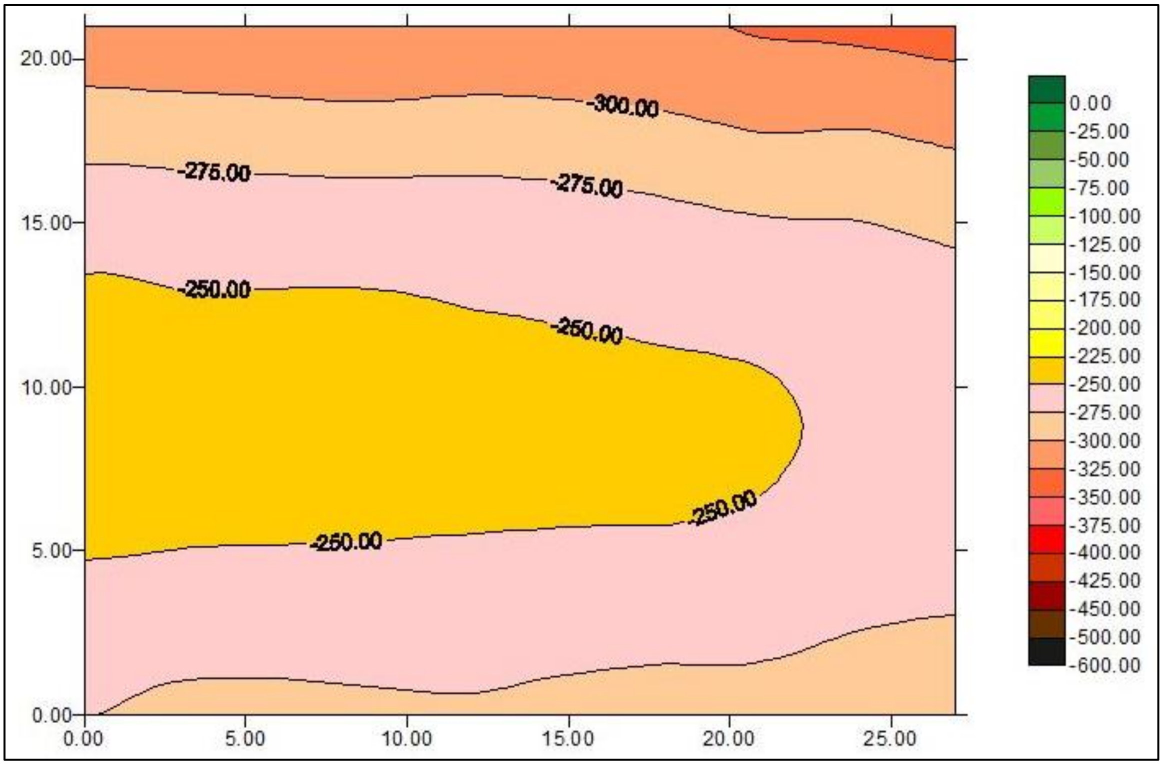


Figure E.149 Shaft 52 test 2 data map

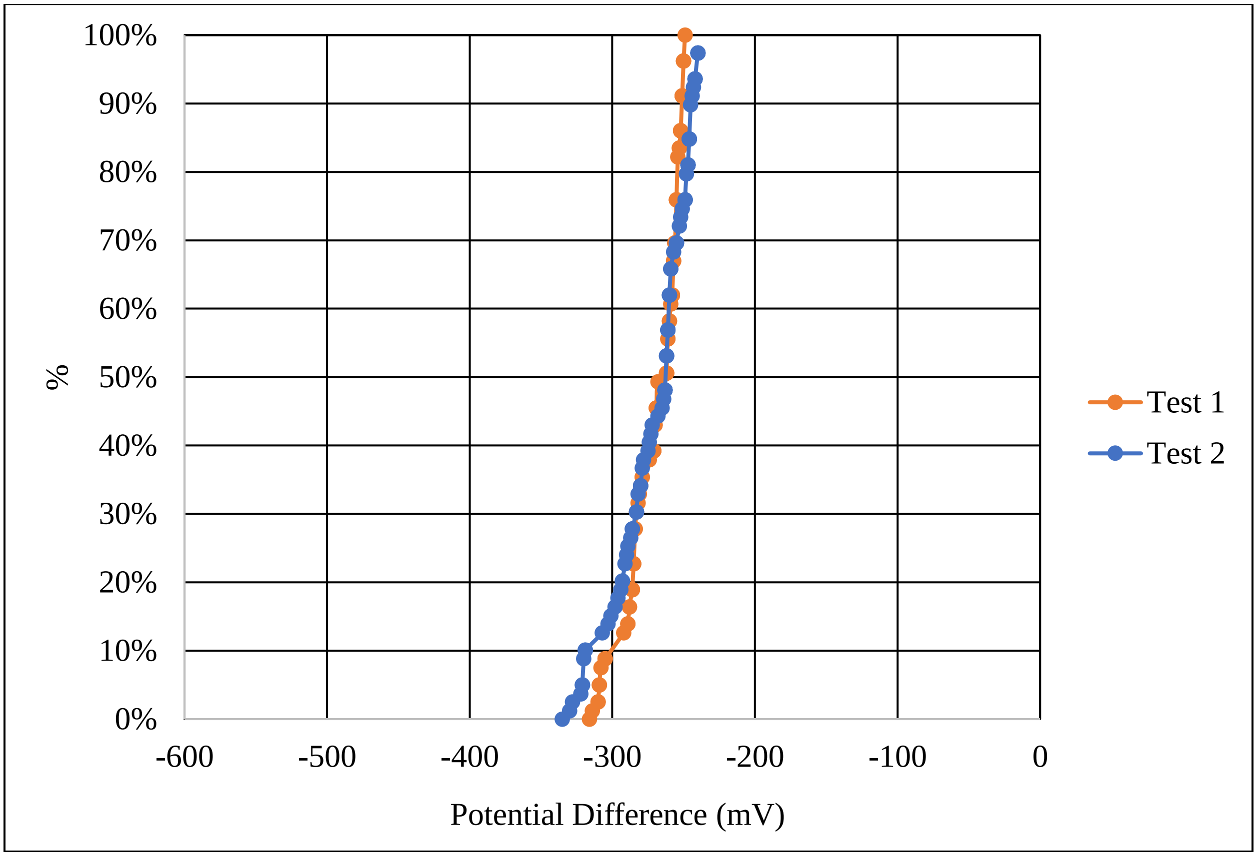


Figure E.150 Shaft 52 percentile distributions

Table E.101 Shaft 53, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-181	-175	-178	-177	-174	-160	-158	-141
3"	-181	-174	-171	-172	-172	-164	-162	-150
6"	-192	-192	-186	-186	-181	-174	-176	-180
9"	-185	-191	-184	-184	-174	-175	-179	-172
12"	-191	-190	-186	-175	-181	-178	-191	-183
15"	-193	192	-189	-190	-185	-183	-186	-182
18"	-195	-191	-189	-184	-178	-181	-185	-158
21"	-190	-194	-191	-177	-174	-187	-187	-129
24"	-205	-201	-197	-180	-186	-189	-190	-158
27"	-202	-196	-200	-165	-184	-184	-183	-120

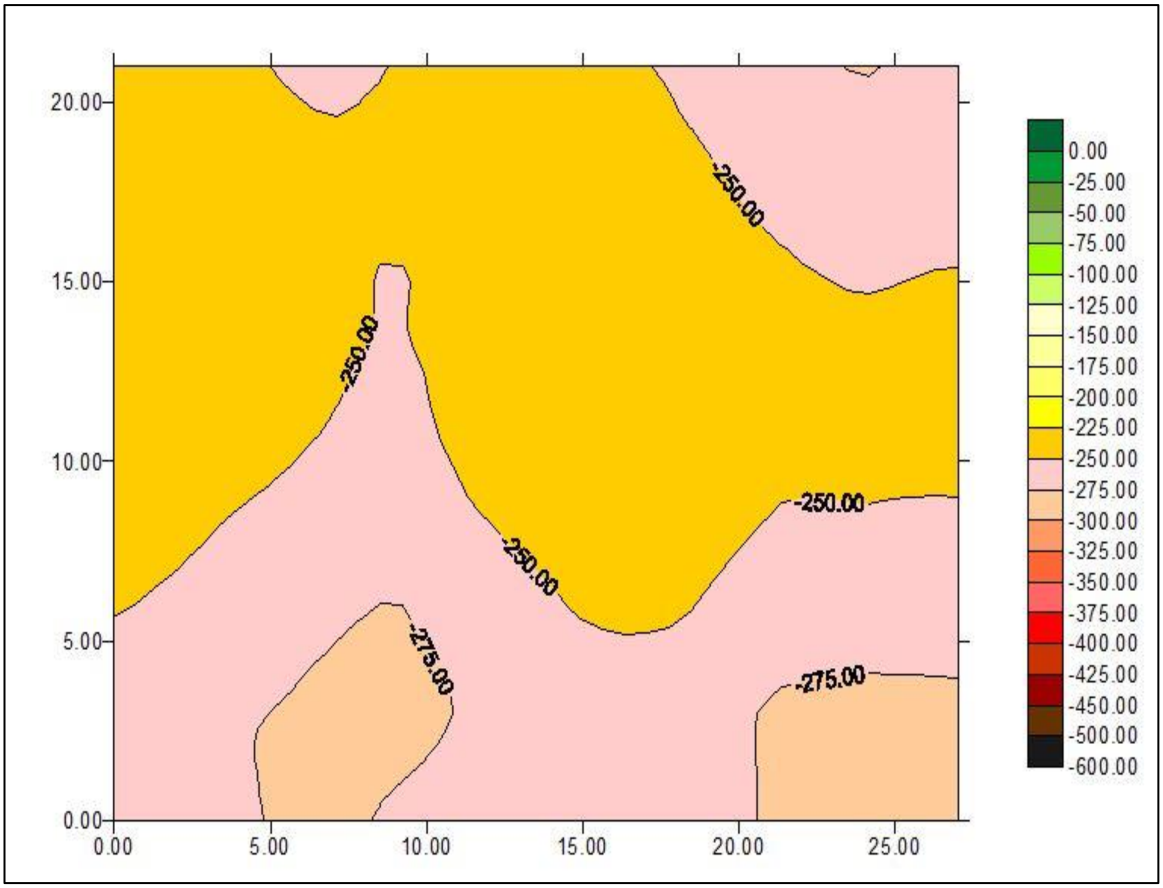


Figure E.151 Shaft 53, test 1 data map

Table E.102 Shaft 53, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-251	-239	-242	-242	-242	-235	-245	-253
3"	-252	-247	-246	-239	-239	-234	-244	-245
6"	-255	-244	-240	-235	-238	-233	-243	-248
9"	-260	-256	-246	-249	-241	-232	-237	-241
12"	-259	-249	-244	-239	-232	-229	-234	-237
15"	-258	-249	-243	-234	-235	-233	-242	-239
18"	-253	-246	-239	-242	-236	-239	-238	-238
21"	-254	-246	-241	-235	-230	-231	-230	-236
24"	-252	-249	-244	-238	-236	-237	-241	-234
27"	-260	-254	-248	-247	-240	-239	-240	-236

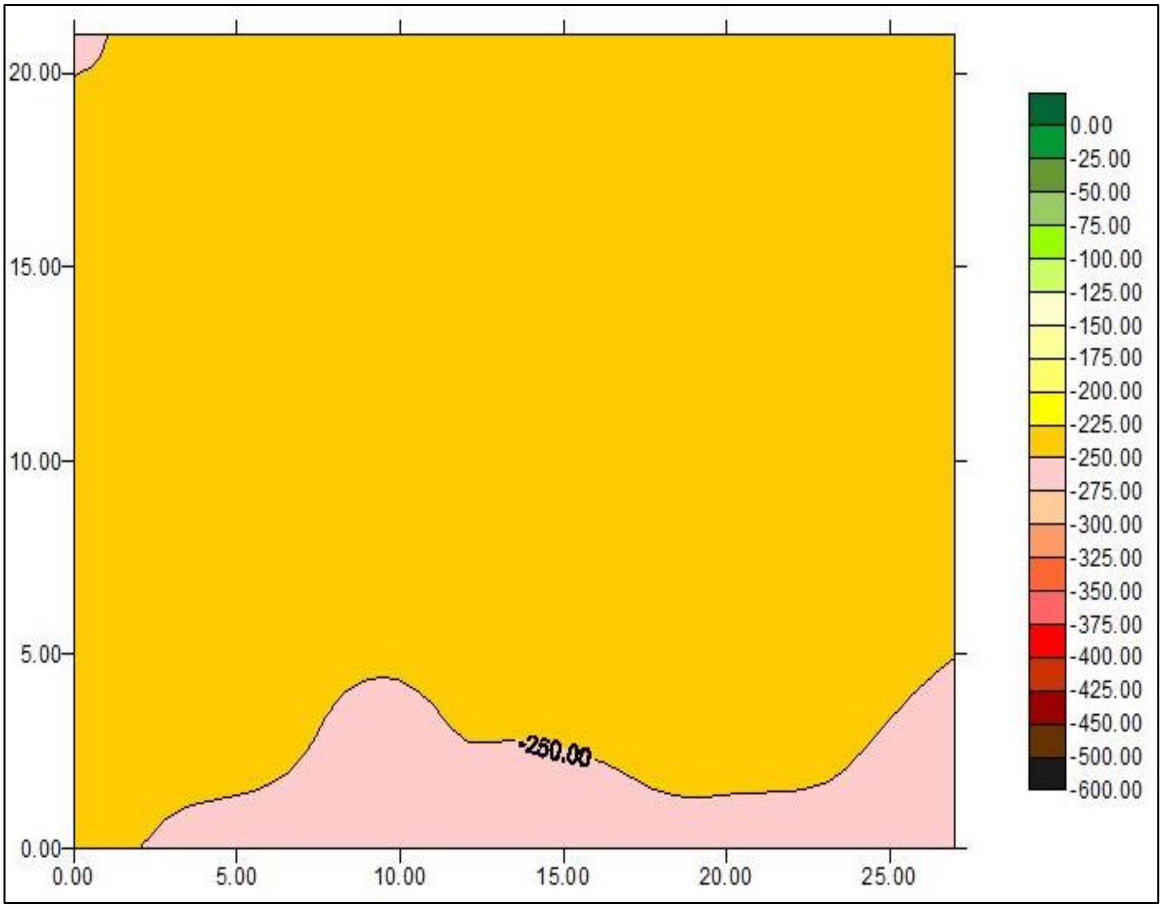


Figure E.152 Shaft 53 test 2 data map

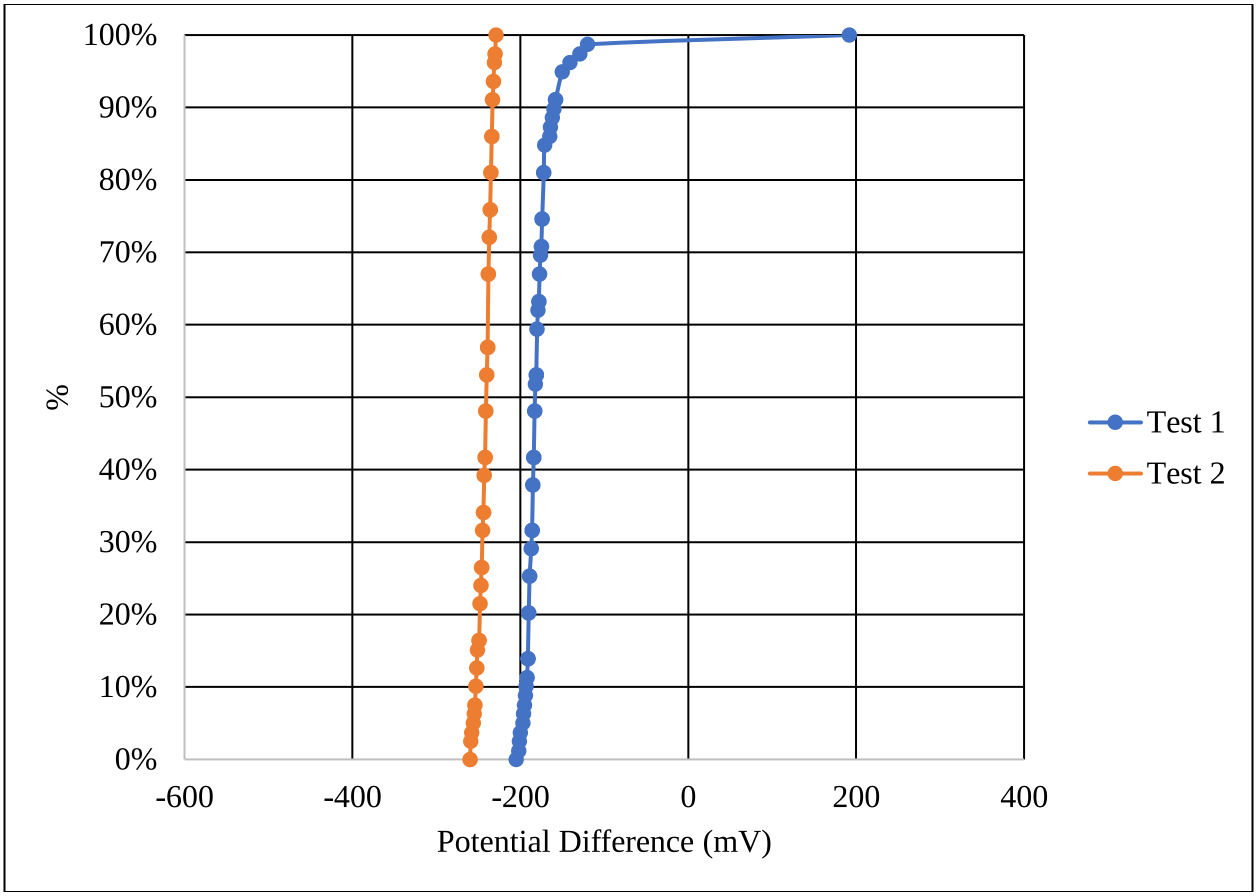


Figure E.153 Shaft 53 percentile distributions

Table E.103 Shaft 54, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-180	-187	-184	-176	-176	-171	-174	-175
3"	-185	-199	-191	-179	-184	-182	-183	-180
6"	-199	-199	-188	-186	-185	-179	-176	-175
9"	-205	-210	-196	-188	-182	-172	-179	-179
12"	-215	-212	-203	-189	-182	-192	-184	-185
15"	-206	-211	-201	-188	-179	-187	-177	-174
18"	-205	-190	-210	-199	-183	-184	-181	-181
21"	-195	-188	-182	-178	-175	-173	-181	-175
24"	-185	-183	-183	-173	-175	-174	-167	-174
27"	-197	-180	-197	-181	-166	-176	-160	-175

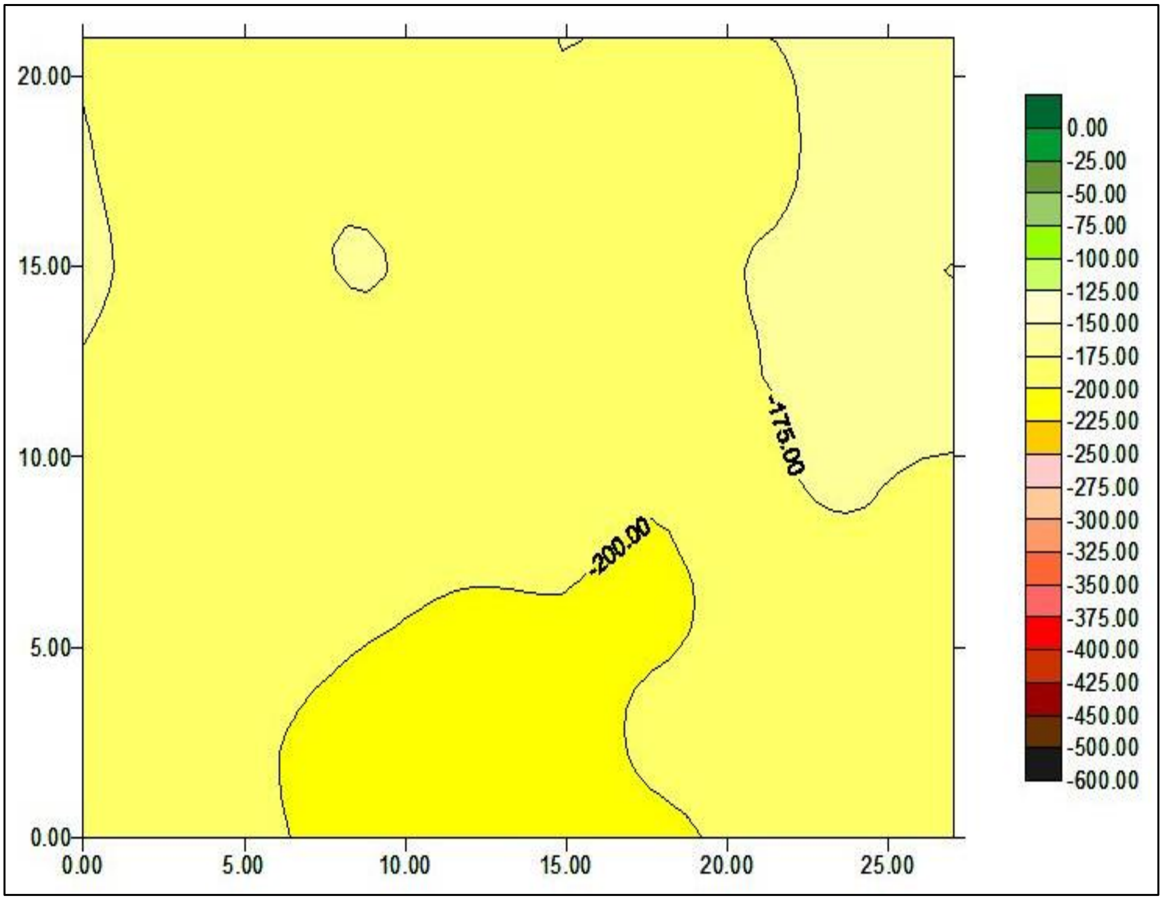


Figure E.154 Shaft 54, test 1 data map

Table E.104 Shaft 54, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-225	-228	-227	-224	-221	-221	-227	-231
3"	-224	-233	-232	-224	-230	-233	-241	-238
6"	-235	-239	-232	-231	-233	-230	-236	-239
9"	-236	-246	-242	-232	-233	-232	-241	-244
12"	-242	-247	-245	-236	-235	-249	-248	-252
15"	-237	-244	-245	-240	-233	-245	-239	-244
18"	-236	-238	-251	-241	-232	-239	-240	-239
21"	-238	-237	-235	-236	-233	-231	-240	-235
24"	-224	-226	-234	-228	-228	-228	-225	-223
27"	-242	-231	-246	-233	-228	-233	-220	-227

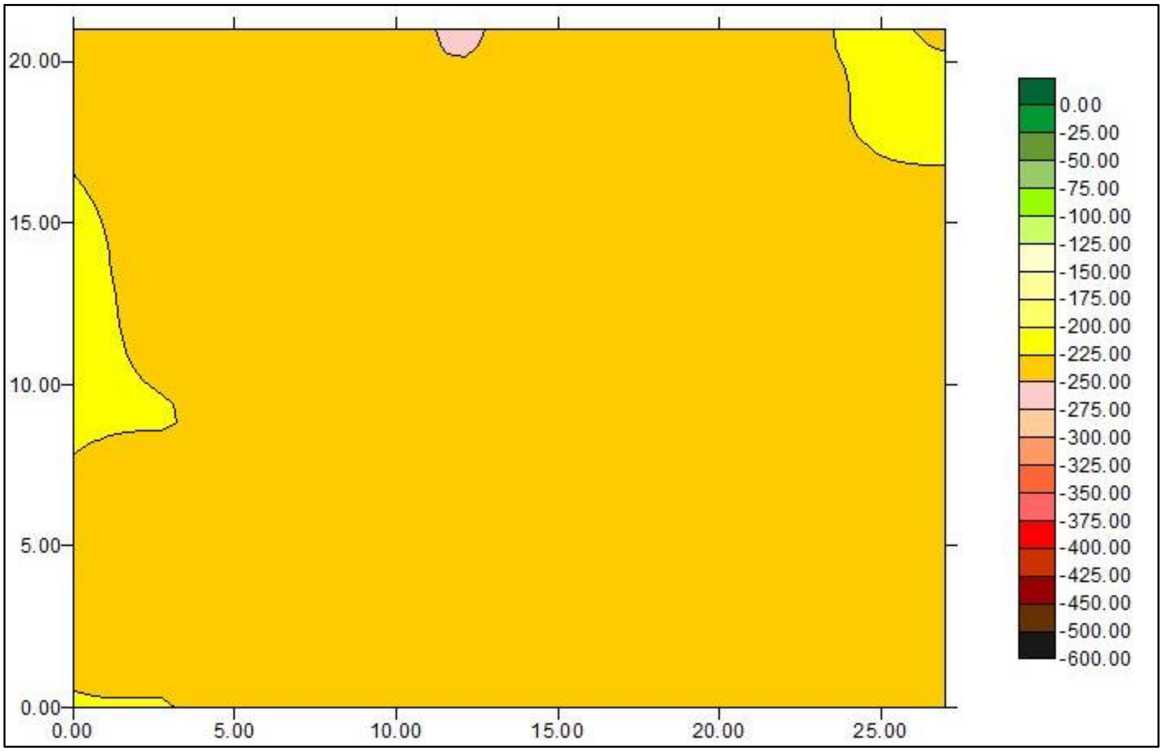


Figure E.155 Shaft 54 test 2 data map

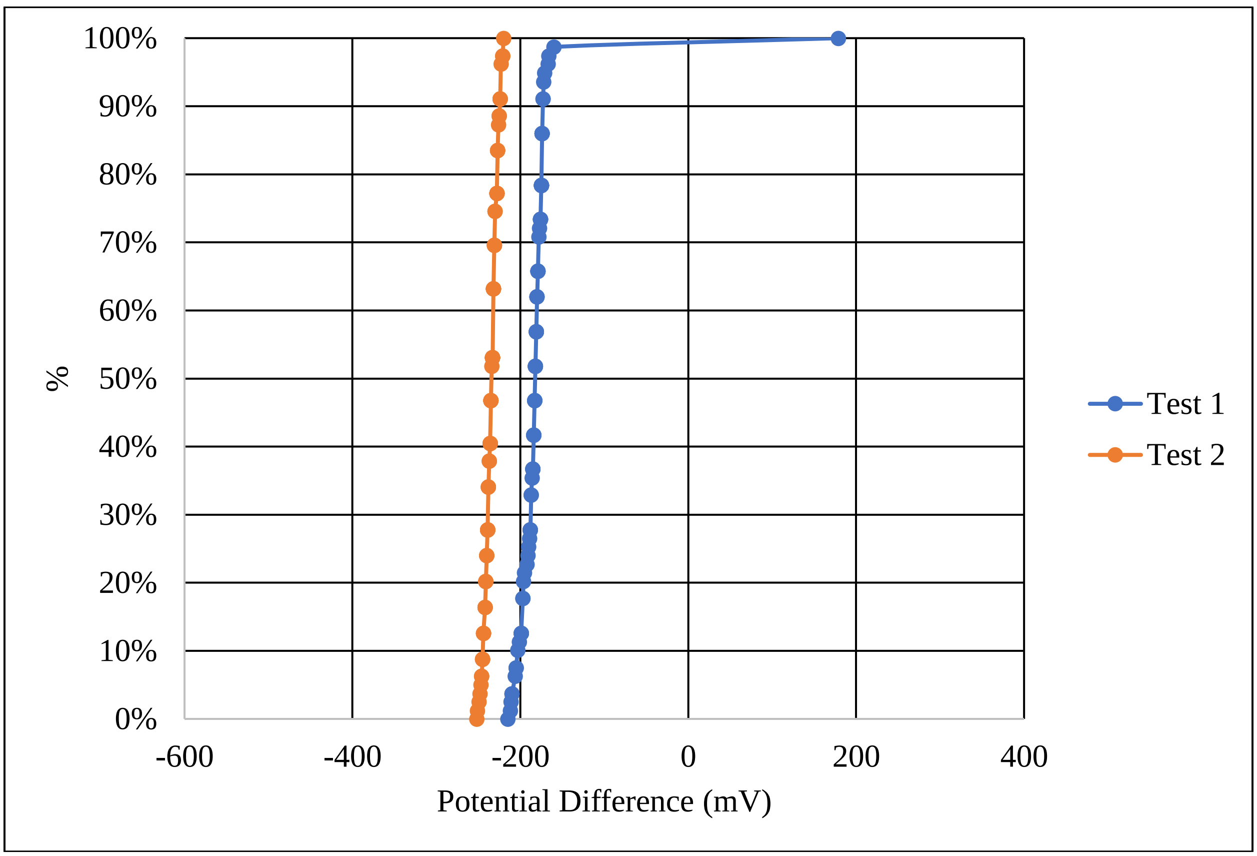


Figure E.156 Shaft 54 percentile distributions

Table E.105 Shaft 55, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-179	-201	-187	-191	-194	-194	-193	-172
3"	-205	-196	-199	-199	-199	-196	-195	-173
6"	-189	-198	-189	-192	-192	-194	-209	-177
9"	-199	-197	-188	-187	-187	-188	-198	-183
12"	-186	-198	-198	-192	-192	-185	-201	-178
15"	-190	-190	-188	-186	-186	-186	-183	-171
18"	-189	-182	-186	-184	-184	-180	-181	-174
21"	-185	-187	-191	-181	-181	-181	-180	-174
24"	-189	-195	-195	-186	-186	-177	-177	-179
27"	-190	-186	-185	-184	-184	-180	-172	-176

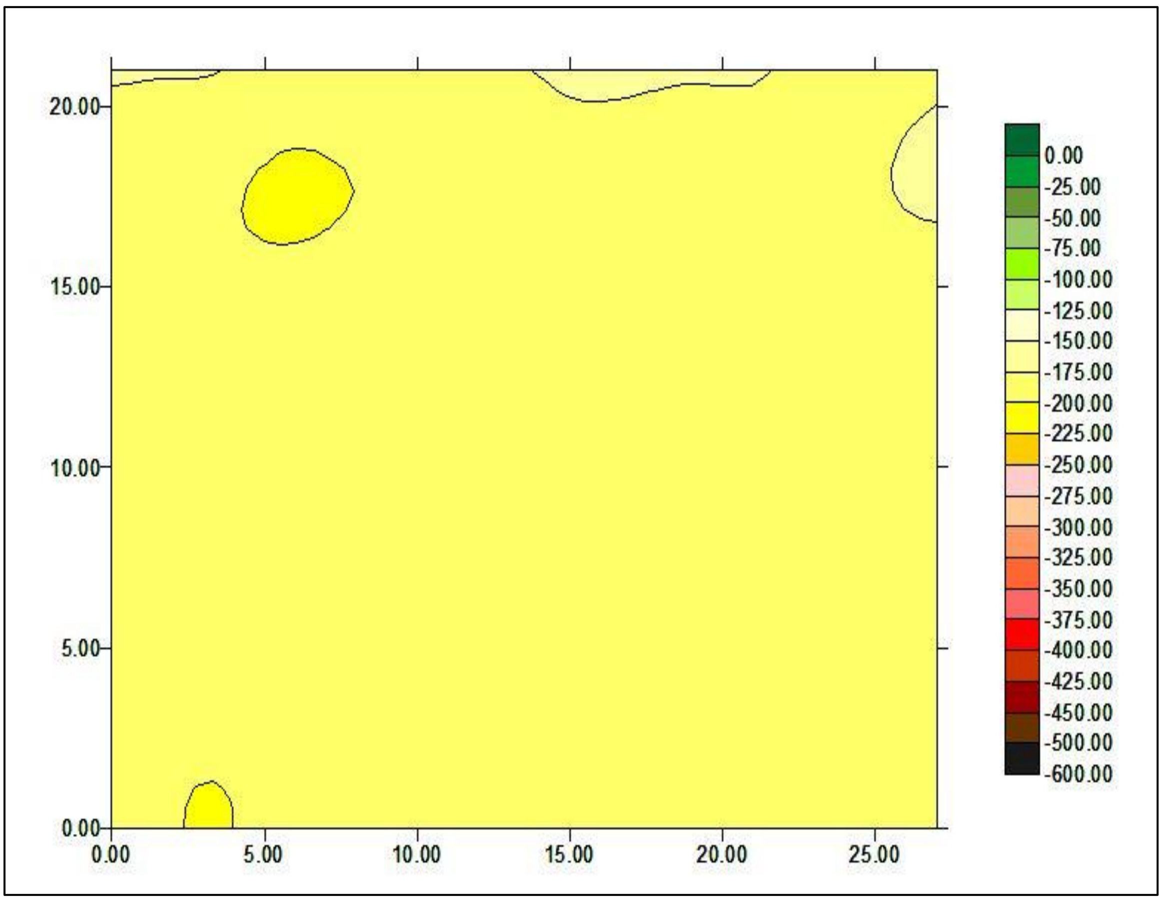


Figure E.157 Shaft 55, test 1 data map

Table E.106 Shaft 55, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-251	-251	-253	-255	-252	-257	-265	-263
3"	-264	-263	-260	-255	-268	-265	-271	-263
6"	-255	-255	-262	-261	-262	-278	-273	-278
9"	-259	-269	-270	-270	-272	-277	-282	-284
12"	-267	-274	-276	-274	-280	-284	-295	-297
15"	-269	-282	-279	-274	-281	-292	-304	-311
18"	-271	-287	-282	-282	-286	-295	-314	-325
21"	-277	-287	-299	-293	-301	-313	-330	-342
24"	-280	-284	-292	-288	-301	-315	-336	-365
27"	-281	-279	-300	-288	-299	-319	-354	-383

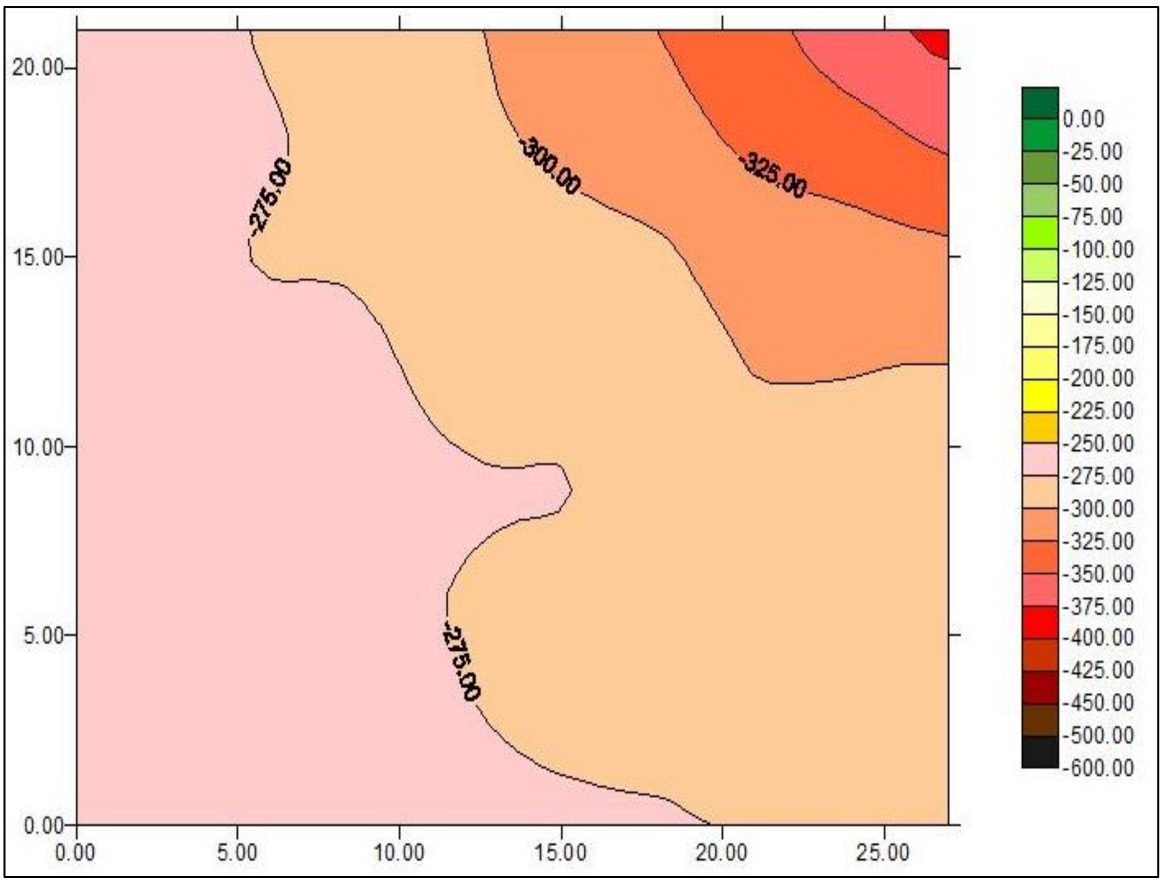


Figure E.158 Shaft 55 test 2 data map

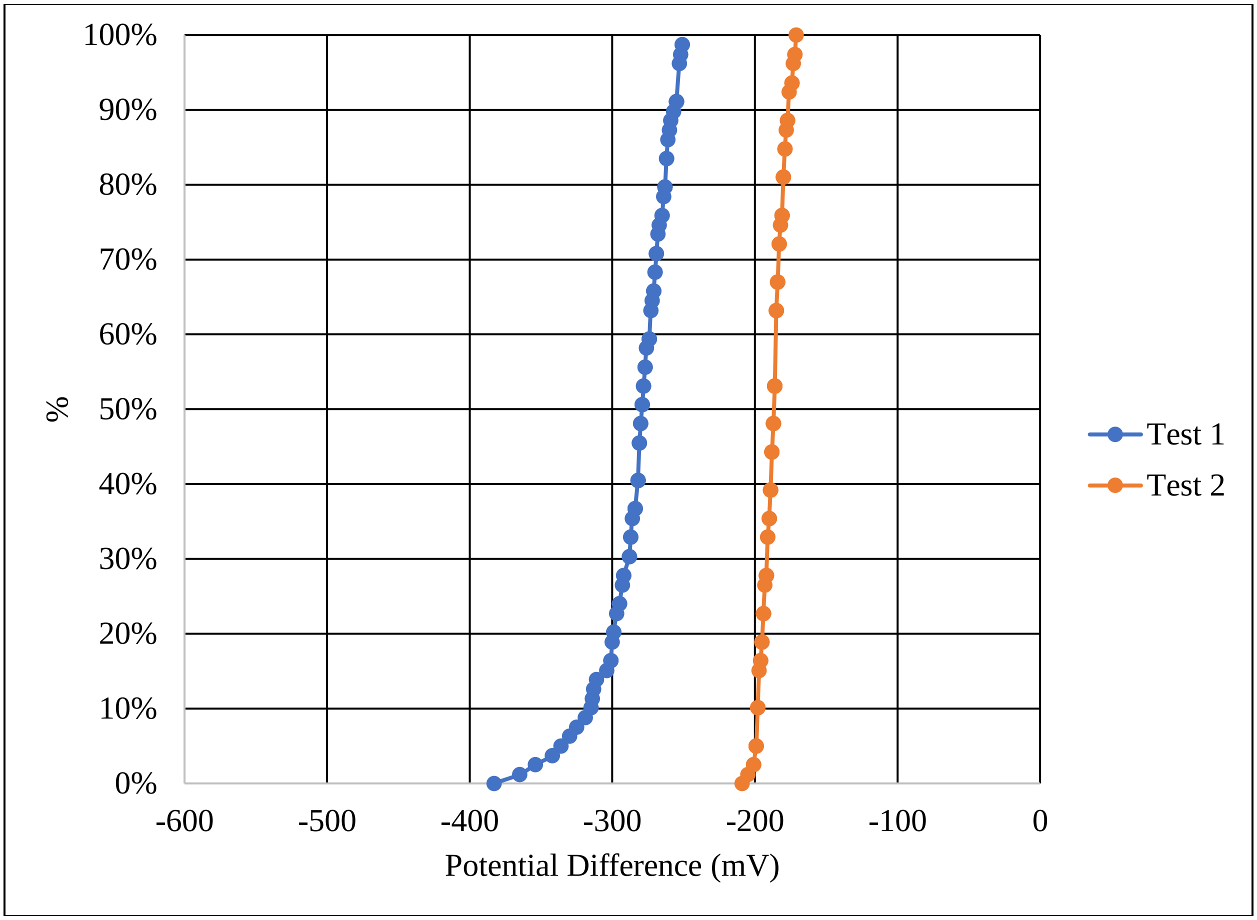


Figure E.159 Shaft 55 percentile distributions

Table E.107 Shaft 56, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-173	-171	-175	-181	-180	-178	-179	-167
3"	-168	-170	-167	-169	-169	-172	-173	-167
6"	-171	-173	-159	-169	-168	-173	-174	-174
9"	-166	-171	-157	-169	-175	-169	-176	-171
12"	-169	-158	-154	-162	-157	-168	-166	-165
15"	-166	-161	-156	-159	-165	-167	-167	-164
18"	-148	-162	-158	-173	-173	-180	-175	-167
21"	-156	-158	-151	-158	-168	-168	-160	-166
24"	-162	-158	-152	-154	-158	-165	-154	-158
27"	-161	-162	-160	-168	-155	-158	-169	-160

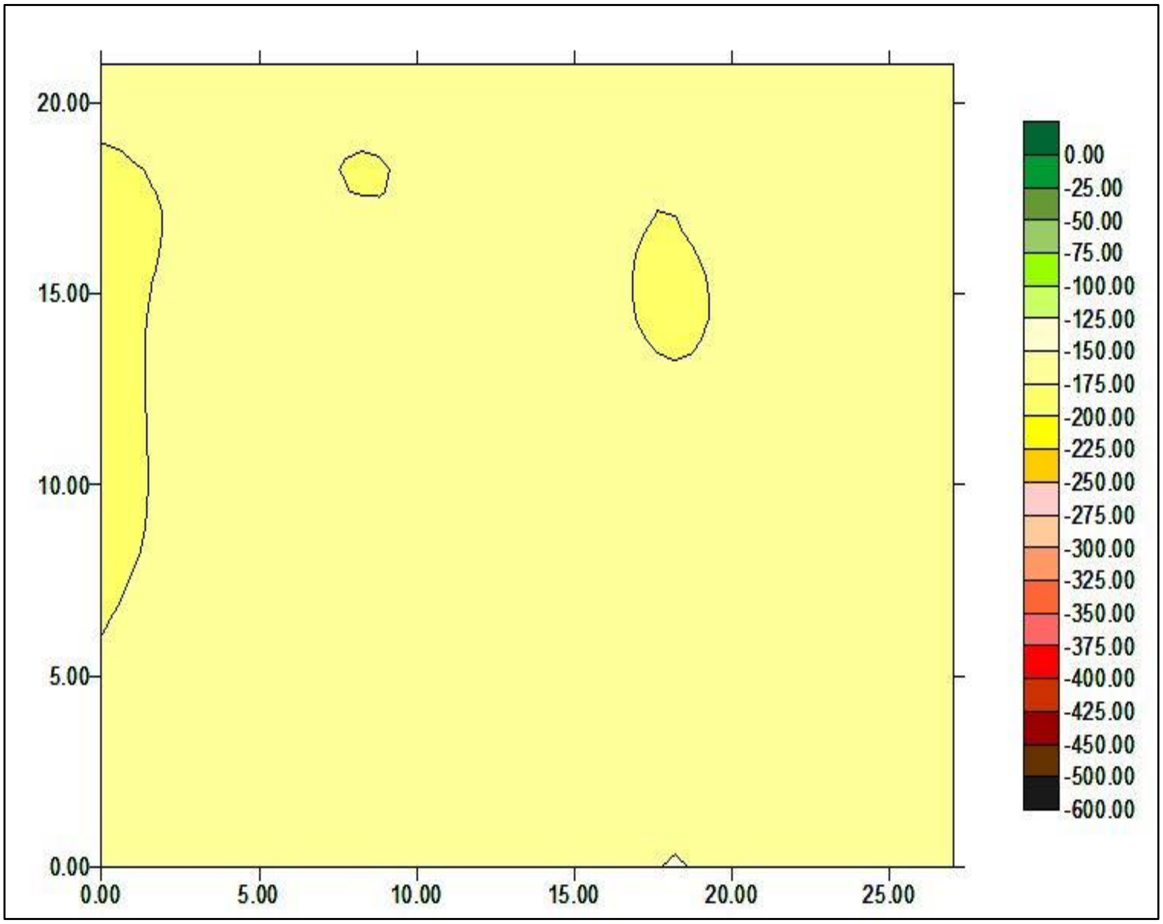


Figure E.160 Shaft 56, test 1 data map

Table E.108 Shaft 56, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-232	-245	-239	-248	-247	-231	-232	-234
3"	-239	-235	-240	-235	-228	-225	-233	-244
6"	-233	-237	-243	-237	-225	-221	-233	-225
9"	-228	-244	-252	-249	-250	-238	-239	-230
12"	-228	-236	-235	-230	-223	-216	-223	-235
15"	-226	-238	-235	-229	-231	-222	-233	-242
18"	-234	-240	-234	-245	-241	-226	-237	-240
21"	-227	-240	-237	-236	-232	-225	-246	-239
24"	-226	-236	-240	-239	-234	-241	-243	-243
27"	-238	-243	-243	-245	-240	-238	-244	-235

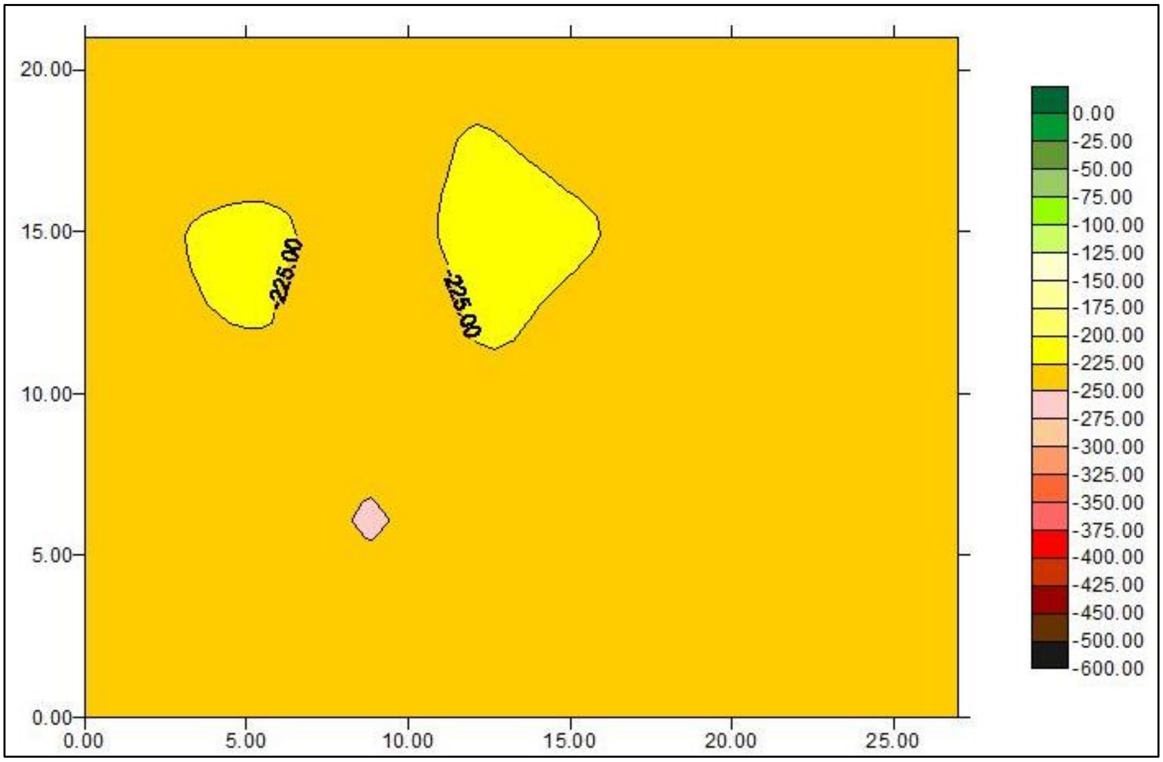


Figure E.161 Shaft 56 test 2 data map

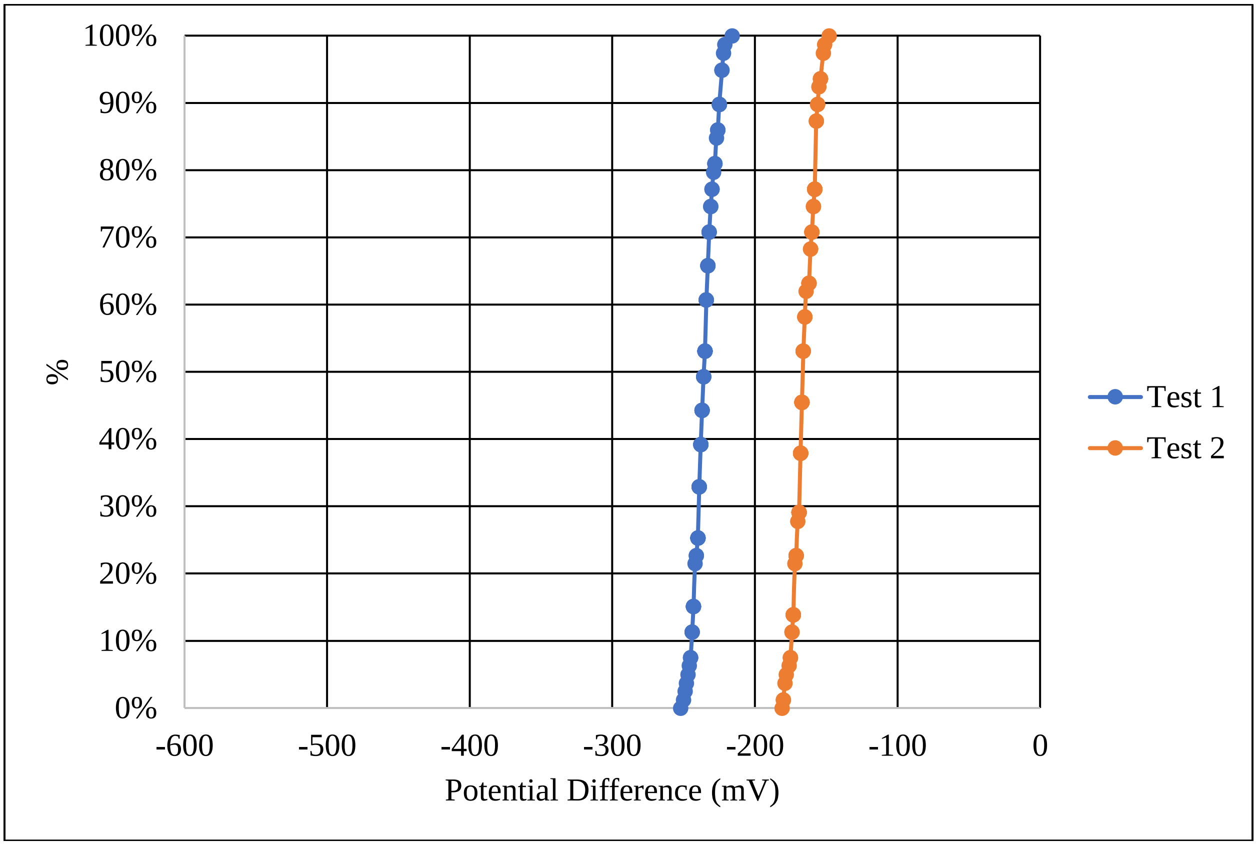


Figure E.162 Shaft 56 percentile distributions

Table E.109 Shaft 57, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-166	-173	-177	-173	-186	-197	-214	-227
3"	-168	-171	-168	-176	-183	-198	-207	-230
6"	-174	-173	-175	-176	-185	-203	-210	-217
9"	-172	-175	-190	-177	-188	-198	-202	-208
12"	-164	-180	-184	-182	-189	-201	-209	-200
15"	-178	-178	-183	-181	-190	-188	-200	-202
18"	-172	-178	-177	-179	-178	-183	-189	-199
21"	-177	-176	-181	-185	-182	-182	-186	-192
24"	-187	-180	-180	-179	-183	-193	-196	-183
27"	-176	-175	-180	-169	-180	-190	-190	-185

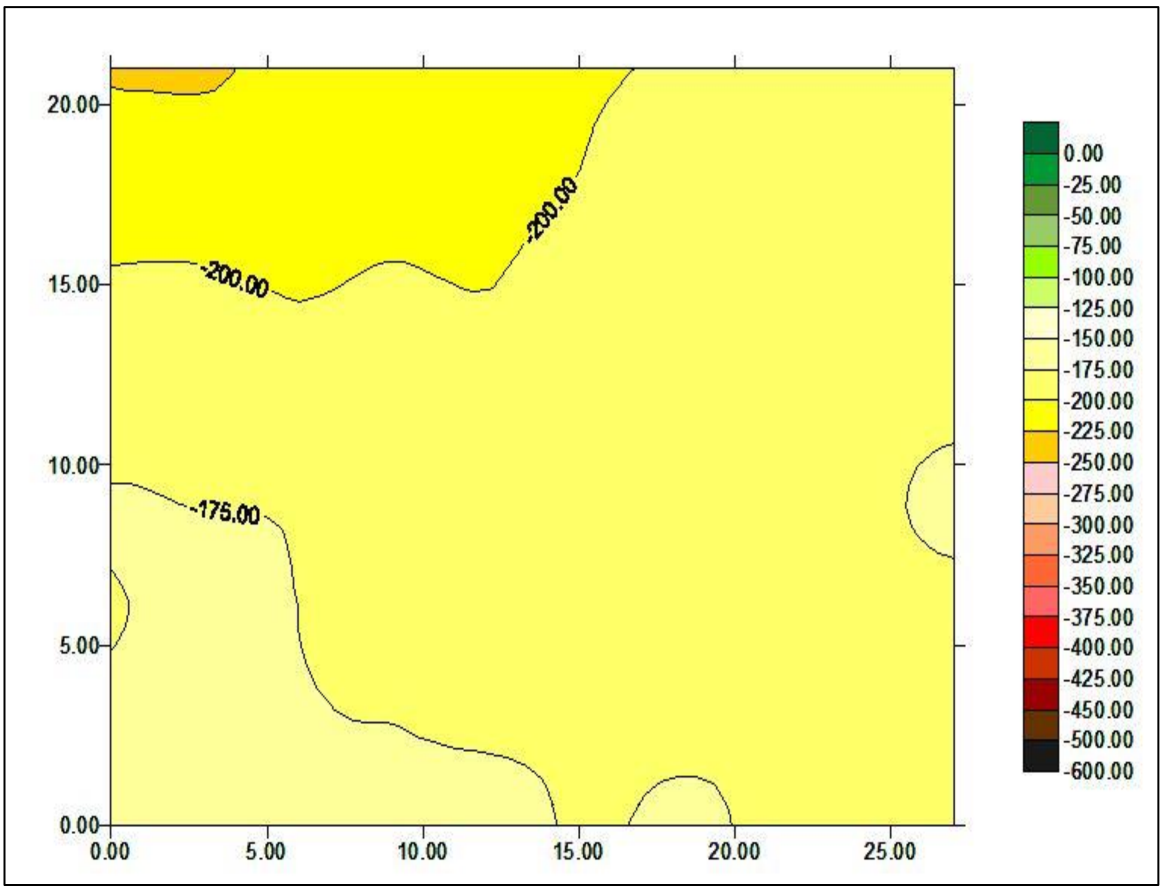


Figure E.163 Shaft 57, test 1 data map

Table E.110 Shaft 57, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-269	-269	-278	-278	-287	-304	-331	-395
3"	-263	-274	-291	-291	-288	-324	-347	-425
6"	-264	-271	-278	-278	-292	-316	-348	-422
9"	-262	-273	-282	-282	-284	-316	-345	-407
12"	-268	-279	-290	-290	-298	-311	-335	-373
15"	-255	-281	-297	-297	-304	-325	-365	-352
18"	-260	-276	-283	-283	-288	-297	-304	-335
21"	-261	-281	-281	-281	-286	-291	-302	-329
24"	-270	-272	-278	-278	-305	-316	-322	-308
27"	-285	-275	-275	-275	-279	-282	-292	-298

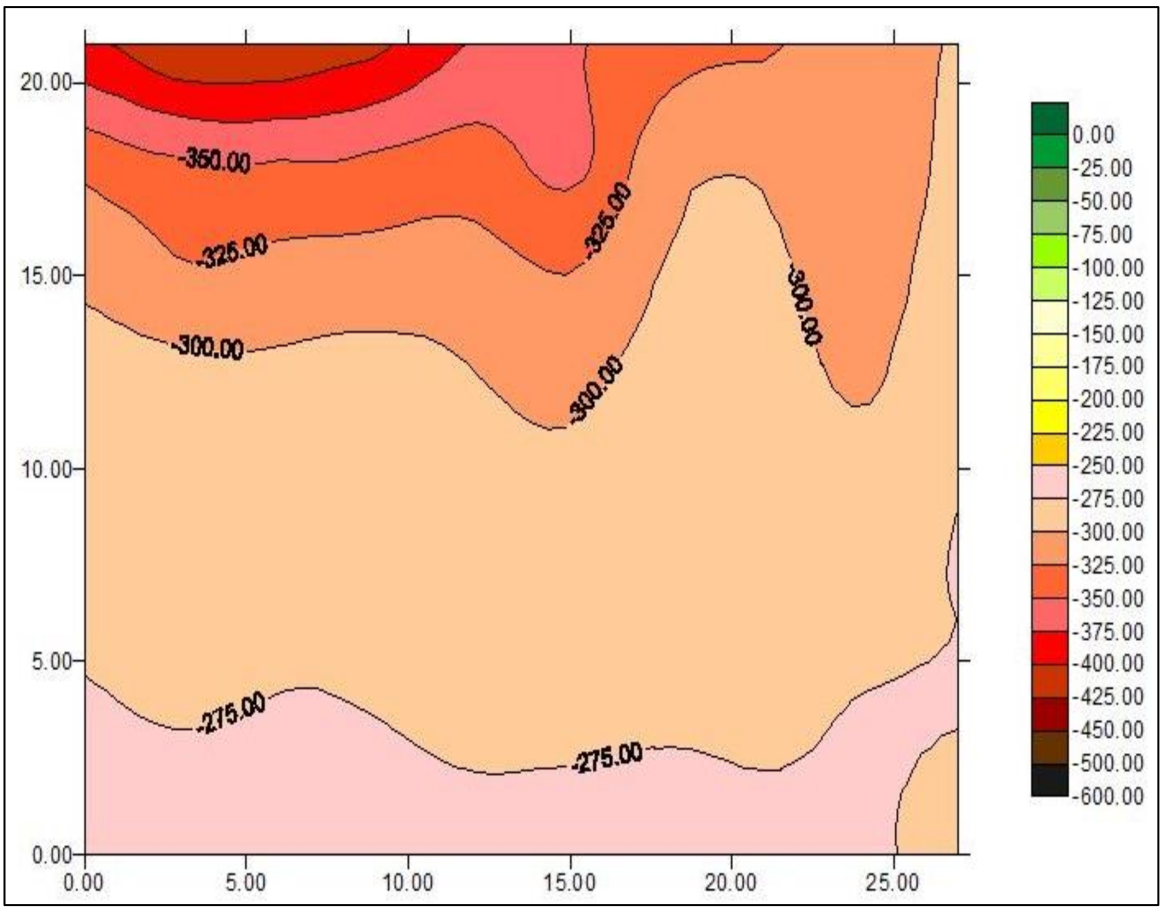


Figure E.164 Shaft 57, test 2 data map

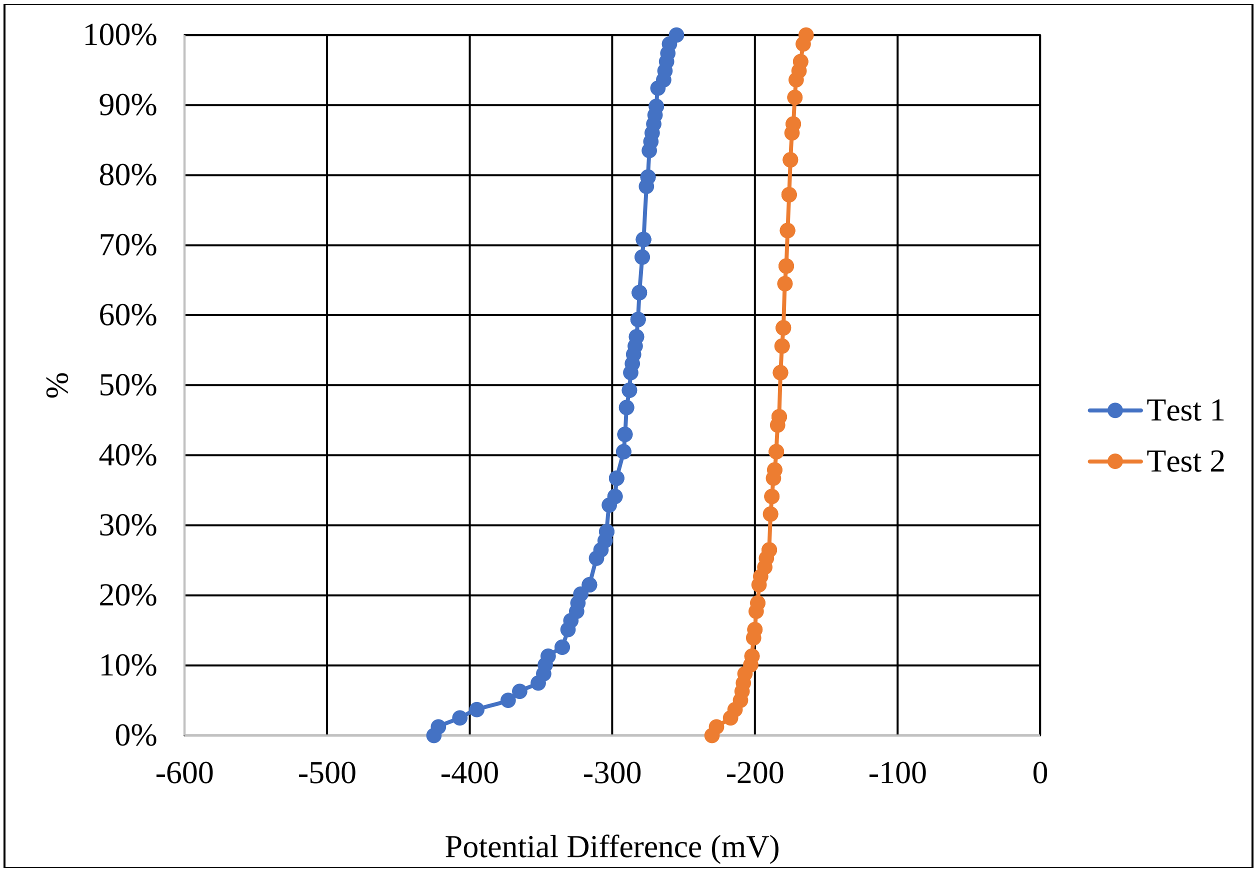


Figure E.165 Shaft 57 percentile distributions

Table E.111 Shaft 58, test 1 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-174	-174	-167	-169	-168	-167	-163	-162
3"	-167	-215	-208	-207	-210	-203	-204	-201
6"	-148	-200	-196	-196	-192	-192	-192	-193
9"	-122	-161	-193	-191	-192	-190	-186	-190
12"	-124	-140	-106	-104	-137	-127	-122	-190
15"	-121	-182	-104	-94	-129	-123	-123	-188
18"	-122	-206	-134	-110	-134	-127	-124	-185
21"	-117	-186	-183	-154	-183	-174	-172	-169
24"	-117	-191	-184	-188	-185	-185	-184	-194
27"	-117	-168	-162	-166	-163	-163	-162	-171

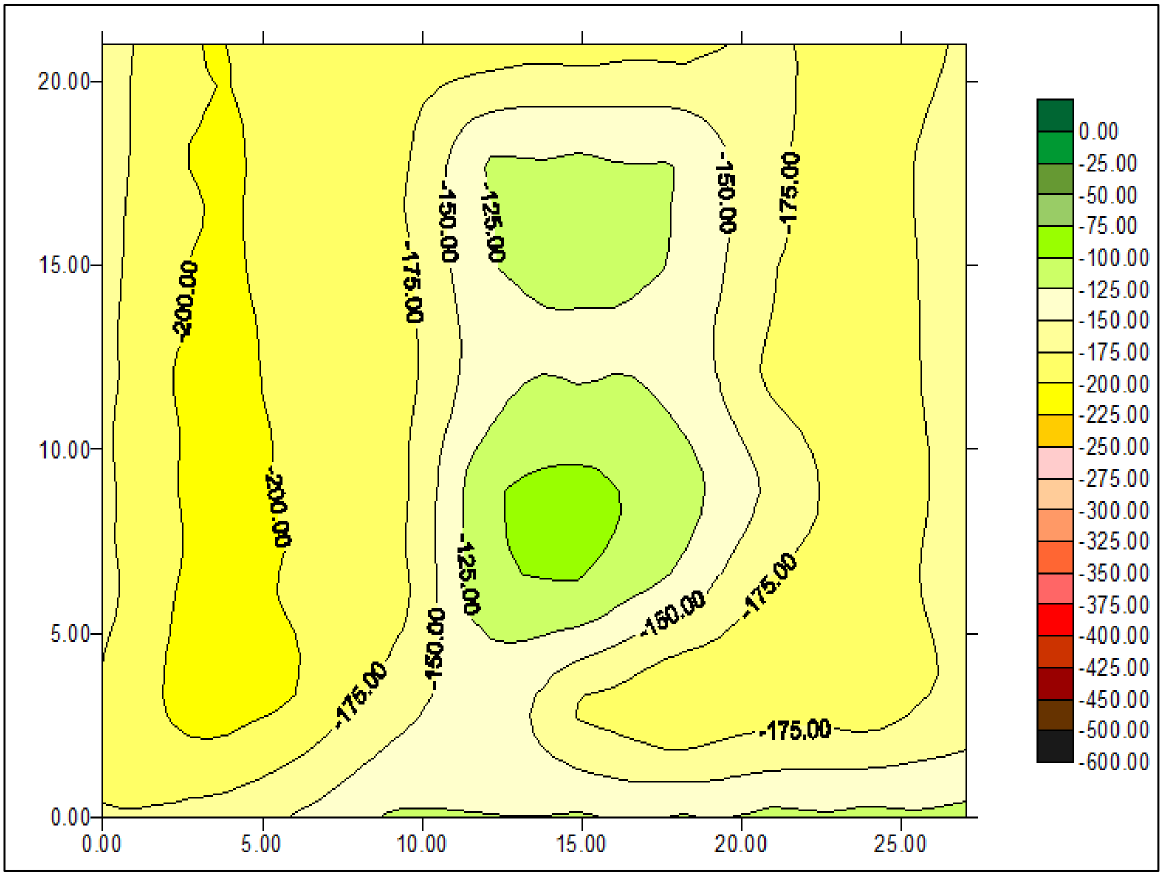


Figure E.166 Shaft 58, test 1 data map

Table E.112 Shaft 58, test 2 raw data (mV)

	1.5	4.5	7.5	10.5	13.5	16.5	19.5	22.5
0"	-230	-233	-224	-227	-225	-224	-220	-219
3"	-234	-233	-226	-225	-228	-221	-222	-219
6"	-223	-229	-225	-225	-221	-221	-221	-222
9"	-229	-229	-222	-220	-221	-219	-215	-222
12"	-232	-242	-195	-191	-237	-223	-217	-222
15"	-228	-247	-191	-178	-226	-218	-218	-220
18"	-229	-236	-233	-200	-233	-223	-220	-217
21"	-222	-225	-219	-220	-220	-222	-218	-218
24"	-222	-226	-218	-223	-220	-220	-218	-229
27"	-222	-222	-214	-216	-216	-219	-226	-221

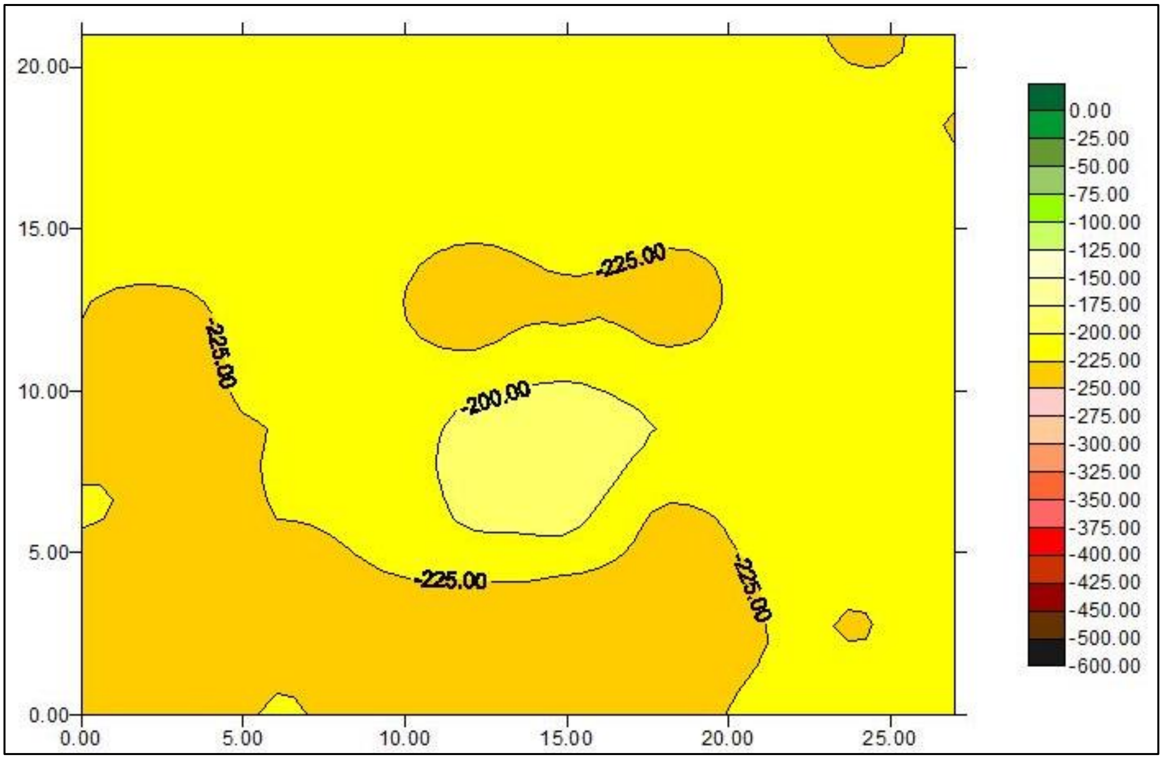


Figure E.167 Shaft 58, test 2 data map

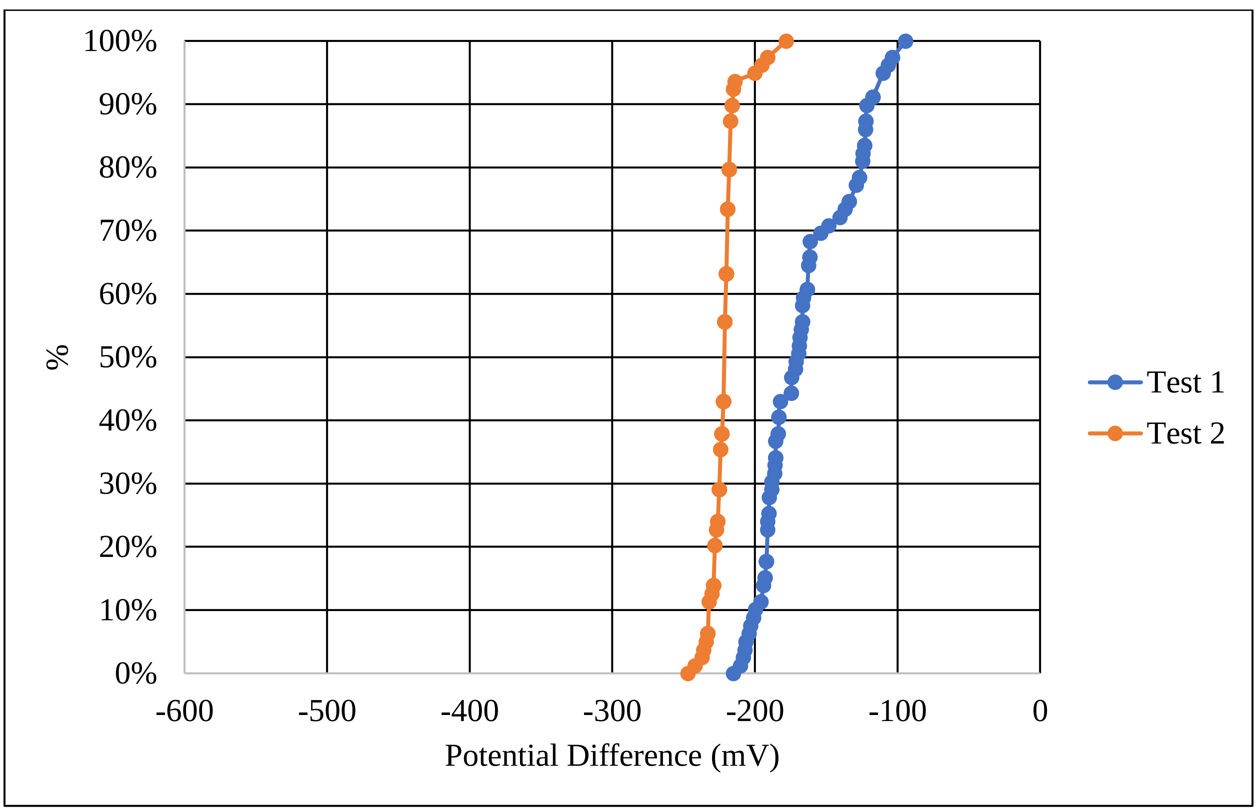


Figure E.168 Shaft 58 percentile distributions

APPENDIX F – CORING DATA TABLES AND PLOTS

Table F.1 Coring data summary

Shaft #	Support fluid	Viscosity	Core #	Location	Titan File #	Date	Average (psi)	Baseline (psi)
1	B	44	1	Crease	0086	6/22/2018	3860.6	
1	B	44	2	cover	0087	6/22/2018	3798.8	
1	B	44	3	cover	0088	6/22/2018	3484.1	
1	B	44	4	Interior	0091	6/22/2018	4341.9	
1	B	44	5	Interior	0092	6/22/2018	4687.9	4687.9
3	B	40	1	Crease	0137	6/29/2018	3710.4	
3	B	40	2	cover	0138	6/29/2018	4437.8	
3	B	40	3	cover	0139	6/29/2018	4453.8	
3	B	40	4	Interior	0140	6/29/2018	4296	
3	B	40	5	Interior	0141	6/29/2018	4282.6	4296
4	B	55	1	Crease	0131	6/29/2018	4506.35	
4	B	55	2	cover	0133	6/29/2018	4601.01	
4	B	55	3	cover	0134	6/29/2018	4318.4	
4	B	55	4	Interior	0135	6/29/2018	4228.6	
4	B	55	5	Interior	0136	6/29/2018	4268	4268
5	B	90	1	Crease	0052	6/20/2018	3043.1	
5	B	90	2	cover	0053	6/20/2018	4711.24	
5	B	90	3	cover	0054	6/20/2018	4134.04	
5	B	90	4	Interior	0055	6/20/2018	4312.12	
5	B	90	5	Interior	0056	6/20/2018	5022.7	
5	B	90	6	Interior	0057	6/20/2018	4640	5022.7
6	W	26	1	Crease	0025	6/12/2018	4048.9	
6	W	26	2	cover	0028	6/12/2018	4144	
6	W	26	3	cover	0029	6/12/2018	4064	
6	W	26	4	Interior	0030	6/12/2018	4383.24	4752
6	W	26	5	Interior	0031	6/12/2018	4752	
7	B	30	1	Cover	0042	5/21/2018	2914.96	
7	B	30	2	Cover	0043	5/21/2018	3796.7	
7	B	30	3	Crease	0044	5/21/2018	2901.2	
7	B	30	4	Interior	0045	5/21/2018	4068.9	

Table F.1 (Continued)

Shaft #	Support fluid	Viscosity	Core #	Location	Titan File #	Date	Average (psi)	Baseline (psi)
7	B	30	5	Interior	0052	5/21/2018	4303.5	**4303.5**
8	B	40	1	Crease	0079	6/22/2018	4008.7	
8	B	40	2	cover	0082	6/22/2018	4124.5	
8	B	40	3	cover	0083	6/22/2018	4138.5	
8	B	40	4	Interior	0084	6/22/2018	4532.3	
9	B	57	5	Interior	0036	6/12/2018	5627	
10	B	90	1	Interior	0053	6/8/2018	3680	
10	B	90	2	Interior	0054	6/8/2018	5142	**5142**
10	B	90	3	Interior	0055	6/8/2018	5142	
10	B	90	4	cover	0056	6/8/2018	4110	
10	B	90	5	cover	0057	6/8/2018	3893	
10	B	90	6	crease	0063	6/8/2018	3961	
11	P	65	1	Crease	0018	6/12/2018	4372.5	
11	P	65	2	cover	0019	6/12/2018	5165	
11	P	65	3	cover	0021	6/12/2018	4463	
11	P	65	4	Interior	0022	6/12/2018	4141.9	**4220**
11	P	65	5	Interior	0023	6/12/2018	4220	
12	P	66	1	Crease	0060	6/20/2018	4589.3	
12	P	66	2	cover	0062	6/20/2018	5747.5	
12	P	66	3	cover	0063	6/20/2018	5750	
12	P	66	4	Interior	0064	6/20/2018	5976.3	
12	P	66	5	Interior	0066	6/20/2018	5400.7	**5976.3**
13	B	30	1	Crease	0045	6/18/2018	3546.4	
13	B	30	2	cover	0046	6/18/2018	3575.14	
13	B	30	3	cover	0049	6/18/2018	3958.43	
13	B	30	4	Interior	0050	6/18/2018	3777.56	
13	B	30	5	Interior	0051	6/18/2018	3814.67	**3796.12**
14	B	30	1	Crease	0073	6/22/2018	3084.25	
14	B	30	2	cover	0075	6/22/2018	4210.9	
14	B	30	3	cover	0076	6/22/2018	4048.7	
14	B	30	4	Interior	0077	6/22/2018	3986.6	
14	B	30	5	Interior	0078	6/22/2018	4511.9	**4511.9**
15	B	56	1	Crease	0080	6/9/2018	4970.2	
15	B	56	2	cover	0081	6/9/2018	5293.9	
15	B	56	3	cover	0083	6/9/2018	4959	
15	B	56	4	cover	0084	6/9/2018	4255	

Table F.1 (Continued)

Shaft #	Support fluid	Viscosity	Core #	Location	Titan File #	Date	Average (psi)	Baseline (psi)
15	B	56	5	Interior	0085	6/9/2018	4742	**4742**
15	B	56	6	Interior	0086	6/9/2018	4682.7	
16	P	85	1	Crease	0142	6/29/2018	3521.4	
16	P	85	2	cover	0143	6/29/2018	3603.9	
16	P	85	3	cover	0144	6/29/2018	3802.9	
17	P	85	4	Interior	0061	6/8/2018	3948.5	**4345.25**
17	P	85	5	Interior	0062	6/8/2018	4742	
18	W	26	1	Crease	0125	6/29/2018	3314.6	
18	W	26	2	cover	0126	6/29/2018	4423	
18	W	26	3	cover	0127	6/29/2018	4417.8	
18	W	26	4	Interior	0128	6/29/2018	3813.13	
18	W	26	5	Interior	0130	6/29/2018	3957.3	**3957.3**
19	P	63	1	Crease	0073	6/9/2018	4822	
19	P	63	2	cover	0076	6/9/2018	4957.2	
19	P	63	3	cover	0077	6/9/2018	5568	
19	P	63	4	Interior	0078	6/9/2018	5733	**5739**
19	P	63	5	Interior	0079	6/9/2018	5739	
20	P	121	1	Crease	0117	6/27/2018	3569.6	
20	P	121	2	cover	0118	6/27/2018	4772.9	
20	P	121	3	cover	0119	6/27/2018	4546.1	
20	P	121	4	Interior	0120	6/27/2018	4719.9	
20	P	121	5	Interior	0121	6/27/2018	4395.6	**4719.9**
21	B	42	1	Crease	0064	6/9/2018	3715.3	
21	B	42	2	cover	0066	6/9/2018	4146.04	
21	B	42	3	cover	0067	6/9/2018	4846.4	
21	B	42	4	Interior	0070	6/9/2018	4468	
21	B	42	5	Interior	0072	6/9/2018	5092.17	**5092.17**
22	W	26	1	Crease	0001	6/11/2018	3126	
22	W	26	2	cover	0002	6/11/2018	4701	
22	W	26	3	cover	0003	6/11/2018	4737	
22	W	26	4	Interior	0005	6/11/2018	4597.6	
22	W	26	5	Interior	0008	6/11/2018	4029.4	**4597.6**
23	W	26	1	Crease	0067	6/20/2018	4637.5	
23	W	26	2	cover	0068	6/20/2018	5020.4	
23	W	26	3	cover	0070	6/20/2018	3511	
23	W	26	4	Interior	0071	6/20/2018	3373	

Table F.1 (Continued)

Shaft #	Support fluid	Viscosity	Core #	Location	Titan File #	Date	Average (psi)	Baseline (psi)
23	W	Viscosity	5	Interior	0072	Date	3449	**3449**
24	B	40	1	Crease	0009	6/12/2018	2802.4	
24	B	40	2	cover	0010	6/12/2018	3352.2	
24	B	40	3	cover	0013	6/12/2018	2386.9	
24	B	40	4	Interior	0016	6/12/2018	3210.7	
24	B	40	5	Interior	0017	6/12/2018	3295.8	**3295.8**
26	W	26	1	Crease	0040	6/18/2018	3542.9	
26	W	26	2	cover	0041	6/18/2018	3544.8	
26	W	26	3	cover	0042	6/18/2018	3434.7	
26	W	26	4	Interior	0043	6/18/2018	3357.08	
26	W	26	5	Interior	0044	6/18/2018	3610.9	**3610.9**
32	W	26	1	Crease	0036	6/18/2018	5703	
32	W	26	2	Cover	0037	6/18/2018	3525	
32	W	26	3	Cover	0038	6/18/2018	4041.3	
32	W	26	5	Interior	0058	6/19/2018	4956.7	
32	W	26	6	Interior	0059	6/20/2018	4422.7	**4956.7**
45	B	37	1	Crease	0104	6/26/2018	4046.97	
45	B	37	2	Cover	0106	6/26/2018	4069.7	
45	B	37	3	Cover	0109	6/26/2018	4329.6	
45	B	37	4	Interior	0110	6/26/2018	4847.1	
45	B	37	5	Interior	0111	6/26/2018	4789.5	**4847.1**
46	B	37	1	Crease	0112	6/26/2018	4584.1	
46	B	37	2	Cover	0113	6/26/2018	4529.5	
46	B	37	3	Cover	0114	6/26/2018	5372.34	
46	B	37	4	Interior	0115	6/26/2018	4730	
46	B	37	5	Interior	0116	6/26/2018	5341.3	**5341.3**
50	P	57	1	Crease	0099	6/26/2018	4000.8	
50	P	90	2	Cover	0100	6/26/2018	3711.8	
50	P	90	3	Cover	0101	6/26/2018	3667.1	
50	P	90	4	Interior	0102	6/26/2018	3845.23	
50	P	90	5	Interior	0103	6/26/2018	3682	**3845.23**
51	P	90	1	Crease	0093	6/26/2018	2488.9	
51	P	90	2	Cover	0094	6/26/2018	3440.12	
51	P	90	3	Cover	0095	6/26/2018	3417.4	
51	P	90	4	Interior	0096	6/26/2018	3733.8	
51	P	90	5	Interior	0097	6/26/2018	4008.9	**4008.9**

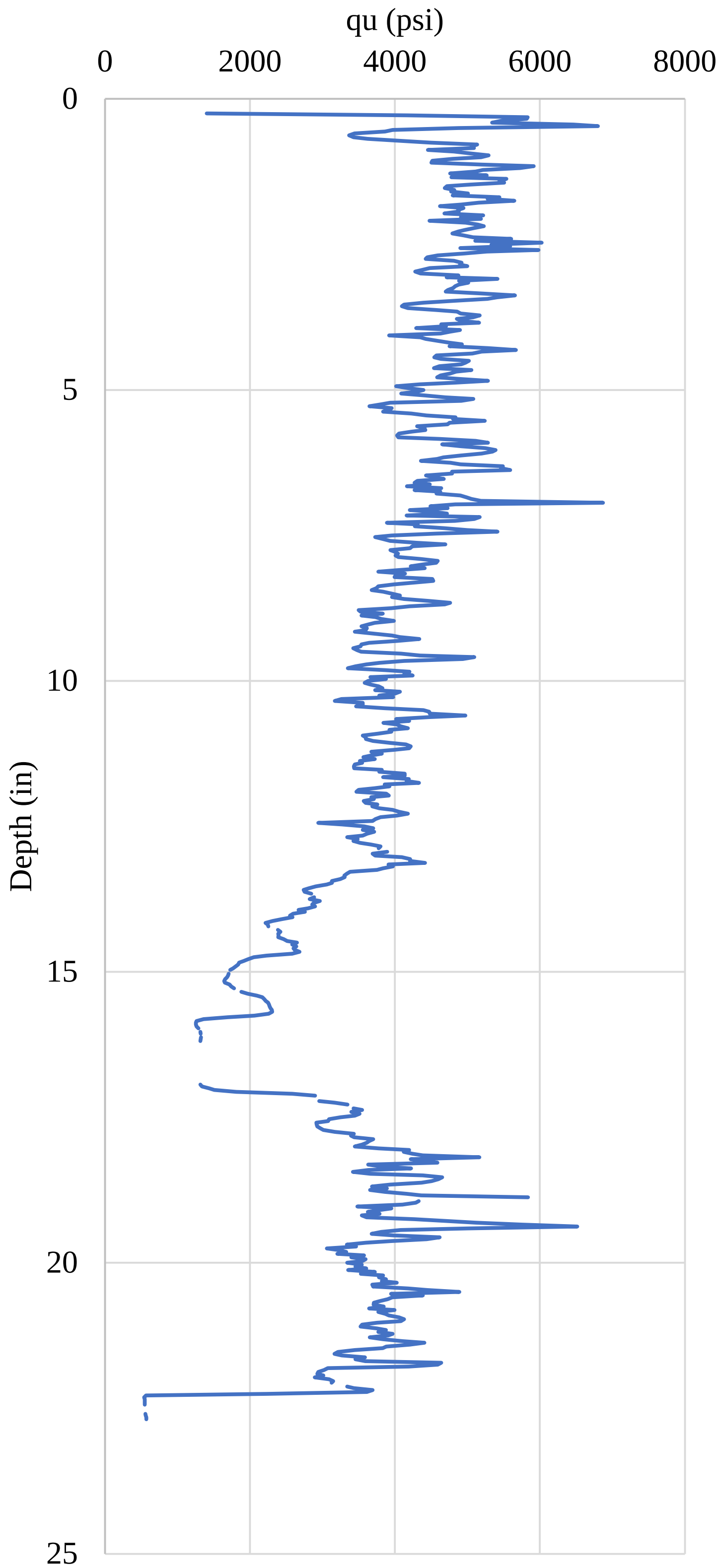


Figure F.1 Core 1-1

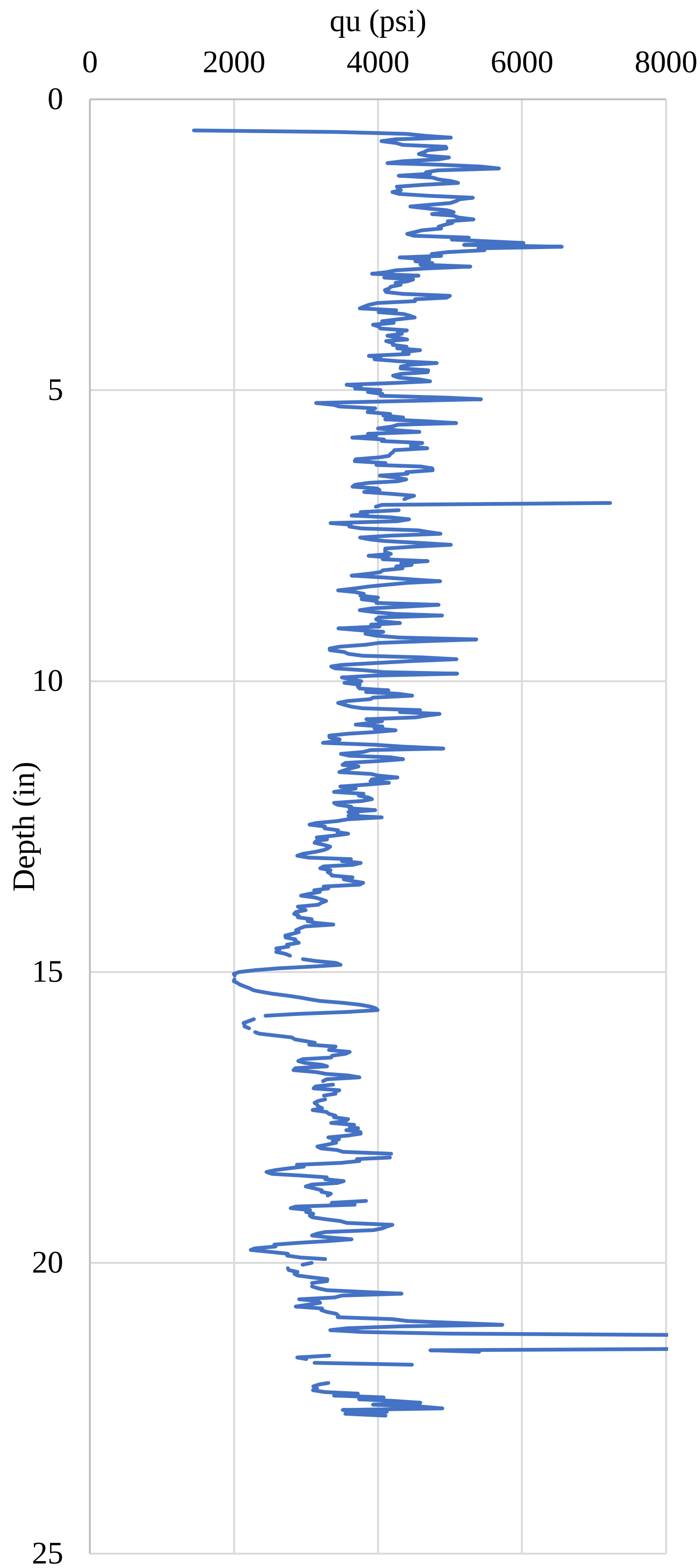


Figure F.2 Core 1-2

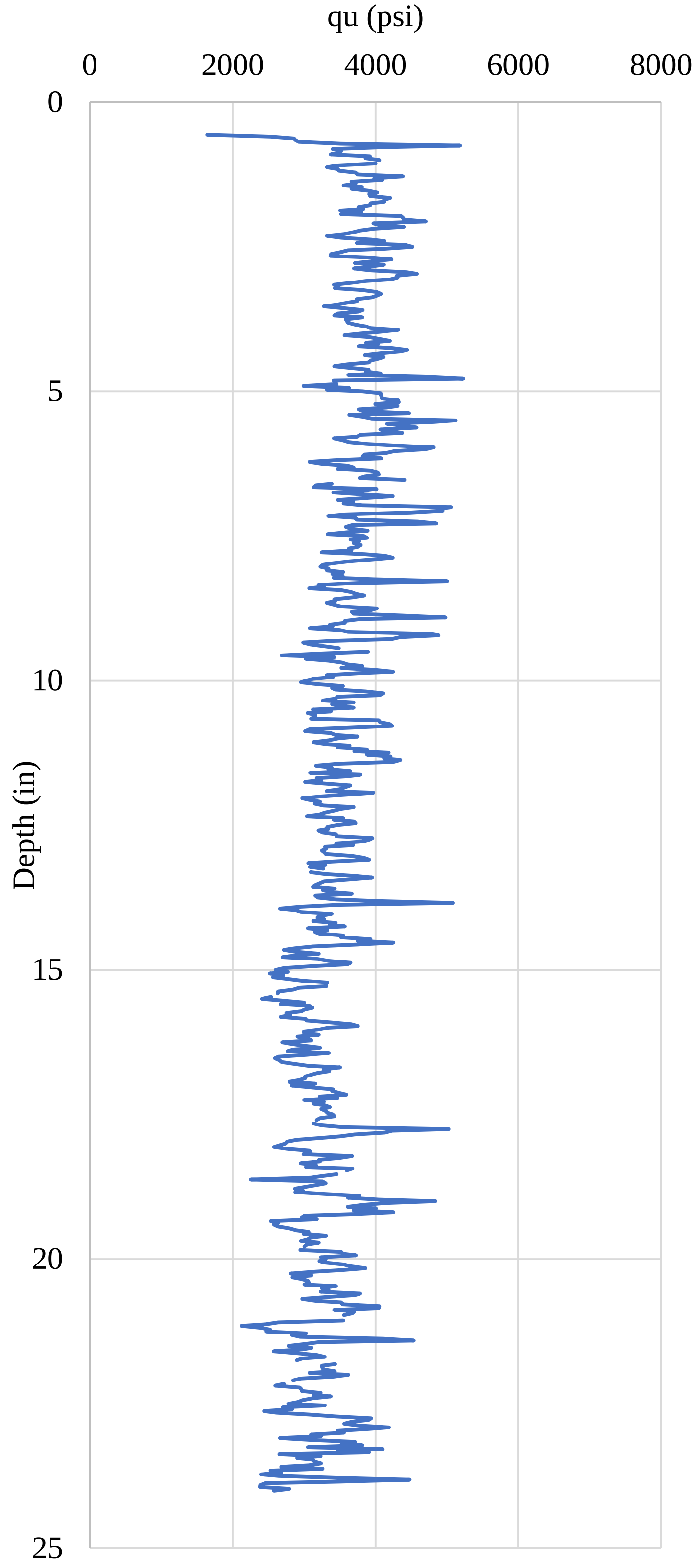


Figure F.3 Core 1-3

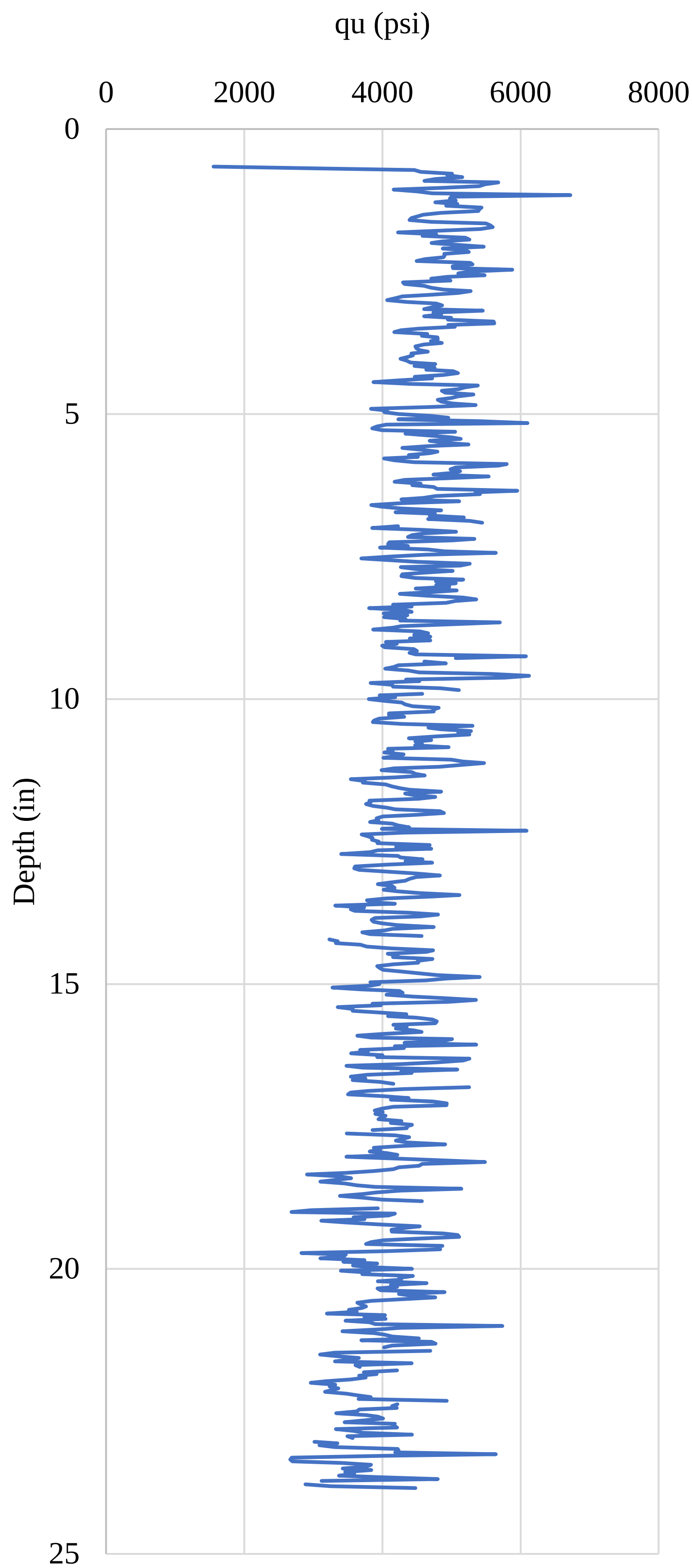


Figure F.4 Core 1-4

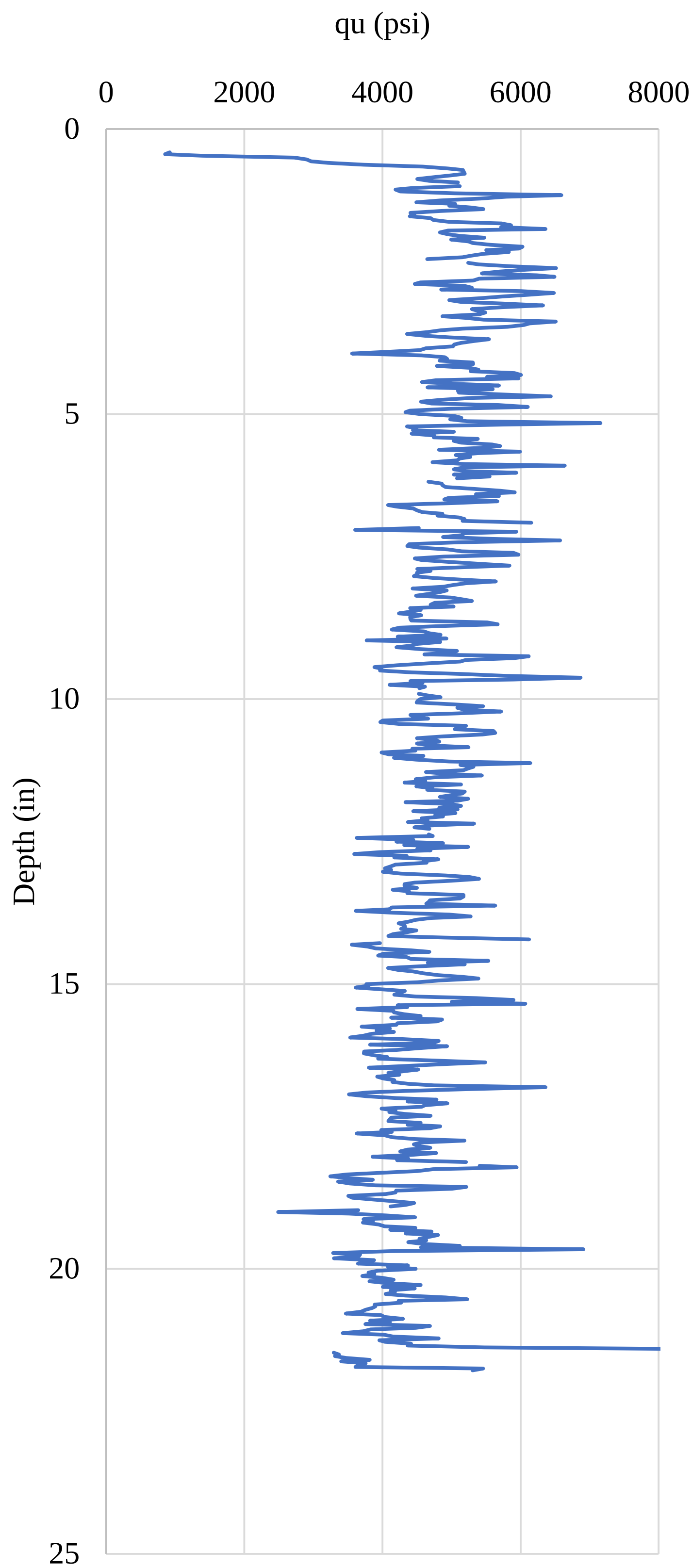


Figure F.5 Core 1-5

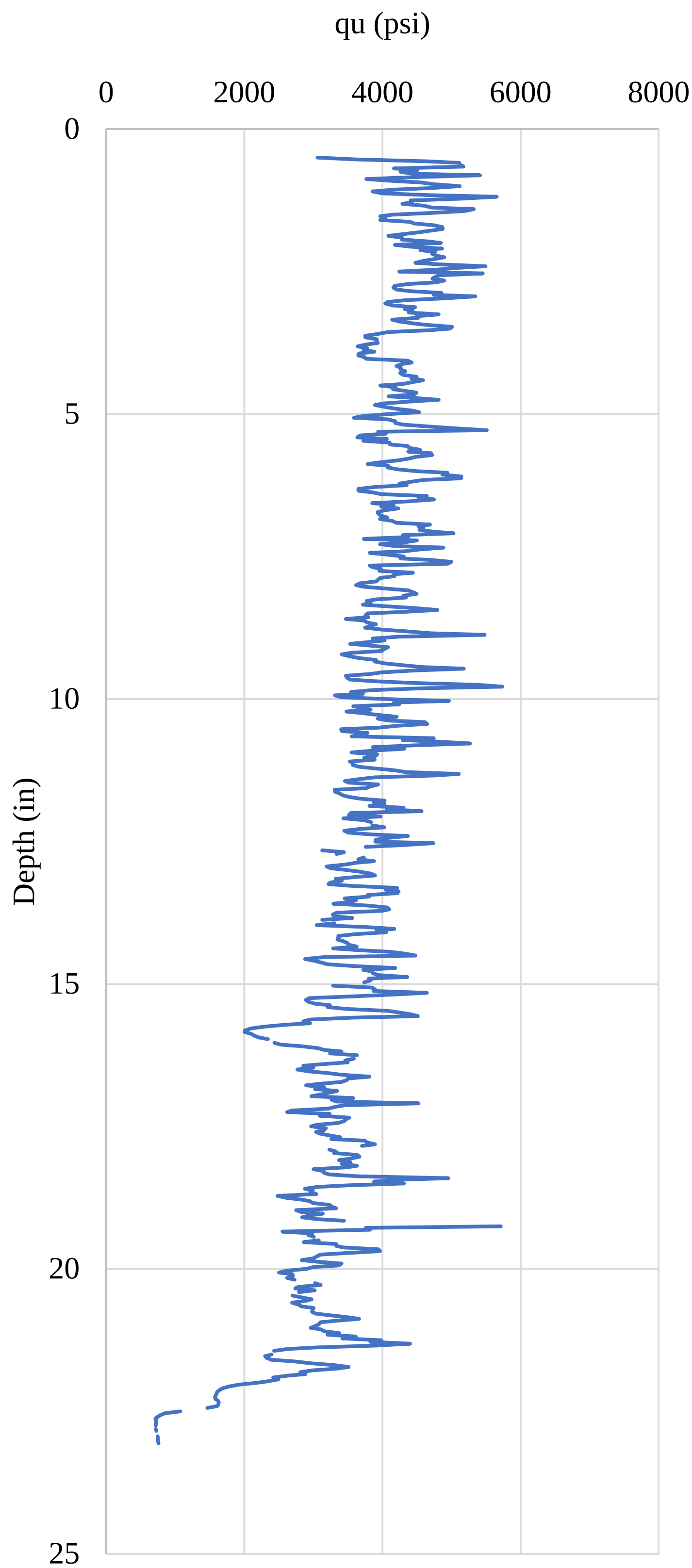


Figure F.6 Core 3-1

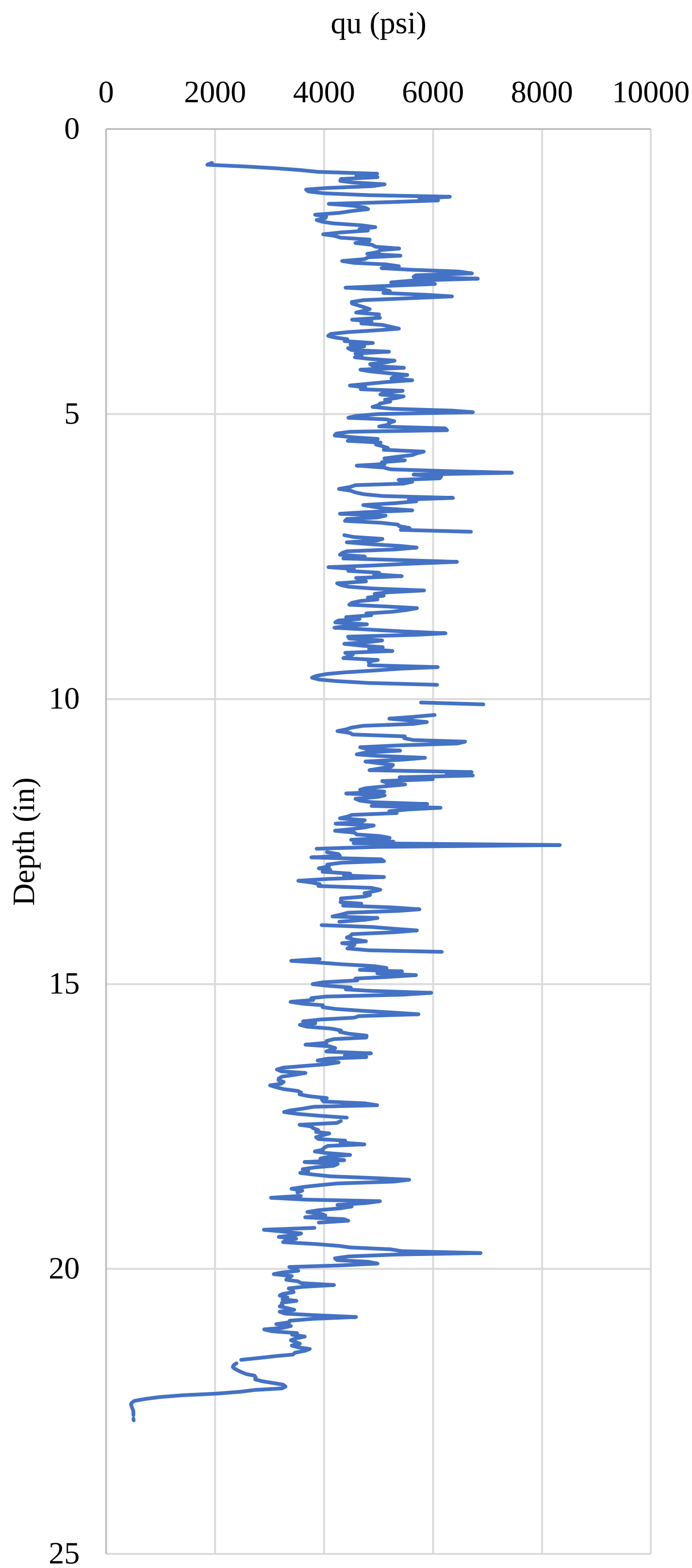


Figure F.7 Core 3-2

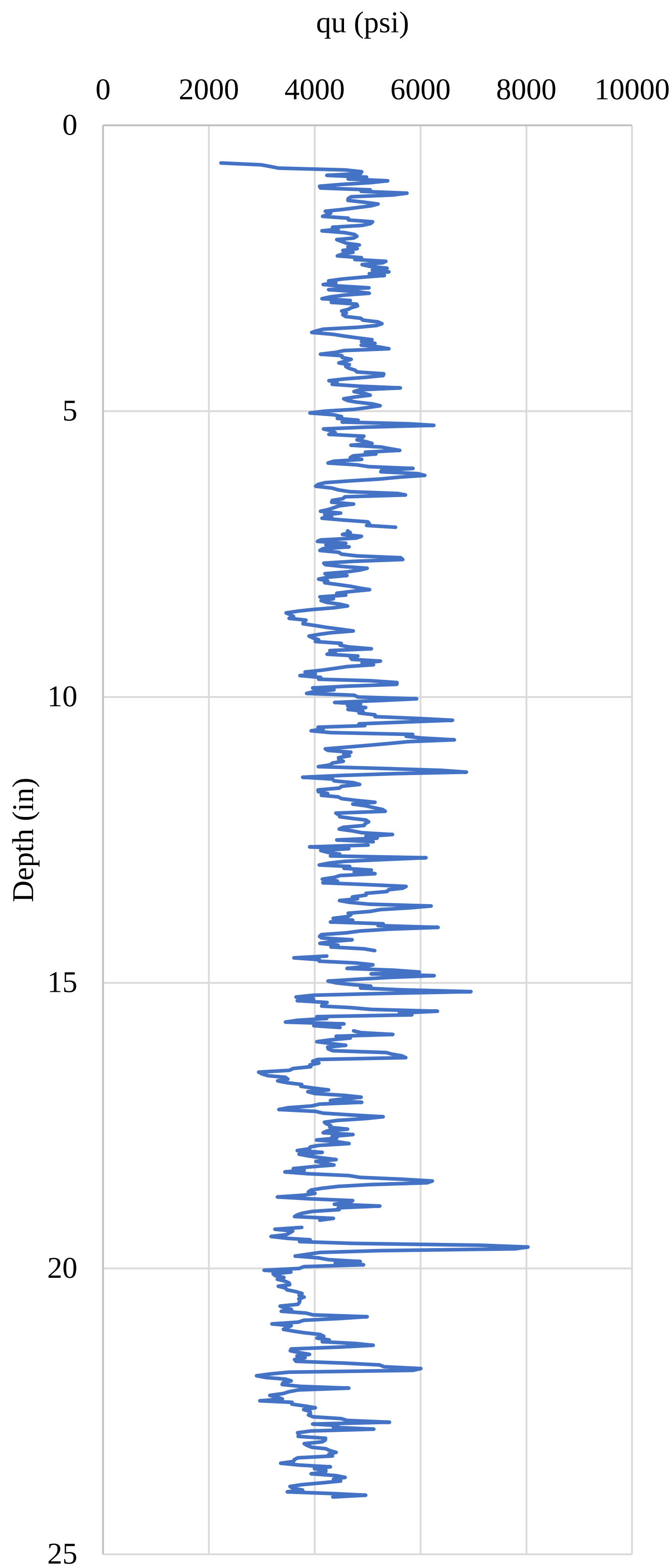


Figure F.8 Core 3-3

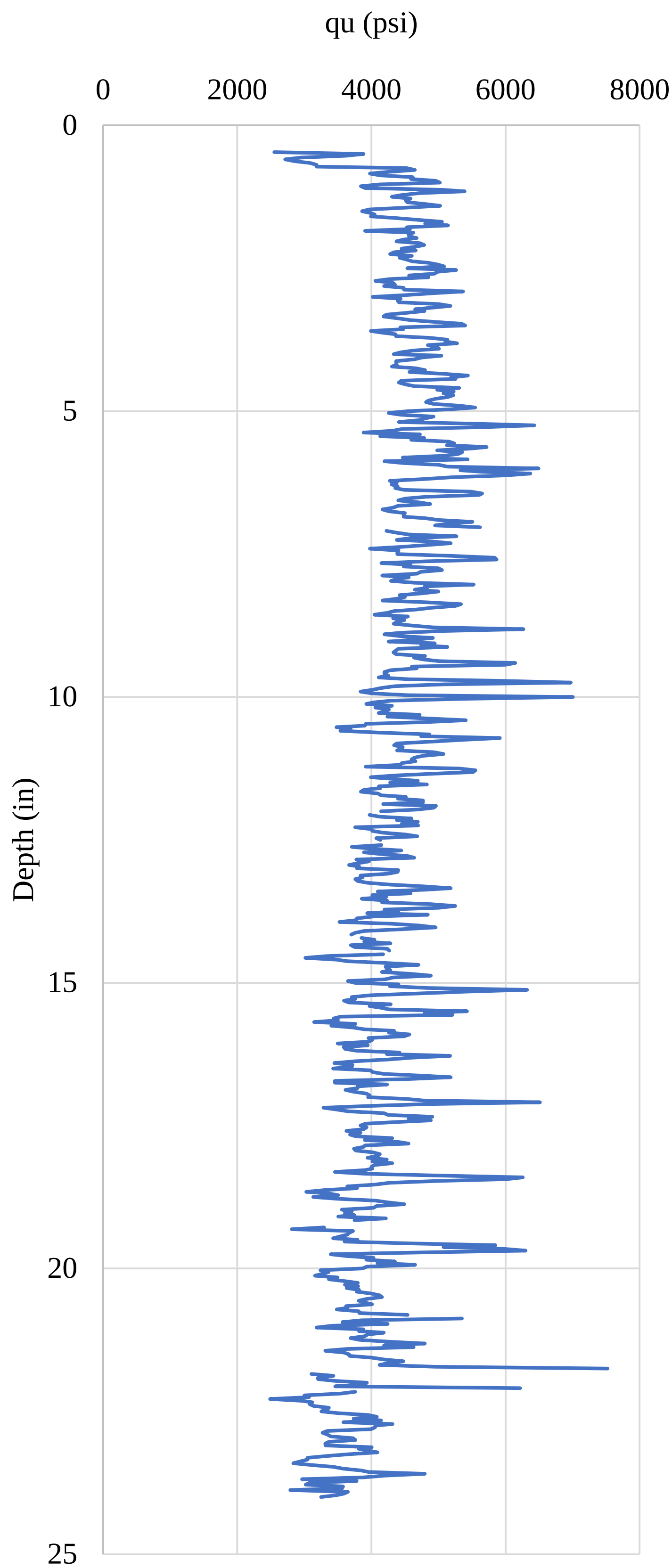


Figure F.9 Core 3-4

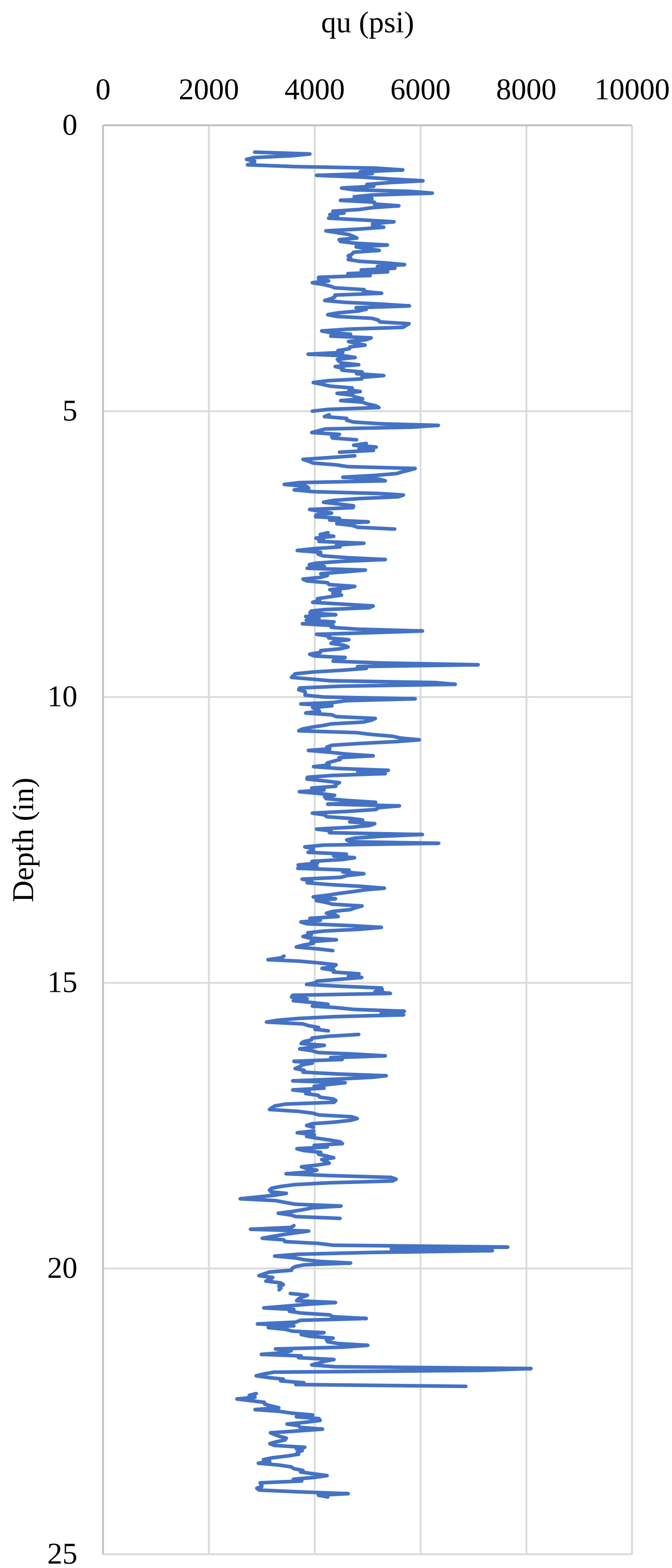


Figure F.10 Core 3-5

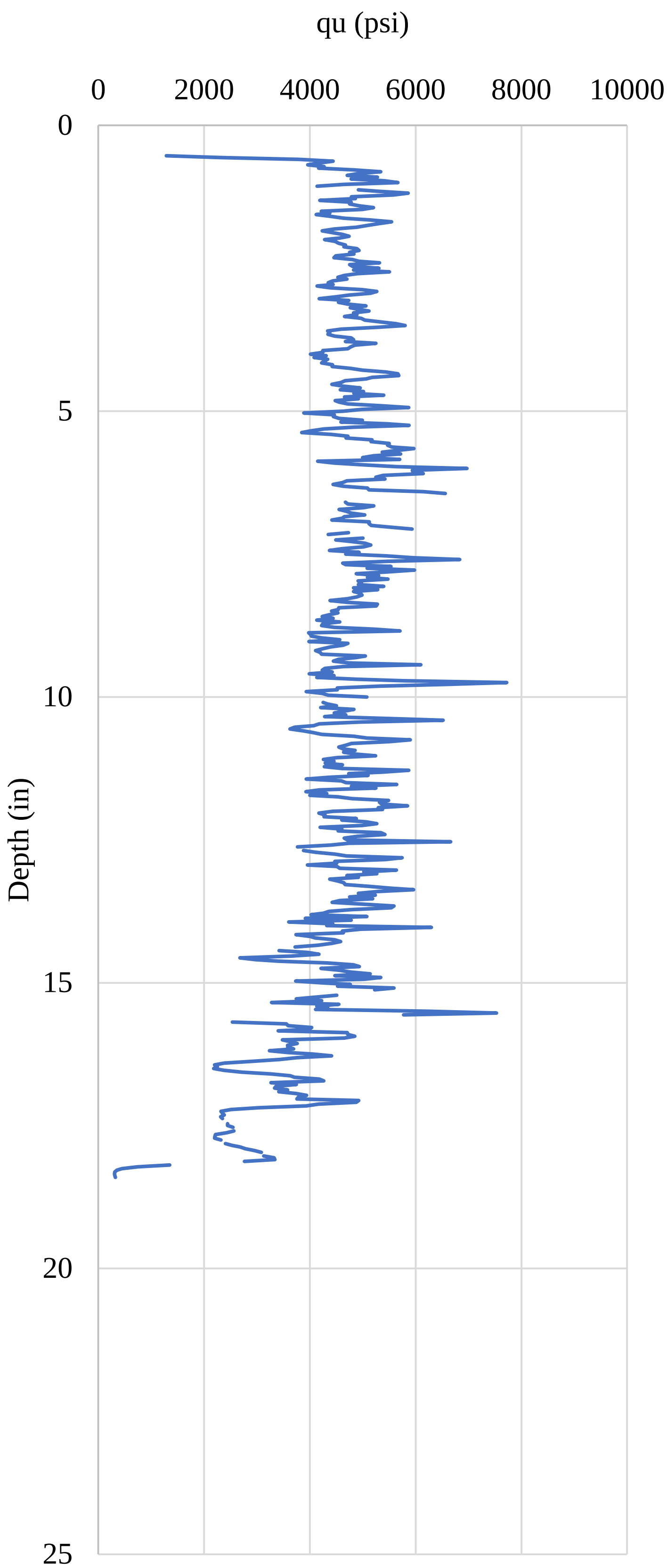


Figure F.11 Core 4-1

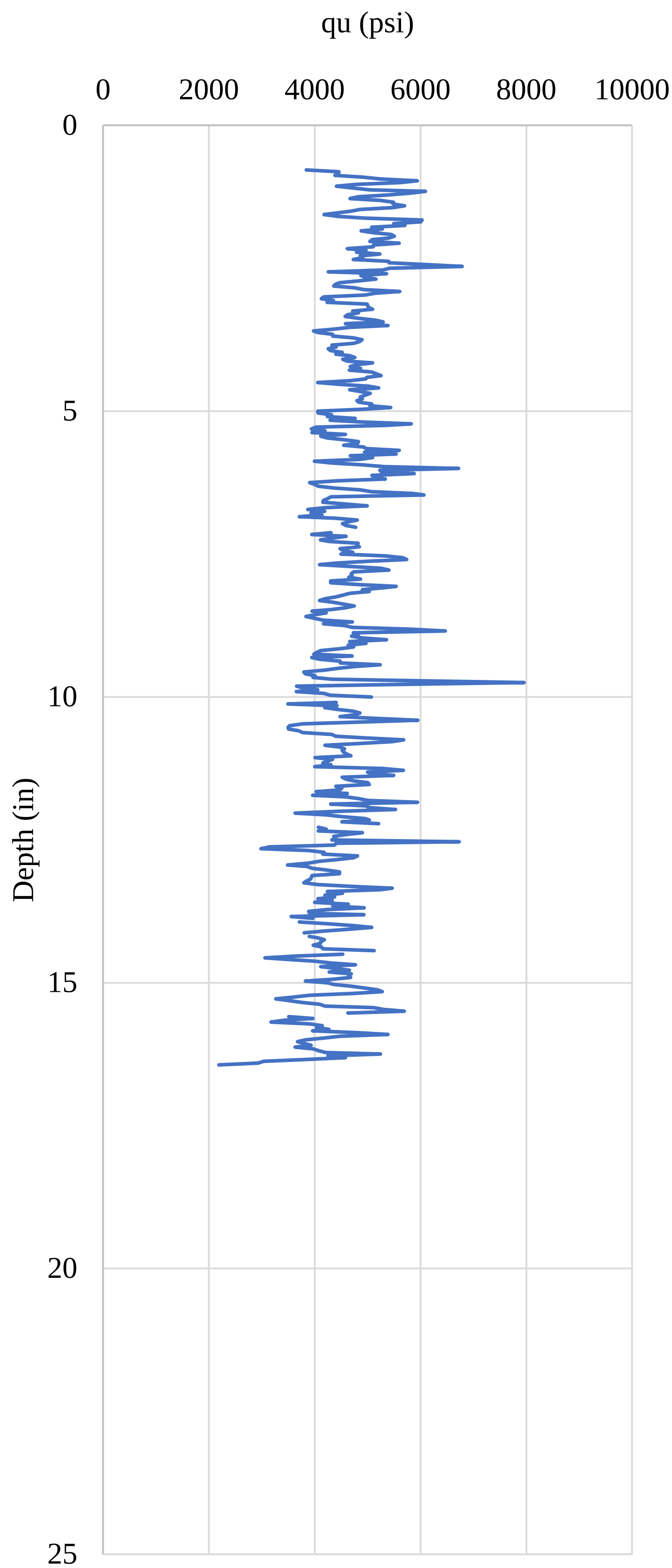


Figure F.12 Core 4-2

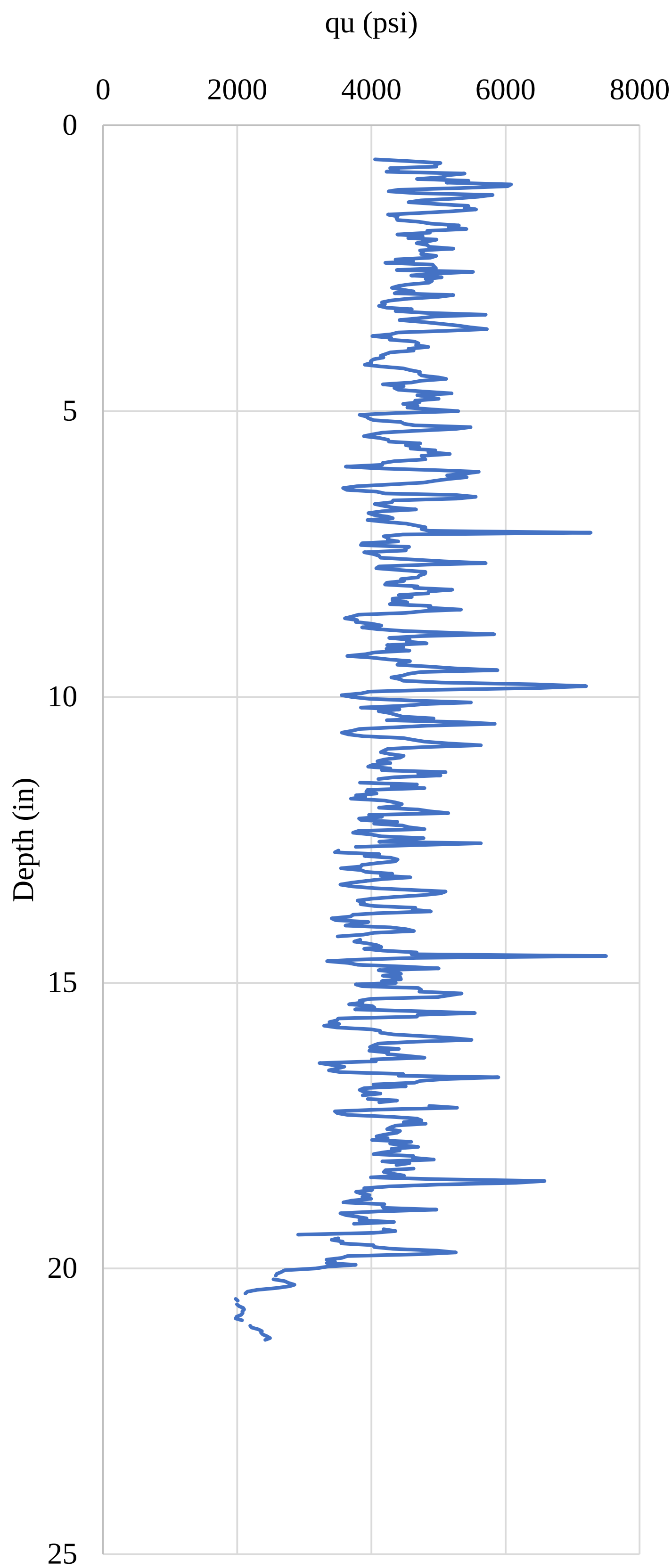


Figure F.13 Core 4-3

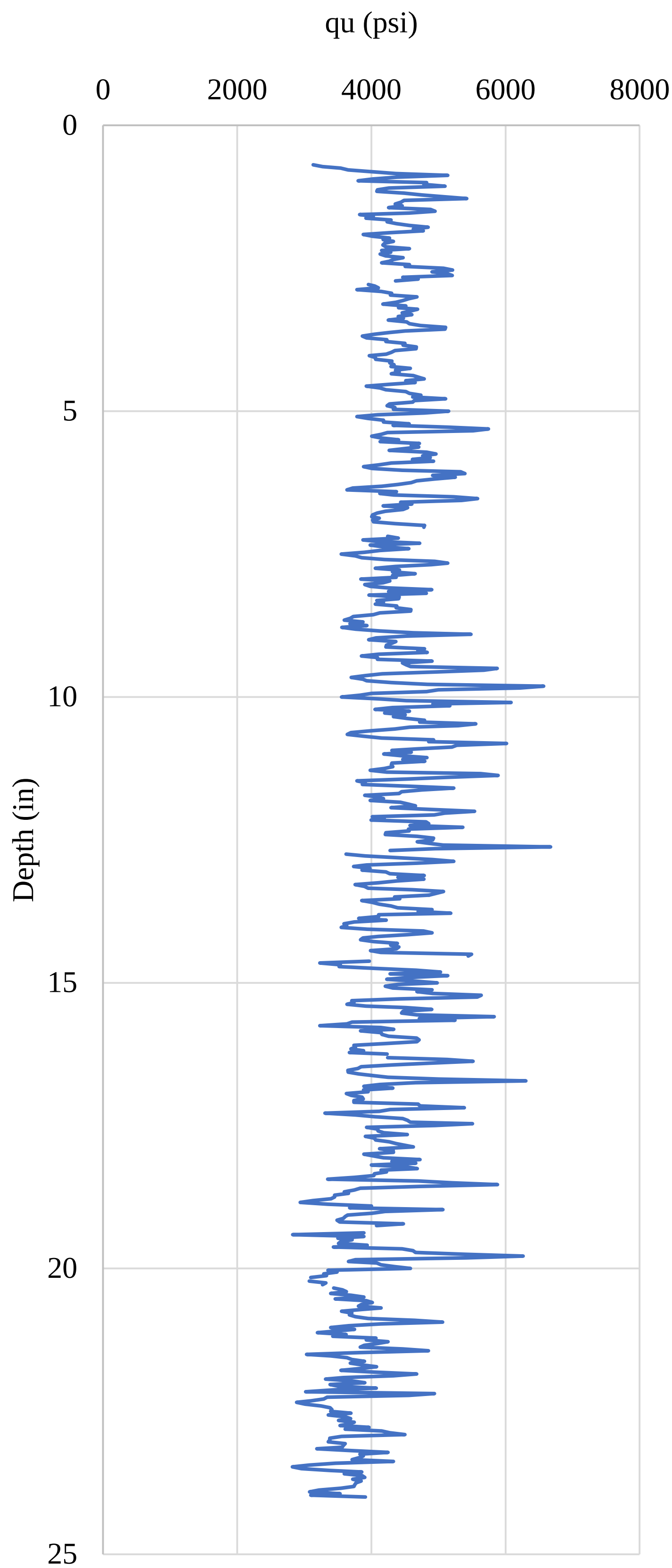


Figure F.14 Core 4-4

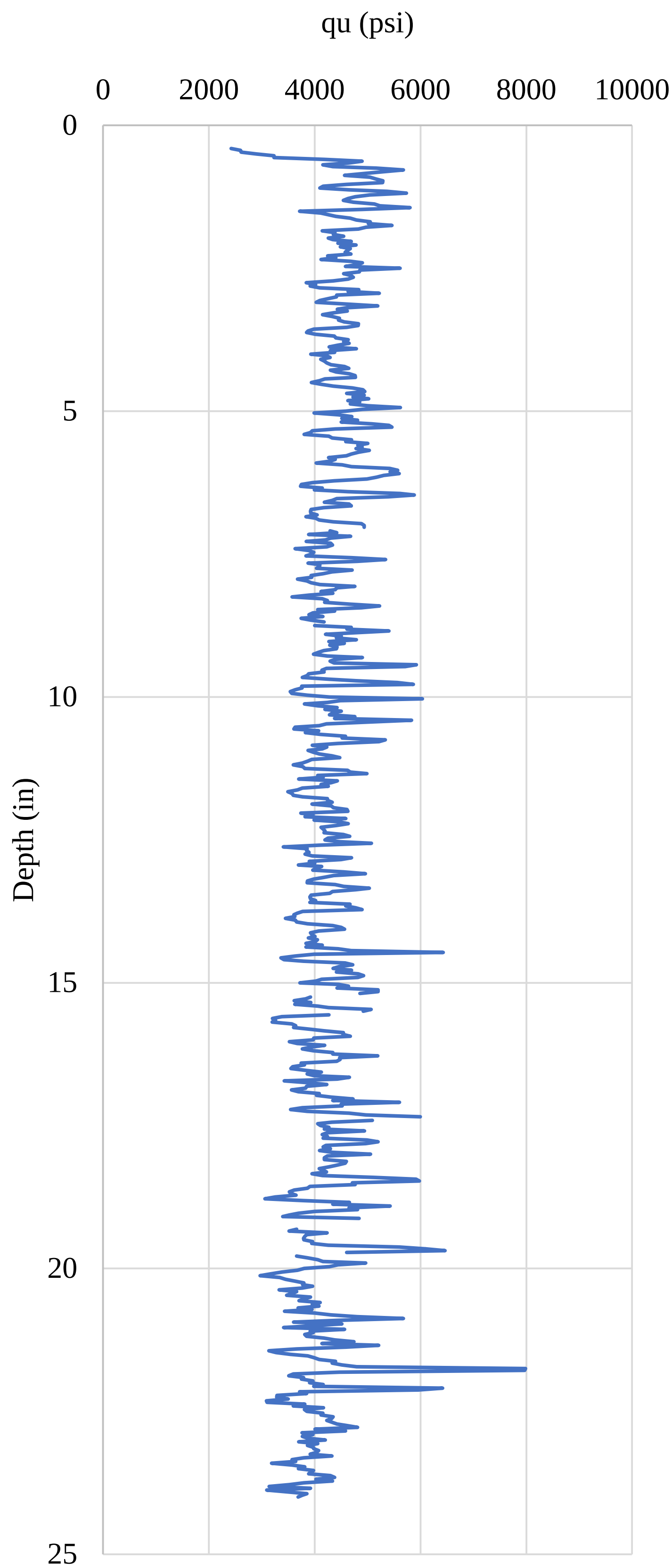


Figure F.15 Core 4-5

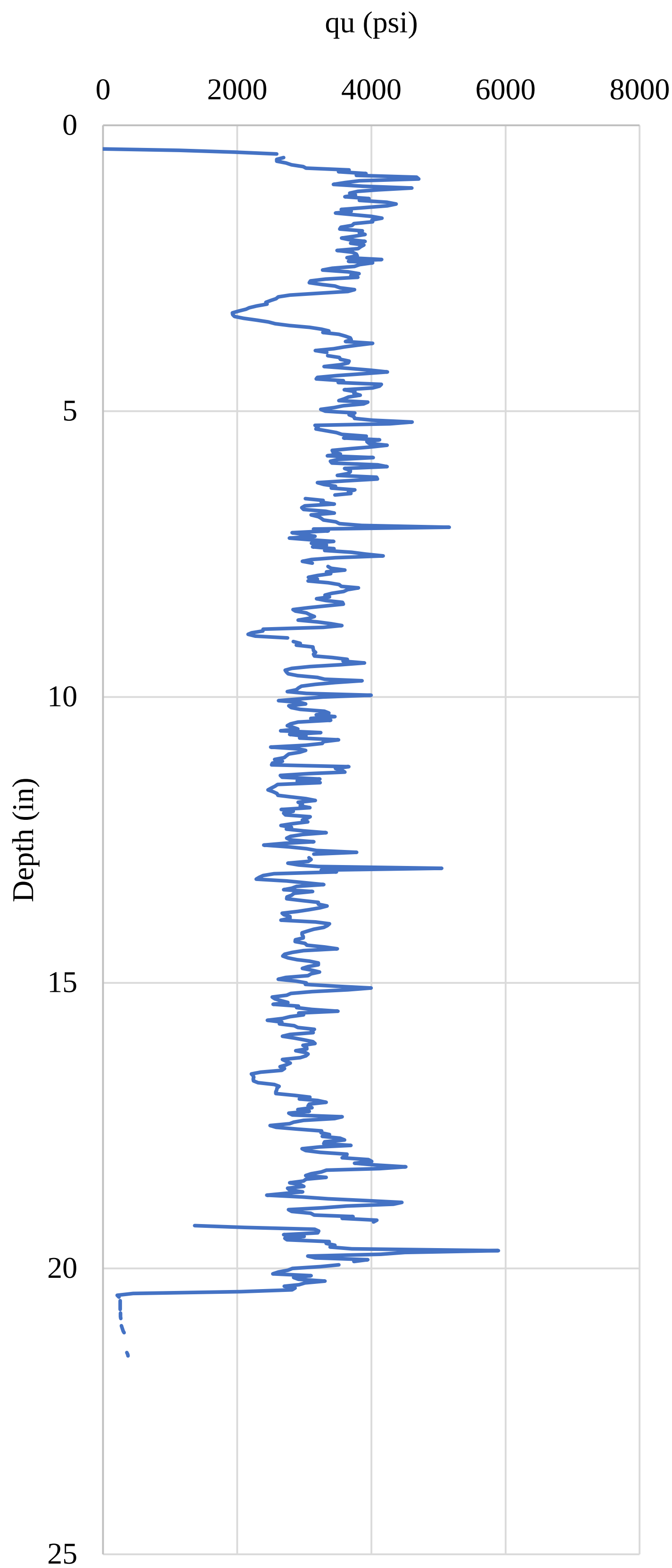


Figure F.16 Core 5-1

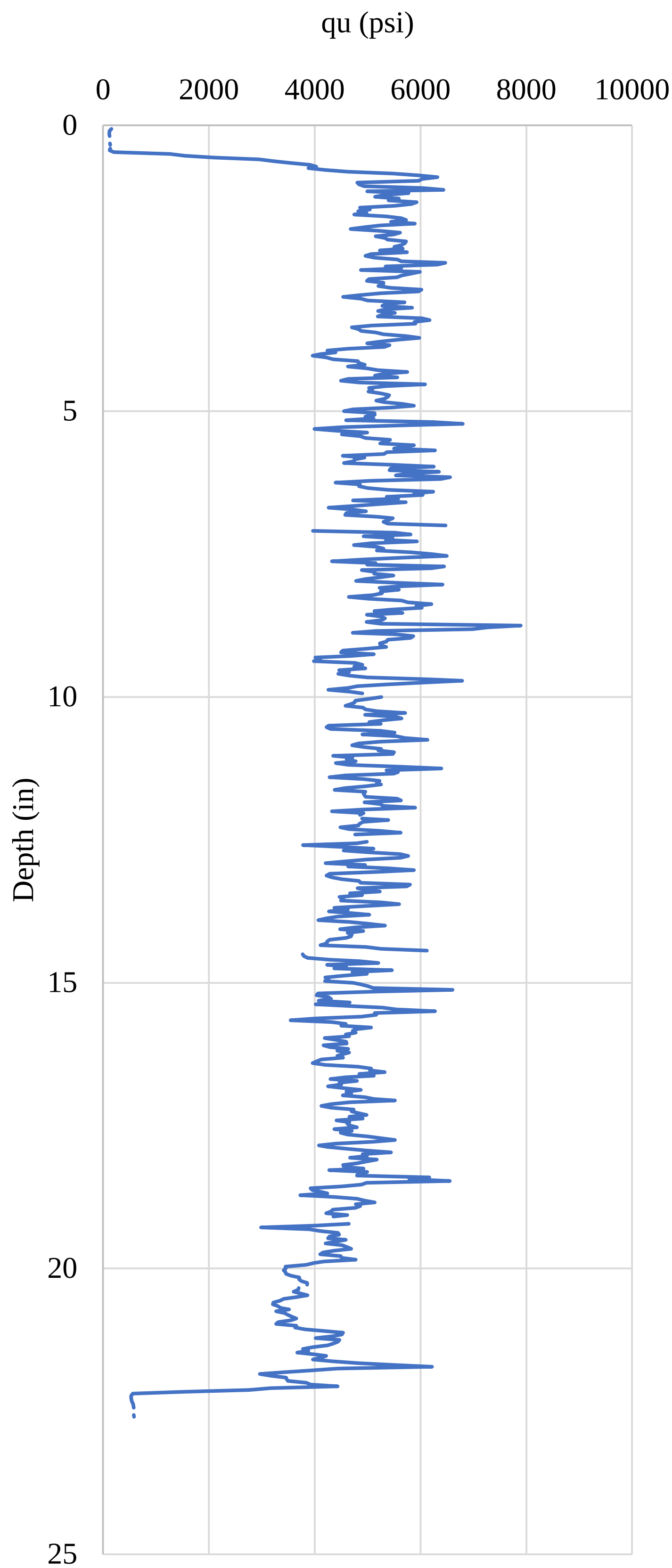


Figure F.17 Core 5-2

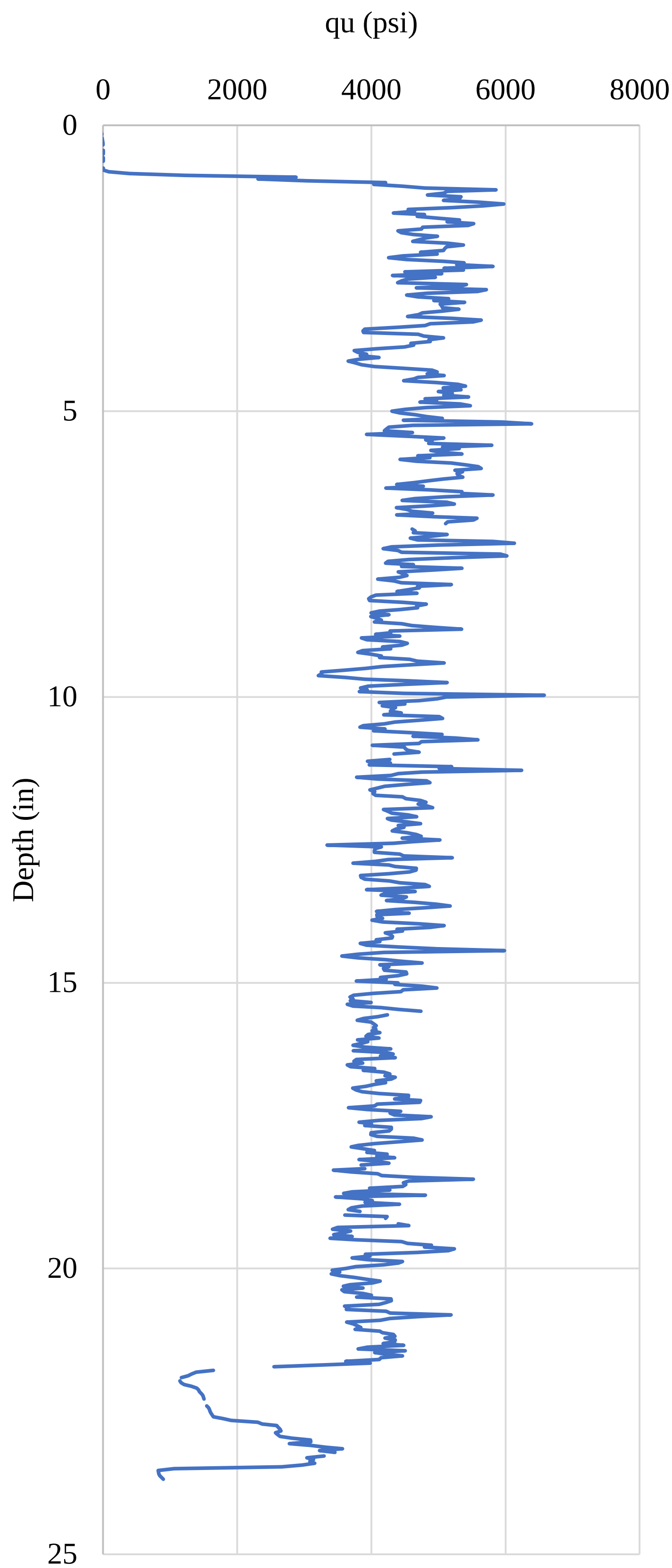


Figure F.18 Core 5-3

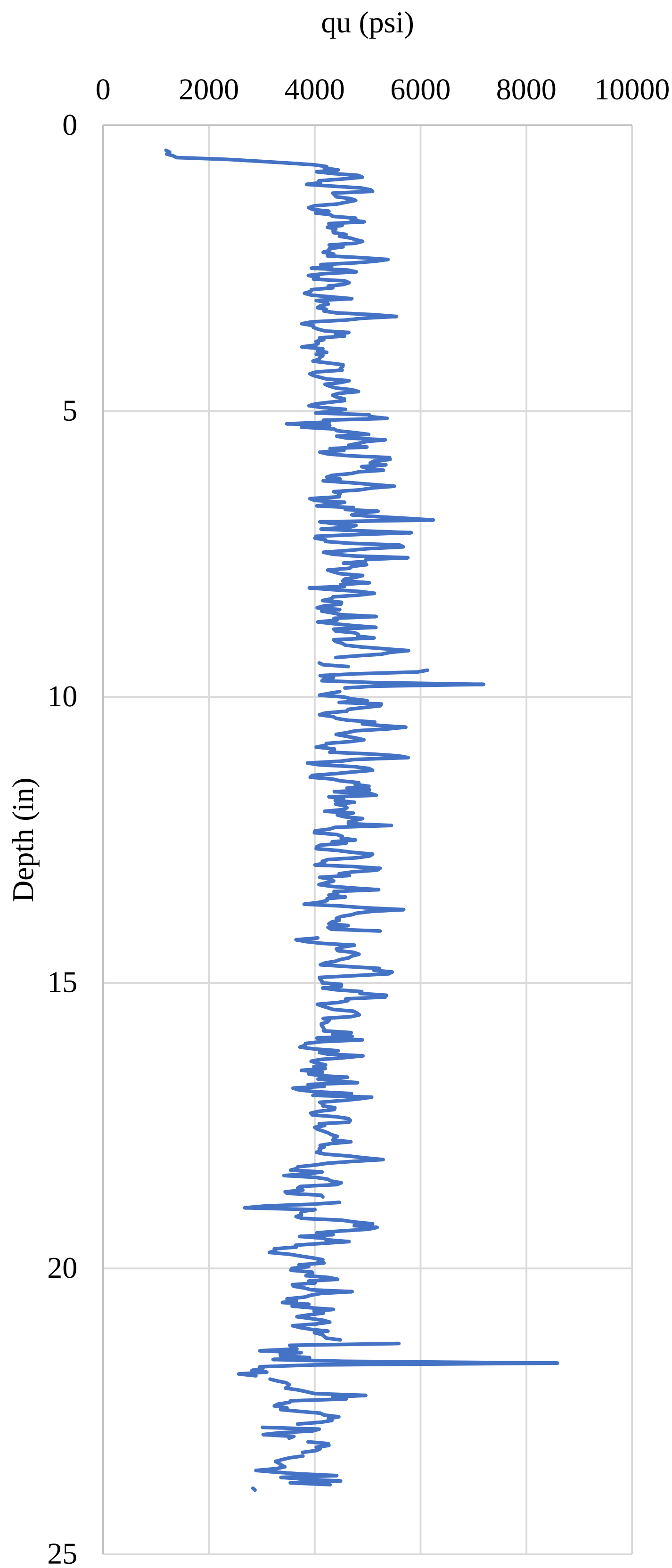


Figure F.19 Core 5-4

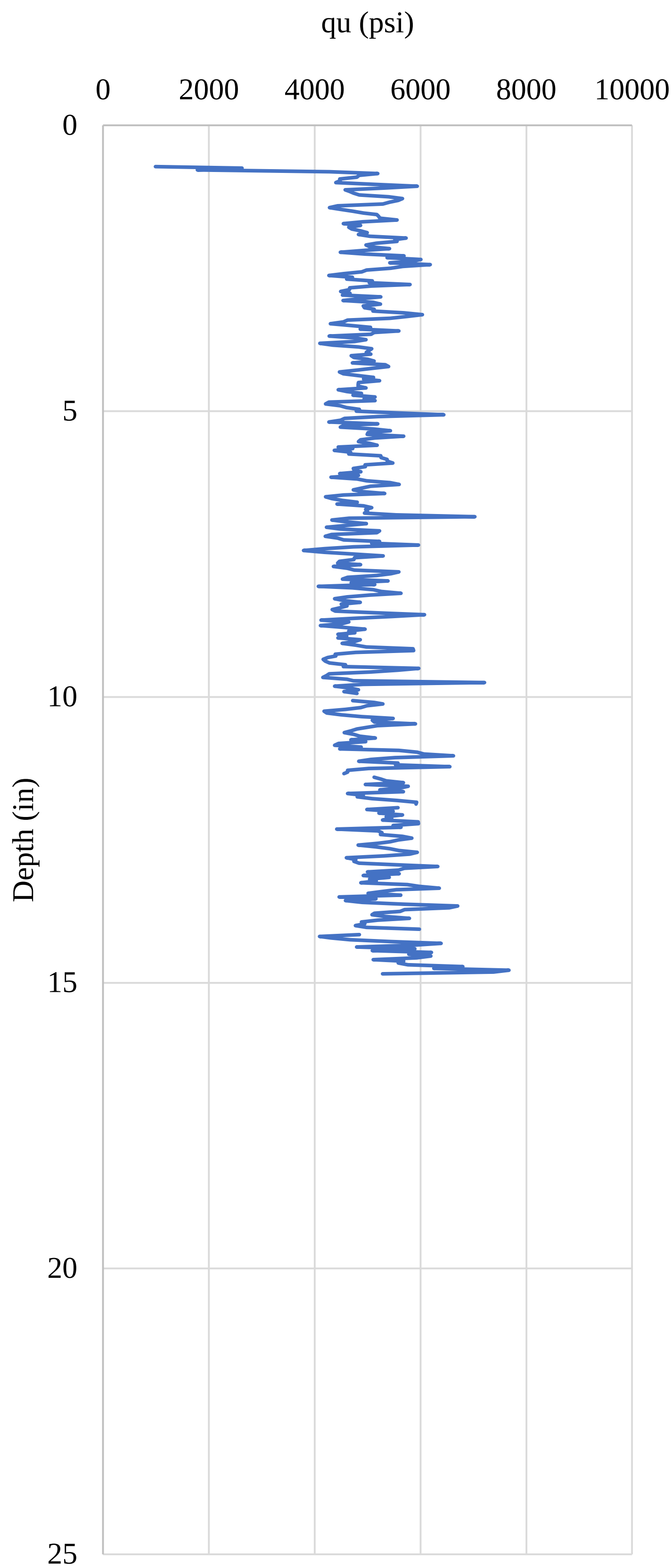


Figure F.20 Core 5-5

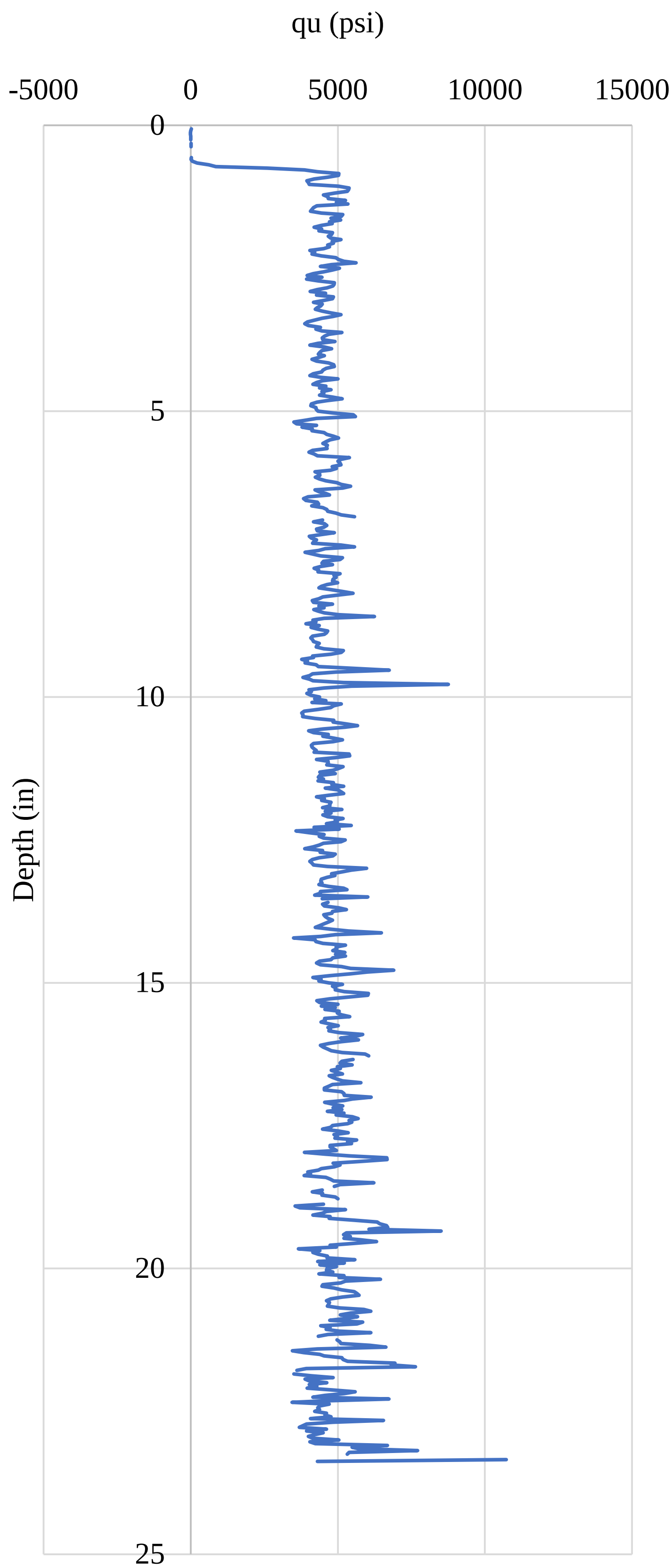


Figure F.21 Core 5-6

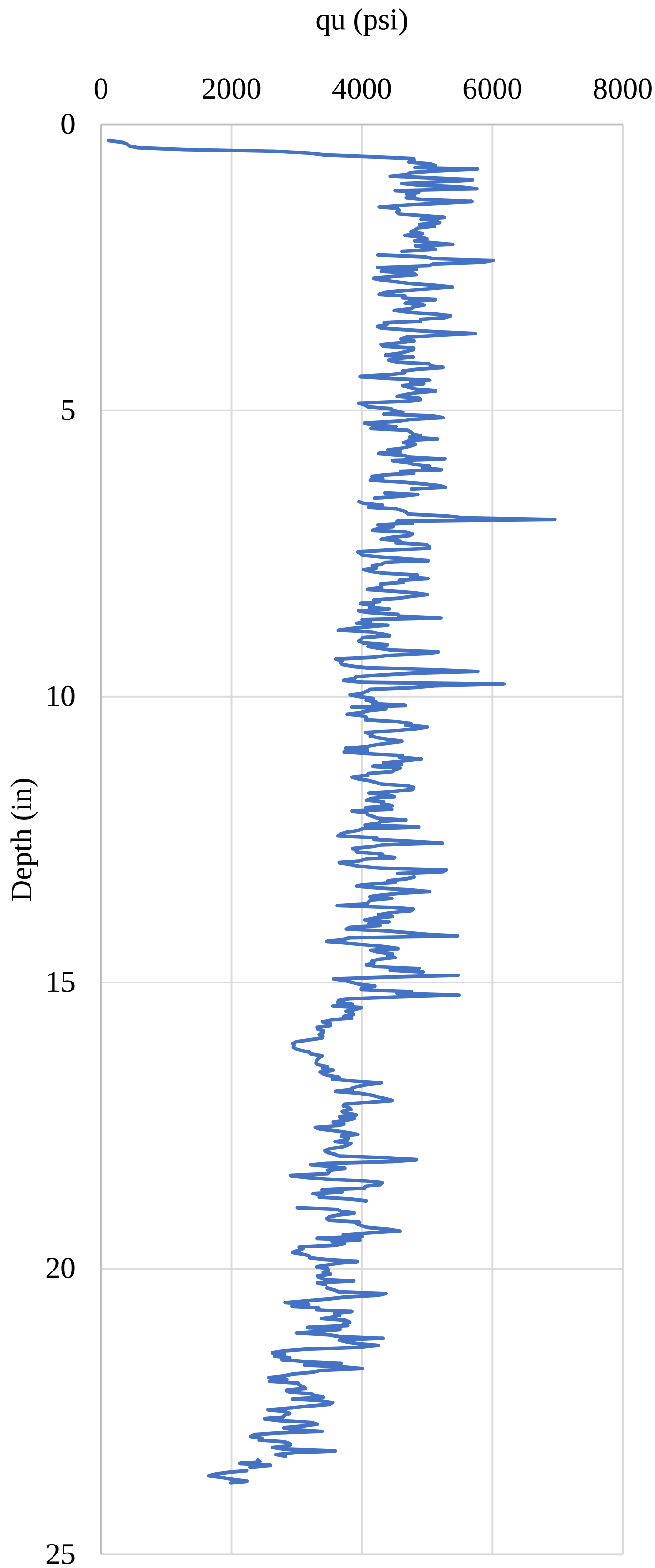


Figure F.22 Core 6-1

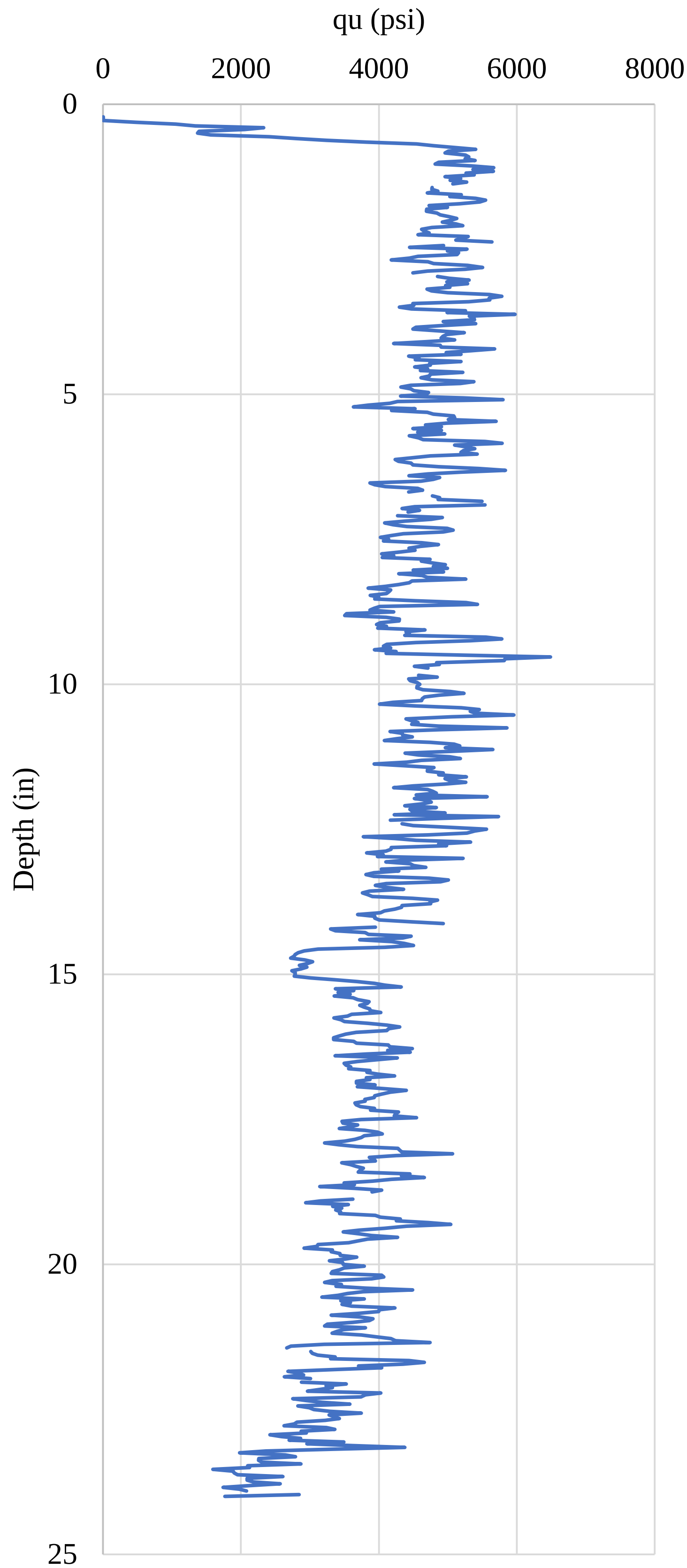


Figure F.23 Core 6-2

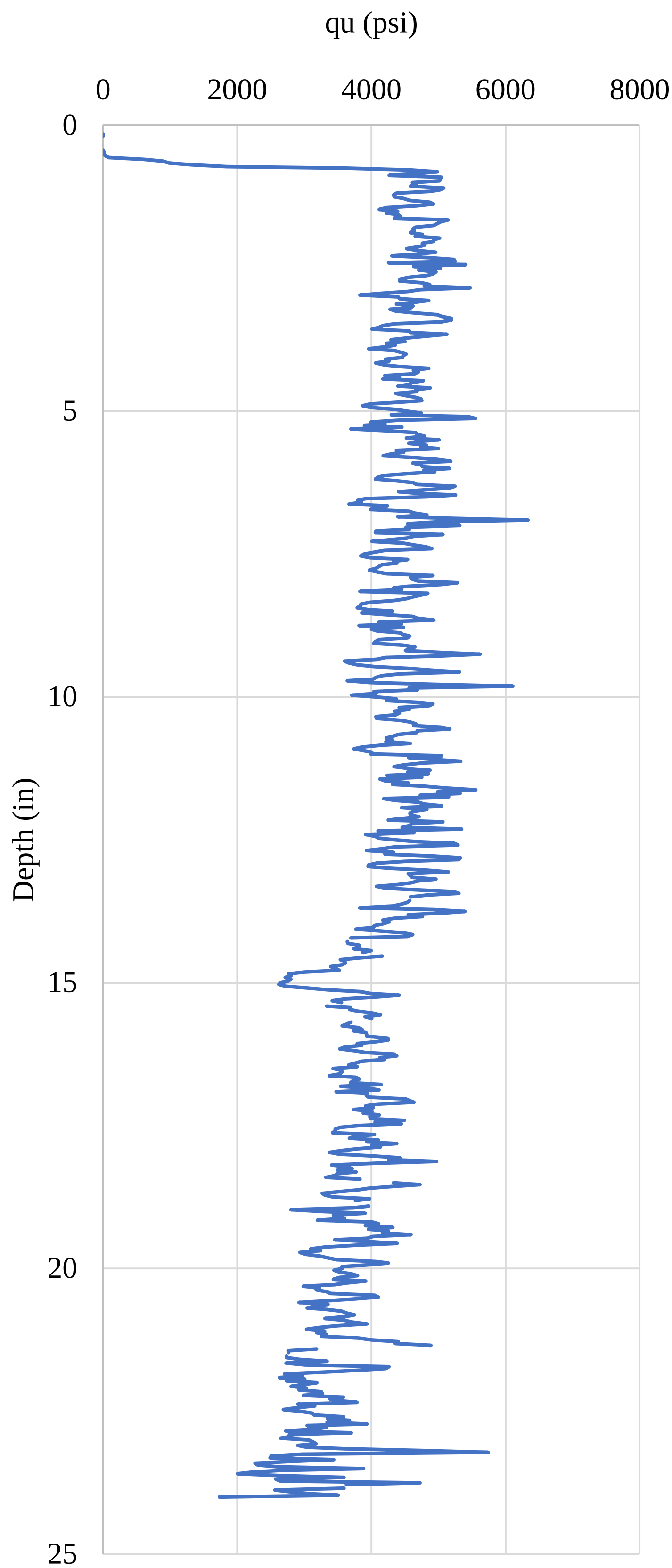


Figure F.24 Core 6-3

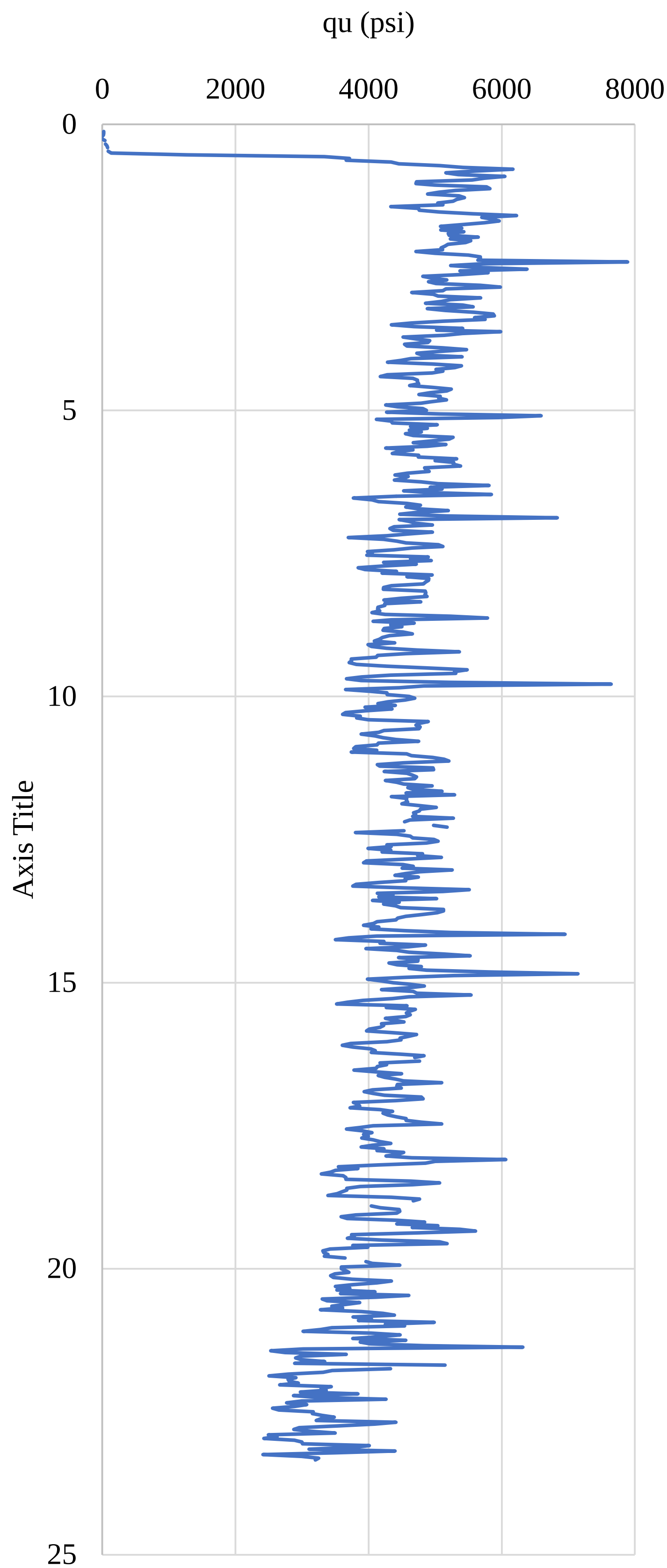


Figure F.25 Core 6-4

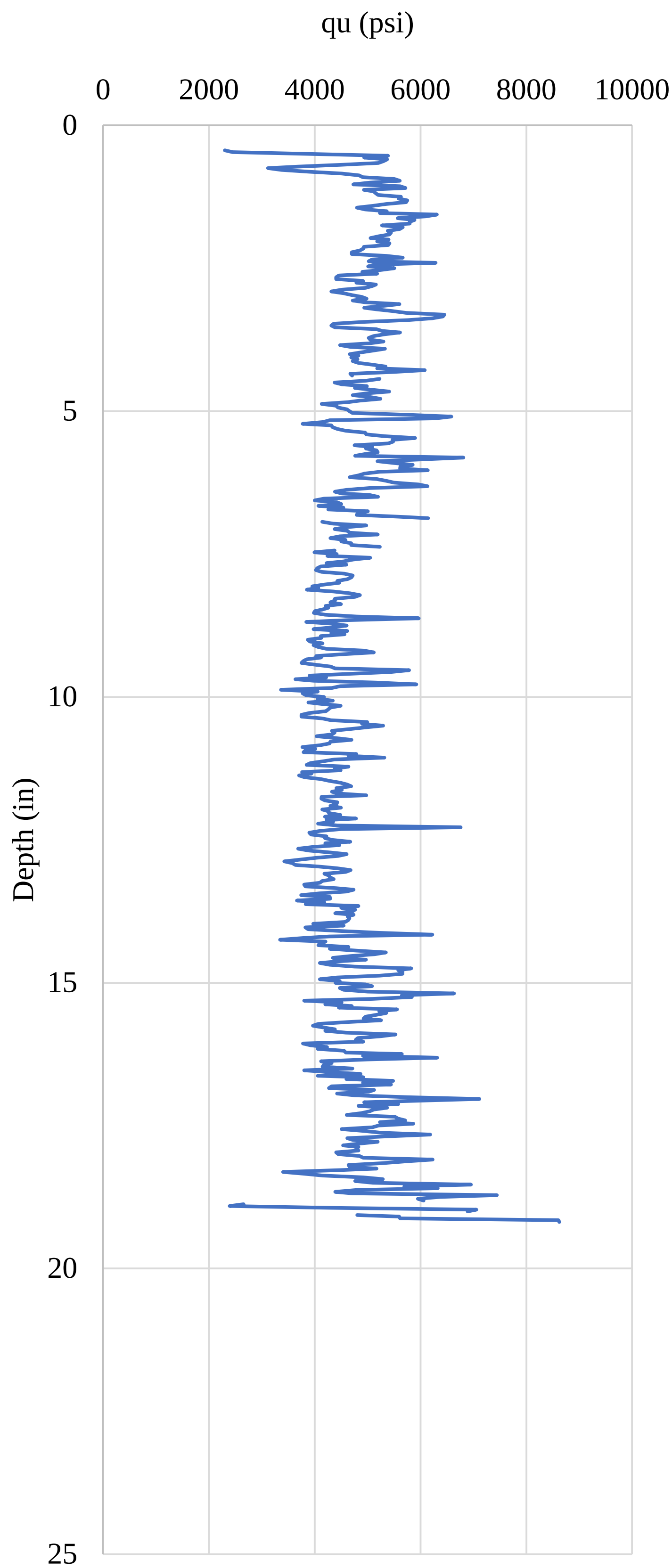


Figure F.26 Core 6-5

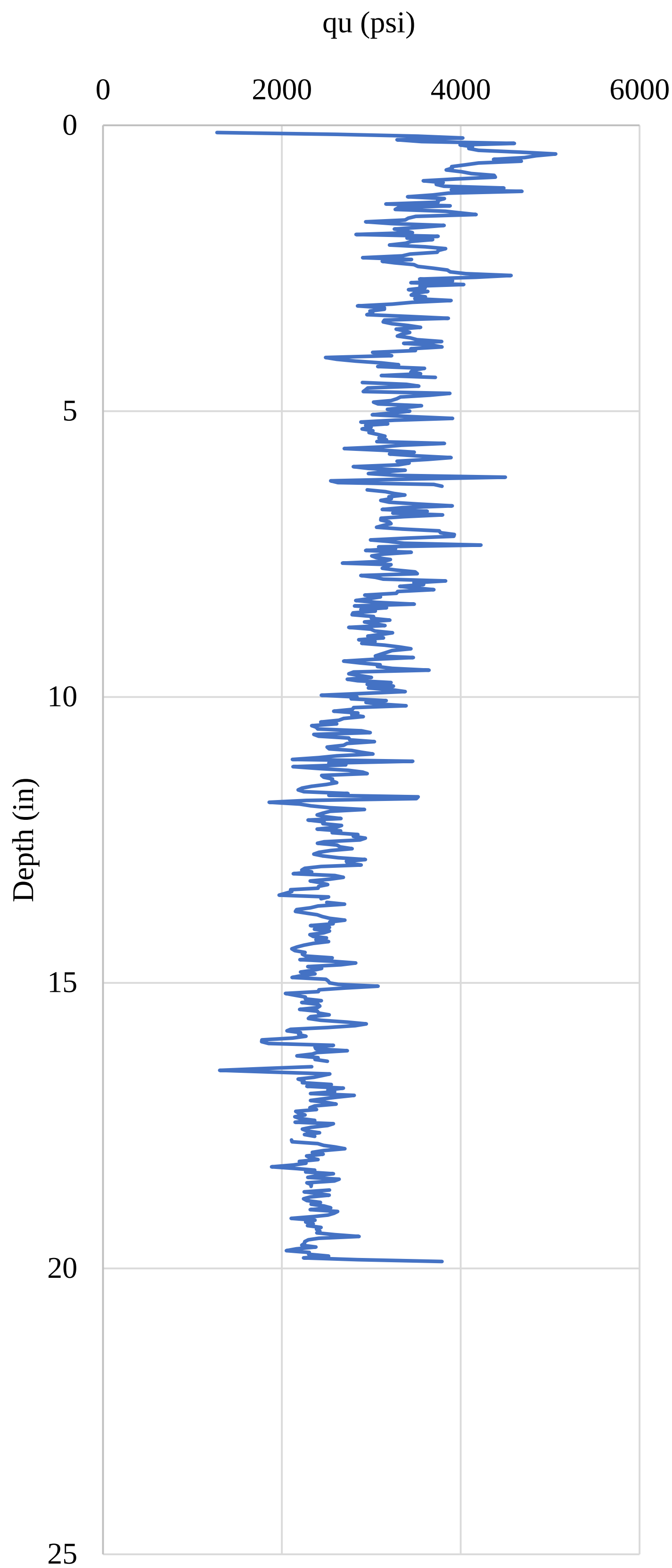


Figure F.27 Core 7-1

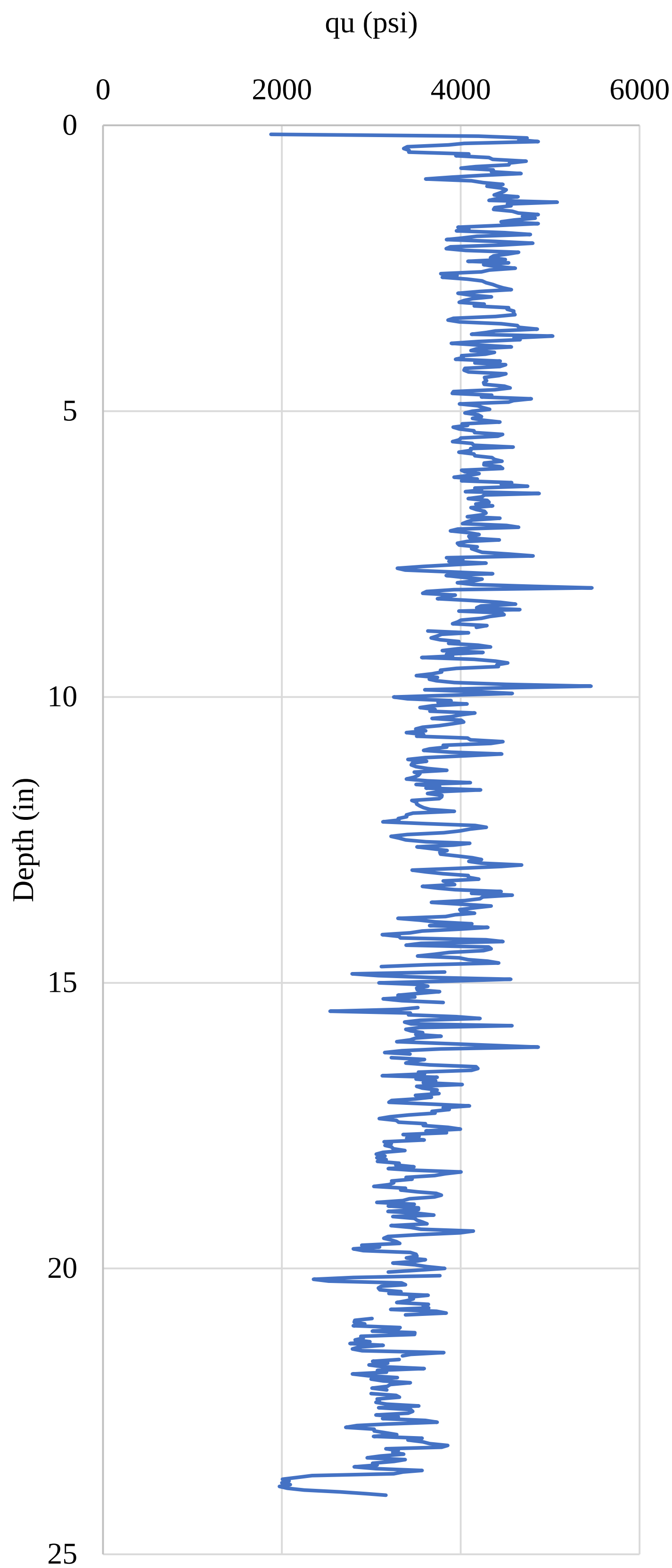


Figure F.28 Core 7-2

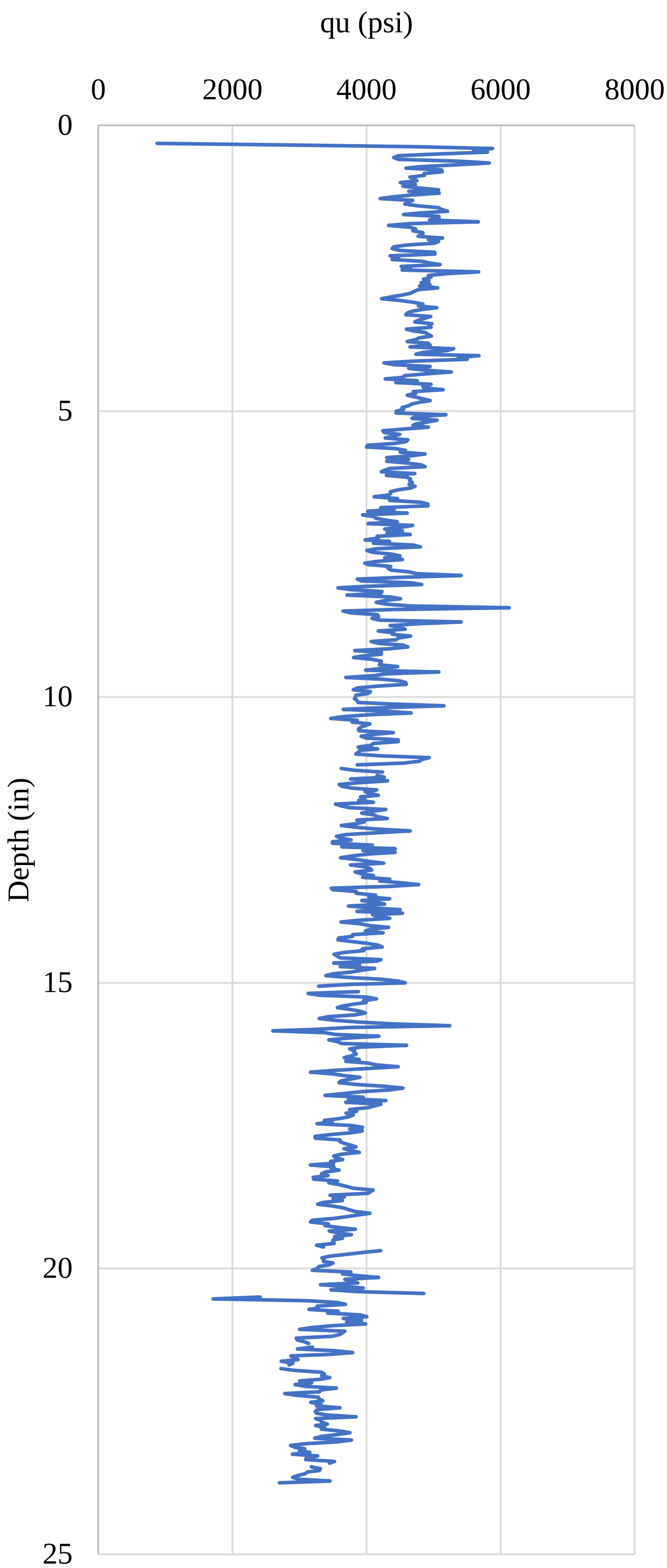


Figure F.29 Core 7-3

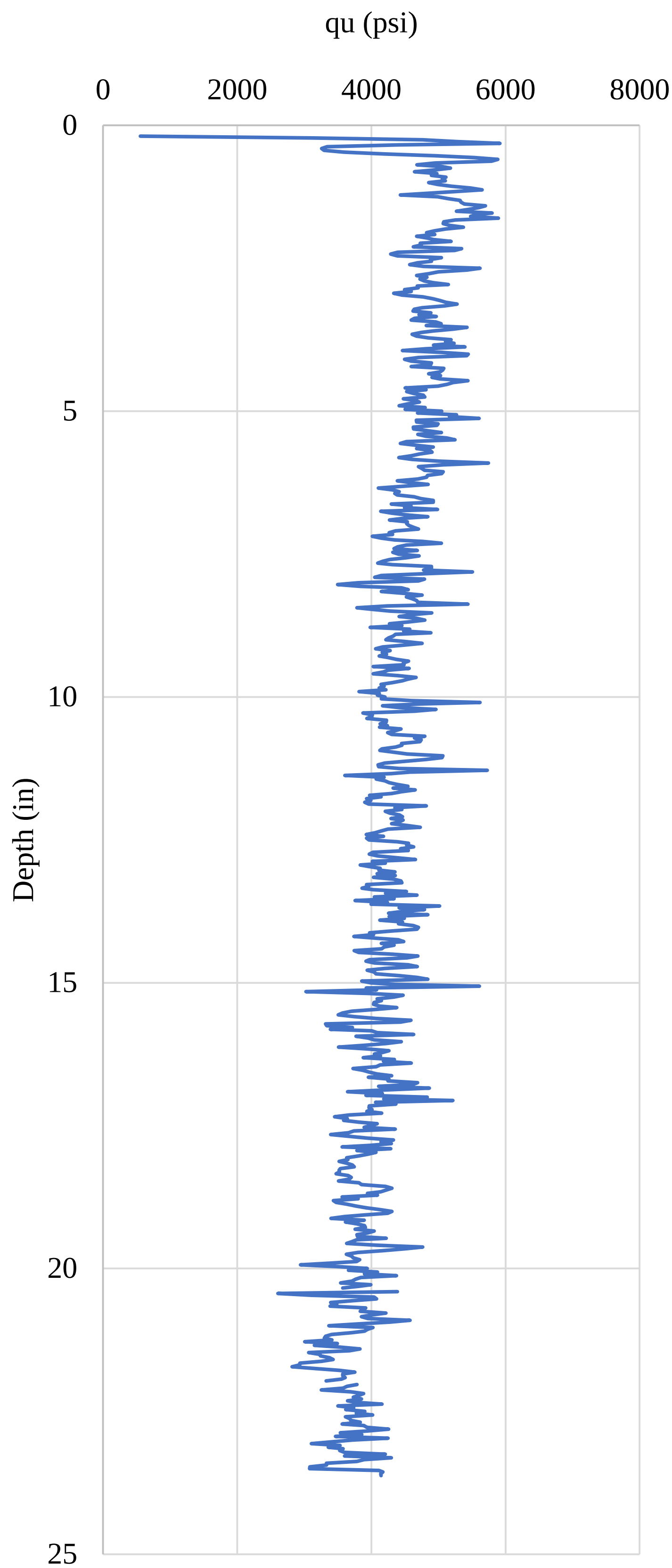


Figure F.30 Core 7-4

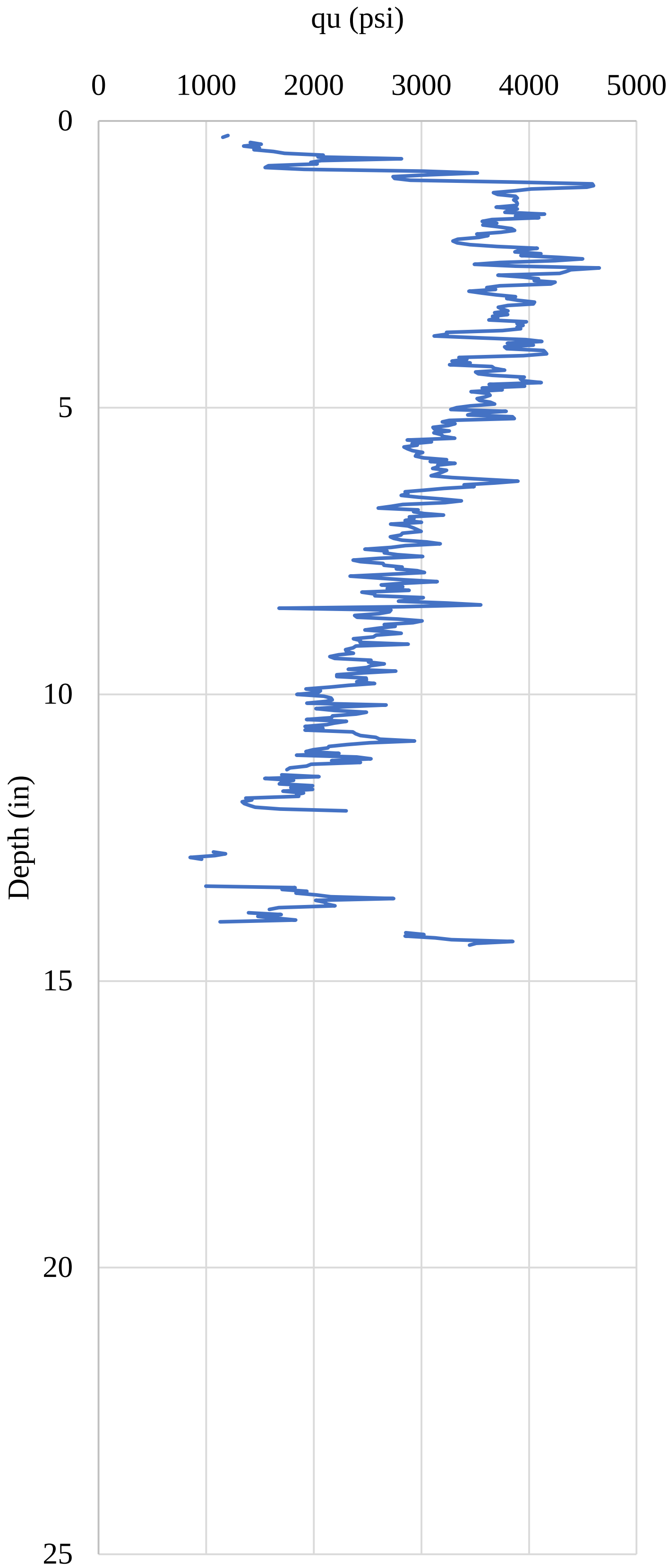


Figure F.31 Core 7-5

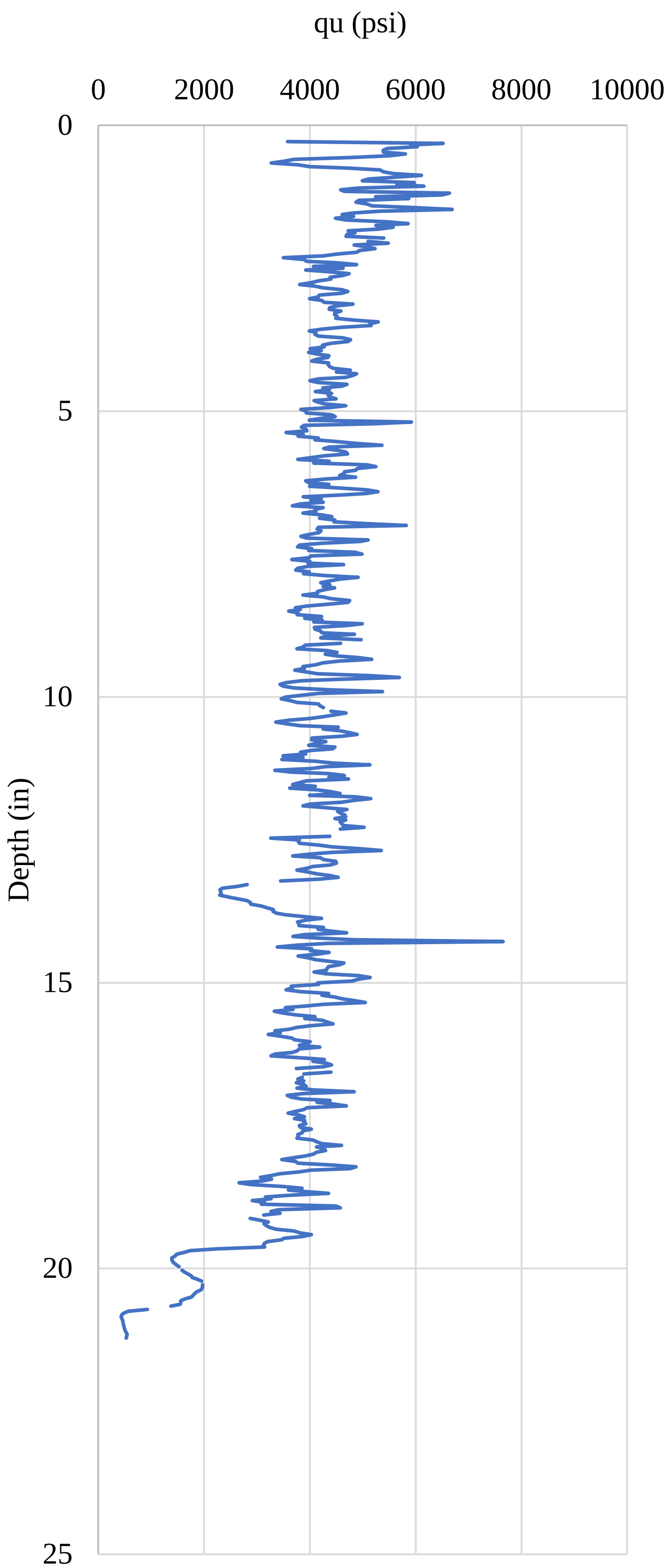


Figure F.32 Core 8-1

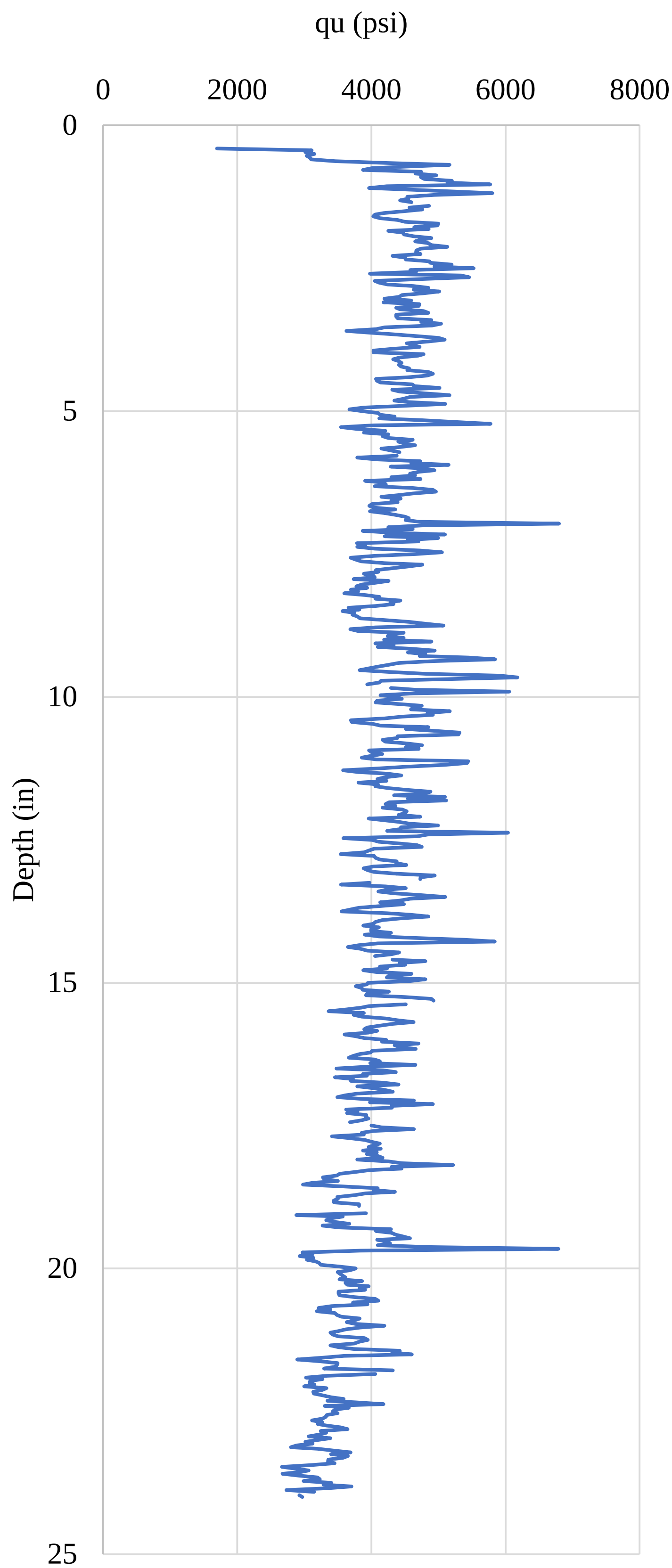


Figure F.33 Core 8-2

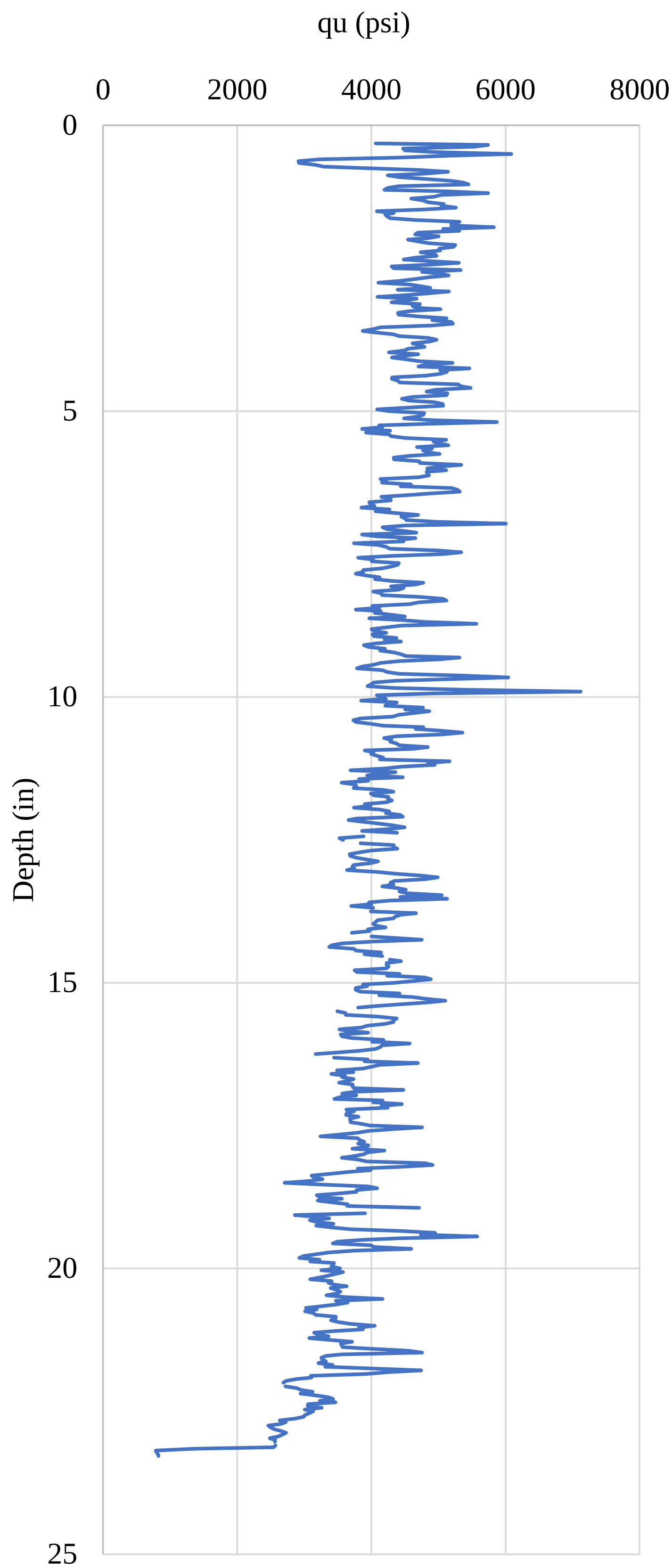


Figure F.34 Core 8-3

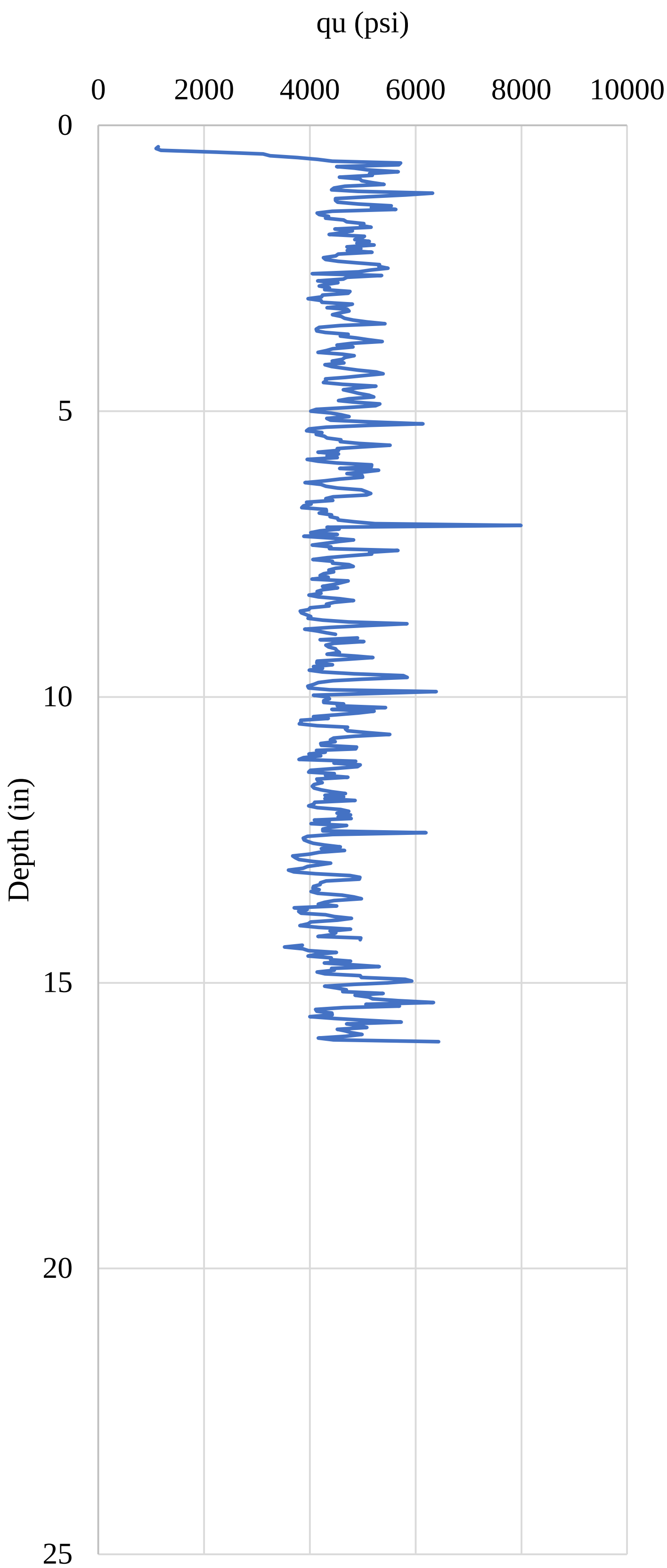


Figure F.35 Core 8-4

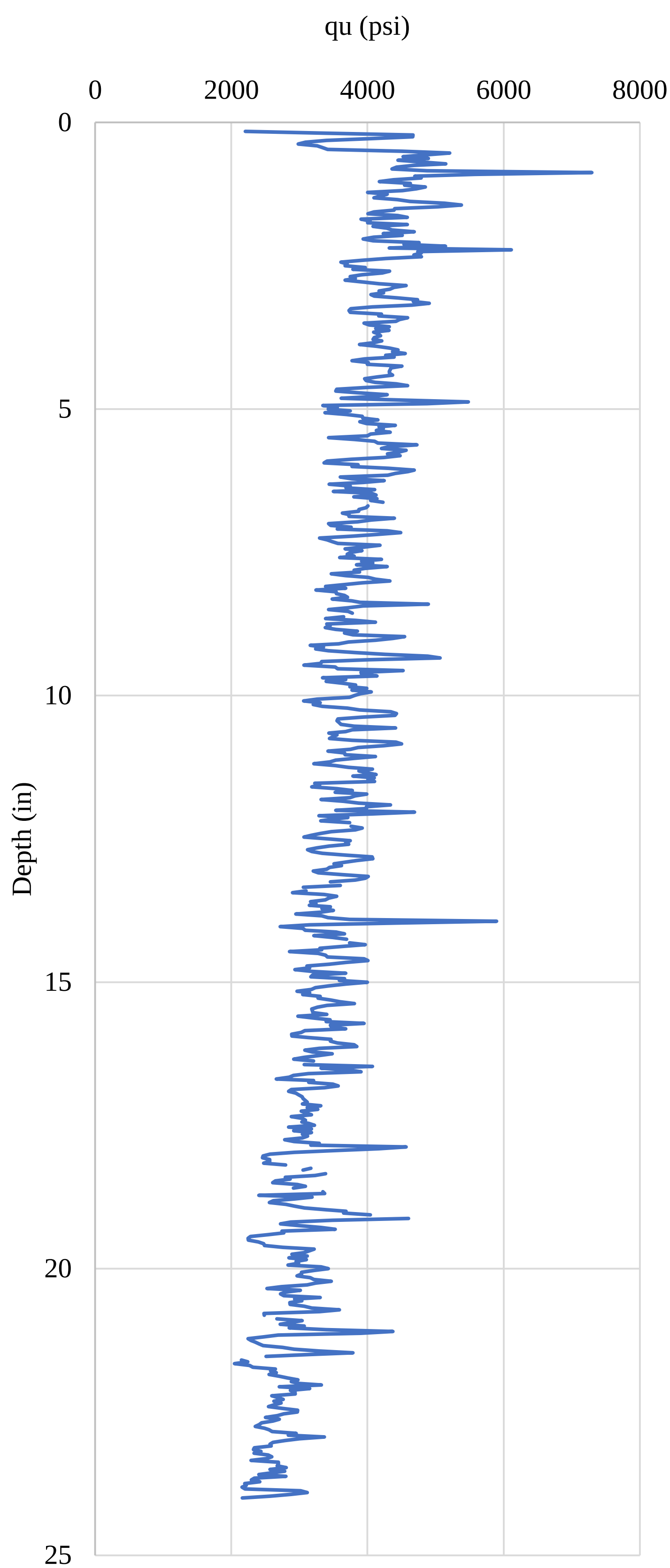


Figure F.36 Core 8-5

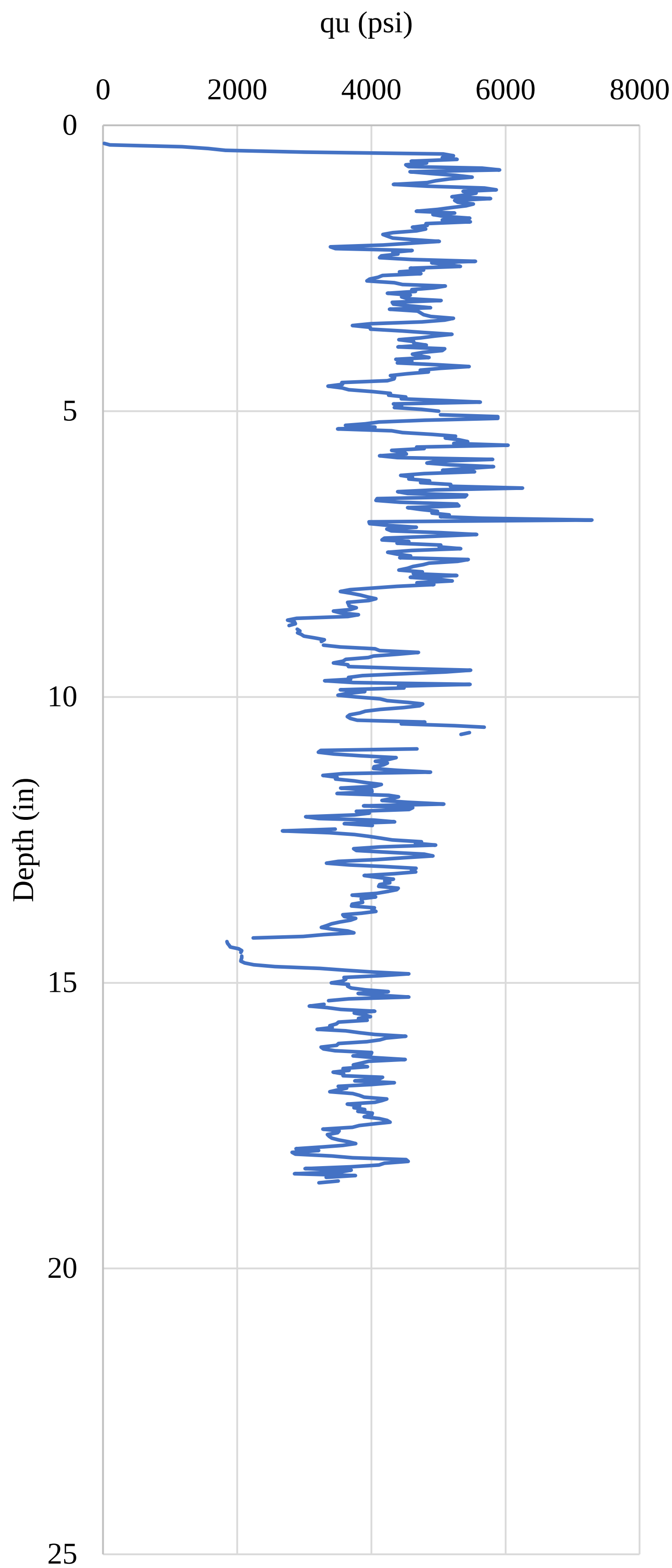


Figure F.37 Core 9-1

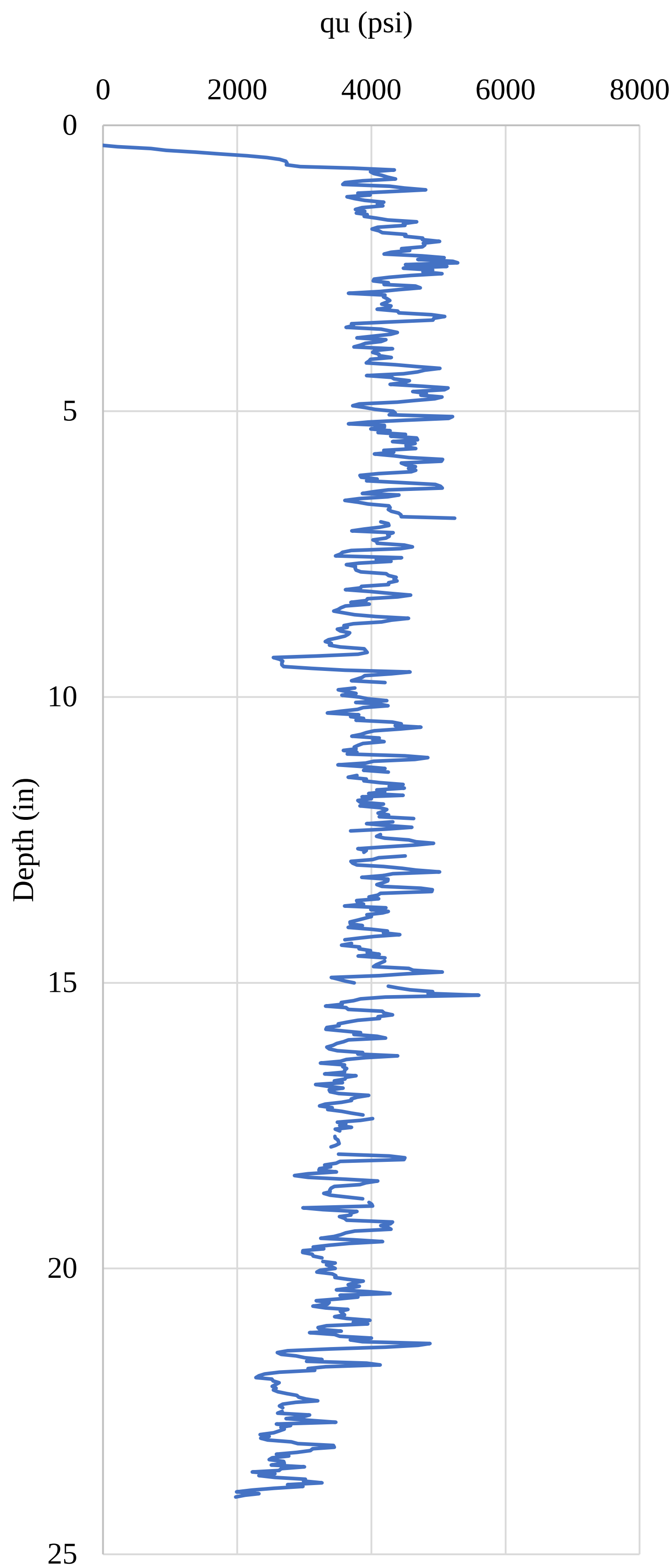


Figure F.38 Core 9-2

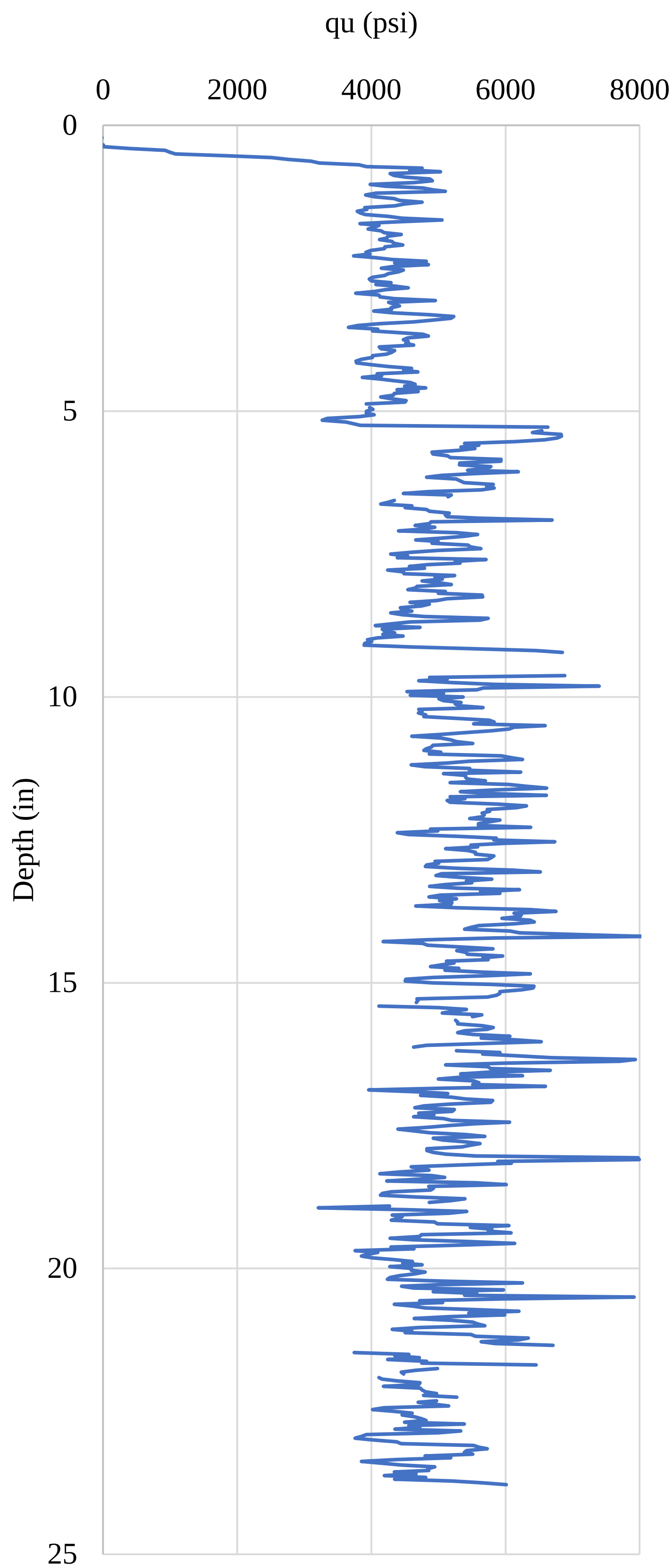


Figure F.39 Core 9-3

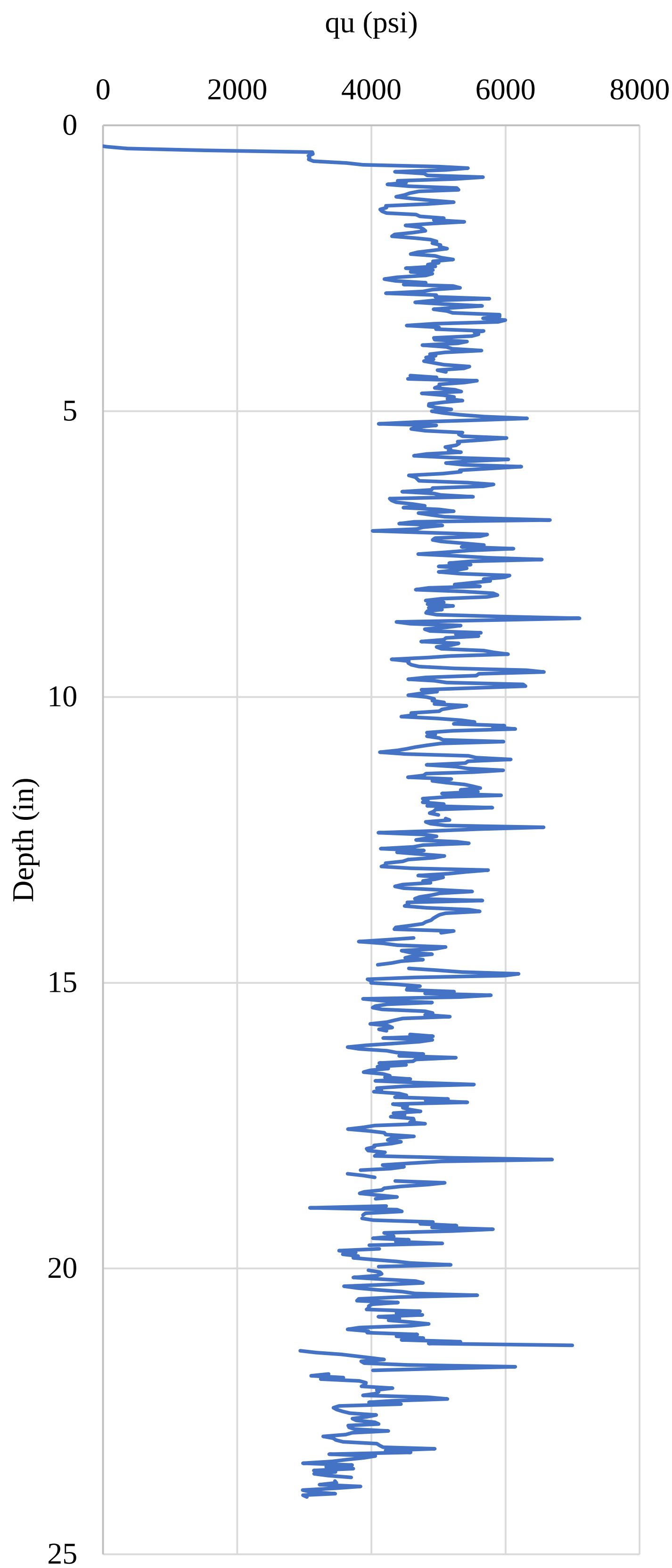


Figure F.40 Core 9-4

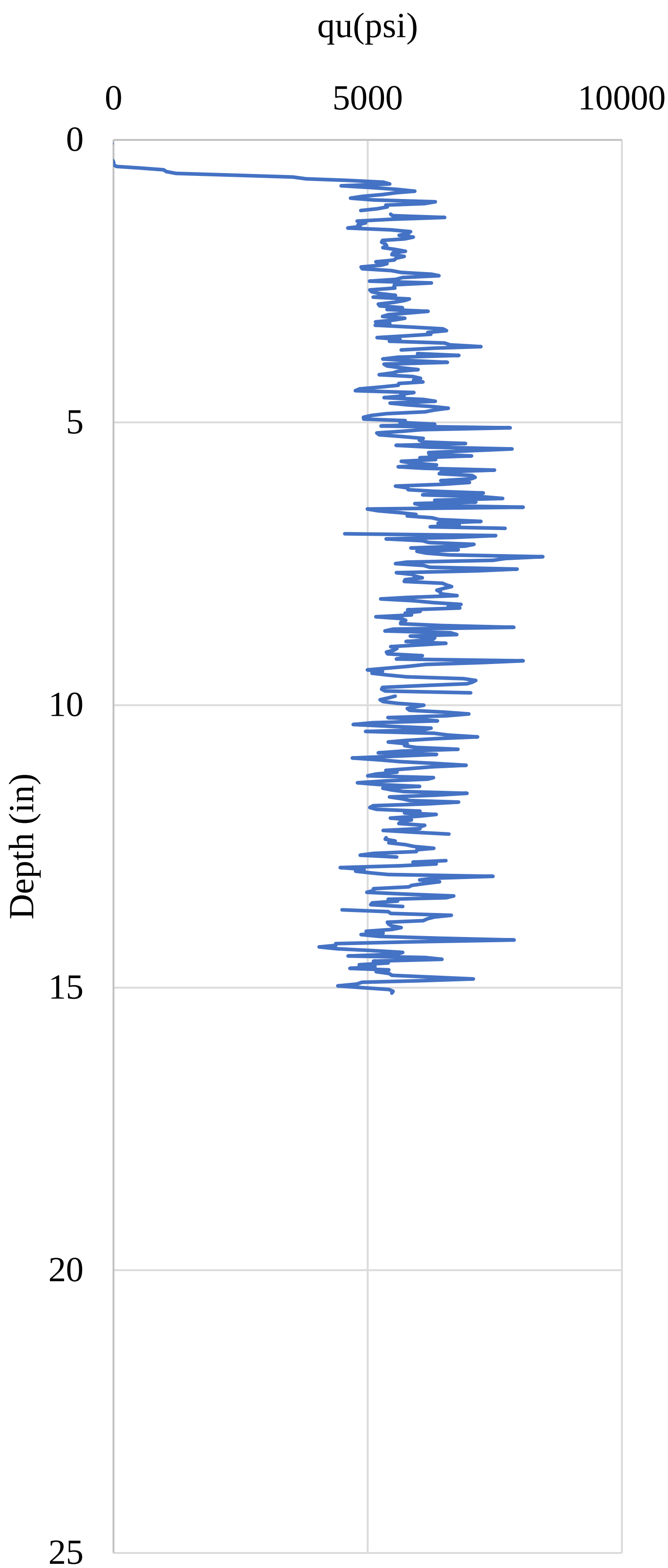


Figure F.41 Core 9-5

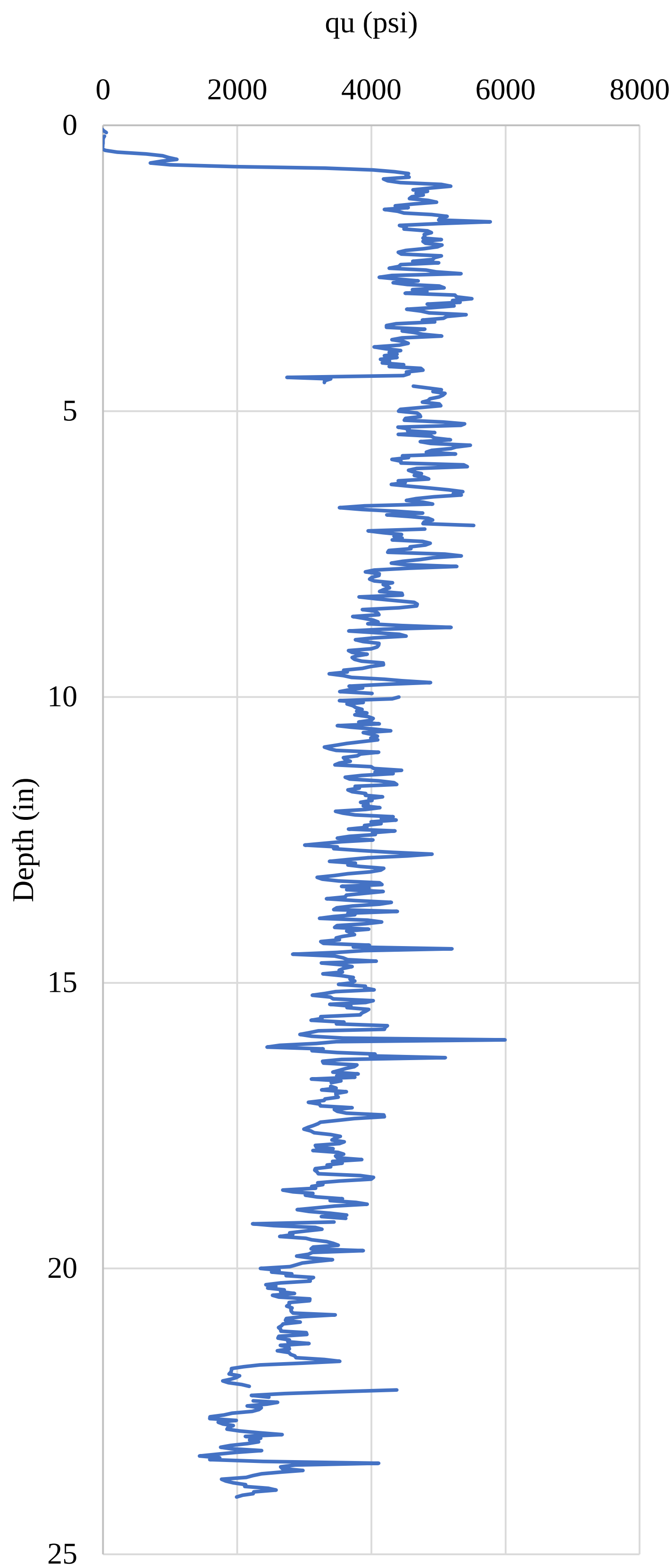


Figure F.42 Core 10-1

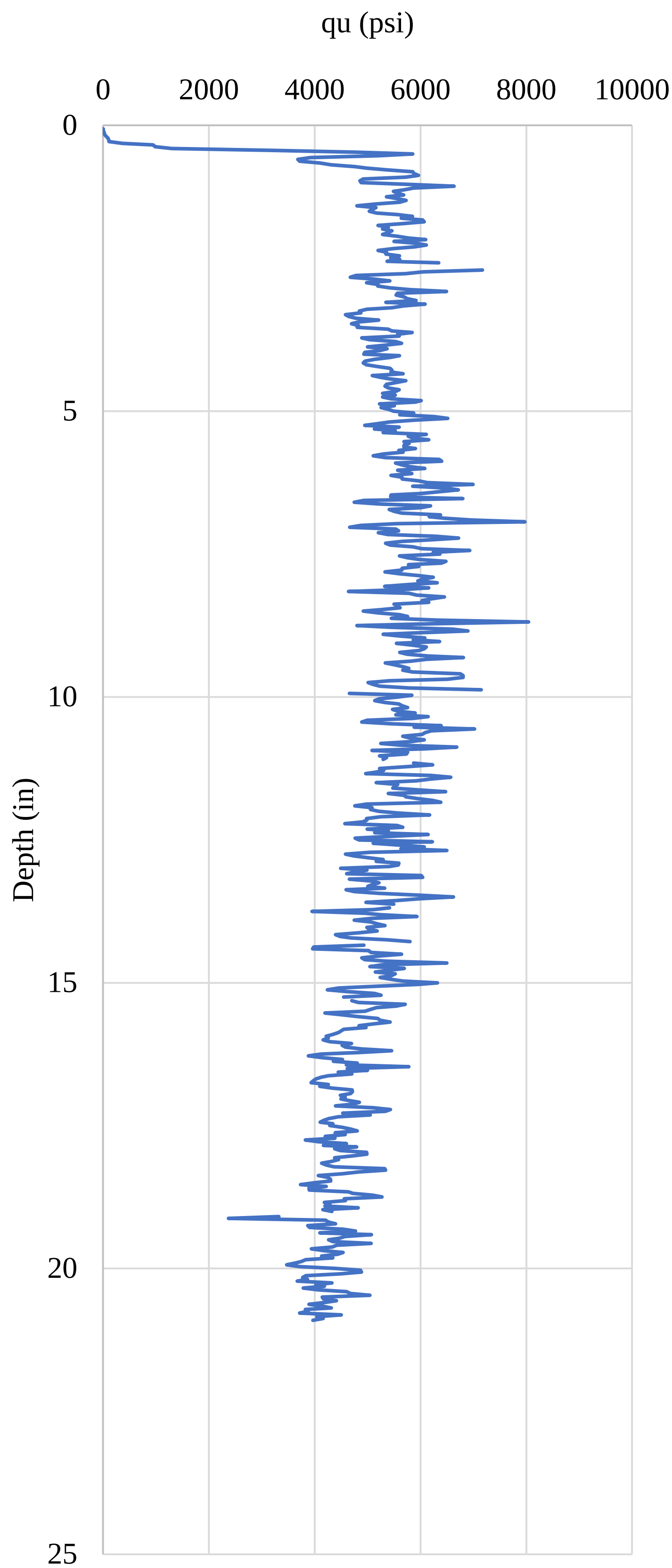


Figure F.43 Core 10-2

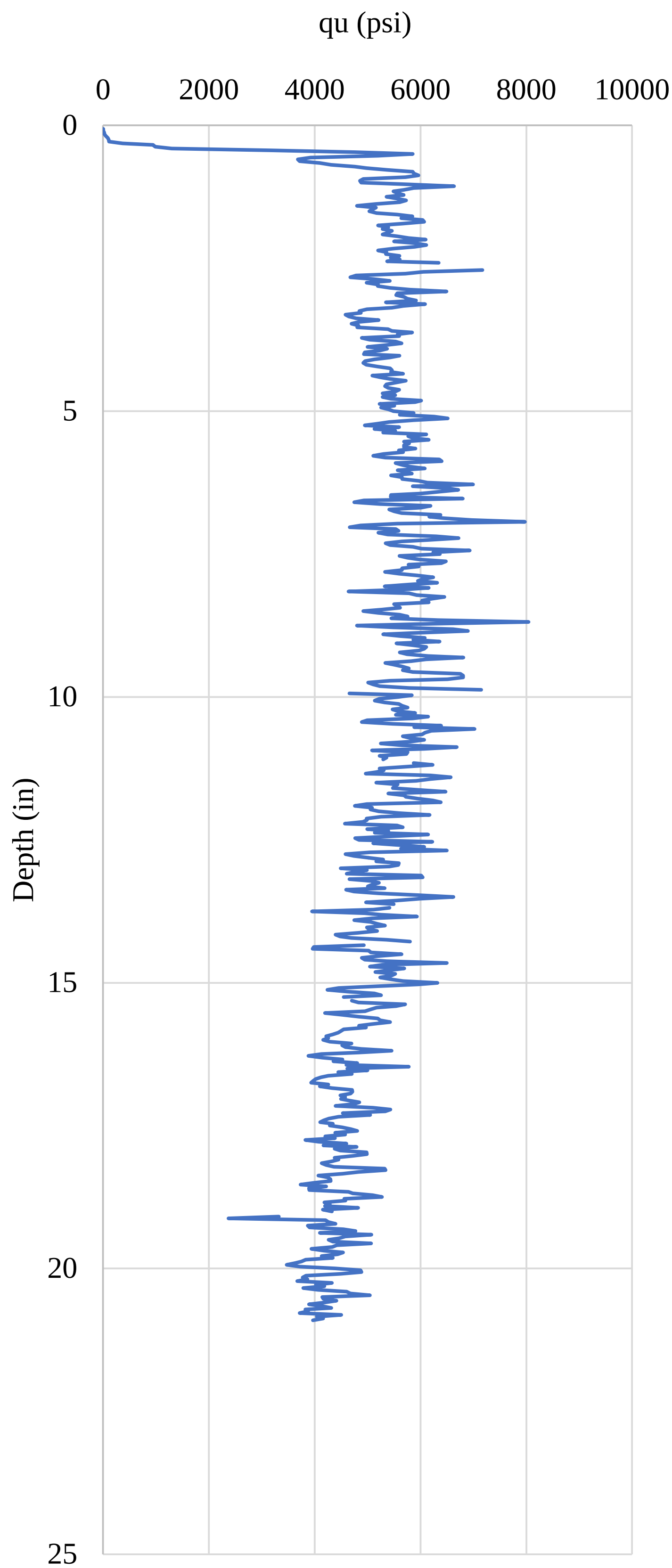


Figure F.44 Core 10-3

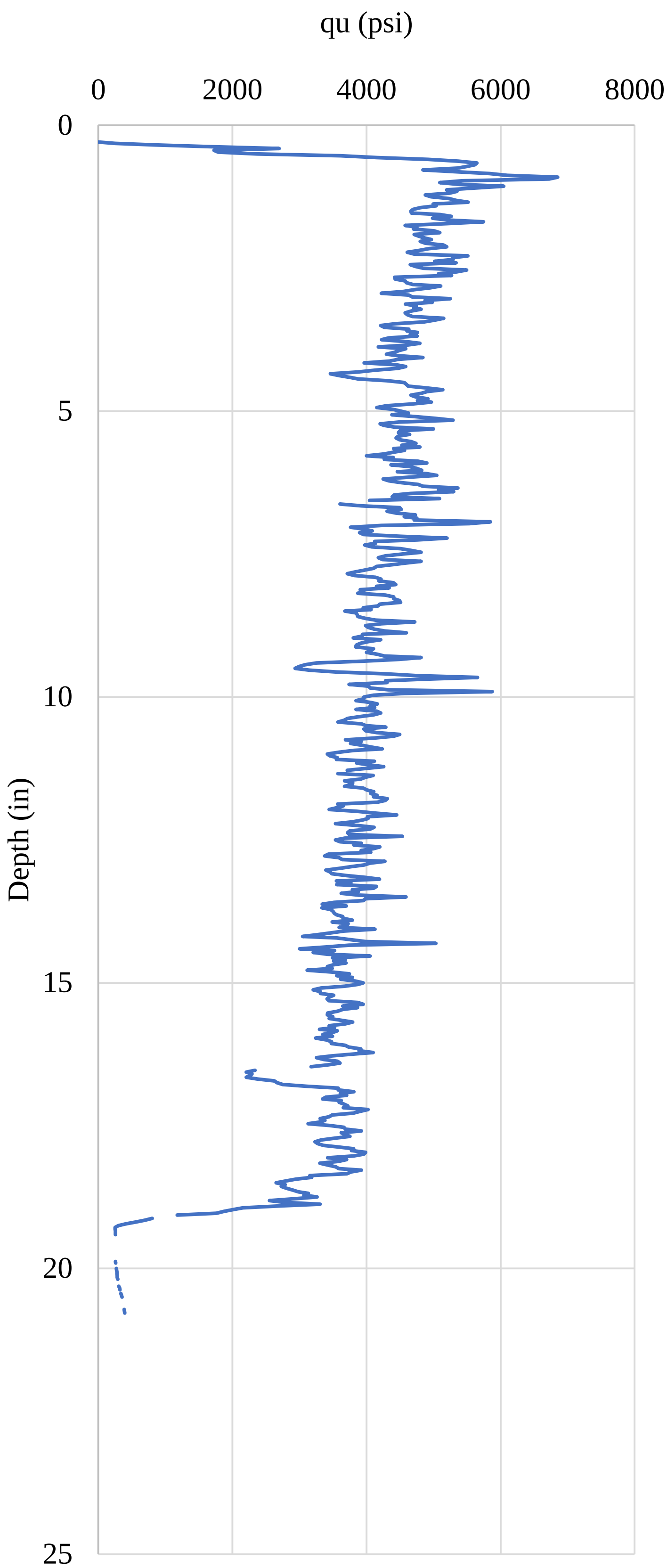


Figure F.45 Core 10-4

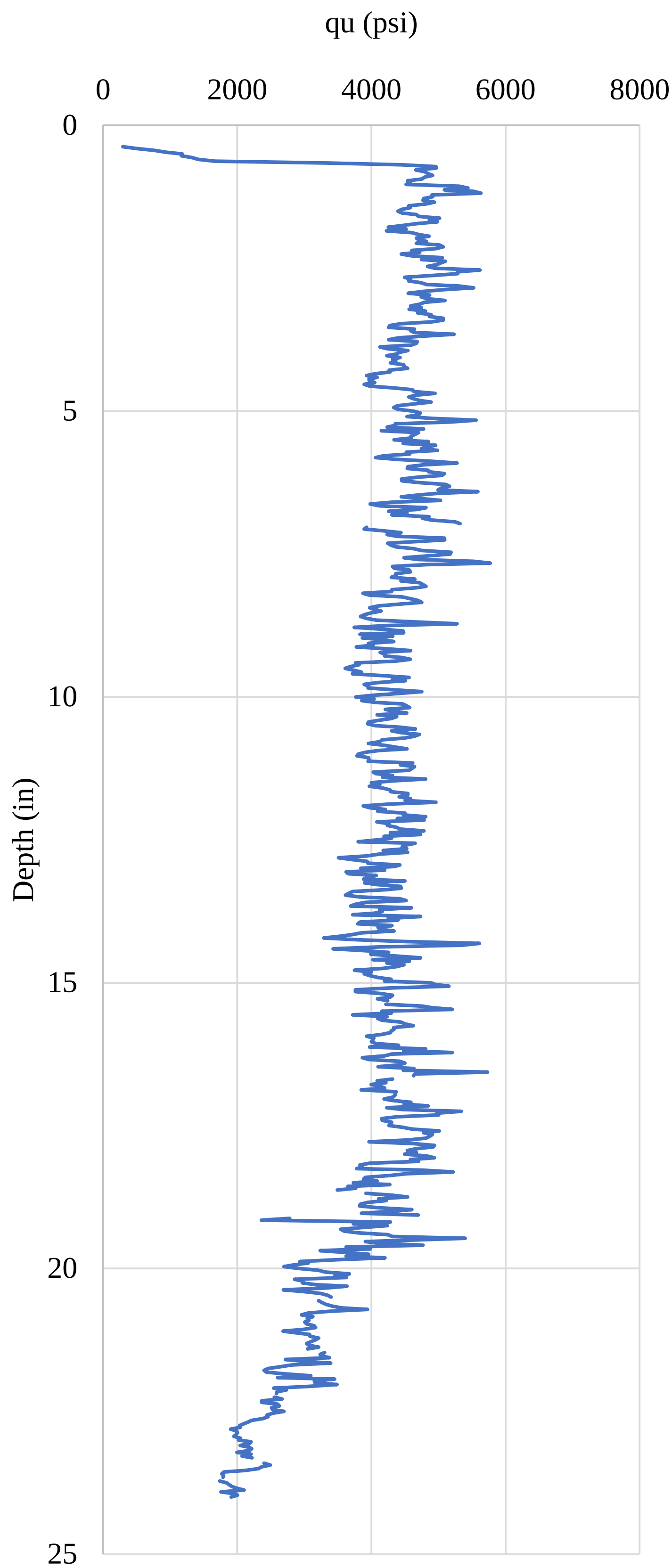


Figure F.46 Core 10-5

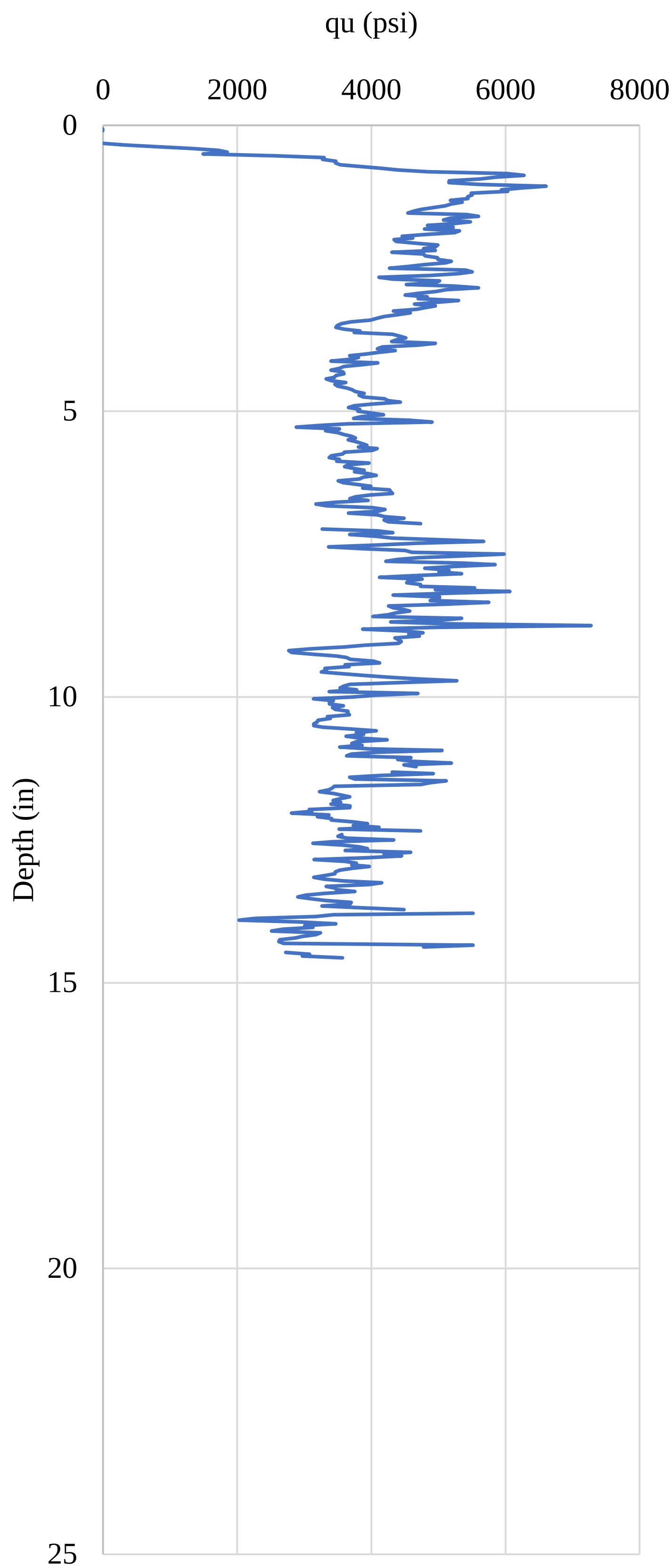


Figure F.47 Core 10-6

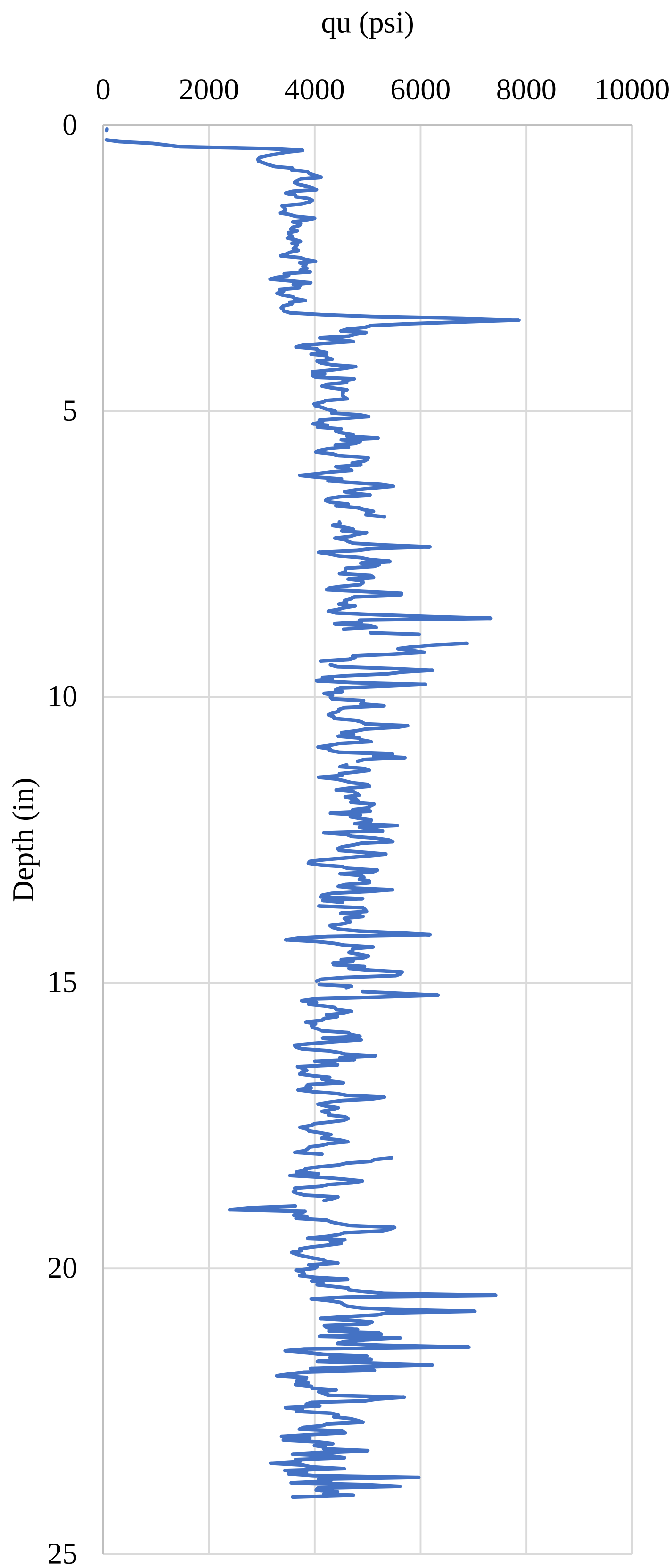


Figure F.48 Core 11-1

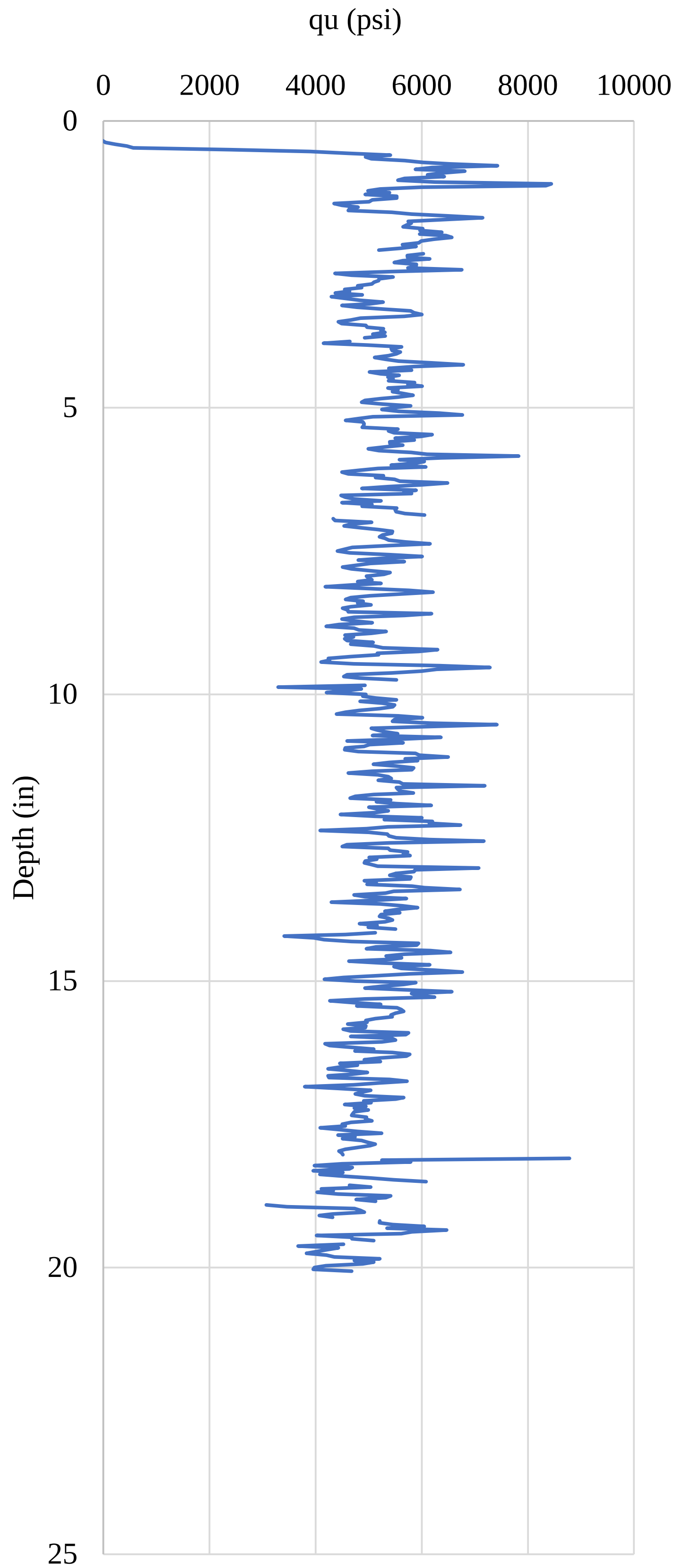


Figure F.49 Core 11-2

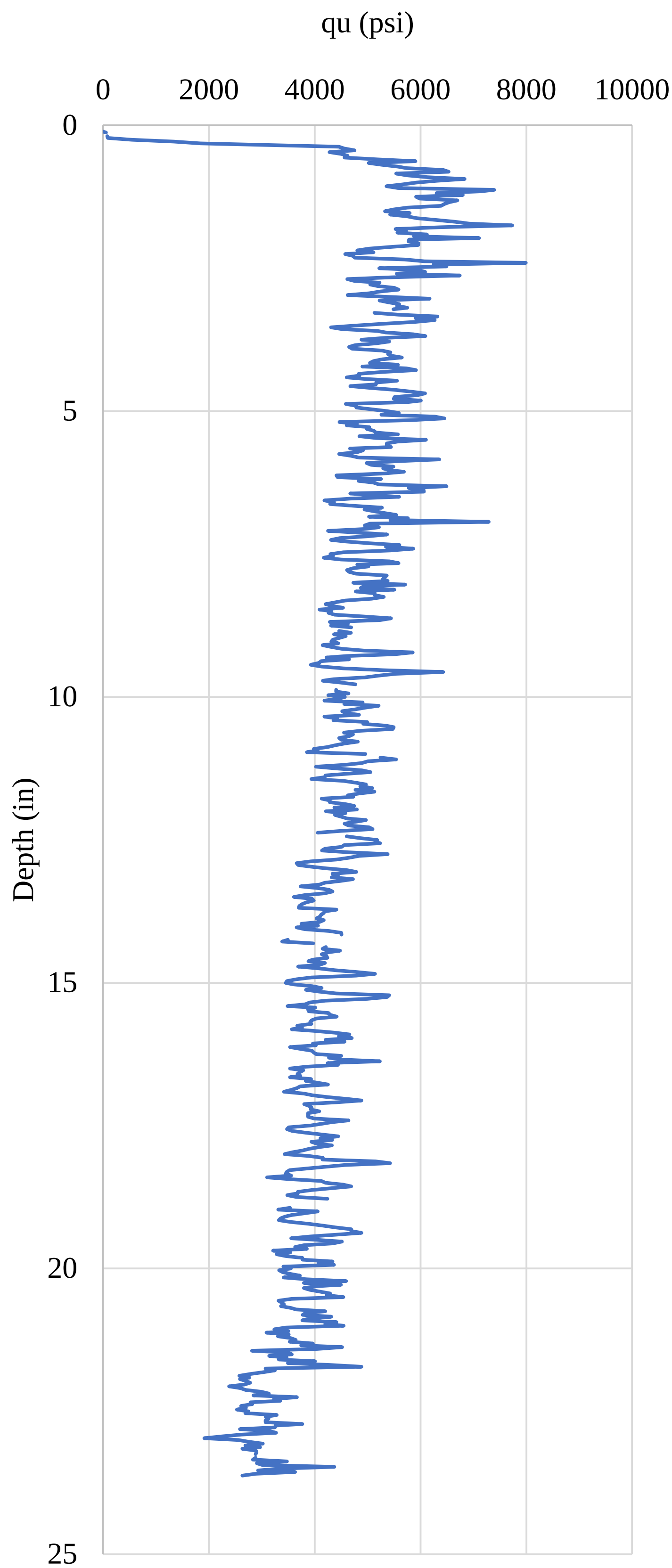


Figure F.50 Core 11-3

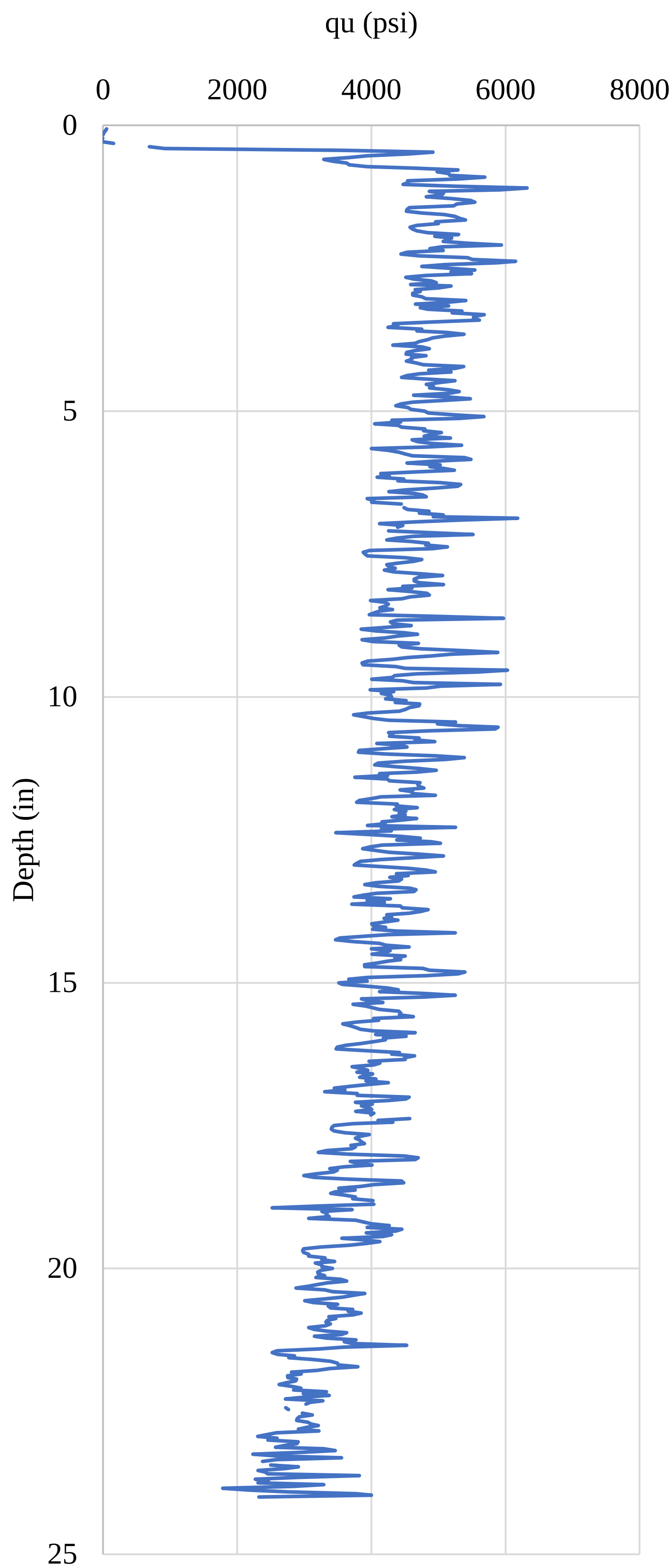


Figure F.51 Core 11-4

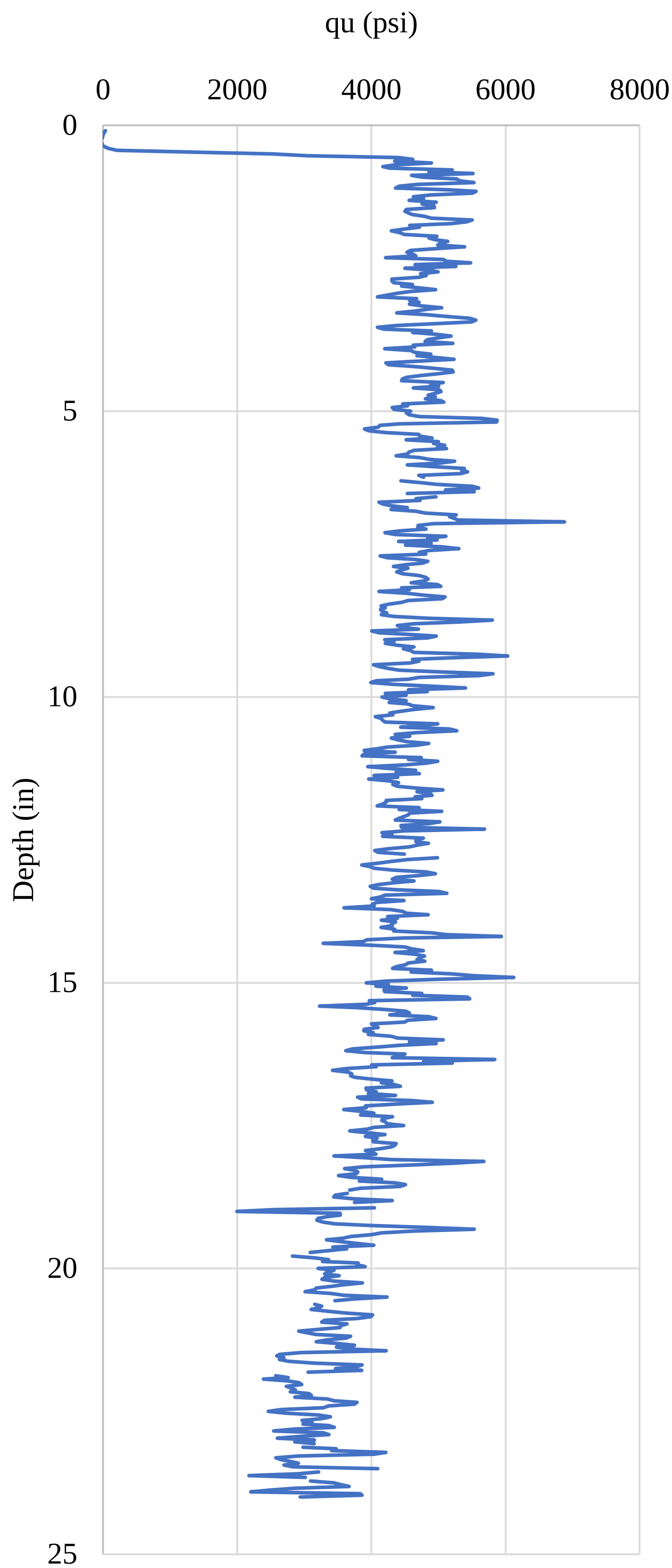


Figure F.52 Core 11-5

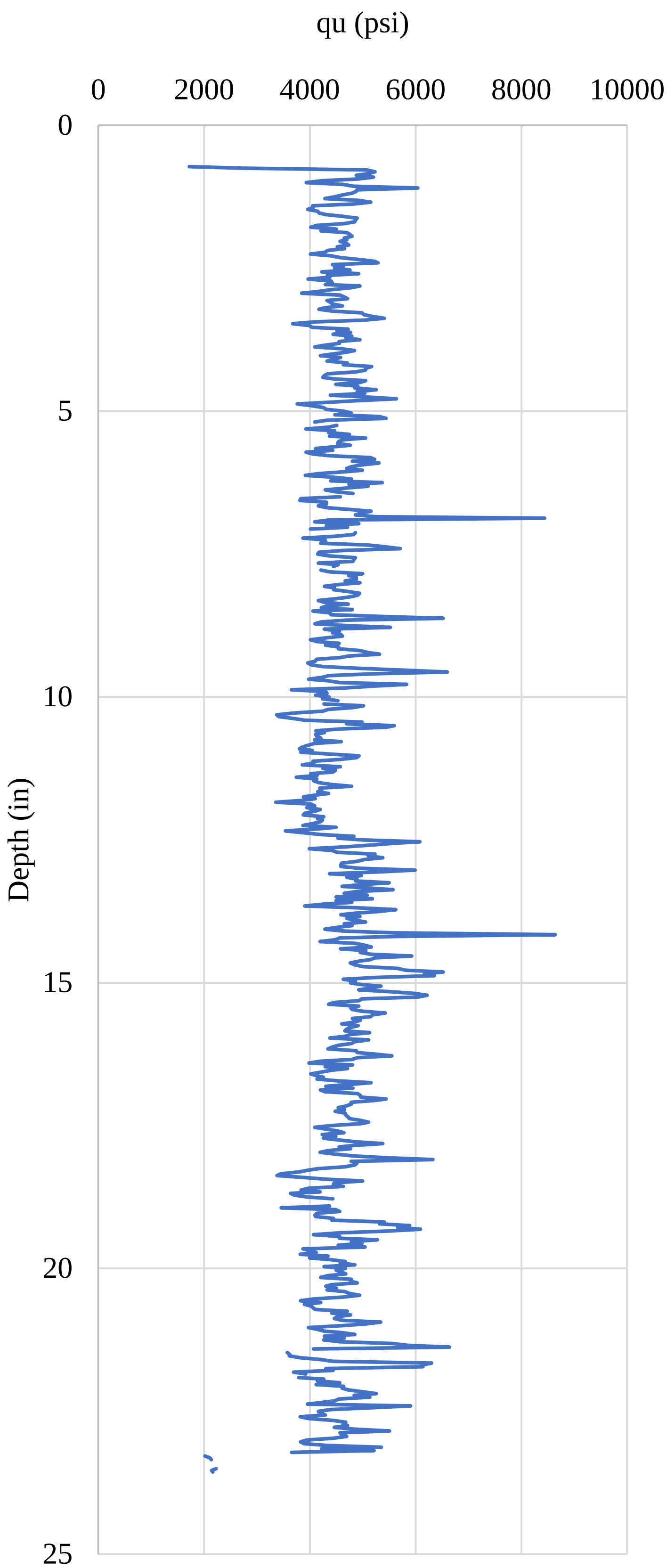


Figure F.53 Core 12-1

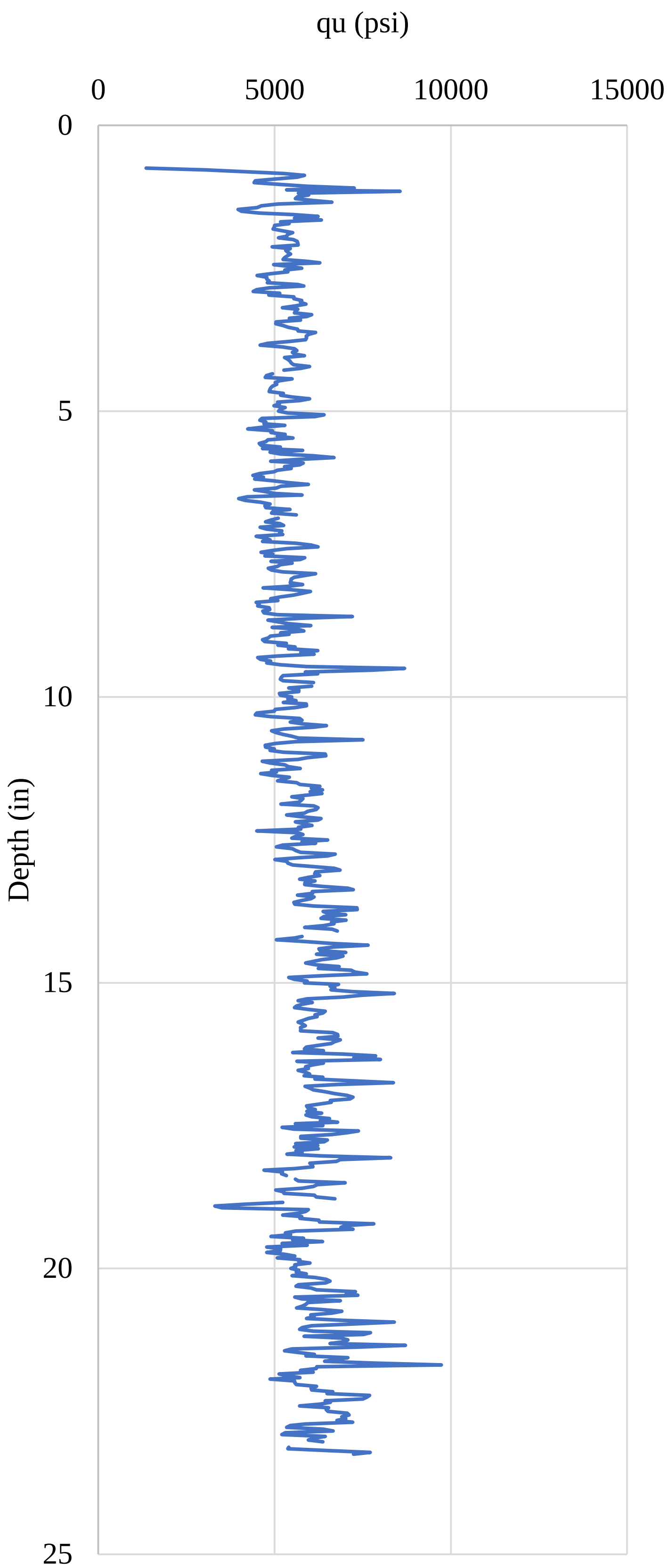


Figure F.54 Core 12-2

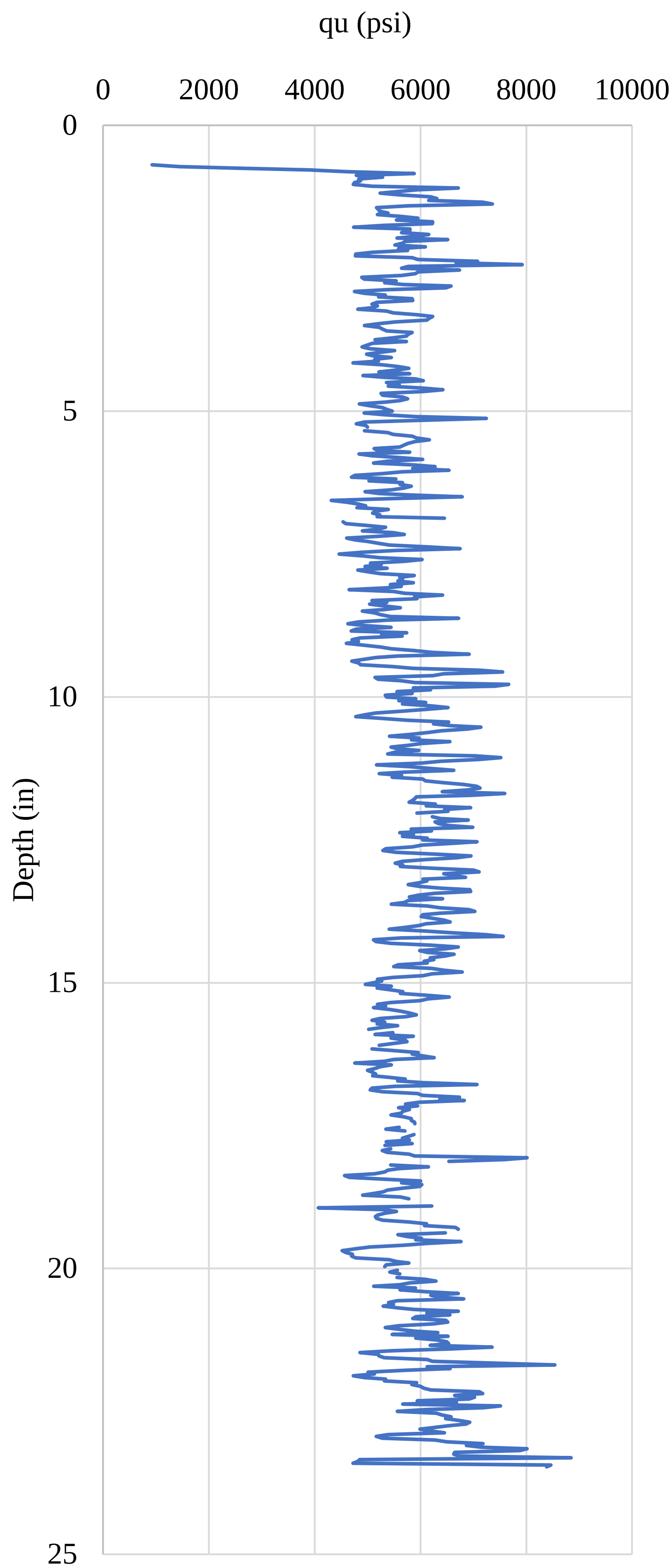


Figure F.55 Core 12-3

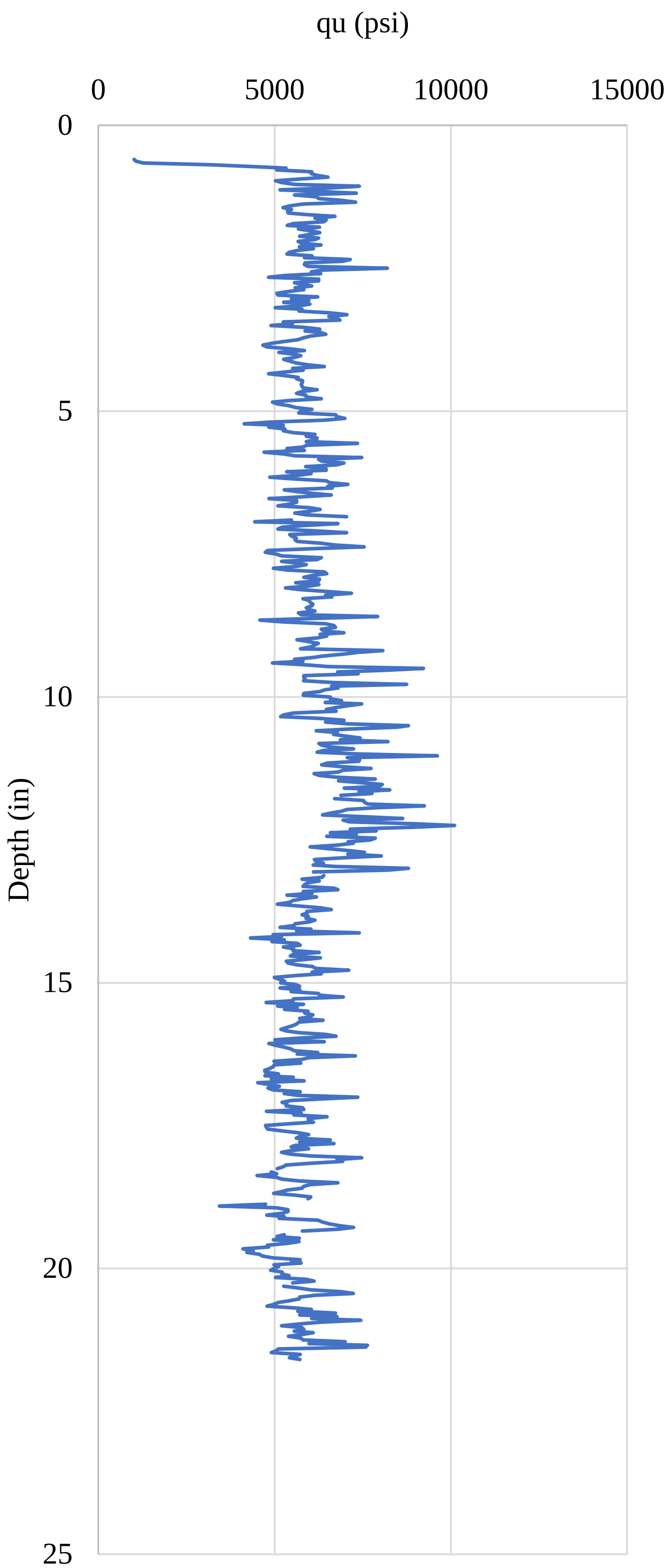


Figure F.56 Core 12-4

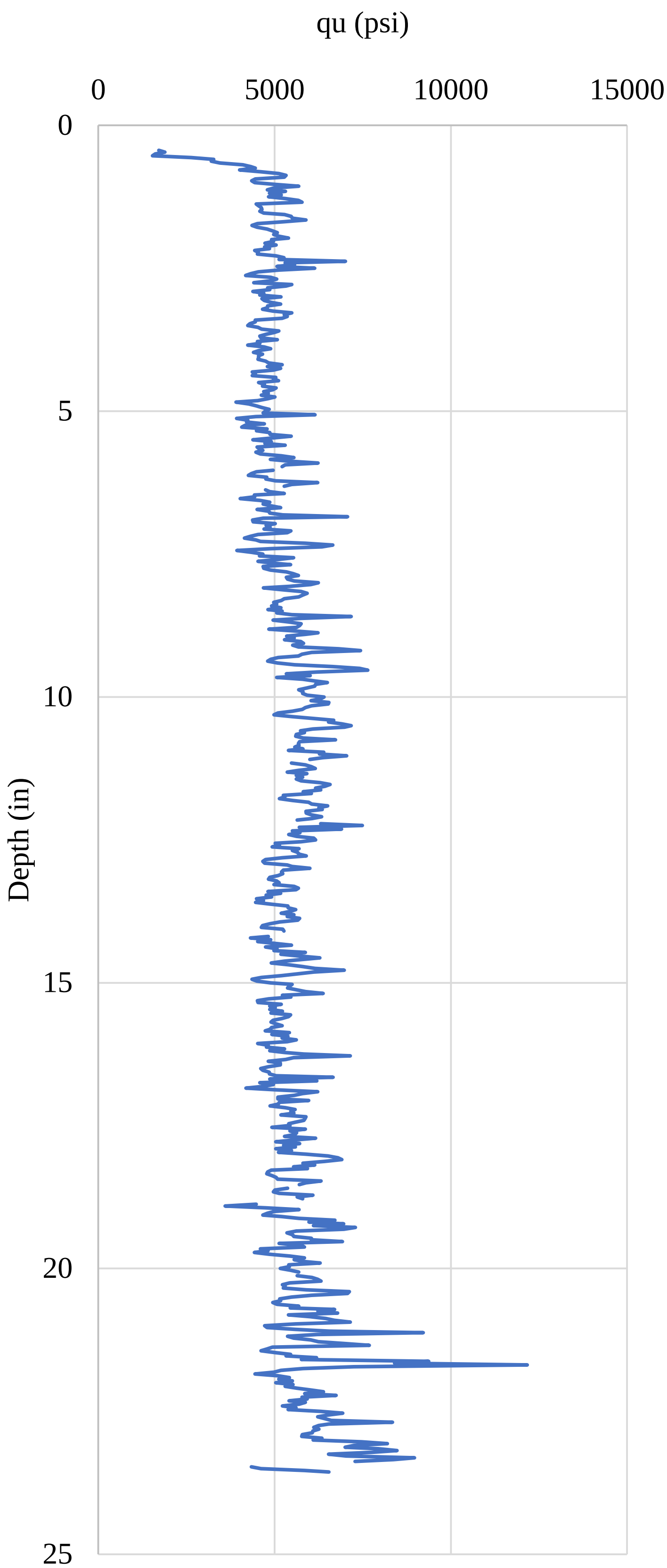


Figure F.57 Core 12-5

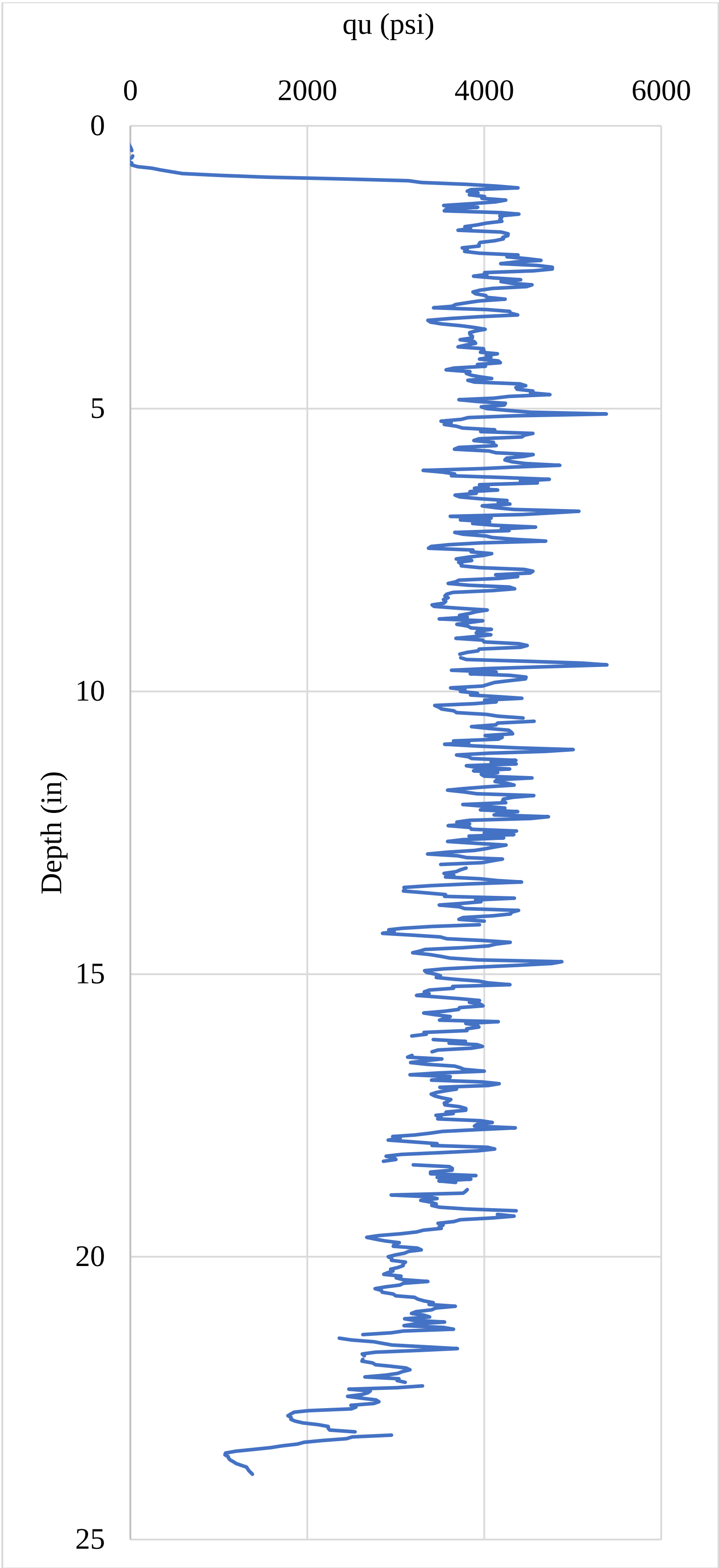


Figure F.58 Core 13-1

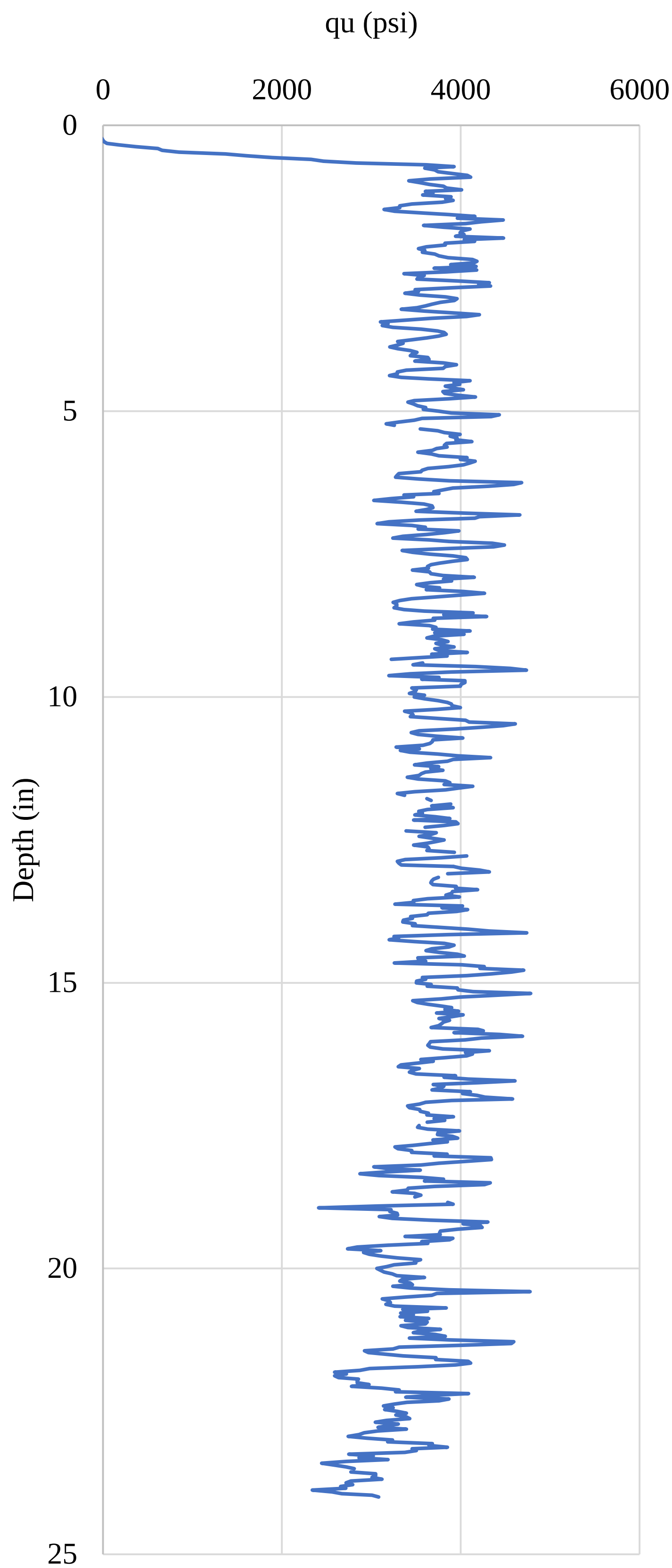


Figure F.59 Core 13-2

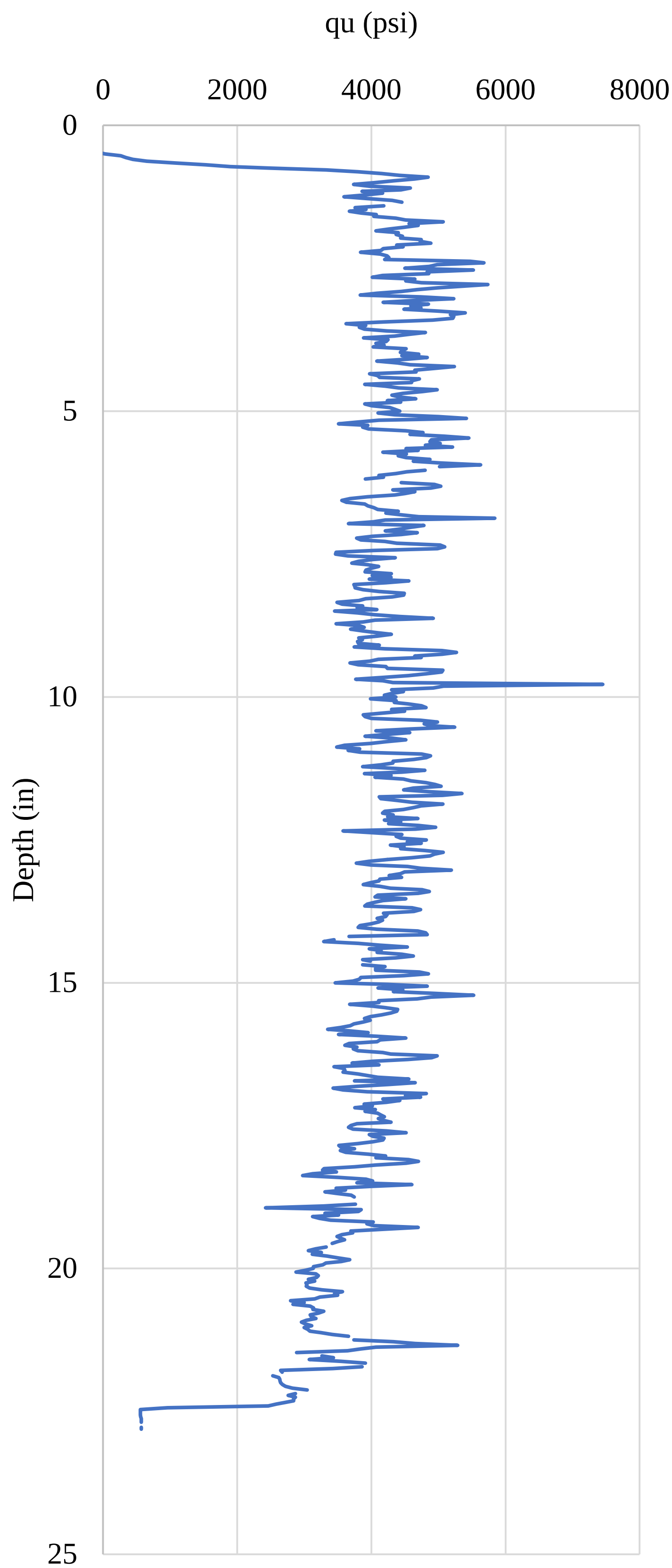


Figure F.60 Core 13-3

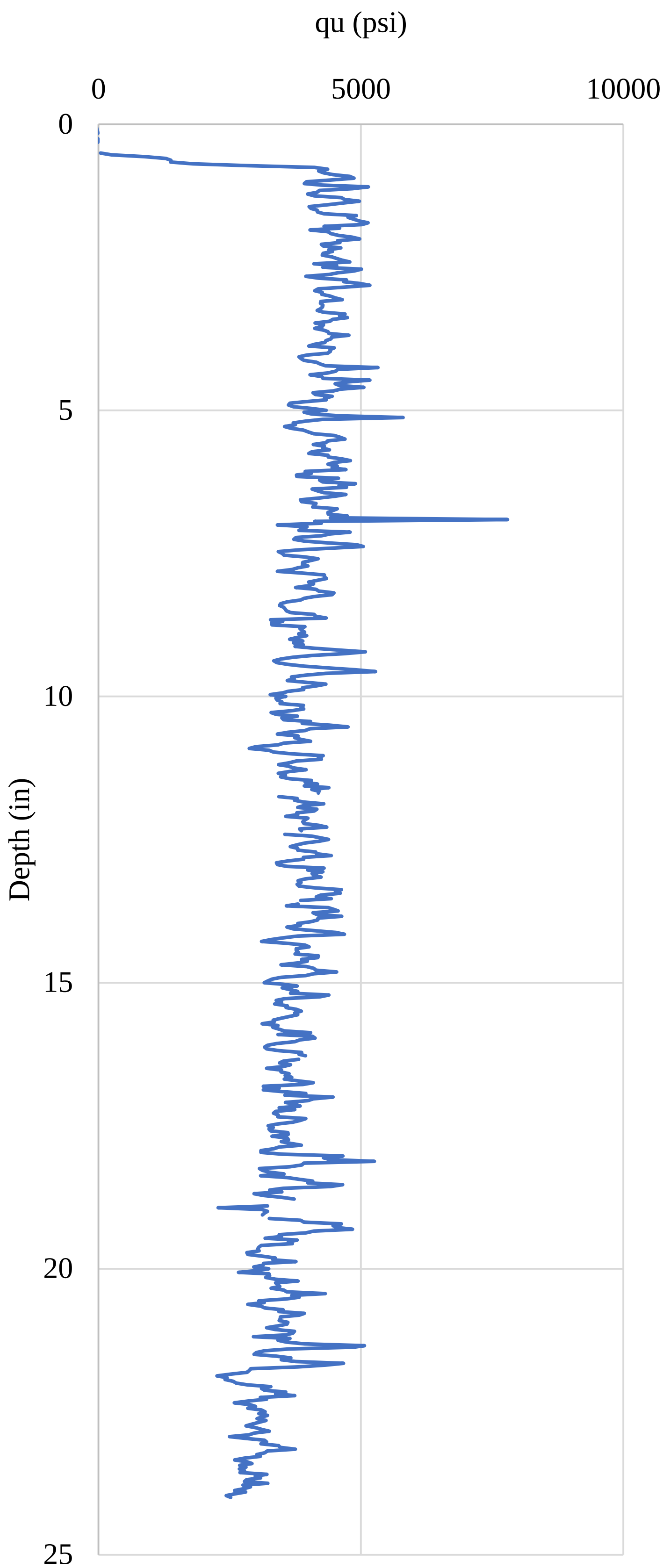


Figure F.61 Core 13-4

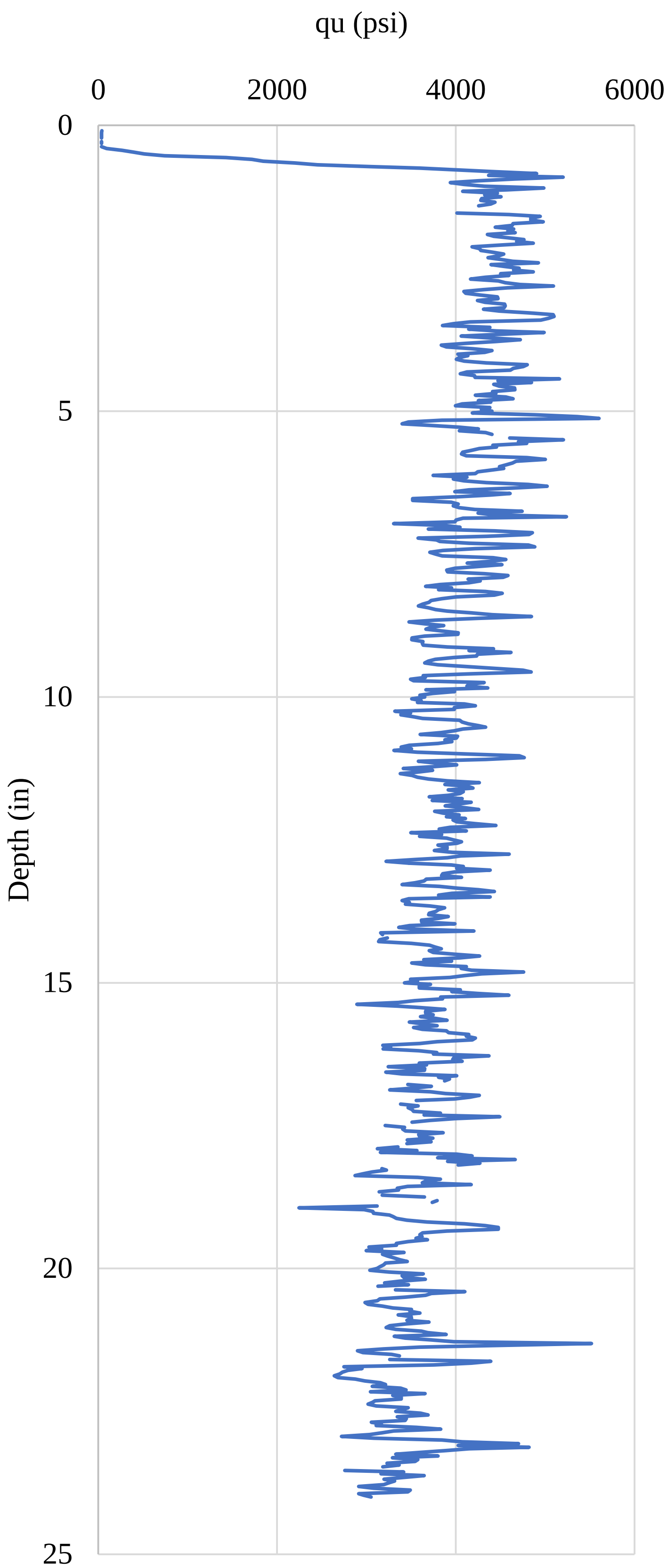


Figure F.62 Core 13-5

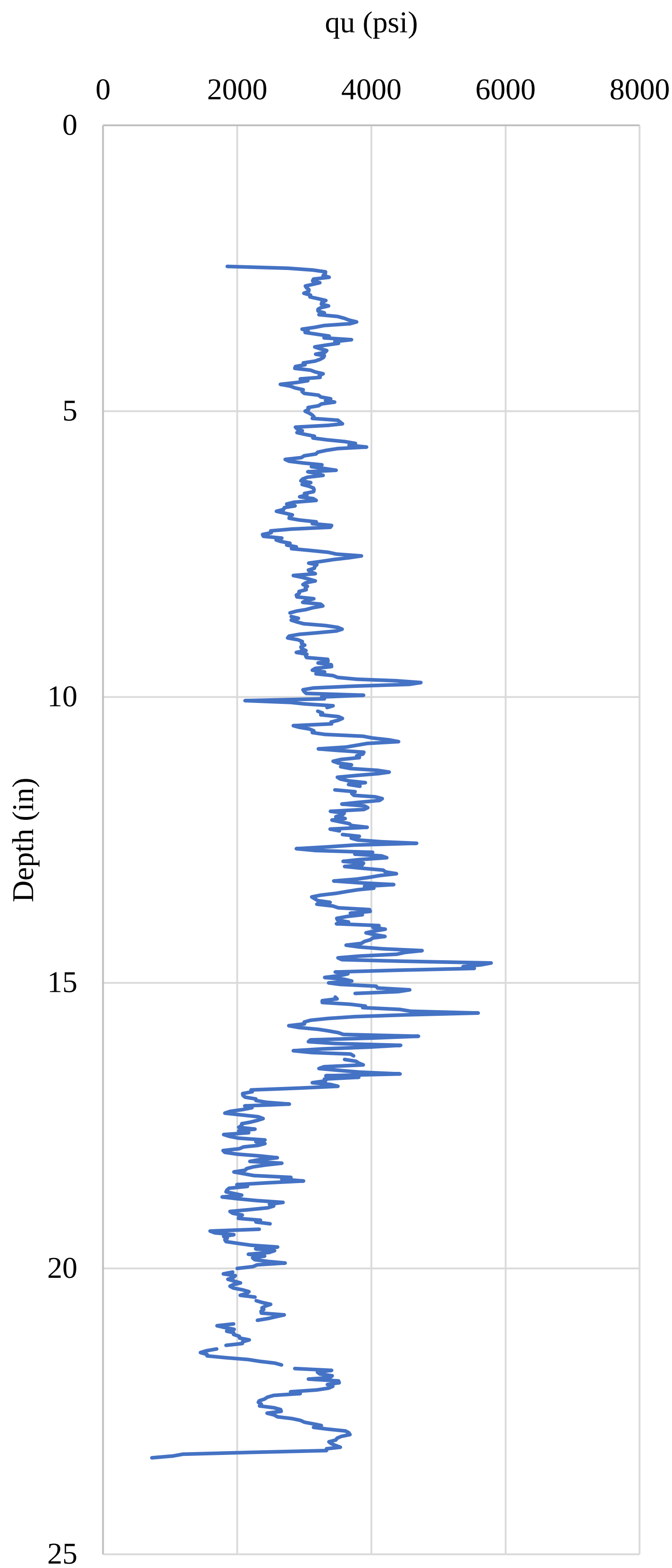


Figure F.63 Core 14-1

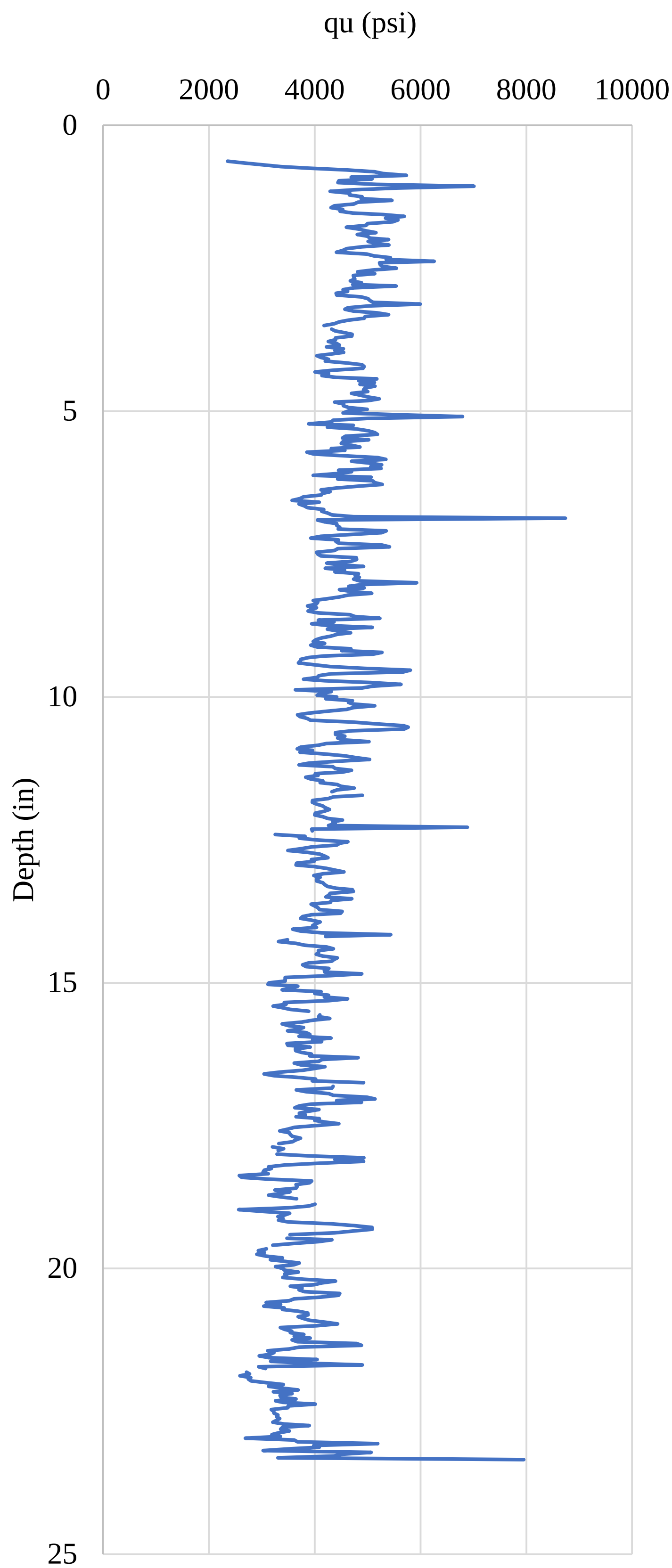
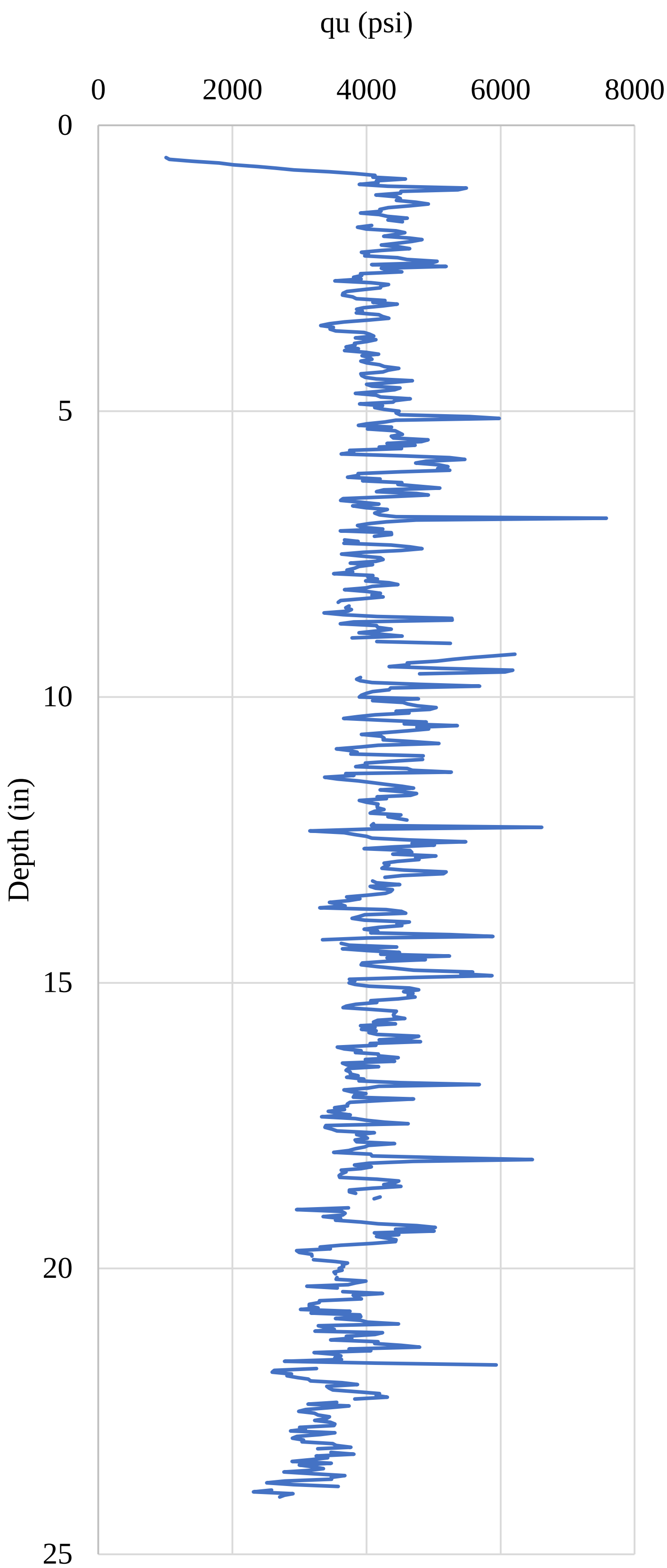


Figure F.64 Core 14-2



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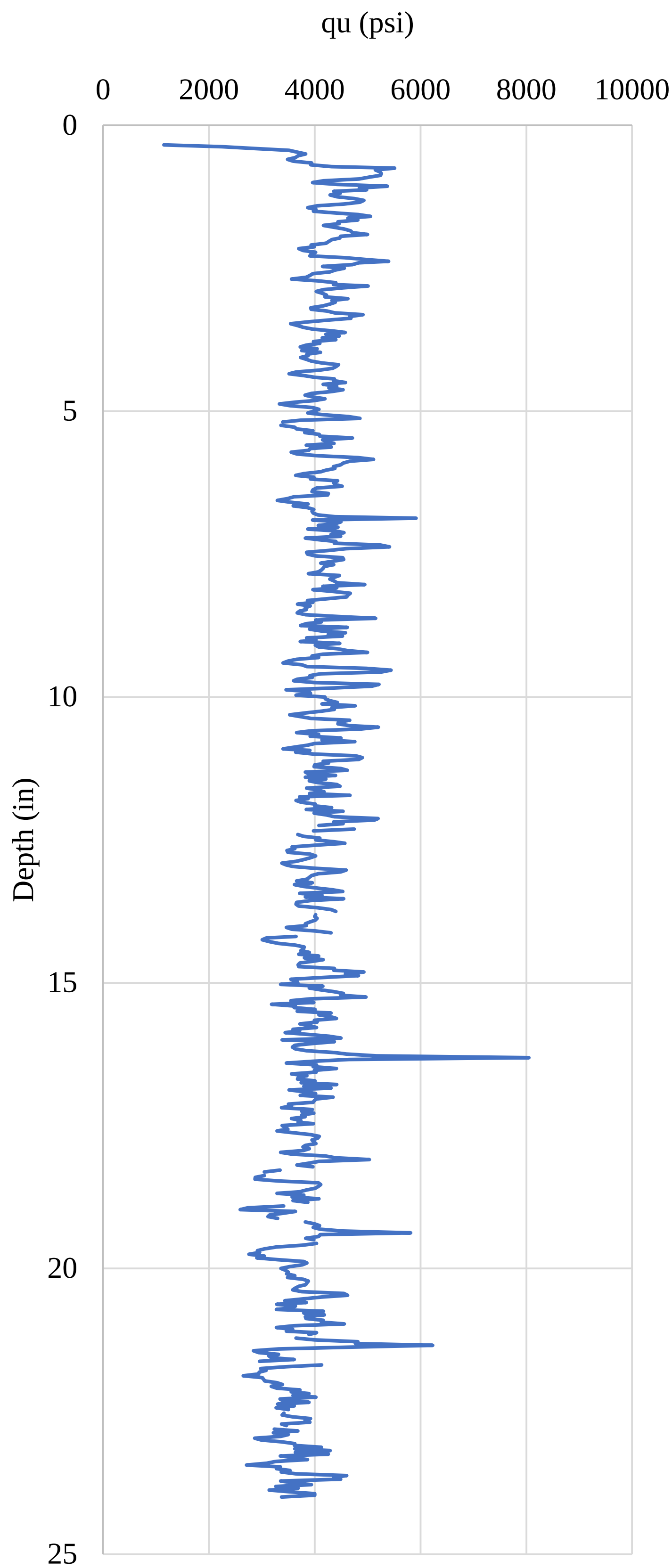
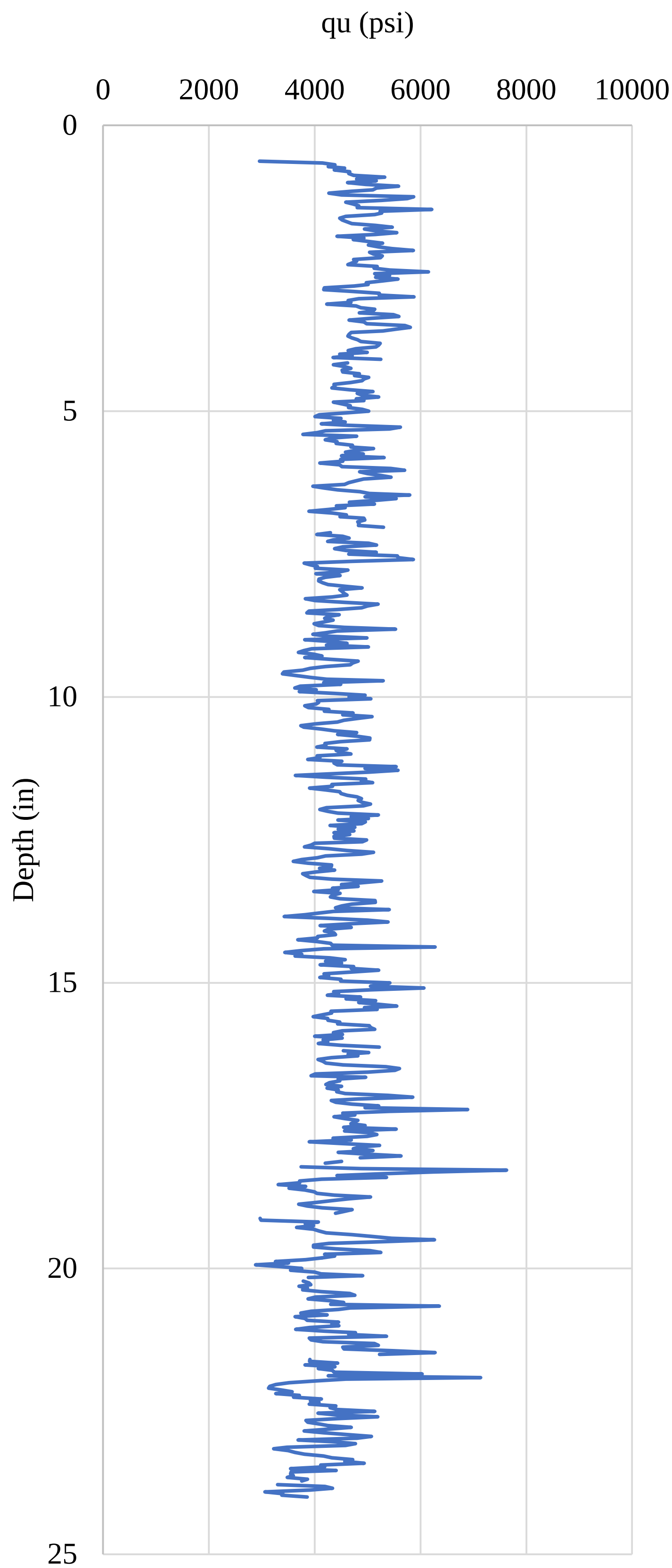


Figure F.66 Core 14-4



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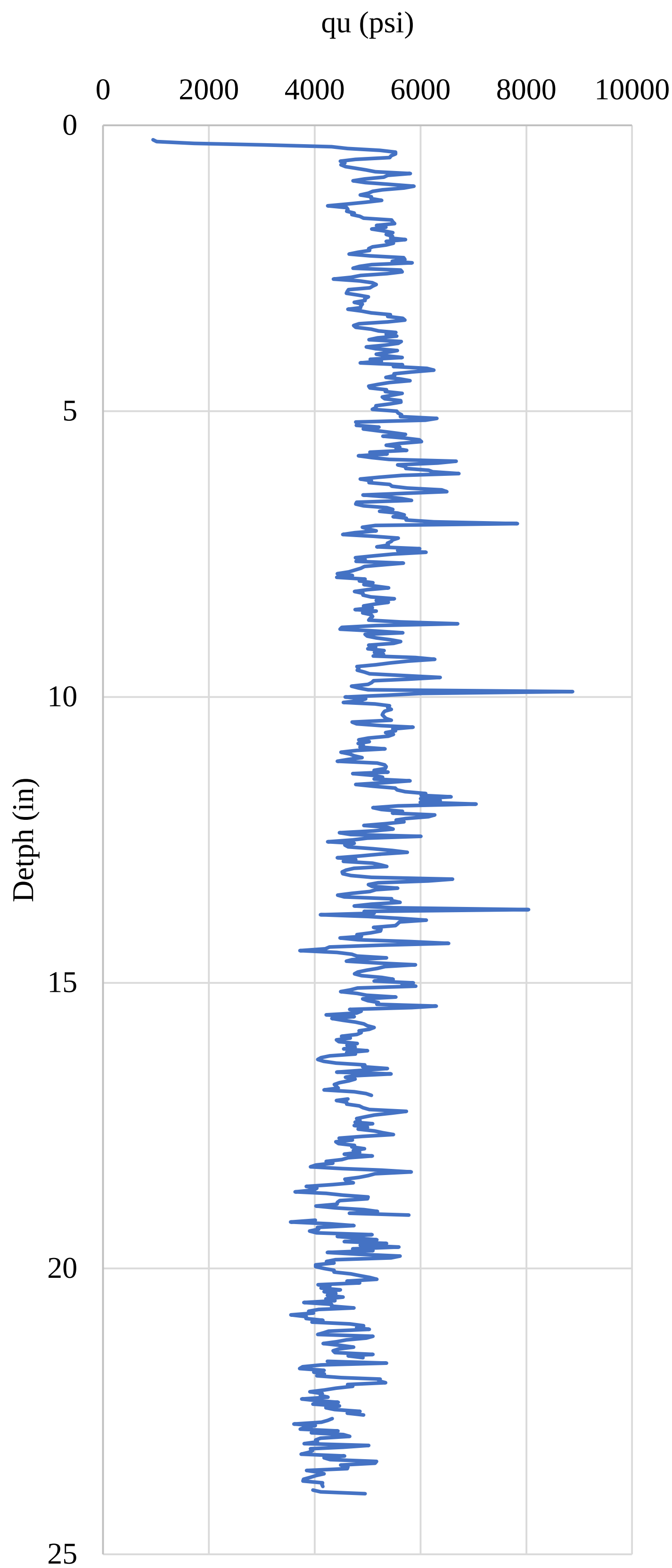


Figure F.68 Core 15-1

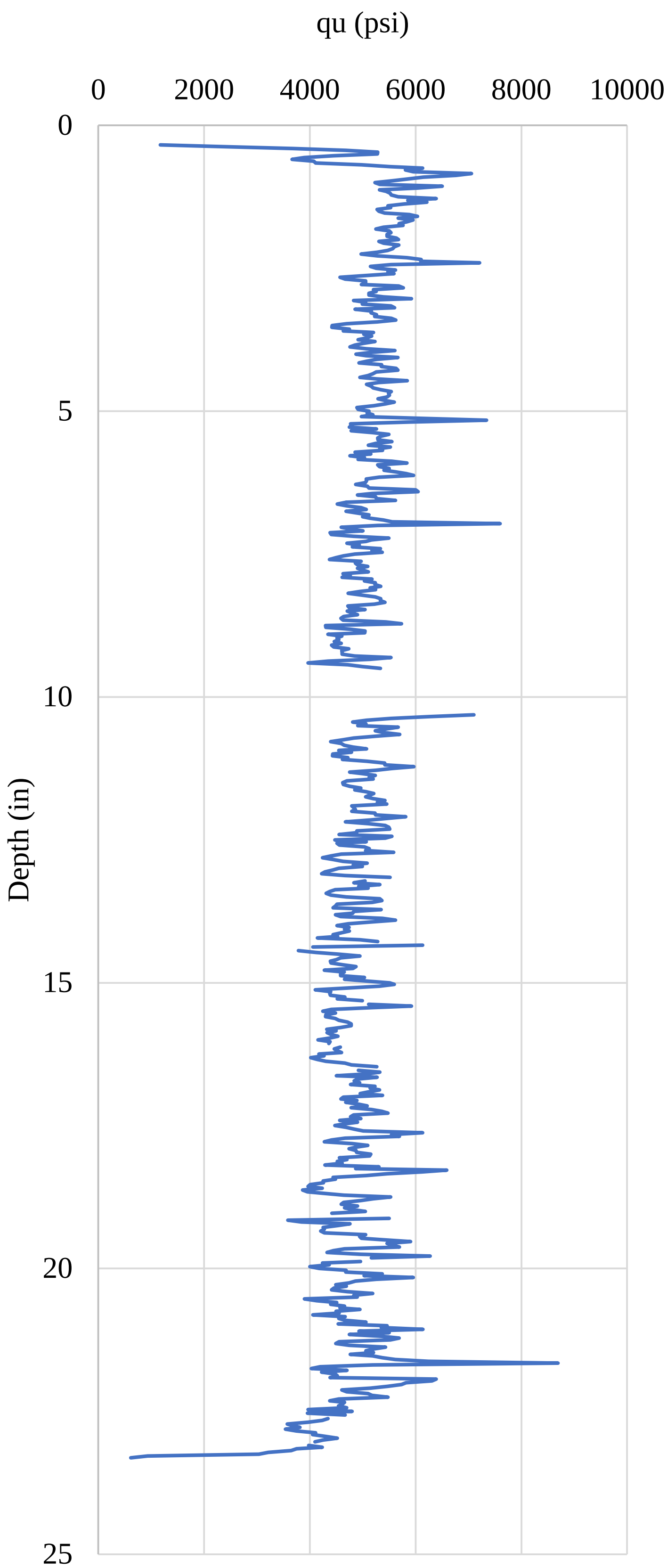


Figure F.69 Core 15-2

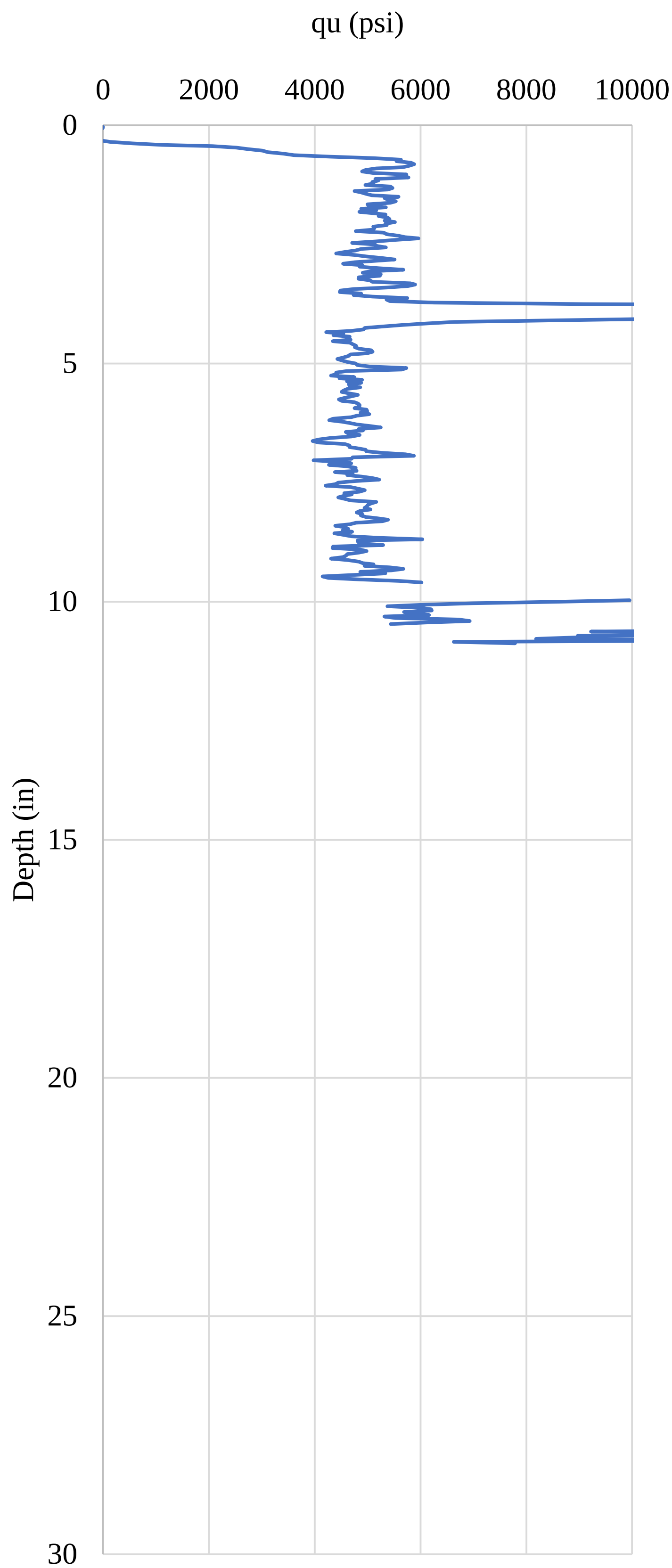


Figure F.70 Core 15-3

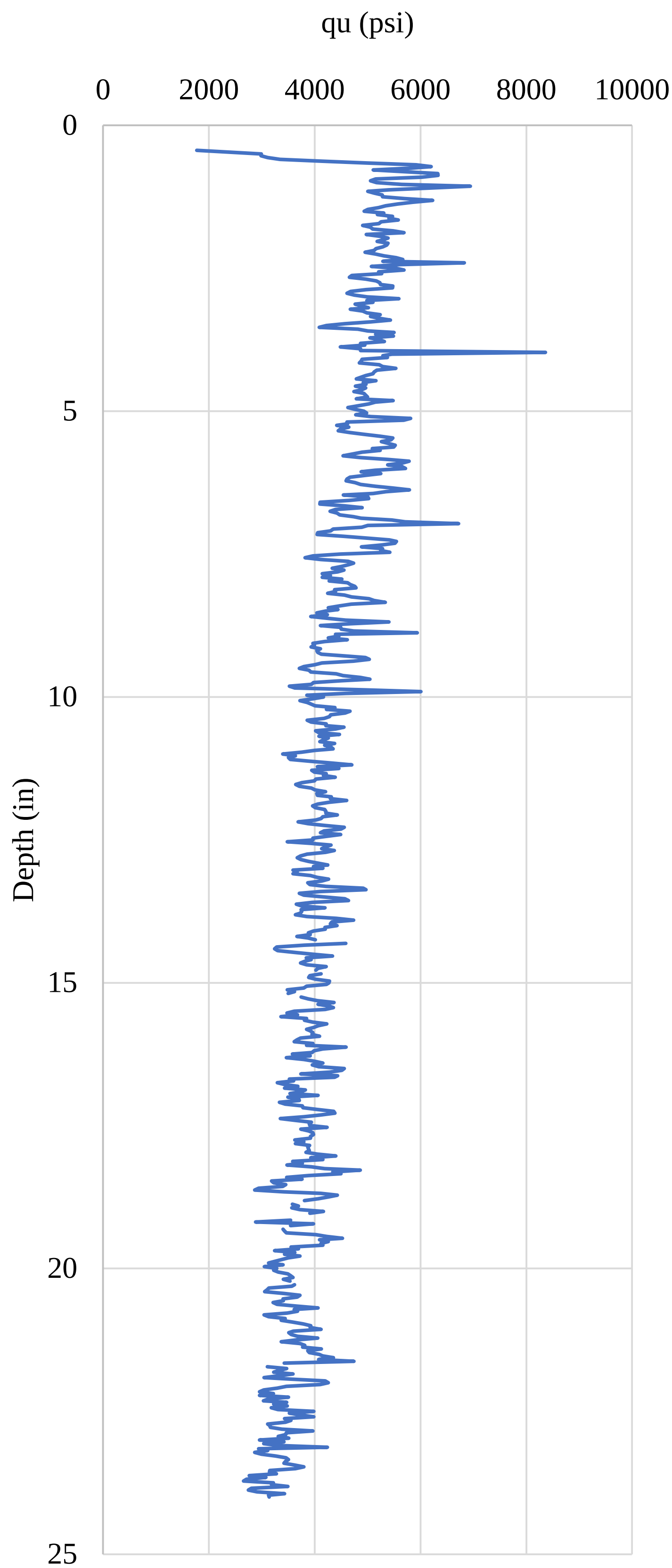


Figure F.71 Core 15-4

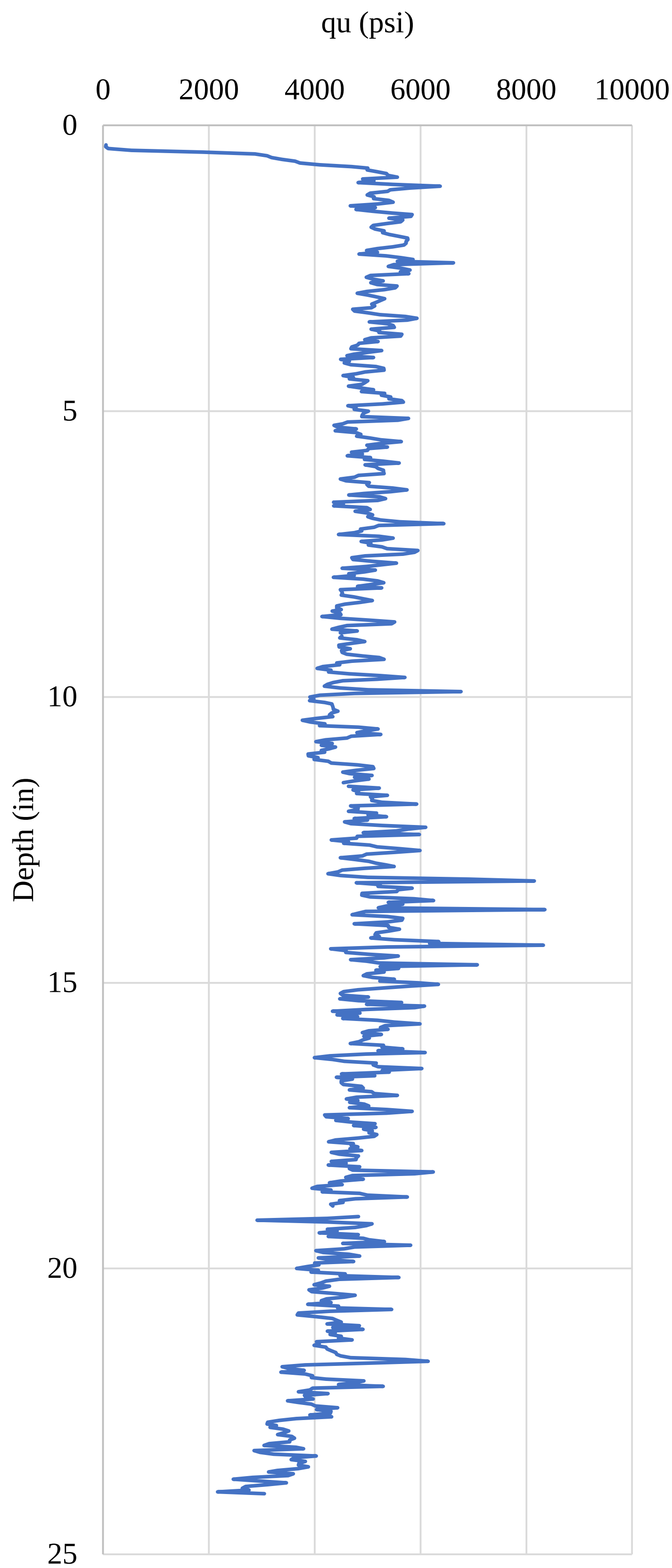


Figure F.72 Core 15-5

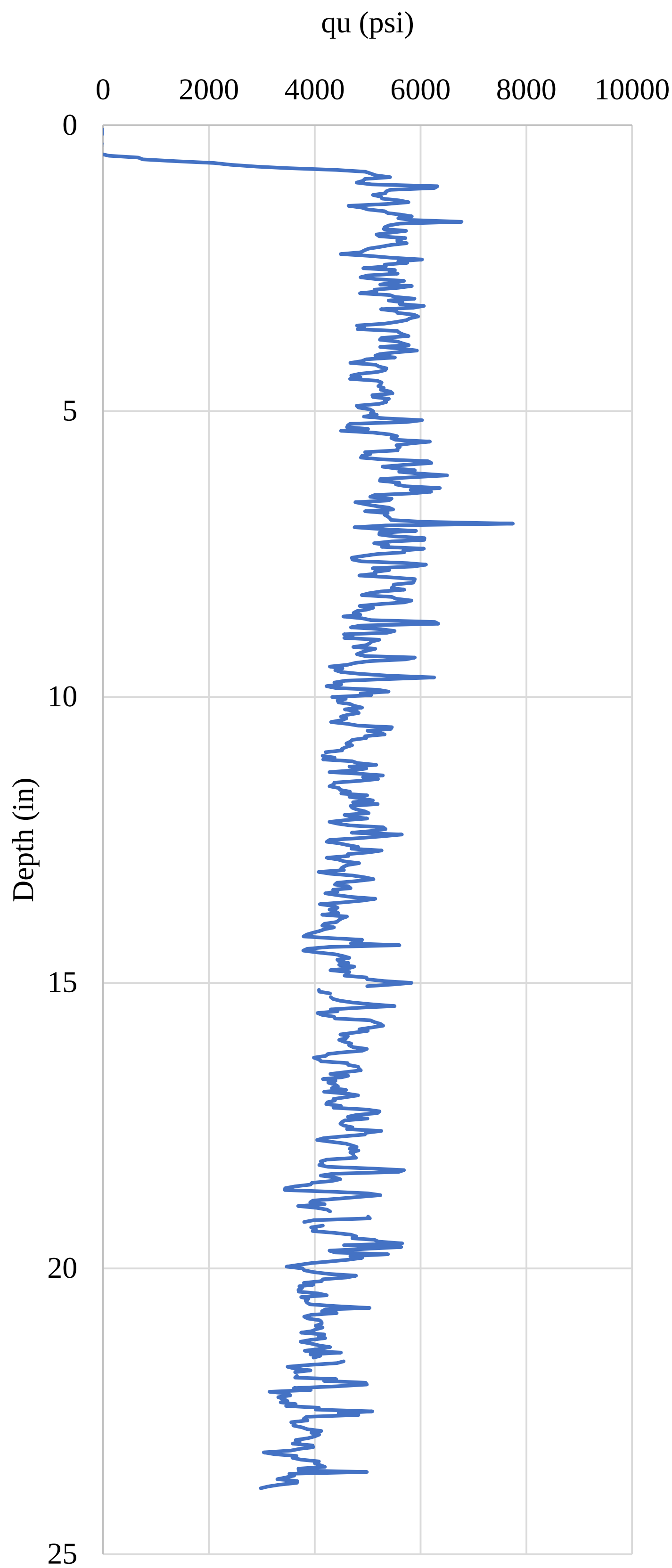
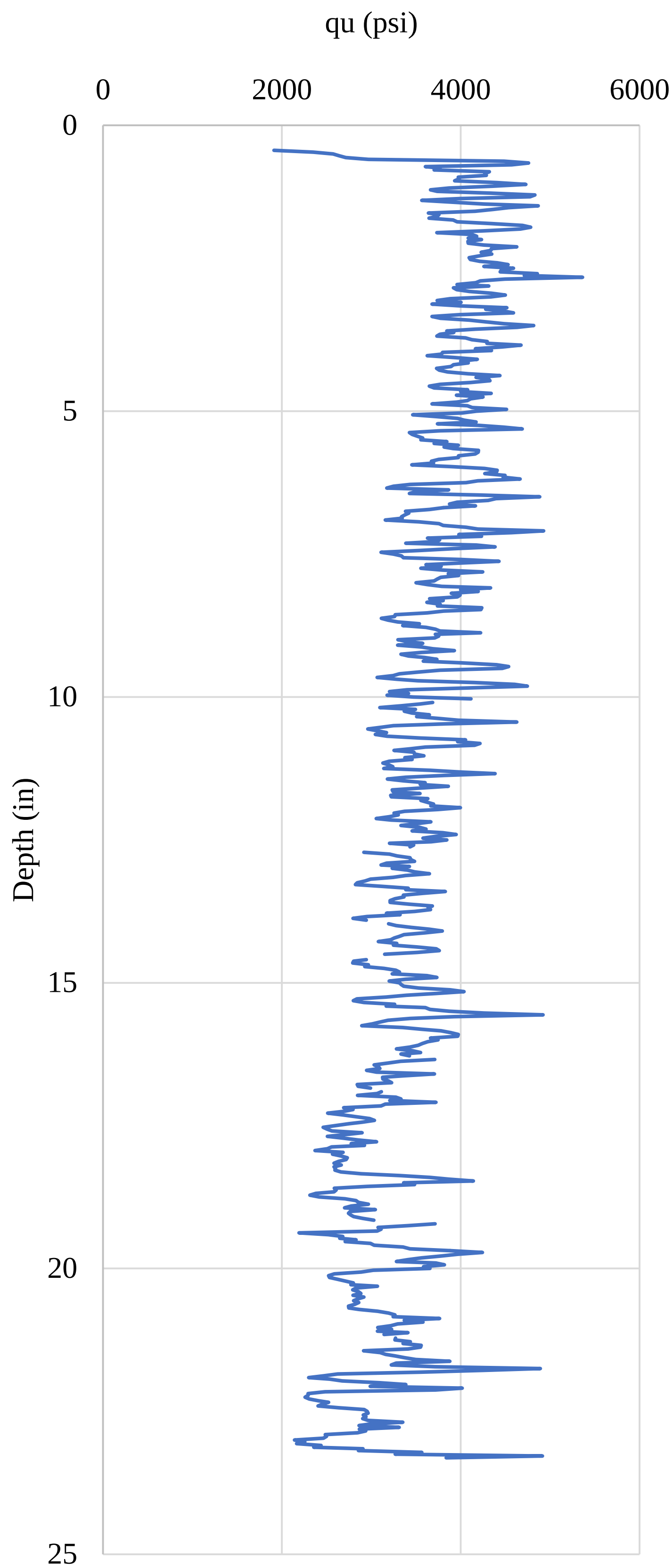


Figure F.73 Core 15-6



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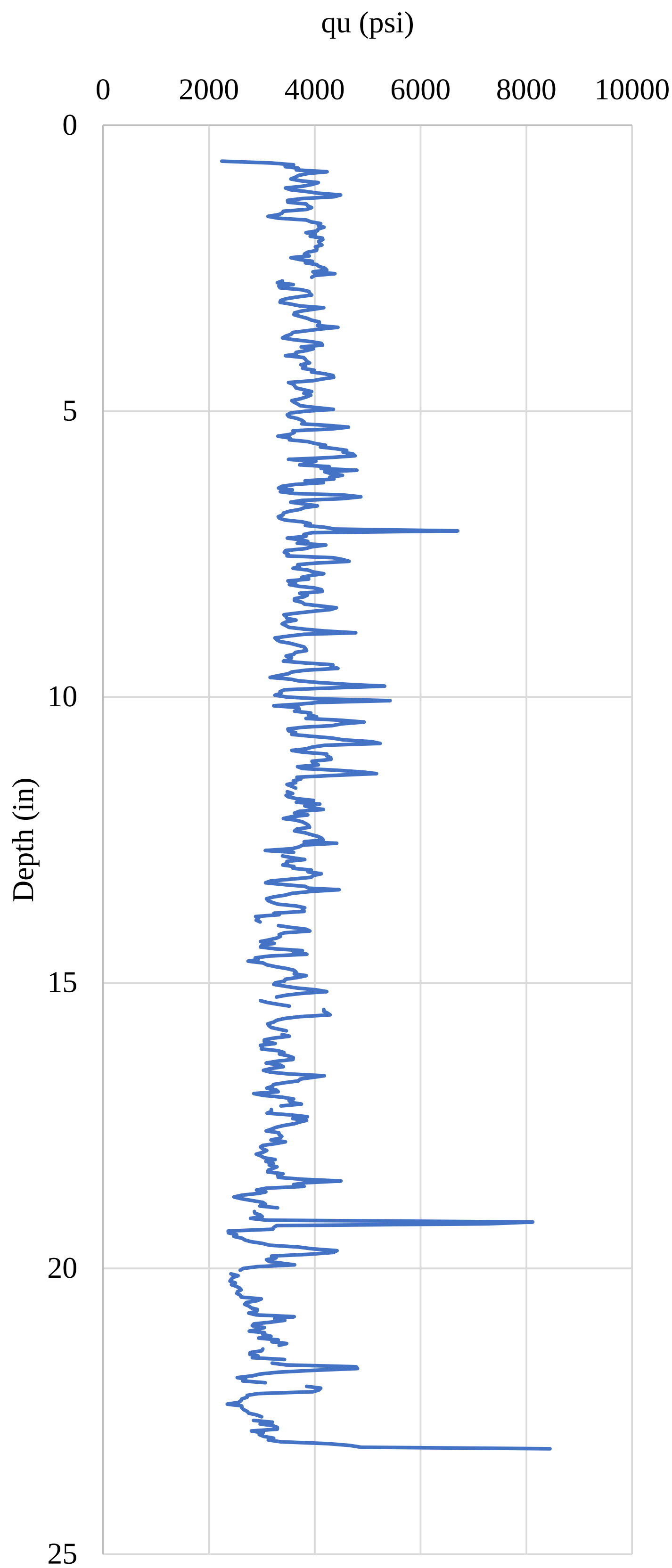
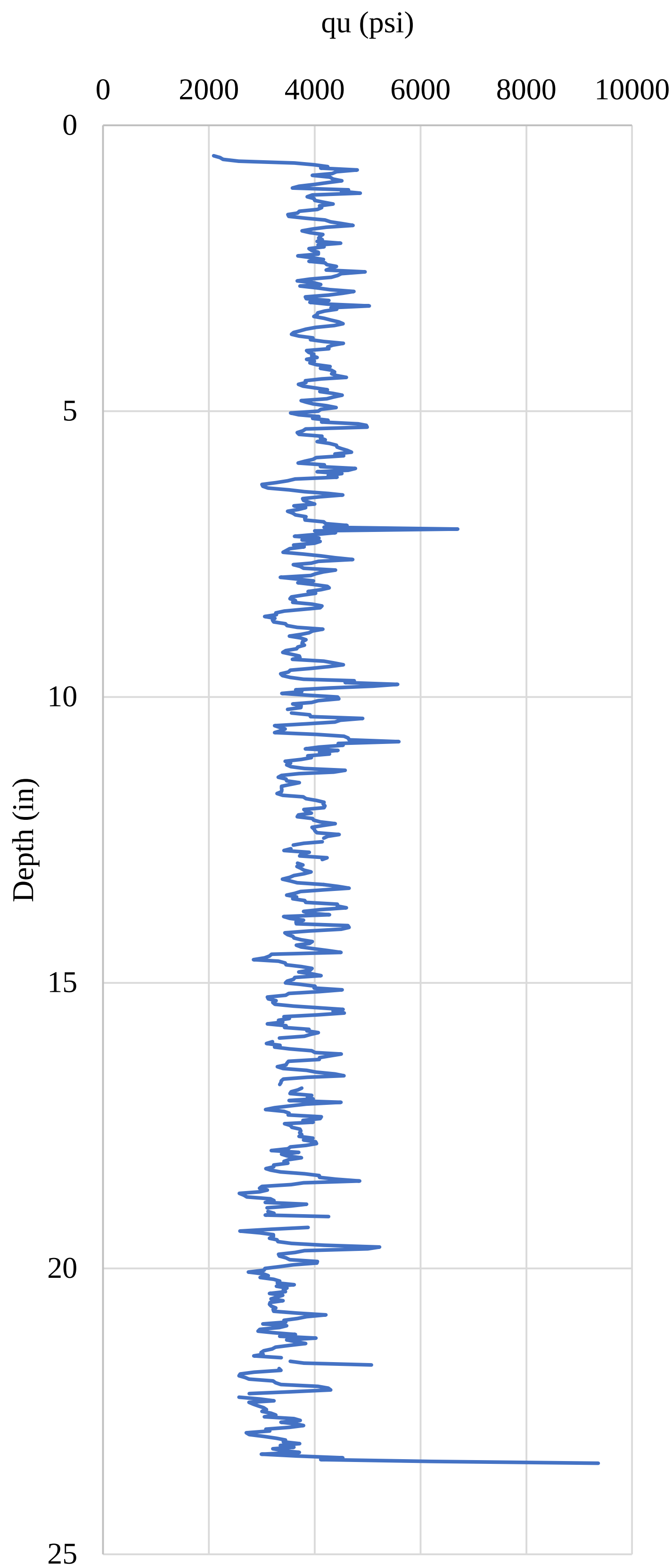


Figure F.75 Core 16-2



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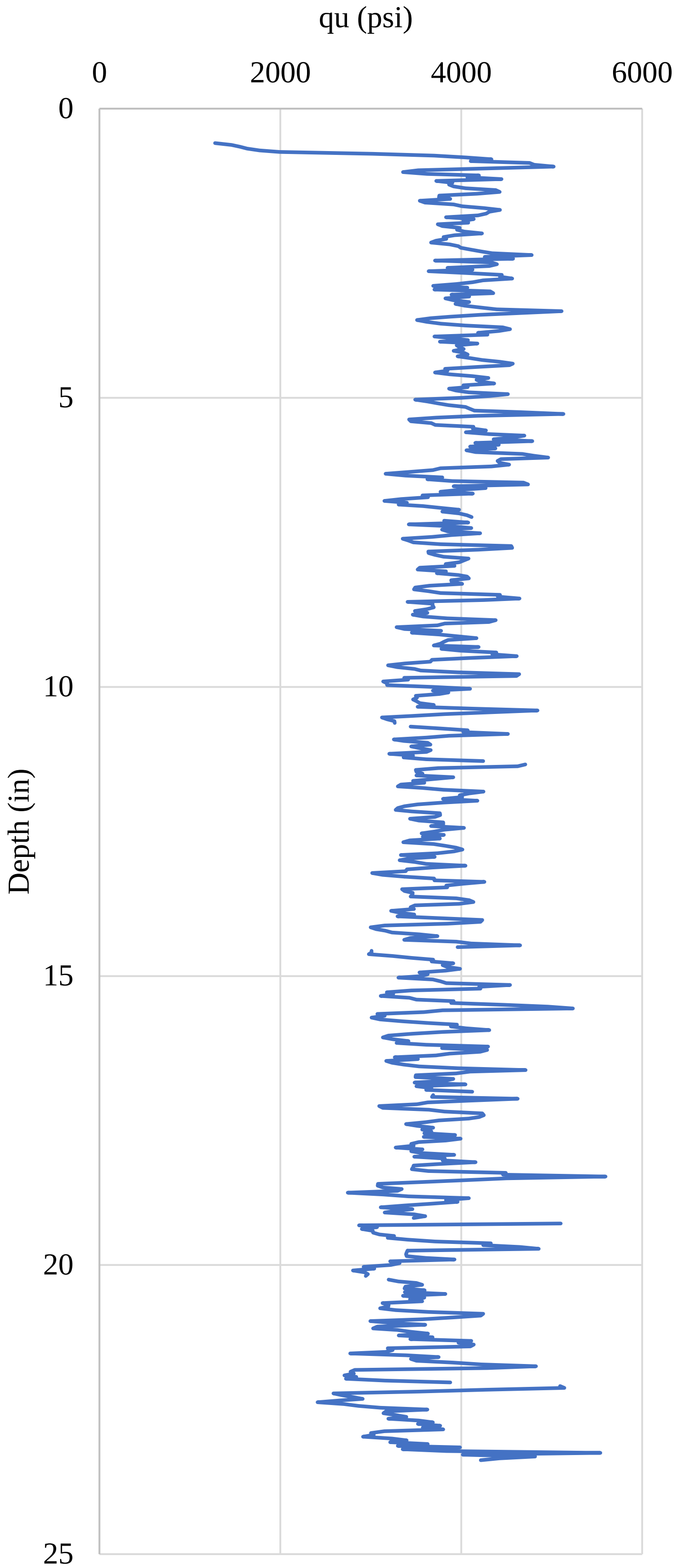


Figure F.77 Core 16-4

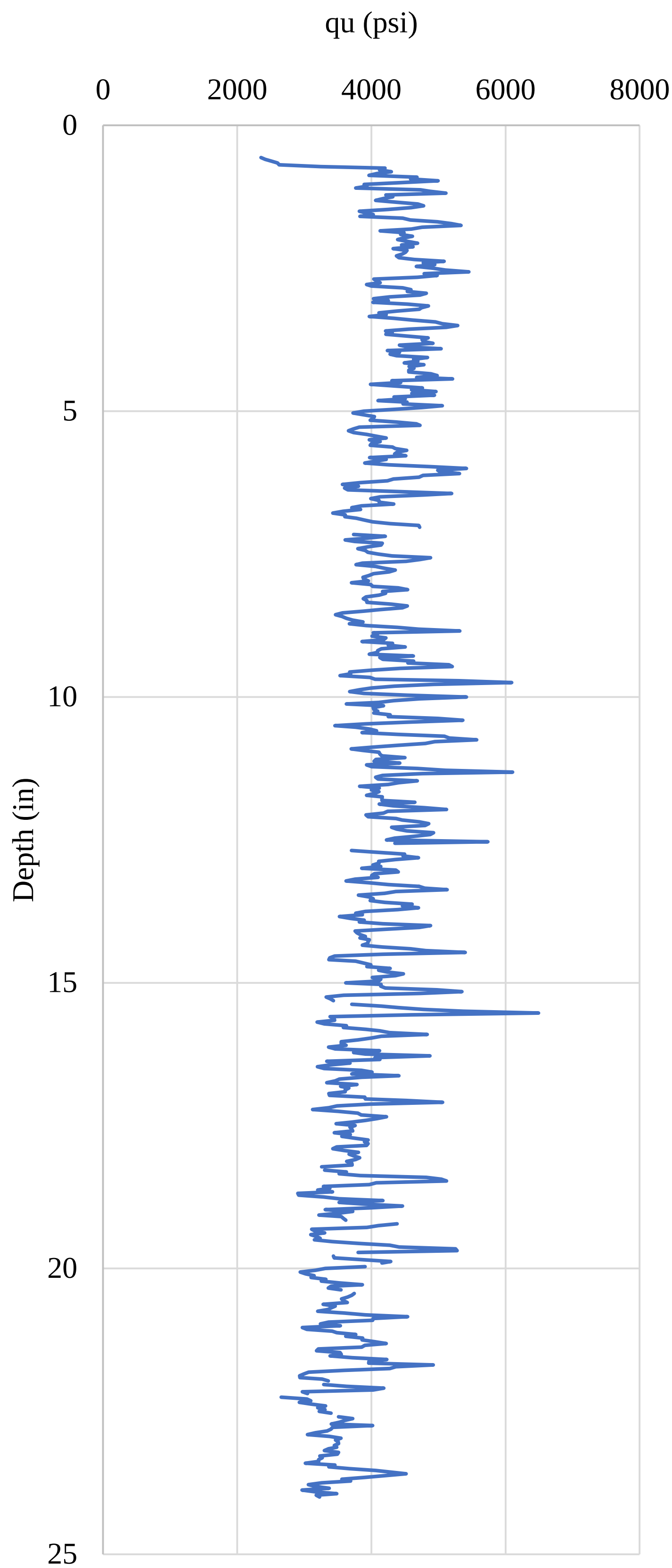
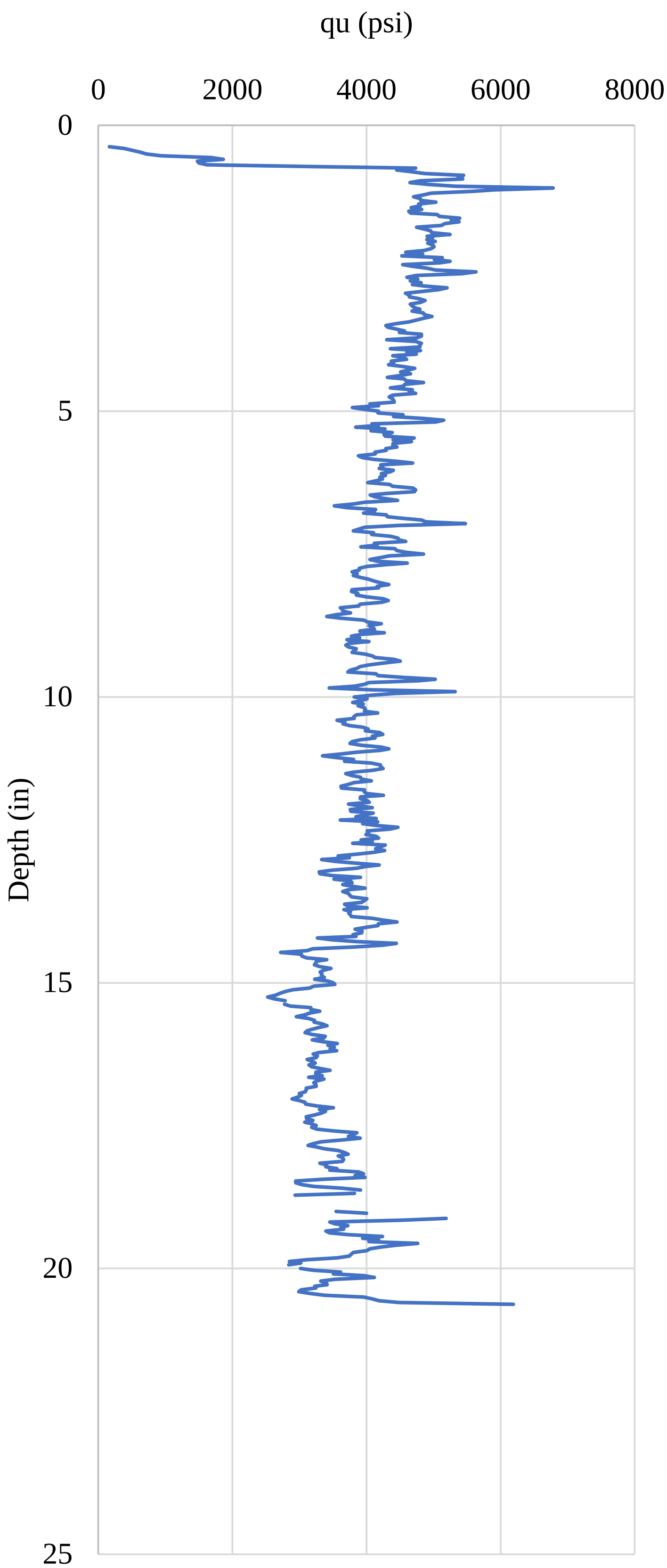


Figure F.78 Core 16-5



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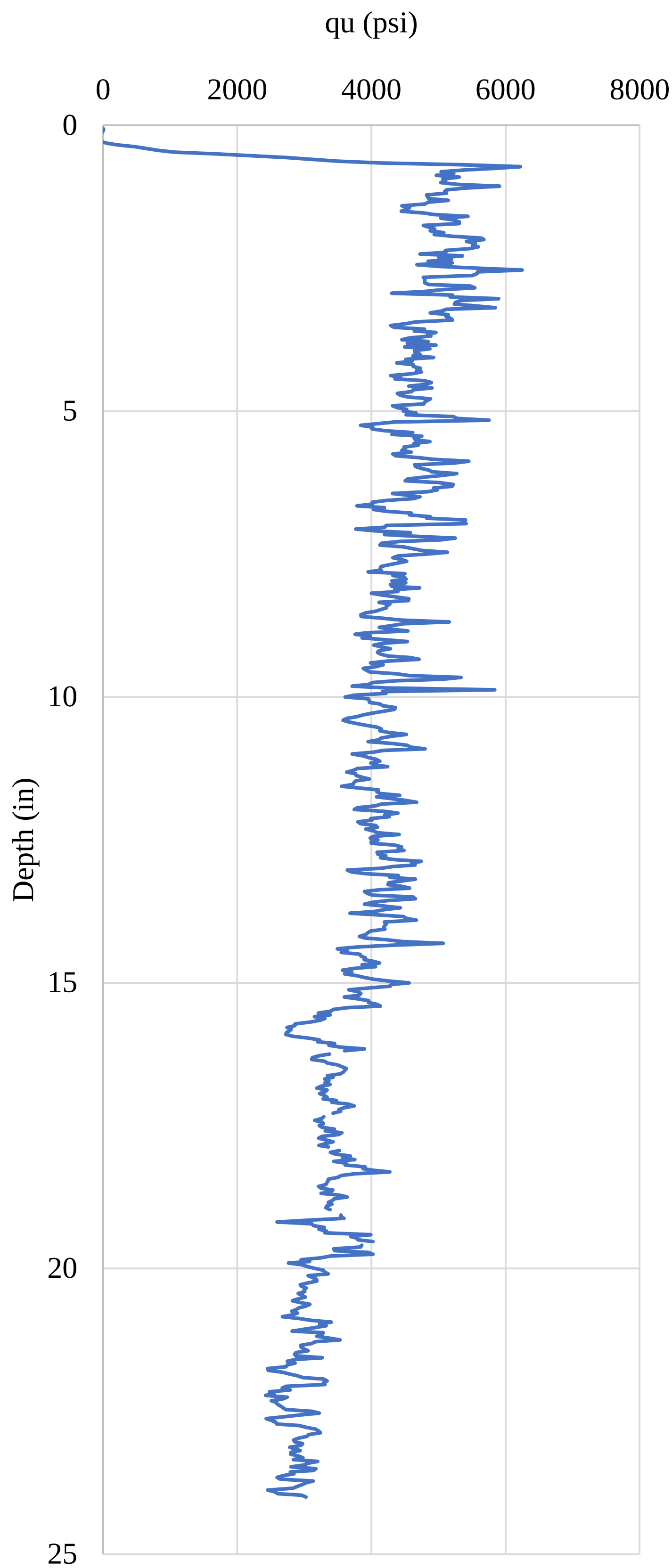

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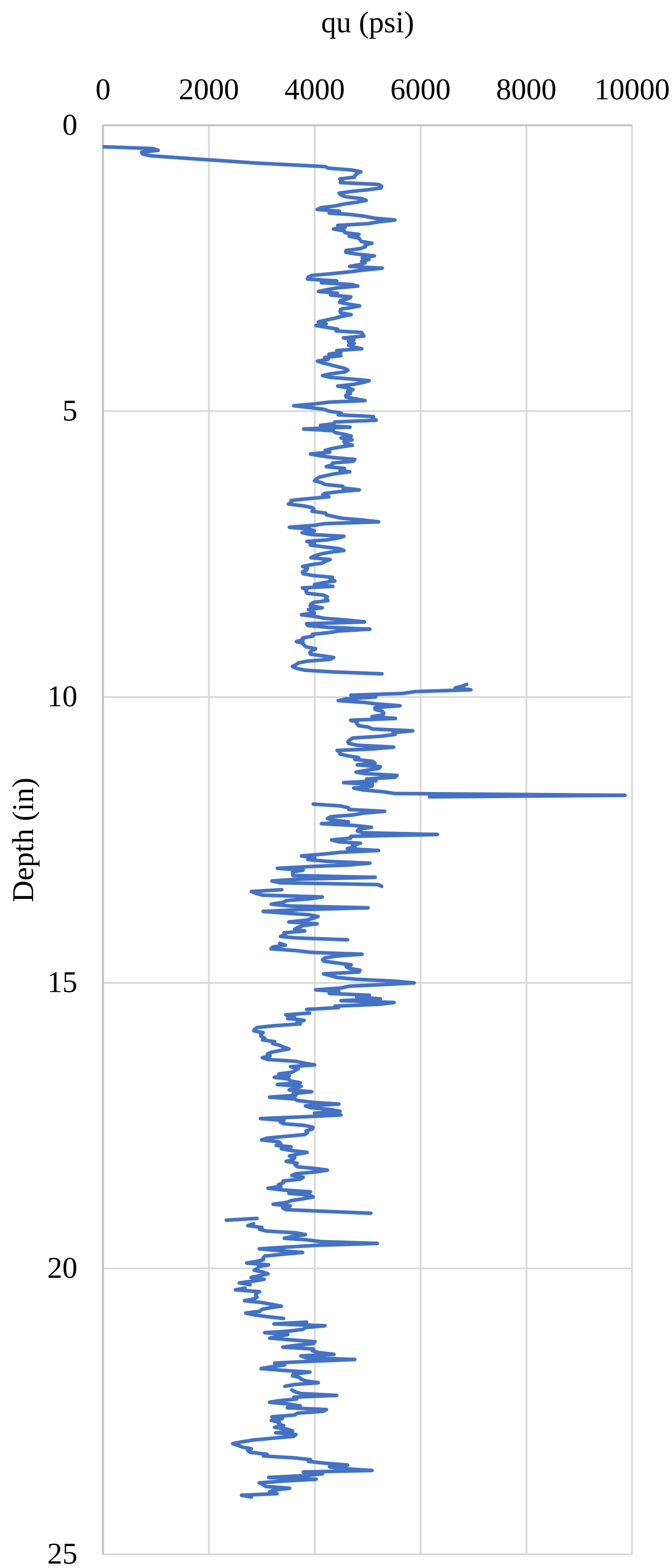


Figure F.81 Core 17-3

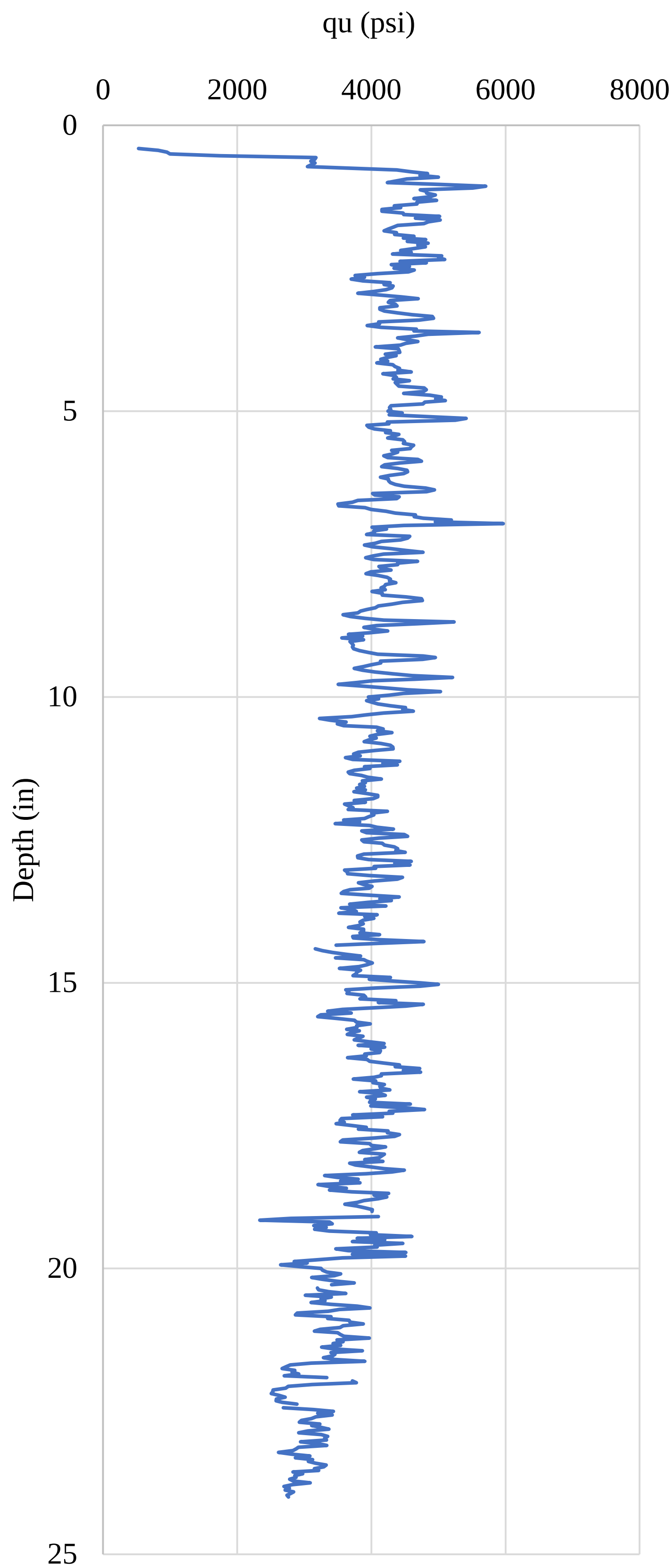


Figure F.82 Core 17-4

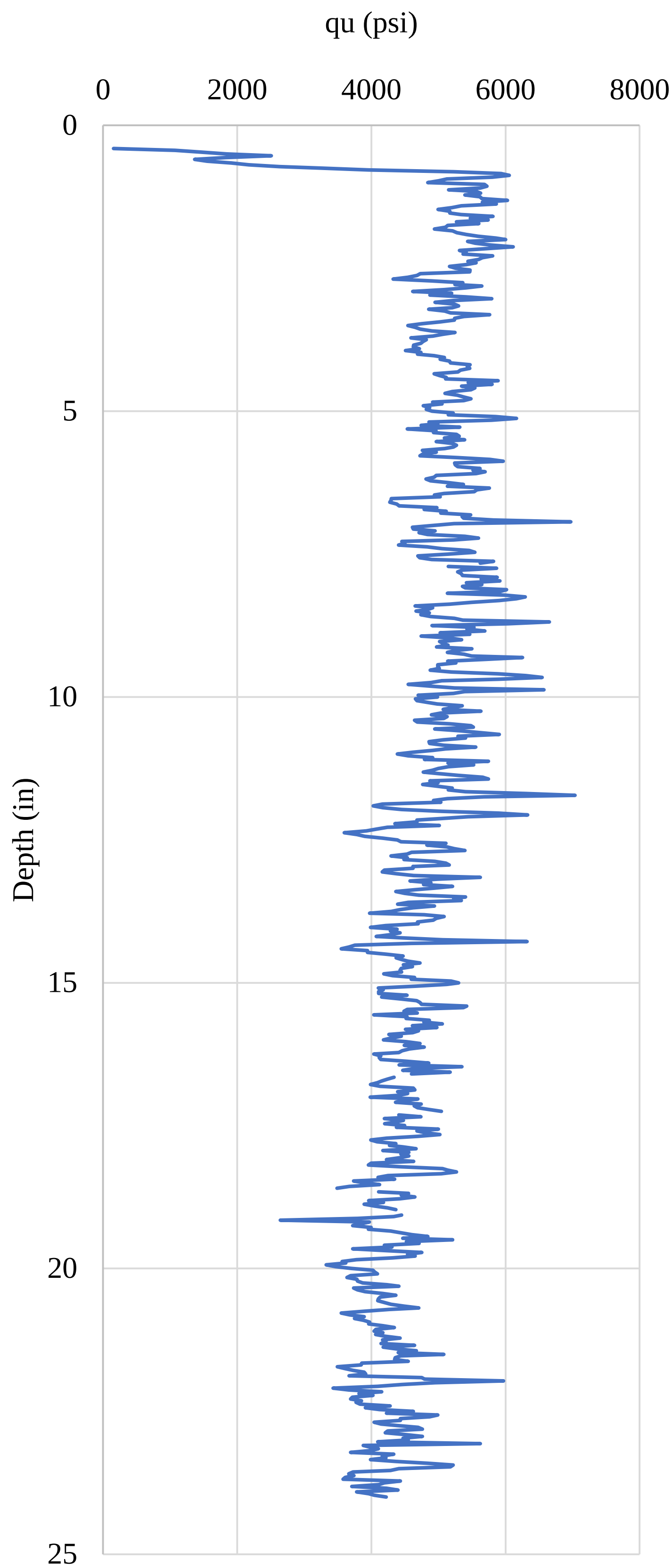
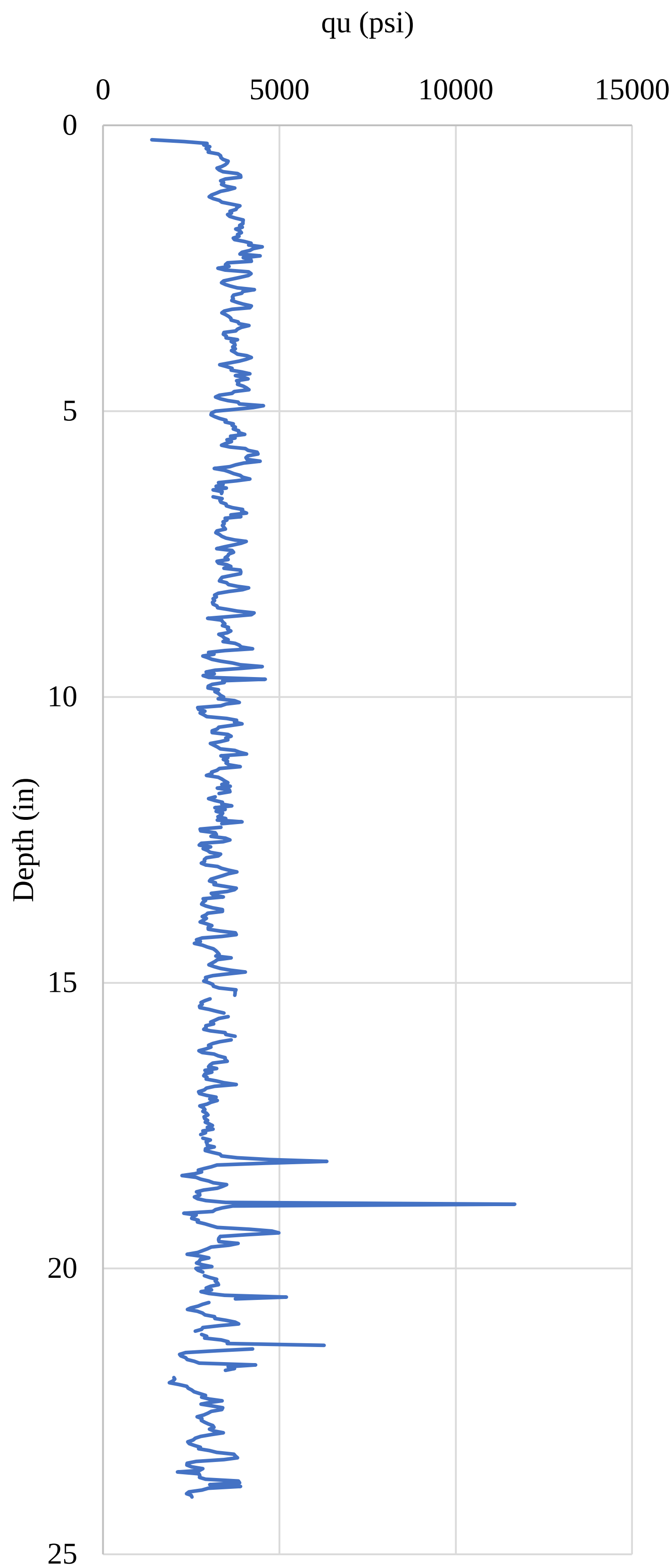


Figure F.83 Core 17-5



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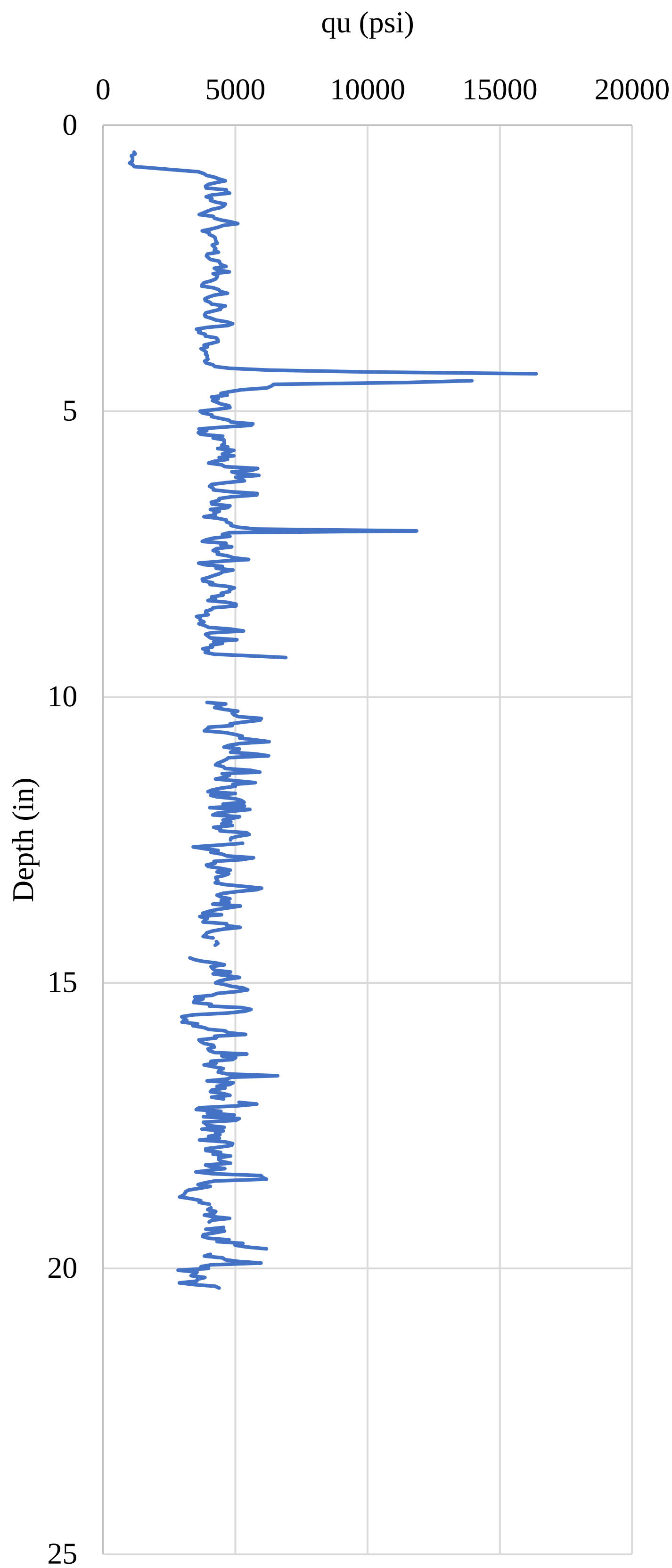
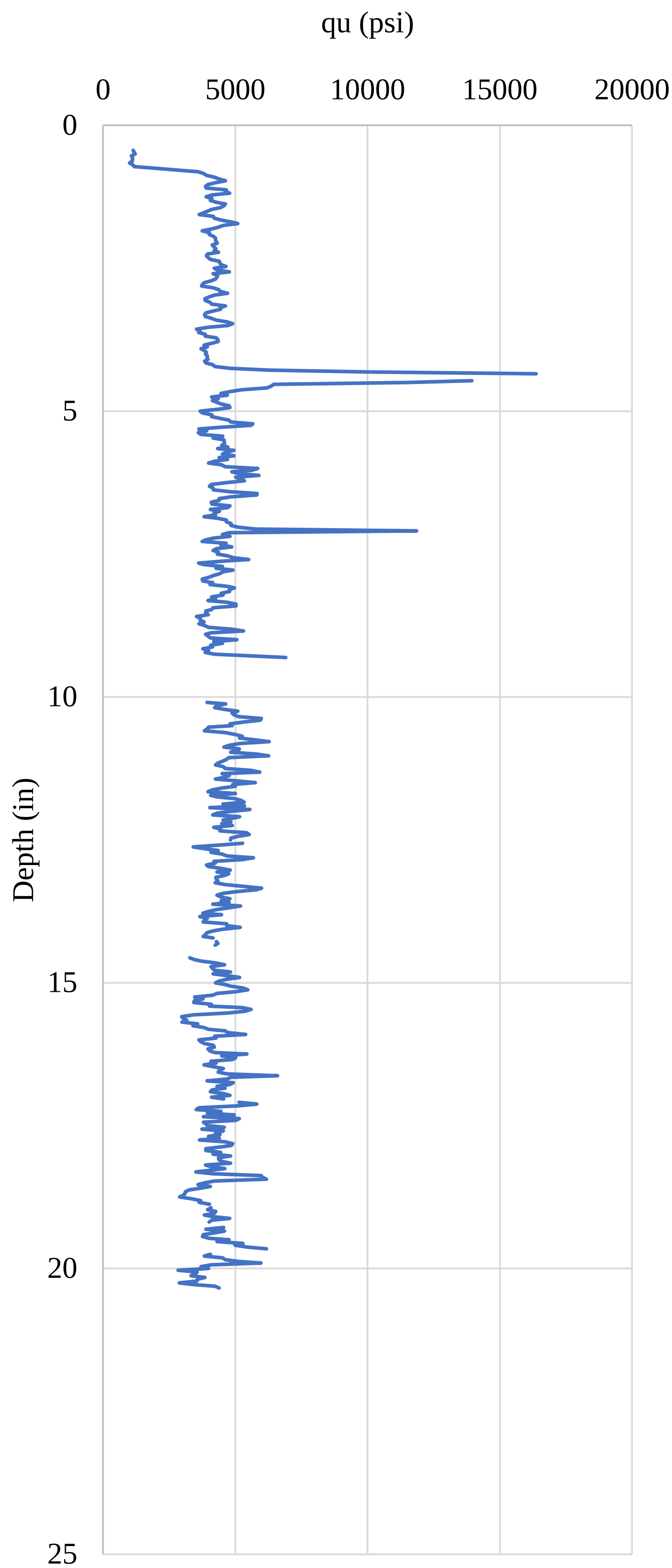


Figure F.85 Core 18-2



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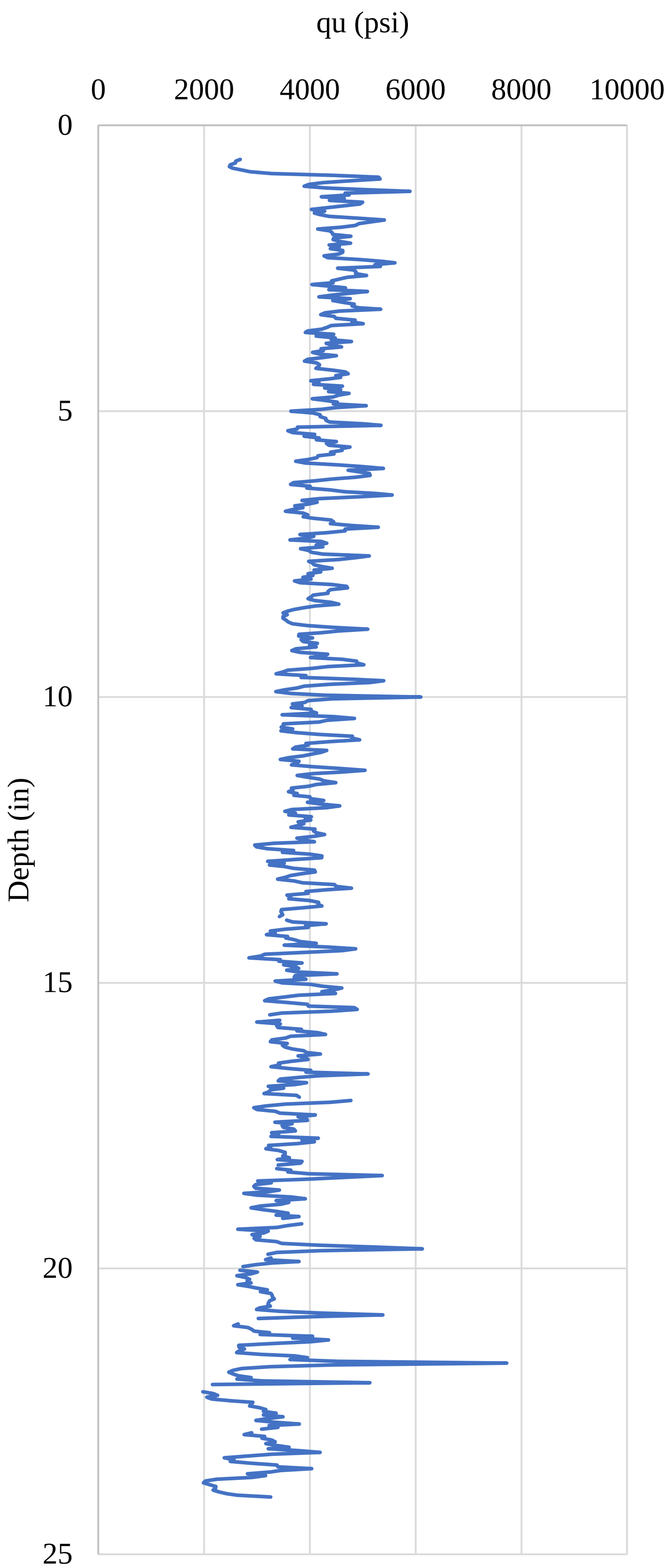

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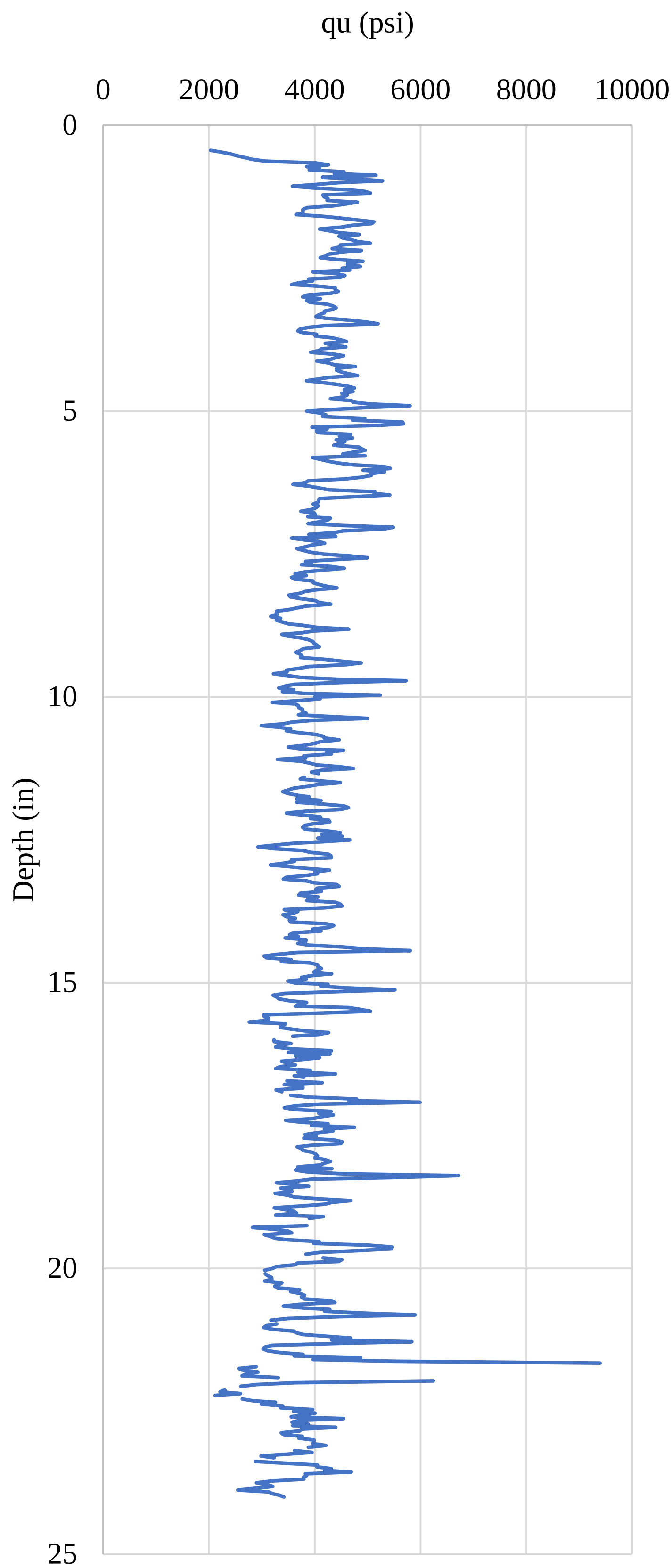

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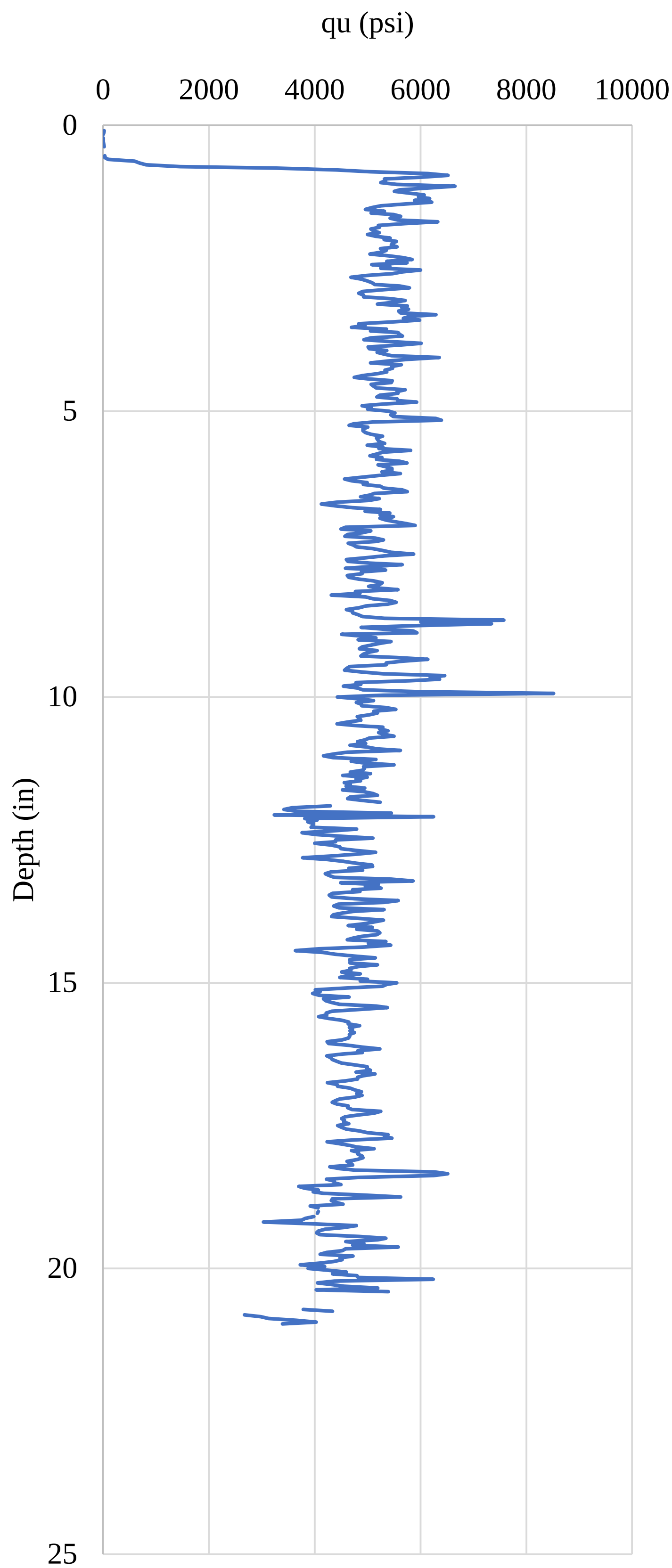


Figure F.89 Core 19-1

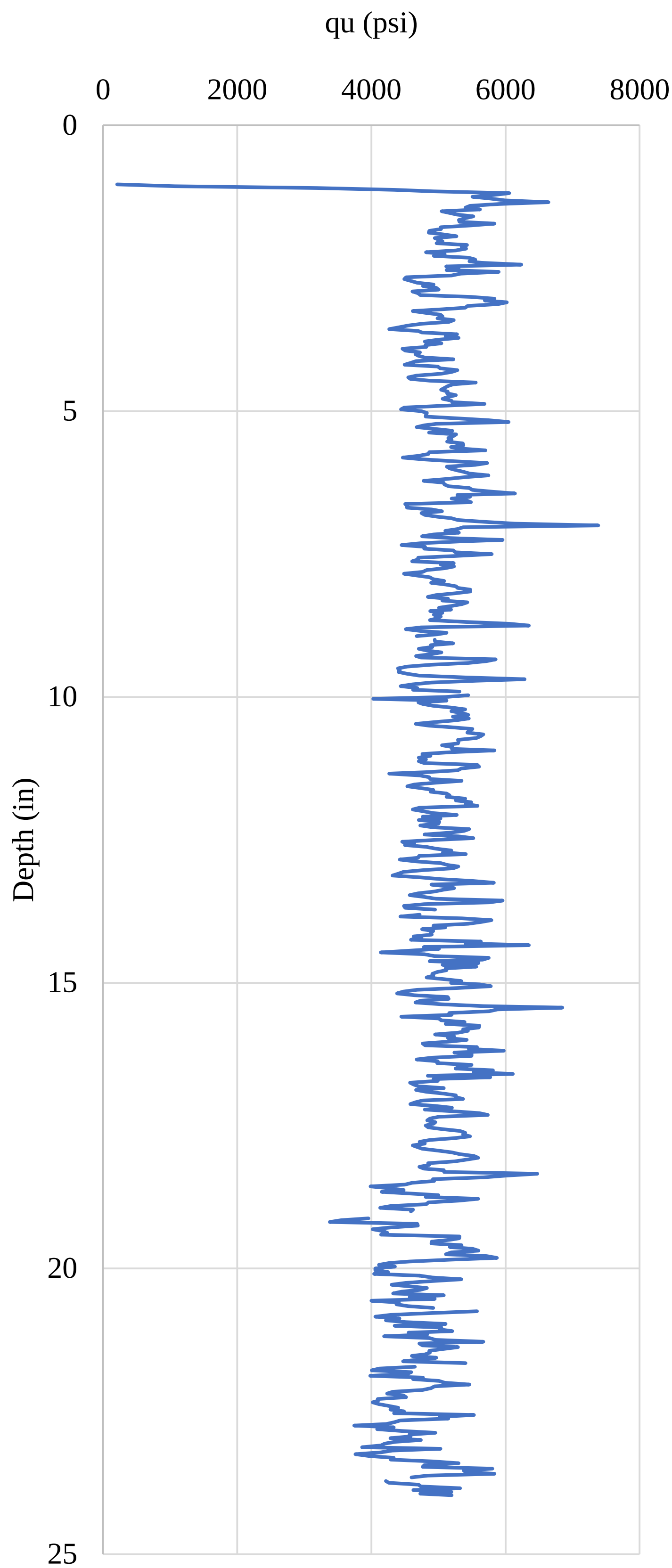


Figure F.90 Core 19-2

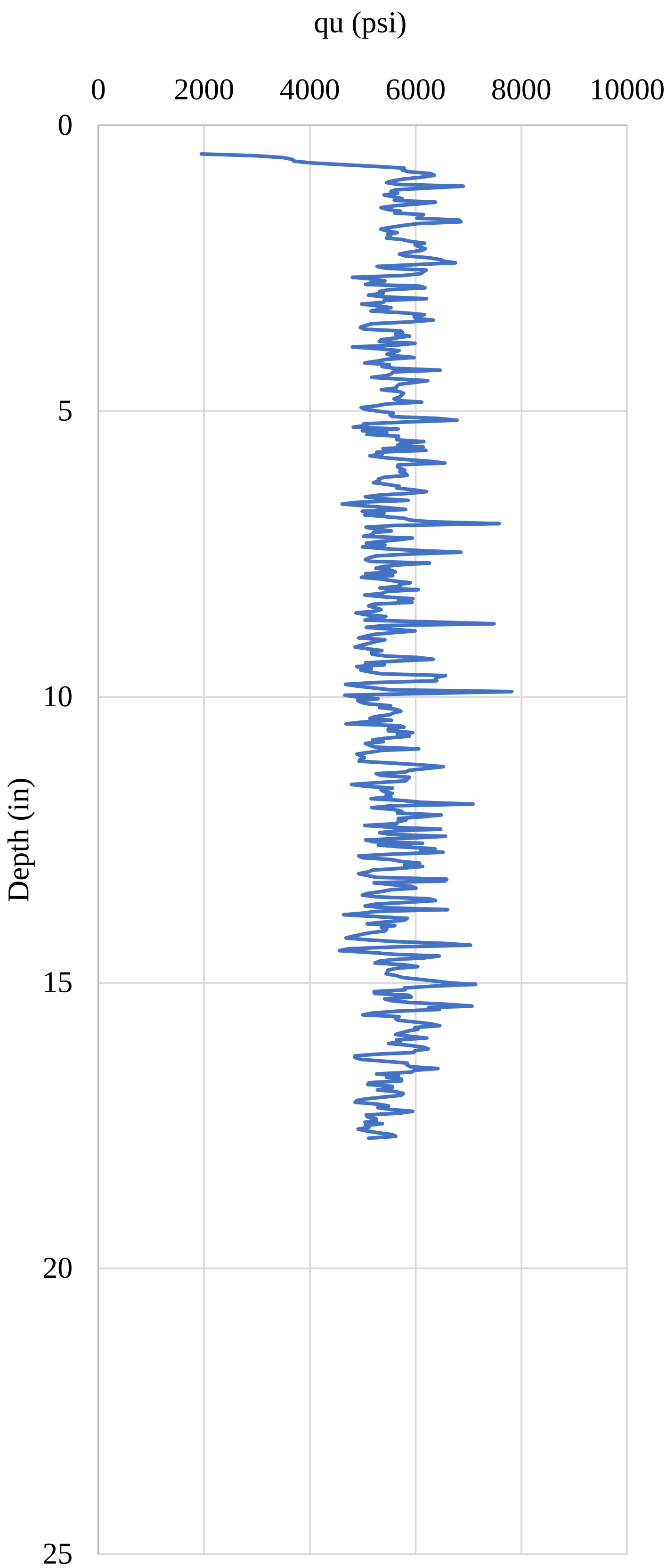


Figure F.91 Core 19-3

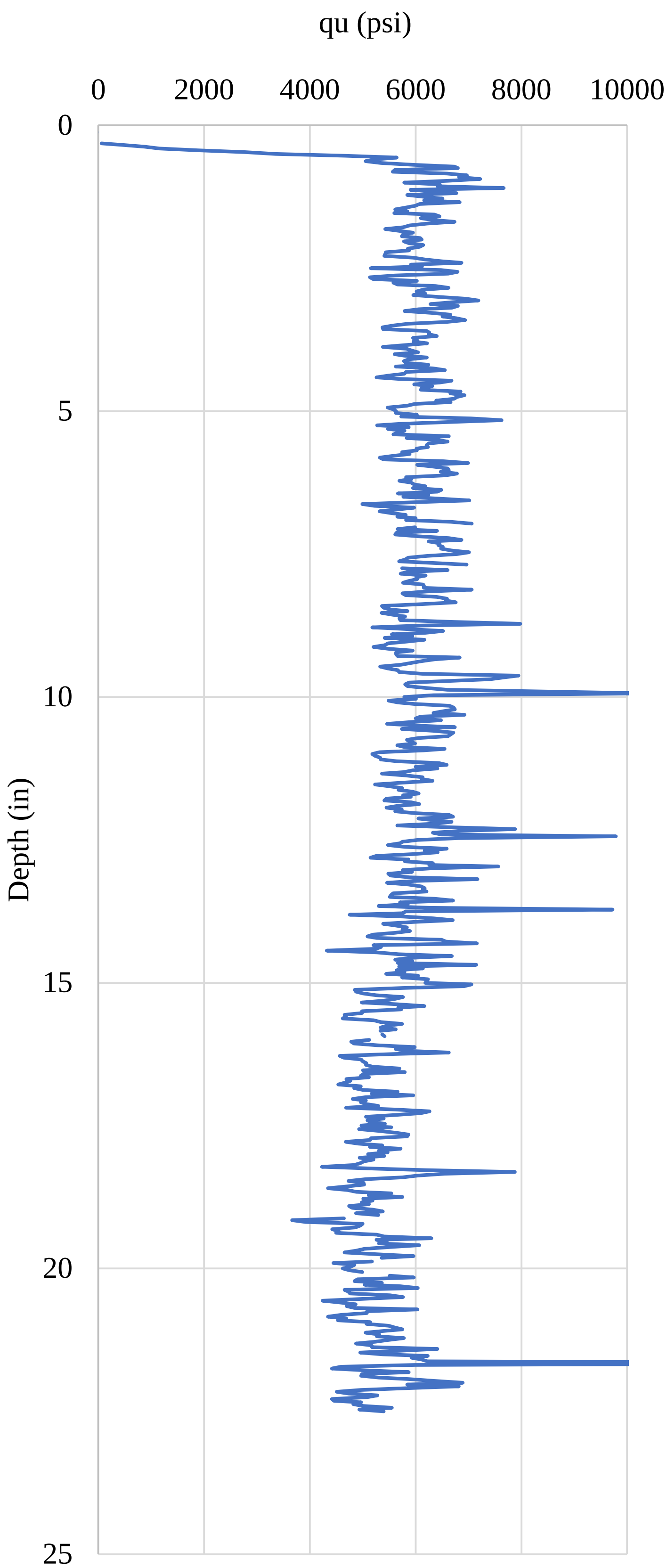


Figure F.92 Core 19-4

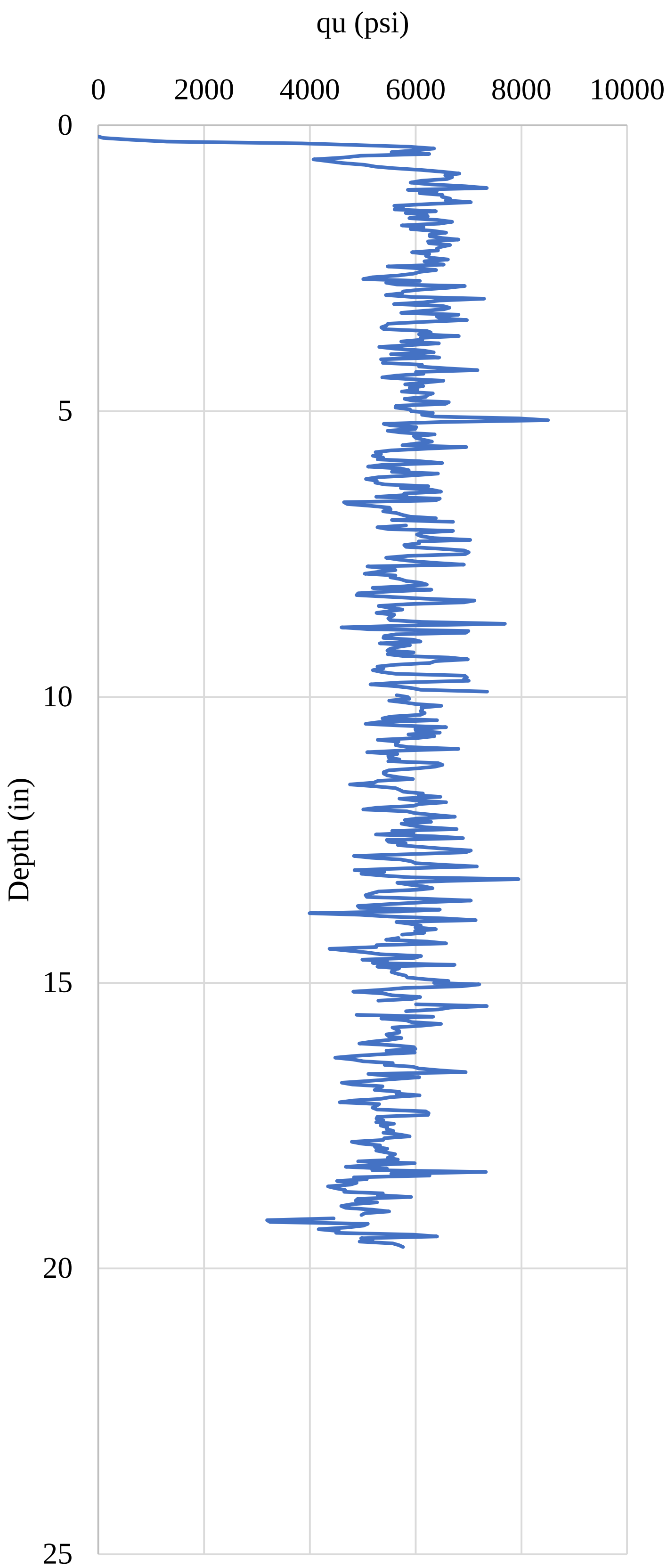


Figure F.93 Core 19-5

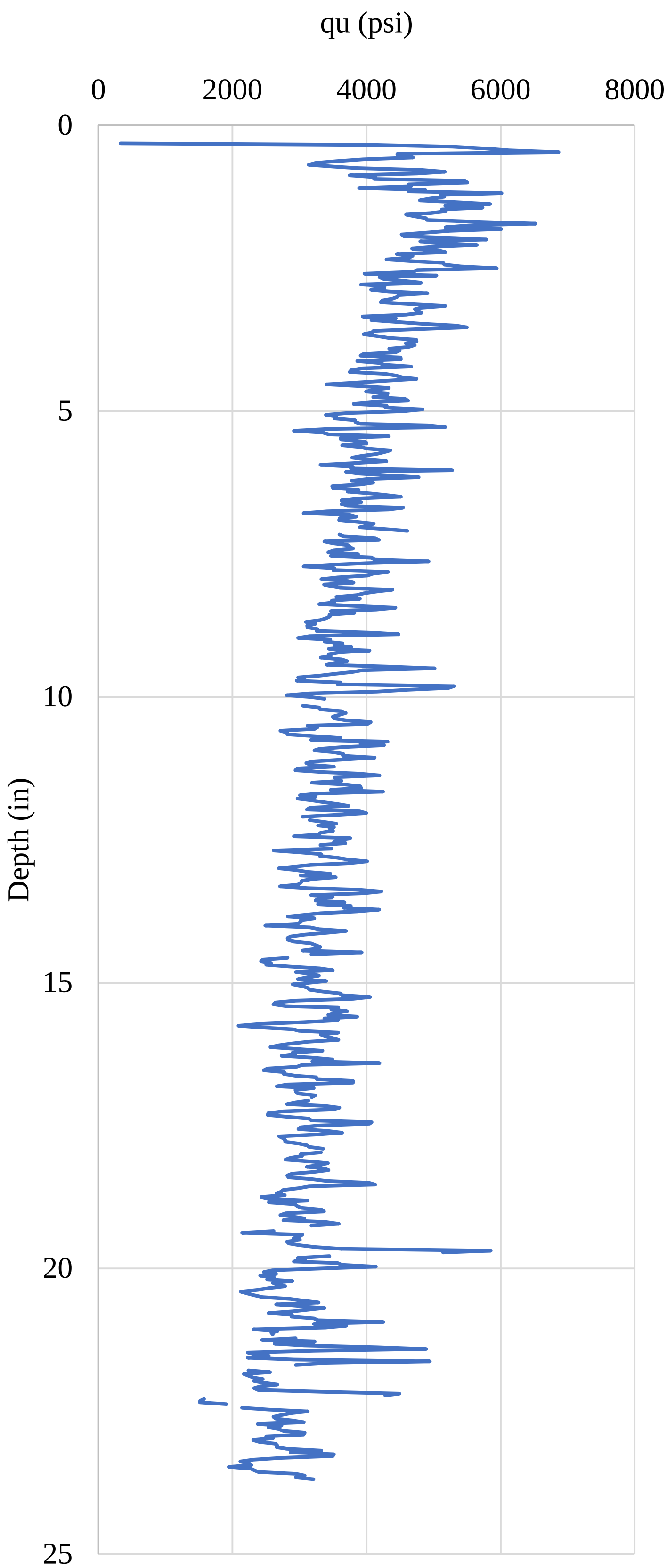


Figure F.94 Core 20-1

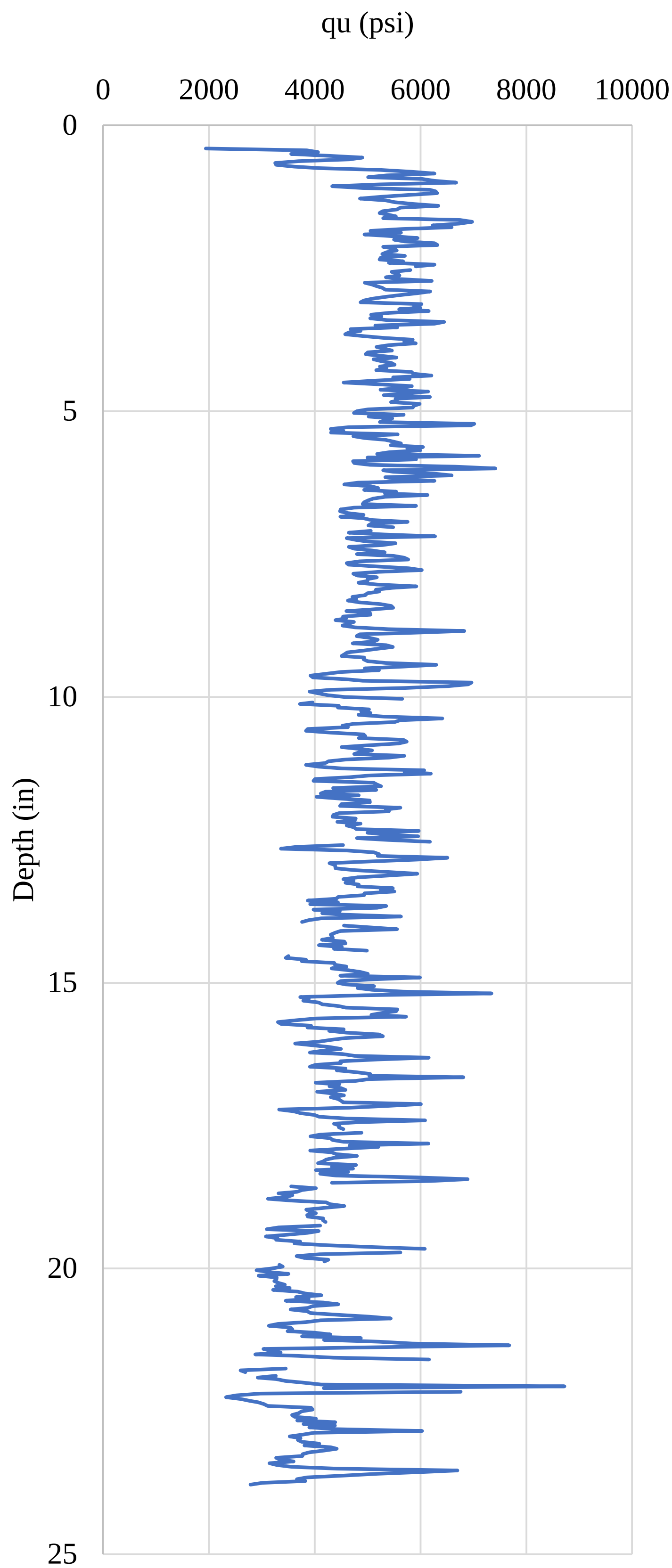


Figure F.95 Core 20-2

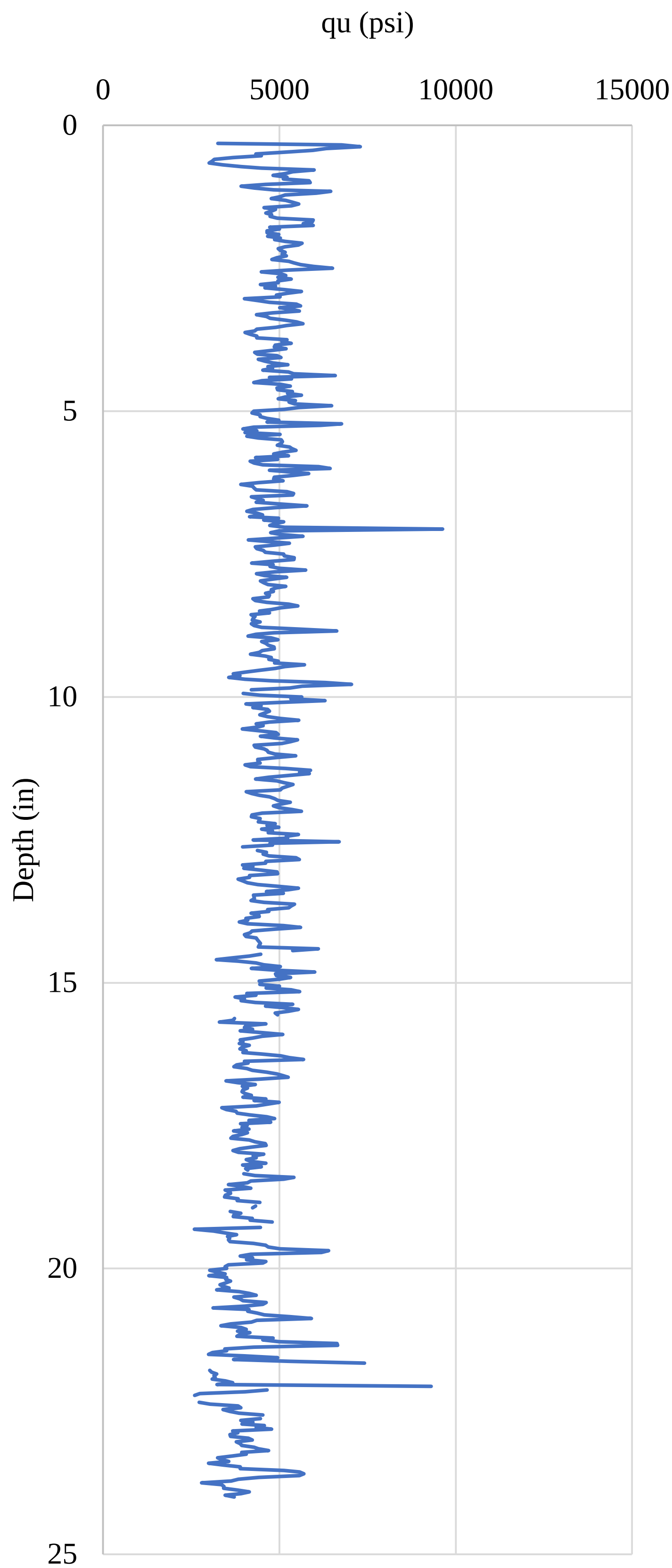


Figure F.96 Core 20-3

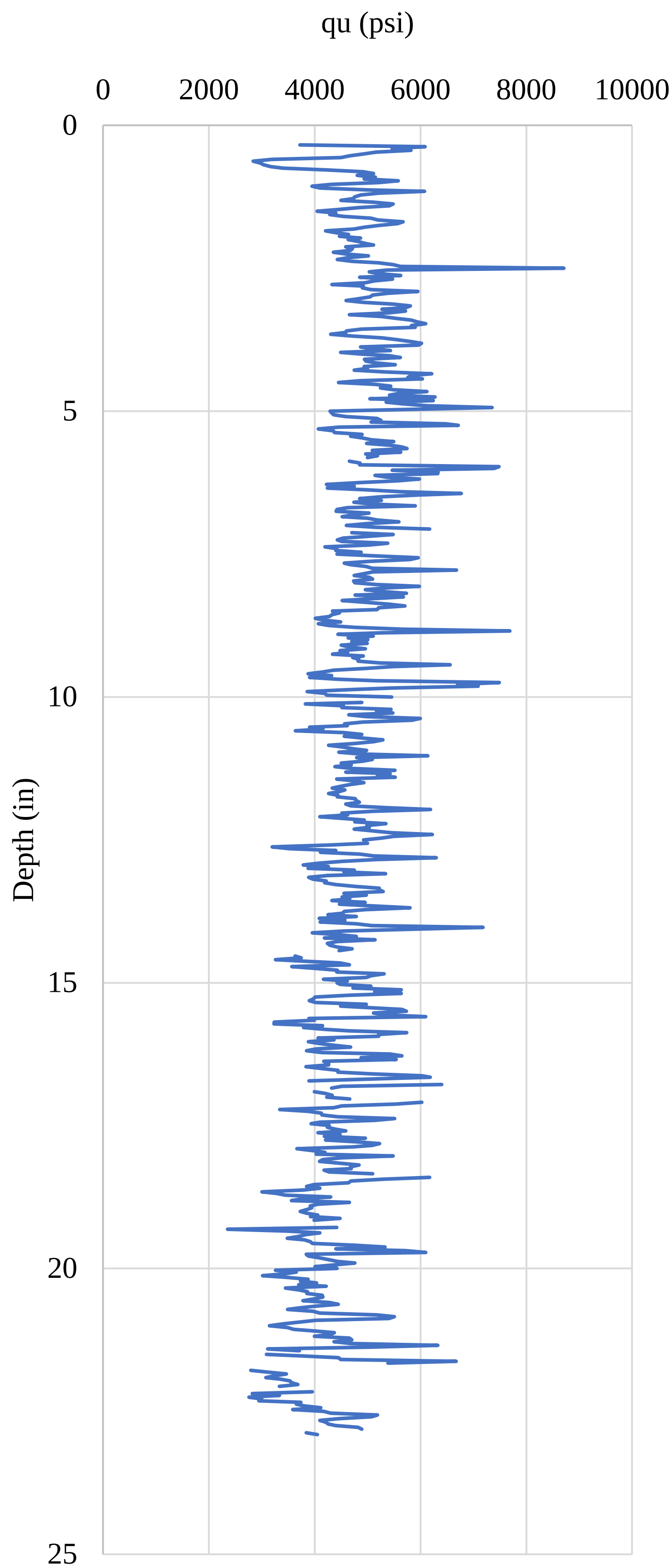


Figure F.97 Core 20-4

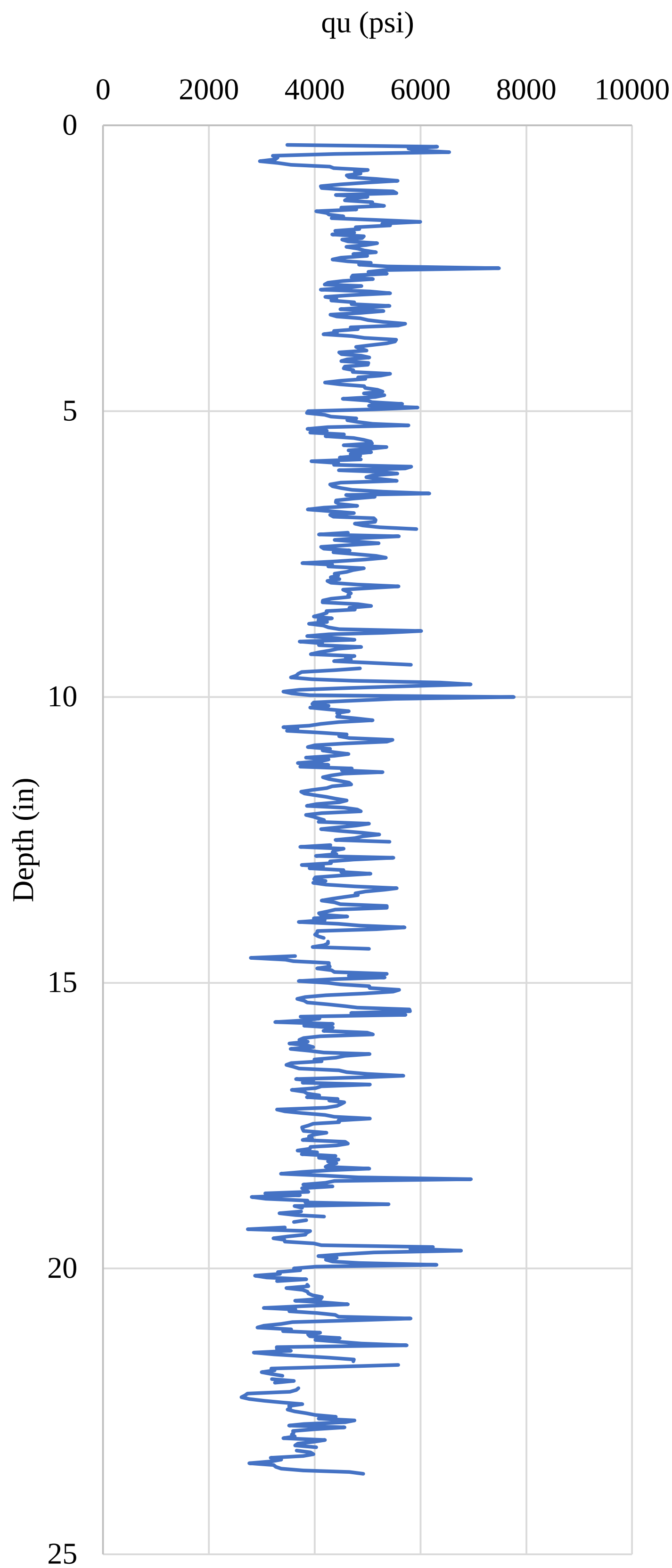
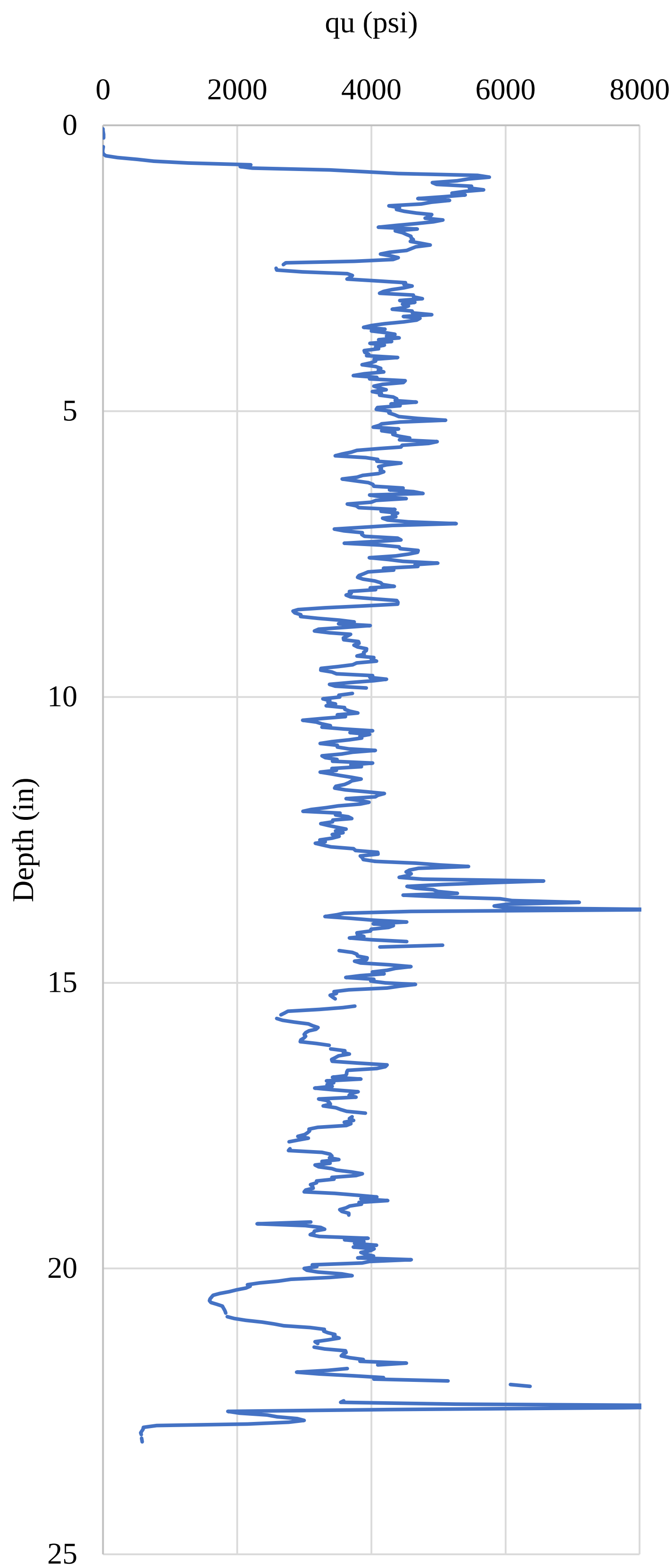


Figure F.98 Core 20-5



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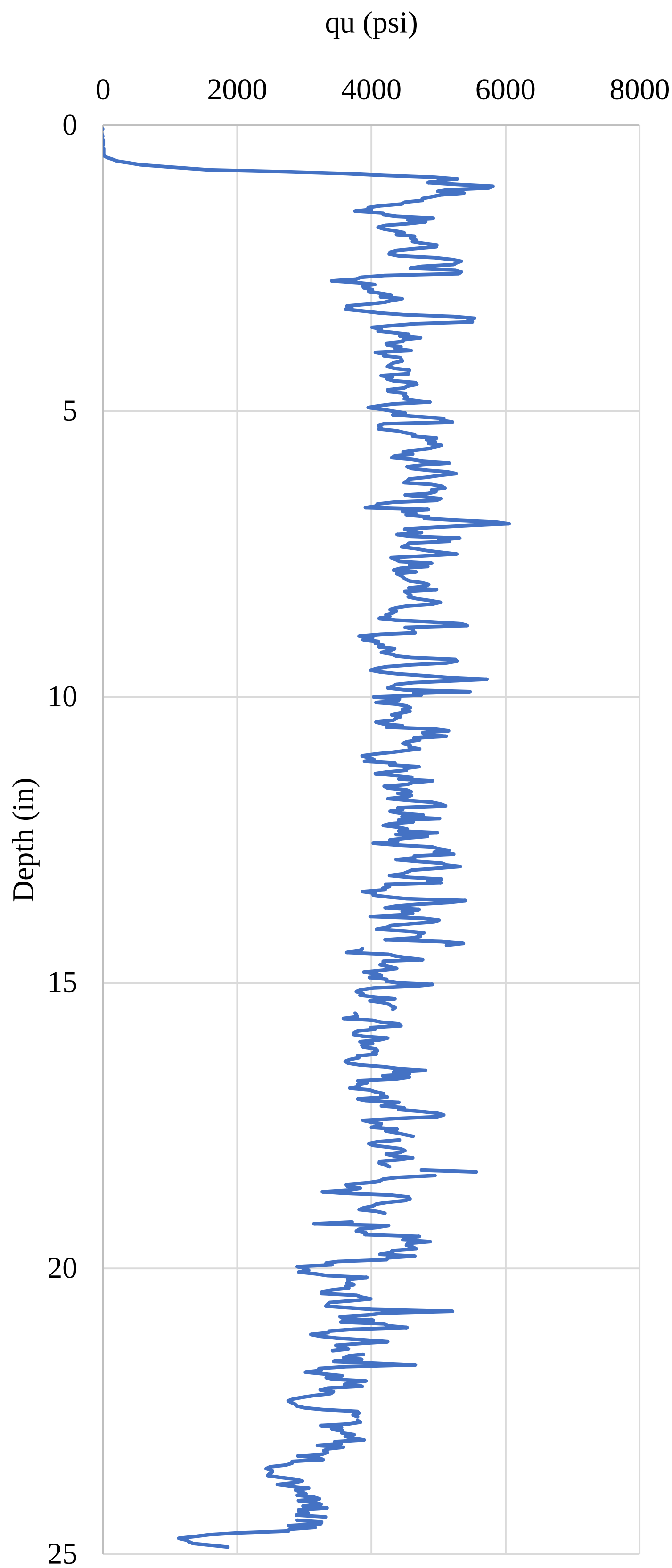


Figure F.100 Core 21-2

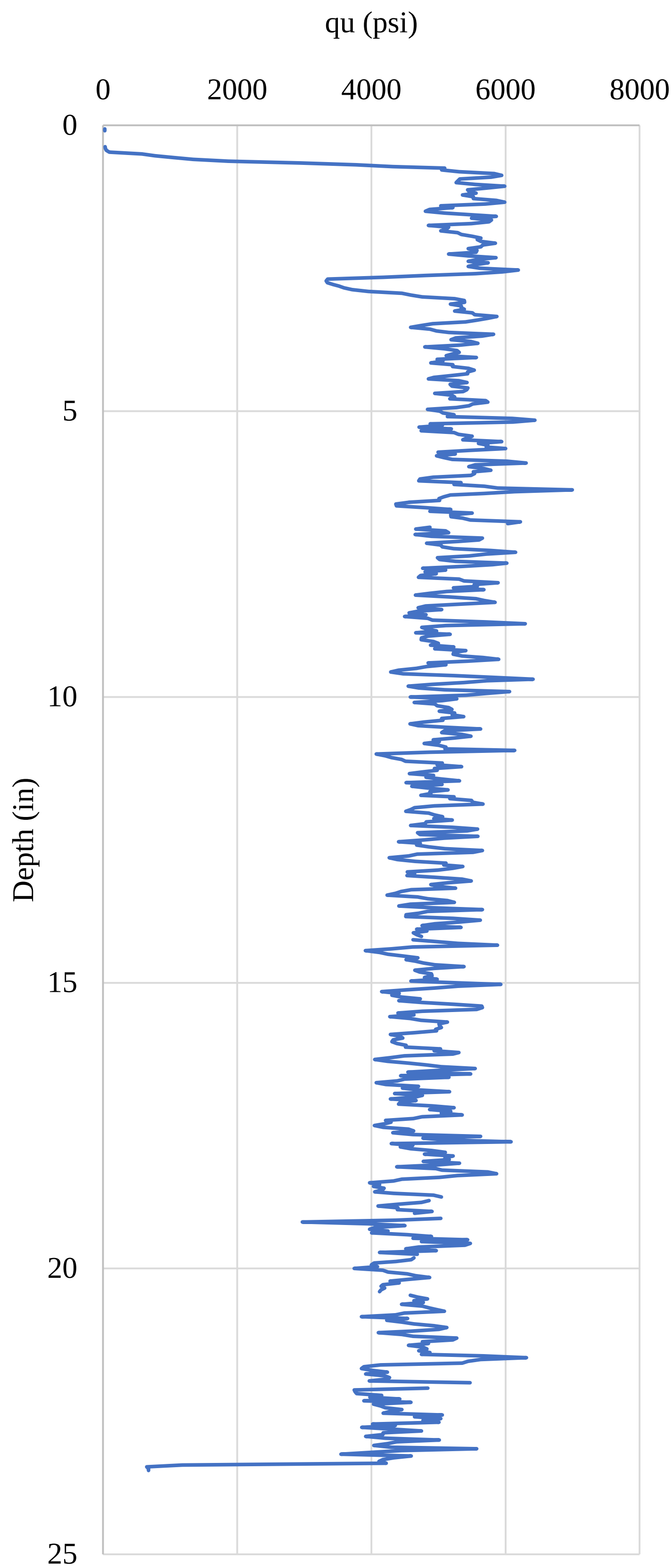


Figure F.101 Core 21-3

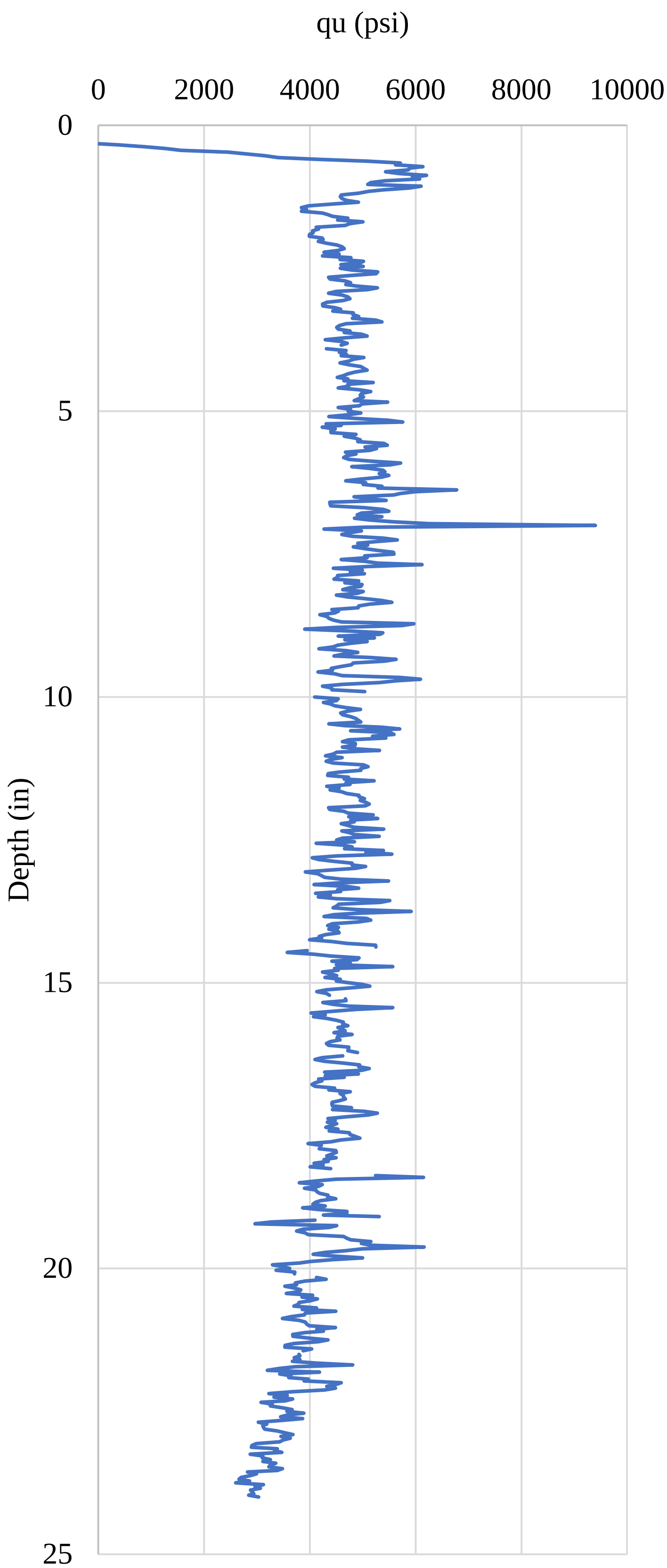


Figure F.102 Core 21-4

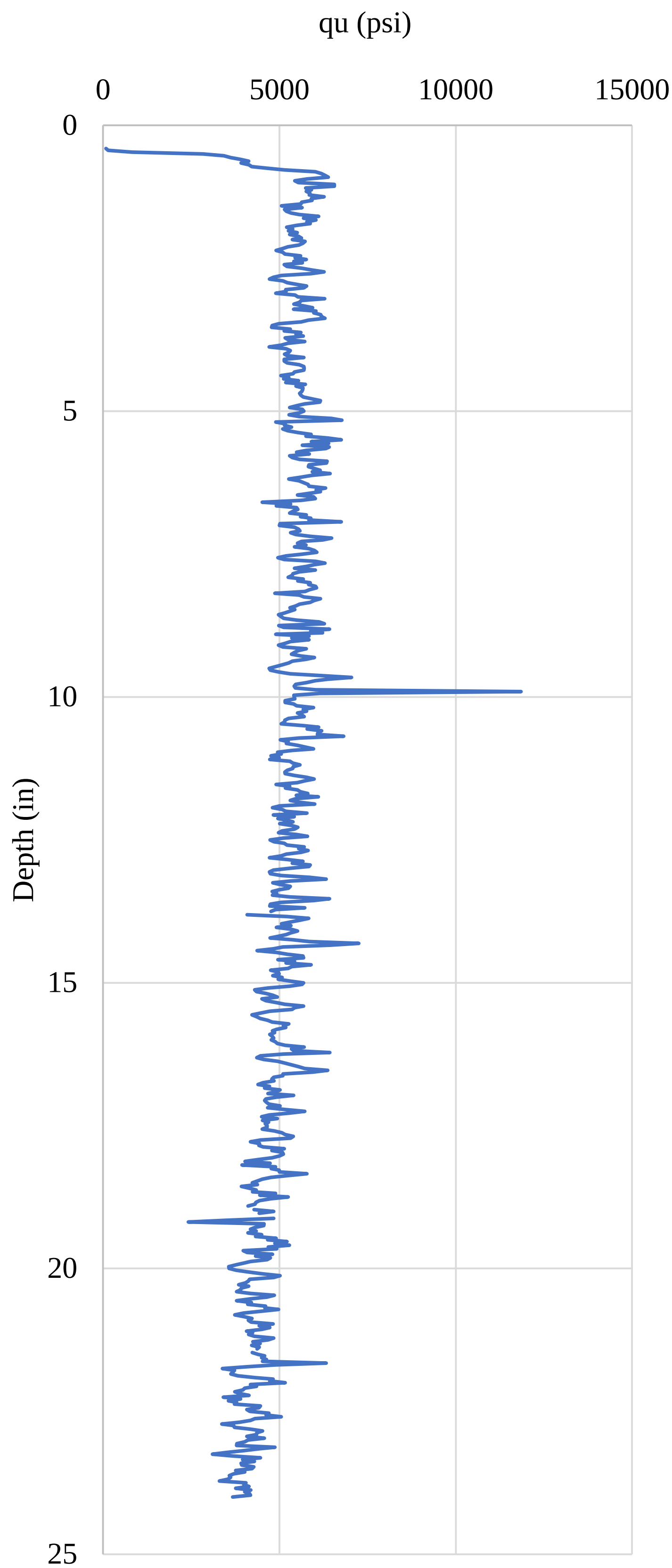


Figure F.103 Core 21-5

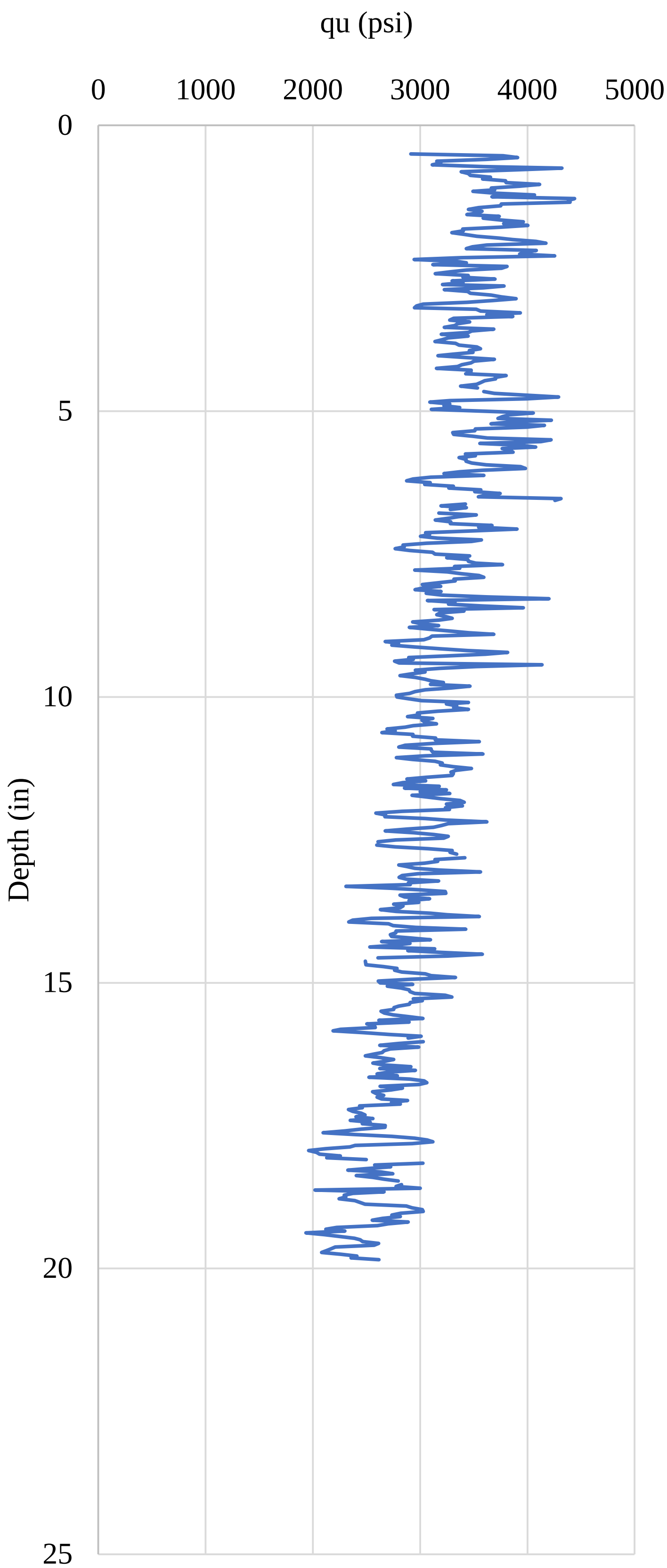


Figure F.104 Core 22-1

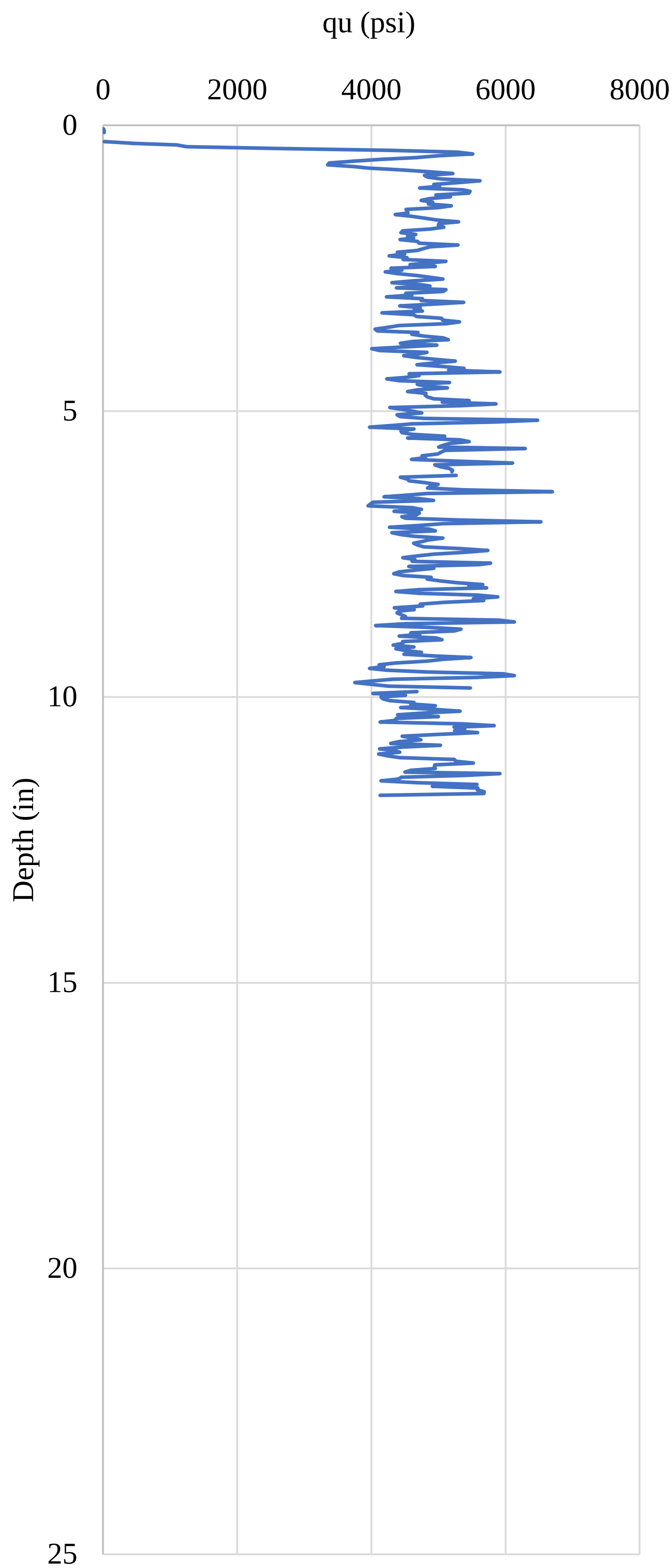


Figure F.105 Core 22-2

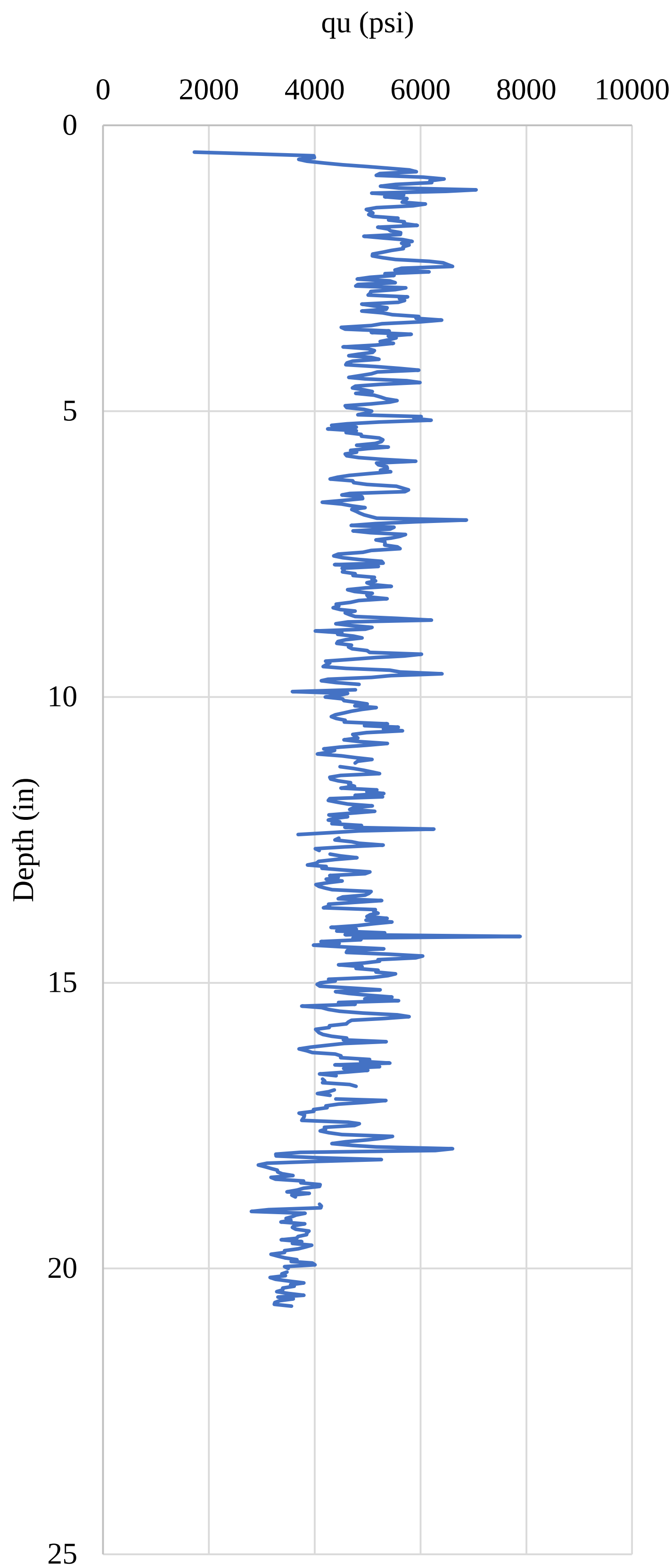


Figure F.106 Core 22-3

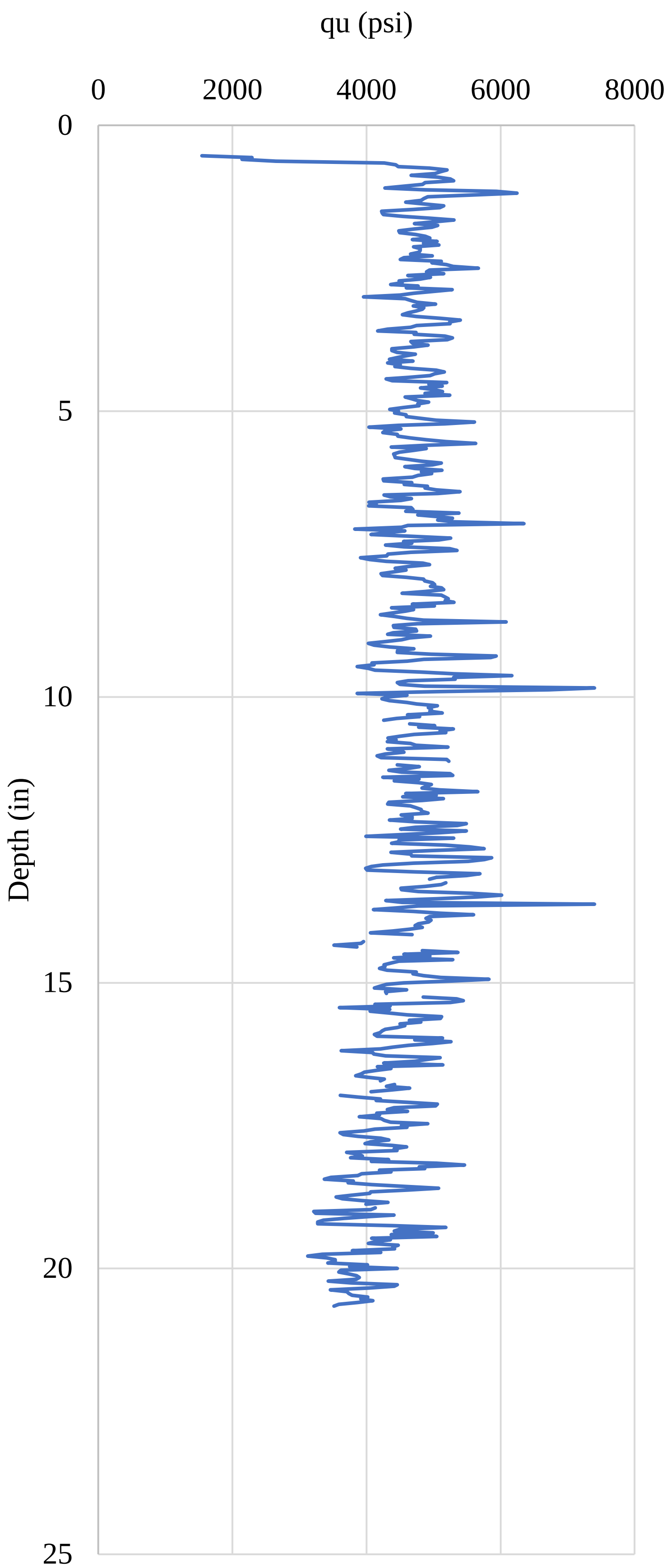


Figure F.107 Core 22-4

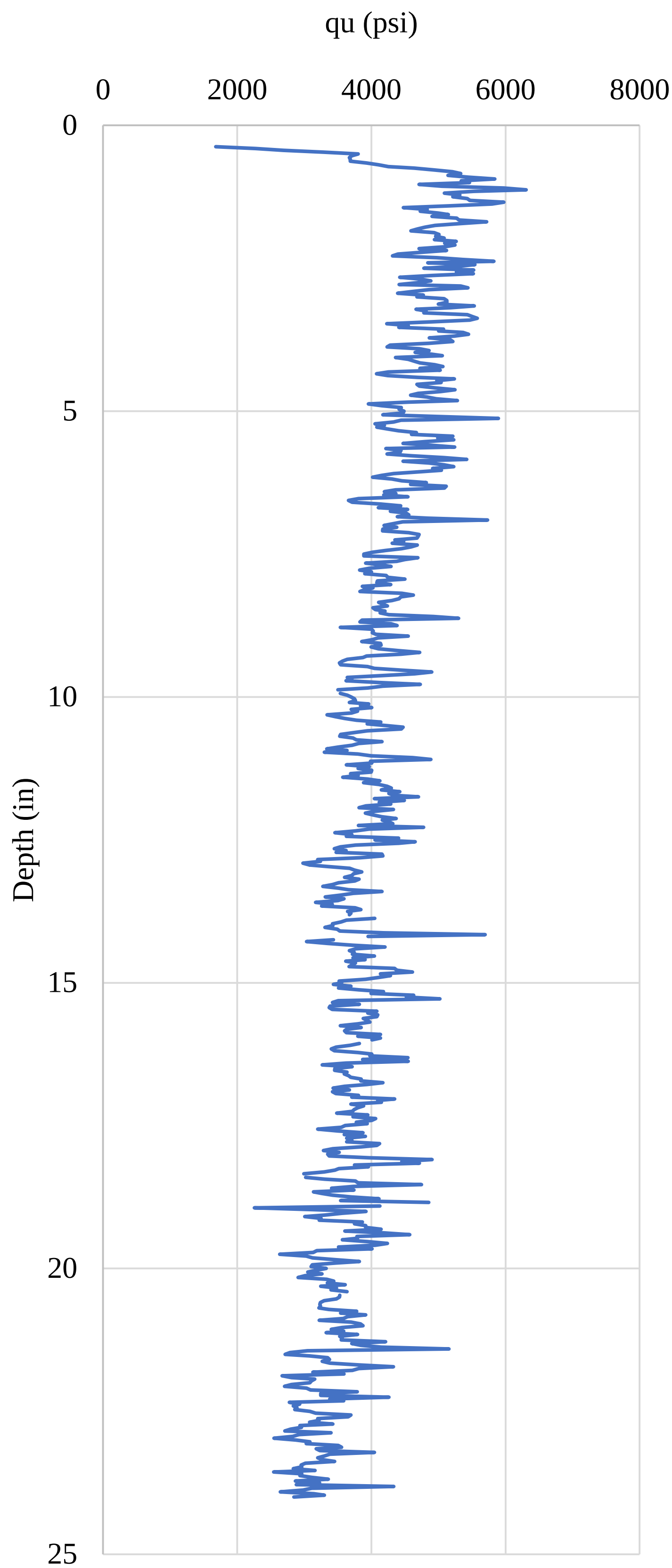
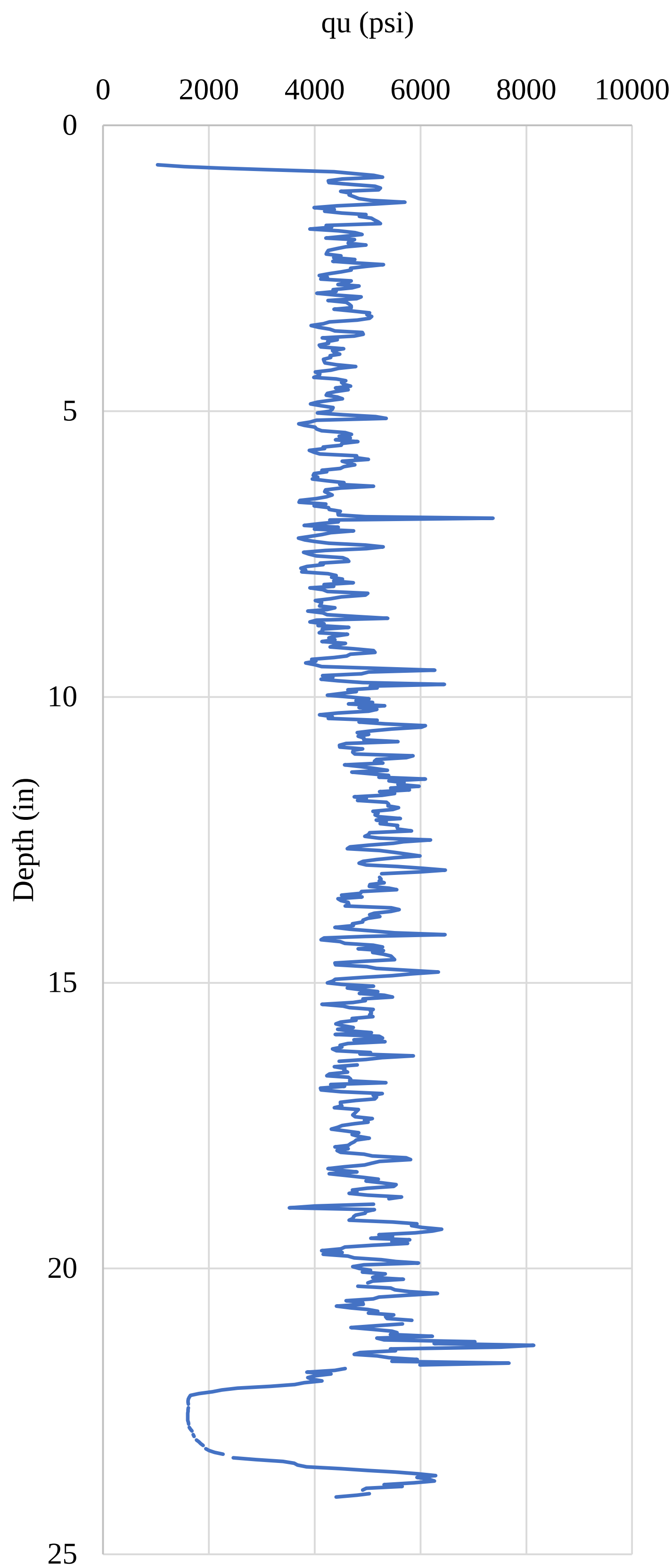


Figure F.108 Core 22-5



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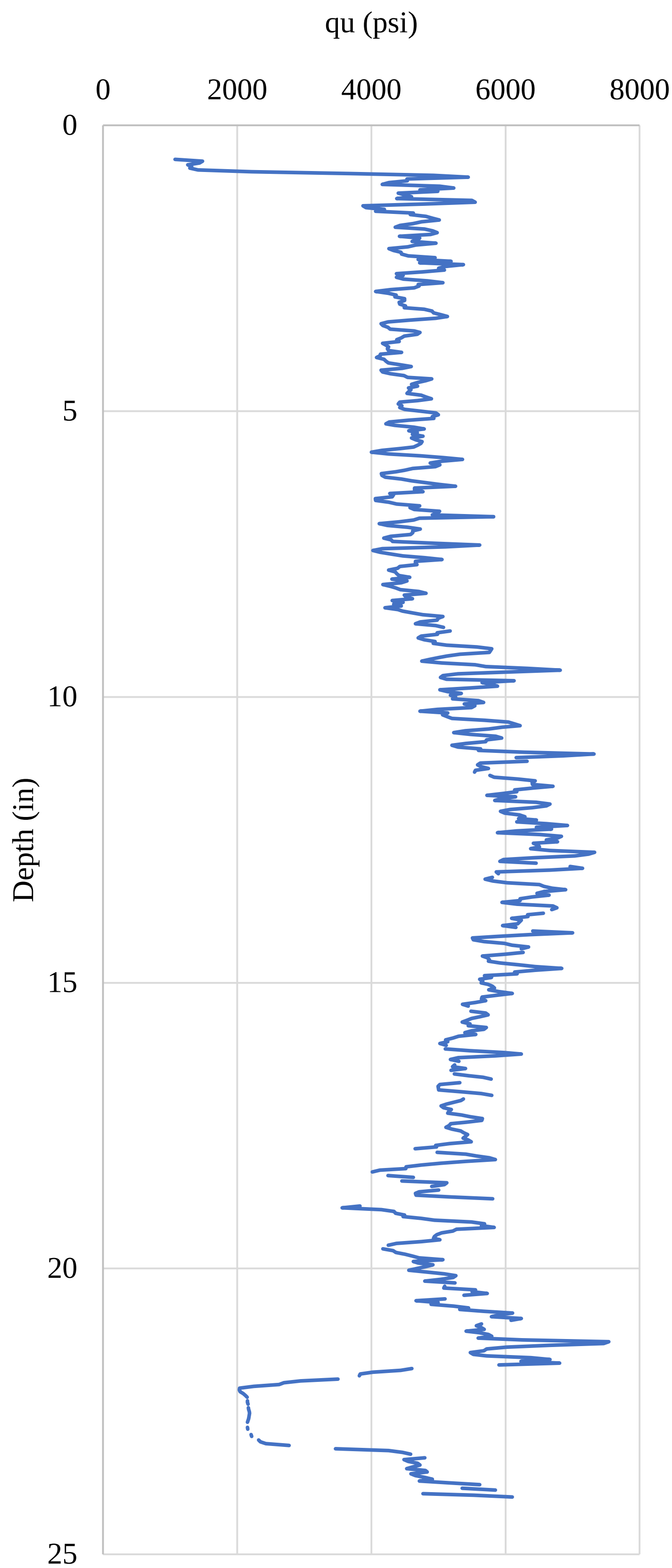


Figure F.110 Core 23-2

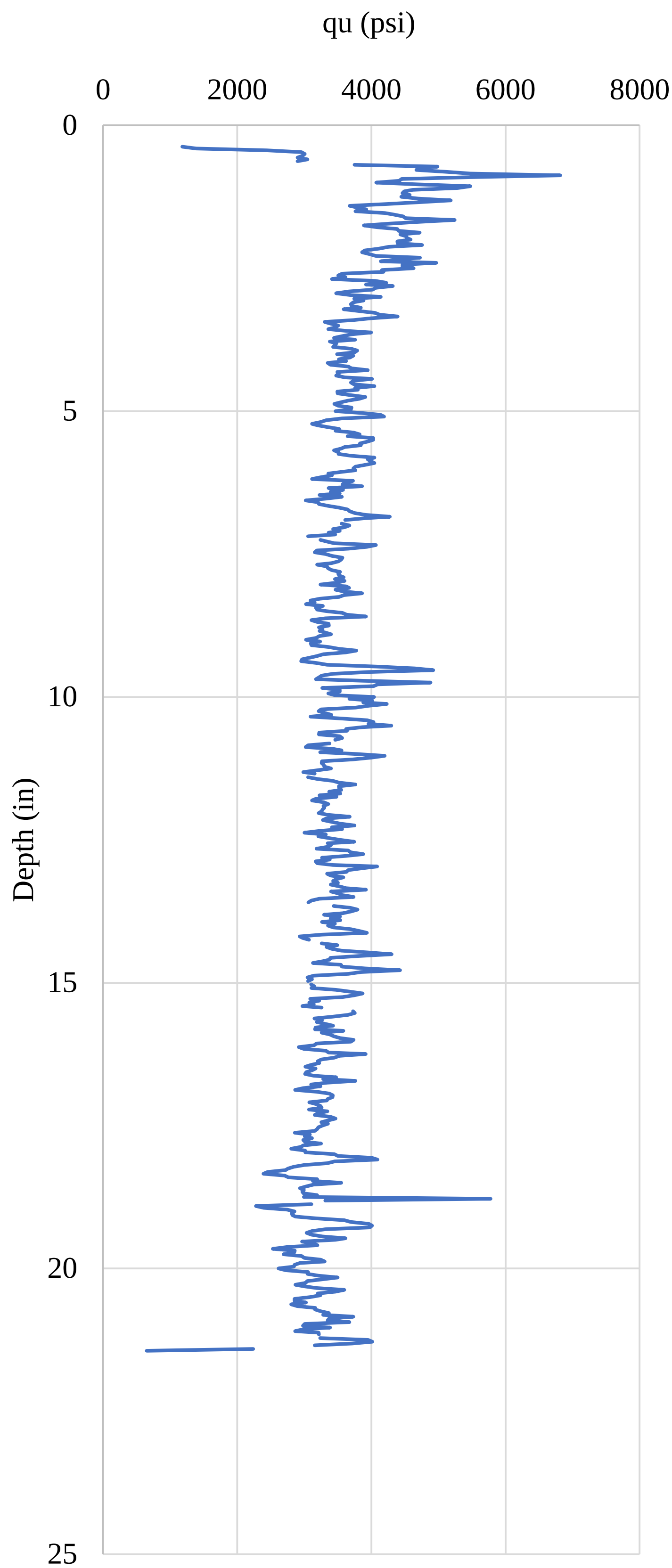


Figure F.111 Core 23-3

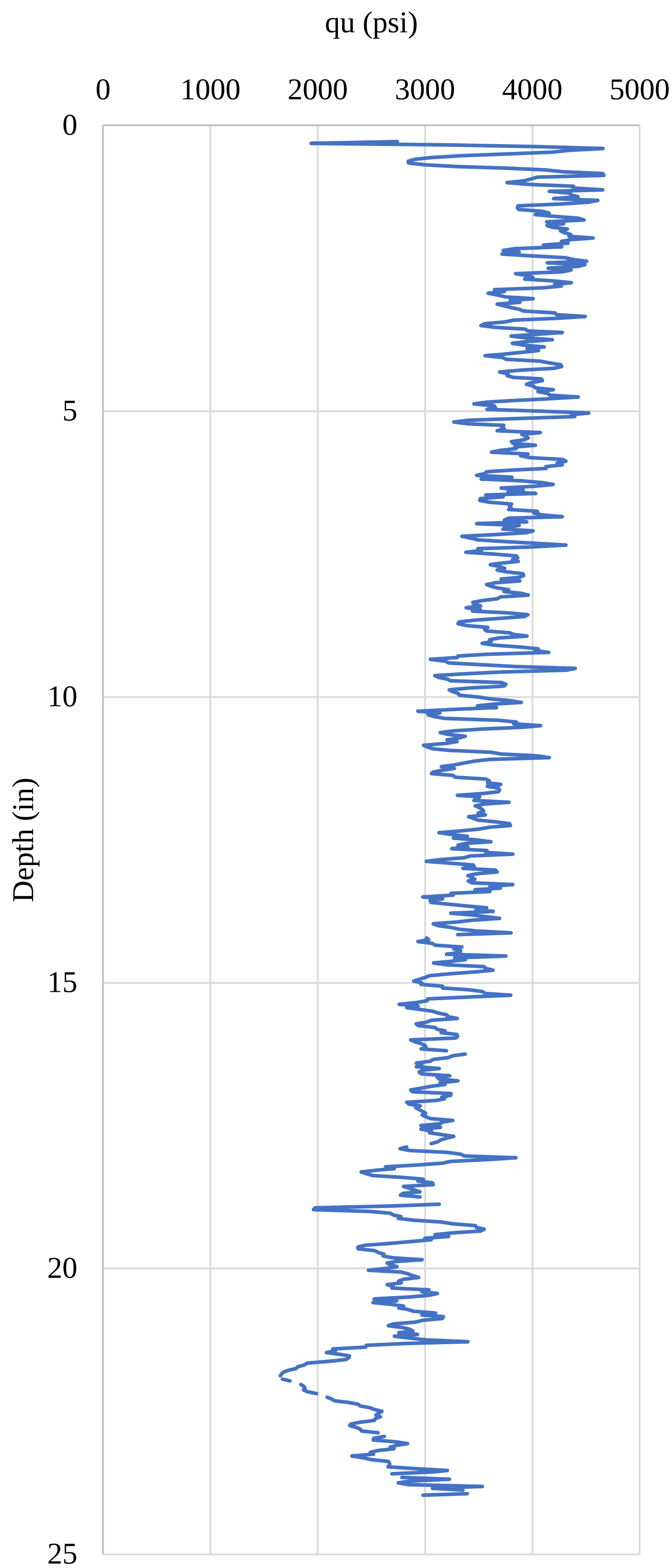


Figure F.112 Core 23-4

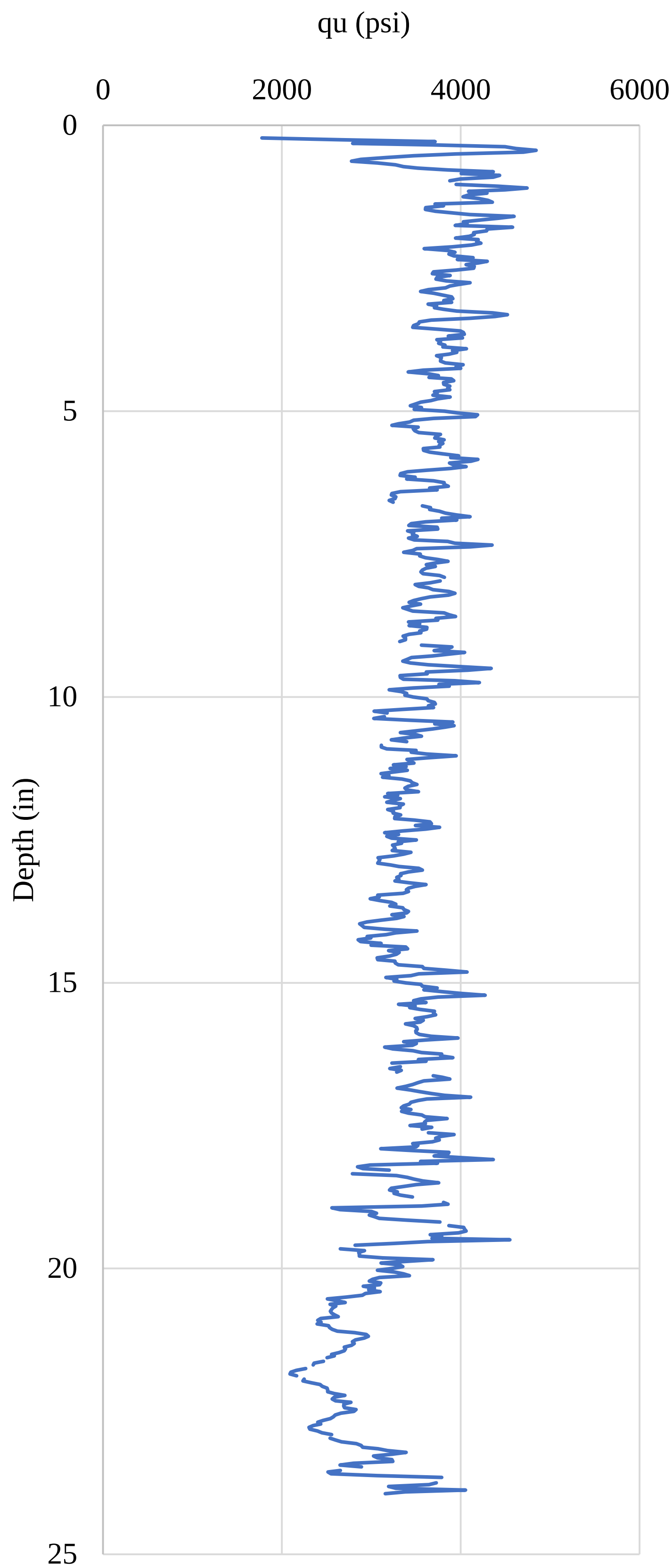


Figure F.113 Core 23-5

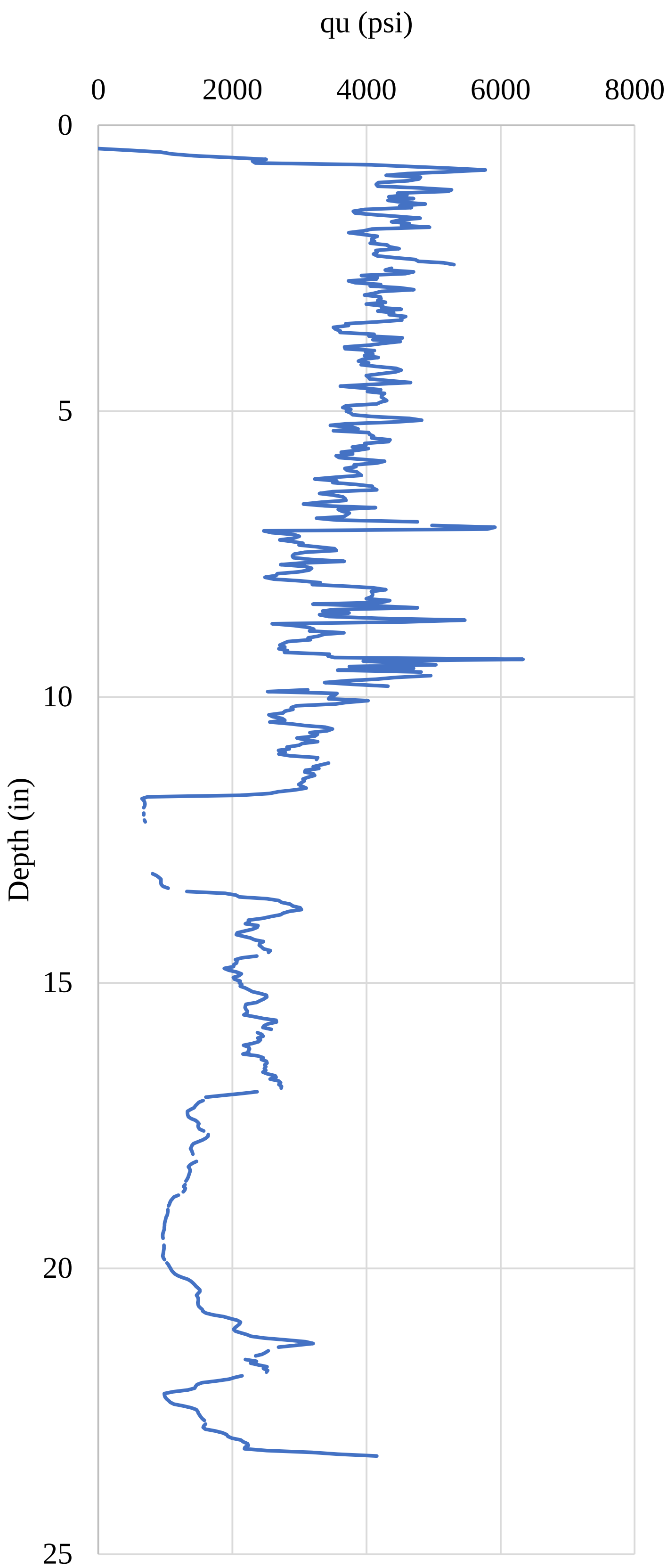


Figure F.114 Core 24-1

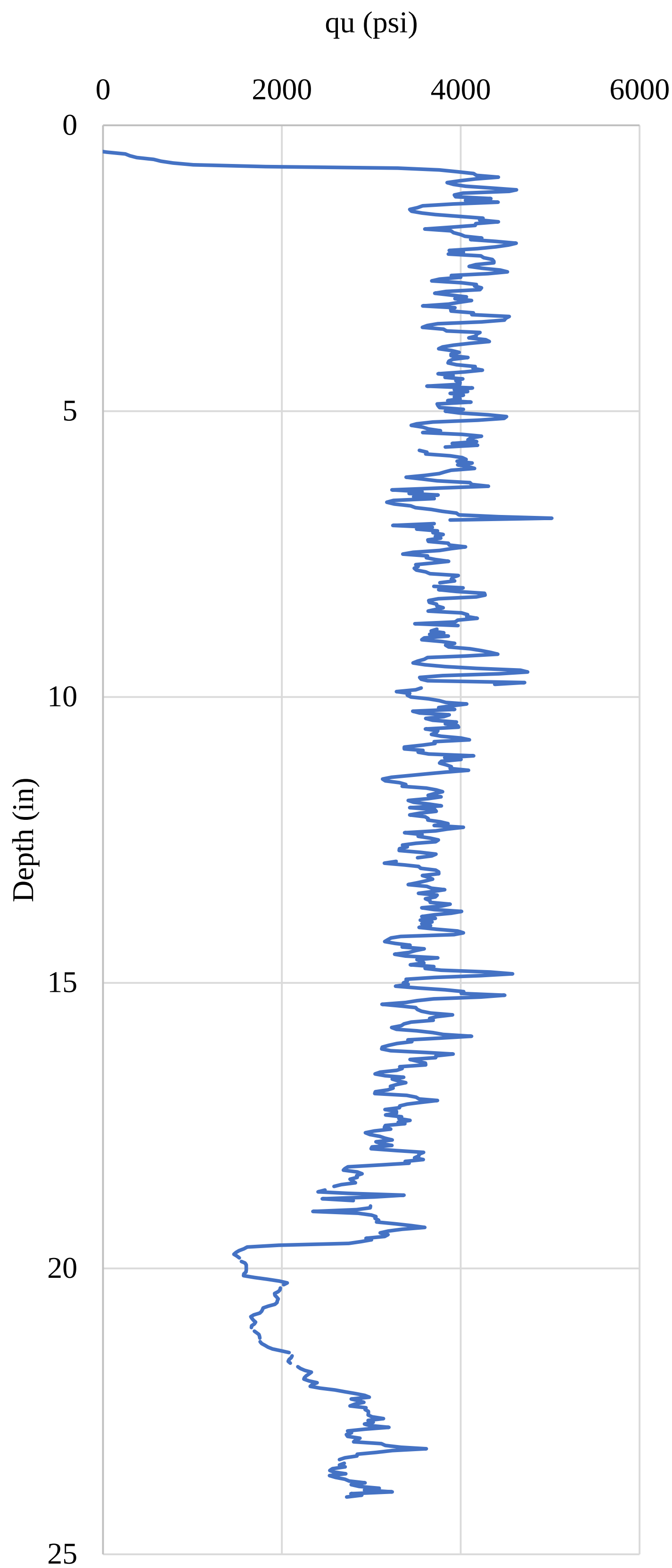


Figure F.115 Core 24-2

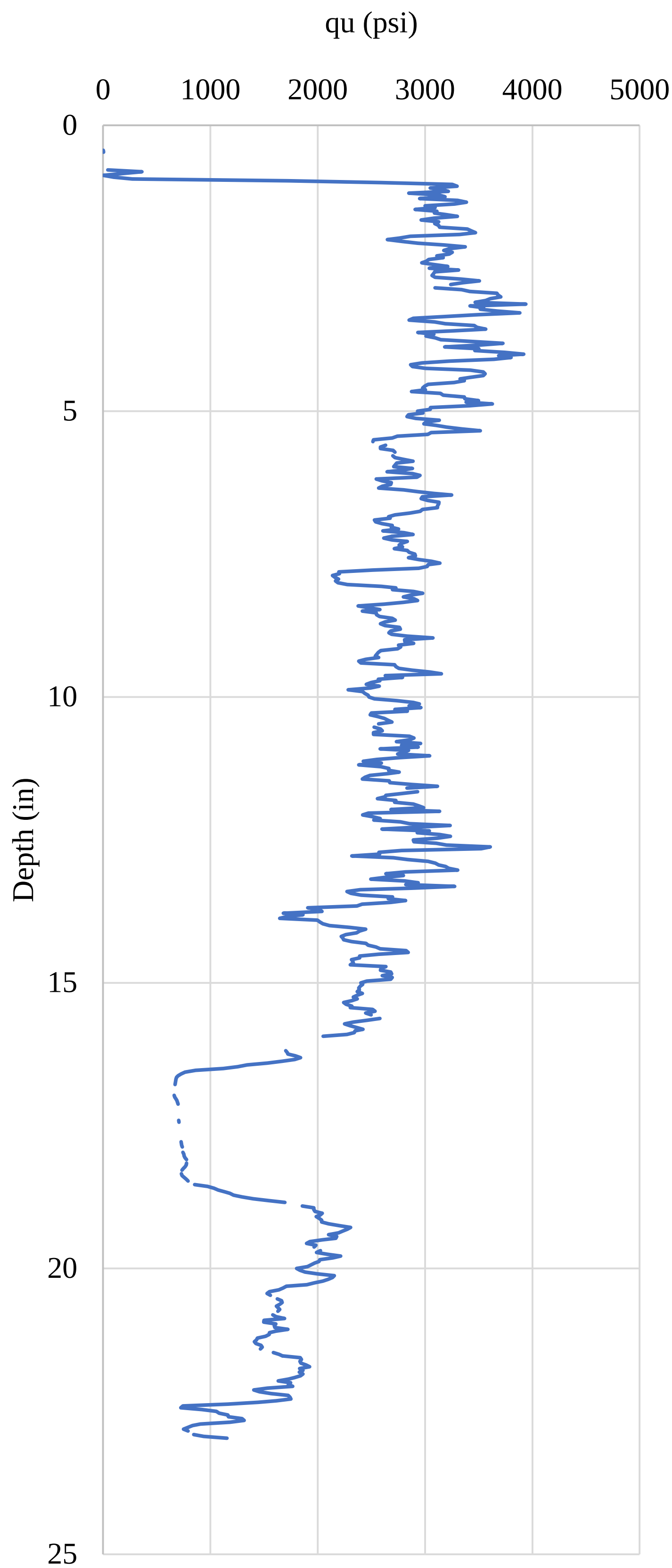


Figure F.116 Core 24-3

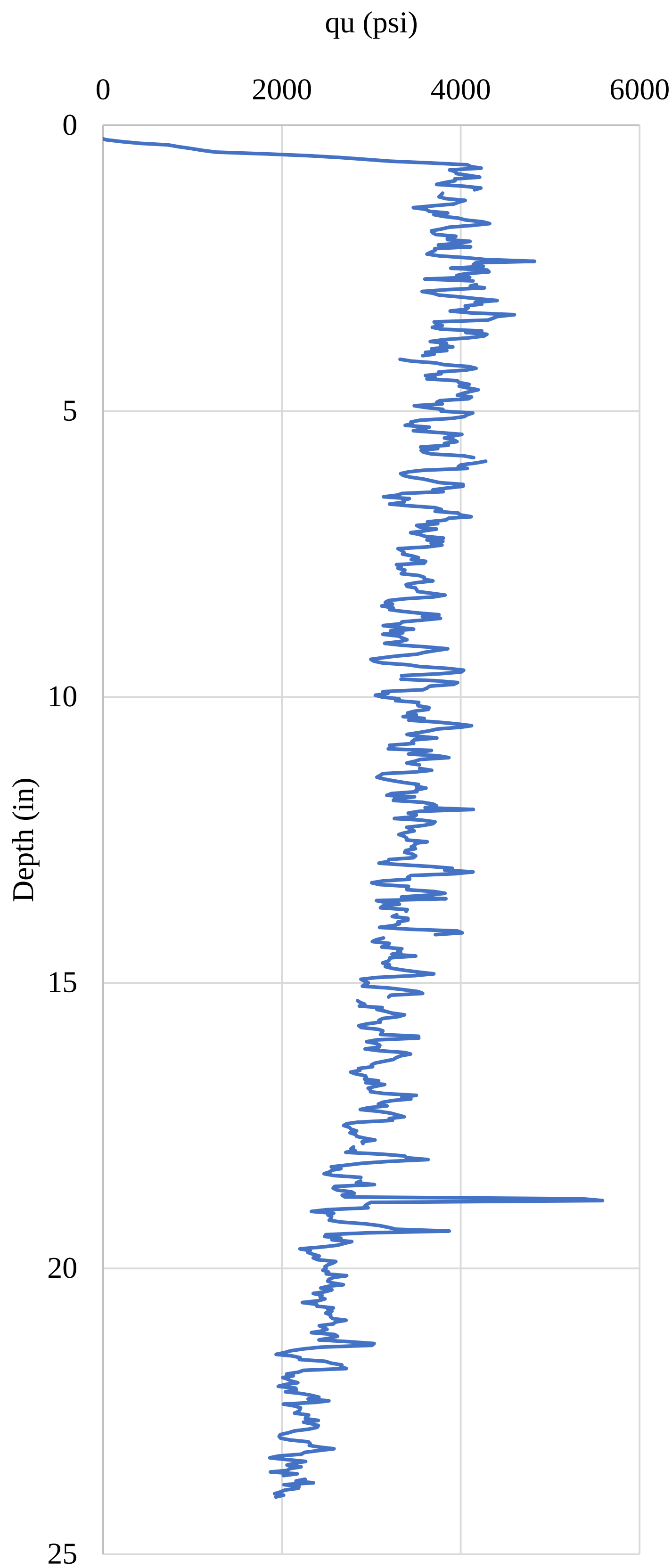


Figure F.117 Core 24-4

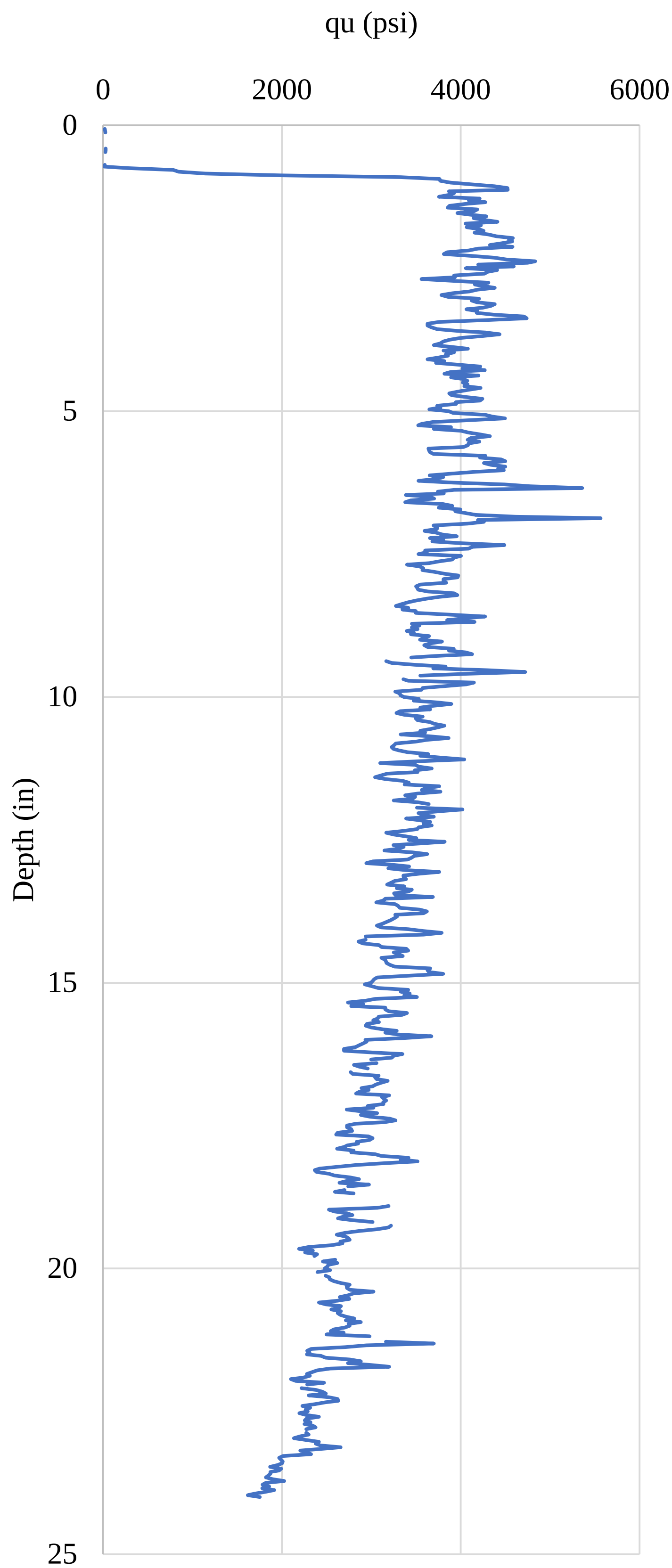


Figure F.118 Core 24-5

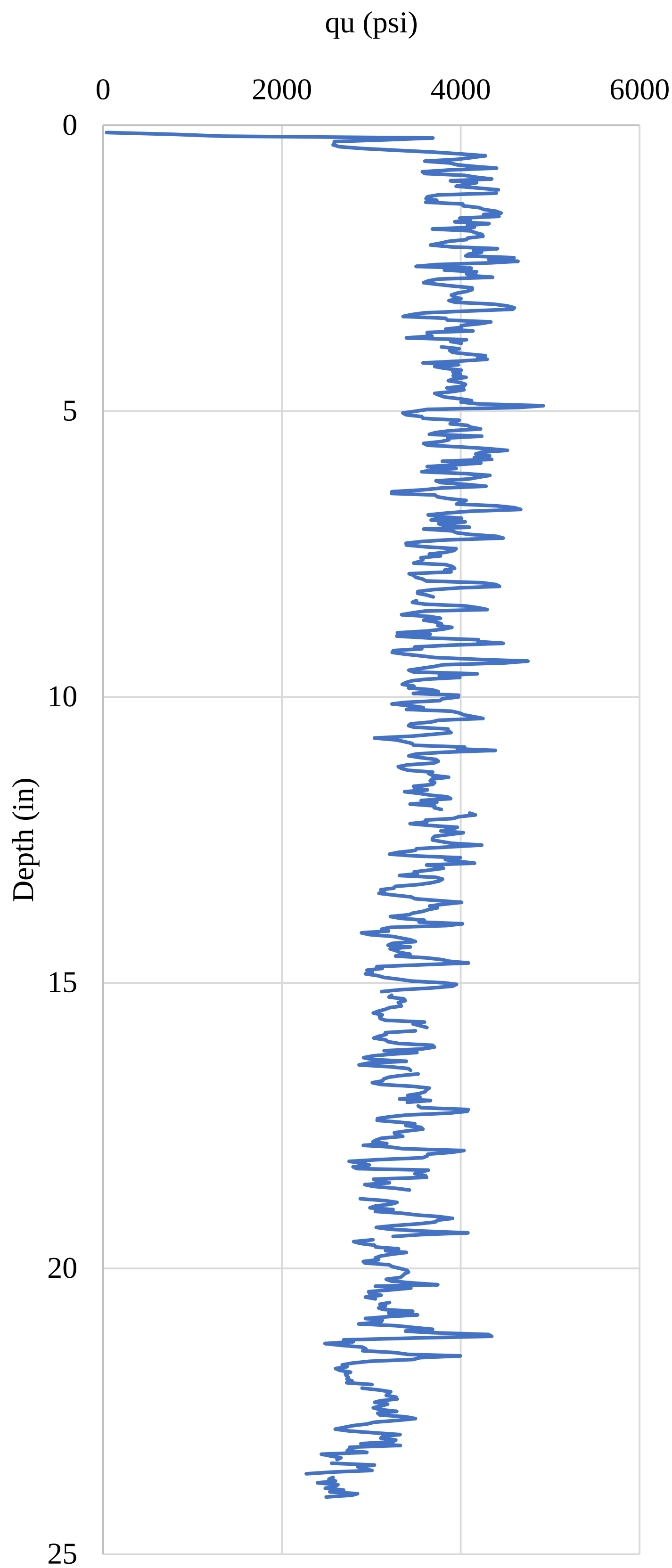


Figure F.119 Core 26-1

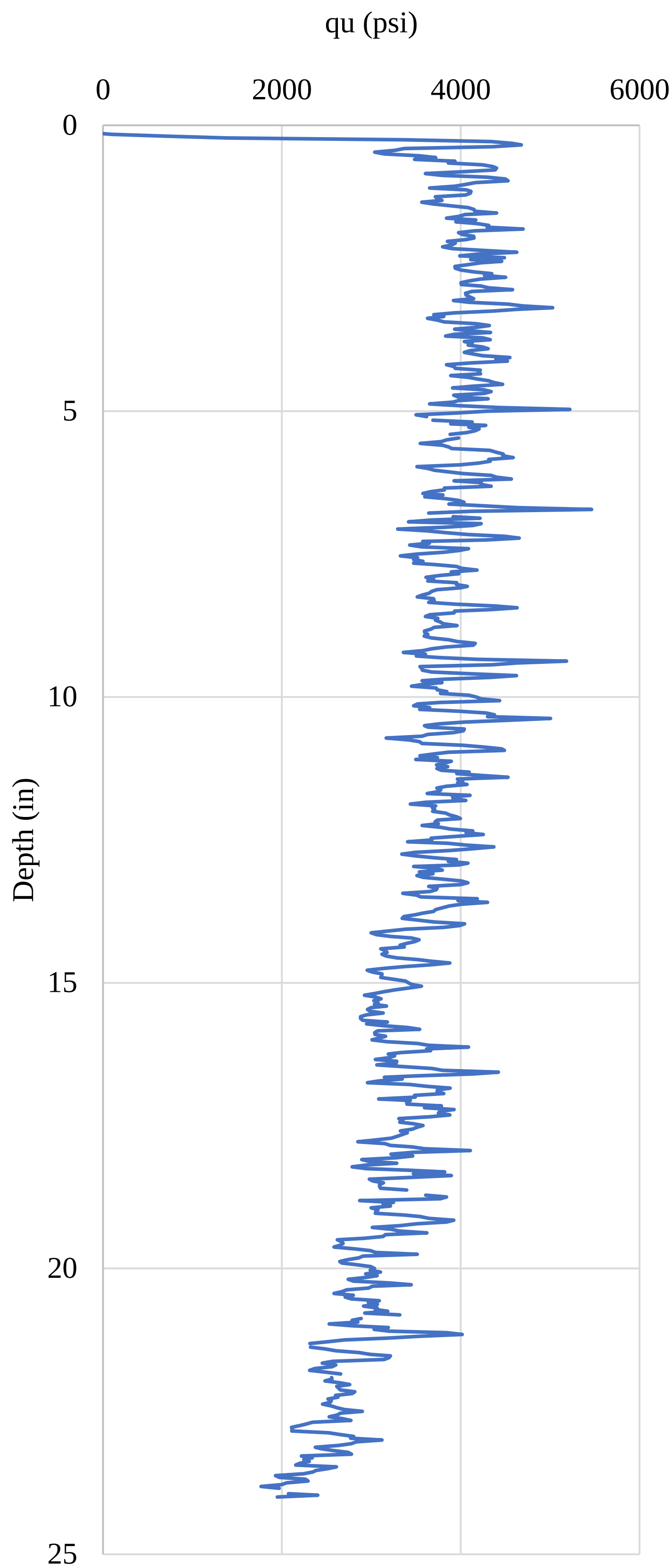


Figure F.120 Core 26-2

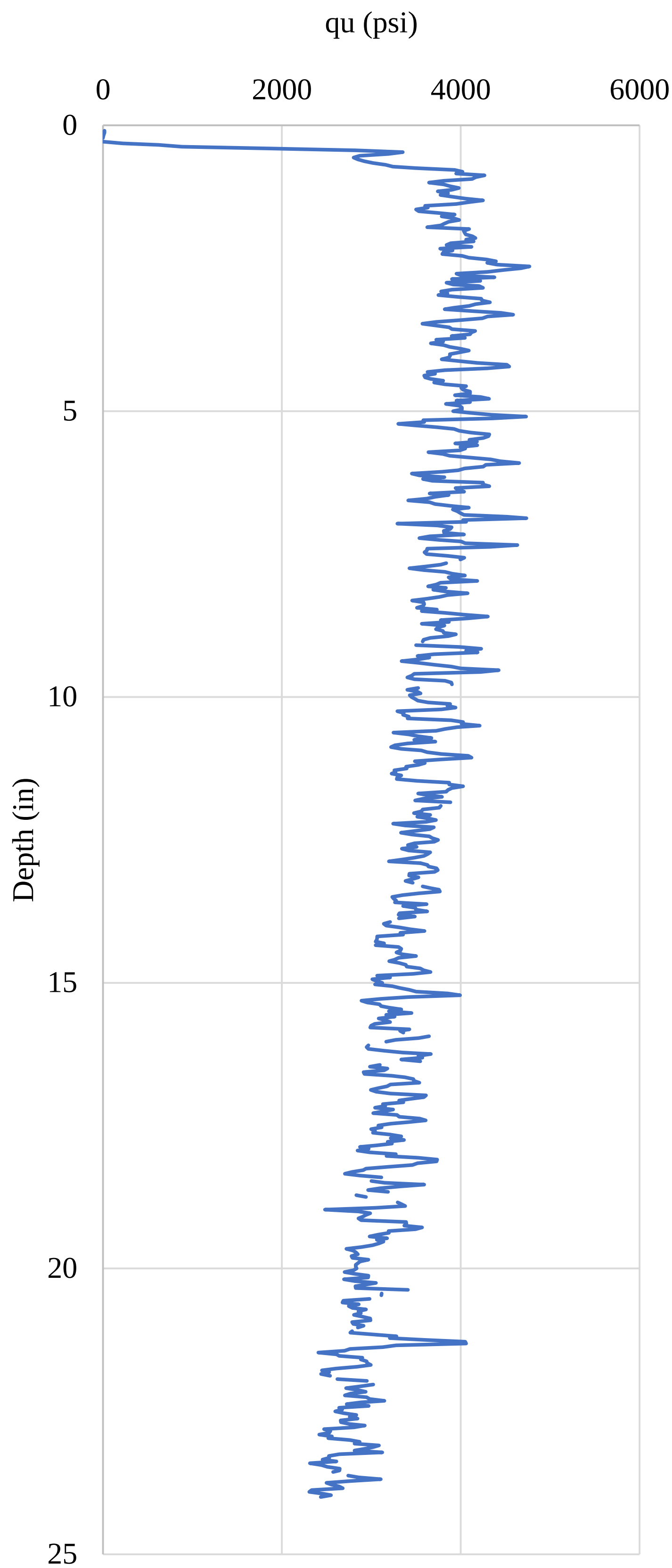


Figure F.121 Core 26-3

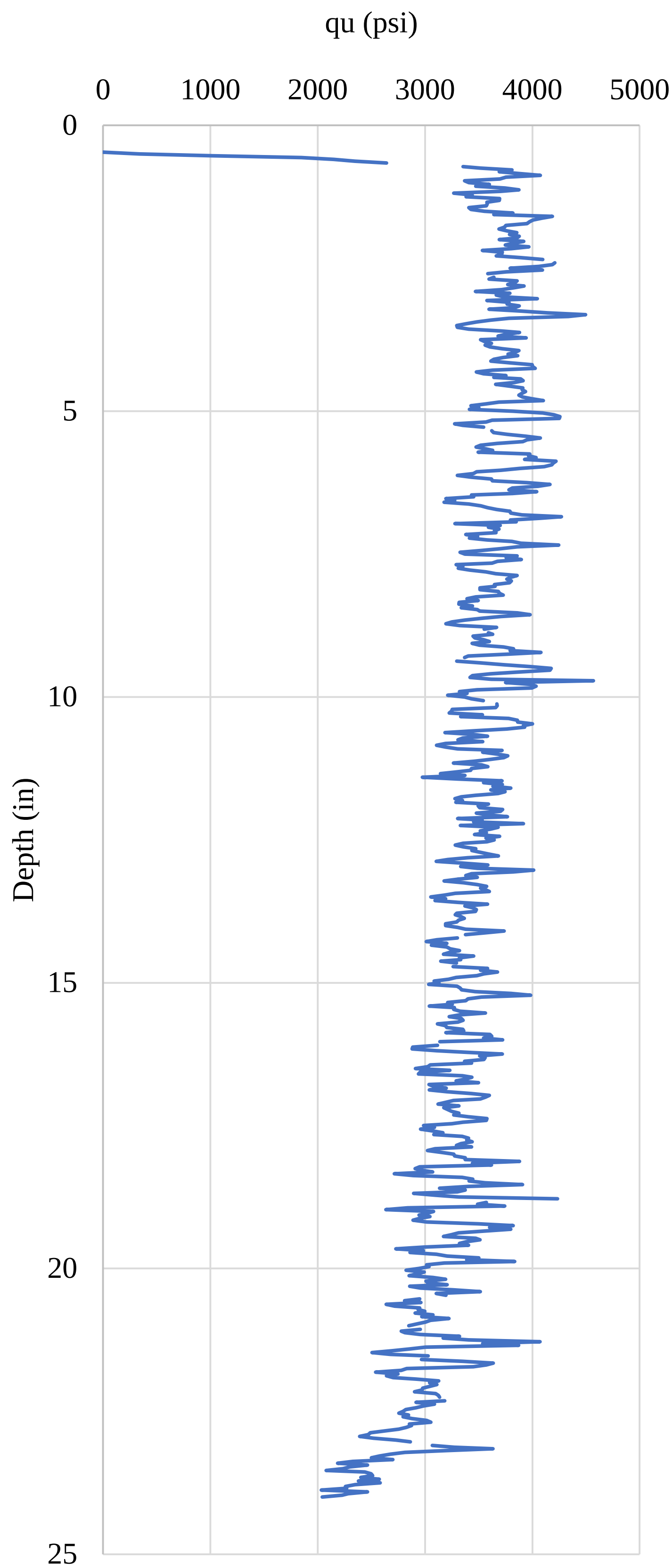


Figure F.122 Core 26-4

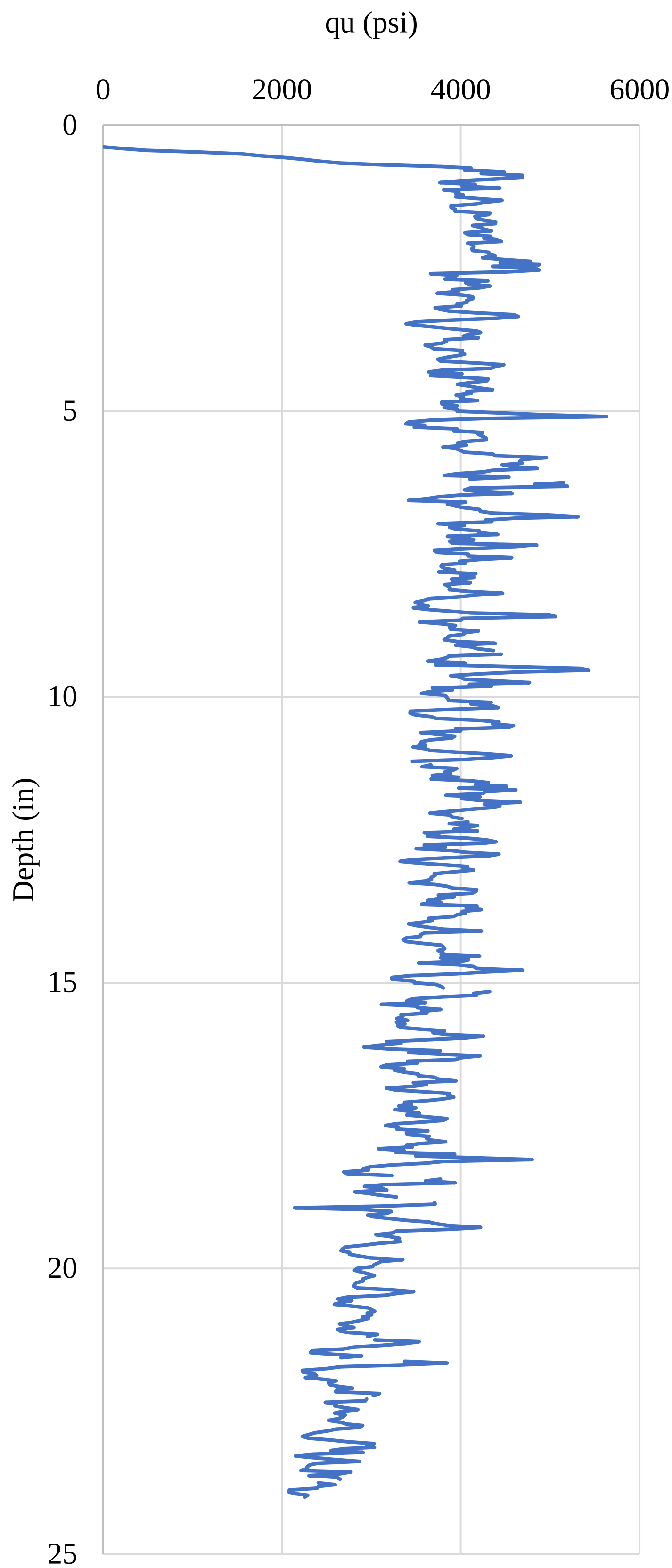


Figure F.123 Core 26-5

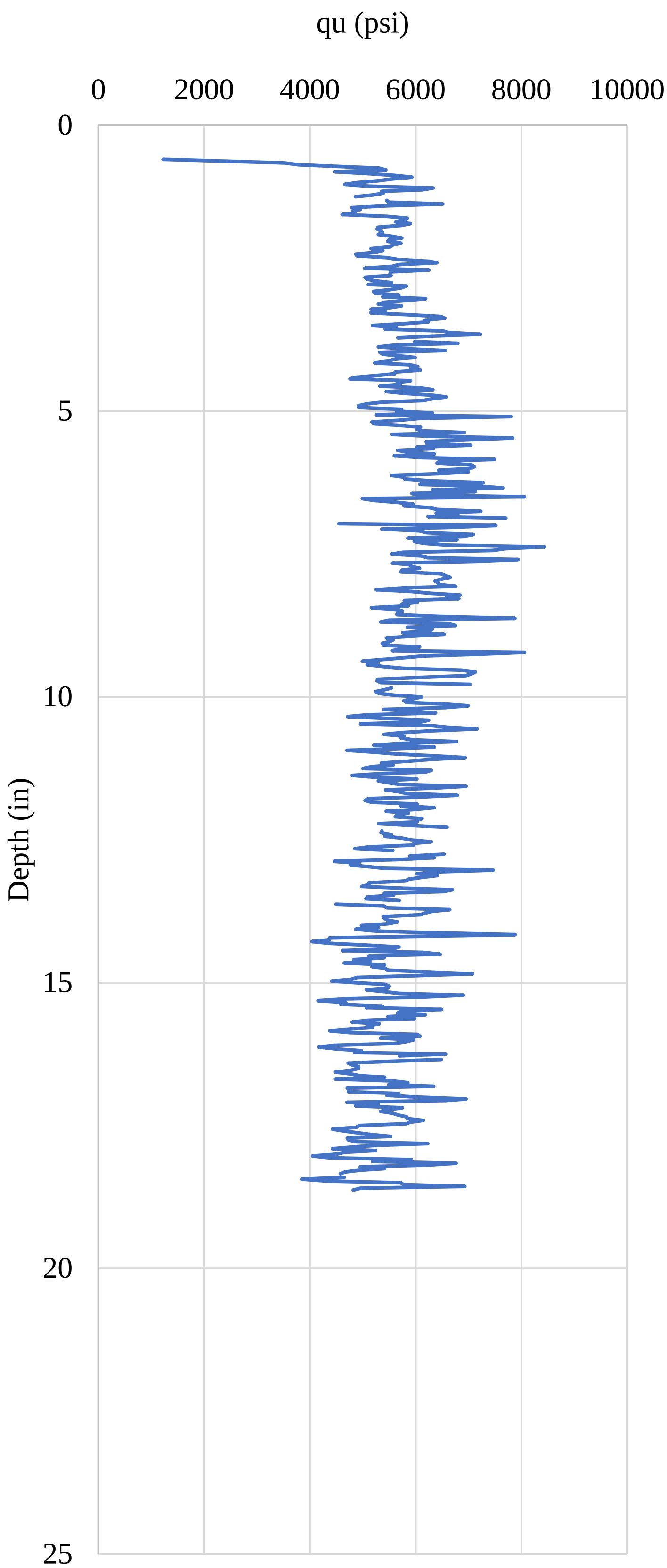


Figure F.124 Core 32-1

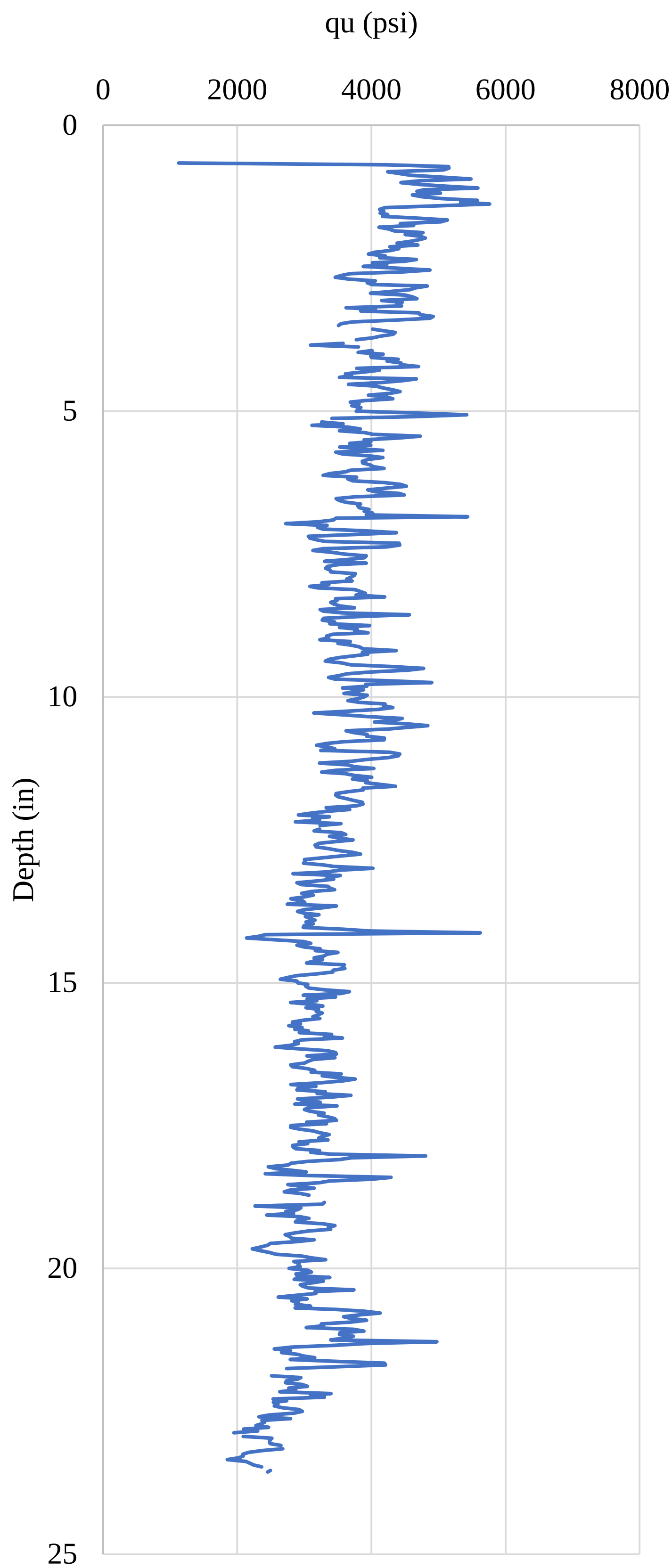


Figure F.125 Core 32-2

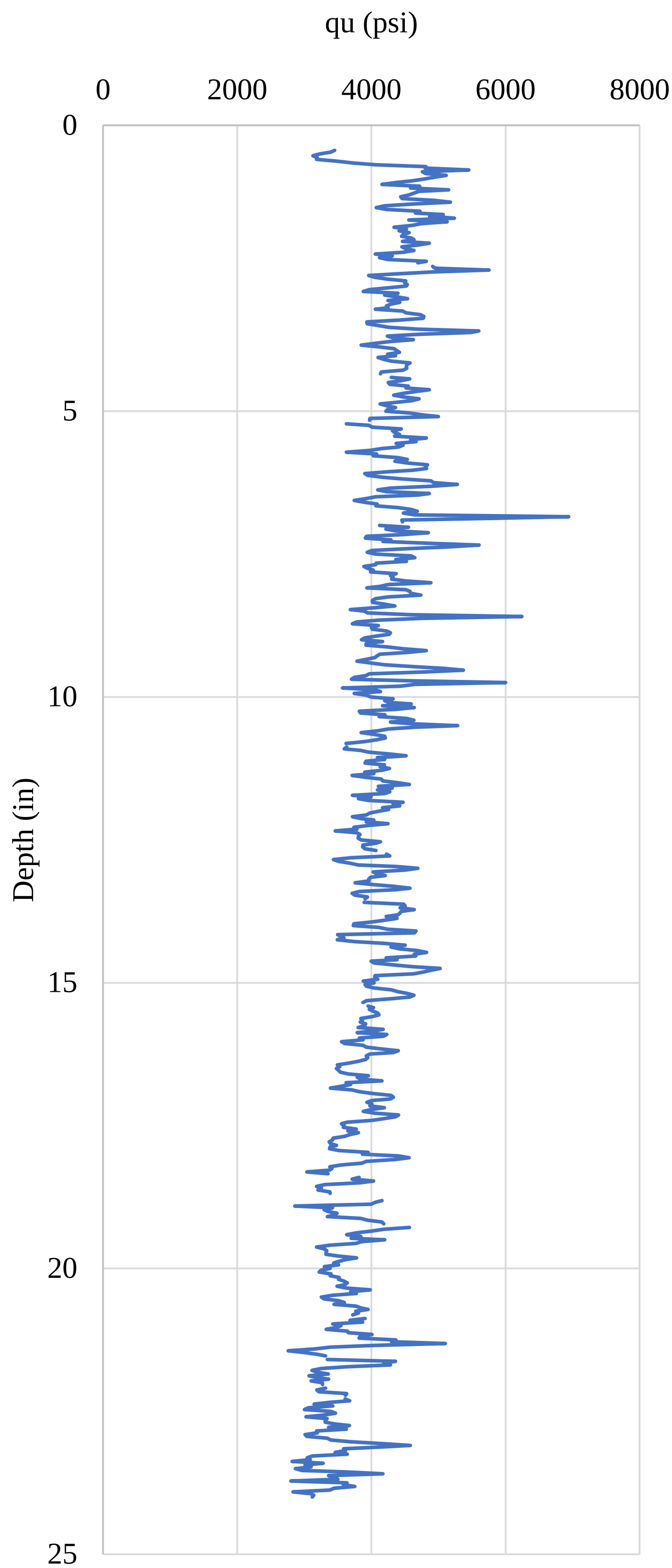


Figure F.126 Core 32-3

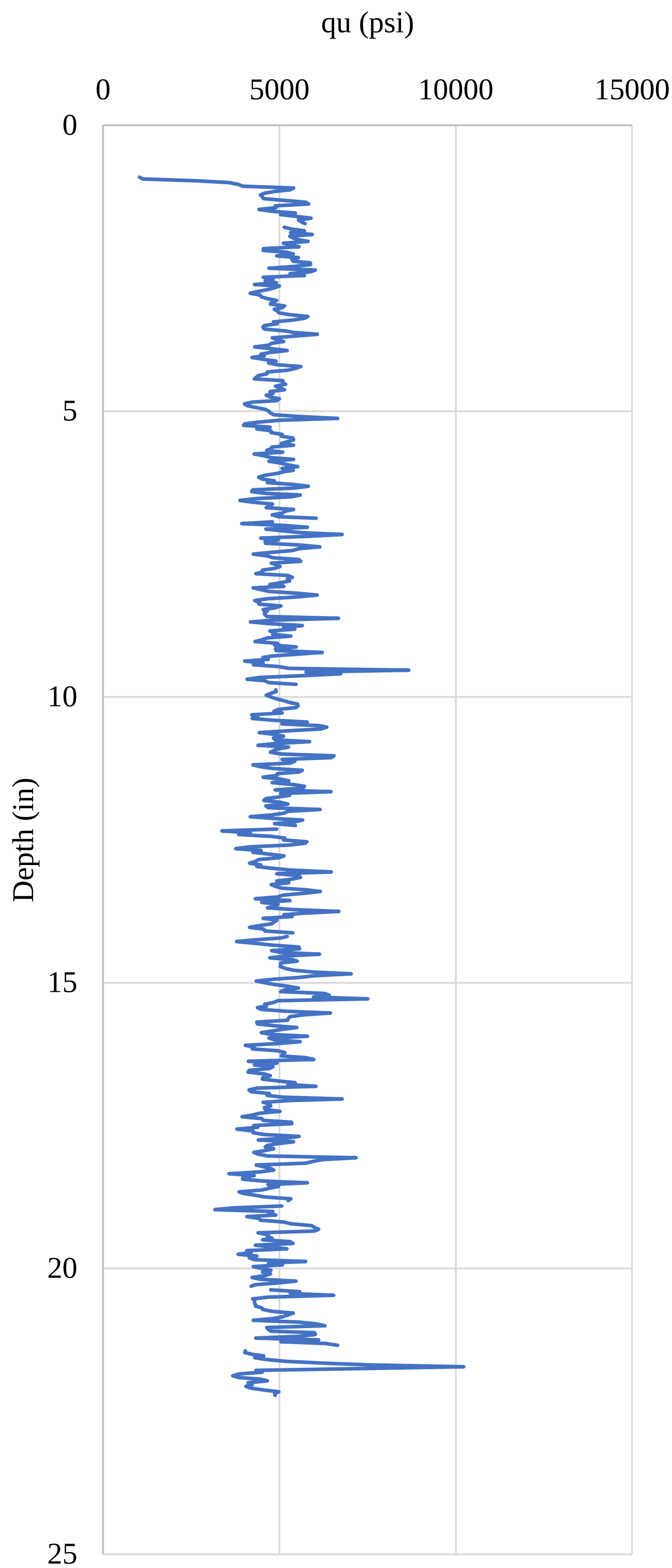


Figure F.127 Core 32-4

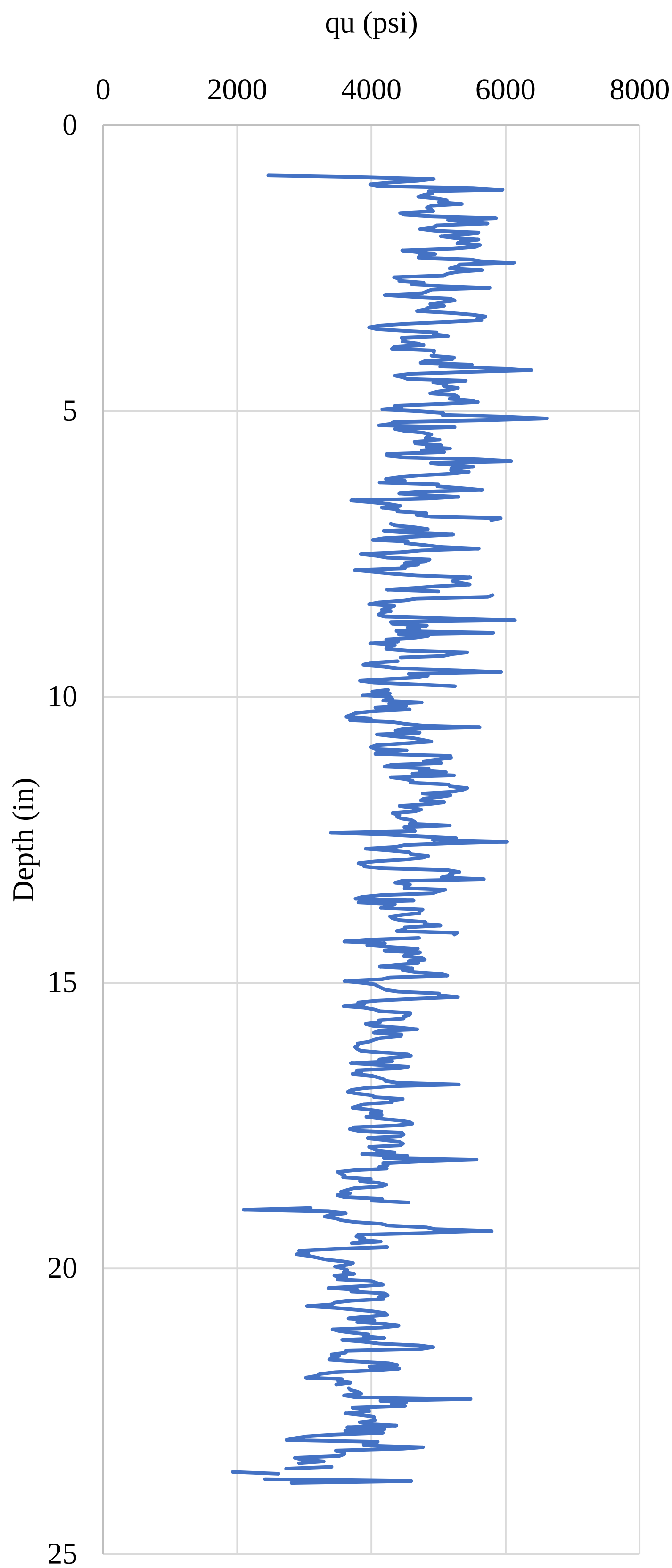


Figure F.128 Core 32-5

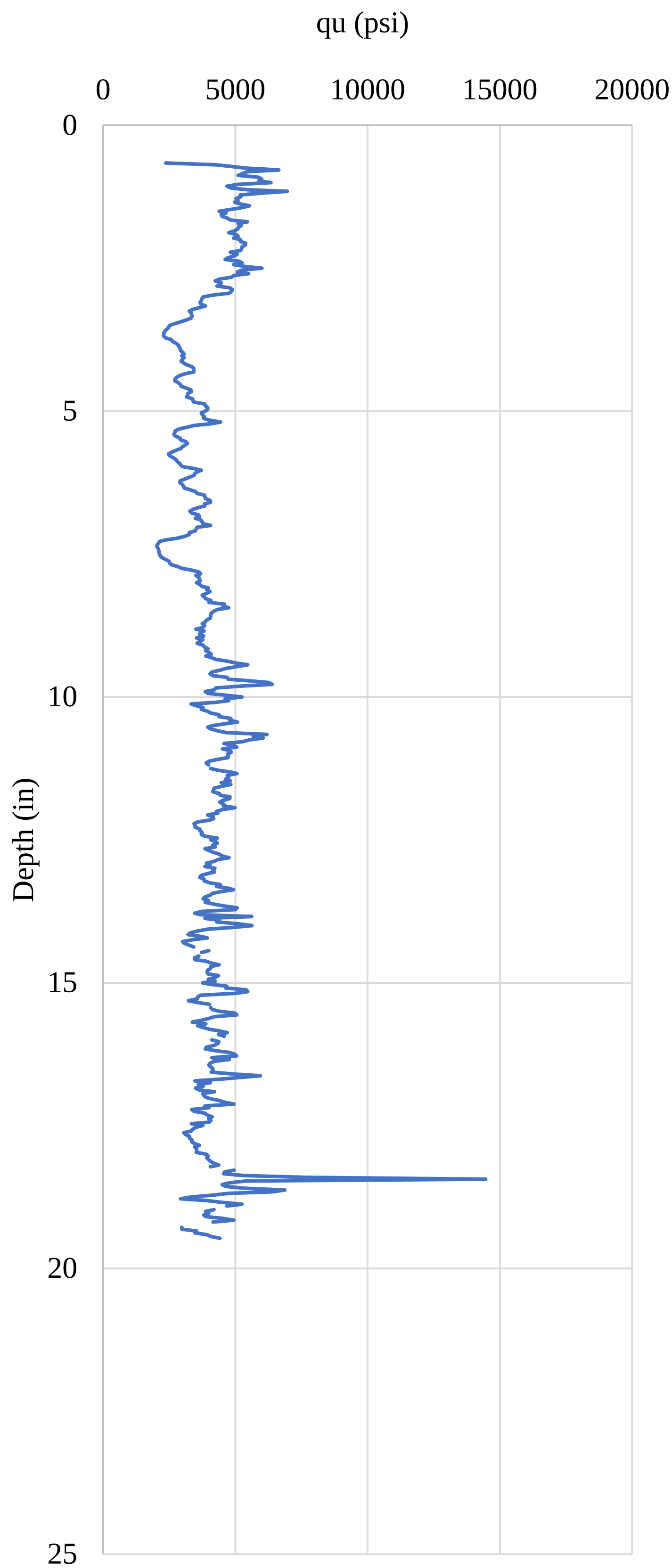


Figure F.129 Core 45-1

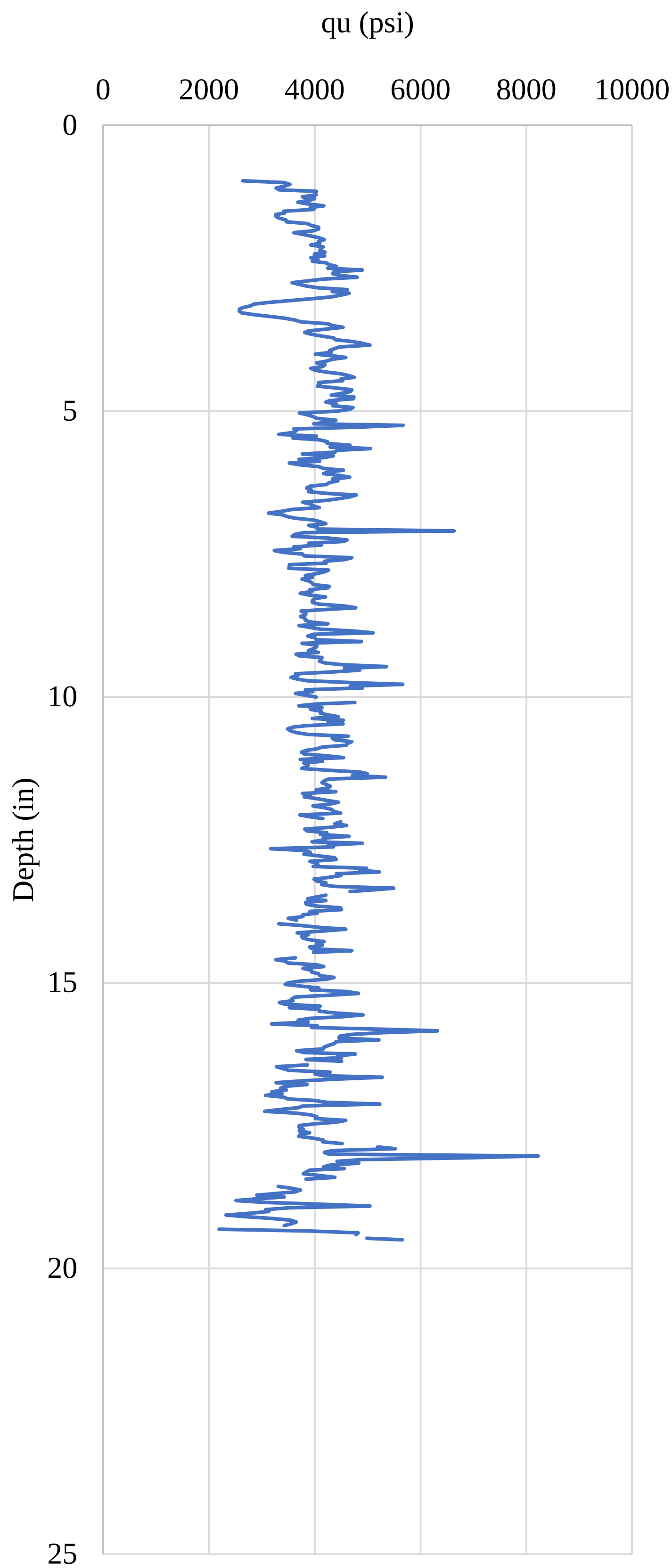


Figure F.130 Core 45-2

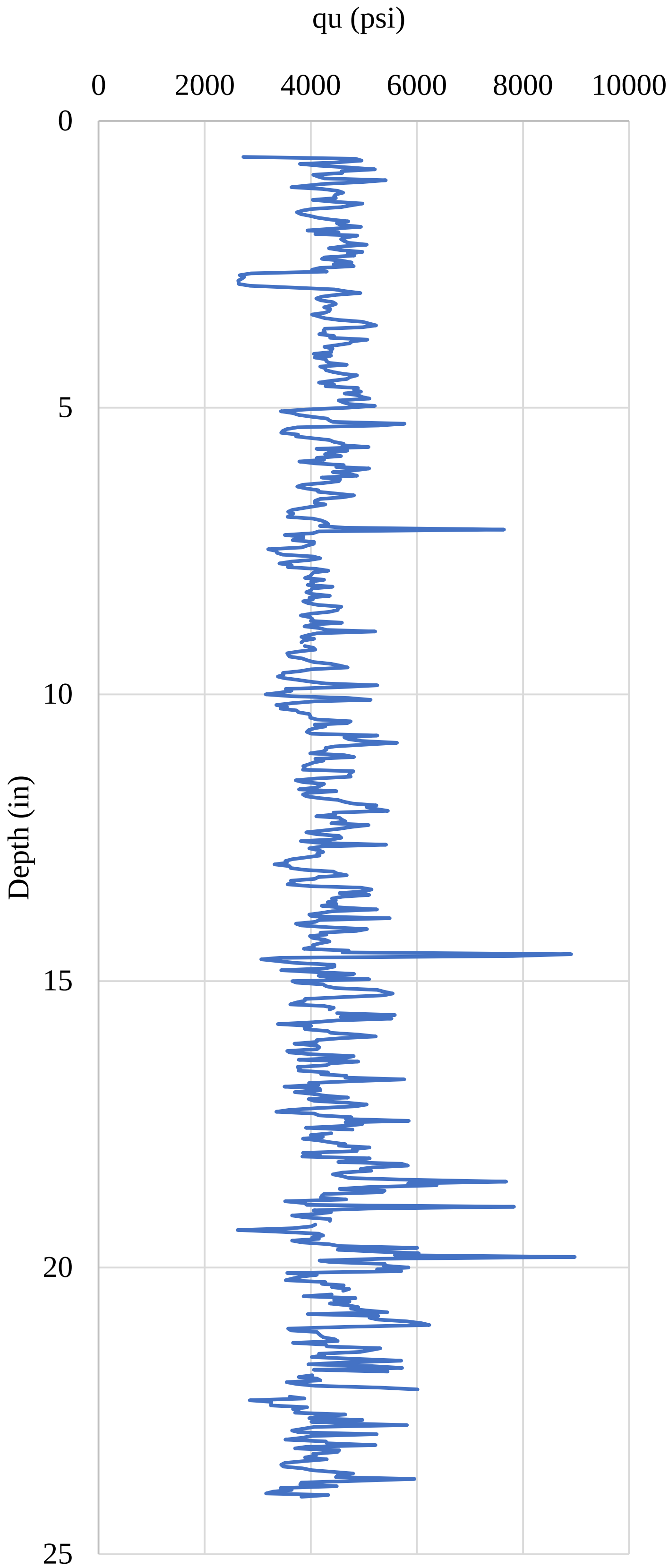


Figure F.131 Core 45-3

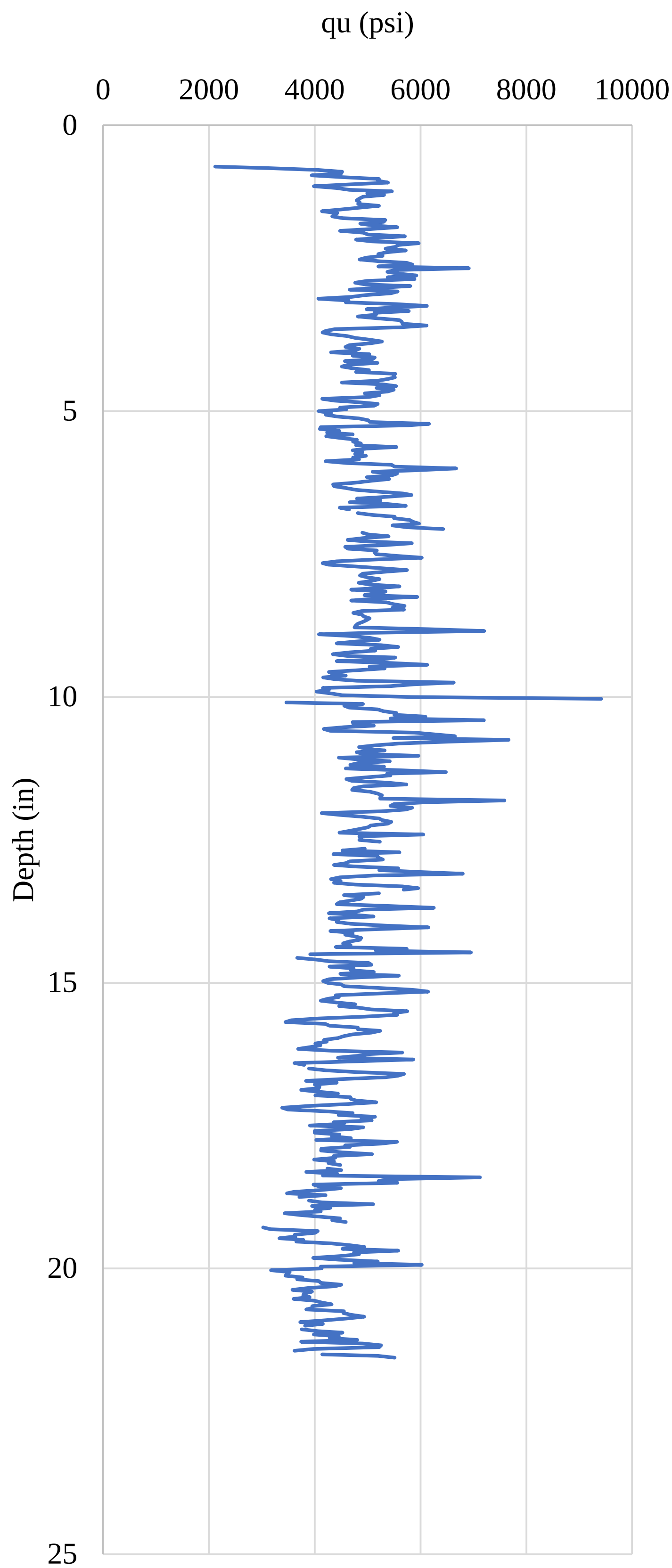


Figure F.132 Core 45-4

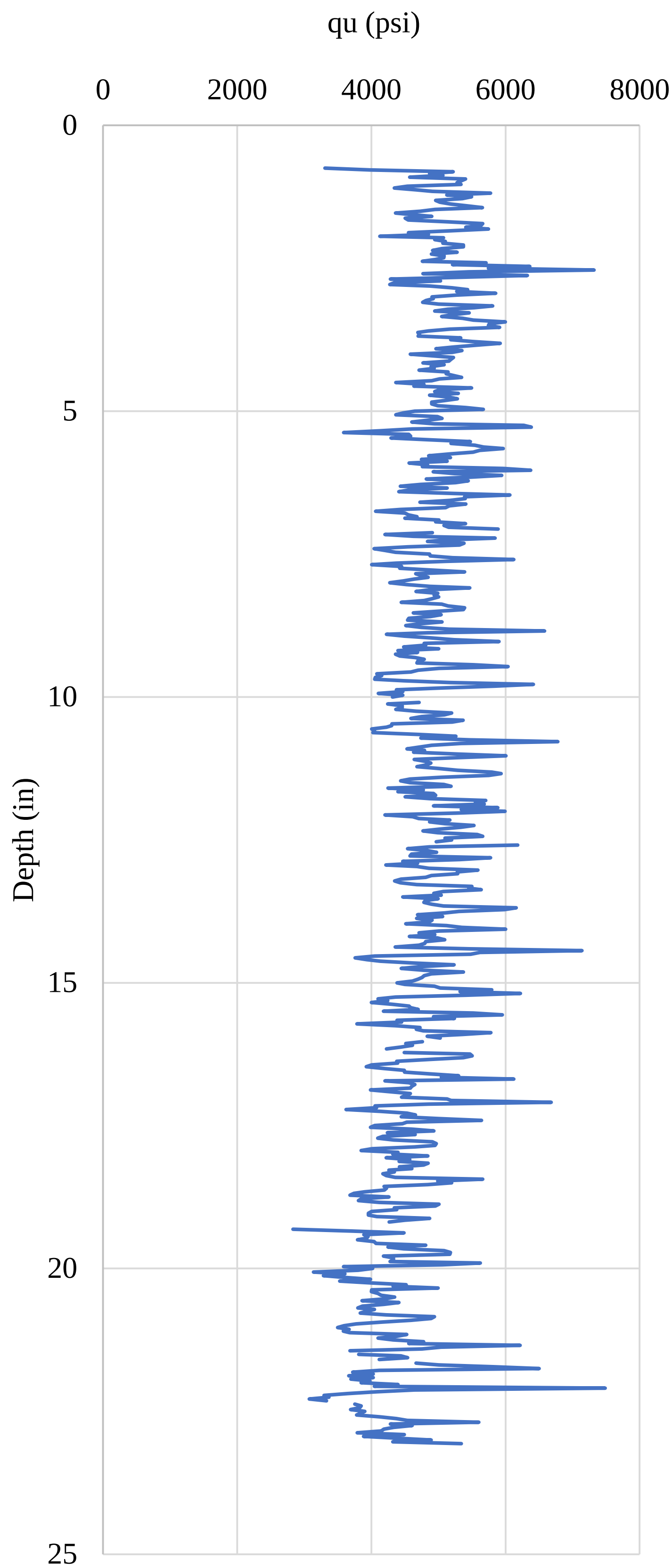


Figure F.133 Core 45-5

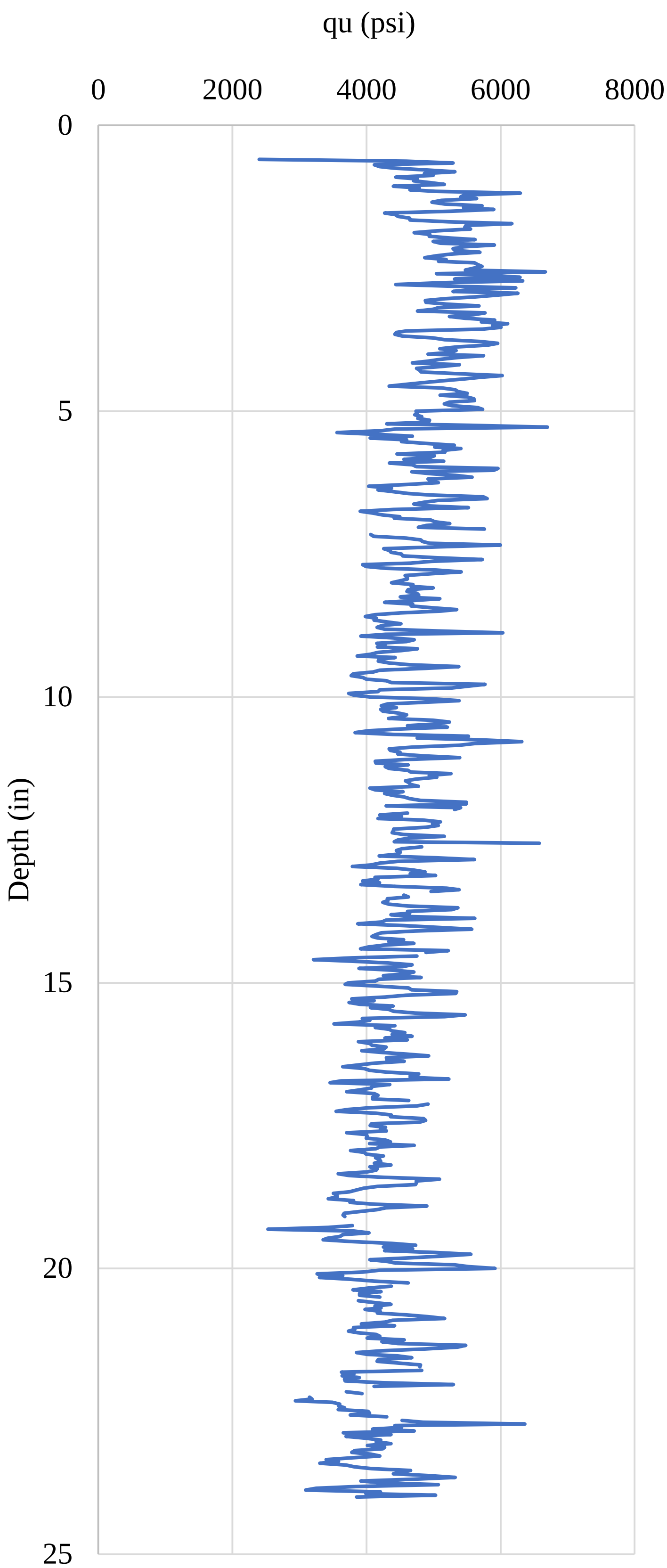


Figure F.134 Core 46-1

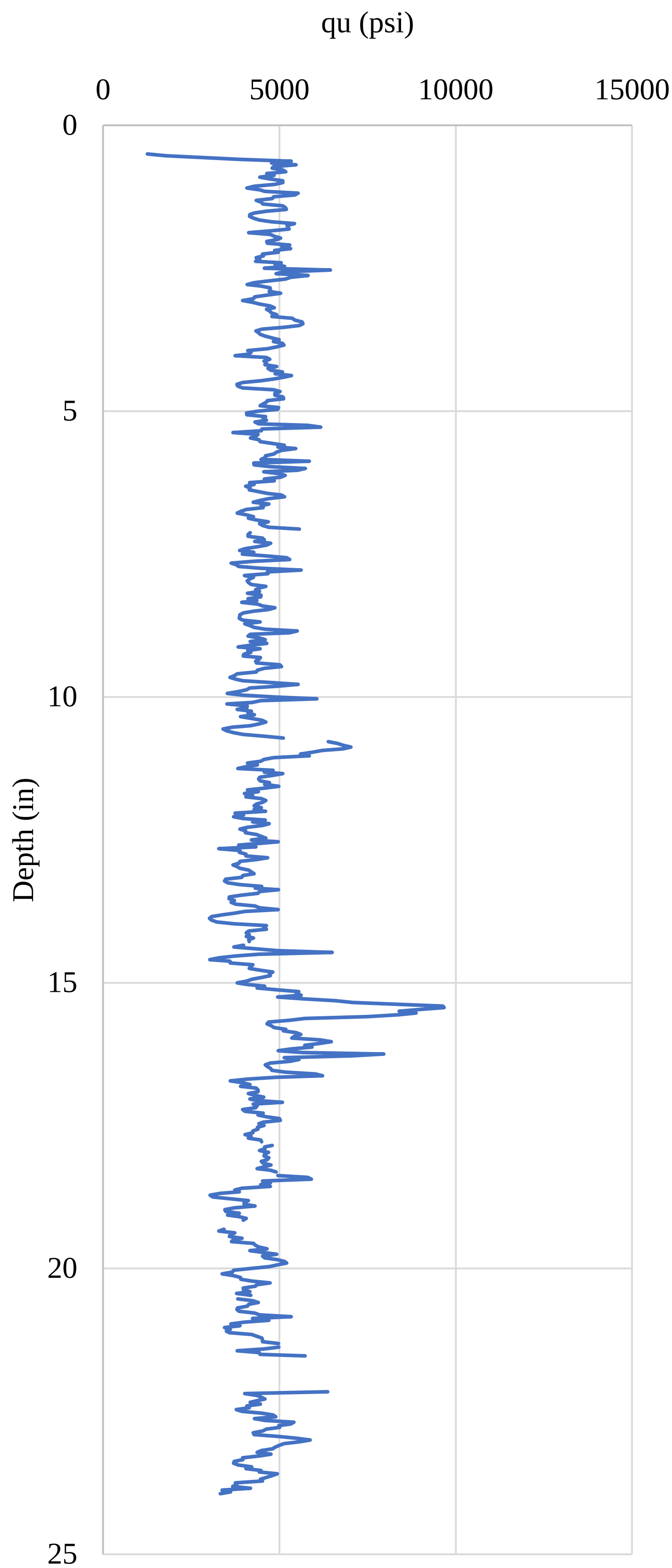


Figure F.135 Core 46-2

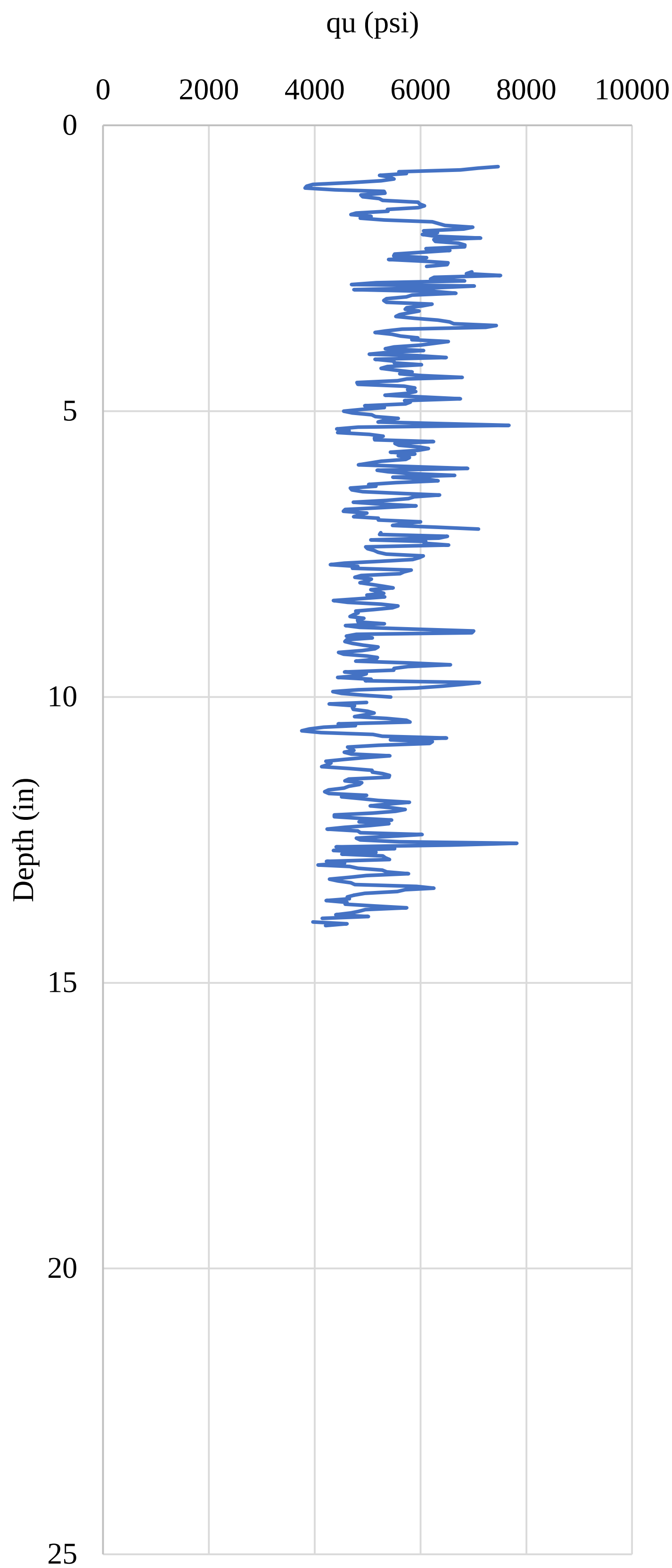


Figure F.136 Core 46-3

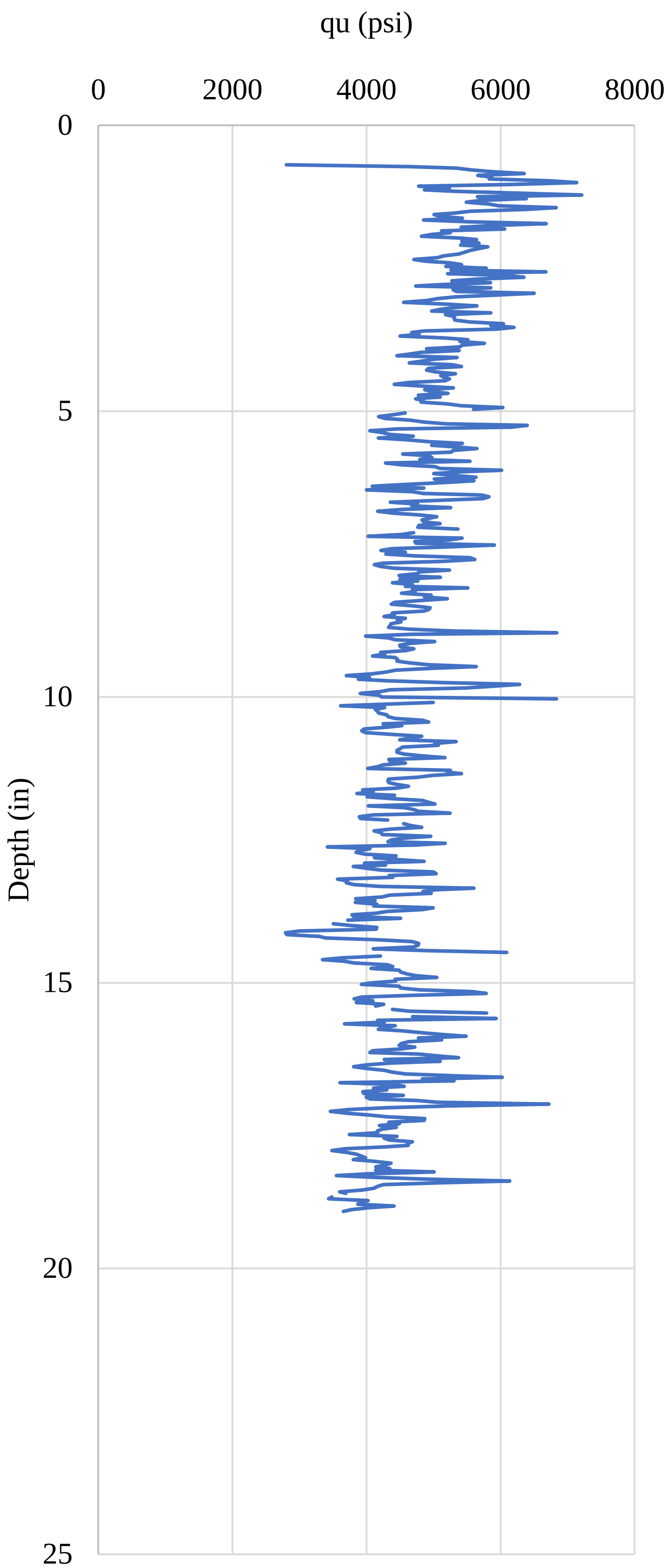


Figure F.137 Core 46-4

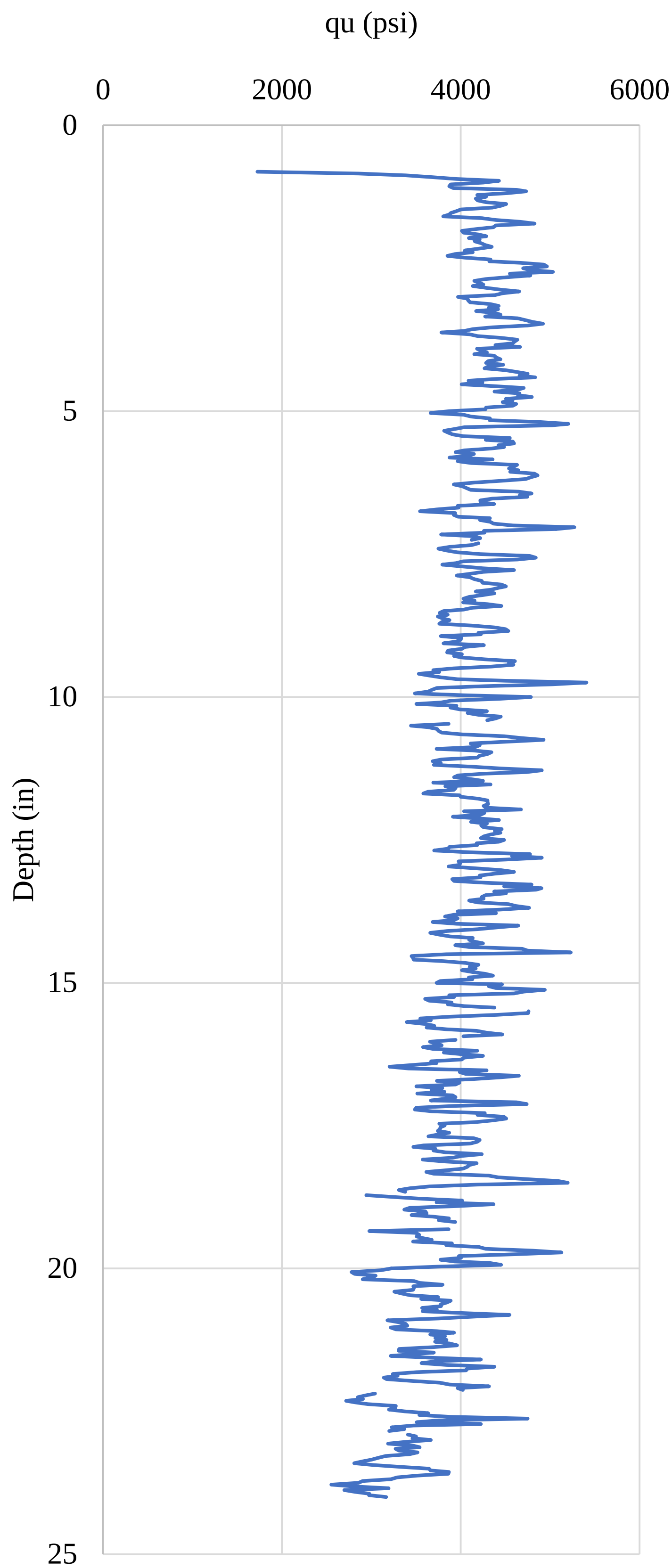


Figure F.138 Core 50-1

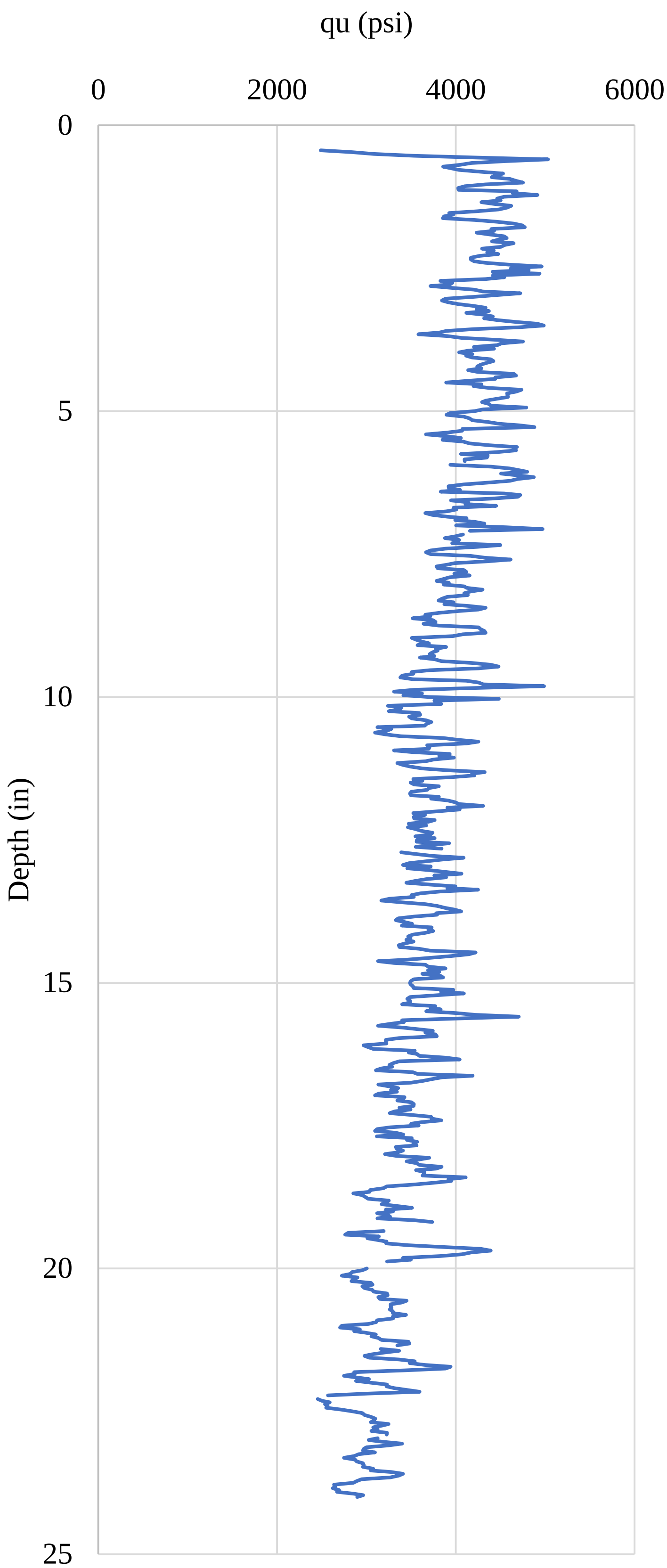


Figure F.139 Core 50-2

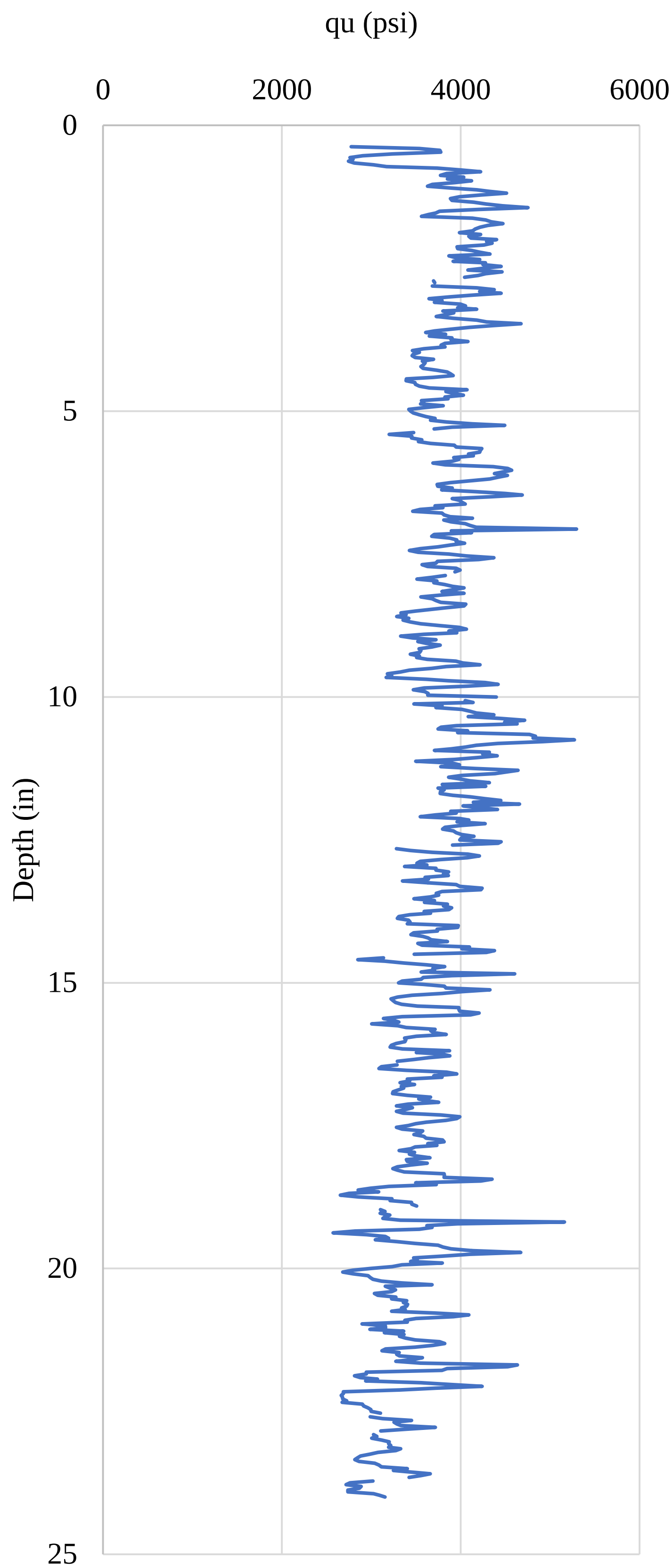


Figure F.140 Core 50-3

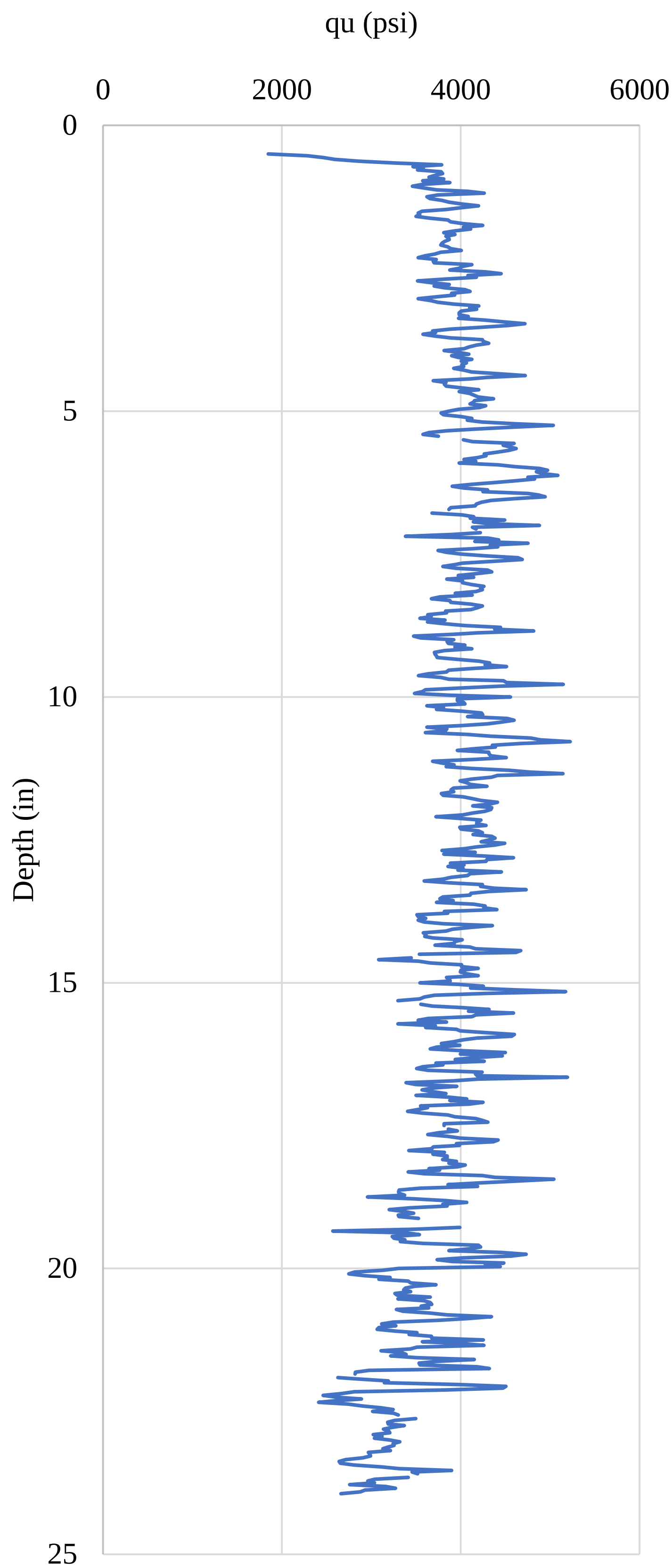


Figure F.141 Core 50-4

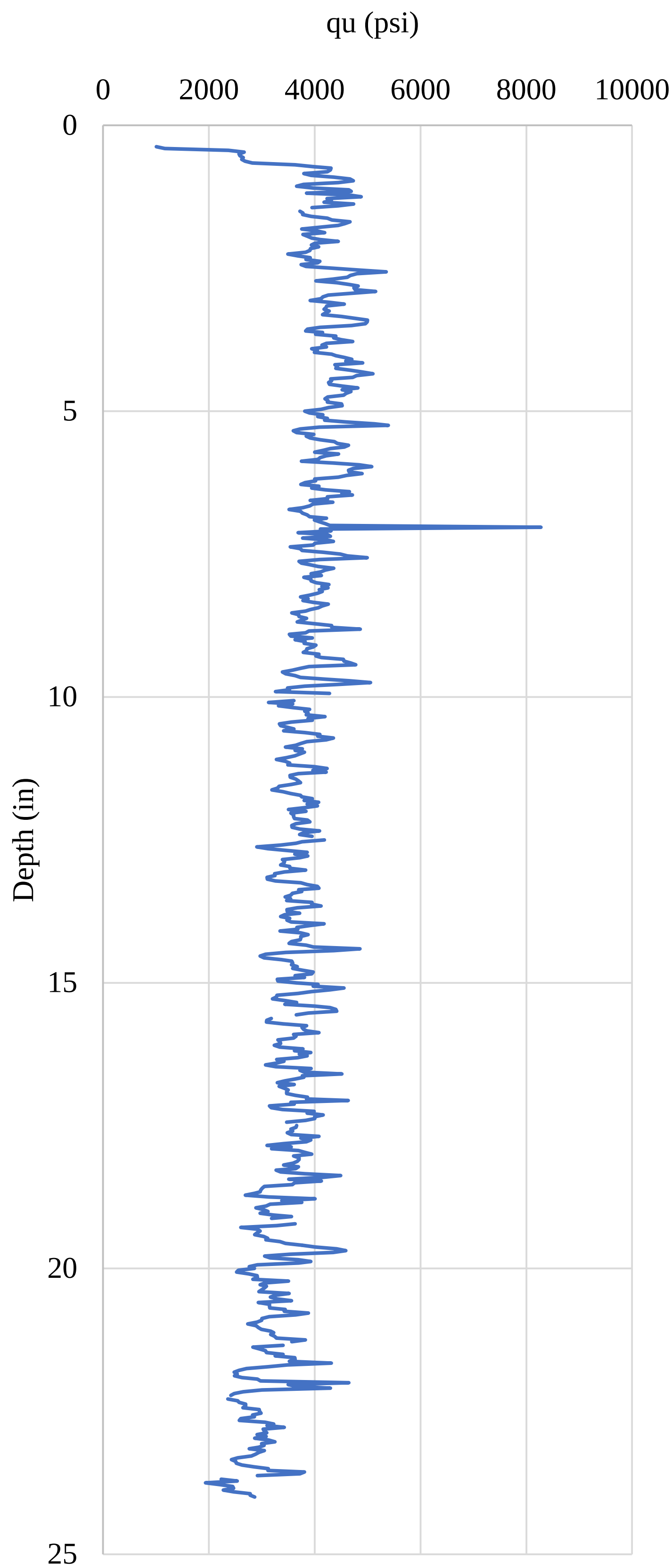


Figure F.142 Core 50-5

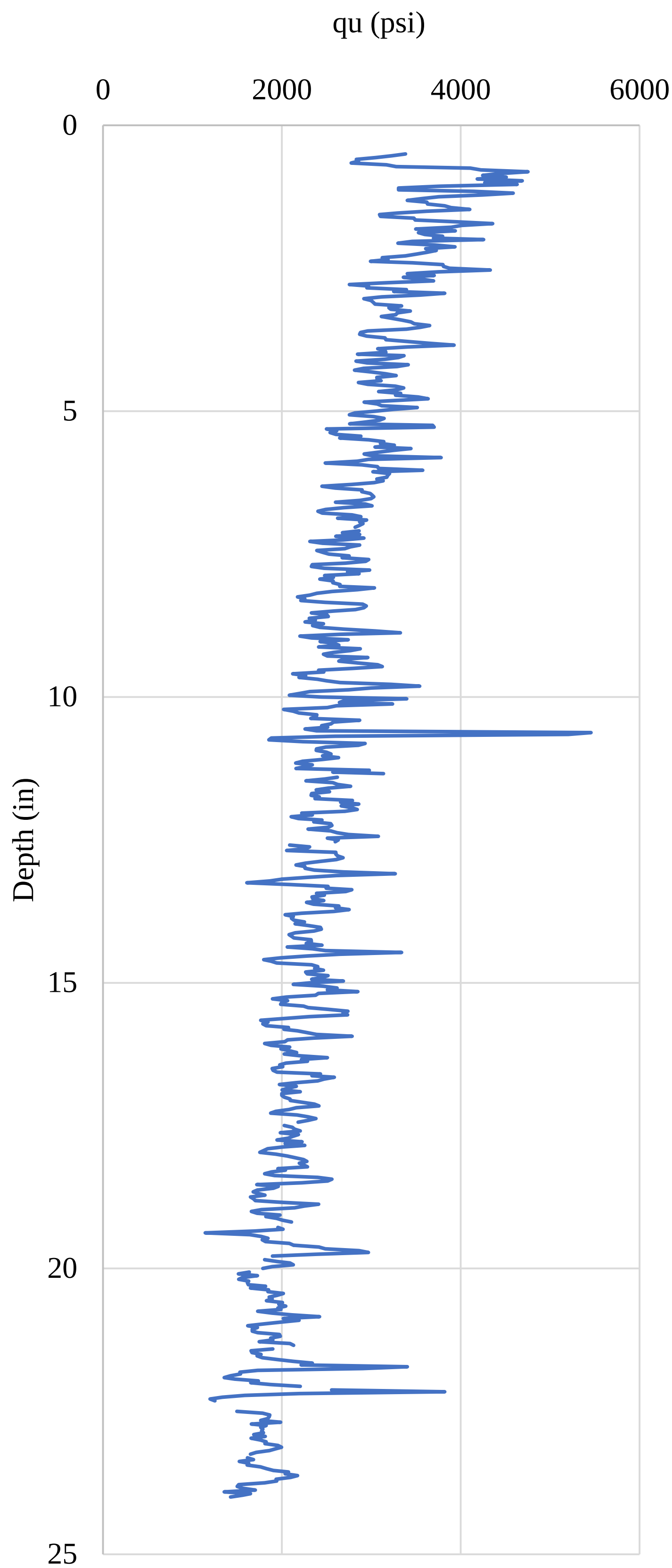


Figure F.143 Core 51-1

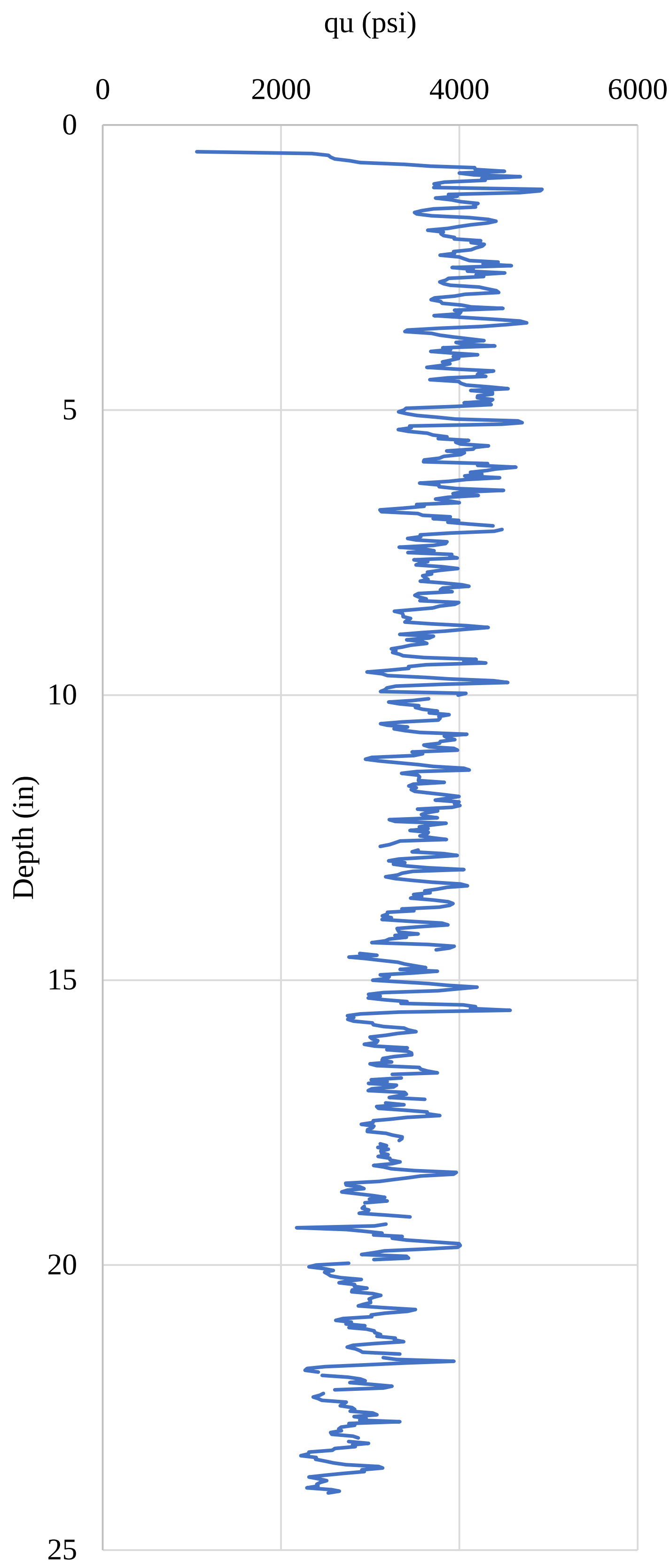


Figure F.144 Core 51-2

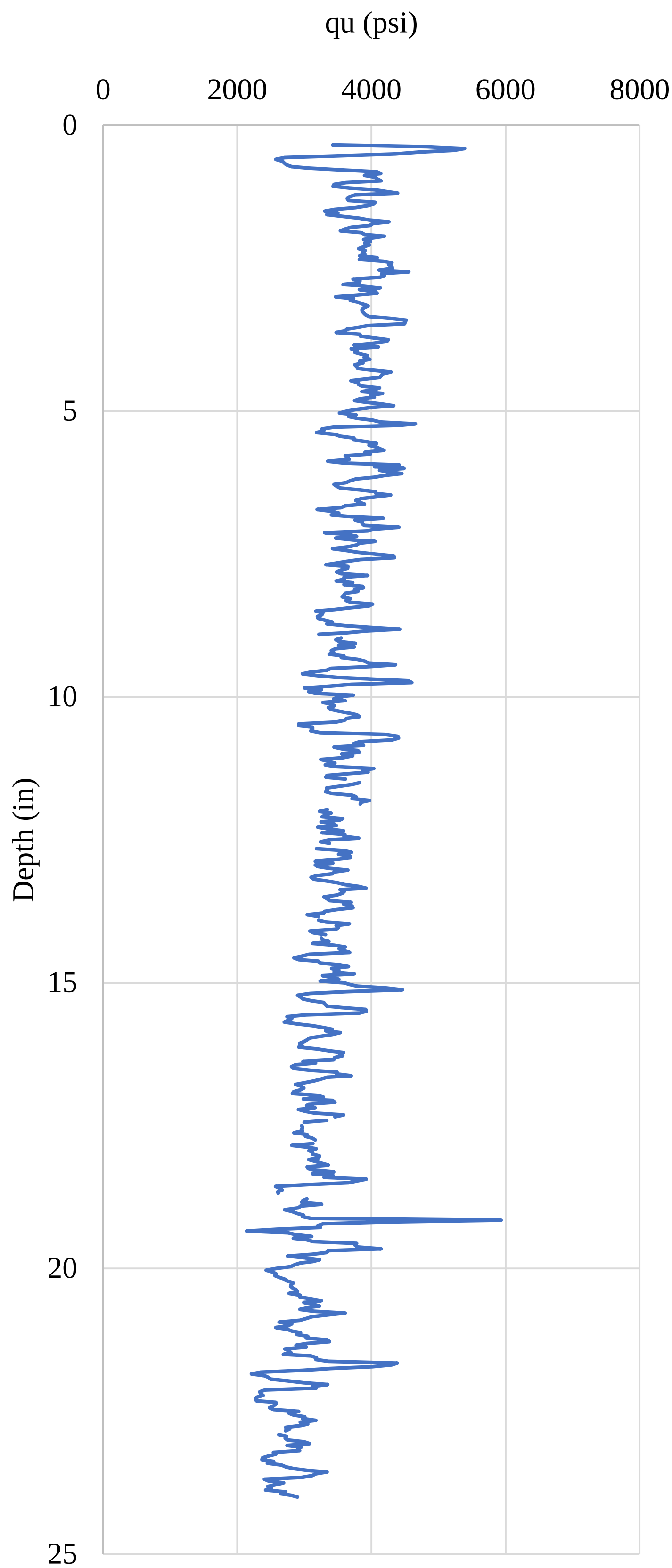


Figure F.145 Core 51-3

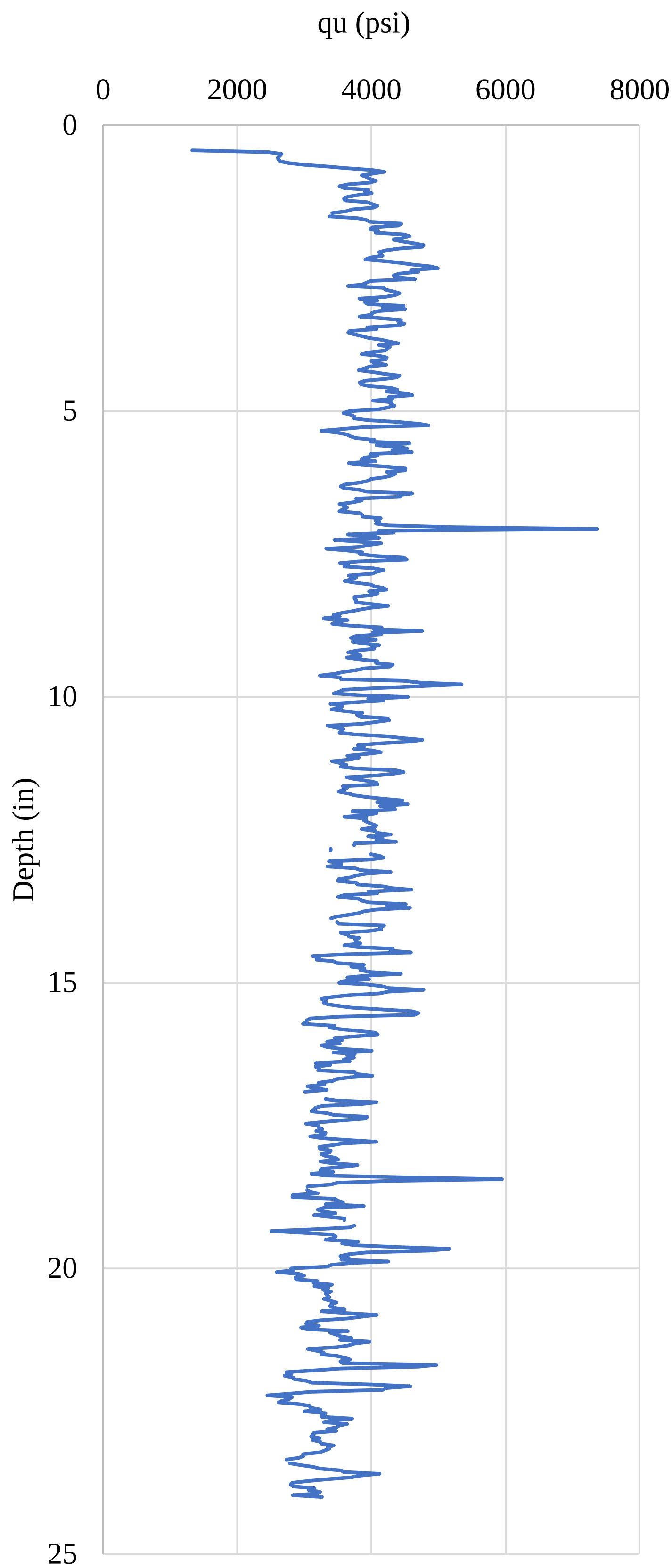


Figure F.146 Core 51-4

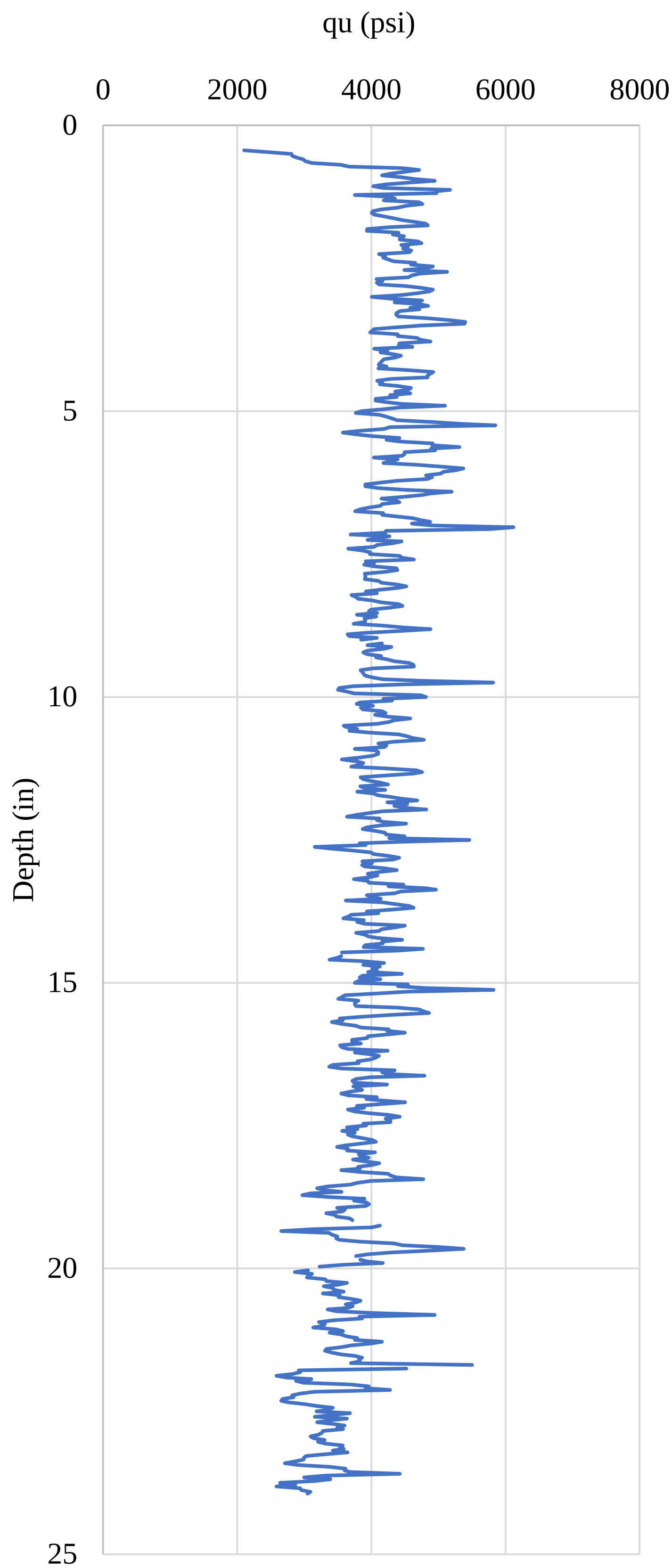
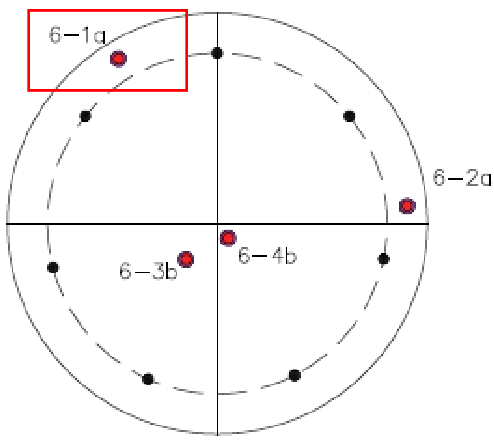
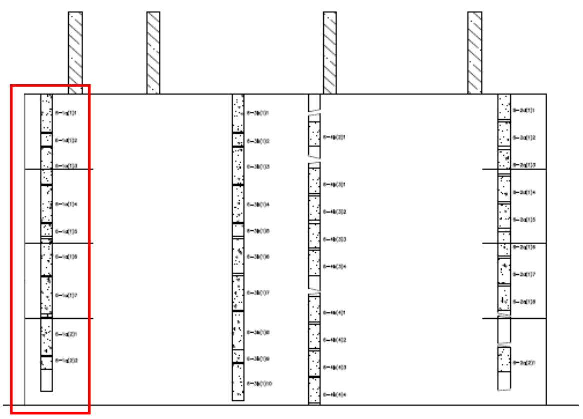


Figure F.147 Core 51-5

APPENDIX G. MIP SAMPLE LOCATION





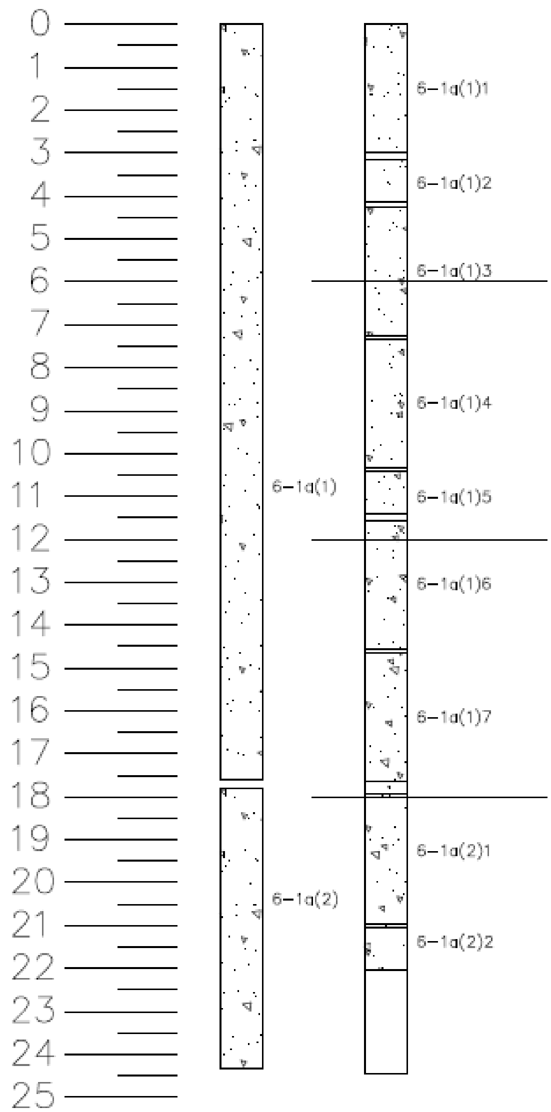


Figure G.1. Core location and profile for 6-1a

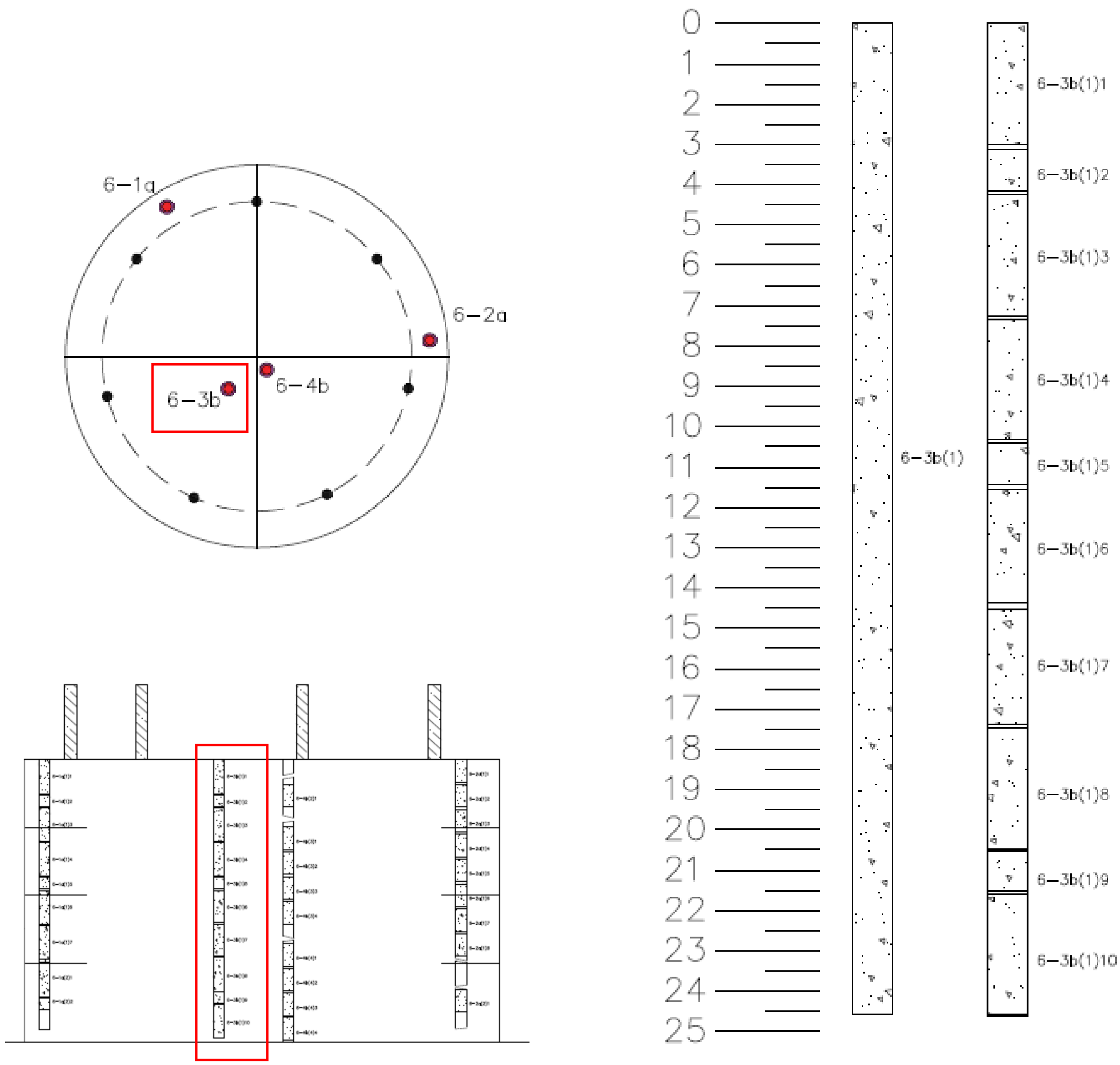


Figure G.2 Core location and profile for 6-3b.

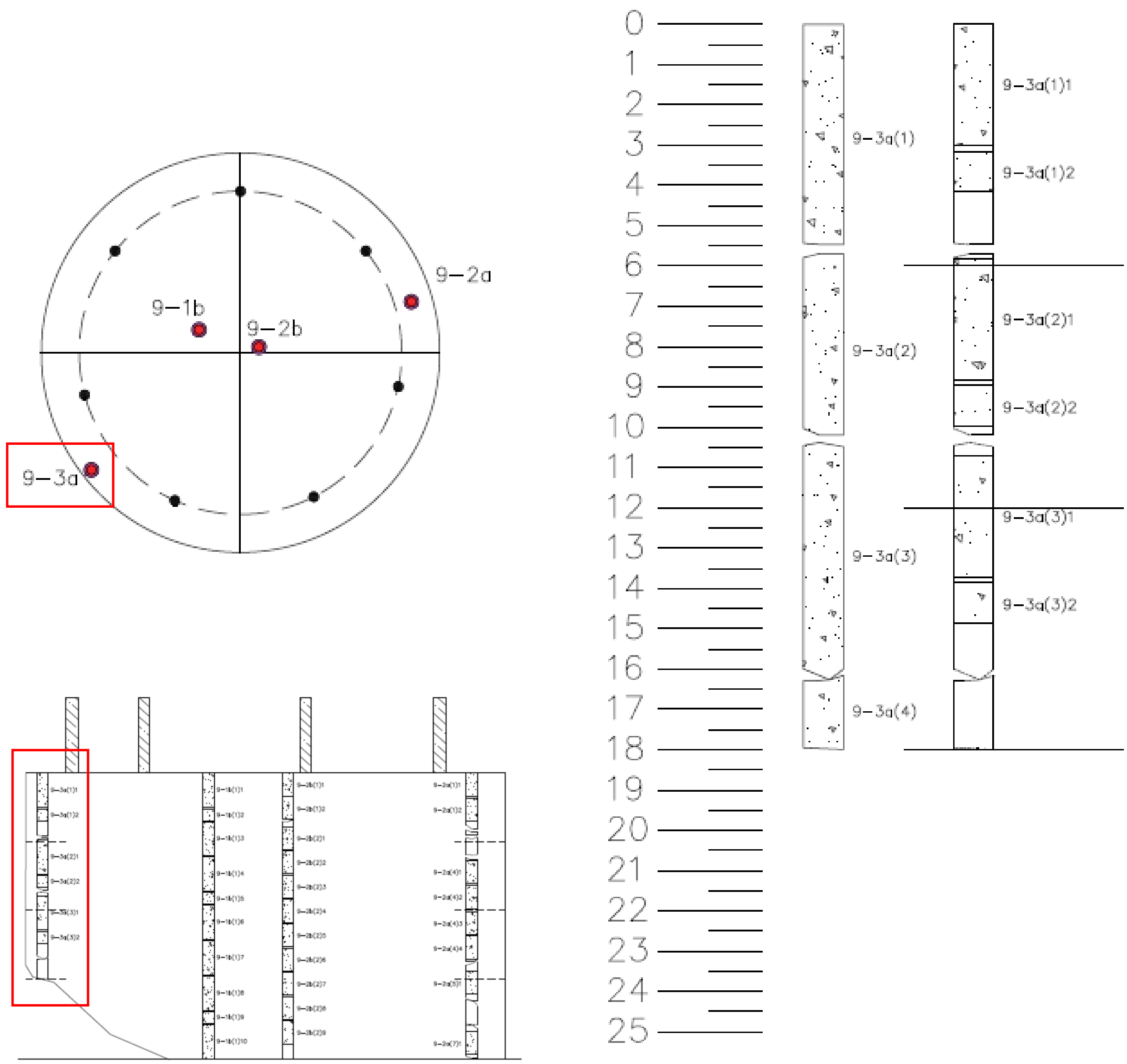


Figure G.3 Core location and profile for 9-3a.

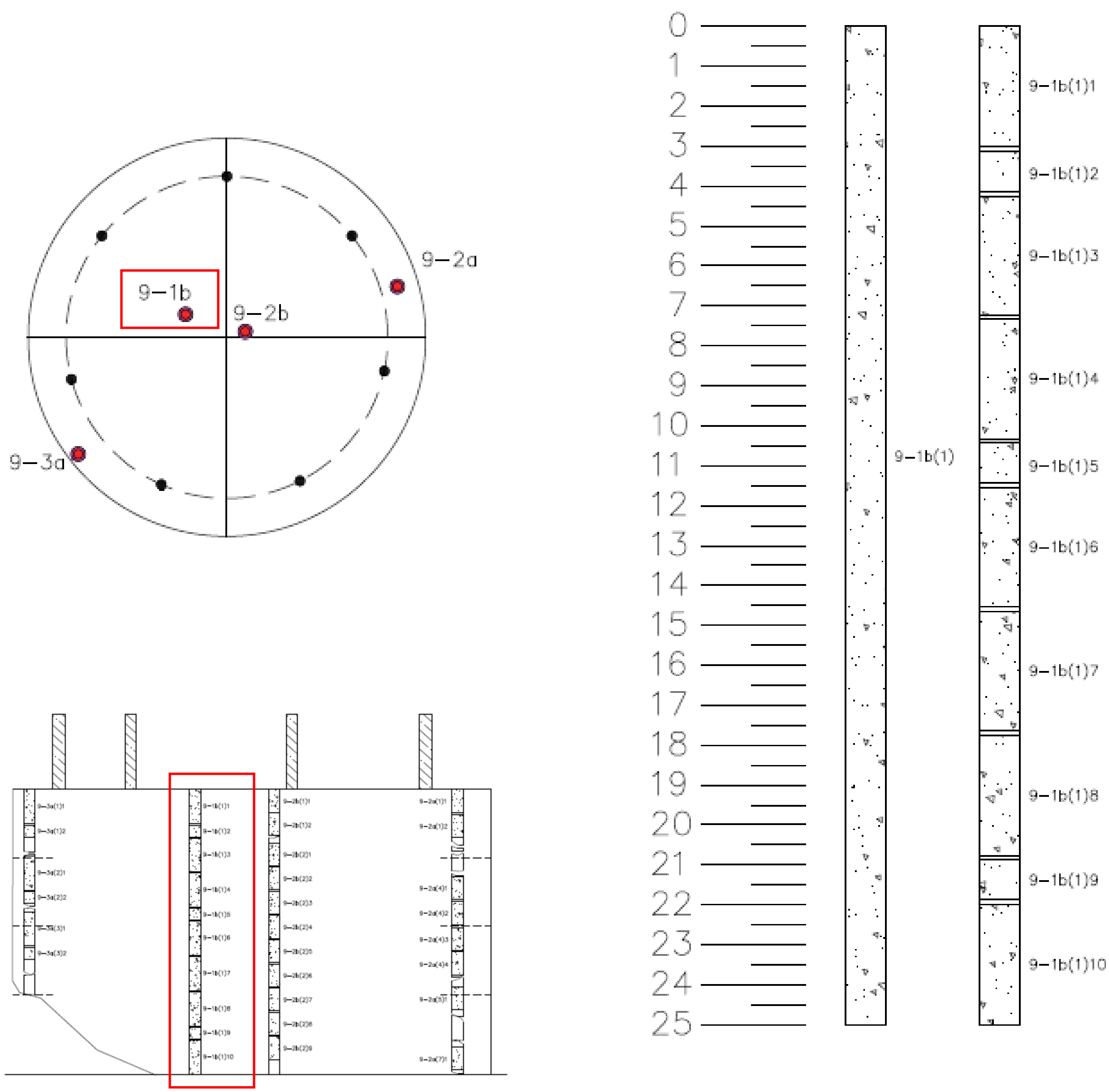


Figure G.4 Core location and profile for 9-1b.

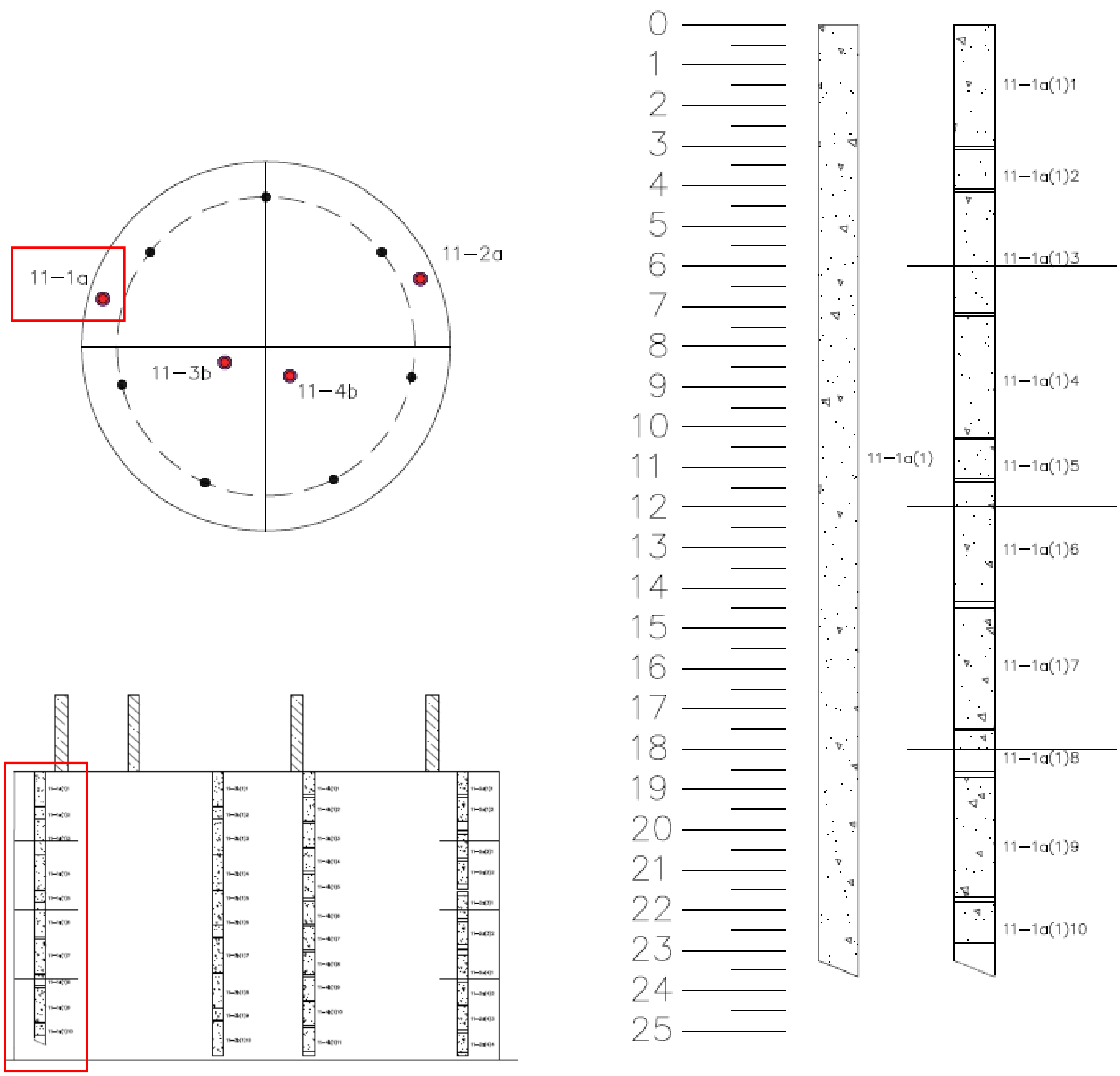


Figure G.5 Core location and profile for 11-1a.

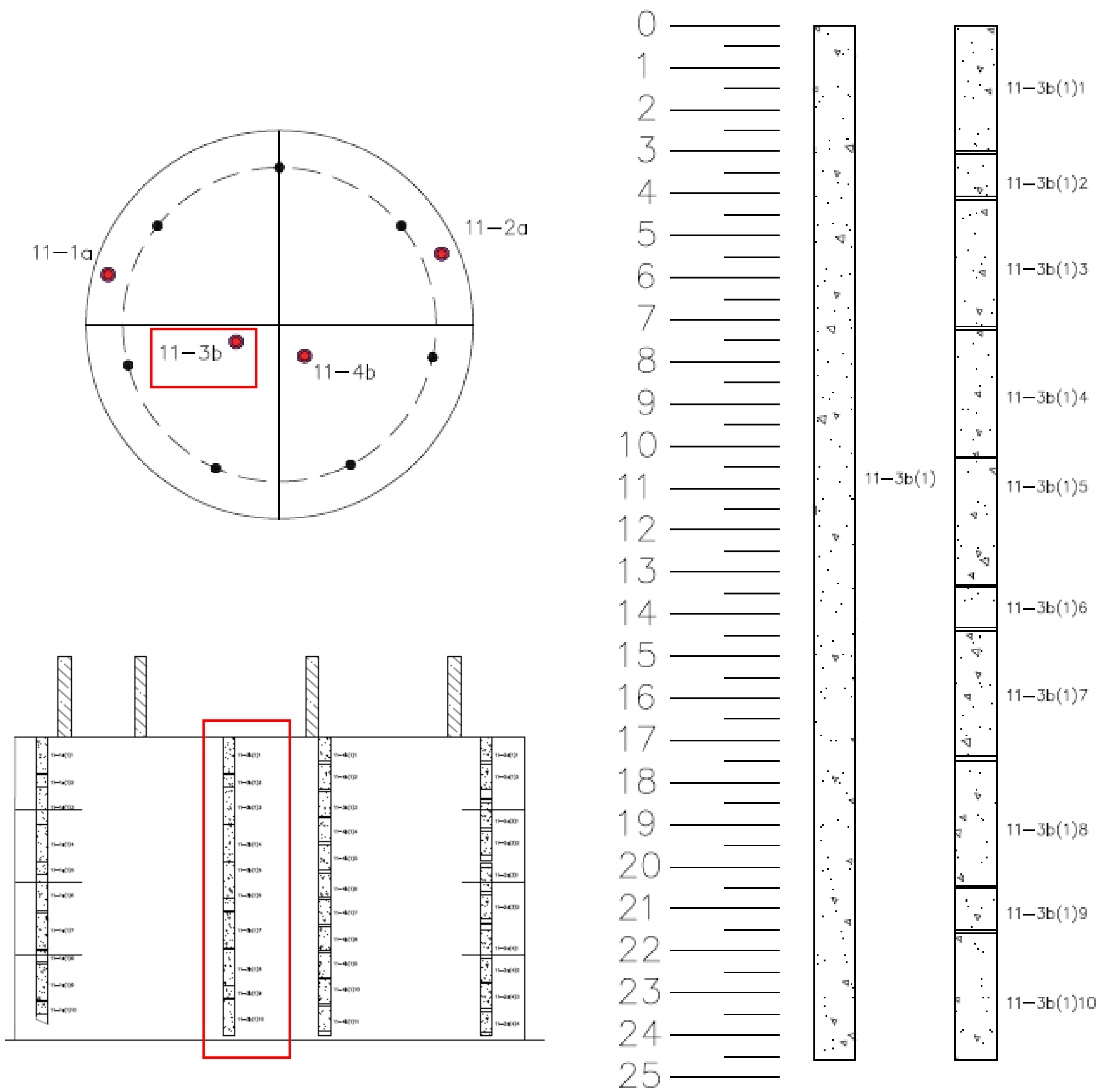


Figure G.6 Core location and profile for 11-3b.

Table G.1 Mix proportions for 25% replacement mix without bentonite using deionized water for cubes 1-9 tested at 3 days, 7 days, and 28 days.

	Weight (g)
Cement	1542.2
Deionized Water	961.2
w/c	0.62

Table G.2 Mix proportions for 50% replacement mix without bentonite using deionized water for cubes 1-3 tested at 3 days.

	Weight (g)
Cement	342.4
Deionized Water	366.5
w/c	1.07

Table G.3 Mix proportions for 50% replacement mix without bentonite using deionized water for cubes 4-9 tested at 7 days and 28 days.

	Weight (g)
Cement	684.9
Deionized Water	733.0
w/c	1.07

Table G.4 Mix proportions for 25% replacement mix without bentonite using soda ash water for cubes tested at 3 days.

	Weight (g)
Cement	514.6
Soda Ash Water	320.6
w/c	0.62

Table G.5 Mix proportions for 50% replacement mix without bentonite using soda ash water for cubes tested at 3 days.

	Weight (g)
Cement	345.0
Soda Ash Water	360.1
w/c	1.04

APPENDIX H – PHYSICAL SURFACE ROUGHNESS DATA TABLES

Table H.1 Physical surface roughness data summary.

Shaft #	beaker tare (g)	beaker volume (mL)	full beaker (g)	Before weight (g)	After weight (g)	putty density (g/mL)	putty used (g)	void volume (cm^3)	void volume (in^3)	Surface Roughness (in^3/sf)
1	38.7	350	490.07	1394.56	1298.36	1.40	96.2	68.70	4.19	26.93
	38.7	350	494.66	1396.81	1182.04	1.41	214.77	151.96	9.27	
2	38.77	350	495.11	1344.03	1164.35	1.41	179.68	127.02	7.75	22.35
	38.77	350	497.79	1347.13	1267.29	1.42	79.84	56.14	3.43	
3	38.7	350	493.31	1394.5	1348.19	1.41	46.31	32.86	2.01	9.25
	38.7	350	492.61	1398.85	1338.44	1.41	60.41	42.92	2.62	
4	38.77	350	489.96	1339.53	1251.95	1.40	87.58	62.56	3.82	14.24
	38.77	350	485.93	1339.22	1264.08	1.39	75.14	54.12	3.30	
5	38.7	350	482.22	1388.47	1327.51	1.38	60.96	44.25	2.70	11.54
	38.7	350	491.82	1399.09	1328.38	1.41	70.71	50.32	3.07	
6	38.77	350	489.03	1338.79	1329.1	1.40	9.69	6.94	0.42	1.91
	38.77	350	490.78	1340.55	1328.36	1.40	12.19	8.69	0.53	
7	38.77	350	493.28	1343.19	1216.5	1.41	126.69	89.89	5.49	22.15
	38.77	350	496.07	1345.79	1215.93	1.42	129.86	91.62	5.59	
8	38.7	350	494.01	1390.84	1323.15	1.41	67.69	47.96	2.93	9.79
	38.7	350	492.37	1396.31	1350.96	1.41	45.35	32.24	1.97	
	38.77	350	498.32	1347.37	1333.91	1.42	13.46	9.45	0.58	

Table H.1 (Continued)

Shaft #	beaker tare (g)	beaker volume (mL)	full beaker (g)	Before weight (g)	After weight (g)	putty density (g/mL)	putty used (g)	void volume (cm^3)	void volume (in^3)	Surface Roughness (in^3/sf)
9	38.77	350	500.25	1349.73	1290.17	1.43	59.56	41.67	2.54	11.83
	38.77	350	499.38	1350.96	1272.1	1.43	78.86	55.27	3.37	
10	38.77	350	500.25	1520	1390	1.43	130	90.95	5.55	24.56
	38.77	350	498.32	1438	1281	1.42	157	110.27	6.73	
11	38.77	350	496.87	1345.32	1320.96	1.42	24.36	17.16	1.05	3.25
	38.77	350	498.32	1347.37	1333.91	1.42	13.46	9.45	0.58	
12	38.77	350	490.03	1340.92	1323.08	1.40	17.84	12.74	0.78	2.70
	38.77	350	489.99	1339.25	1326.1	1.40	13.15	9.39	0.57	
13	38.7	350	488.31	1388.08	1340.11	1.40	47.97	34.38	2.10	7.68
	38.7	350	493.42	1393.91	1353.62	1.41	40.29	28.58	1.74	
14	38.7	350	485.23	1383.62	1307.7	1.39	75.92	54.76	3.34	13.62
	38.7	350	487.09	1386.89	1307.82	1.39	79.07	56.82	3.47	
15	38.7	350	491.2	1392.76	1324.62	1.40	68.14	48.55	2.96	11.97
	38.7	350	489.37	1393.83	1324.61	1.40	69.22	49.51	3.02	
16	38.77	350	491.21	1340.6	1320.26	1.40	20.34	14.49	0.88	3.30
	38.77	350	490.18	1339.9	1322.34	1.40	17.56	12.54	0.77	
17	38.7	350	486.51	1385.83	1376.91	1.39	8.92	6.42	0.39	1.74
	38.7	350	489.34	1388.96	1378.01	1.40	10.95	7.83	0.48	
18	38.77	350	489.86	1339.19	1325.42	1.40	13.77	9.84	0.60	2.16
	38.77	350	490.44	1337.62	1326.64	1.40	10.98	7.84	0.48	
19	38.7	350	487.53	1388.28	1377.1	1.39	11.18	8.03	0.49	1.97
	38.7	350	490.84	1392.6	1381.23	1.40	11.37	8.11	0.49	
20	38.77	350	490.69	1332.3	1323.26	1.40	9.04	6.45	0.39	1.49
	38.77	350	489.05	1338.4	1330.4	1.40	8	5.73	0.35	

Table H.1 (Continued)

Shaft #	beaker tare (g)	beaker volume (mL)	full beaker (g)	Before weight (g)	After weight (g)	putty density (g/mL)	putty used (g)	void volume (cm^3)	void volume (in^3)	Surface Roughness (in^3/sf)
21	38.7	350	484.64	1374.17	1200.68	1.38	173.49	125.29	7.65	26.27
	38.7	350	481.74	1379.9	1256.14	1.38	123.76	89.92	5.49	
22	38.77	350	483.61	1333.62	1324.43	1.38	9.19	6.65	0.41	1.35
	38.77	350	495.75	1345.36	1339.17	1.42	6.19	4.37	0.27	
23	38.77	350	498.89	1348.44	1336.57	1.43	11.87	8.33	0.51	2.19
	38.77	350	498.14	1347.7	1334.07	1.42	13.63	9.58	0.58	
25	38.6	350	510.4	1026.3	955	1.46	71.3	48.89	2.98	7.59
	38.6	350	499	1016.6	997.6	1.43	19	13.33	0.81	
26	38.6	350	510	1011.2	994.8	1.46	16.4	11.25	0.69	2.88
	38.6	350	510.6	1013.2	995.2	1.46	18	12.34	0.75	
27	38.6	350	512	1016	978.1	1.46	37.9	25.91	1.58	6.83
	38.6	350	517.7	1021.9	977.4	1.48	44.5	30.08	1.84	
28	38.6	350	514.3	1030.7	1019.6	1.47	11.1	7.55	0.46	2.36
	38.6	350	502	1021	1004.1	1.43	16.9	11.78	0.72	
29	38.6	350	504.4	1010	984.6	1.44	25.4	17.62	1.08	6.16
	38.6	350	505.9	1007.4	959.9	1.45	47.5	32.86	2.01	
30	38.6	350	507.2	891.6	753.1	1.45	138.5	95.57	5.83	15.51
	38.6	350	498.3	883.4	838.5	1.42	44.9	31.54	1.92	
31	38.6	350	505.4	1024.2	1011.2	1.44	13	9.00	0.55	2.22
	38.6	350	505.9	1025.6	1012.3	1.45	13.3	9.20	0.56	
32	38.6	350	520.6	994.5	983.4	1.49	11.1	7.46	0.46	1.75
	38.6	350	510.7	984.6	974.6	1.46	10	6.85	0.42	
33	38.6	350	512.6	896.6	850.9	1.46	45.7	31.20	1.90	5.54
	38.6	350	507.1	890.9	870.4	1.45	20.5	14.15	0.86	

Table H.1 (Continued)

Shaft #	beaker tare (g)	beaker volume (mL)	full beaker (g)	Before weight (g)	After weight (g)	putty density (g/mL)	putty used (g)	void volume (cm^3)	void volume (in^3)	Surface Roughness (in^3/sf)
34	38.6	350	504.4	887.6	819.7	1.44	67.9	47.12	2.88	7.25
	38.6	350	505.7	890.1	872.3	1.44	17.8	12.32	0.75	
35	38.6	350	519.1	993.8	898.2	1.48	95.6	64.46	3.93	14.90
	38.6	350	519.2	994.2	908.7	1.48	85.5	57.64	3.52	
36	38.6	350	496.1	879.8	851.5	1.42	28.3	19.97	1.22	3.28
	38.6	350	507.2	891.9	881.9	1.45	10	6.90	0.42	
37	38.6	350	518.8	991.4	974.9	1.48	16.5	11.13	0.68	2.29
	38.6	350	515	988	976.8	1.47	11.2	7.61	0.46	
38	38.6	350	518.9	994	949.2	1.48	44.8	30.22	1.84	6.82
	38.6	350	515.4	988.4	950.6	1.47	37.8	25.67	1.57	
39	38.6	350	512.7	897.3	862.5	1.46	34.8	23.76	1.45	5.27
	38.6	350	510.7	899	870.6	1.46	28.4	19.46	1.19	
40	38.6	350	505	891.2	753	1.44	138.2	95.78	5.84	16.35
	38.6	350	512.6	896.6	840.7	1.46	55.9	38.17	2.33	
41	38.6	350	511.1	984.9	963.5	1.46	21.4	14.65	0.89	3.32
	38.6	350	522	995.6	976.9	1.49	18.7	12.54	0.77	
42	38.6	350	514.1	1018.1	983.5	1.47	34.6	23.56	1.44	6.44
	38.6	350	511.5	988.7	946	1.46	42.7	29.22	1.78	
43	38.6	350	514.9	988	943.5	1.47	44.5	30.25	1.85	7.57
	38.6	350	515.7	988.2	941.4	1.47	46.8	31.76	1.94	
45	38.6	350	511.4	894.7	857.7	1.46	37	25.32	1.55	6.67
	38.6	350	509	893.5	850.9	1.45	42.6	29.29	1.79	
46	38.6	350	506.3	889.8	869.4	1.45	20.4	14.10	0.86	3.81
	38.6	350	504.2	888.1	863.5	1.44	24.6	17.08	1.04	

Table H.1 (Continued)

Shaft #	beaker tare (g)	beaker volume (mL)	full beaker (g)	Before weight (g)	After weight (g)	putty density (g/mL)	putty used (g)	void volume (cm^3)	void volume (in^3)	Surface Roughness (in^3/sf)
47	38.6	350	506.3	979.6	962.4	1.45	17.2	11.89	0.73	2.93
	38.6	350	513.8	986.3	968.5	1.47	17.8	12.13	0.74	
48	38.6	350	511.8	985.5	954.2	1.46	31.3	21.40	1.31	5.86
	38.6	350	503.1	977.7	939.5	1.44	38.2	26.58	1.62	
49	38.6	350	514.4	987.2	950.3	1.47	36.9	25.11	1.53	4.63
	38.6	350	516.3	988.8	969.9	1.48	18.9	12.81	0.78	
50	38.6	350	509.5	894.8	879.3	1.46	15.5	10.65	0.65	2.49
	38.6	350	513.7	899	884.7	1.47	14.3	9.74	0.59	
51	38.6	350	514.5	898.8	888.4	1.47	10.4	7.07	0.43	1.60
	38.6	350	511.3	895.5	886.7	1.46	8.8	6.02	0.37	
52	38.6	350	517.4	989.9	963.5	1.48	26.4	17.86	1.09	4.07
	38.6	350	518.5	992.4	969.5	1.48	22.9	15.46	0.94	
53	38.63	350	508.2	1021.3	994.6	1.45	26.7	18.39	1.12	4.81
	38.63	350	512.6	890.2	859.4	1.46	30.8	21.03	1.28	
54	38.63	350	517.7	985.6	965.5	1.48	20.1	13.59	0.83	3.92
	38.63	350	505.2	913.4	886.7	1.44	26.7	18.50	1.13	
55	38.63	350	513.6	938.2	915.4	1.47	22.8	15.54	0.95	3.38
	38.63	350	501.3	1018.7	1001.3	1.43	17.4	12.15	0.74	
56	38.63	350	511.7	1006.1	981.4	1.46	24.7	16.89	1.03	3.94
	38.63	350	502.6	975.9	953.8	1.44	22.1	15.39	0.94	
57	38.63	350	505.9	985.8	942.2	1.45	43.6	30.16	1.84	6.23
	38.63	350	516.3	972.6	941.8	1.48	30.8	20.88	1.27	
58	38.63	350	499.8	984.6	961.0	1.43	23.6	16.53	1.01	3.65
	38.63	350	516.6	963.7	944.0	1.48	19.7	13.35	0.81	

APPENDIX I – DIGITAL SURFACE ROUGHNESS RENDERINGS

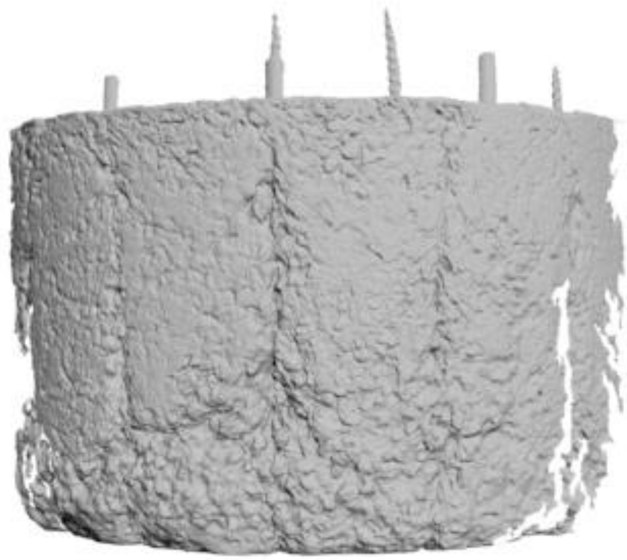


Figure I.1 Shaft 1 digital rendering

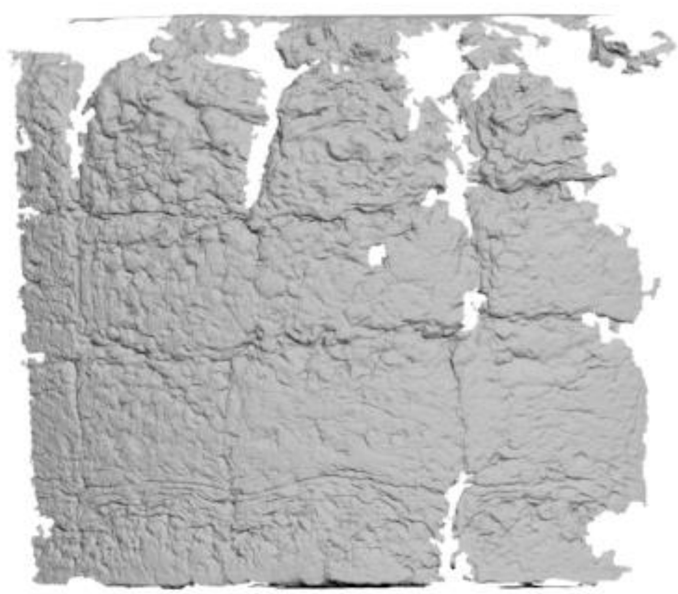


Figure I.2 Shaft 2 digital rendering

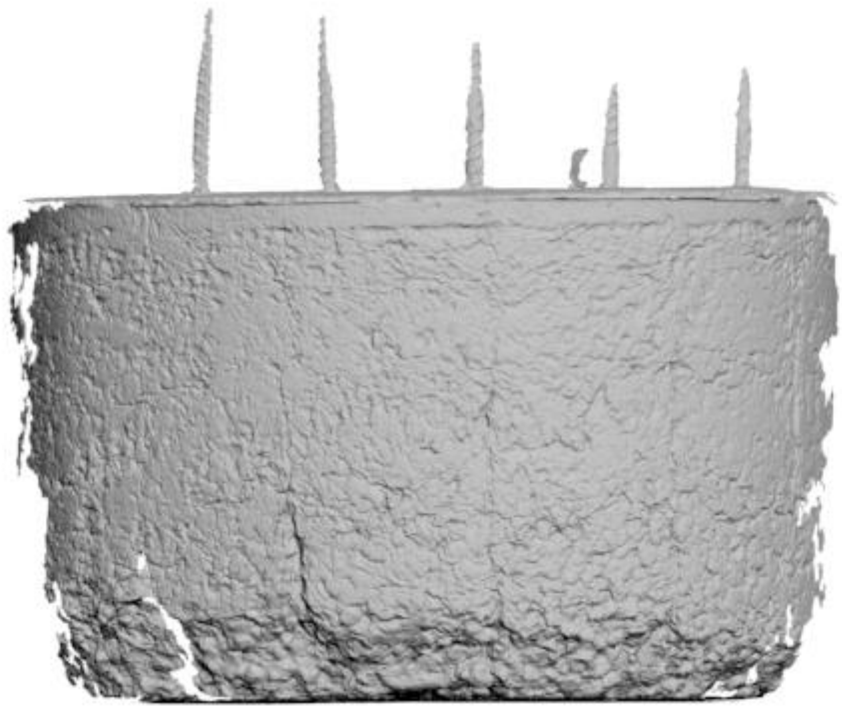


Figure I.3 Shaft 3 digital rendering

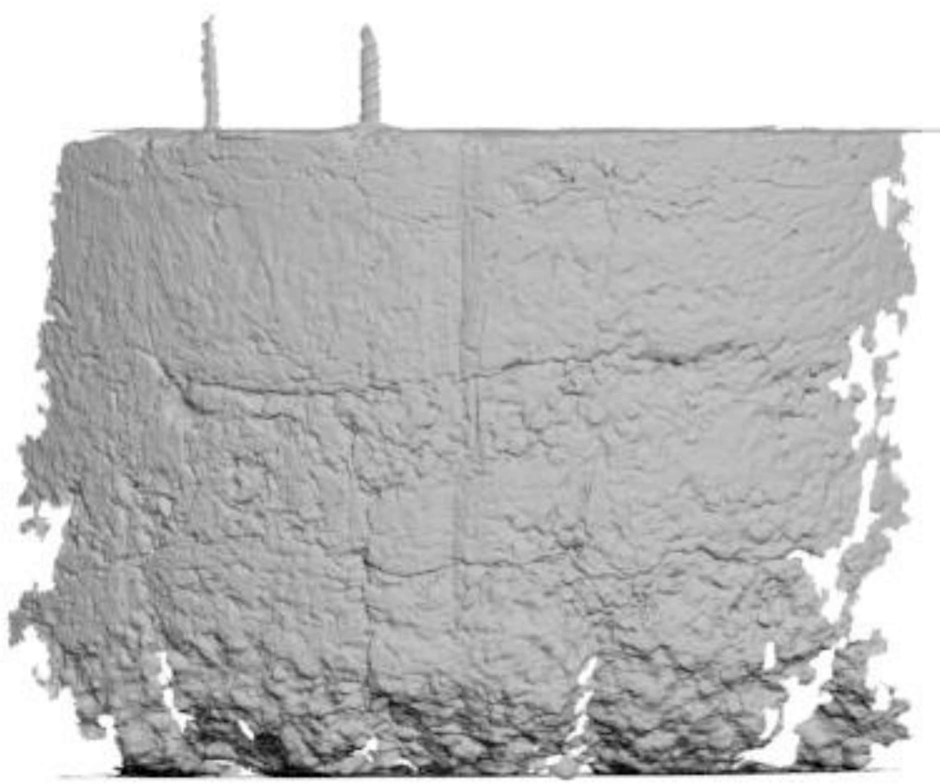


Figure I.4 Shaft 4 digital rendering

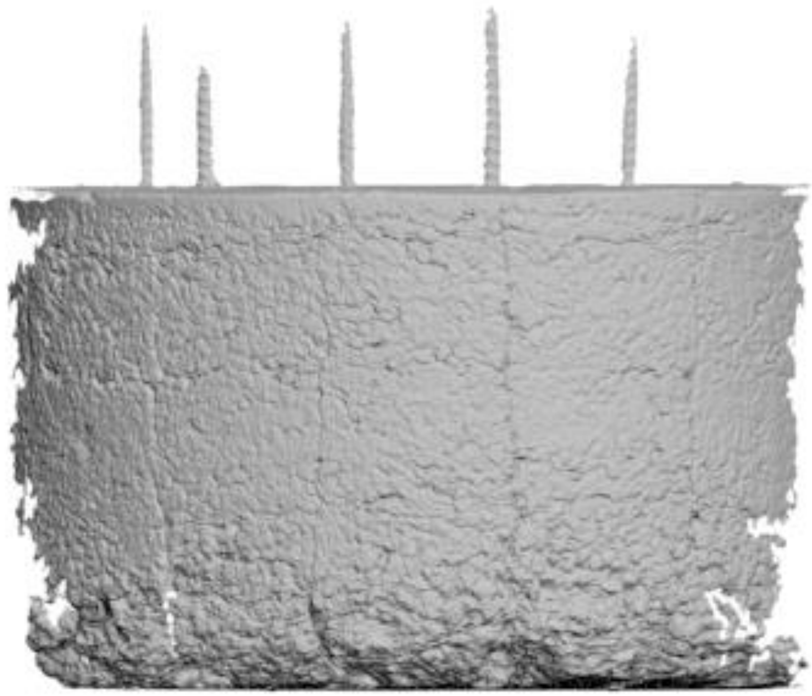


Figure I.5 Shaft 5 digital rendering

Figure I.6 Shaft 6 digital rendering

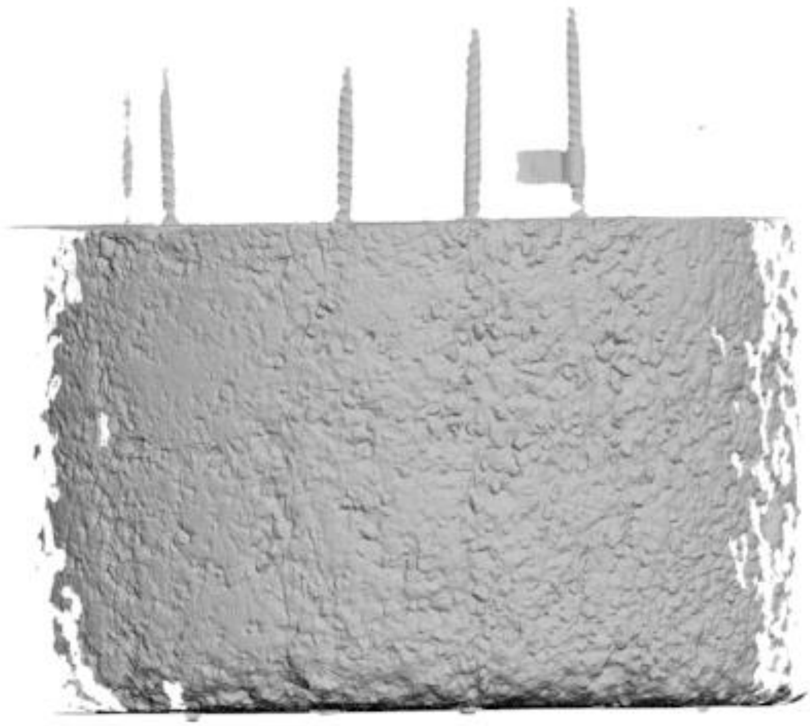


Figure I.7 Shaft 7 digital rendering

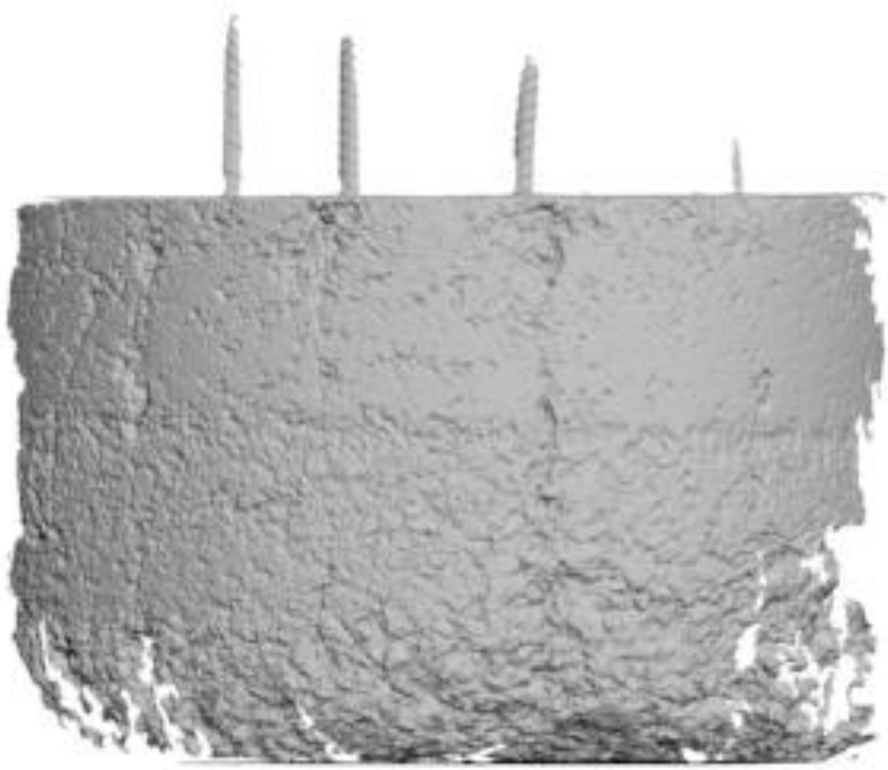


Figure I.8 Shaft 8 digital rendering

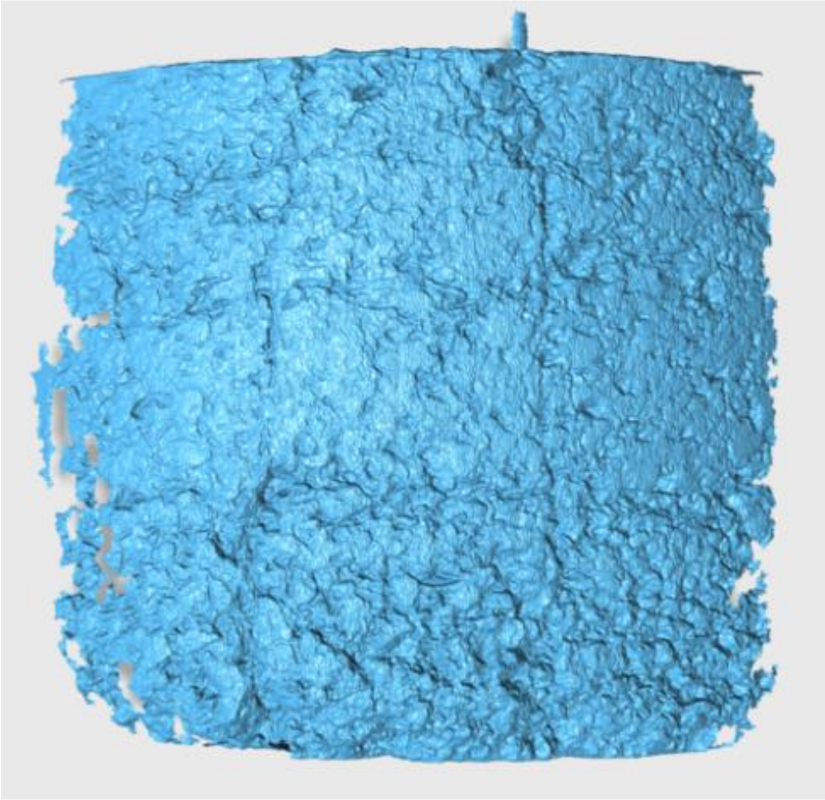


Figure I.9 Shaft 9 digital rendering

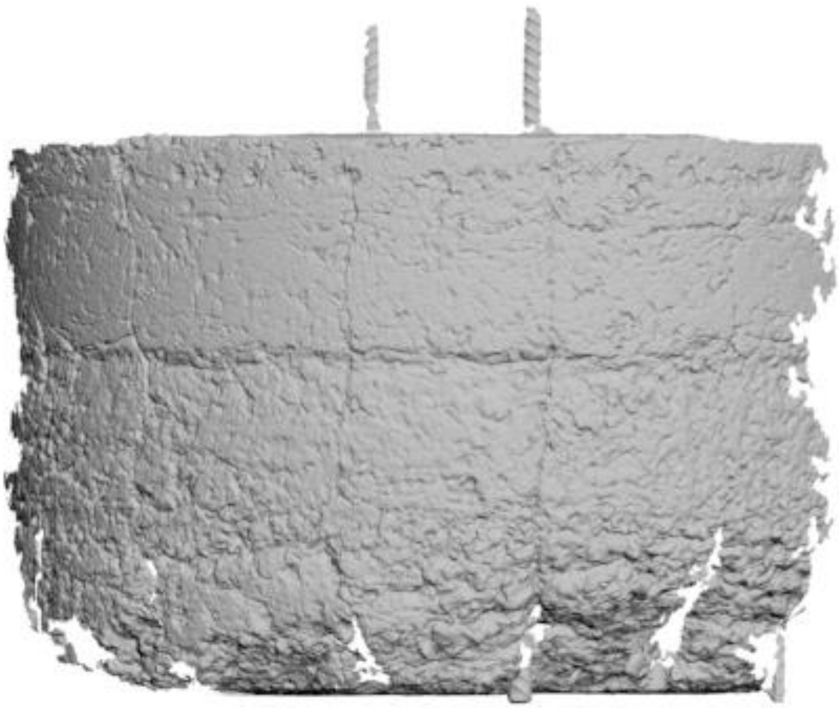


Figure I.10 Shaft 10 digital rendering

Figure I.11 Shaft 10 digital rendering

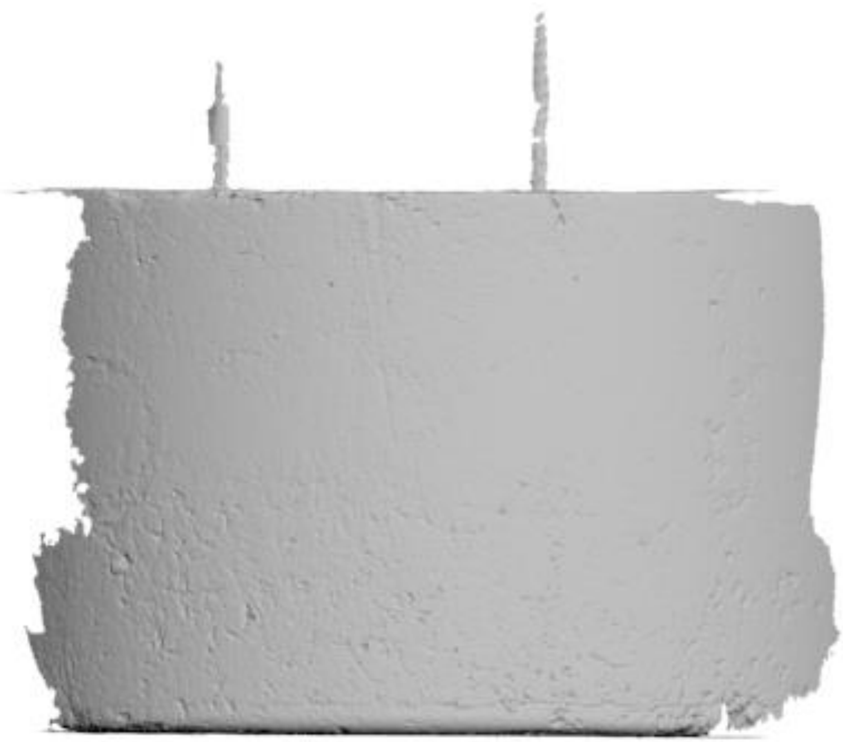


Figure I.12 Shaft 12 digital rendering

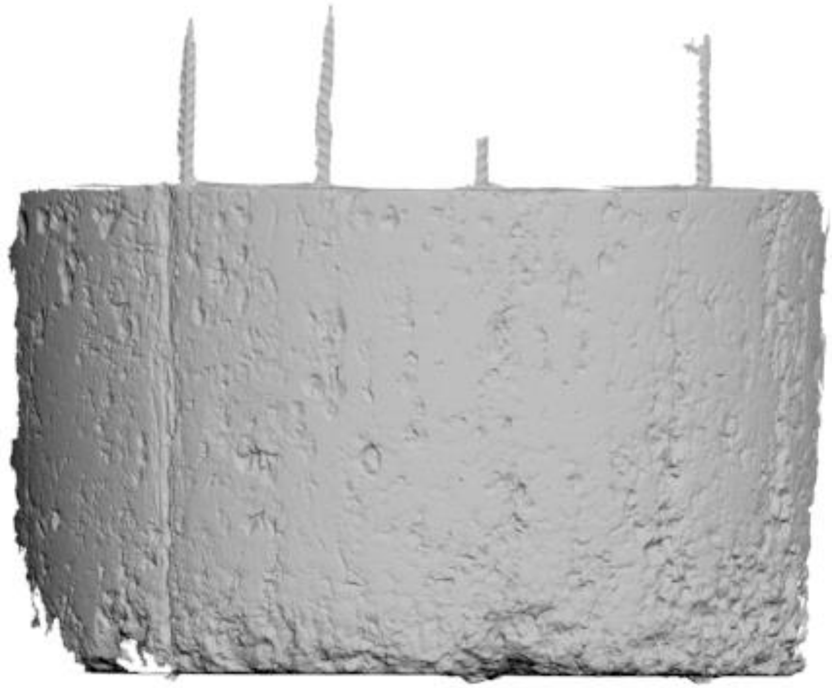


Figure I.13 Shaft 13 digital rendering

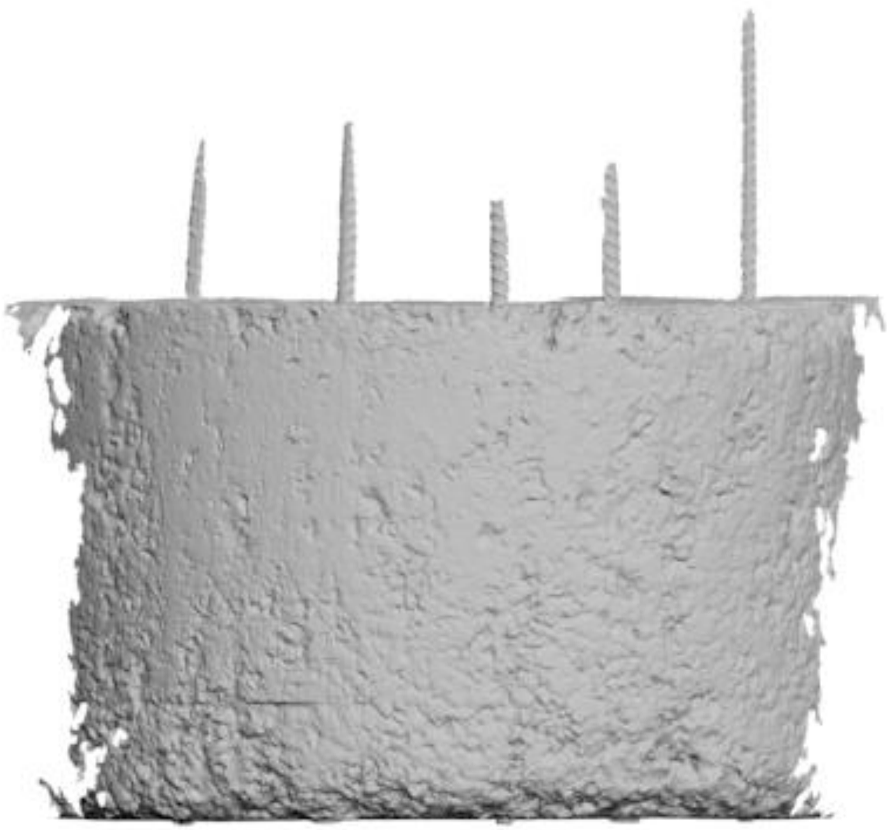


Figure I.14 Shaft 14 digital rendering

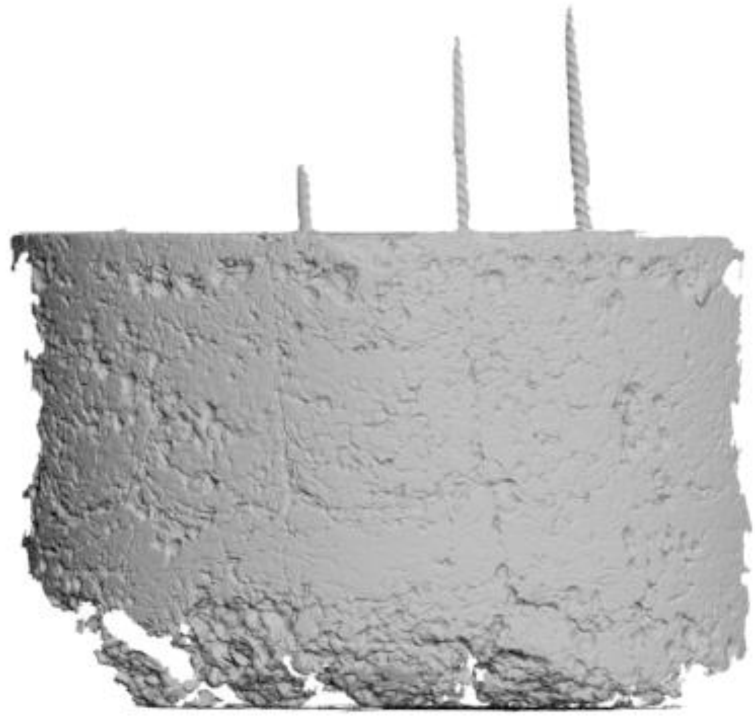


Figure I.15 Shaft 15 digital rendering

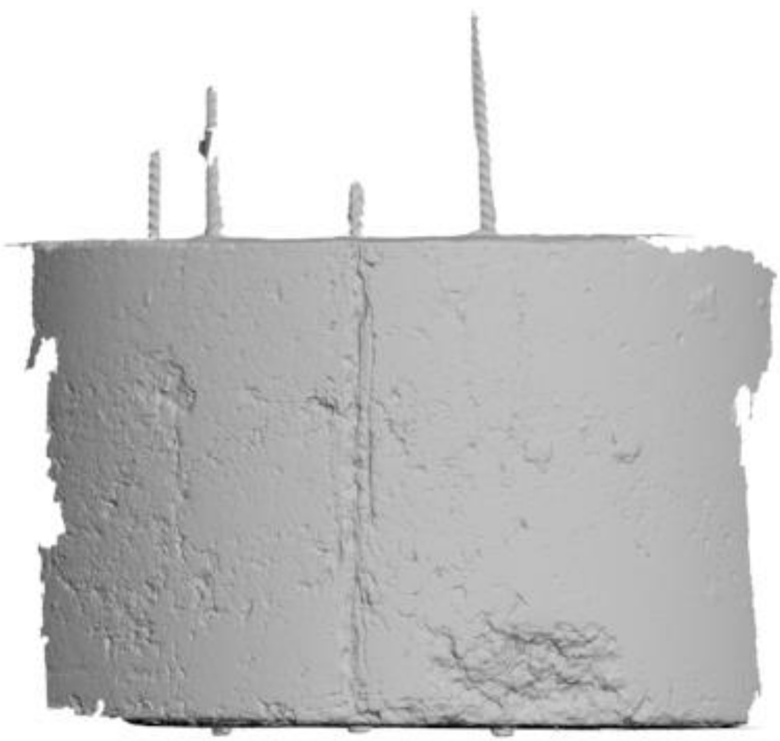


Figure I.16 Shaft 16 digital rendering

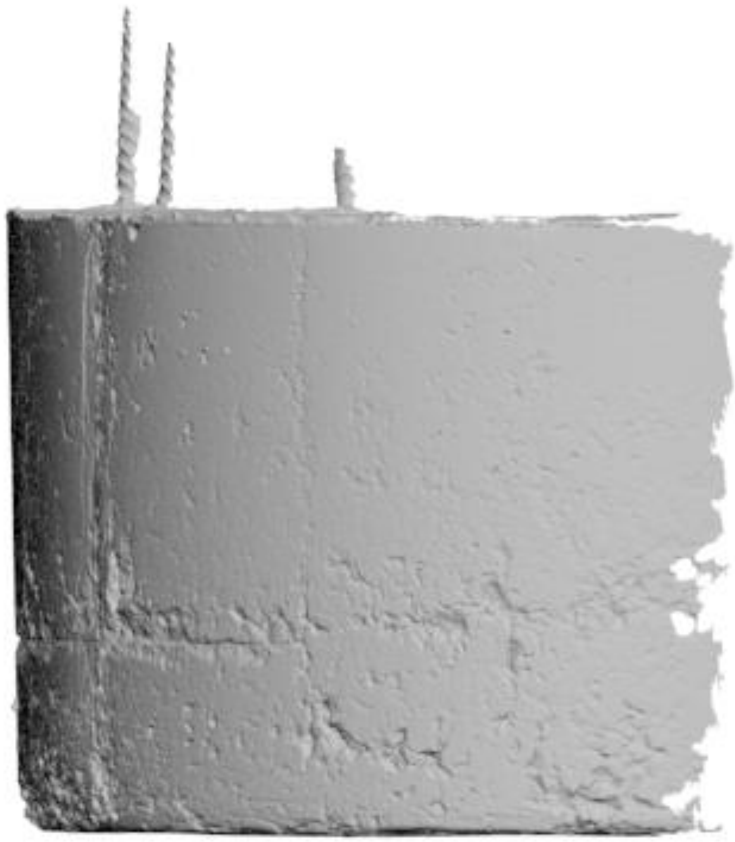


Figure I.17 Shaft 17 digital rendering

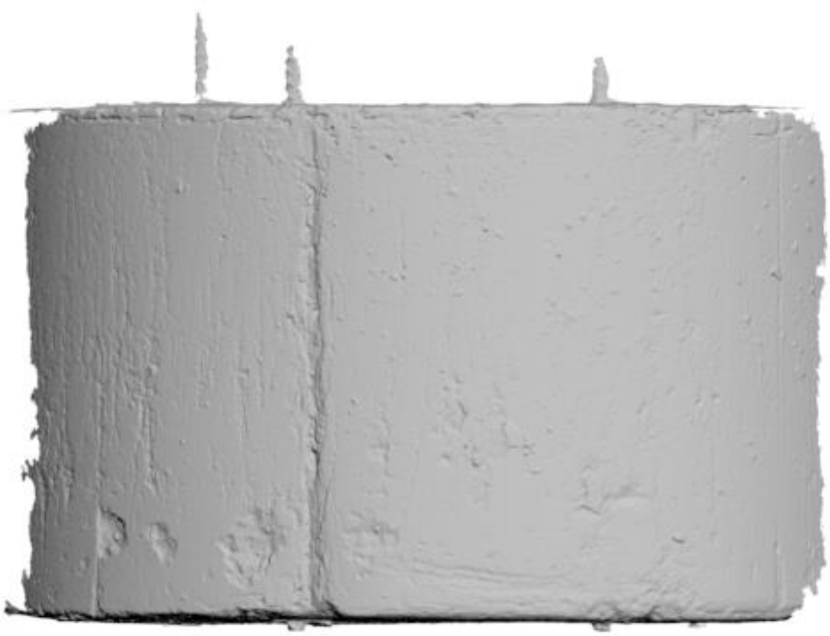


Figure I.18 Shaft 18 digital rendering

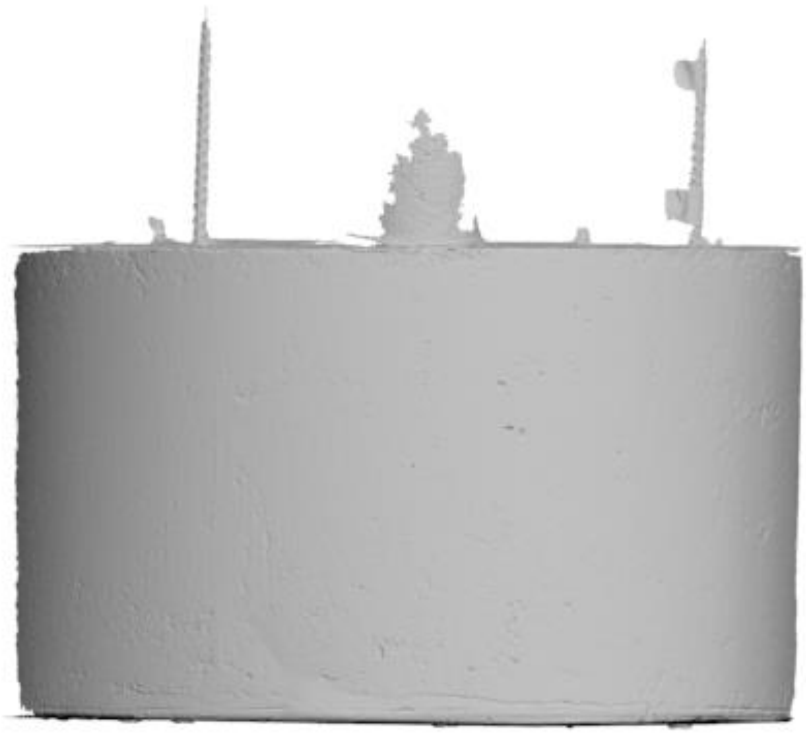


Figure I.19 Shaft 19 digital rendering

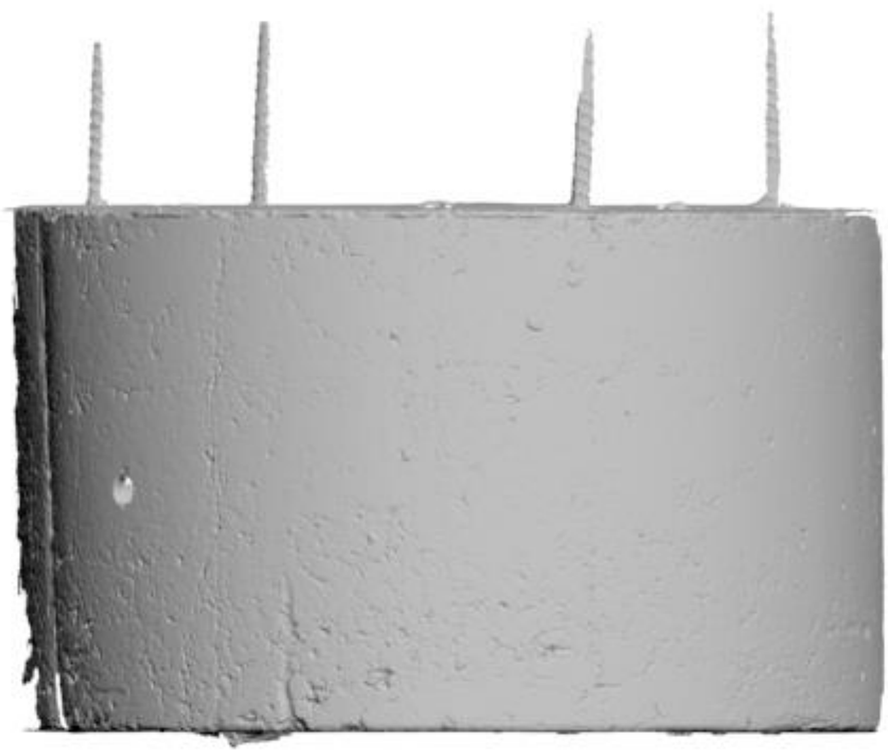


Figure I.20 Shaft 20 digital rendering

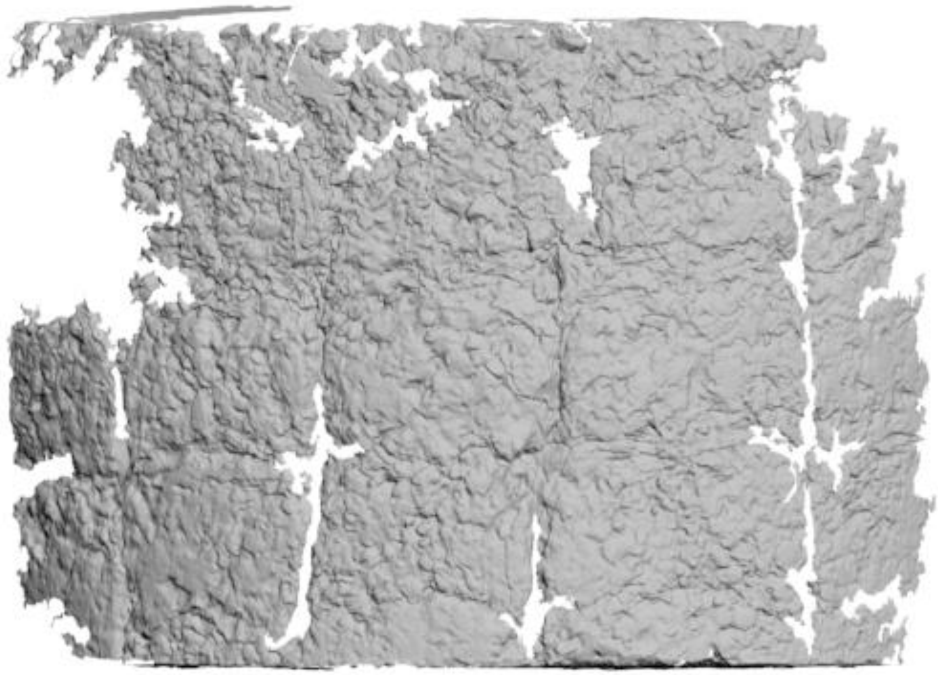


Figure I.21 Shaft 21 digital rendering

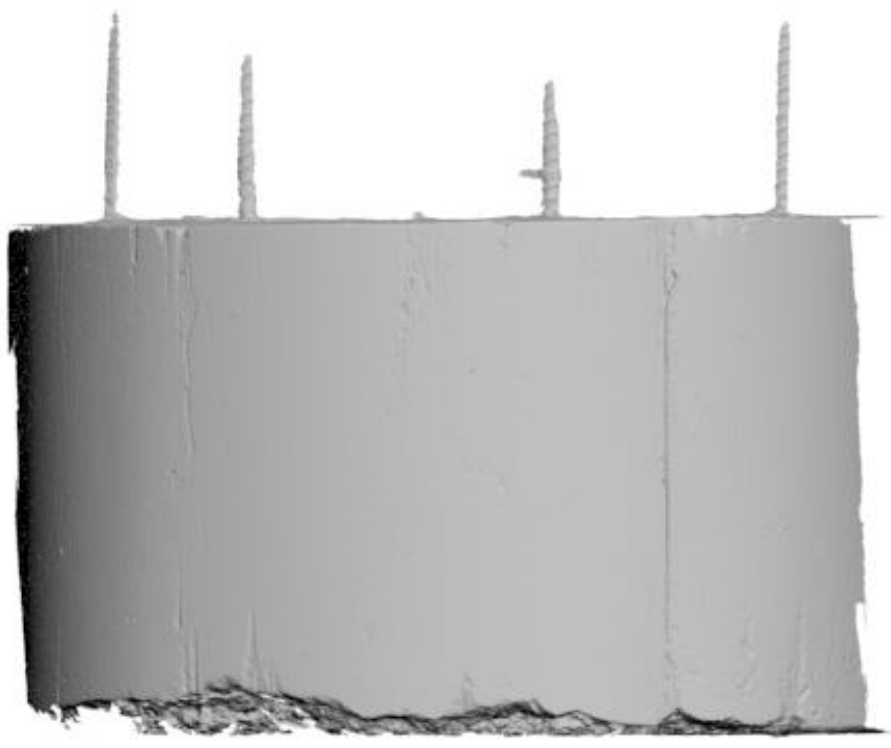


Figure I.22 Shaft 22 digital rendering

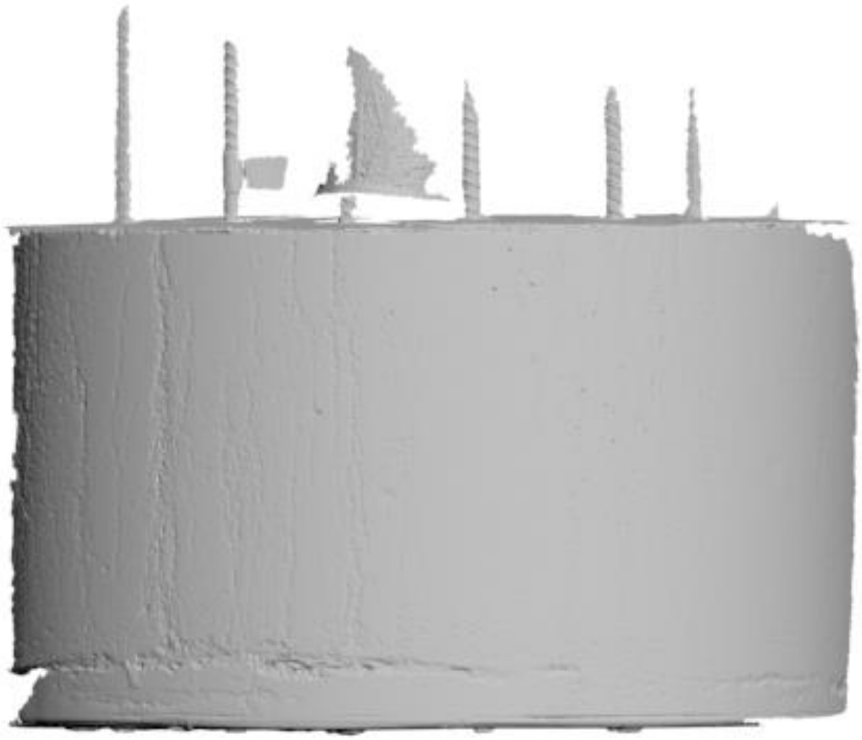


Figure I.23 Shaft 23 digital rendering

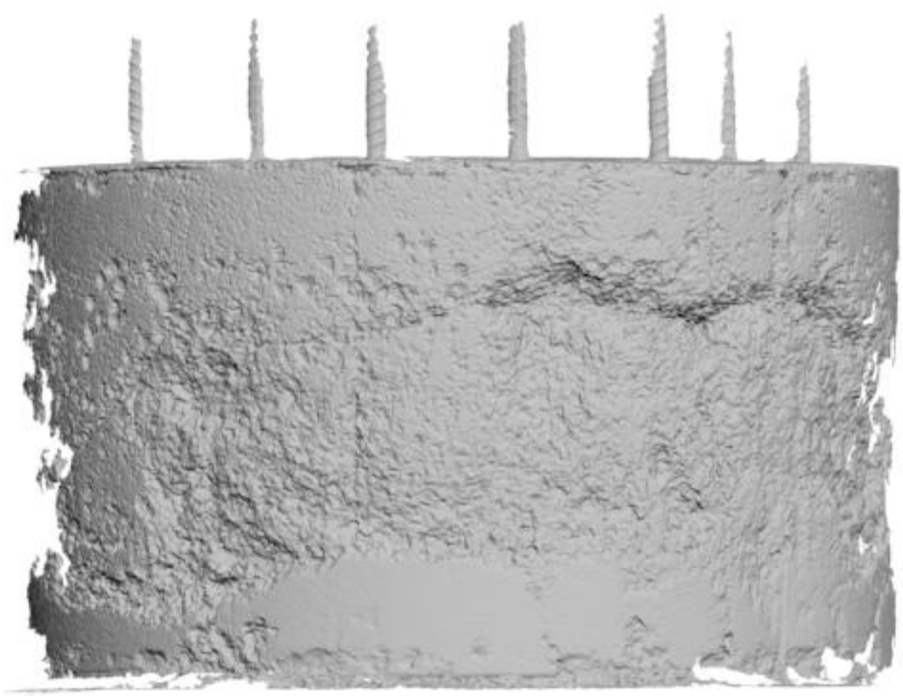


Figure I.24 Shaft 24 digital rendering

APPENDIX J – DIGITAL SURFACE ROUGHNESS DETERMINATION

J.1 Data Decimation Process

The shaft scans started as .obj files; however, their large size requires decimation in order to make them workable in AutoCAD. Originally, the data files have up to 30 million vertices and the desired number is below 2.5 million.

The process to decimation the data is as follows.

1. Open Meshlab and import the .obj file. (Note the orientation of the working face)
2. Navigate to Filters – Remeshing, Simplification and Reconstruction – Quadric Edge Collapse Decimation
3. In the Quadric Edge Collapse Decimation Tab, set the “Quality Threshold” and the “Boundary Preserving Weight” equal to 1. Then check the “Preserve Normal”, “Preserve Topology”, “Optimal Position of Simplified Vertices”, “Planar Simplification”, “Weighted Simplification”, and “Post-Simplification Cleaning” boxes. Apply the decimation settings and wait for the data to be decimated.
4. Repeat steps two and three until the number of vertices are below five hundred thousand.
5. Export the mesh as a .xyz file.

J.2 Surface Roughness Determination Process

Autodesk Civil 3D was used for determining surface roughness for the shaft specimens detailed in this research, but this process could be used for any scan data. Start by opening a new drawing file under the Void Volume Template (void volume.dwt) and then by creating a surface (Figure J.1). Name the surface to correspond to the shaft name or any other identifiers (Figure J.2).

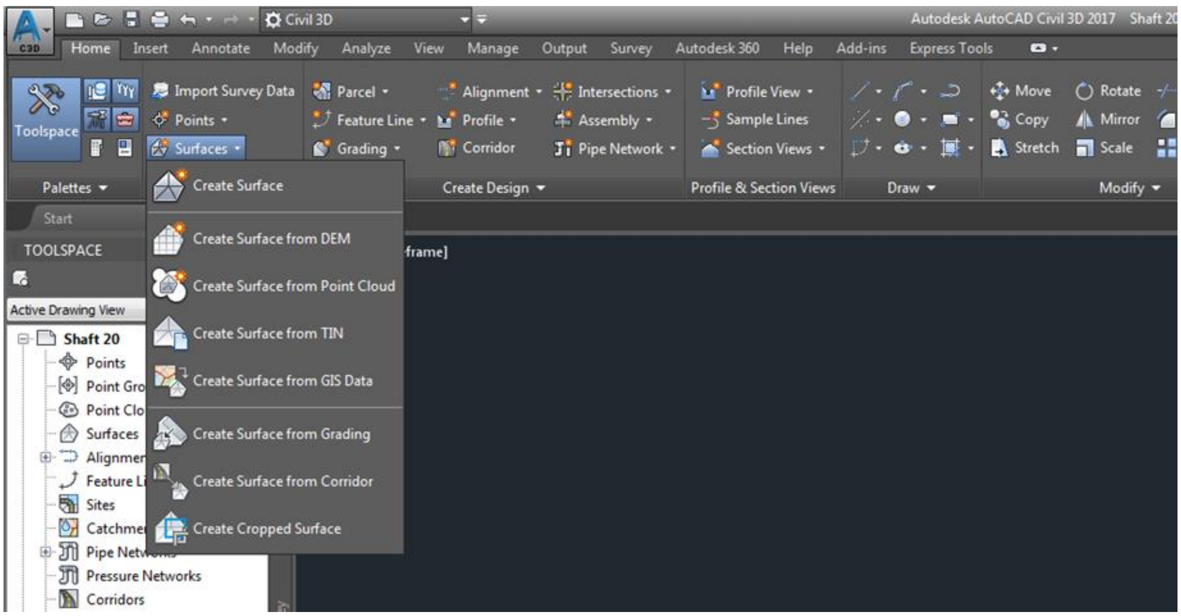


Figure J.1 Create a surface

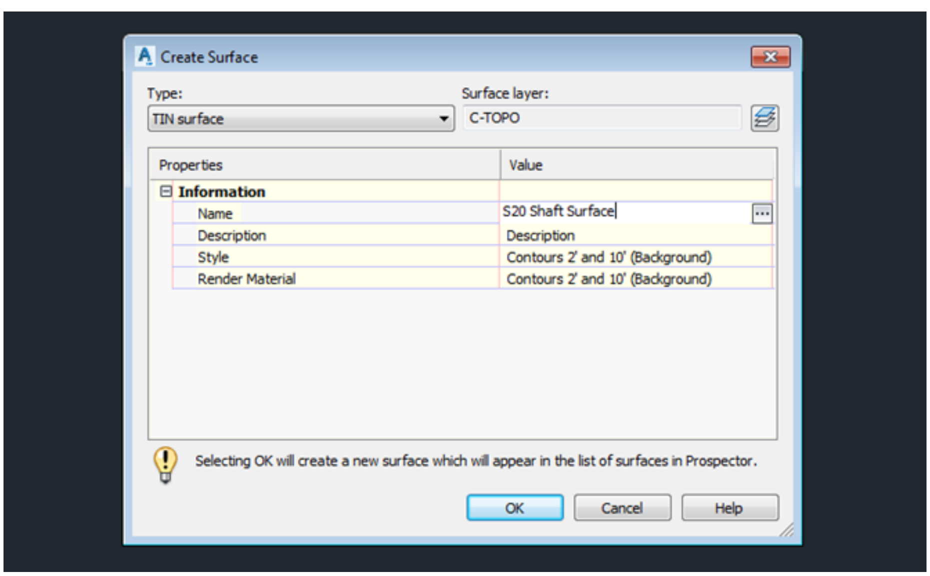


Figure J.2 Name the surface

To define the surface that was just created, add a .xyz file as a point file. Expand the "Surfaces", "Your Surface Name" and "Definition" tabs to find the "Point Files" option. Right click and add a point file (Figure J.3).

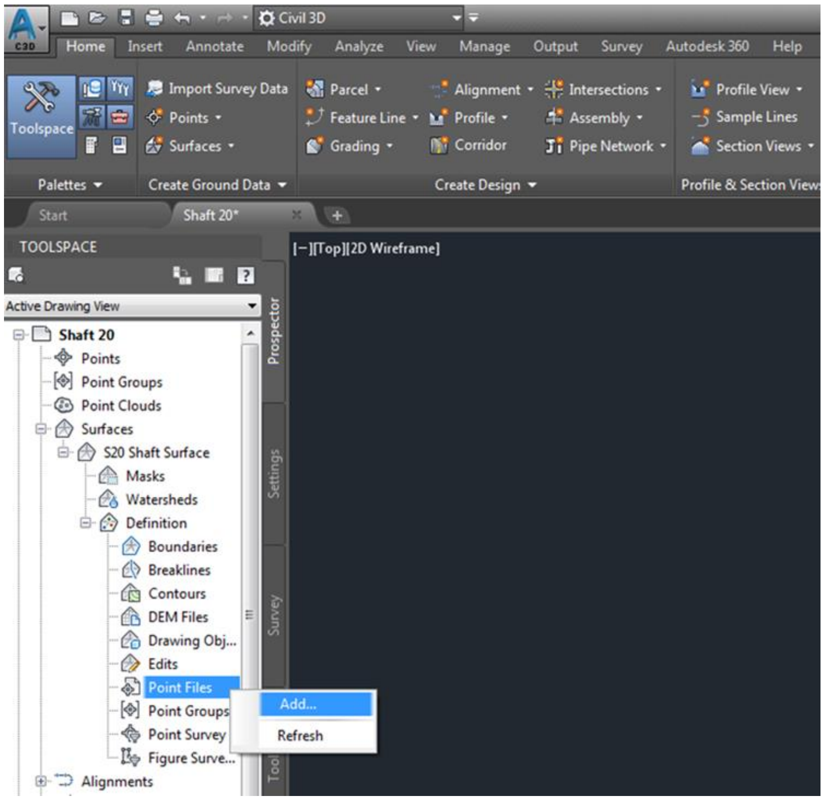


Figure J.3 Add a point file to your surface definition

When prompted to choose a file, click on the plus on the right side of the window and then navigate to the directory your file is in. Choose the file and hit "Open" and the "Ok" (Figure J.4). It is important to make sure the working face of your surface is in the XY plane. You should have noted the orientation of the working face during your data decimation. If the working face is already in the XY plane, then you can upload your data file as a PNEZ – the NEZ stands for northing, easting and elevation and this file designation indicates that autoCAD will read your xyz file as x is northing, y is easting and z is elevation. If you need to rotate your surface all you have to do is change the designation. You do this by clicking the plus sign by the file format (Figure J.4) and then creating a new file format (Figure J.5).

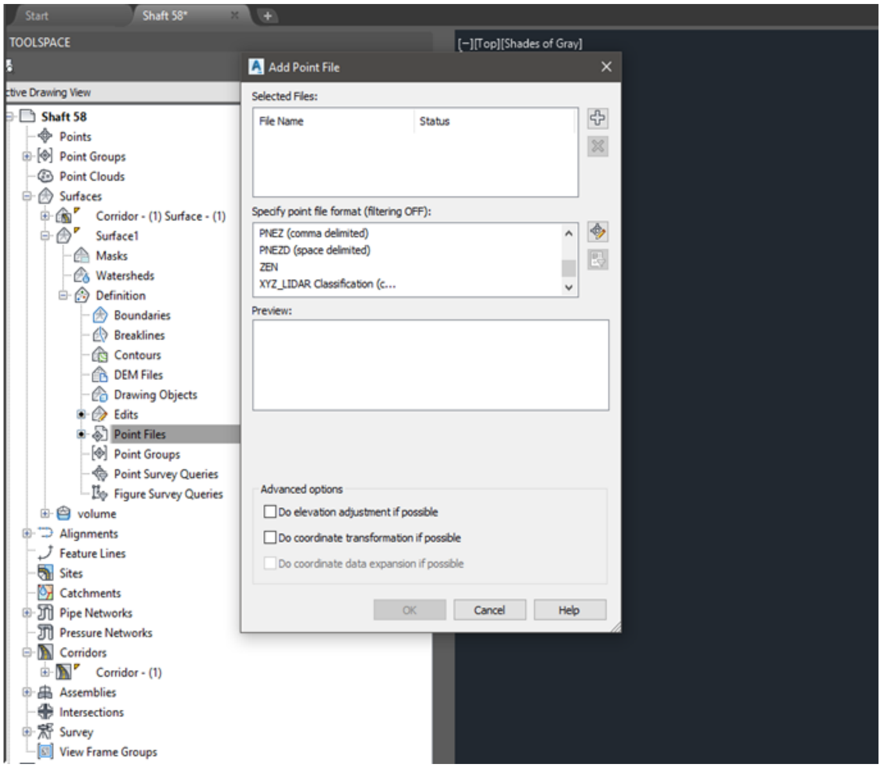


Figure J.4 Adding a file designation

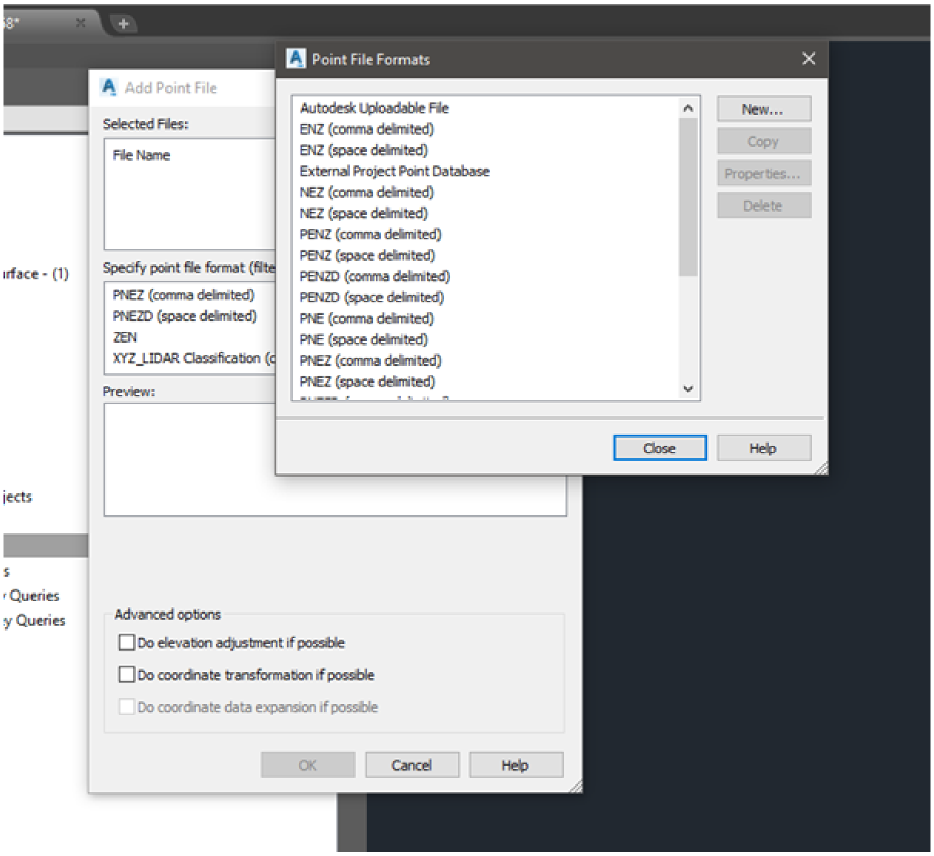


Figure J.5 Creating a new format

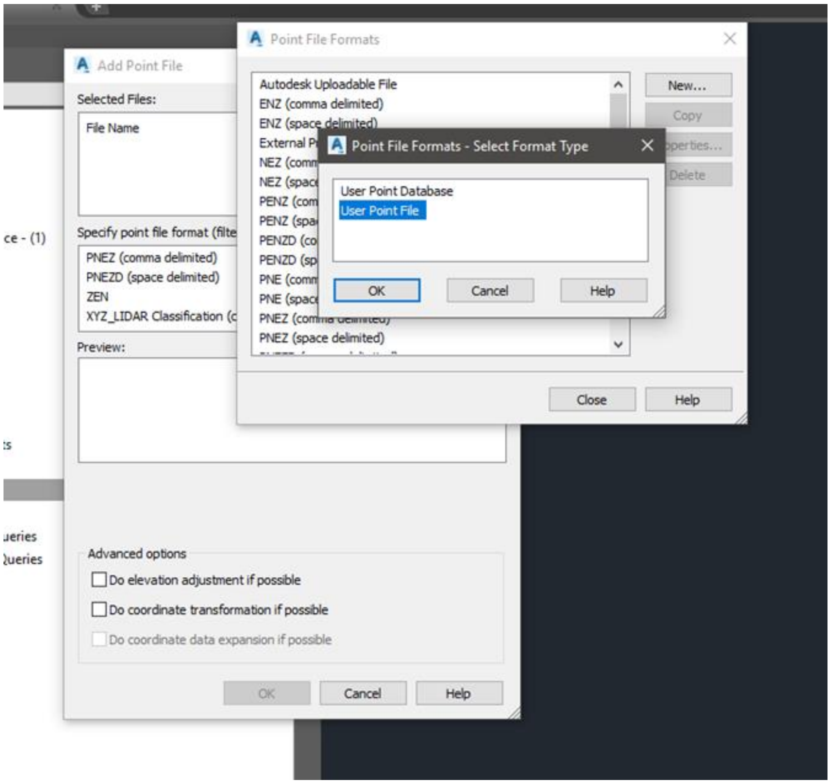


Figure J.6 User point file

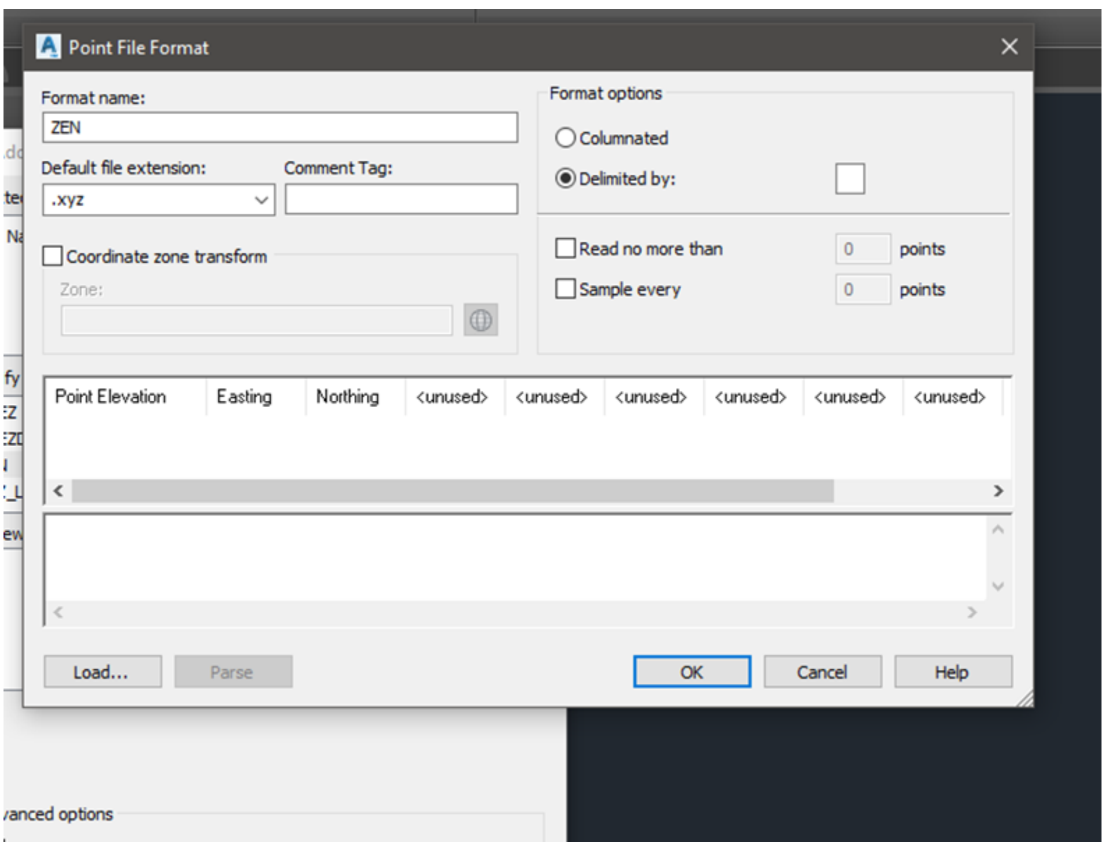
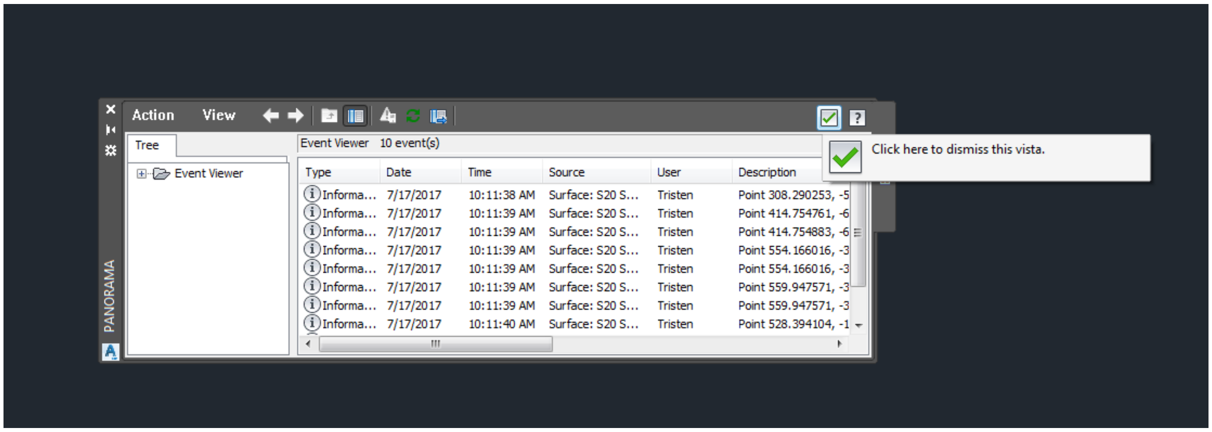


Figure J.7 Point file format

When the point file is added, an “Event Viewer” will display (Figure J.8). Click the green check in the upper right corner.



Type	Date	Time	Source	User	Description
Informa...	7/17/2017	10:11:38 AM	Surface: S20 S...	Tristen	Point 308.290253, -5
Informa...	7/17/2017	10:11:39 AM	Surface: S20 S...	Tristen	Point 414.754761, -6
Informa...	7/17/2017	10:11:39 AM	Surface: S20 S...	Tristen	Point 414.754883, -6
Informa...	7/17/2017	10:11:39 AM	Surface: S20 S...	Tristen	Point 554.166016, -3
Informa...	7/17/2017	10:11:39 AM	Surface: S20 S...	Tristen	Point 554.166016, -3
Informa...	7/17/2017	10:11:39 AM	Surface: S20 S...	Tristen	Point 559.947571, -3
Informa...	7/17/2017	10:11:39 AM	Surface: S20 S...	Tristen	Point 559.947571, -3
Informa...	7/17/2017	10:11:40 AM	Surface: S20 S...	Tristen	Point 528.394104, -1

Figure J.8 Event viewer

Once the shaft renders, draw a line from the top of the shaft to the bottom of the shaft. When the points are imported the scale will most likely be off. The shaft shown has a height of 625 inches (Figure J.9).

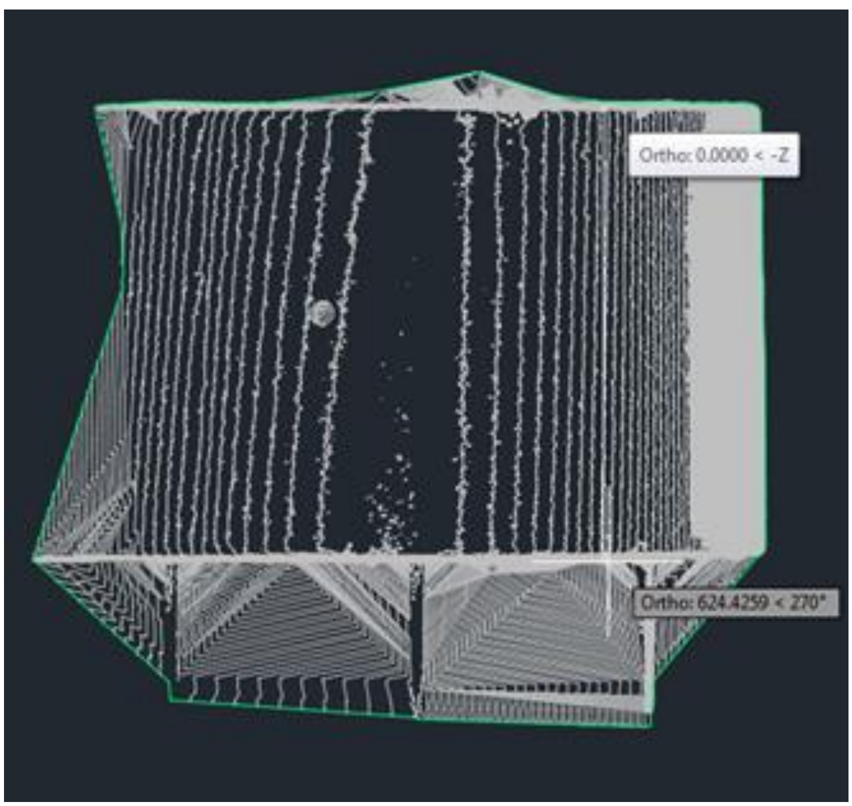


Figure J.9 Surface height check

Use the scale command by typing in SCALE. Choose a base point and enter a scale factor, in this case it was 24/625. After scaling the surface, draw another line from the top of the shaft to the bottom to verify that it was scaled correctly. Next, draw two polylines, using the PLINE command (Figure J.10), creating horizontal and vertical lines through the shaft.

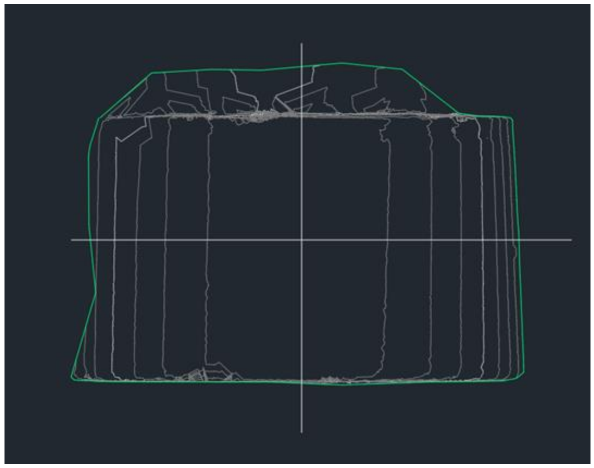


Figure J.10 Draw a horizontal and vertical line through the shaft.

Under the Alignment tab, choose "Create Alignment from Objects" (Figure J.11). After choosing the alignment tool, click on one of the polylines and an arrow will appear on the line. Pay attention to which way the arrow is facing. The alignment needs to be read from left to right or the top to the bottom and therefore the arrow needs to point in the correct direction. If it does not, enter an R into the command line to reverse it. After enter is pressed, a screen with alignment

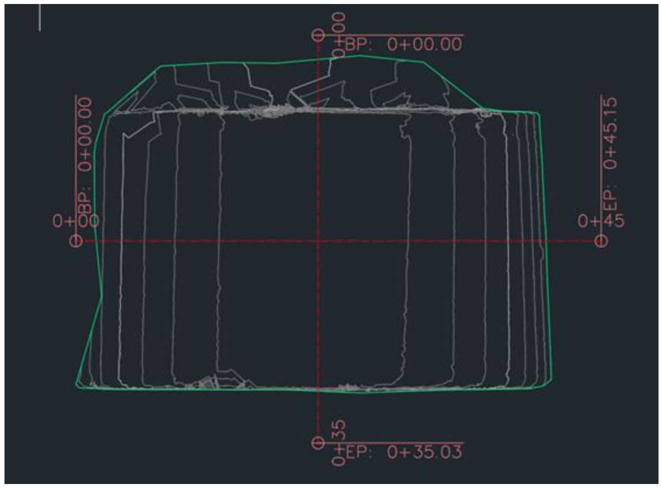


Figure J.11 Turn those lines into alignments.

options will pop up (Figure J.12). Change the name to identify the alignment on a certain shaft and then press ok to create the alignment.

The next step is to create a surface profile. Go to the profile button on the home tab and select Create Surface Profile. (Figure J.12). When the surface profile window appears, select the alignement from the drop down. For this profile we will use “Shaft 20 Upper Alignment”. Next click the add buttom midway down from the right hand side of the window (Figure J.13).

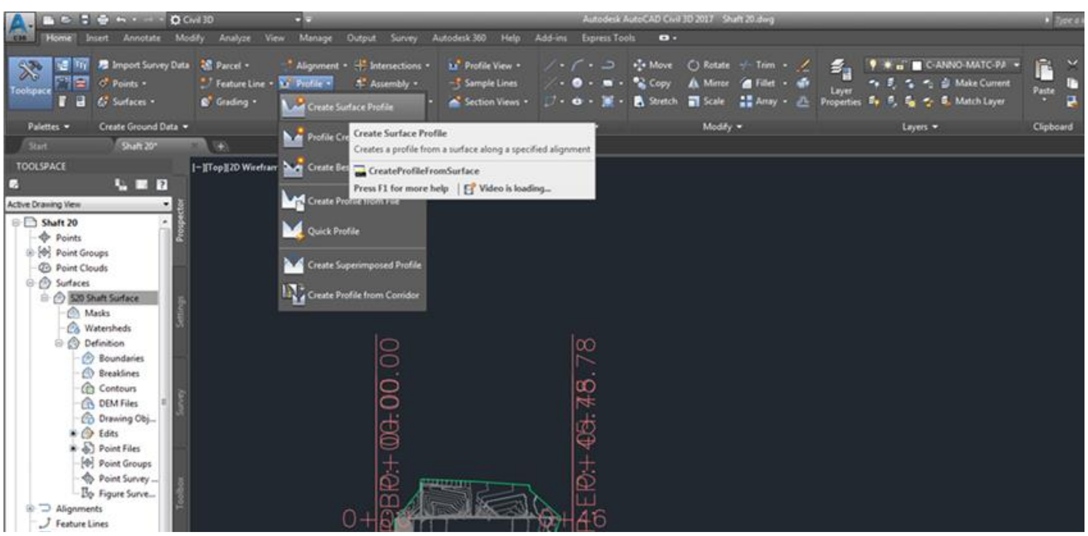


Figure J.12 Create surface profile

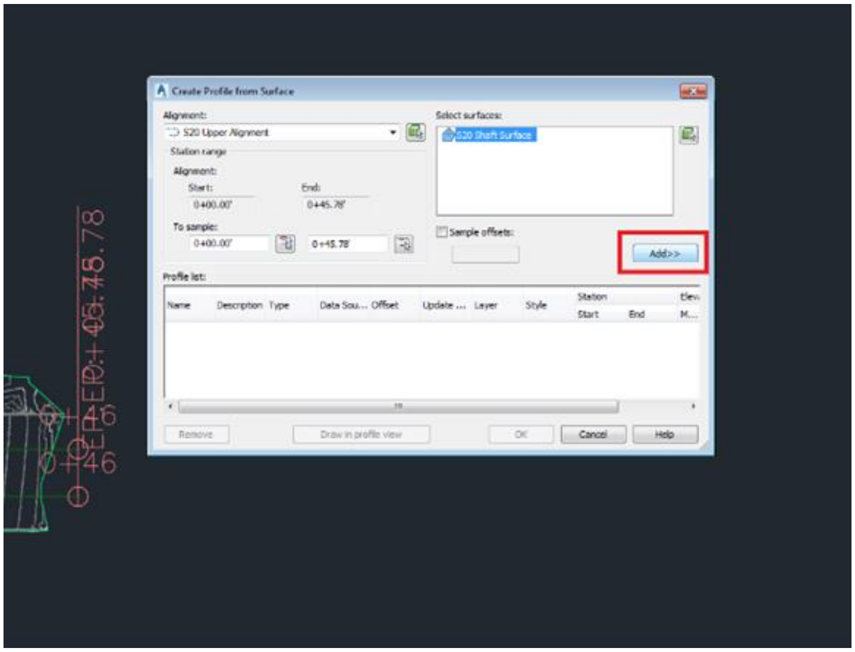


Figure J.13 Profile selection

After the surface profile has been added to the profile list, select the draw in profile view button on the bottom of the window (Figure J.14). Change the name of the profile and then press apply (Figure J.15).

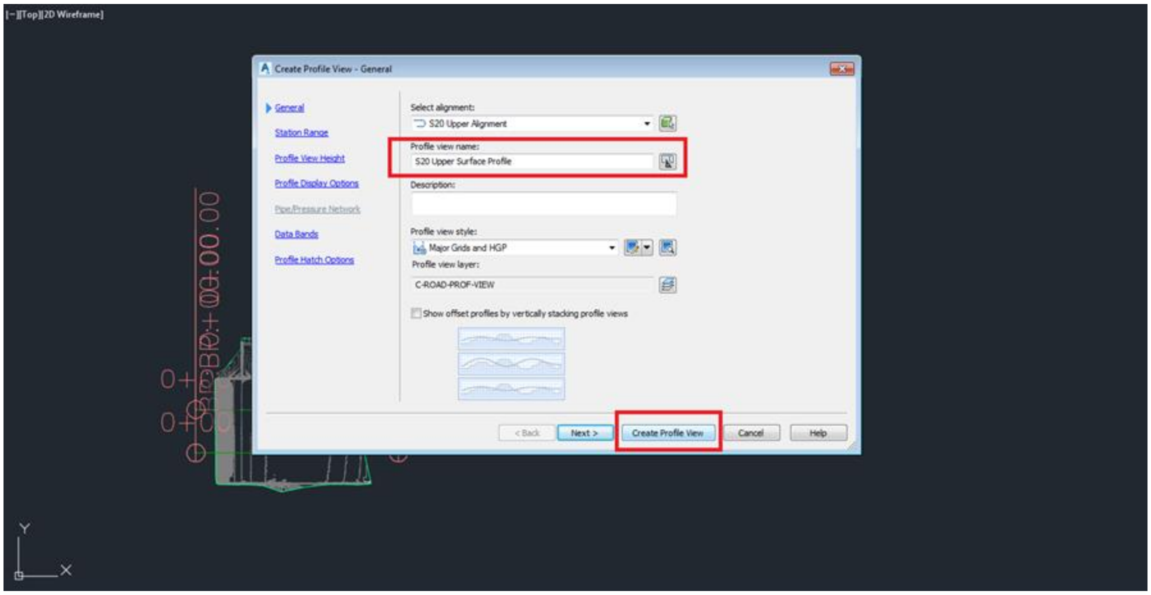


Figure J.14 Draw in profile view

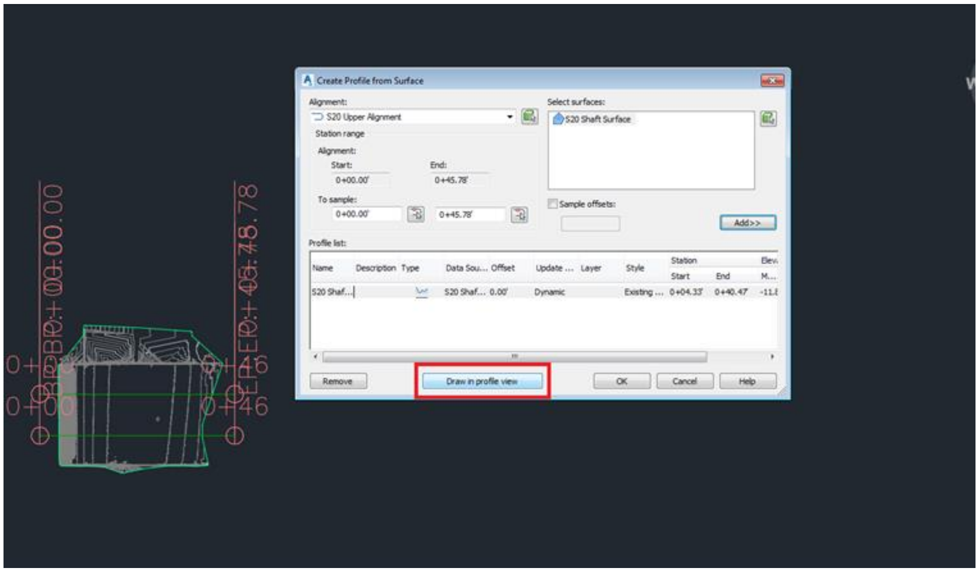


Figure J.15 Change profile name

You may need to change the profile scale. You can do so by right clicking on the profile and selecting edit profile style (Figure J.16). The correct profile should show a curved surface without any vertical exaggeration (Figure J.17).

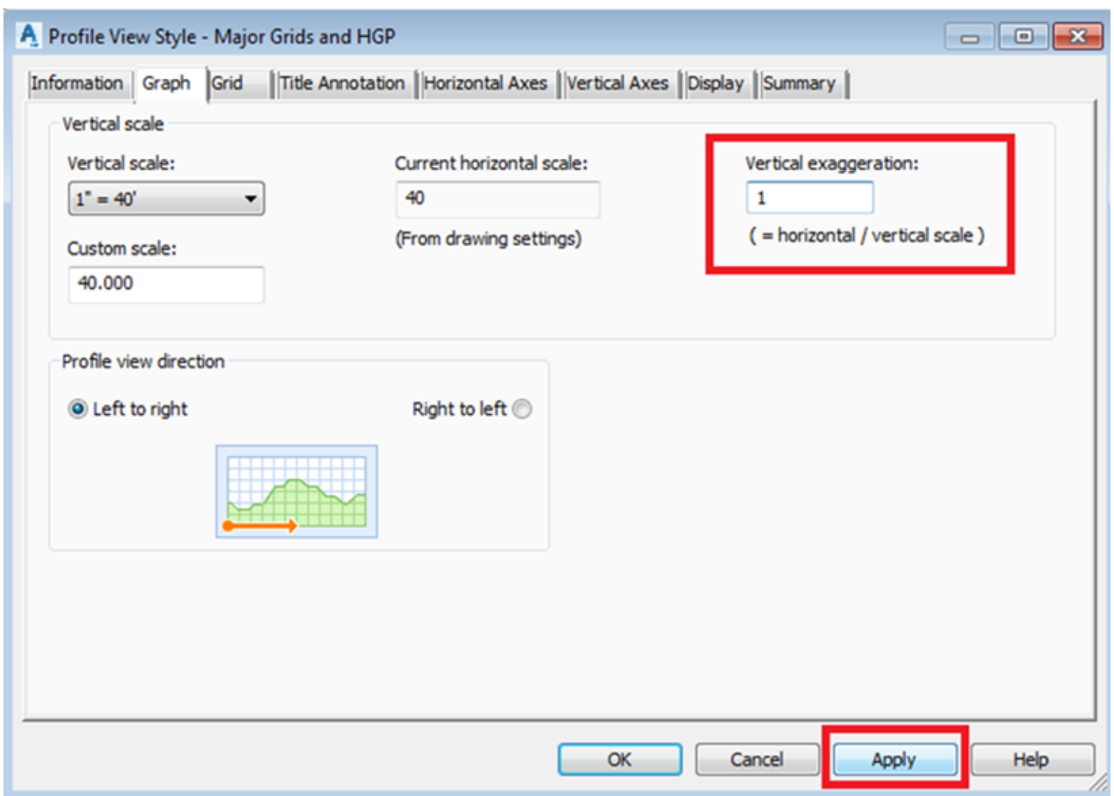


Figure J.16 Edit profile view scale

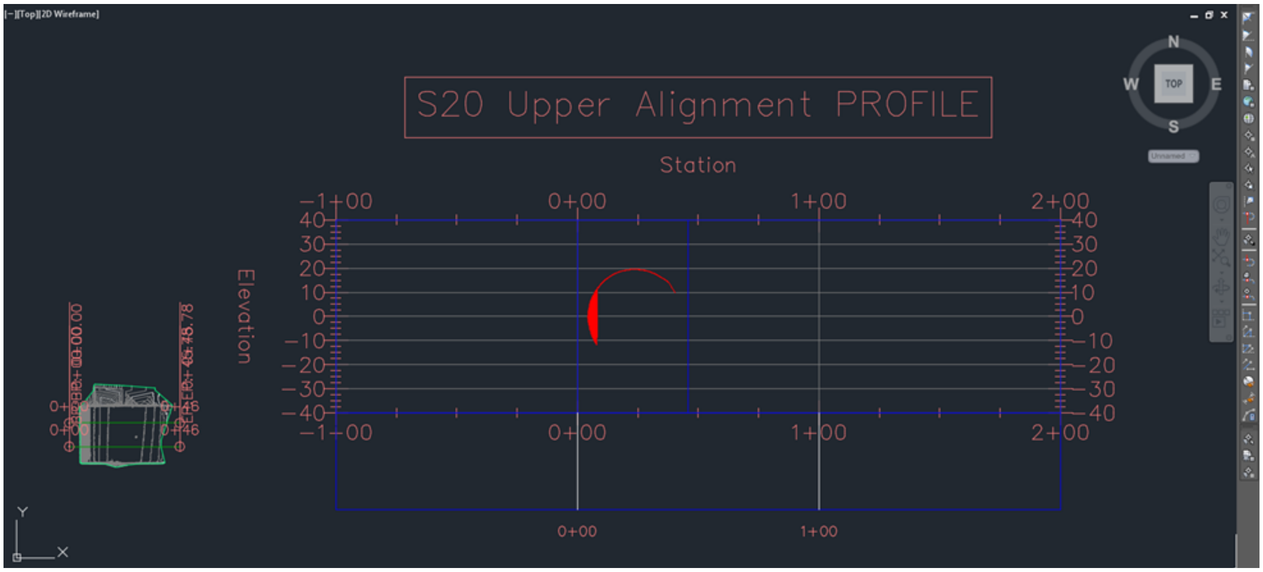


Figure J.17 Correct profile view

Now that you have a surface profile you need to create the perfect surface for comparison. You do this by clicking on the profile creation tools (Figure J.18). You will be prompted to select the profile that you are working with- select the surface profile that you have just created.

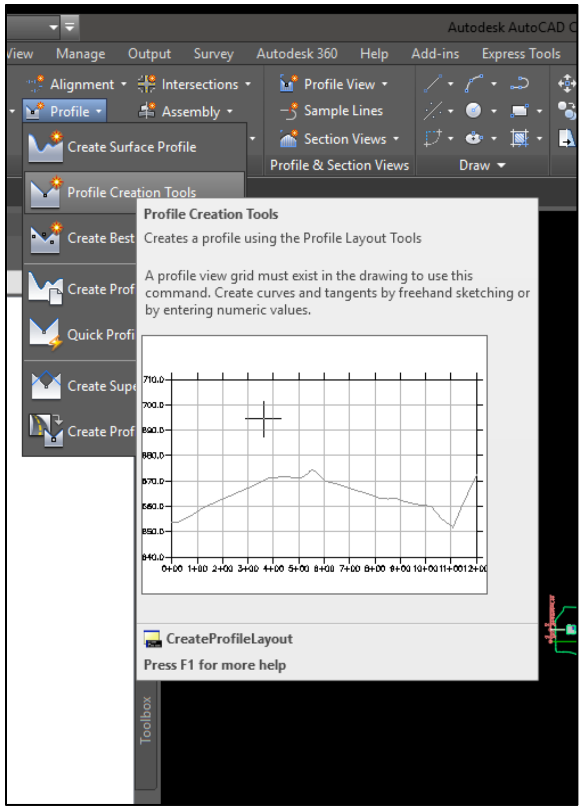


Figure J.18 Profile creation tools

Next you need to draw a curve using the three point curve tool in the profile creation tool bar (Figure J.19). Start by choosing three points along the curve of your surface. This will not be a perfect fit on the first try. You will need to adjust the vertices until you get a curve that fits just above the surface , touching but not cutting into the highest points in your subject range (Figure J.20). This does not have to fit the length of your profile, only a 12 inch section.

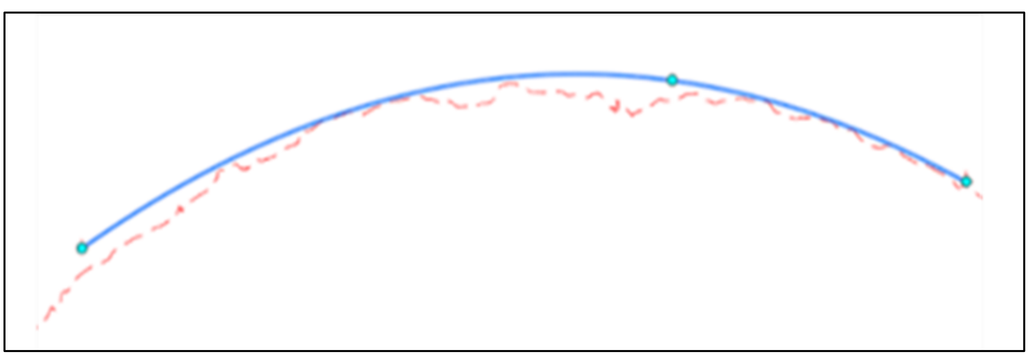


Figure J.19 Three-point curve fit.

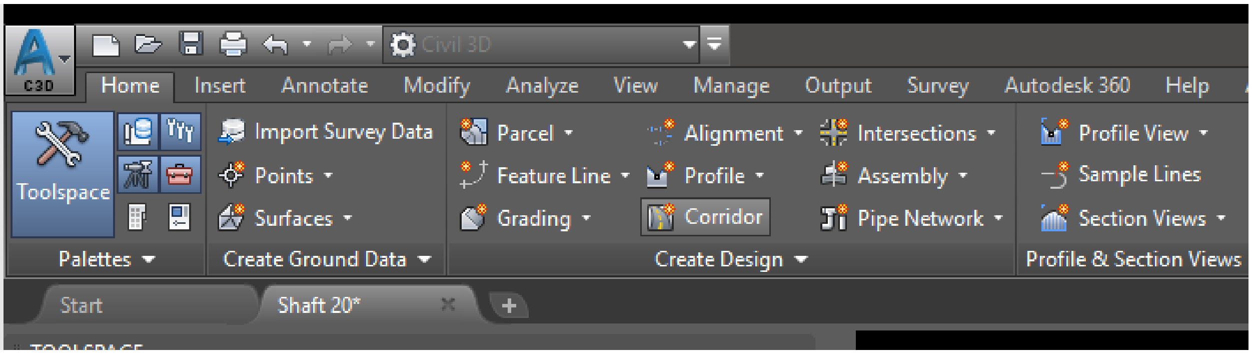


Figure J.20 Corridor tool

Now you are ready to build your corridor. Click on the corridor tool in the home tab (Figure J.21) You will need three pieces of information initially – the base surface, the profile you have just drawn and the assembly that you want to use. If you have started with a void volume template the assembly will already be in the drawing so you just need to select it (Figure J.22).

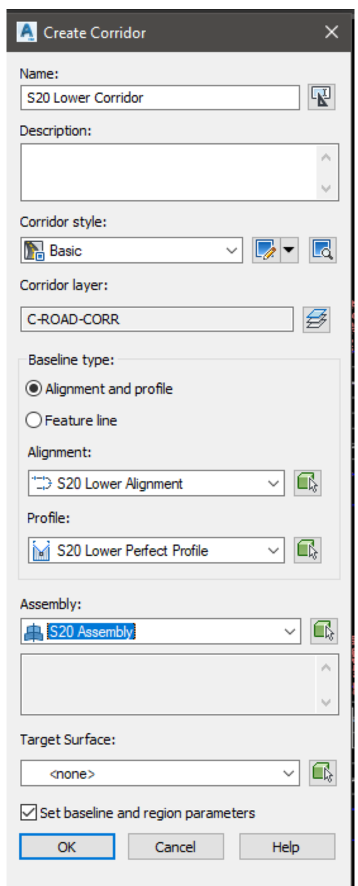


Figure J.21 Assembly selection

After the initial screen you will get a dialog box where you can set the range of the corridor. You need to choose a 12 inch section within your profile. You can do this by typing in the stationing or by selection the stationing in your model space. It is important that this section is only 12 inches because the assembly is 12 inches wide and that is how you get a 1' by 1' square test area. Once the stationing is selected you can run the corridor. A square should show up on the rendering of your shaft surface (Figure J. 23).

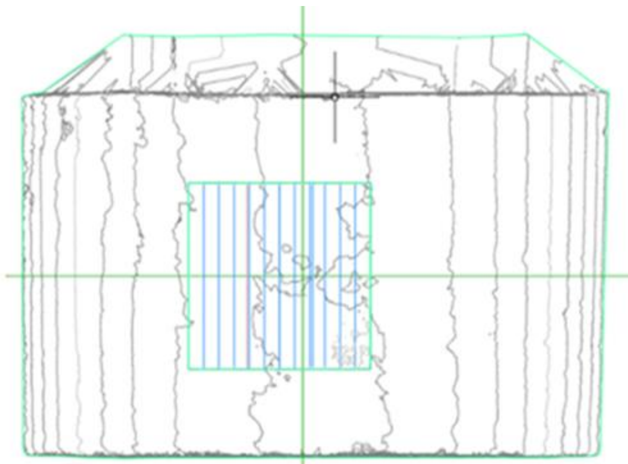


Figure J.22 Corridor

In order to get a volume you need to create a surface out of your corridor. You do this by clicking on the corridor and then create a surface in the corridor ribbon (Figure J.24). You will need to create the surface and then add the top tag (Figure J.25). Now that the surface is created, you can do a surface comparison using the volumes dashboard under the analyze tab (Figure J.26.) Once open, this will ask you to create a new volume, then you add a base surface (the shaft) and a comparison surface (the corridor) this will generate a total volume of Cut/fill (Figure J.27).

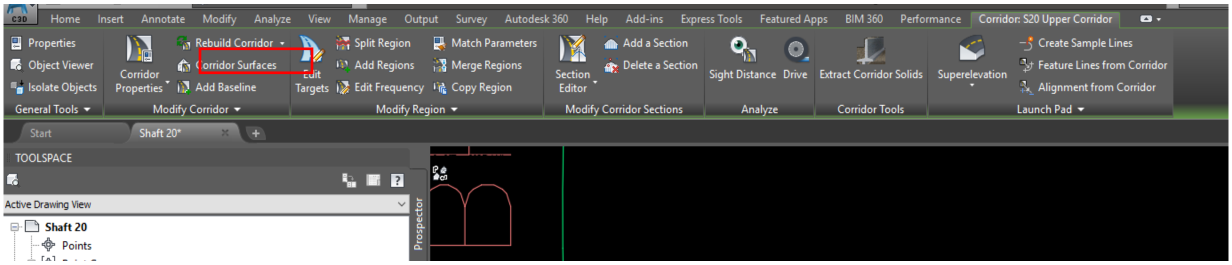


Figure J.23 Corridor surface tool

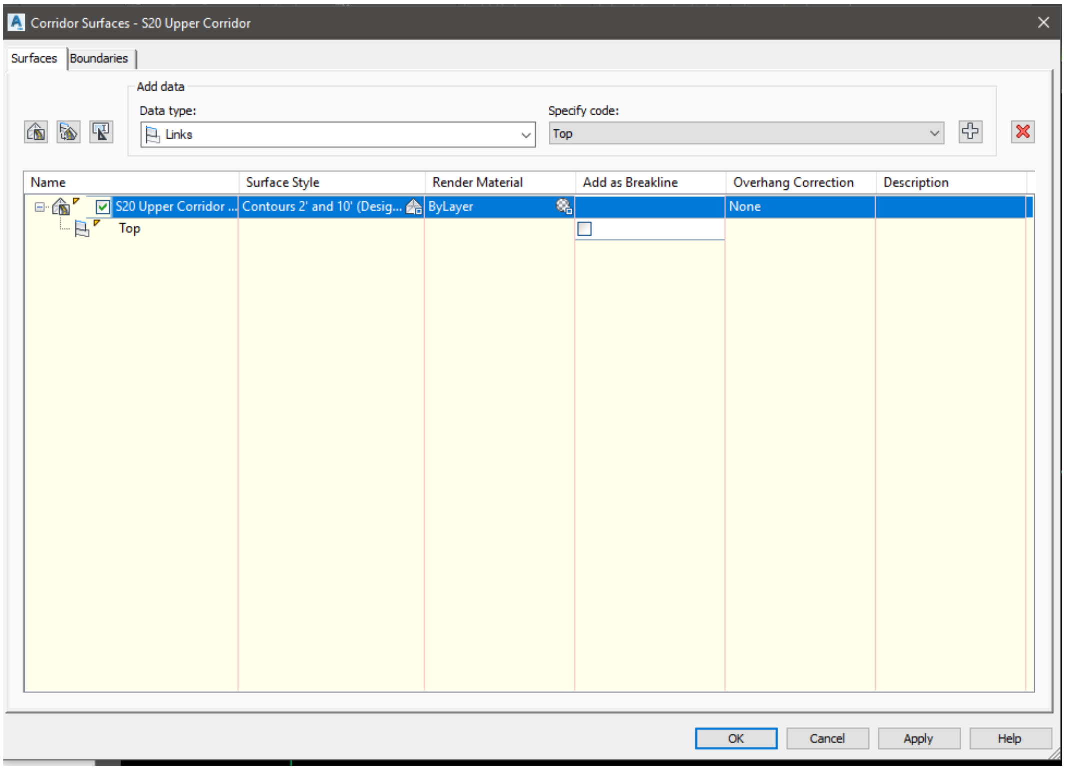


Figure J.24 Corridor surface top tag

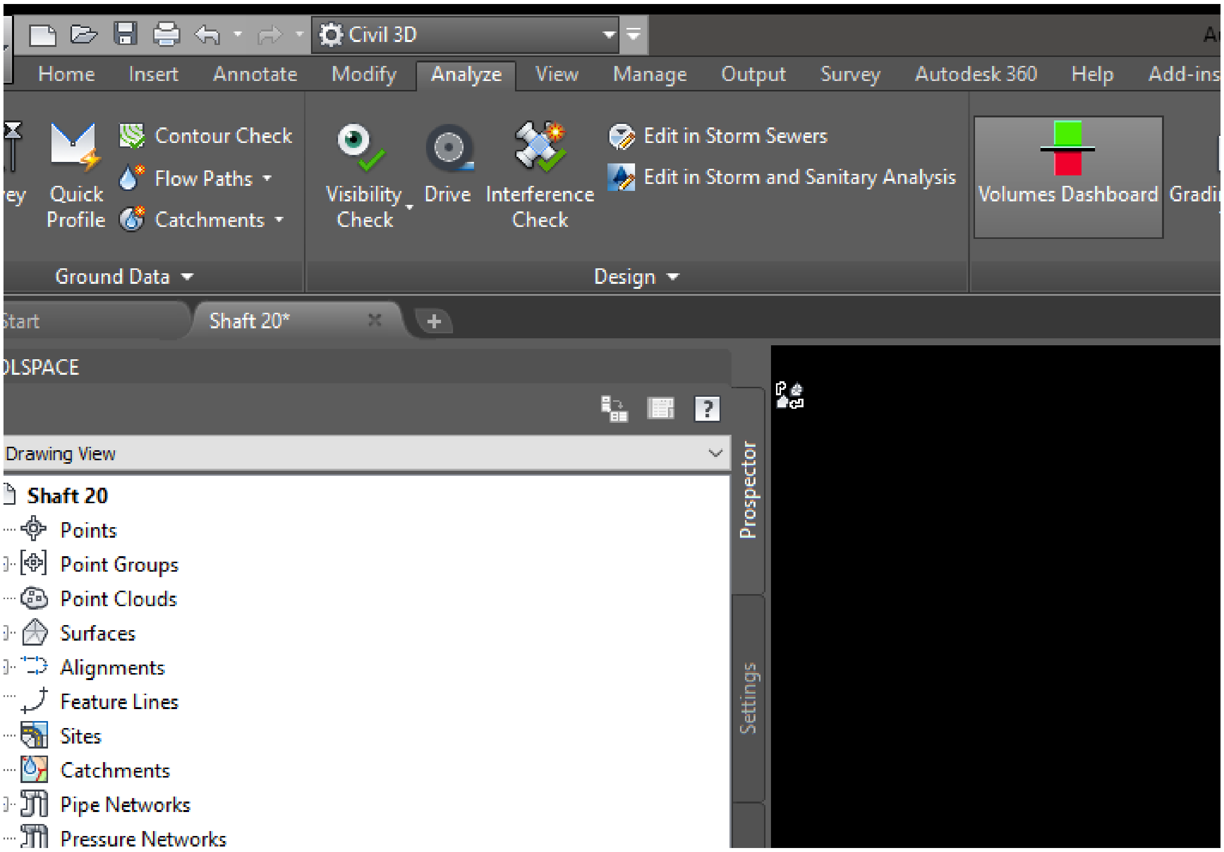


Figure J.25 Volumes dashboard

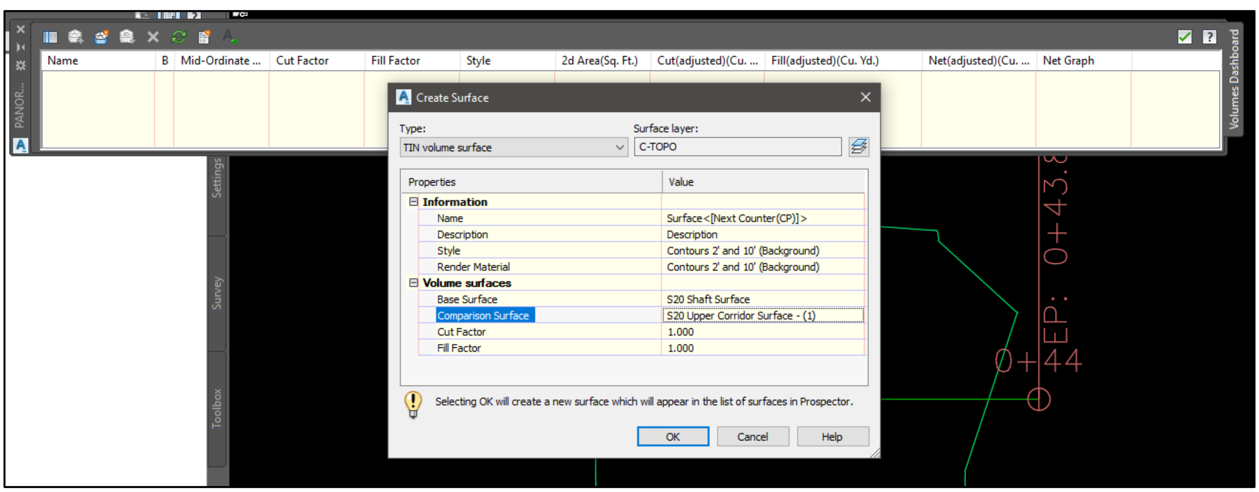


Figure J.26 Volumes dashboard set up

You should only have fill. If you have cut, then your profile is too close to the existing surface you will need to adjust the three point curve and then update the corridor and the volume (there will be yellow exclamation points prompting you to update). The volume is given in cubic yards so you will need to convert to cubic feet and that will be your surface roughness (in^3/sf).

APPENDIX K – BRIDGE PLANS COMPLETE

FRACTURE CRITICAL INSPECTION

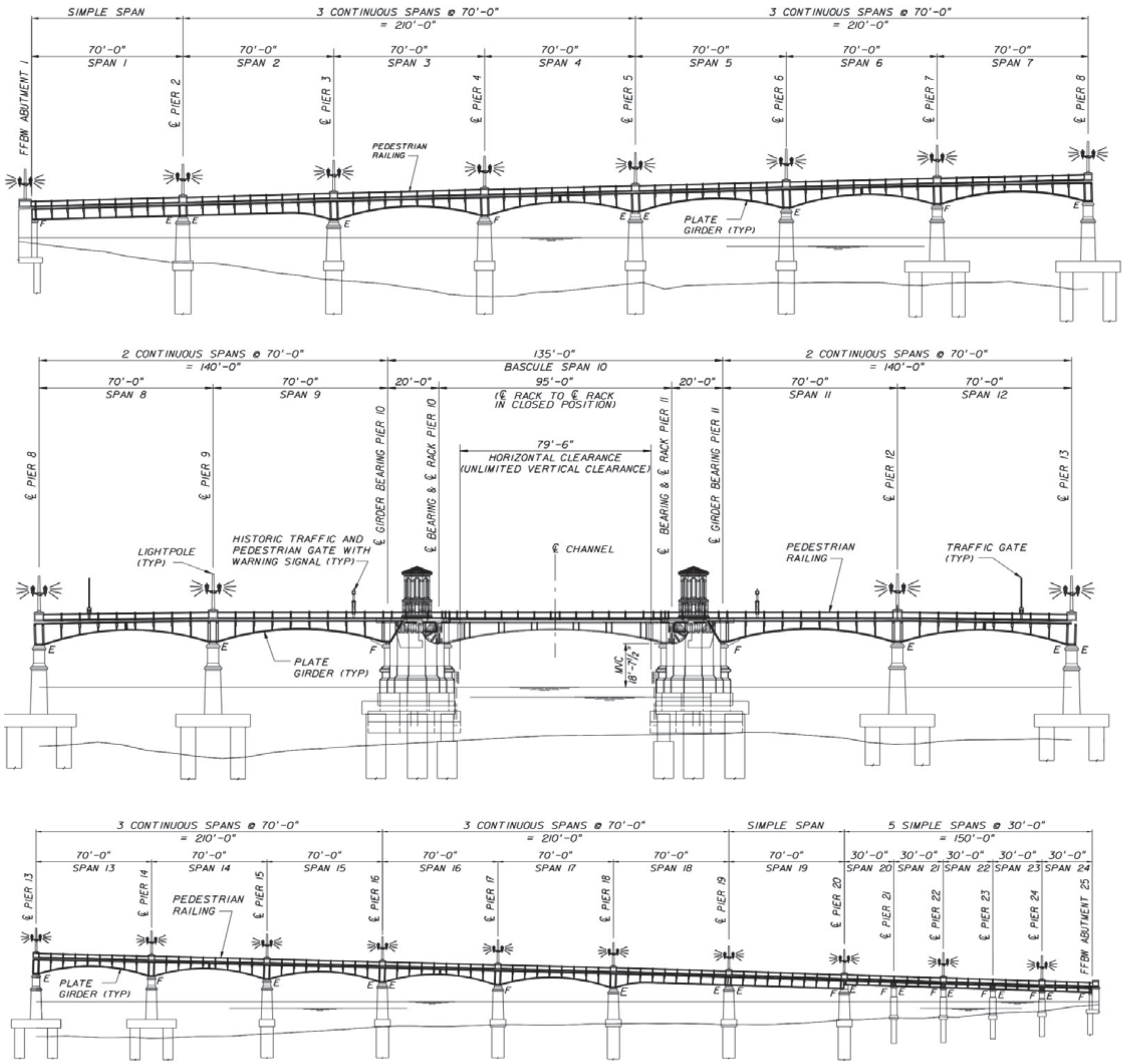


FIGURE 1. BRIDGE ELEVATION LOOKING SOUTH

(ALL REINFORCING STEEL BARS ARE BOTH TOP AND BOTTOM EXCEPT OTHERWISE NOTED)

NOTES:

1. FOR DETAILS OF FENDER MOUNTING BLOCK SEE SHEET BP52.
2. FOR DETAILS OF FILLING EXISTING OPENINGS WITH CONCRETE SEE SHEET BP53.
3. THE SURFACES OF EXISTING FOOTING AND CAISSONS CONTACTING THE NEW FOOTING SHALL BE CLEANED BY WATER JETTING TO REMOVE ALL DELETERIOUS MATERIALS SUCH AS MARINE GROWTH, LOOSE RUST, LOOSE CONCRETE, ETC. TO THE SATISFACTION OF THE ENGINEER. COLLECT ALL MATERIAL AND RUNOFF. THIS WORK IS INCIDENTAL TO THE FOOTING CONCRETE ITEM. 1

AS-BUILT

THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.

AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC. AB

NOTES (CONT.): 1

4. THE NEW AND EXISTING CONCRETE SHALL BE IN CONTACT THROUGHOUT WITH NO VOIDS BETWEEN THEM. SPALLS, VOIDS & IRREGULARITIES GREATER THAN ½" DEEP IN THE UNDERSIDE OF THE EXISTING FOOTING SHALL BE PATCHED TO MATCH THE ADJACENT SURFACES. THIS WORK IS INCIDENTAL TO THE FOOTING CONCRETE ITEM.
5. HIGH SLUMP CONCRETE IS REQUIRED TO ENSURE FLOW OF NEW CONCRETE UNDER EXISTING FOOTING. HIGH SLUMP CONCRETE SHALL MEET THE REQUIREMENTS OF THE SPECIFICATIONS AND SHALL NOT EXCEED 7" SLUMP.

BRIDGE NO. 780074

GARY ALAN KLEVENS LICENSE No 47187 STATE OF FLORIDA PROFESSIONAL ENGINEER 07/20/11

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
03/19/04	RWM	1 ADDED, DELETED AND REVISED NOTES			
07/20/11	JH	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	CLP	08/03
CHECKED BY	RWM	08/03
DESIGNED BY	JH	08/03
CHECKED BY	RWM	08/03
APPROVED BY	G.A. KLEVENS P.E.	

ENGINEER OF RECORD:
LICHTENSTEIN CONSULTING ENGINEERS, INC.
2700 W. Cypress Creek Rd., Suite D-140
Fort Lauderdale, Florida 33309
Authorization No. 00006065
G. Alan Klevens, P.E.
P.E. License No. 47187

FLORIDA DEPARTMENT OF TRANSPORTATION

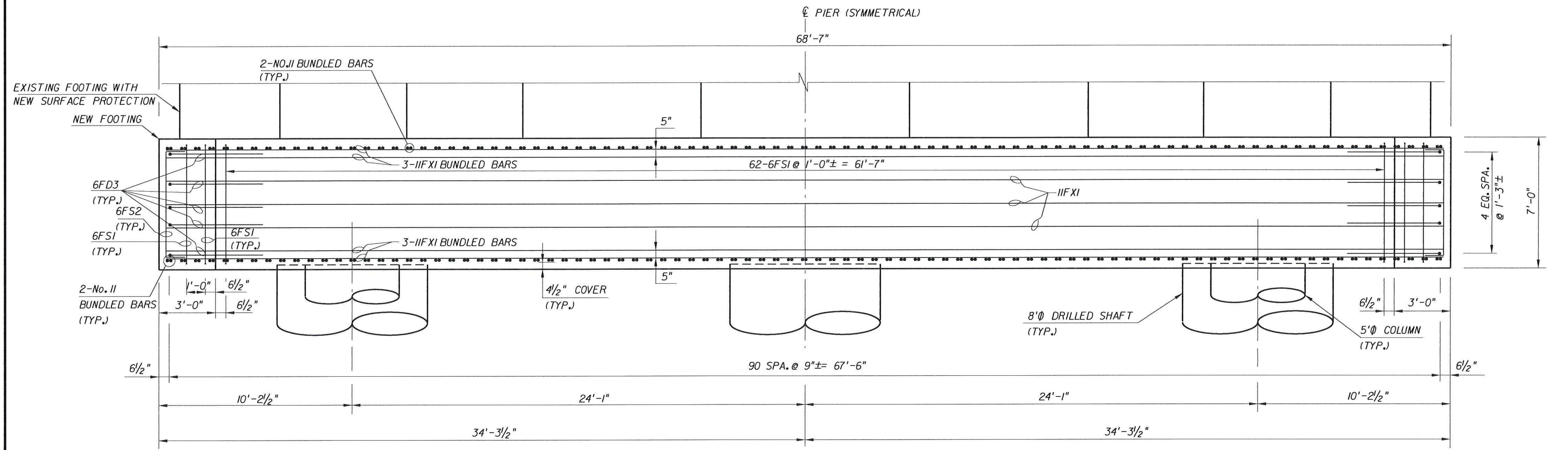
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: BASCULE PIER NEW FOOTING REINFORCING STEEL 1 OF 6

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. BP48

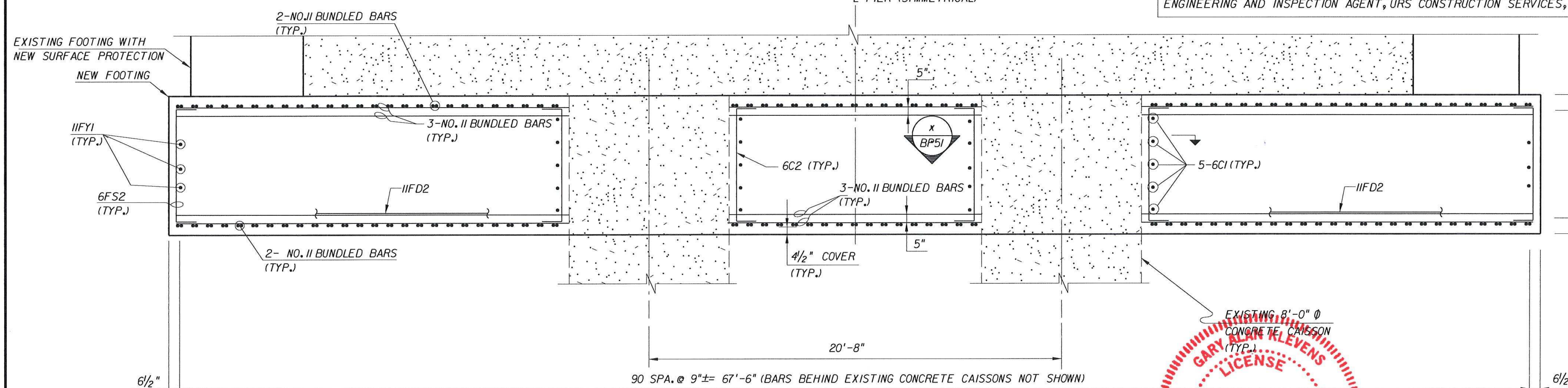
7/20/2011 9:37:24 AM $FILEABBREV$



VIEW (A / BP48)

(EXISTING 8'Ø CONCRETE CAISSONS AND FENDER MOUNTING BLOCKS NOT SHOWN FOR CLARITY)

AS-BUILT
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.



SECTION (B / BP48)

(NEW DRILLED SHAFTS AND FAR EXISTING CONCRETE CAISSONS NOT SHOWN FOR CLARITY)

LEGEND:

EXISTING

NEW

BAR DESIGNATION NOTES:

1. DESIGNATION 2-IIFY** DENOTES A PAIR OF BUNDLED #II BARS (2 BARS TOTAL PER LOCATION).
2. DESIGNATION 6-IIFX** DENOTES 2 ROWS OF 3 BARS BUNDLED (6 BARS TOTAL PER LOCATION). SEE SECTIONS FOR DETAILS.

NOTES:

1. WORK THIS SHEET WITH SHEET BP48.

GARY ALAN KLEVENS LICENSE No 47187 STATE OF FLORIDA PROFESSIONAL ENGINEER 07/20/11

BRIDGE NO. 780074

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
03/19/04	RWM	1 ADDED BAR DESIGNATION NOTES			
07/20/11	JH	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	CLP	08/03
CHECKED BY	RWM	08/03
DESIGNED BY	JH	08/03
CHECKED BY	RWM	08/03
APPROVED BY	G.A. KLEVENS P.E.	

ENGINEER OF RECORD:
LICHTENSTEIN CONSULTING ENGINEERS, INC.
2700 W. Cypress Creek Rd., Suite D-140
Fort Lauderdale, Florida 33309
Authorization No. 00006065
G. Alan Klevens, P.E.
P.E. License No. 47187

FLORIDA DEPARTMENT OF TRANSPORTATION

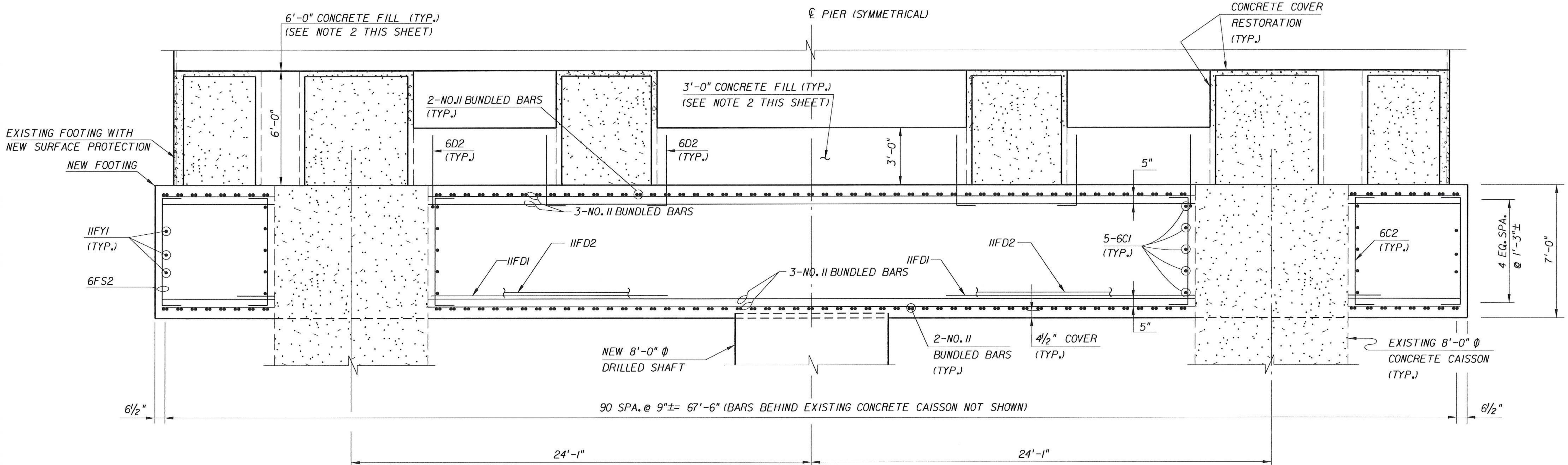
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: BASCULE PIER NEW FOOTING REINFORCING STEEL 2 OF 6

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

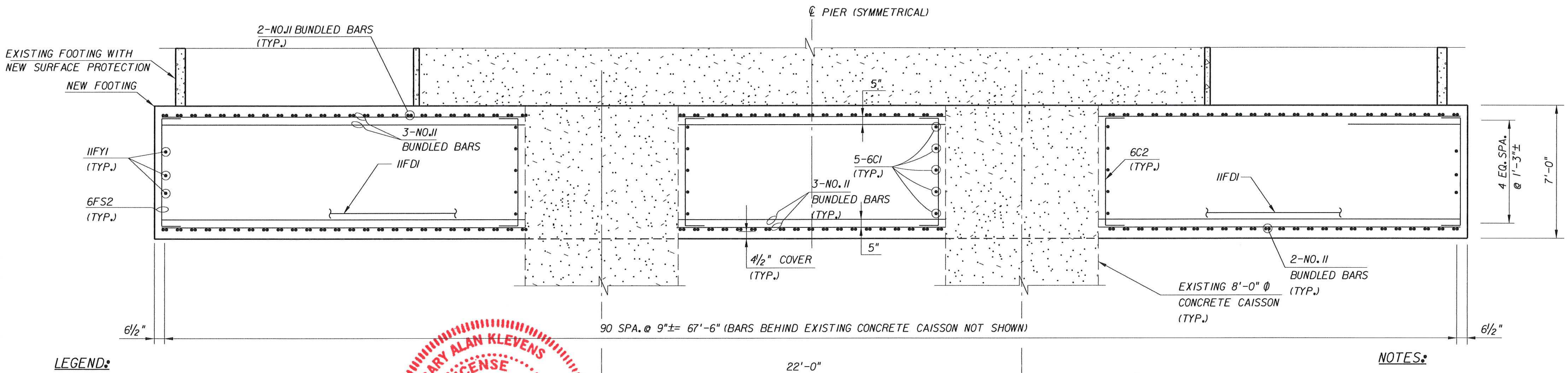
SHEET NO. BP49

7/20/2011 9:38:17 AM $FILEABBREV$



SECTION ℄ / BP48

(FAR EXISTING CONCRETE CAISSONS NOT SHOWN FOR CLARITY)



SECTION D / BP48

(NEW DRILLED SHAFTS NOT SHOWN FOR CLARITY)

LEGEND:

- EXISTING
- NEW
- NEW CONCRETE COVER RESTORATION

GARY ALAN KLEVENS LICENSE No 47187 STATE OF FLORIDA PROFESSIONAL ENGINEER 07/20/11

AS-BUILT

THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.

AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.

NOTES:

1. WORK THIS SHEET WITH SHEET BP48.
2. FOR DETAILS OF FILLING EXISTING OPENINGS WITH CONCRETE SEE SHEET BP53.

BRIDGE NO. 780074

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
07/20/11	JH	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	CLP	08/03
CHECKED BY	RWM	08/03
DESIGNED BY	JH	08/03
CHECKED BY	RWM	08/03
APPROVED BY	G.A. KLEVENS P.E.	

ENGINEER OF RECORD:
LICHTENSTEIN CONSULTING ENGINEERS, INC.
2700 W. Cypress Creek Rd., Suite D-140
Fort Lauderdale, Florida 33309
Authorization No. 00006065
G. Alan Klevens, P.E.
P.E. License No. 47187

FLORIDA DEPARTMENT OF TRANSPORTATION

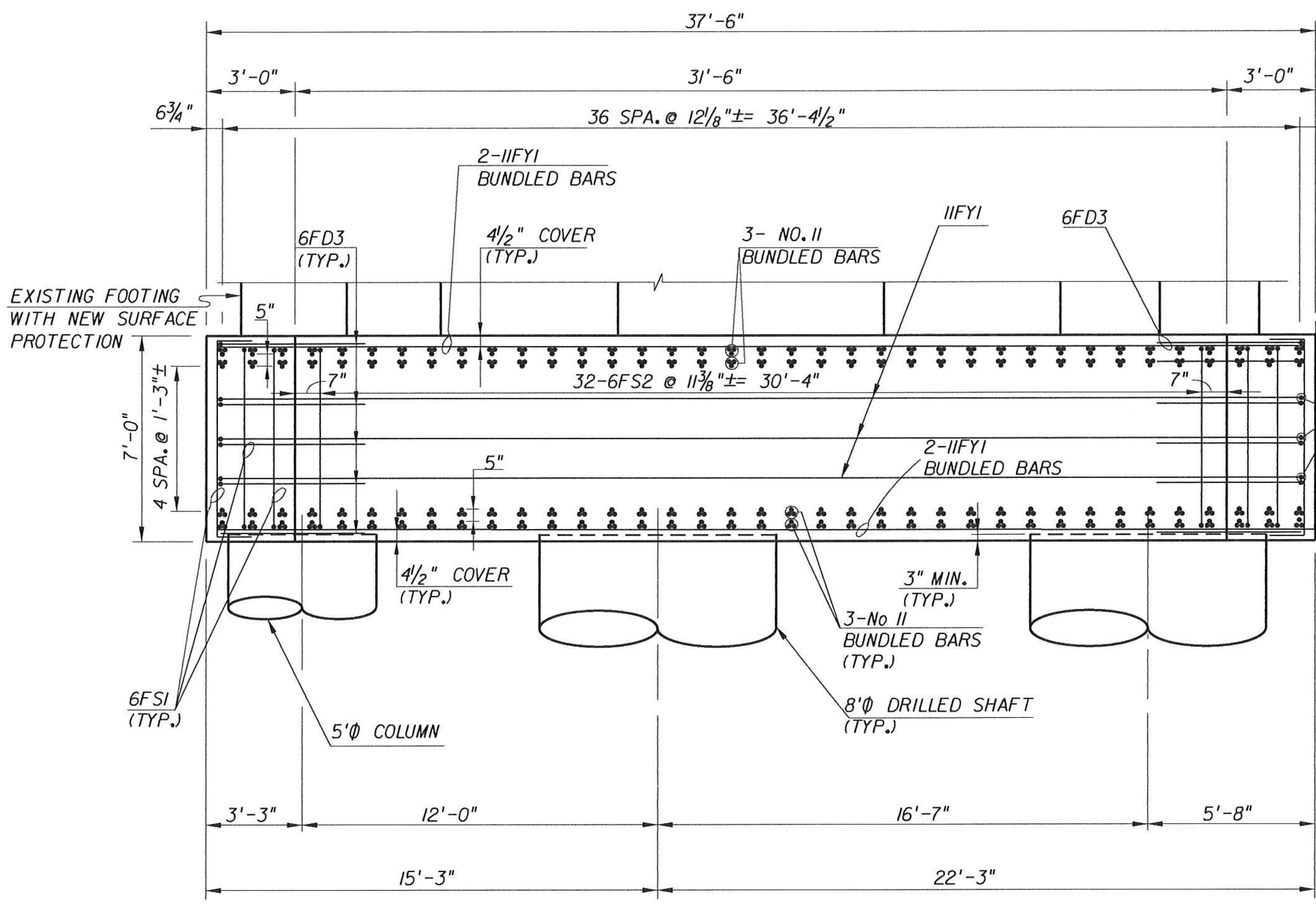
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: BASCULE PIER NEW FOOTING REINFORCING STEEL 3 OF 6

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

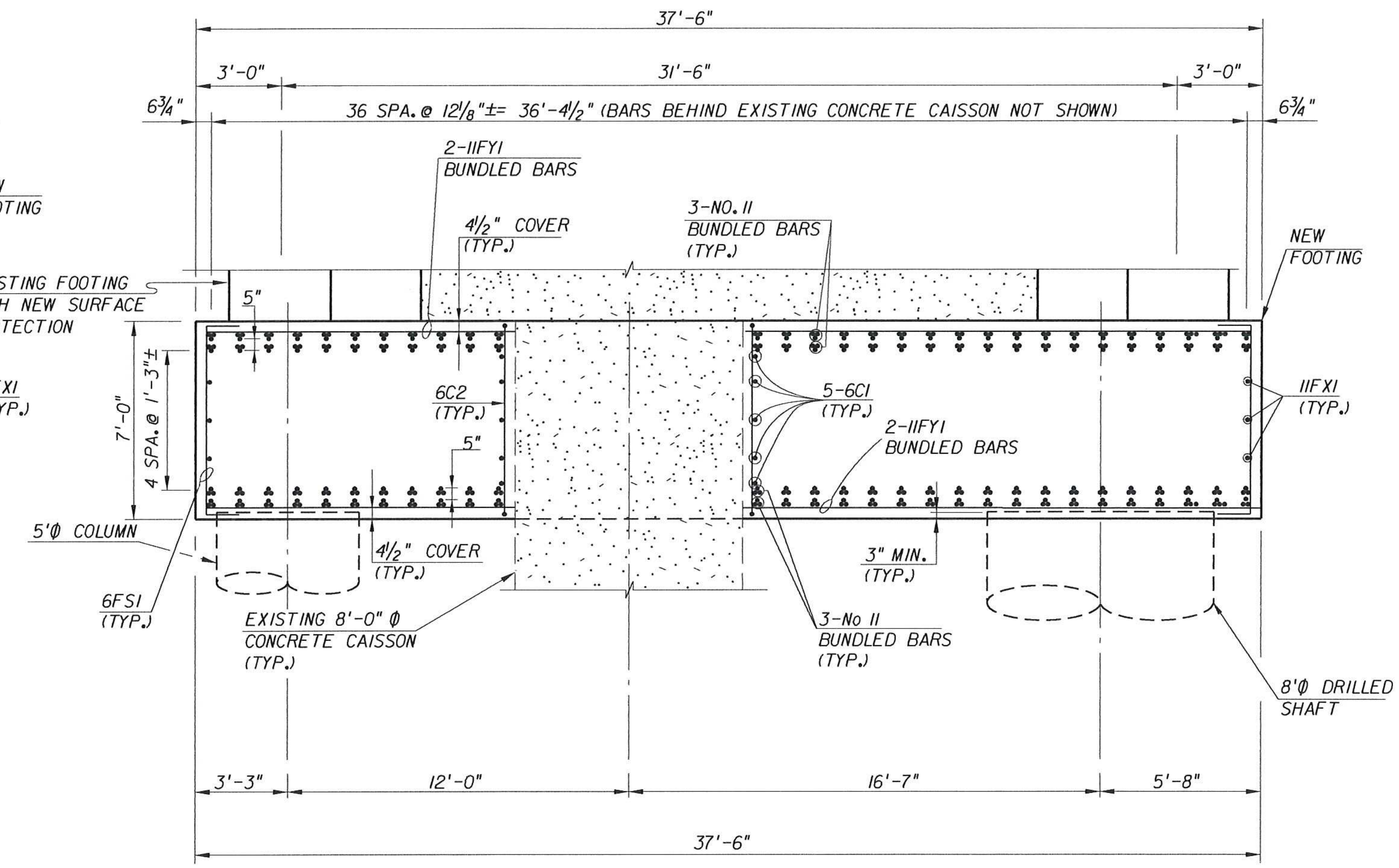
SHEET NO. BP50

7/20/2011 9:39:30 AM $FILEABBREV$



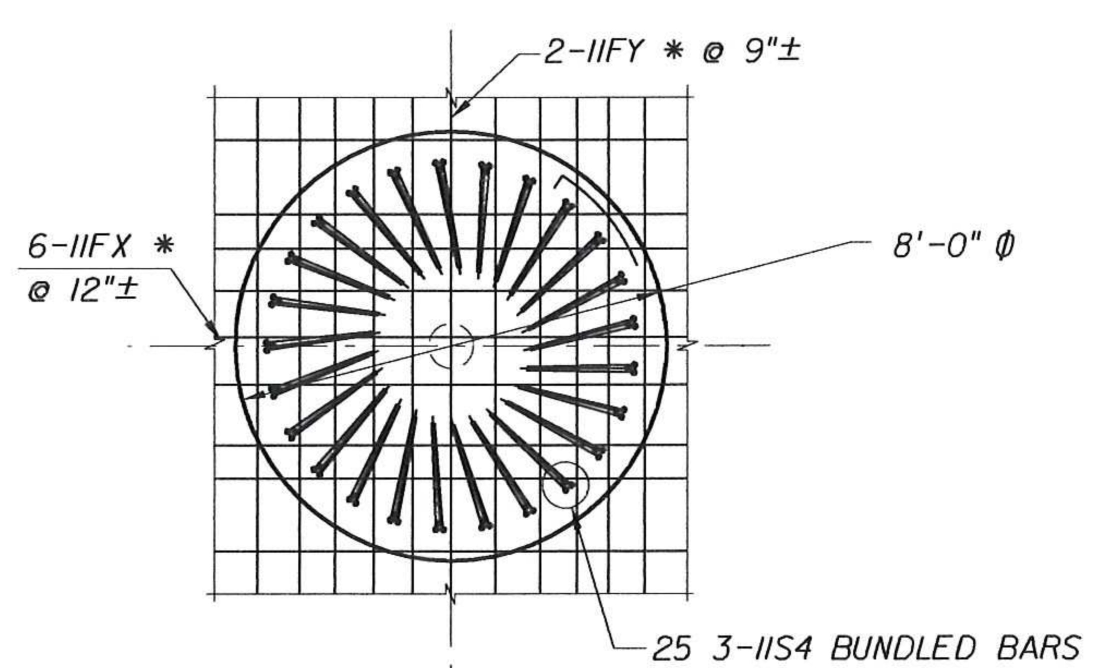
VIEW E / BP48

(EXISTING 8'Ø CONCRETE CAISSONS AND FENDER MOUNTING BLOCKS NOT SHOWN FOR CLARITY)

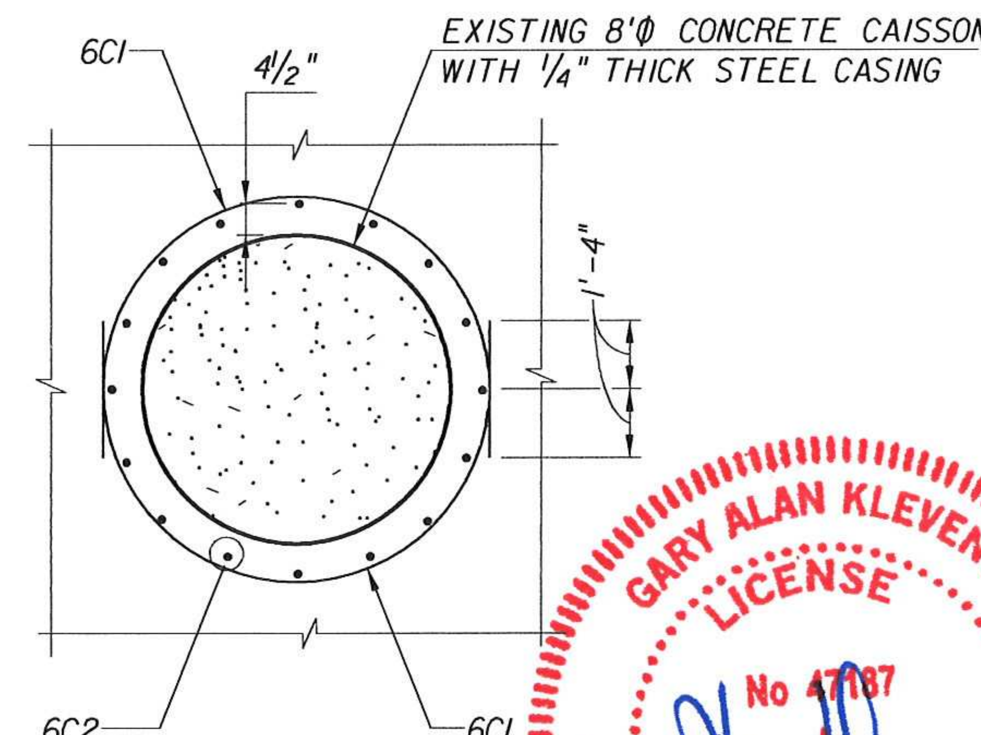


SECTION F / BP48

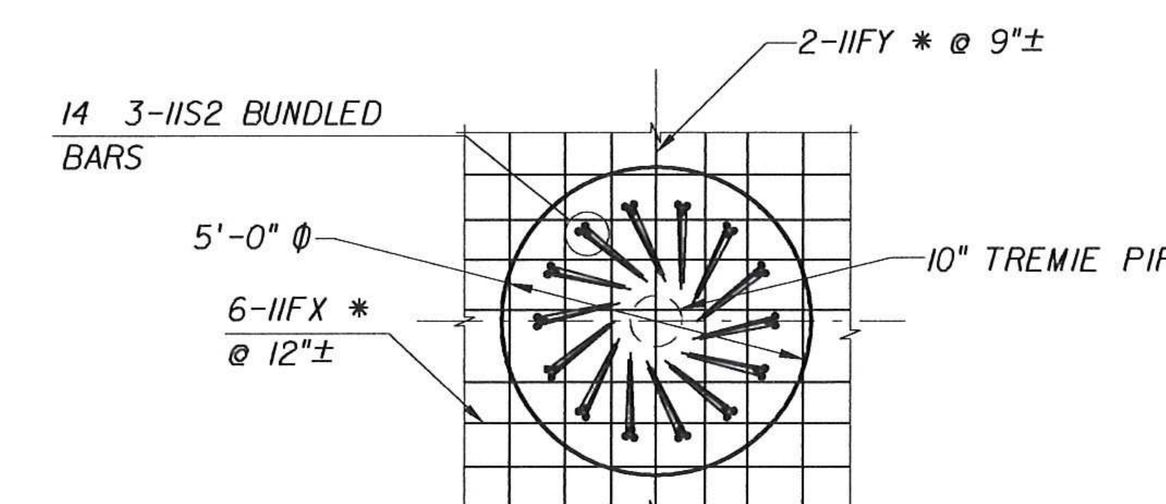
(FAR EXISTING 8'Ø CONCRETE CAISSONS NOT SHOWN FOR CLARITY)



REINFORCING STEEL DETAIL AT THE INTERFACE OF 8' Ø DRILLED SHAFT TO FOOTING



SECTION X / BP49



REINFORCING STEEL DETAIL AT THE INTERFACE OF 5' Ø COLUMN TO FOOTING

LEGEND:

EXISTING

NEW

AB

AS-BUILT

THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.

AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.

GARY ALAN KLEVENS LICENSE No 47187 STATE OF FLORIDA PROFESSIONAL ENGINEER

07/26/11

NOTES:

1. WORK THIS SHEET WITH SHEET BP48.

BRIDGE NO. 780074

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
07/20/11	JH	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	CLP	08/03
CHECKED BY	RWM	08/03
DESIGNED BY	JH	08/03
CHECKED BY	RWM	08/03
APPROVED BY	G.A. KLEVENS P.E.	

ENGINEER OF RECORD:
LICHTENSTEIN CONSULTING ENGINEERS, INC.
2700 W. Cypress Creek Rd., Suite D-140
Fort Lauderdale, Florida 33309
Authorization No. 00006065
G. Alan Klevens, P.E.
P.E. License No. 47187

FLORIDA DEPARTMENT OF TRANSPORTATION

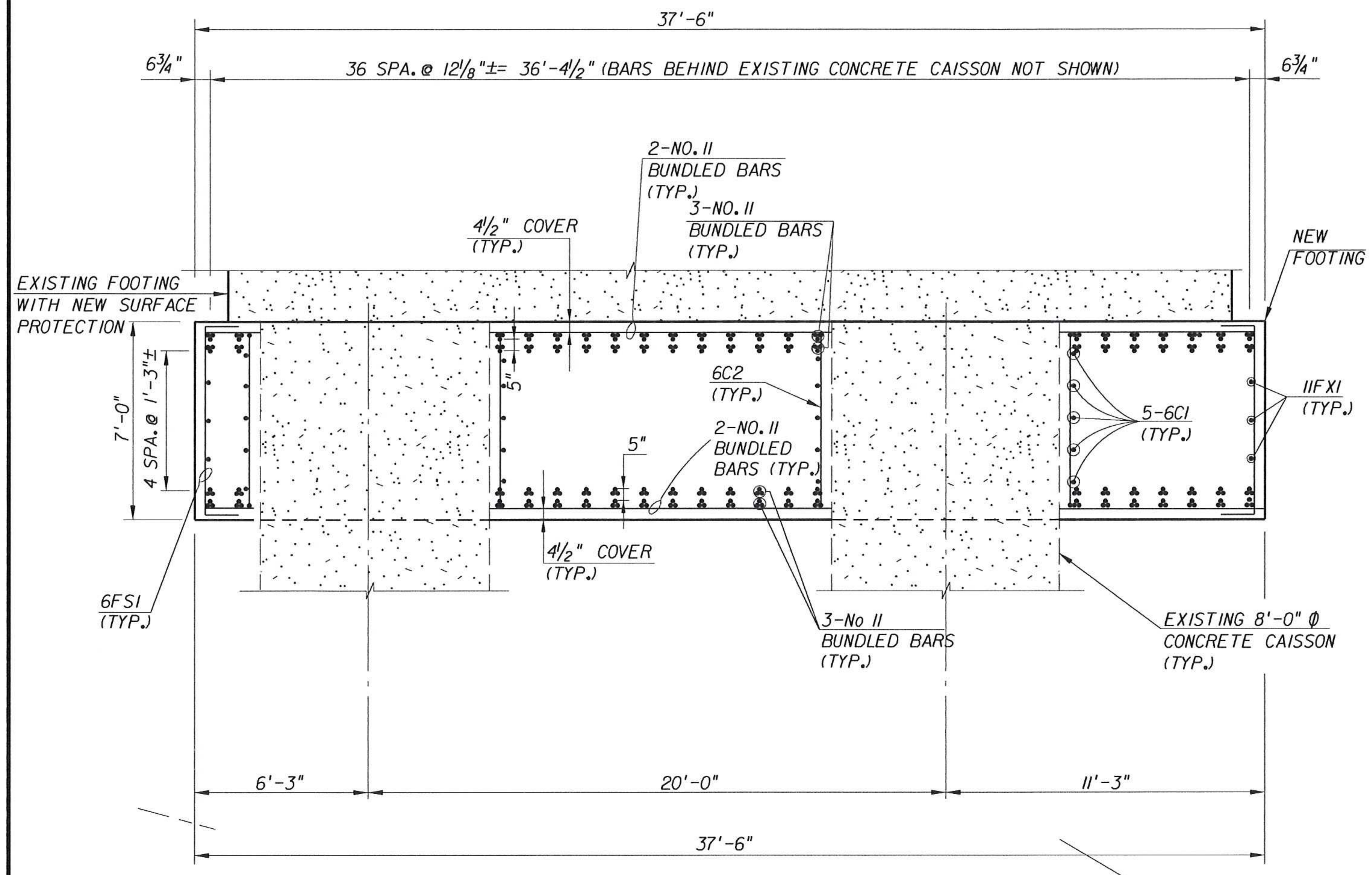
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: BASCULE PIER NEW FOOTING REINFORCING STEEL 4 OF 6

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

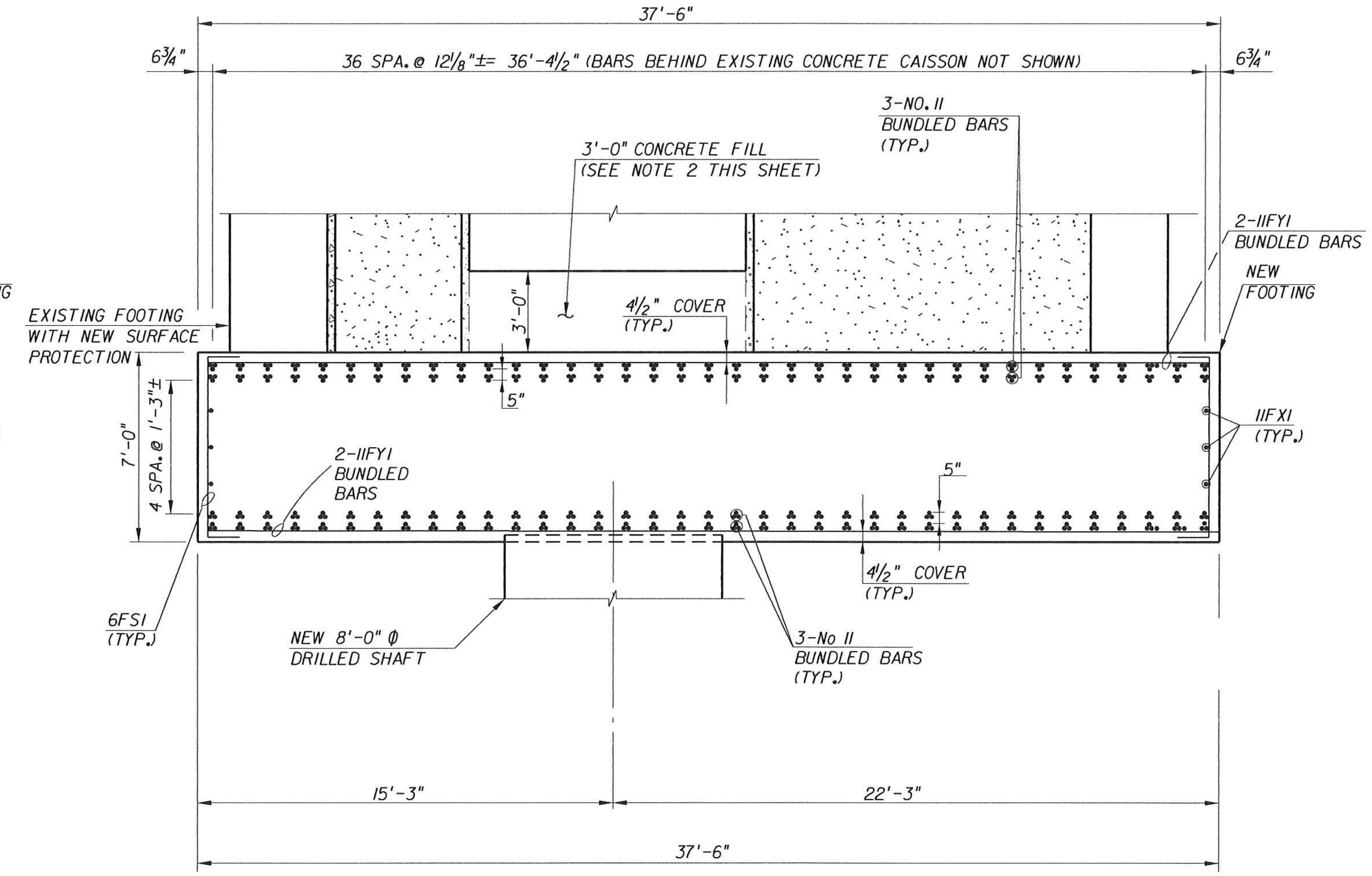
SHEET NO. BP51

7/20/2011 9:42:07 AM $FILEABBREV$



SECTION G / BP48

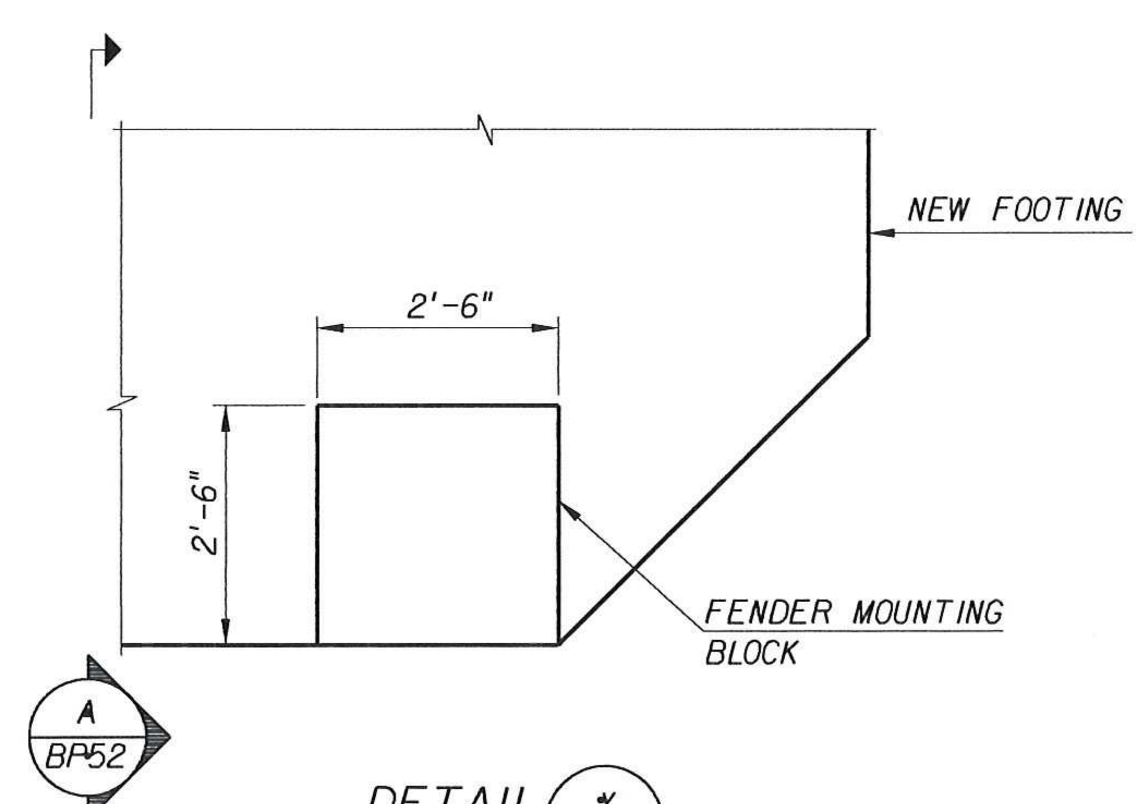
(NEW DRILLED SHAFTS AND FAR EXISTING 8'Ø CONCRETE CAISSONS NOT SHOWN FOR CLARITY)



SECTION H / BP48

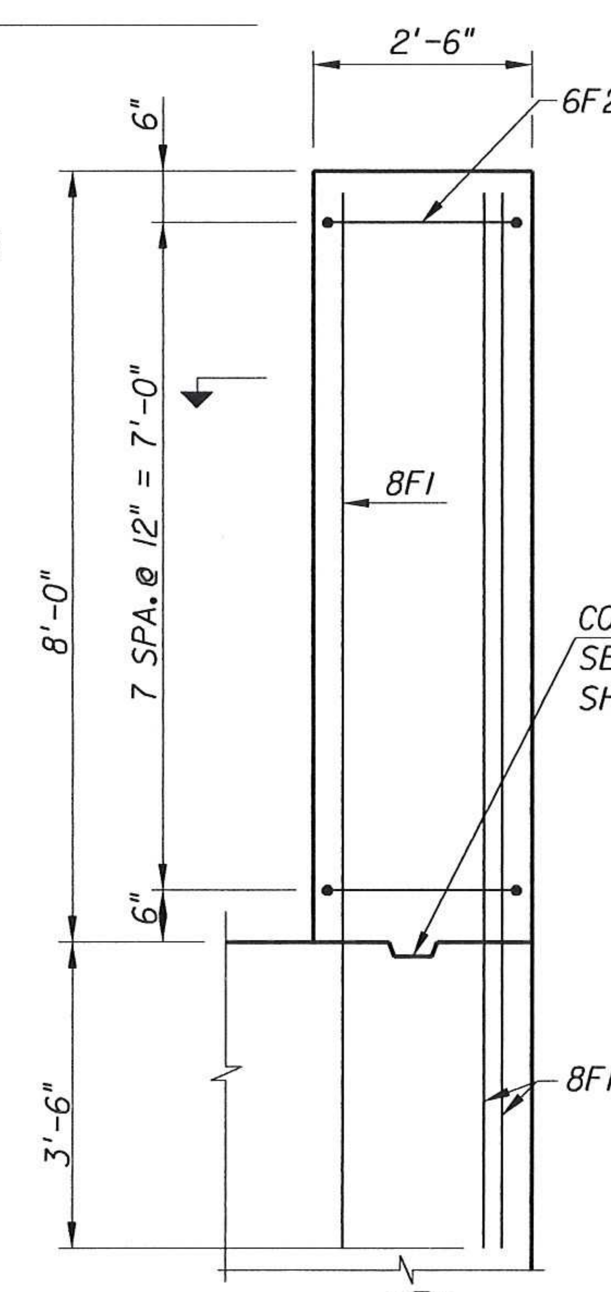
(FAR NEW DRILLED SHAFTS AND EXISTING 8'Ø CONCRETE CAISSONS NOT SHOWN FOR CLARITY)

AS-BUILT
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.

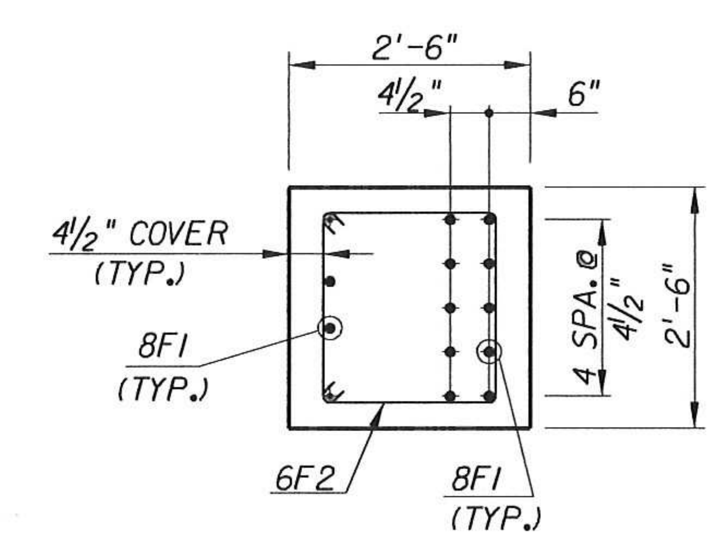


DETAIL X / BP48

(4 REQUIRED PER FOOTING)



VIEW A / BP52



SECTION B / BP52

GARY ALAN KLEVENS
LICENSE
No 47187
STATE OF
FLORIDA
PROFESSIONAL ENGINEER
07/20/11

LEGEND:

EXISTING

NEW

NEW CONCRETE COVER RESTORATION

NOTES:

1. WORK THIS SHEET WITH SHEET BP48.
2. FOR DETAILS OF FILLING EXISTING OPENINGS WITH CONCRETE, SEE SHEET BP53.

BRIDGE NO. 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
07/20/11	JH	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	CLP	08/03
CHECKED BY	RWM	08/03
DESIGNED BY	JH	08/03
CHECKED BY	RWM	08/03
APPROVED BY	G.A. KLEVENS P.E.	

ENGINEER OF RECORD:
LICHTENSTEIN CONSULTING ENGINEERS, INC.
2700 W. Cypress Creek Rd., Suite D-140
Fort Lauderdale, Florida 33309
Authorization No. 00006065
G. Alan Klevens, P.E.
P.E. License No. 47187

FLORIDA DEPARTMENT OF TRANSPORTATION

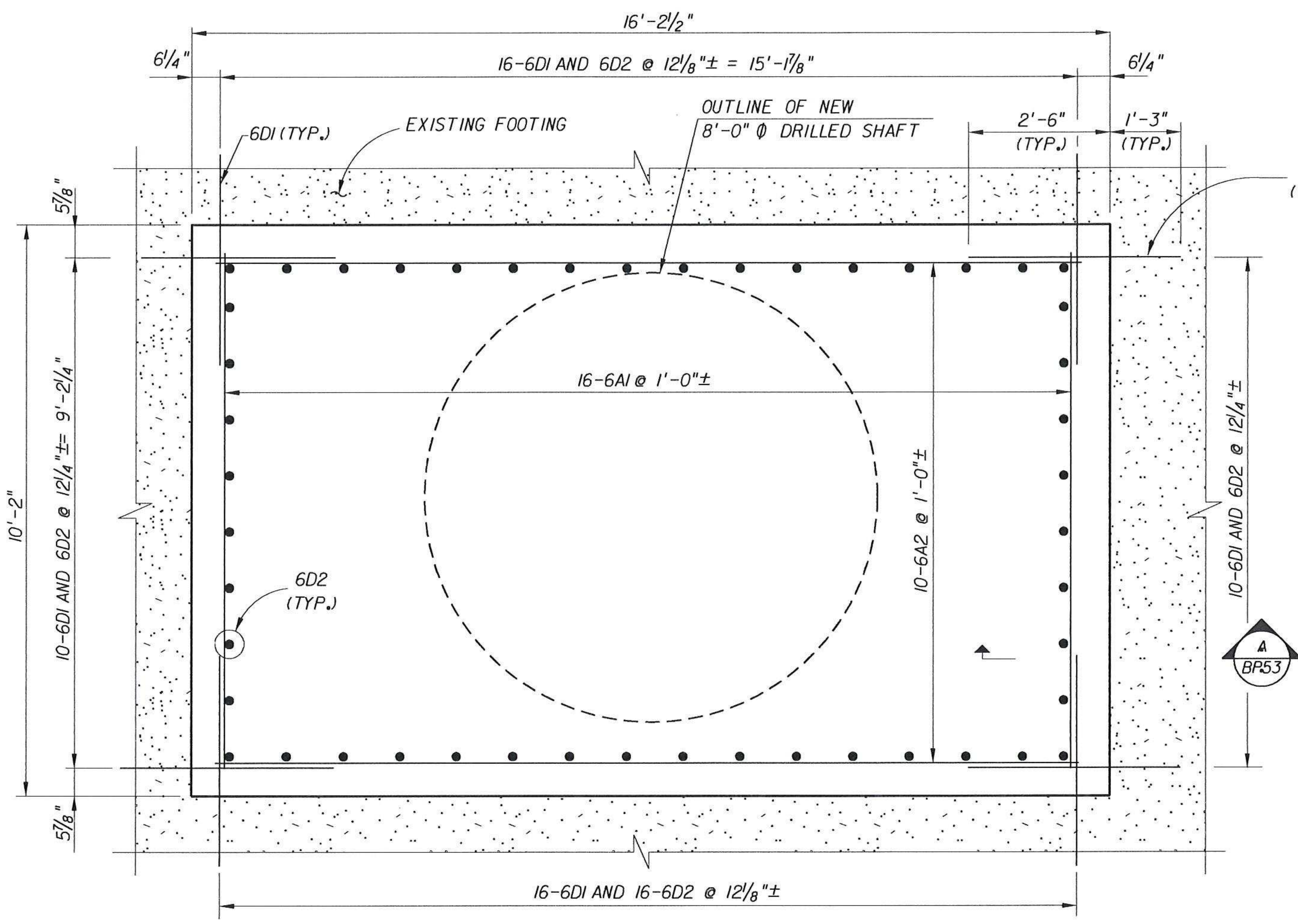
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: BASCULE PIER NEW FOOTING REINFORCING STEEL 5 OF 6

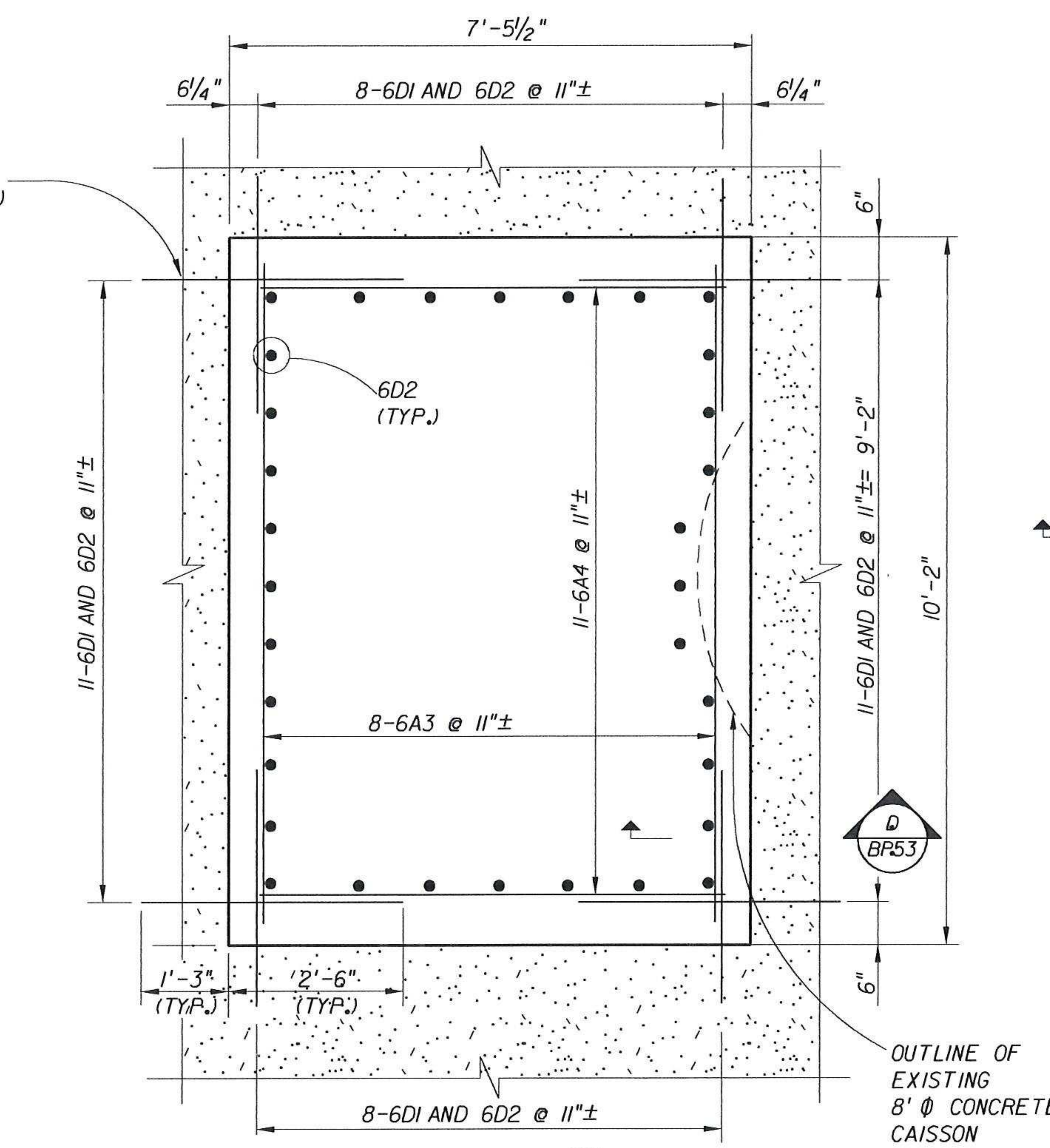
PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. BP52

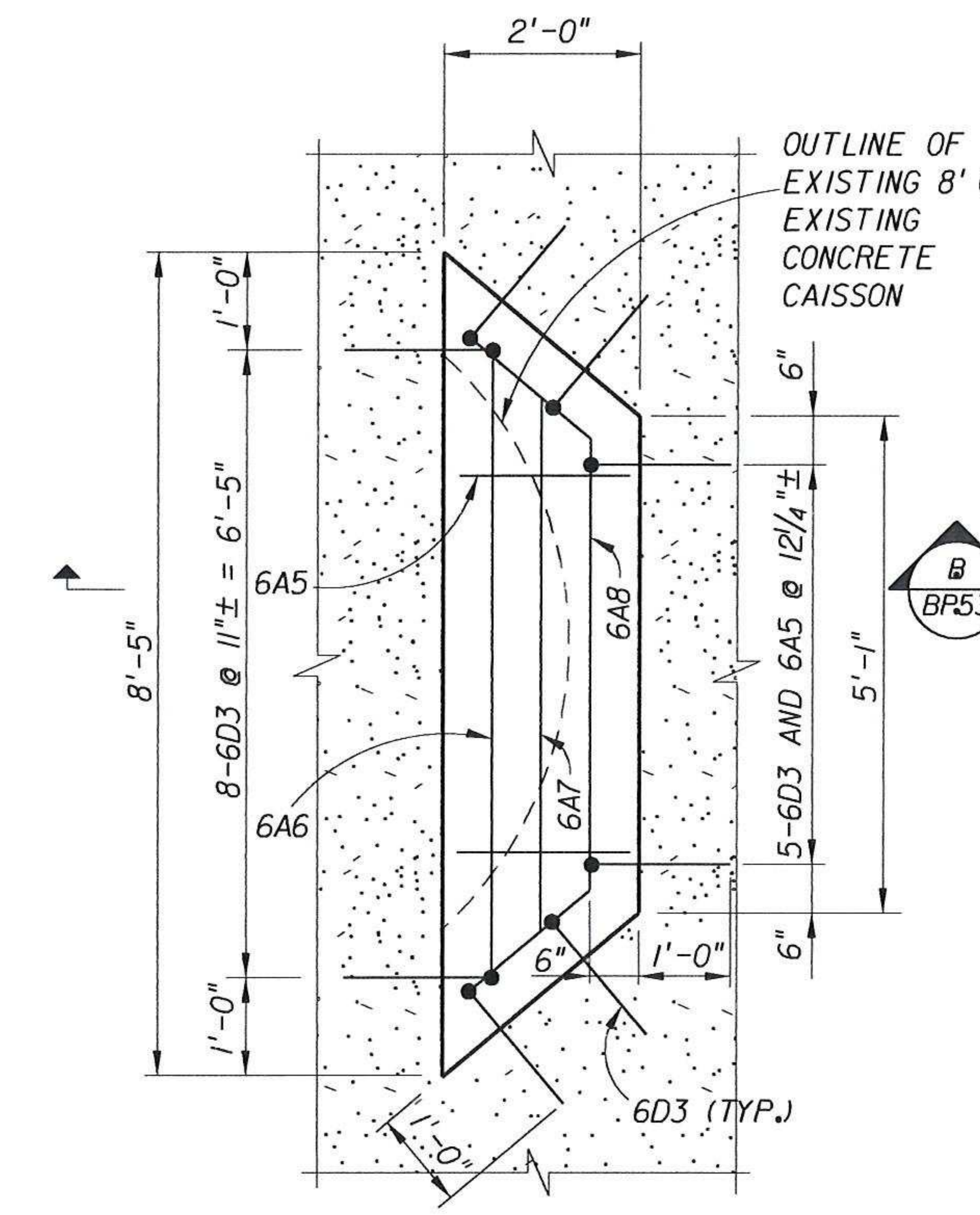
7/20/2011 9:43:15 AM $FILEABBREV$



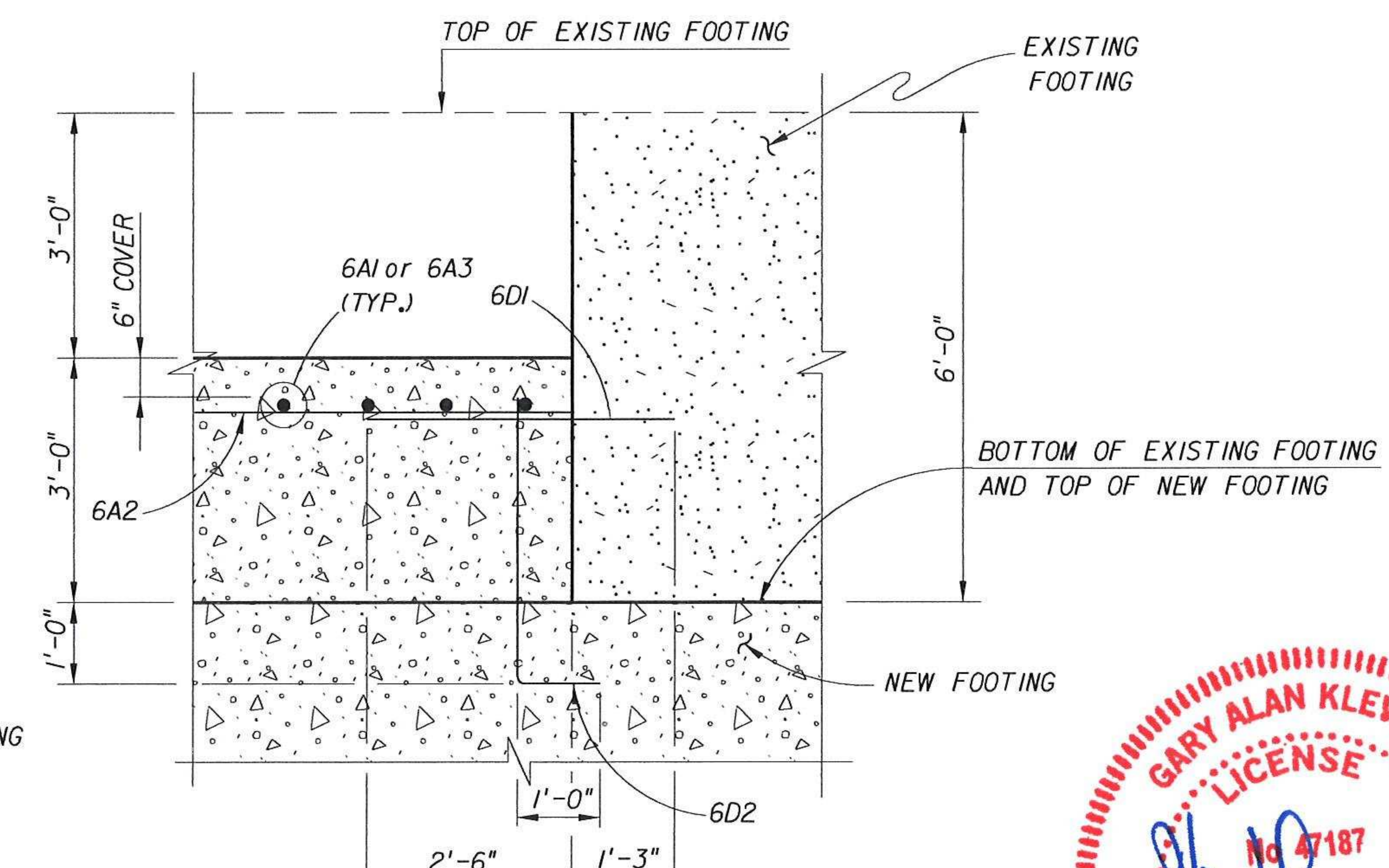
DETAIL 1 BP48



DETAIL 2 BP48



DETAIL 3 BP48

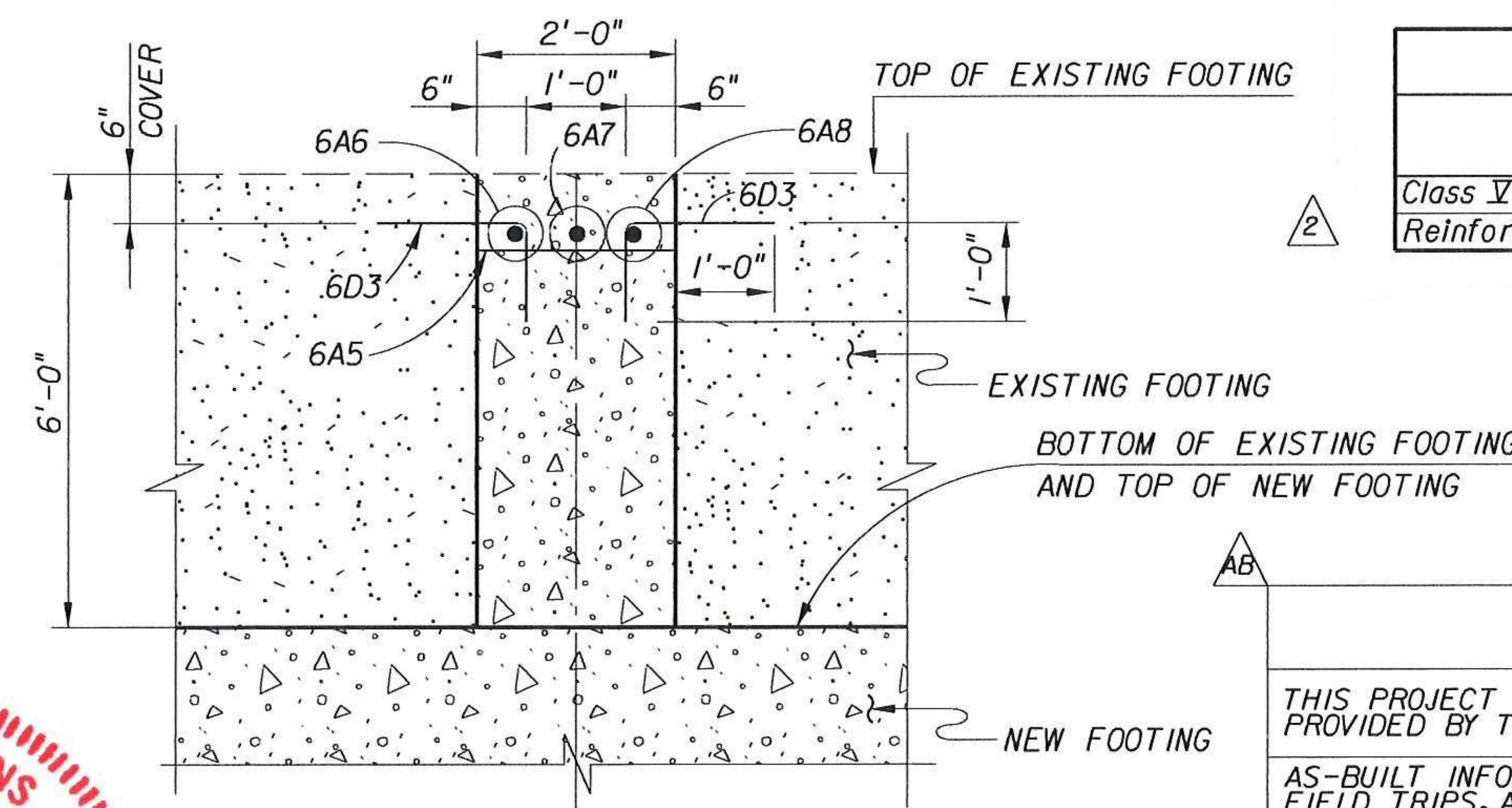


LEGEND:

EXISTING

NEW

SECTION A BP53 D BP53



SECTION B BP53

ESTIMATED QUANTITIES			
ITEM	UNIT	QUANTITIES	
		PIER 15	PIER 16
Class IV Concrete (Substructure) Mass Concrete	CY	639.0	639.0
Reinforcing Steel (Substructure)	LB	223,250	223,250

AS-BUILT

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GARY ALAN KLEVENS LICENSE No 47187 STATE OF FLORIDA PROFESSIONAL ENGINEER 07/20/11

BRIDGE NO. 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
04/02/04	PSF	2 CHANGED CLASS OF CONCRETE			
07/20/11	JH	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	CLB	08/03
CHECKED BY	RWM	08/03
DESIGNED BY	JH	08/03
CHECKED BY	RWM	08/03
APPROVED BY	G.A. KLEVENS P.E.	

ENGINEER OF RECORD:
LICHTENSTEIN CONSULTING ENGINEERS, INC.
2700 W. Cypress Creek Rd., Suite D-140
Fort Lauderdale, Florida 33309
Authorization No. 00006065
G. Alan Klevens, P.E.
P.E. License No. 47187

FLORIDA DEPARTMENT OF TRANSPORTATION

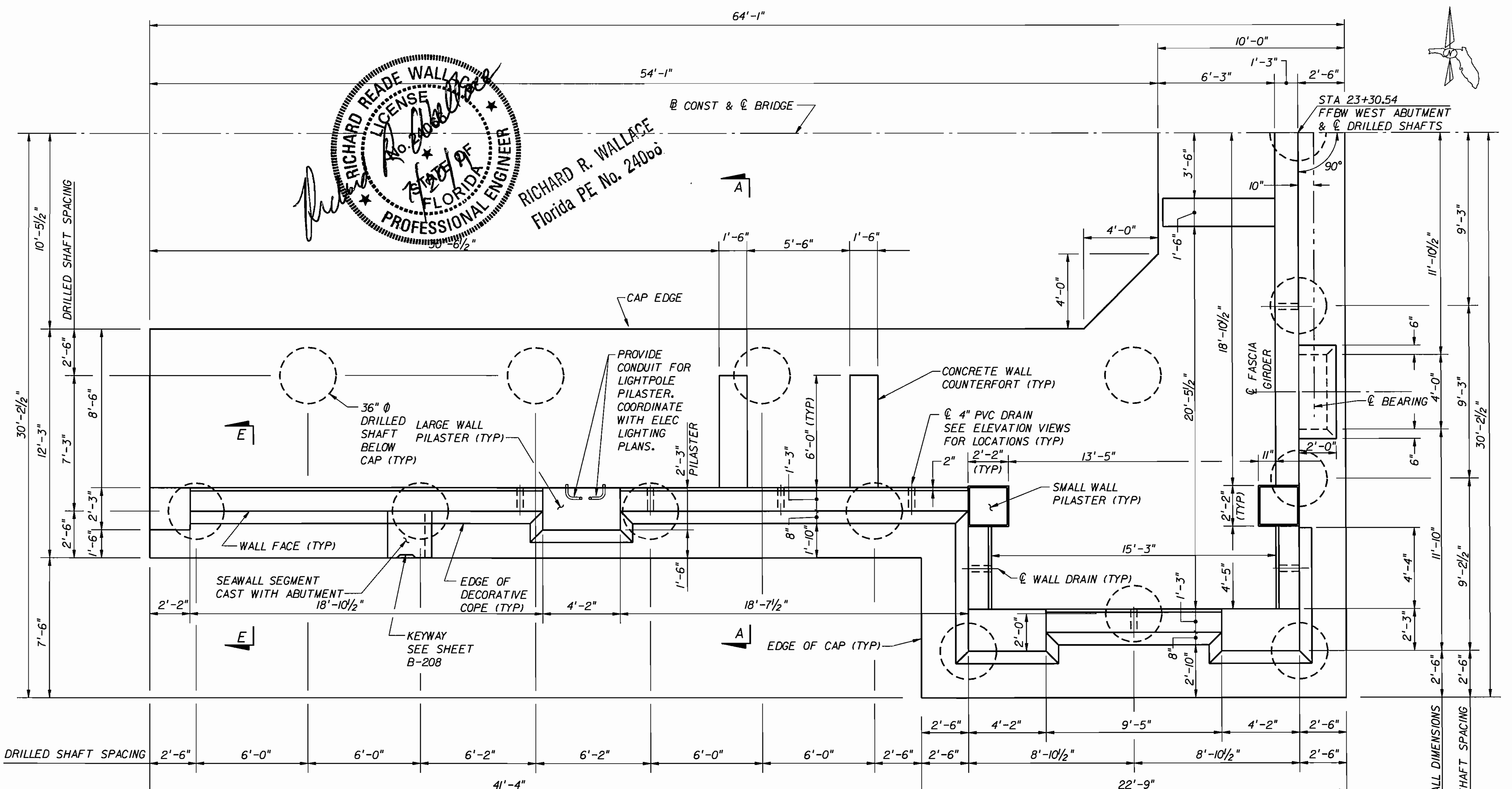
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: BASCULE PIER NEW FOOTING REINFORCING STEEL 6 OF 6

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. BP53

7/20/2011 9:43:49 AM $FILEABBREV$



NOTES:

1. WORK THIS SHEET WITH SHEETS B-31, B-34, B-36 & B-185 FOR SEAWALL SEGMENT DETAILS.
2. SEE SHEET B-36 FOR SECTION A-A.
3. SEE SHEET B-37 FOR SECTION E-E.
4. COST OF ALL LABOR AND MATERIALS NECESSARY TO INSTALL WALL DRAINS WILL BE INCLUDED WITH UNIT PRICE FOR CONCRETE CLASS V (SUBSTRUCTURE).

HALF PLAN--SYMMETRICAL ABOUT ℄ BRIDGE (AB)

AS-BUILT
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.
IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.

ESTIMATED QUANTITIES FOR WEST ABUTMENT

ITEM	UNIT	QUANTITY
Class V Concrete (Substructure)	C.Y.	197.8
Class V Concrete (Microsilica-Substructure)	C.Y.	141.6
Reinforcing Steel (Substructure)	LB.	72,151
Class III Concrete (Seal)	C.Y.	197.4

BRIDGE NO. 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	(AB) AS-BUILT			

	NAMES	DATES
DRAWN BY	GBL	09-03
CHECKED BY	JPT	10-03
DESIGNED BY	JPT	09-03
CHECKED BY	GBL	09-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Jeffrey P. Toussant, Florida PE No. 36367

FLORIDA DEPARTMENT OF TRANSPORTATION

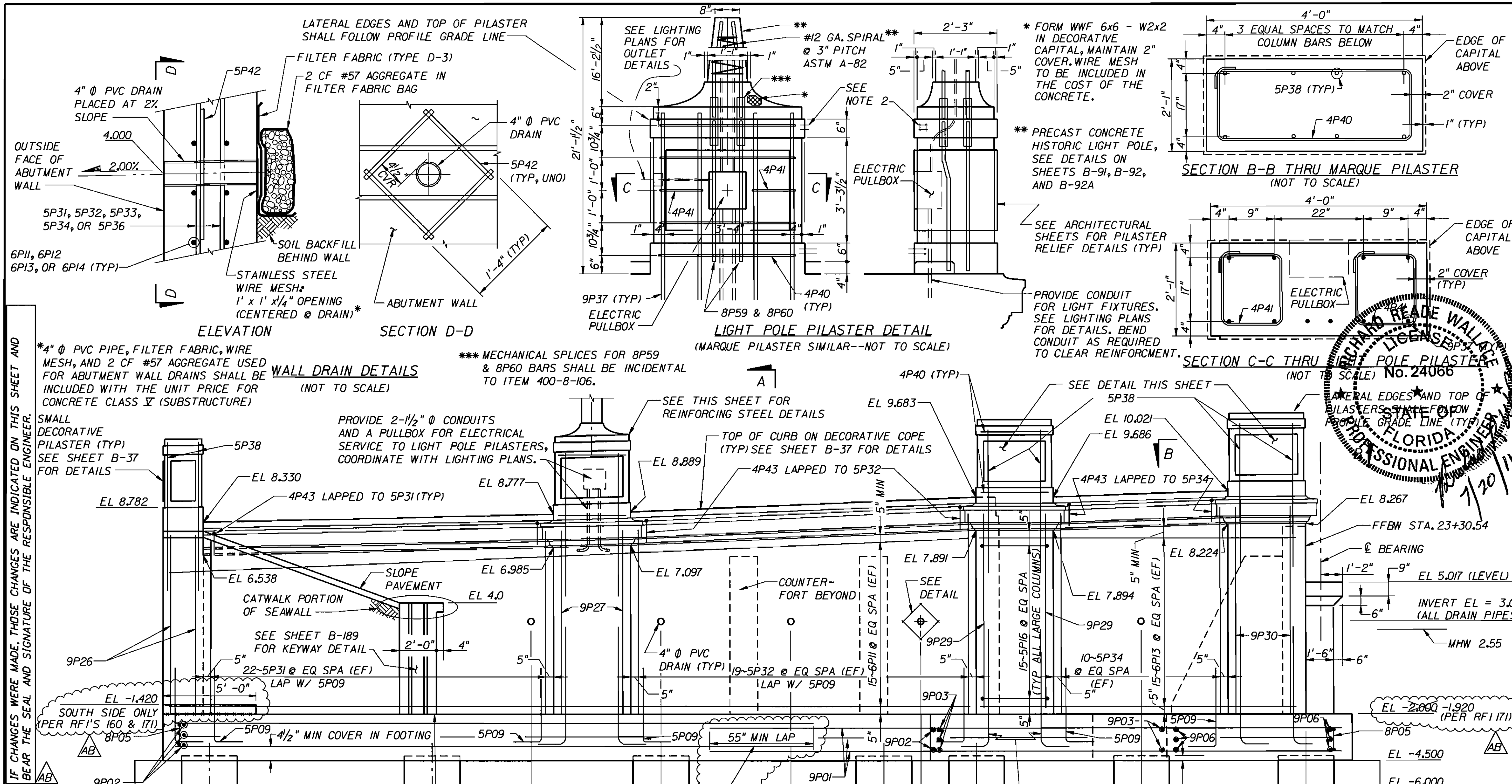
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: WEST ABUTMENT 1 - HALF PLAN

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. B-30

•DGN$SPECIFICATION•••••• •DD-MMM-YYYY HH:MM•



NOTES:

1. SEE SHEET B-36 FOR SECTION A-A.
2. POSTS AND RAILING NOT SHOWN FOR CLARITY. SEE SHEET B-36 FOR POST SPACING. SEE SHEET A-35A FOR INSERTS.
3. CONTRACTOR MAY SHIFT HORIZONTAL LOCATION OF DRAIN AS NECESSARY TO AVOID CONFLICTS WITH REINFORCING STEEL.
4. CONCRETE PILASTERS SHALL BE CAST INTEGRAL WITH WALL.
5. 2'-6" PORTION OF SEAWALL TO BE CAST AND CURED (72 HOURS) PRIOR TO BACKFILLING INTERIOR OF ABUTMENT.

2'-6" SEAWALL SEGMENT TO BE CAST WITH ABUTMENT SEE SHEETS B-34 AND B-36 FOR REINFORCEMENT DETAILS

NOTE: LOCATION OF SPLICE MAY BE FIELD ADJUSTED (PER RFI 161)

SOUTH ELEVATION
(NORTH ELEVATION MIRROR OPPOSITE)

1'-6" THICK, CLASS III CONCRETE SEAL SHALL BE USED BELOW FOOTING AS NECESSARY TO PROVIDE DRY WORK AREA. (SEE SHEET B-29A, NOTE 5)

RICHARD R. WALLACE
Florida PE No. 24066

IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.

AS-BUILT
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.

AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.

BRIDGE NO. 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	PAW	09-03
CHECKED BY	JPT	10-03
DESIGNED BY	PAW	03-03
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Jeffrey P. Toussant, Florida PE No. 36387

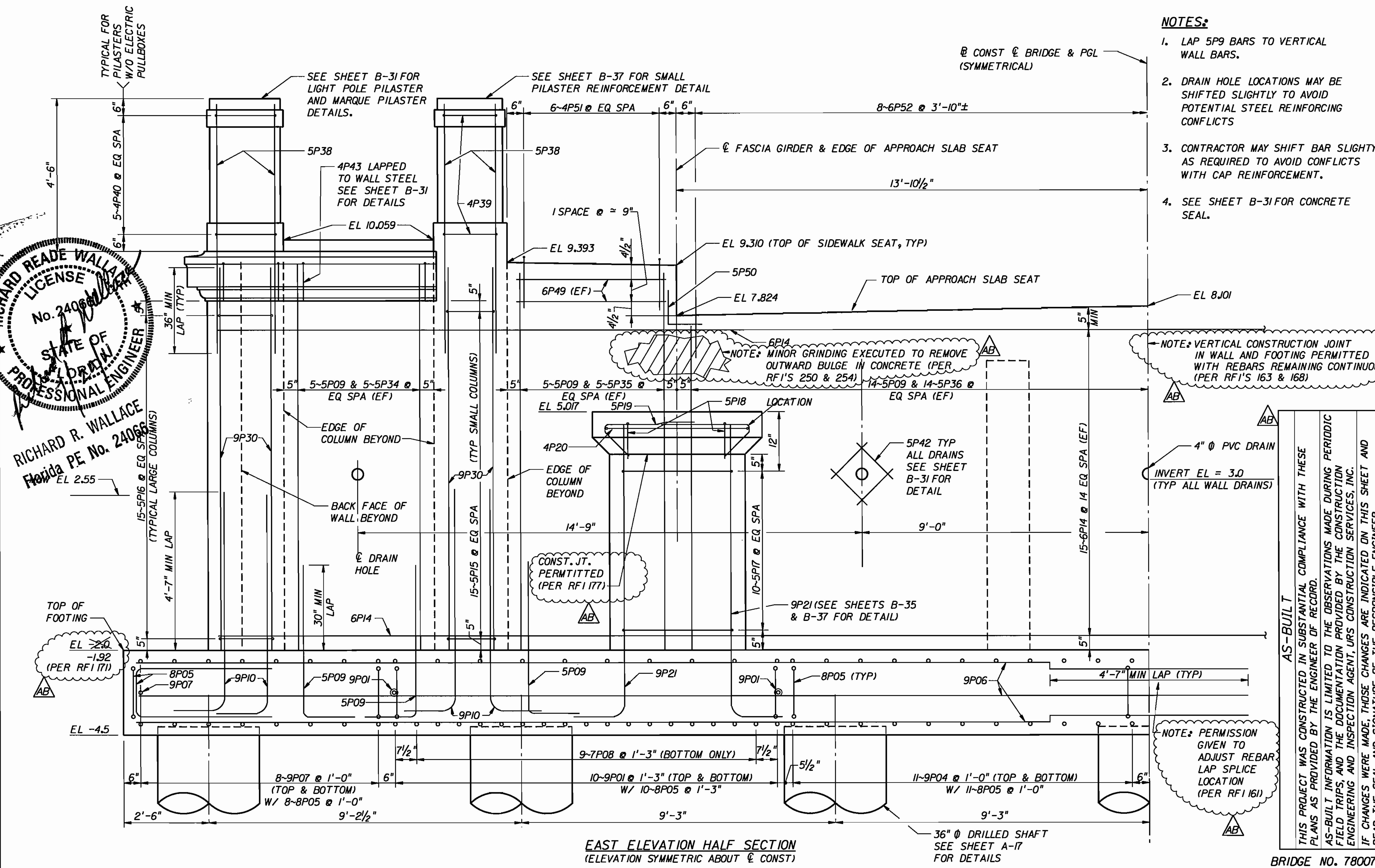
FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: WEST ABUTMENT 1 - ELEVATION (1 OF 2)

PROJECT NAME: PERMANENT BRIDGE
BRIDGE OF LIONS REHABILITATION

SHEET NO. B-31



EAST ELEVATION HALF SECTION
(ELEVATION SYMMETRIC ABOUT ℄ CONST)

NOTES:

1. LAP 5P9 BARS TO VERTICAL WALL BARS.
2. DRAIN HOLE LOCATIONS MAY BE SHIFTED SLIGHTLY TO AVOID POTENTIAL STEEL REINFORCING CONFLICTS
3. CONTRACTOR MAY SHIFT BAR SLIGHTY AS REQUIRED TO AVOID CONFLICTS WITH CAP REINFORCEMENT.
4. SEE SHEET B-31 FOR CONCRETE SEAL.

AS-BUILT

THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
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IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.

RICHARD READE WALLACE LICENSE No. 24066 STATE OF FLORIDA PROFESSIONAL ENGINEER
RICHARD R. WALLACE
Florida PE No. 24066

BRIDGE NO. 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	BMS	09-03
CHECKED BY	JPT	10-03
DESIGNED BY	PAW	03-03
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Jeffrey P. Toussant, Florida PE No. 36387

FLORIDA DEPARTMENT OF TRANSPORTATION

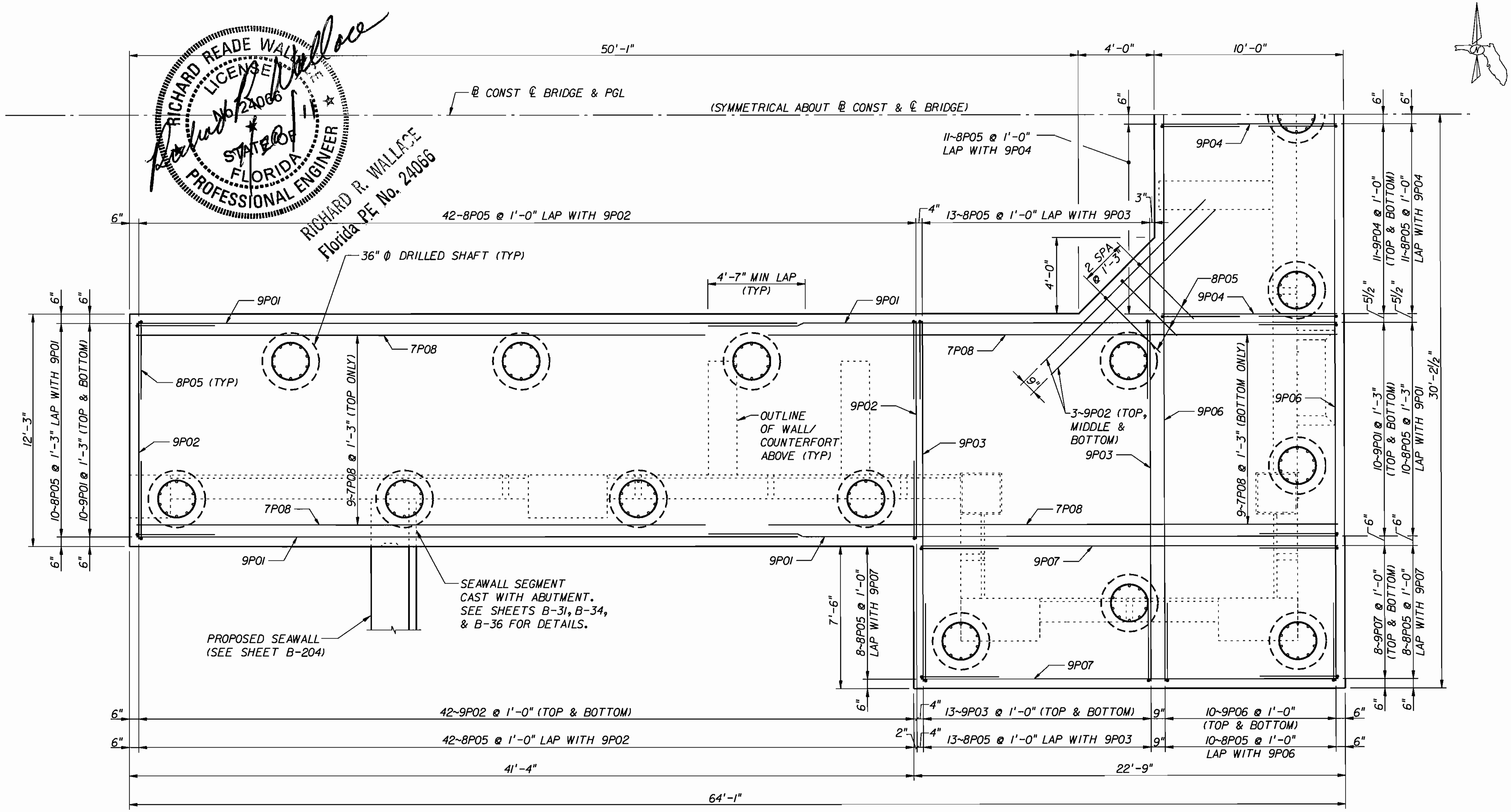
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: WEST ABUTMENT 1 - ELEVATION (2 OF 2)

PROJECT NAME: PERMANENT BRIDGE
BRIDGE OF LIONS REHABILITATION

SHEET NO. B-32

•DD-MMM-YYYY HH:MM• •DGN$SPECIFICATION•••••



NOTES:

1. CONTRACTOR MAY SHIFT BARS AS NECESSARY TO AVOID POTENTIAL REBAR CONFLICTS AT DRILLED SHAFTS.
2. CAP SHALL BE CAST MONOLITHICALLY.
3. COVER SHALL BE 4½" UNLESS NOTED OTHERWISE.

WEST ABUTMENT FOOTING REINFORCEMENT DETAILS

(SOUTH HALF SECTION SHOWN, NORTH SECTION SYMMETRIC ABOUT ℄ CONST)

AS-BUILT

THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.

AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.

AB IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.

BRIDGE NO. 780074

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	GBL	10-03
CHECKED BY	JPT	10-03
DESIGNED BY	JPT	09-03
CHECKED BY	GBL	09-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Jeffrey P. Toussant, Florida PE No. 38367

FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: WEST ABUTMENT 1 - DETAILS (1 OF 5)

PROJECT NAME: PERMANENT BRIDGE
BRIDGE OF LIONS REHABILITATION

SHEET NO. B-33

•DGN$SPECIFICATION•••••• •DD-MMM-YYYY HH:MM•

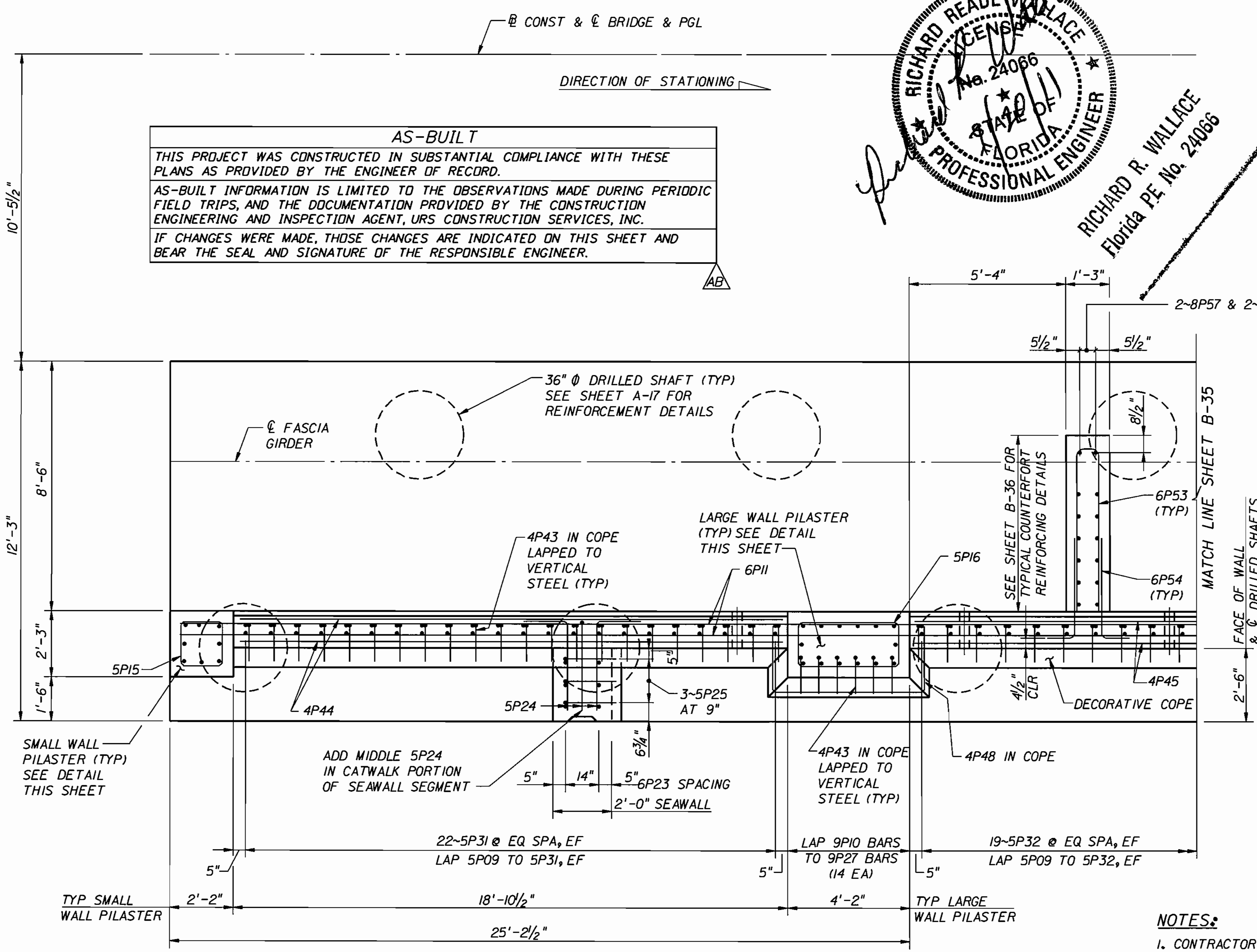
AS-BUILT

AS-BUILT
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.
IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.

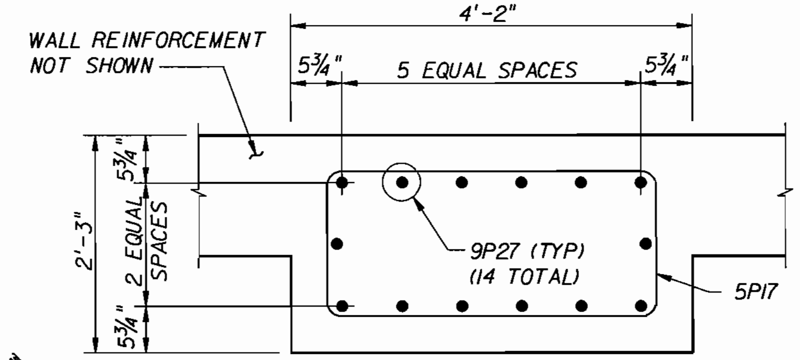
AB

RICHARD READE WALLACE LICENSE No. 24066 STATE OF FLORIDA PROFESSIONAL ENGINEER

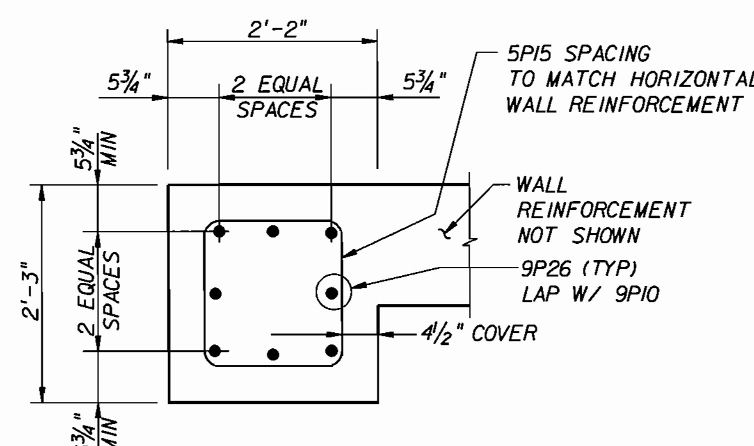
RICHARD R. WALLACE
Florida PE No. 24066



WEST ABUTMENT REINFORCEMENT DETAILS



TYPICAL LARGE WALL PILASTER REINFORCEMENT
(COLUMN SUPPORTING LIGHT POLE PILASTER SHOWN, OTHERS SIMILAR)



TYPICAL SMALL WALL PILASTER REINFORCEMENT
(COLUMN AT WEST END OF ABUTMENT SHOWN, OTHERS SIMILAR)

NOTES:

1. CONTRACTOR MAY SHIFT BARS SLIGHTLY AS REQUIRED TO AVOID CONFLICTS WITH CAP REINFORCEMENT.
2. COVER SHALL BE 4½" UNLESS OTHERWISE NOTED.

BRIDGE NO. 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	GBL	09-03
CHECKED BY	JPT	10-03
DESIGNED BY	PAW	03-03
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Jeffrey P. Toussant, Florida PE No. 36387

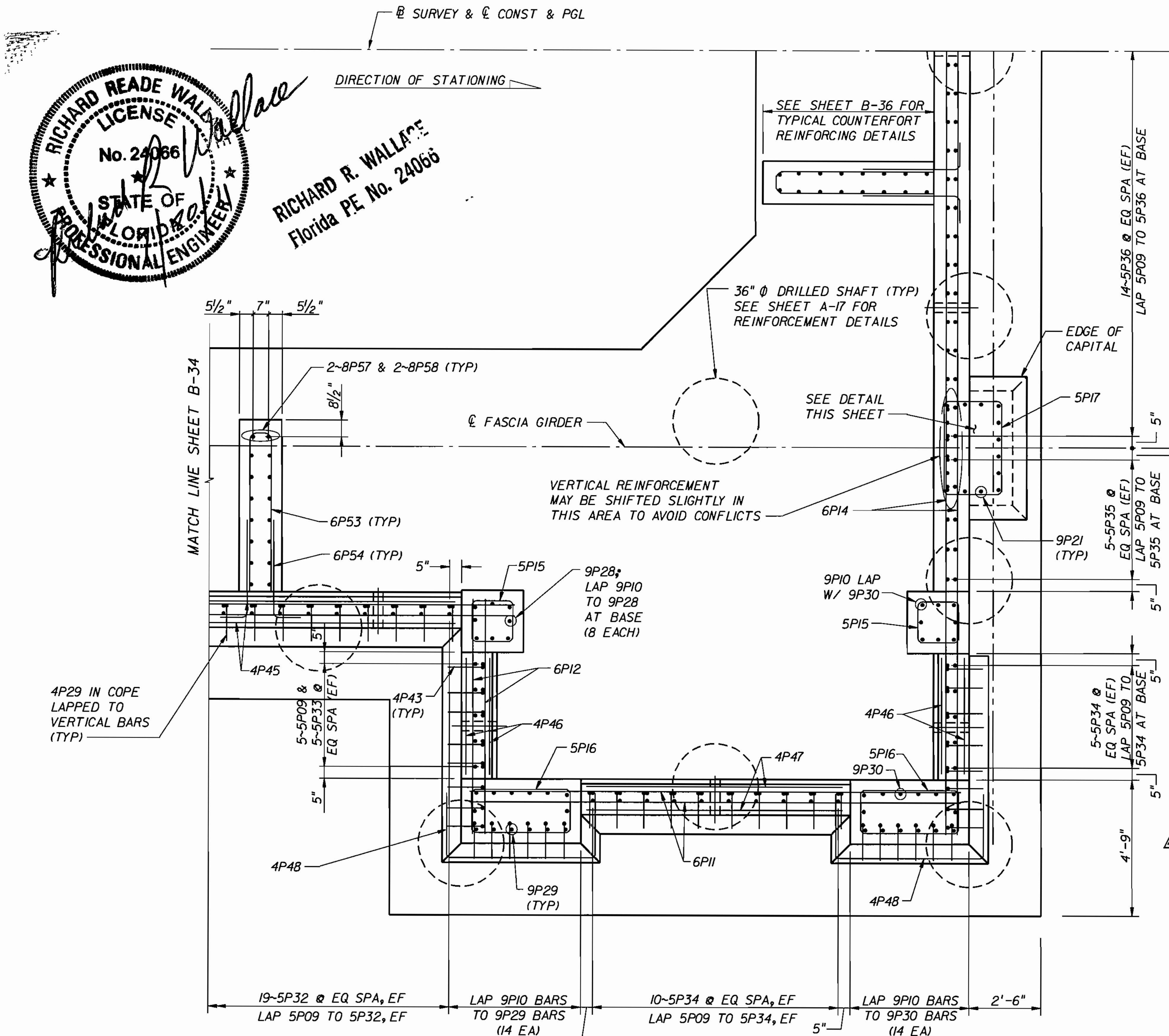
FLORIDA DEPARTMENT OF TRANSPORTATION		
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: WEST ABUTMENT 1 - DETAILS (2 OF 5)

PROJECT NAME: PERMANENT BRIDGE
BRIDGE OF LIONS REHABILITATION

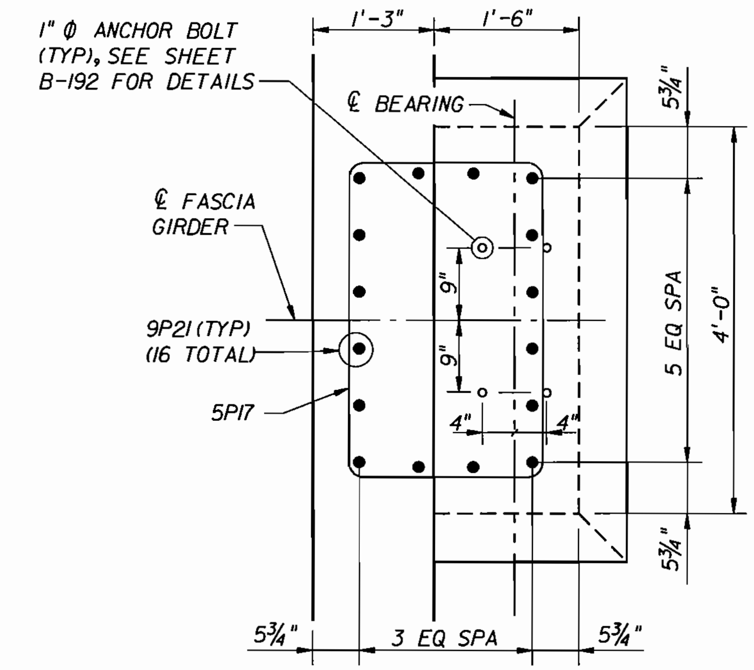
SHEET NO. B-34

*DGN$SPECIFICATION*******
DD-MMM-YYYY HH:MM



WEST ABUTMENT REINFORCEMENT DETAILS

(REINFORCEMENT SYMMETRICAL ABOUT ℄ CONST)



TYPICAL BEARING PEDESTAL REINFORCEMENT

(WALL REINFORCEMENT NOT SHOWN FOR CLARITY)

AS-BUILT

THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.

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IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.

AB

NOTES:

1. CONTRACTOR MAY SHIFT VERTICAL WALL REINFORCING BARS SLIGHTLY AS REQUIRED TO AVOID CONFLICTS WITH CAP REINFORCEMENT.
2. COVER SHALL BE 4½" UNLESS OTHERWISE NOTED.

BRIDGE NO. 780074

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT			

	NAMES	DATES
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DESIGNED BY	PAW	03-03
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Jeffrey P. Toussant, Florida PE No. 36367

FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: WEST ABUTMENT 1 - DETAILS (3 OF 5)

PROJECT NAME: PERMANENT BRIDGE
BRIDGE OF LIONS REHABILITATION

SHEET NO. B-35

AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.

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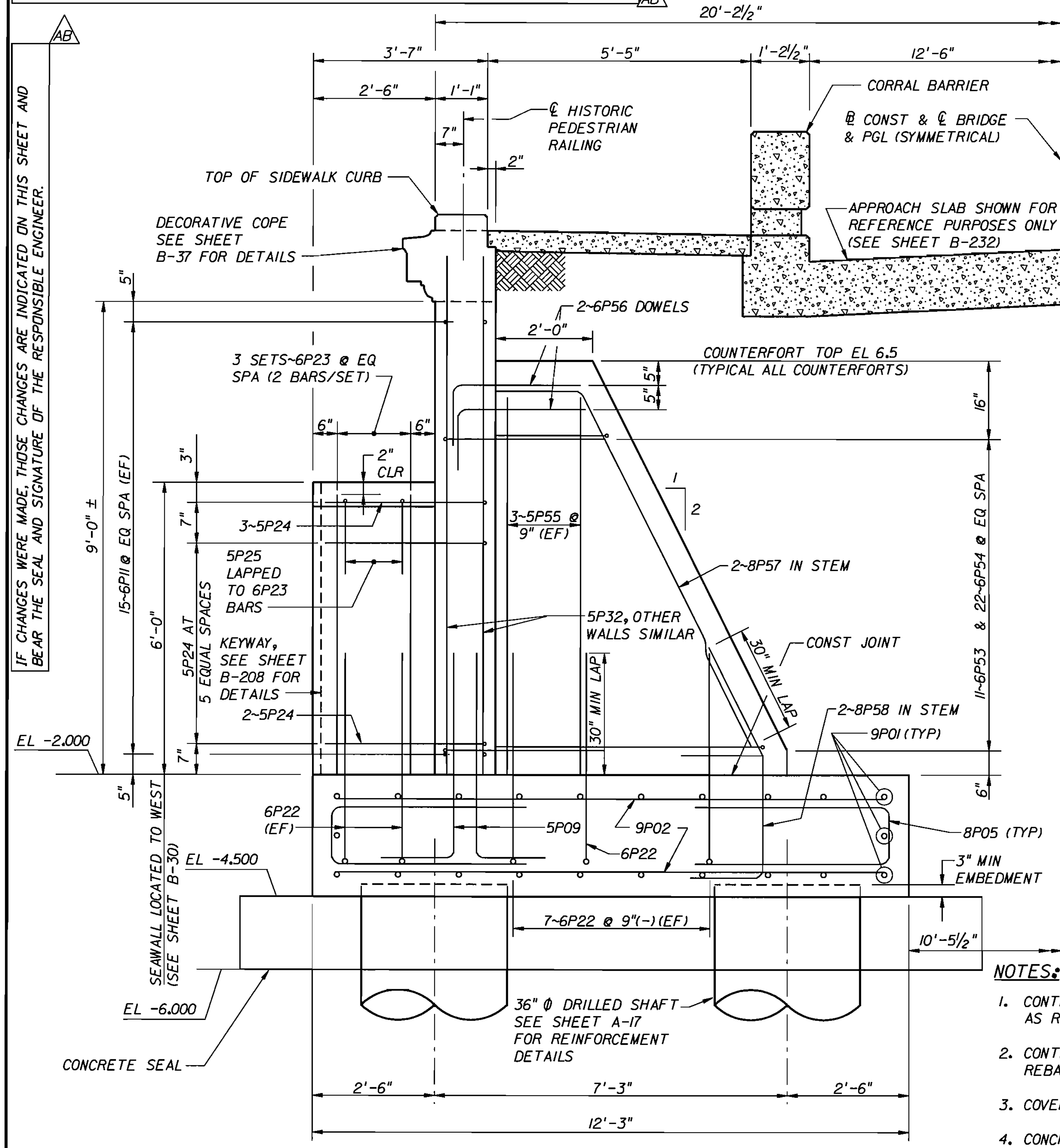
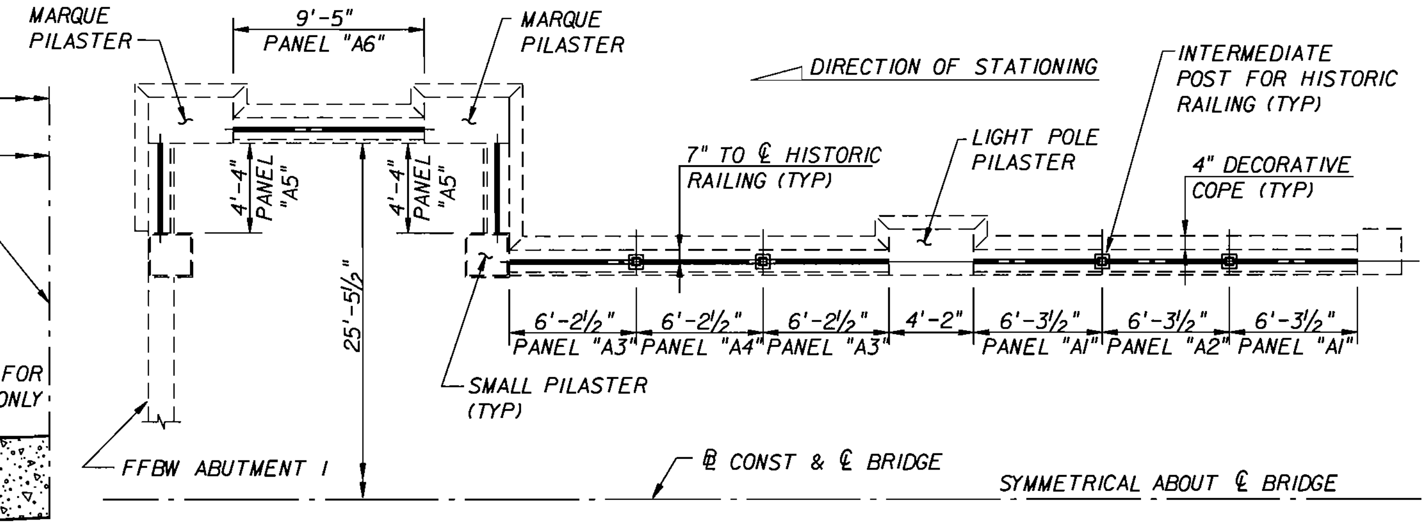


TYPICAL SECTION A-A THRU WINGWALL

(SECTIONS THRU BACKWALL SIMILAR)



WEST ABUTMENT 1 HISTORICAL PEDESTRIAN RAILING DETAIL

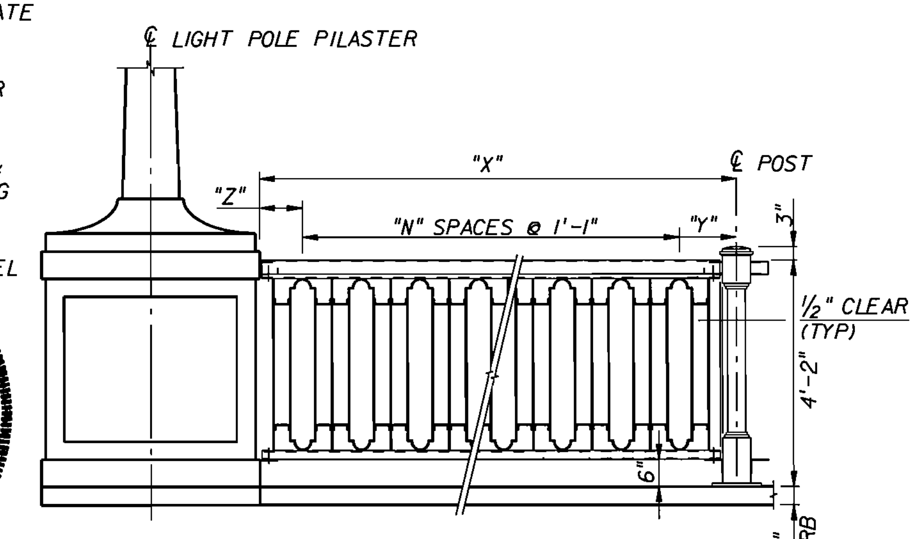
(NOT TO SCALE)

HISTORIC PEDESTRIAN RAILING DATA FOR WEST ABUTMENT							
PANEL	"X" DIMENSION	"Y"	"Z"	NO. OF PANELS	N	REMARKS	RAIL SLOPE FT/FT
A1	6'-3½"	1'-1¼"	10¼"	4	4	END @ PILASTER	0.0356
A2	6'-3½"	11¾"	N/A	2	4	INTERMEDIATE	0.0356
A3	6'-2½"	1'-0¾"	9¾"	4	4	END @ PILASTER	0.0420
A4	6'-2½"	11¼"	N/A	2	4	INTERMEDIATE	0.0420
A5	4'-4"	N/A	1'-1"	4	2	MARQUE TO PILASTER	0.0000
A6	9'-5"	N/A	11"	2	7	MARQUE TO MARQUE	0.0237

N - NUMBER OF 1'-1" SEGMENTS PER PANEL

RAILING NOTES:

1. SEE SHEET A-34 FOR INTERMEDIATE POST DETAILS.
2. SEE DETAILS ON SHEET A-35 FOR PANEL DIMENSIONS.
3. SEE DETAILS ON SHEETS A-35A & B-90 FOR ATTACHMENT OF RAILING TO CONCRETE PILASTERS.
4. RAILING TO BE PARALLEL TO TOP OF SIDEWALK CURB. INTERMEDIATE POSTS SHALL BE PLUMB. SEE NOTE 4 ON SHEET A-34.



TYPICAL ELEVATION END PANELS

(PANEL AT LIGHT POLE PILASTER AND RAILING POST SHOWN, PANEL BETWEEN TWO PILASTERS SIMILAR)

RICHARD R. WALLACE LICENSE No. 24066 STATE OF FLORIDA PROFESSIONAL ENGINEER 7/20/11

RICHARD R. WALLACE

Florida PE No. 24066

NOTES:

1. CONTRACTOR MAY SHIFT WALL AND COUNTERFORT BARS SLIGHTLY AS REQUIRED TO AVOID CONFLICTS WITH FOOTING REINFORCEMENT.
2. CONTRACTOR MAY SHIFT BARS AS NECESSARY TO AVOID POTENTIAL REBAR CONFLICTS WITH DRILLED SHAFTS.
3. COVER IS 4½" UNLESS NOTED OTHERWISE.
4. CONCRETE COUNTERFORT SHALL BE CONSOLIDATED IN 4 EQUAL LIFTS, FORMING SLOPED SURFACE IN 4 SEGMENTS. AT CONTRACTOR'S OPTION, COUNTERFORT MAY BE PLACED MONOLITHIC WITH WALL CONCRETE.

AS-BUILT

THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.

BRIDGE NO. 780074

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	GBL	10-03
CHECKED BY	JPT	10-03
DESIGNED BY	JPT	09-03
CHECKED BY	GBL	09-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD: RS&H Reynolds, Smith and Hills, Inc. 10748 Deerwood Park Blvd. South Jacksonville, Florida 32256-0597 904-256-2500 FL Cert. No. EB0005620 Jeffrey P. Toussant, Florida PE No. 38367

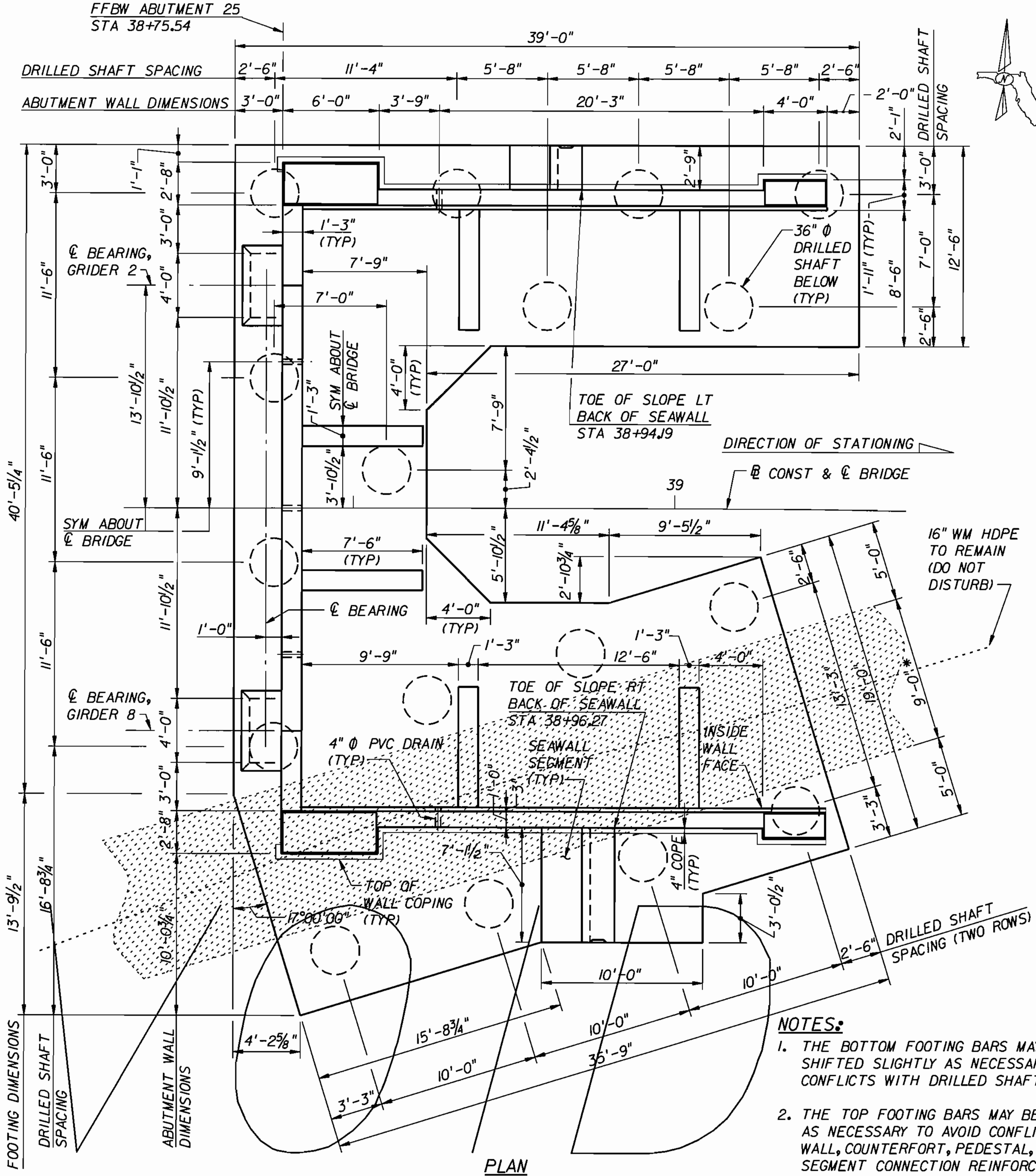
FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: WEST ABUTMENT 1 - DETAILS (4 OF 5)

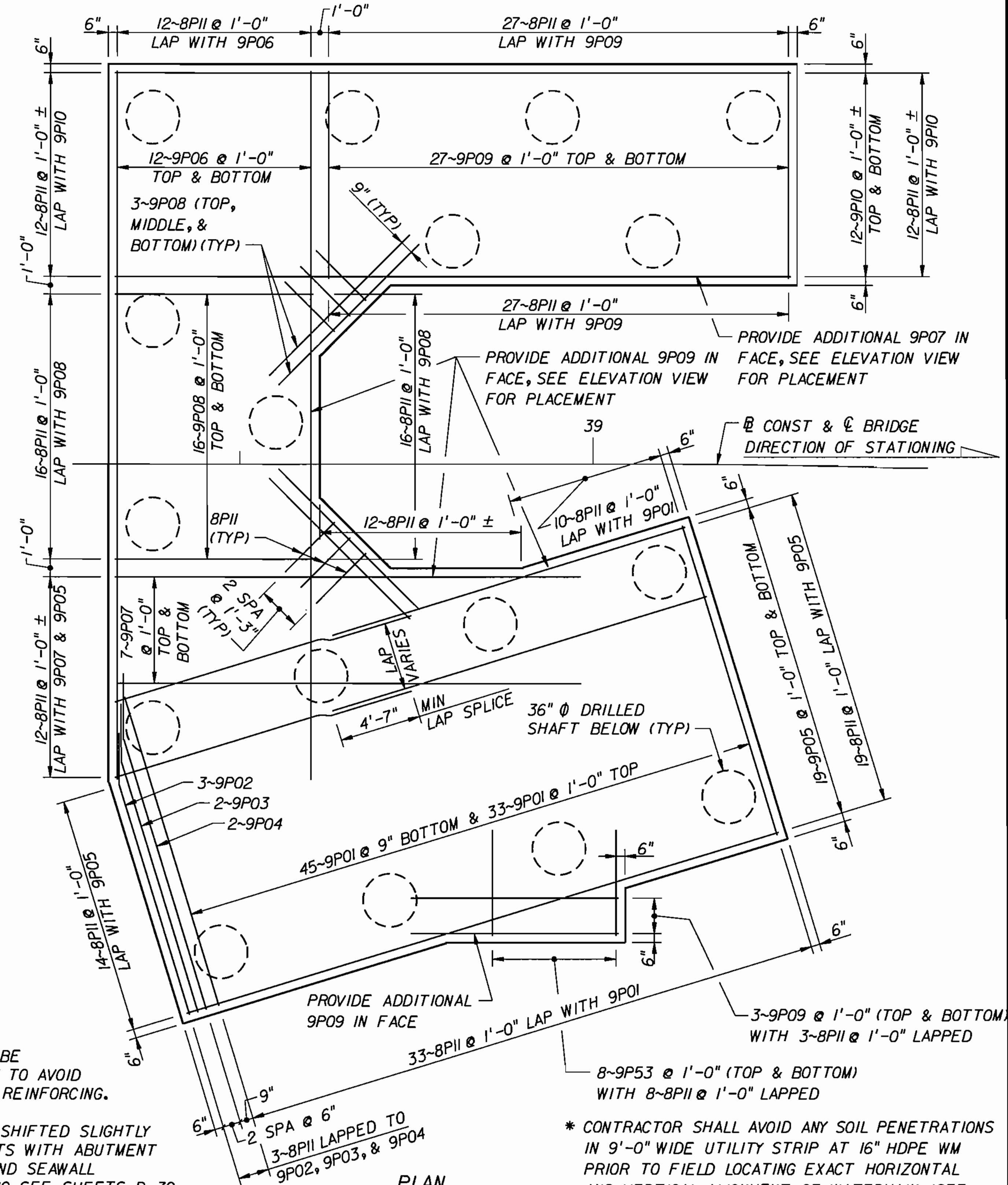
PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. B-36



PLAN

(REINFORCING STEEL OMITTED FOR CLARITY)



PLAN

TOP AND BOTTOM REINFORCING STEEL

NOTES:

1. THE BOTTOM FOOTING BARS MAY BE SHIFTED SLIGHTLY AS NECESSARY TO AVOID CONFLICTS WITH DRILLED SHAFT REINFORCING.
2. THE TOP FOOTING BARS MAY BE SHIFTED SLIGHTLY AS NECESSARY TO AVOID CONFLICTS WITH ABUTMENT WALL, COUNTERFORT, PEDESTAL, AND SEAWALL SEGMENT CONNECTION REINFORCING. SEE SHEETS B-39 THRU B-43 FOR LOCATION OF CONNECTION DOWEL BARS.
3. REFER TO SHEETS B-205 THRU B-211 FOR SEAWALL DETAILS.

* CONTRACTOR SHALL AVOID ANY SOIL PENETRATIONS IN 9'-0" WIDE UTILITY STRIP AT 16" HDPE WM PRIOR TO FIELD LOCATING EXACT HORIZONTAL AND VERTICAL ALIGNMENT OF WATERMAIN, (SEE SHEET B-40). SEE SHEET B-216 FOR PROTECTION DURING CONSTRUCTION.

BRIDGE NO. 780074

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

	NAMES	DATES
DRAWN BY	GBL	10-03
CHECKED BY	JPT	10-03
DESIGNED BY	JPT	09-03
CHECKED BY	GBL	09-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Jeffrey P. Toussant, Florida PE No. 36367

FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: EAST ABUTMENT 25 - PLAN

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. B-38

•DGN$SPECIFICATION••••••

•DD-MMM-YYYY HH:MM•

IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.

AS-BUILT

THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.

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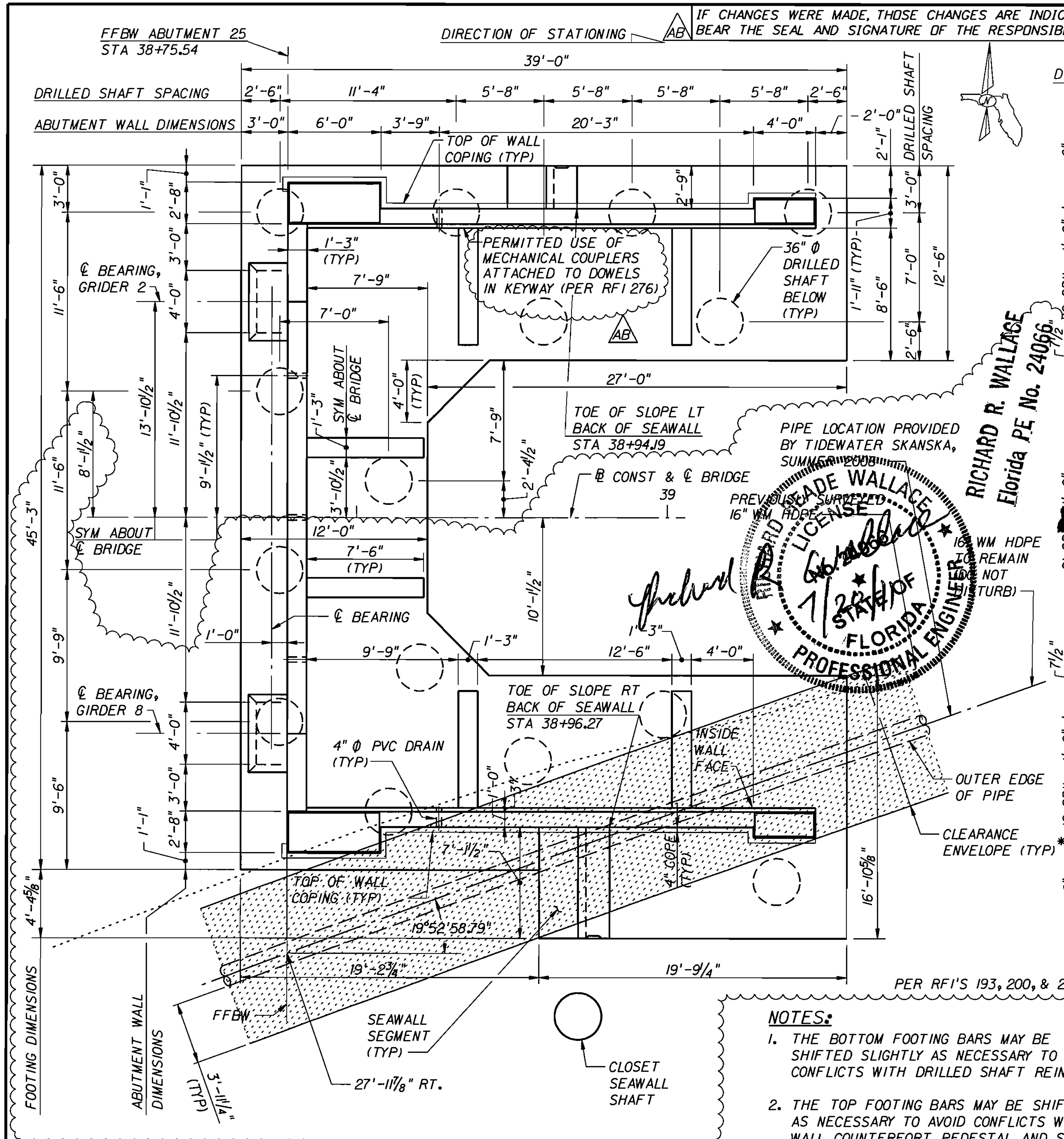
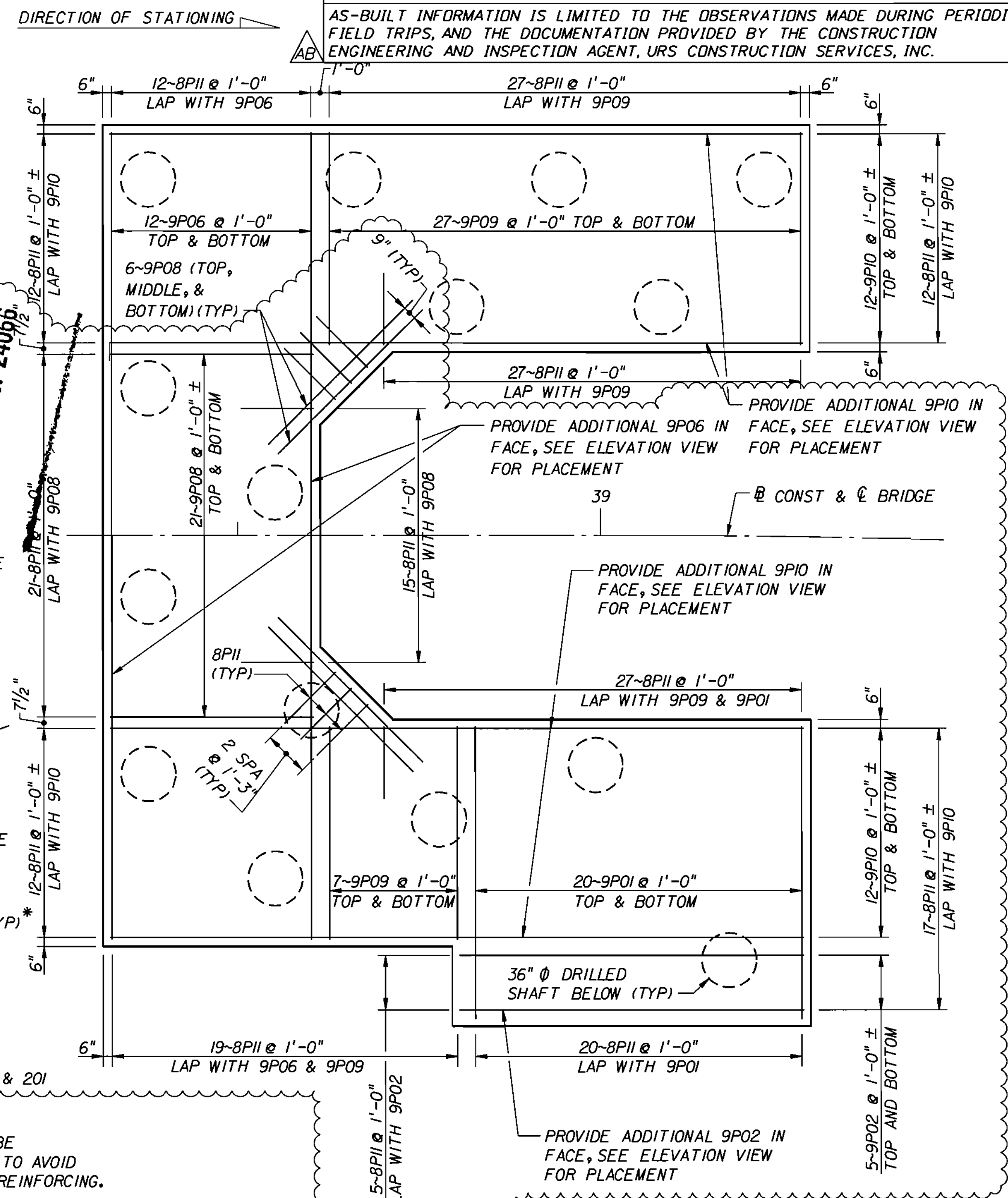


PLAN

(REINFORCING STEEL OMITTED FOR CLARITY)



PLAN

TOP AND BOTTOM REINFORCING STEEL

NOTES:

1. THE BOTTOM FOOTING BARS MAY BE SHIFTED SLIGHTLY AS NECESSARY TO AVOID CONFLICTS WITH DRILLED SHAFT REINFORCING.
2. THE TOP FOOTING BARS MAY BE SHIFTED SLIGHTLY AS NECESSARY TO AVOID CONFLICTS WITH ABUTMENT WALL, COUNTERFORT, PEDESTAL, AND SEAWALL SEGMENT CONNECTION REINFORCING. SEE SHEETS B-39 THRU B-43 FOR LOCATION OF CONNECTION DOWEL BARS.
3. REFER TO SHEETS B-205 THRU B-211 FOR SEAWALL DETAILS.

* CONTRACTOR SHALL AVOID ANY SOIL PENETRATIONS IN 9'-2" CLEARANCE ENVELOPE AT 16" HDPE WM PRIOR TO FIELD LOCATING EXACT HORIZONTAL AND VERTICAL ALIGNMENT OF WATERMAIN, (SEE SHEET B-40). SEE SHEET B-216 FOR PROTECTION DURING CONSTRUCTION.

BRIDGE NO. 780074

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
12-04-06	BMS	6 MODIFIED SHAFTS AND FOOTING			
7-20-11	RRW	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	GBL	10-03
CHECKED BY	JPT	10-03
DESIGNED BY	JPT	09-03
CHECKED BY	GBL	09-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Bryan M. Sturm, Florida PE No. 64086

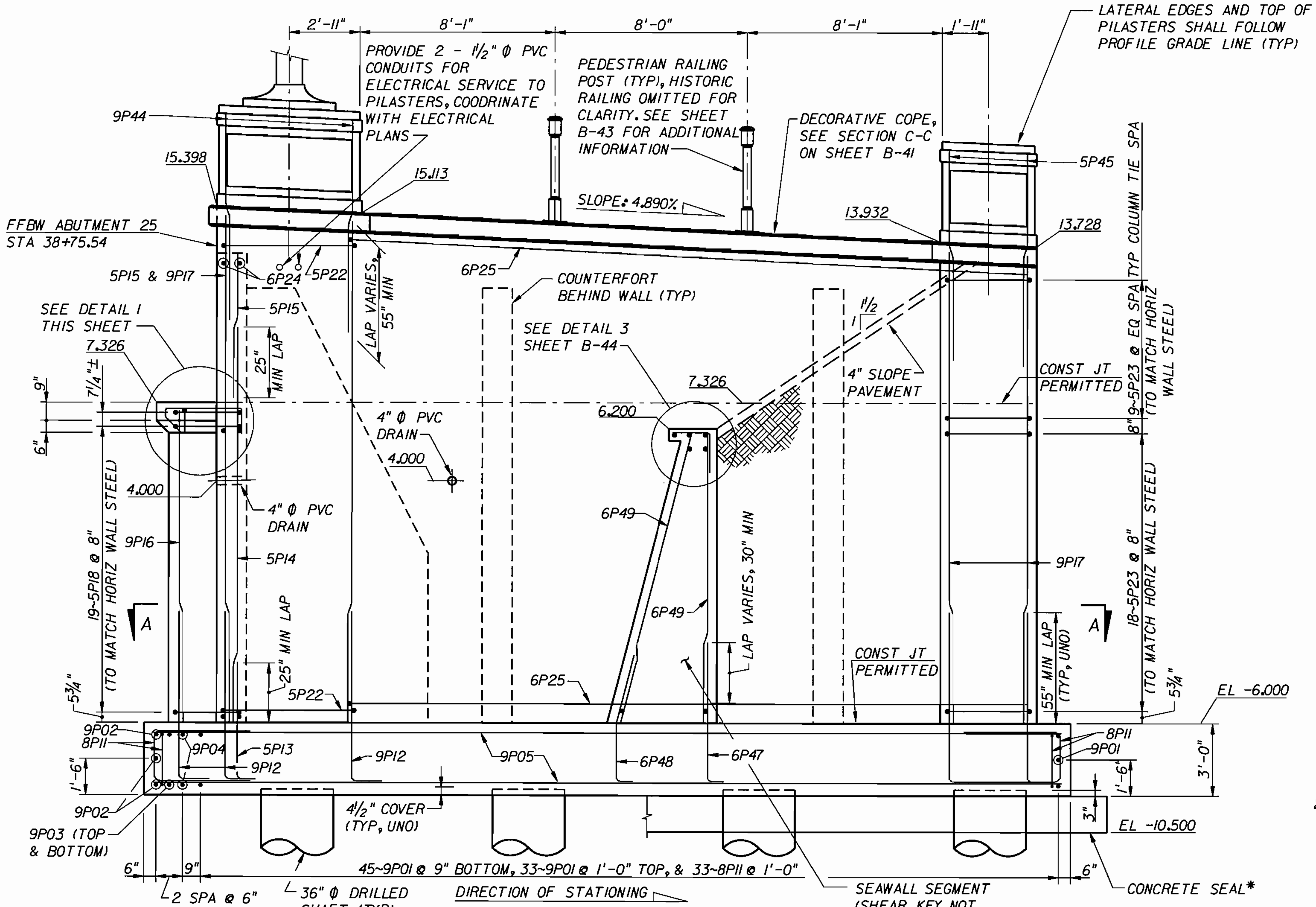
FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

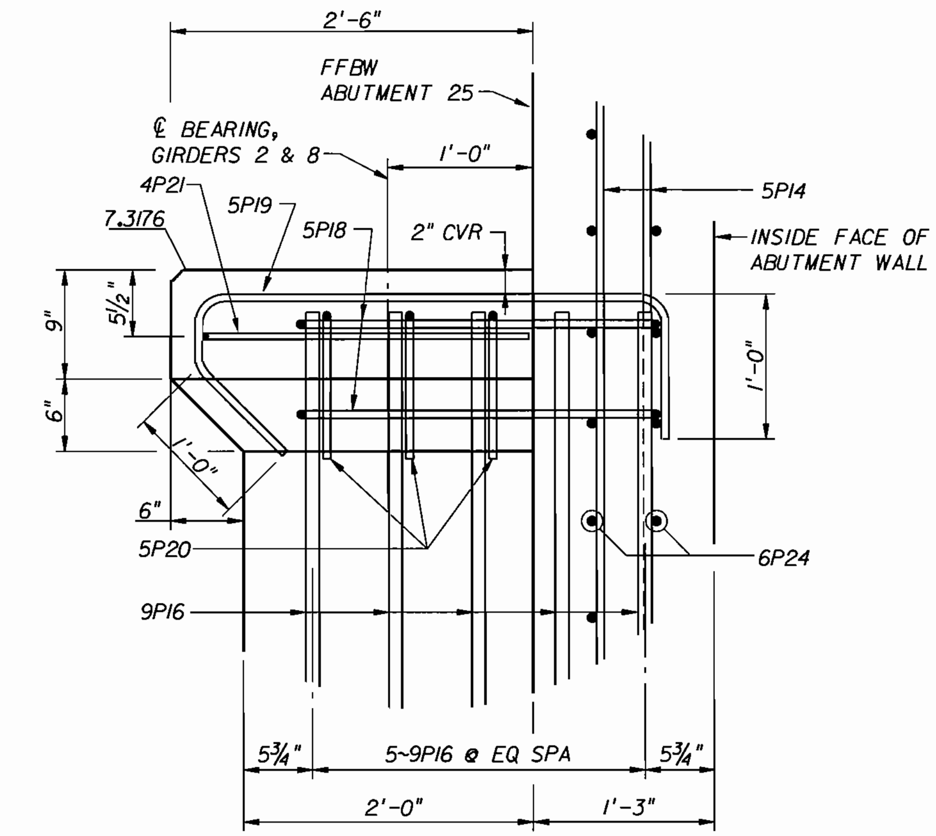
SHEET TITLE: EAST ABUTMENT 25 - PLAN

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

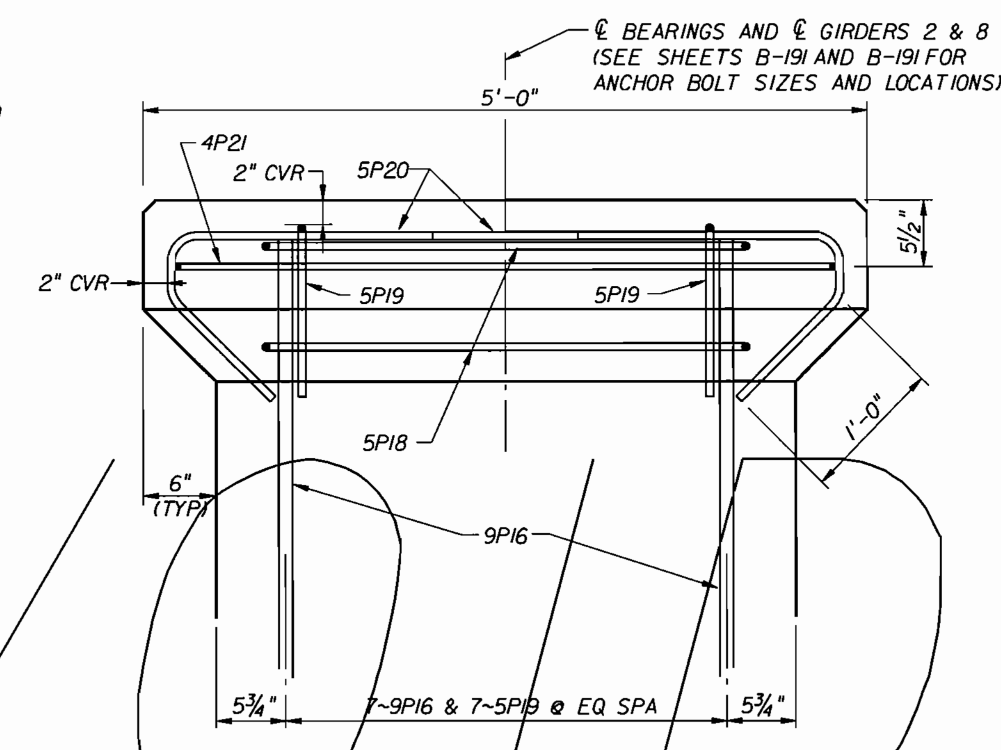
SHEET NO. B-38



SOUTH ELEVATION
(VIEW LOOKING NORTH)
(DRILLED SHAFT, COUNTERFORT, AND ABUTMENT WALL REINFORCEMENT OMITTED FOR CLARITY)



DETAIL 1, BEARING PEDESTAL CAPITAL
(NOT TO SCALE)



DETAIL 2, BEARING PEDESTAL CAPITAL
(NOT TO SCALE)

NOTES:

1. SEE SHEET B-42 FOR SECTION A-A.
2. SEE SHEET B-44 FOR PILASTER REINFORCEMENT DETAILS.
3. SEE SHEET A-17 FOR REINFORCEMENT LAYOUT AND CONNECTION DETAILS IN DRILLED SHAFTS.
4. BOTTOM FOOTING BARS MAY BE SHIFTED SLIGHTLY AS NECESSARY TO AVOID CONFLICTS WITH DRILLED SHAFT BARS.
5. TOP FOOTING BARS MAY BE SHIFTED SLIGHTLY AS NECESSARY TO AVOID CONFLICTS WITH ABUTMENT WALL, COUNTERFORT, PEDESTAL, AND SEAWALL SEGMENT CONNECTION REINFORCING.

ESTIMATED QUANTITIES FOR EAST ABUTMENT

ITEM	UNIT	QUANTITY
Class V Concrete (Substructure)	C.Y.	161.6
Class V Concrete (Microsilica-Substructure)	C.Y.	175.0
Reinforcing Steel (Substructure)	LB.	66,255
Class III Concrete (Seal)	C.Y.	299.6

* 1'-6" MINIMUM THICKNESS CLASS III CONCRETE SEAL SHALL BE USED BELOW FOOTING AS NECESSARY TO PROVIDE DRY WORK AREA.

BRIDGE NO. 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

	NAMES	DATES
DRAWN BY	GBL	05-03
CHECKED BY	JPT	10-03
DESIGNED BY	PAW	02-03
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Jeffrey P. Toussant, Florida PE No. 36367

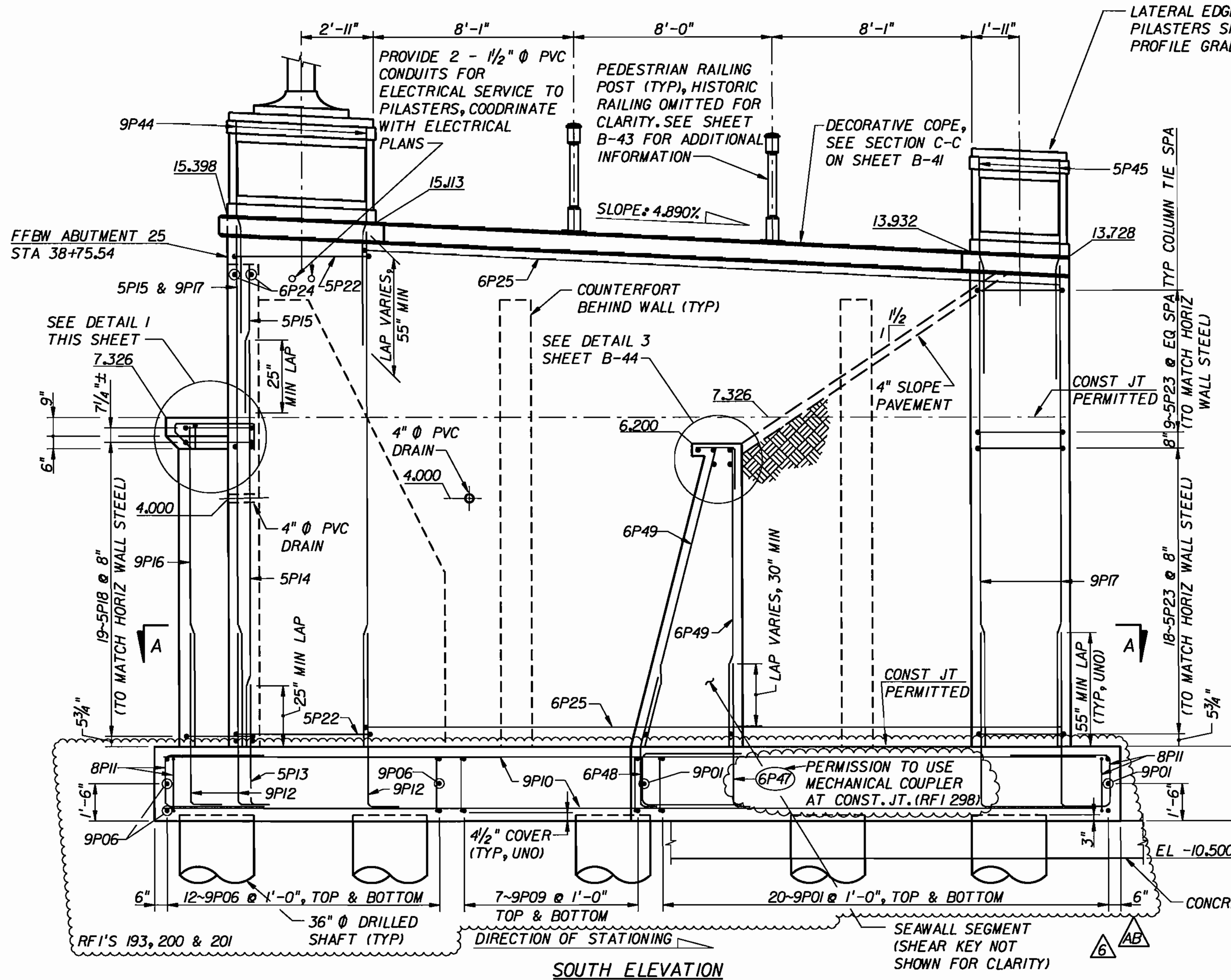
FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: EAST ABUTMENT 25 - ELEVATION (1 OF 3)

PROJECT NAME: PERMANENT BRIDGE
BRIDGE OF LIONS REHABILITATION

SHEET NO. B-39



SOUTH ELEVATION

(VIEW LOOKING NORTH)

(DRILLED SHAFT, COUNTERFORT, AND ABUTMENT WALL REINFORCEMENT OMITTED FOR CLARITY)

AS-BUILT

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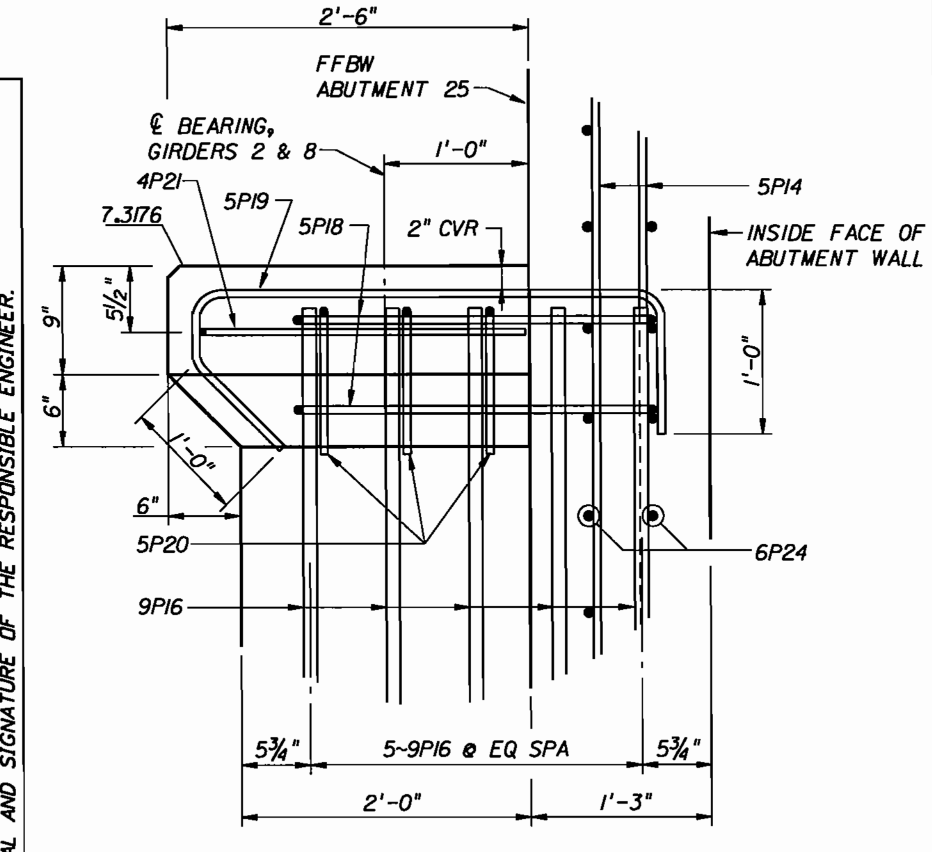
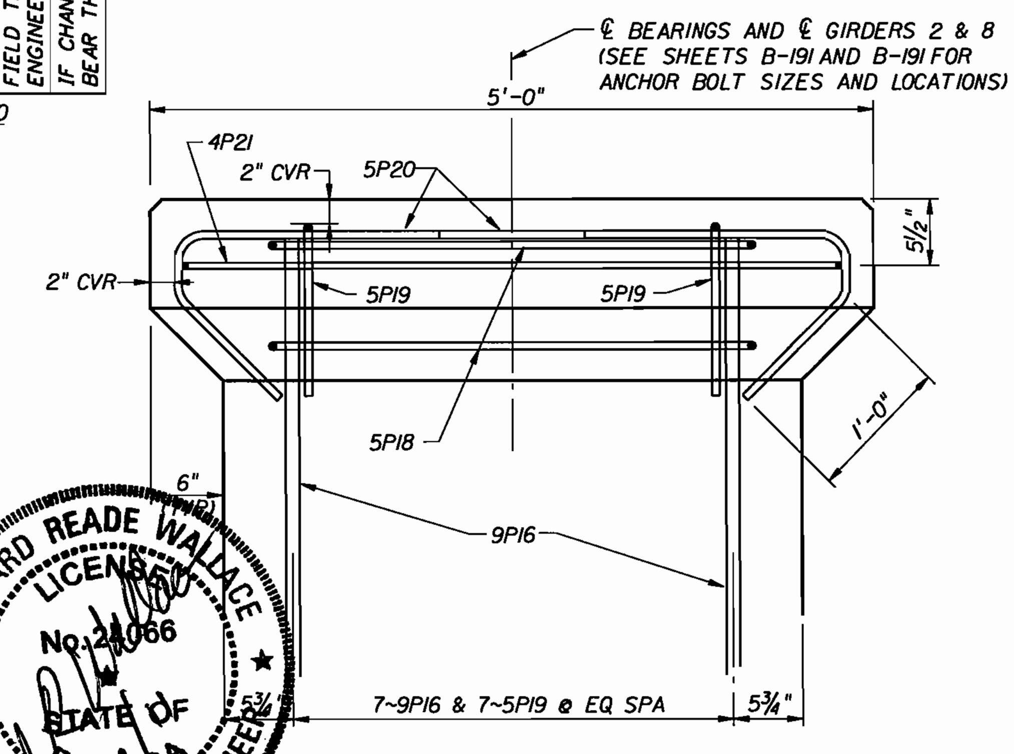


DETAIL 1, BEARING PEDESTAL CAPITAL

(NOT TO SCALE)



DETAIL 2, BEARING PEDESTAL CAPITAL

(NOT TO SCALE)

RICHARD READE WALLACE
LICENSE
No. 24066
STATE OF FLORIDA
PROFESSIONAL ENGINEER

RICHARD R. WALLACE
Florida PE No. 24066

NOTES:

1. SEE SHEET B-42 FOR SECTION A-A.
2. SEE SHEET B-44 FOR PILASTER REINFORCEMENT DETAILS.
3. SEE SHEET A-17 FOR REINFORCEMENT LAYOUT AND CONNECTION DETAILS IN DRILLED SHAFTS.
4. BOTTOM FOOTING BARS MAY BE SHIFTED SLIGHTLY AS NECESSARY TO AVOID CONFLICTS WITH DRILLED SHAFT BARS.
5. TOP FOOTING BARS MAY BE SHIFTED SLIGHTLY AS NECESSARY TO AVOID CONFLICTS WITH ABUTMENT WALL, COUNTERFORT, PEDESTAL, AND SEAWALL SEGMENT CONNECTION REINFORCING.

ESTIMATED QUANTITIES FOR EAST ABUTMENT

ITEM	UNIT	QUANTITY
Class V Concrete (Substructure)	C.Y.	146.8
Class V Concrete (Microsilica-Substructure)	C.Y.	175.0
Reinforcing Steel (Substructure)	LB.	61,318
Class III Concrete (Seal)	C.Y.	299.6

AB 6

* 1'-6" MINIMUM THICKNESS CLASS III CONCRETE SEAL SHALL BE USED BELOW FOOTING AS NECESSARY TO PROVIDE DRY WORK AREA.

BRIDGE NO. 780074

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
12-04-06	BMS	6 MODIFIED SHAFTS AND FOOTING			
7-20-11	RRW	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	GBL	05-03
CHECKED BY	JPT	10-03
DESIGNED BY	PAW	02-03
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
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Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Bryan M. Sturm, Florida PE No. 64066

FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

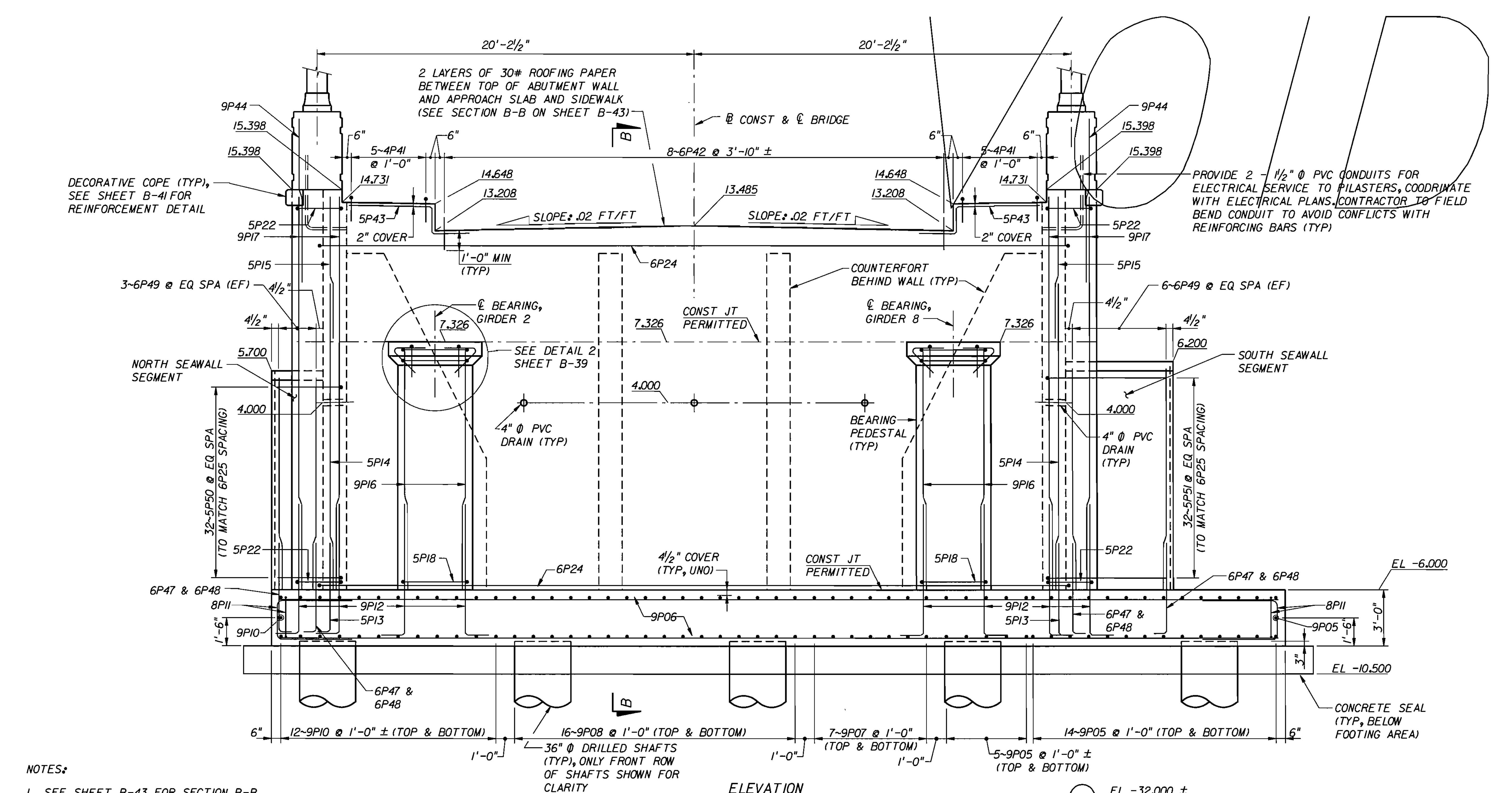
SHEET TITLE: EAST ABUTMENT 25 - ELEVATION (1 OF 3)

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. B-39

•DGN$SPECIFICATION••••••

•DD-MMM-YYYY HH:MM•



ELEVATION

(VIEW LOOKING EAST, DIRECTION OF STATIONING)
(DRILLED SHAFT, COUNTERFORT, AND ABUTMENT WALL REINFORCEMENT OMITTED FOR CLARITY)

NOTES:

1. SEE SHEET B-43 FOR SECTION B-B.
2. SEE SHEET B-44 FOR PILASTER REINFORCEMENT DETAILS.
3. SEE SHEET A-17 FOR REINFORCEMENT LAYOUT AND CONNECTION DETAILS IN DRILLED SHAFTS.
4. BOTTOM FOOTING BARS MAY BE SHIFTED SLIGHTLY AS NECESSARY TO AVOID CONFLICTS WITH DRILLED SHAFT BARS.
5. TOP FOOTING BARS MAY BE SHIFTED SLIGHTLY AS NECESSARY TO AVOID CONFLICTS WITH ABUTMENT WALL, COUNTERFORT, PEDESTAL, AND SEAWALL SEGMENT CONNECTION REINFORCING.

* CONTRACTOR SHALL FIELD LOCATE WATERMAIN PRIOR TO DRIVING SHEET PILING OR DRILLING FOR DRILLED SHAFT INSTALLATION. (SEE SHEET B-216 FOR PROTECTION DURING CONSTRUCTION)

BRIDGE NO. 780074

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

	NAMES	DATES
DRAWN BY	GBL	05-03
CHECKED BY	JPT	10-03
DESIGNED BY	PAW	02-03
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Jeffrey P. Toussant, Florida PE No. 36367

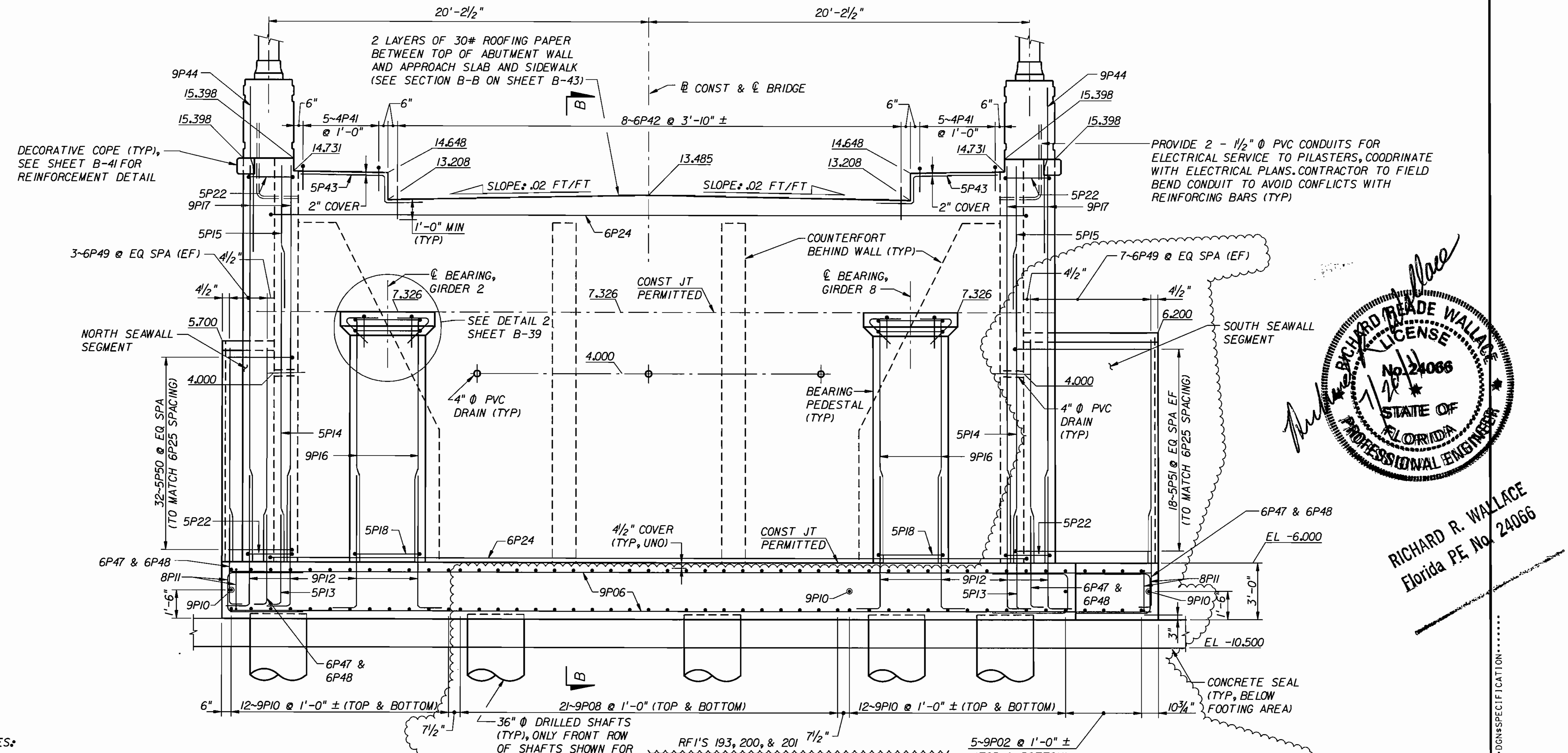
FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: EAST ABUTMENT 25 - ELEVATION (2 OF 3)

PROJECT NAME: PERMANENT BRIDGE
BRIDGE OF LIONS REHABILITATION

SHEET NO. B-40



ELEVATION
(VIEW LOOKING EAST, DIRECTION OF STATIONING)
(DRILLED SHAFT, COUNTERFORT, AND ABUTMENT WALL REINFORCEMENT OMITTED FOR CLARITY)

AB 6

NOTES:

1. SEE SHEET B-43 FOR SECTION B-B.
2. SEE SHEET B-44 FOR PILASTER REINFORCEMENT DETAILS.
3. SEE SHEET A-17 FOR REINFORCEMENT LAYOUT AND CONNECTION DETAILS IN DRILLED SHAFTS.
4. BOTTOM FOOTING BARS MAY BE SHIFTED SLIGHTLY AS NECESSARY TO AVOID CONFLICTS WITH DRILLED SHAFT BARS.
5. TOP FOOTING BARS MAY BE SHIFTED SLIGHTLY AS NECESSARY TO AVOID CONFLICTS WITH ABUTMENT WALL, COUNTERFORT, PEDESTAL, AND SEAWALL SEGMENT CONNECTION REINFORCING.

AS-BUILT
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.
IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER. AB

* CONTRACTOR SHALL FIELD LOCATE WATERMAIN PRIOR TO DRIVING SHEET PILING OR DRILLING FOR DRILLED SHAFT INSTALLATION. (SEE SHEET B-216 FOR PROTECTION DURING CONSTRUCTION)

BRIDGE NO. 780074

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
12-04-06	BMS	6 MODIFIED SHAFTS AND FOOTING, RESCALED SOUTH SEAWALL SEGMENT			
7-20-11	RRW	AB AS-BUILT (PER RFI'S 193, 200, & 201)			

	NAMES	DATES
DRAWN BY	GBL	05-03
CHECKED BY	JPT	10-03
DESIGNED BY	PAW	02-03
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Bryan M. Sturm, Florida PE No. 64066

FLORIDA DEPARTMENT OF TRANSPORTATION

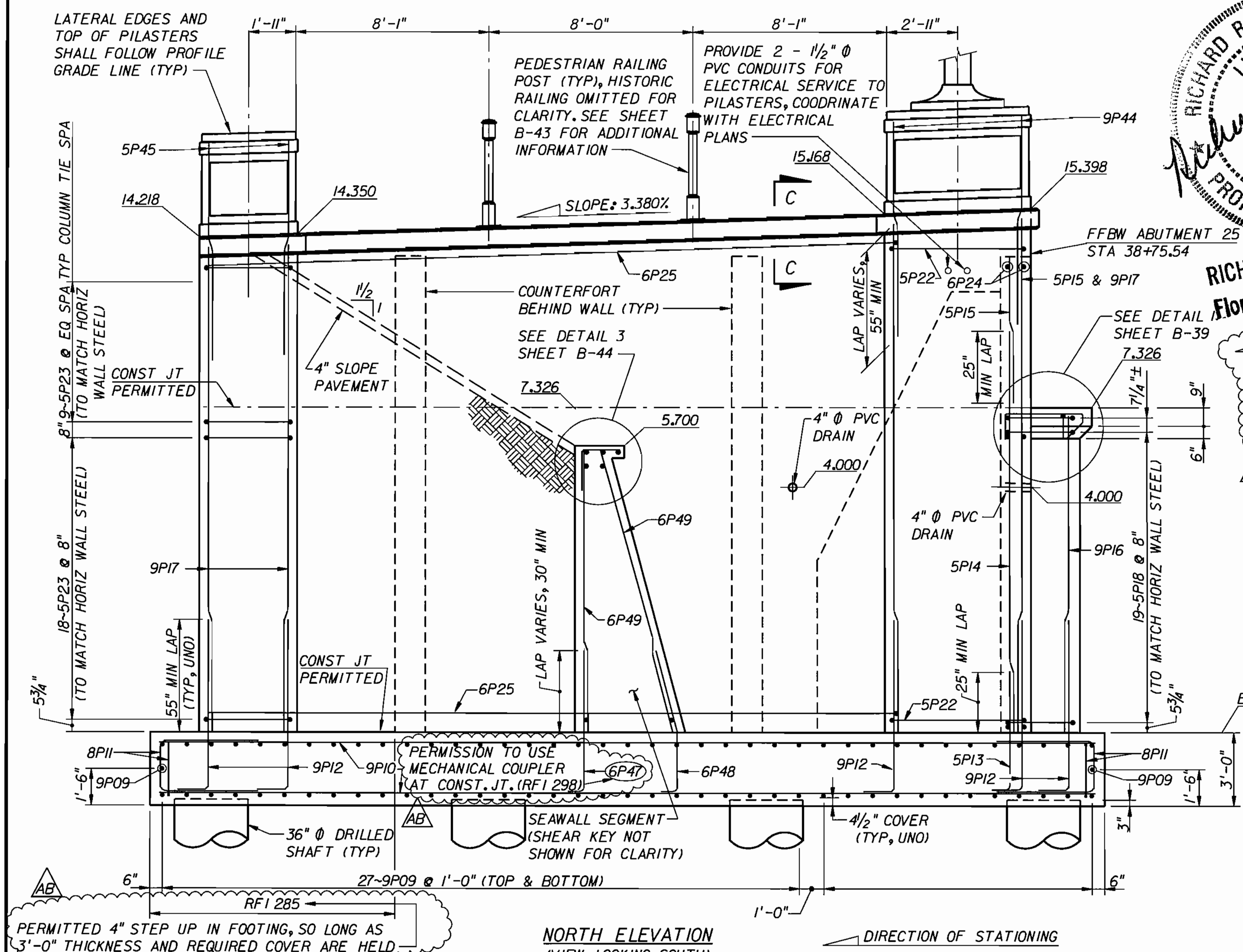
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: EAST ABUTMENT 25 - ELEVATION (2 OF 3)

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. B-40

DD-MMM-YYYY HH:MM
DGN$SPECIFICATION



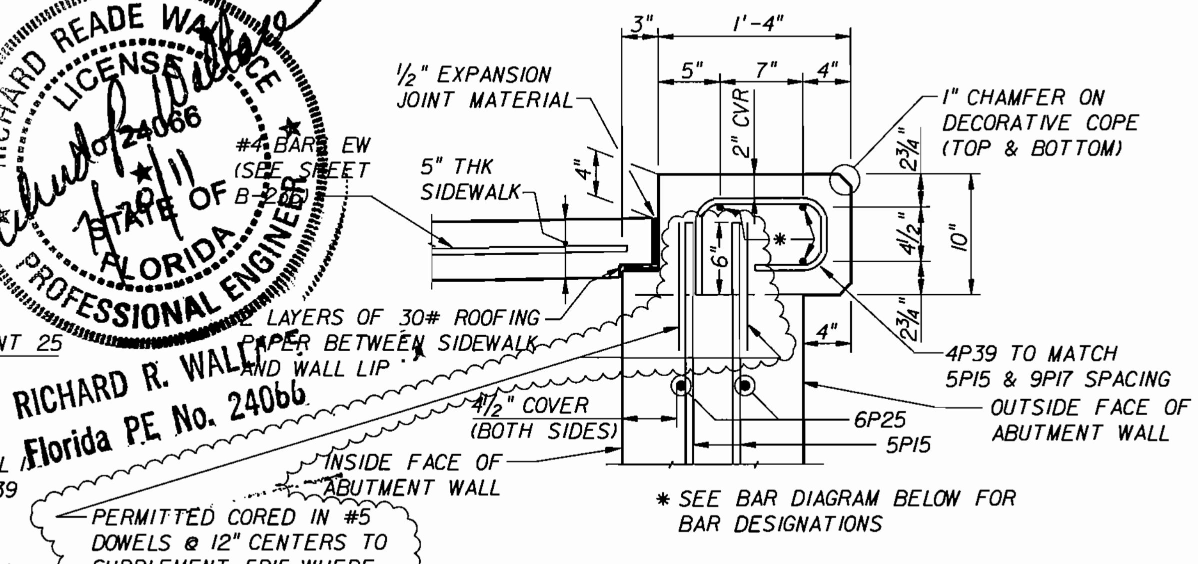
NORTH ELEVATION

(VIEW LOOKING SOUTH)

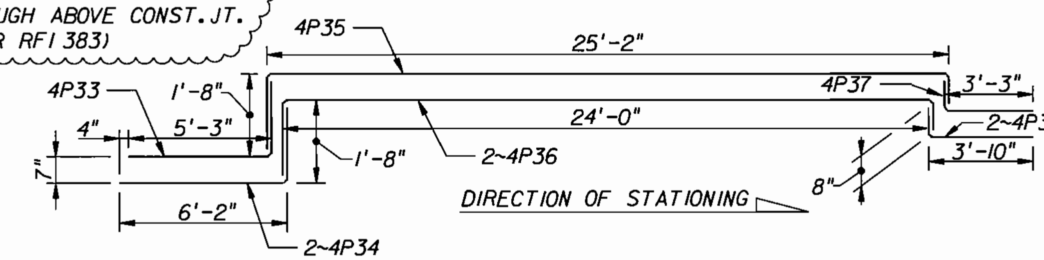
(DRILLED SHAFT, COUNTERFORT, AND ABUTMENT WALL REINFORCEMENT AND SEAL OMITTED FOR CLARITY)

RICHARD READE WALLACE LICENSE No. 24066 STATE OF FLORIDA PROFESSIONAL ENGINEER 7/20/11

RICHARD R. WALLACE Florida PE No. 24066



SECTION C-C

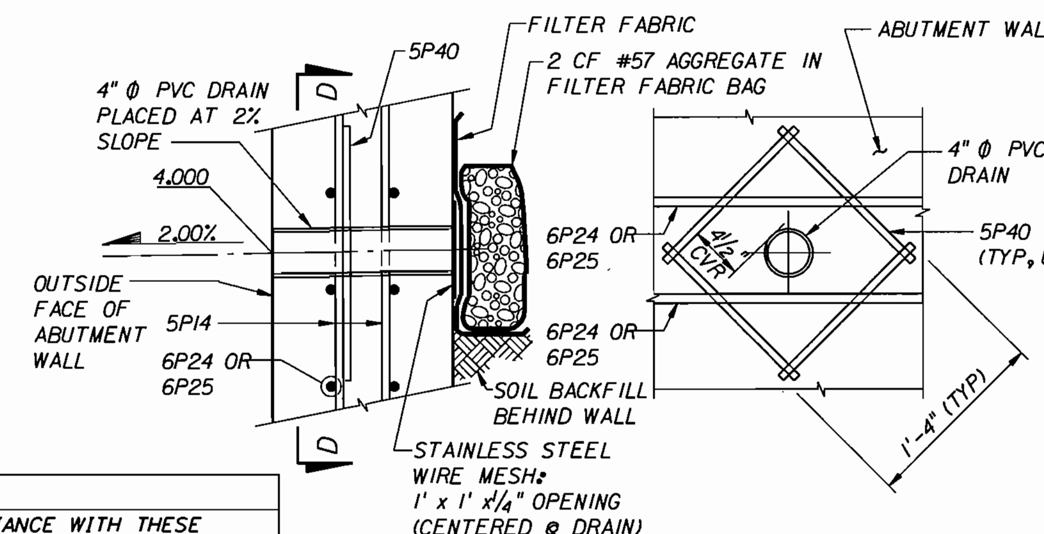


BAR DIAGRAM - PLAN VIEW

(LONGITUDINAL BARS IN DECORATIVE COPING, SOUTH COPING SHOWN, NORTH COPING SIMILAR)

DECORATIVE COPE DETAILS

(NOT TO SCALE)



ELEVATION SECTION D-D

WALL DRAIN DETAILS

(NOT TO SCALE)

NOTES:

1. SEE SHEET B-44 FOR PILASTER REINFORCEMENT DETAILS.
2. SEE SHEET A-17 FOR REINFORCEMENT LAYOUT AND CONNECTION DETAILS IN DRILLED SHAFTS.
3. BOTTOM FOOTING BARS MAY BE SHIFTED SLIGHTLY AS NECESSARY TO AVOID CONFLICTS WITH DRILLED SHAFT BARS.
4. TOP FOOTING BARS MAY BE SHIFTED SLIGHTLY AS NECESSARY TO AVOID CONFLICTS WITH ABUTMENT WALL, COUNTERFORT, PEDESTAL, AND SEAWALL SEGMENT CONNECTION REINFORCING.
5. 4" Ø PVC PIPE, FILTER FABRIC, WIRE MESH, AND 2 CF #57 AGGREGATE USED FOR ABUTMENT WALL DRAINS SHALL BE INCLUDED WITH THE UNIT PRICE FOR CONCRETE CLASS V (SUBSTRUCTURE).

AS-BUILT
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.
IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.

BRIDGE NO. 780074

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	GBL	05-03
CHECKED BY	JPT	10-03
DESIGNED BY	PAW	02-03
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Jeffrey P. Toussant, Florida PE No. 36367

FLORIDA DEPARTMENT OF TRANSPORTATION

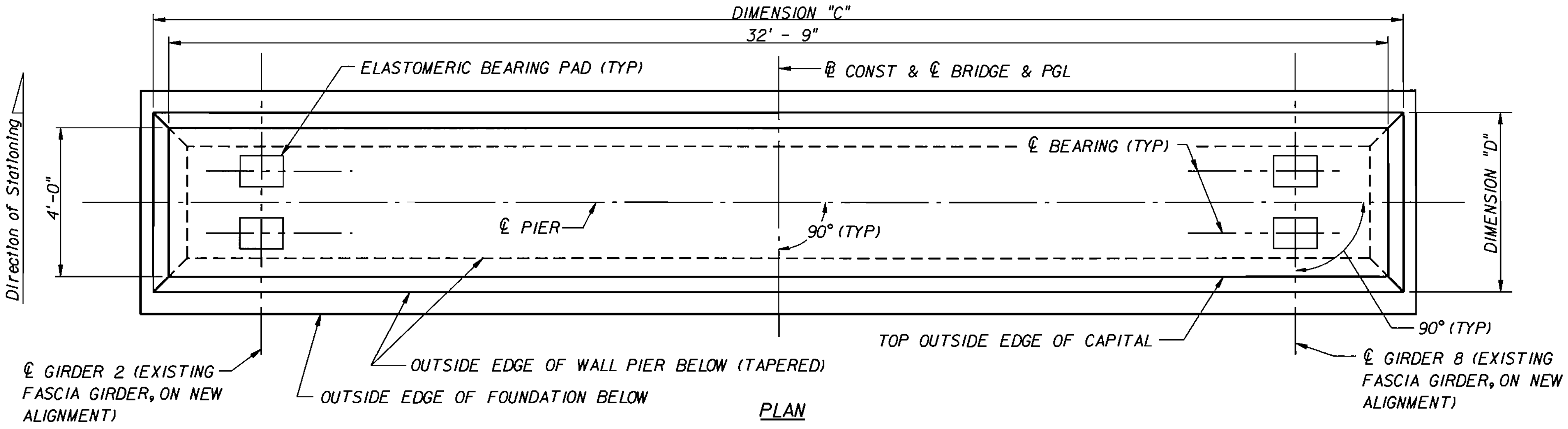
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: EAST ABUTMENT 25 - ELEVATION (3 OF 3)

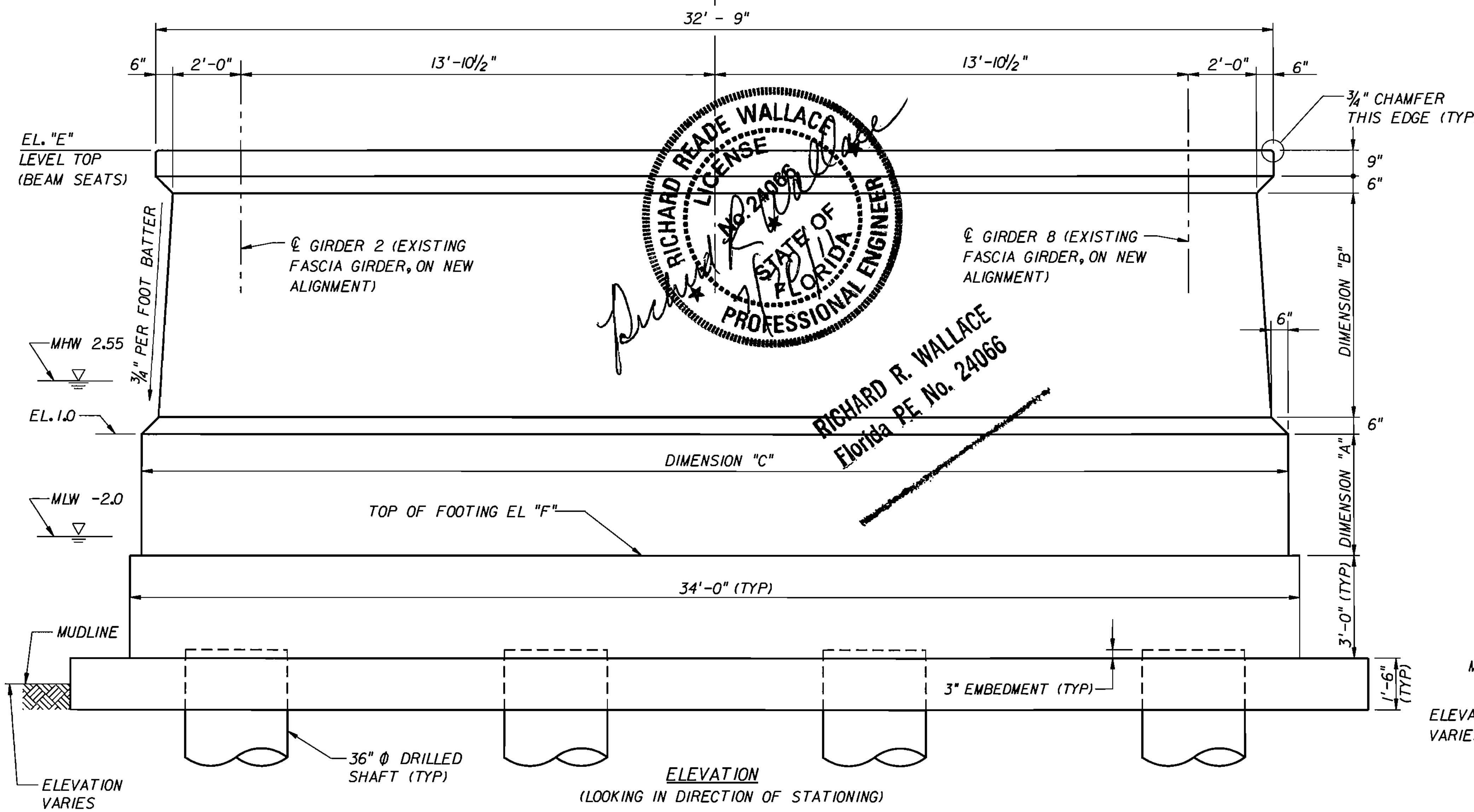
PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. B-41

DD-MMM-YYYY HH:MM DGN$SPECIFICATION



PLAN



ELEVATION
(LOOKING IN DIRECTION OF STATIONING)

NOTES:

1. FOUNDATION SHOWN FOR REFERENCE PURPOSES ONLY. SEE SHEET B-48 FOR FOUNDATION DETAILS.
2. BEARINGS SHOWN IN PLAN VIEW ONLY FOR ILLUSTRATIVE PURPOSES. SEE SHEETS B-191 THRU B-196 FOR BEARING DETAILS.
3. ALL CHAMFERS SHALL BE ¾" UNO.

PIER	DIMENSION "A"	DIMENSION "B"	DIMENSION "C"	DIMENSION "D"	ELEVATION "E"	ELEVATION "F"
2	6'-0"	3'-3¾"	33'-2"	4'-5"	6.056	-5.0
3	8'-0"	4'-4⅞"	33'-3⅝"	4'-6⅝"	7.157	-7.0
4	8'-0"	5'-6⅛"	33'-5¼"	4'-8¼"	8.258	-7.0
5	8'-0"	6'-7¼"	33'-7"	4'-10"	9.359	-7.0

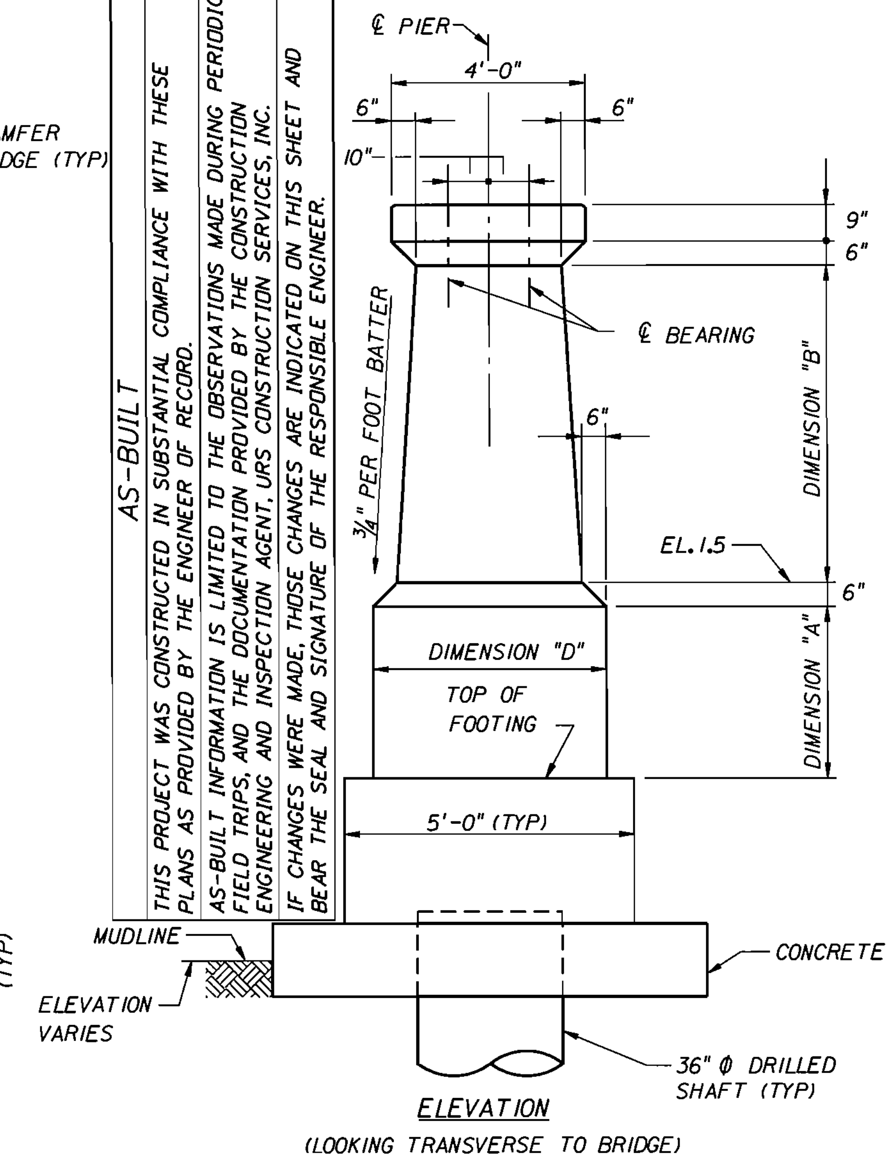
AB

AS-BUILT

THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.

AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.

IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.



ELEVATION
(LOOKING TRANSVERSE TO BRIDGE)

BRIDGE NO. 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	PAW	11-02
CHECKED BY	JPT	06-03
DESIGNED BY	PAW	11-02
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Peter A. Winkler, Florida PE No. 56720

FLORIDA DEPARTMENT OF TRANSPORTATION

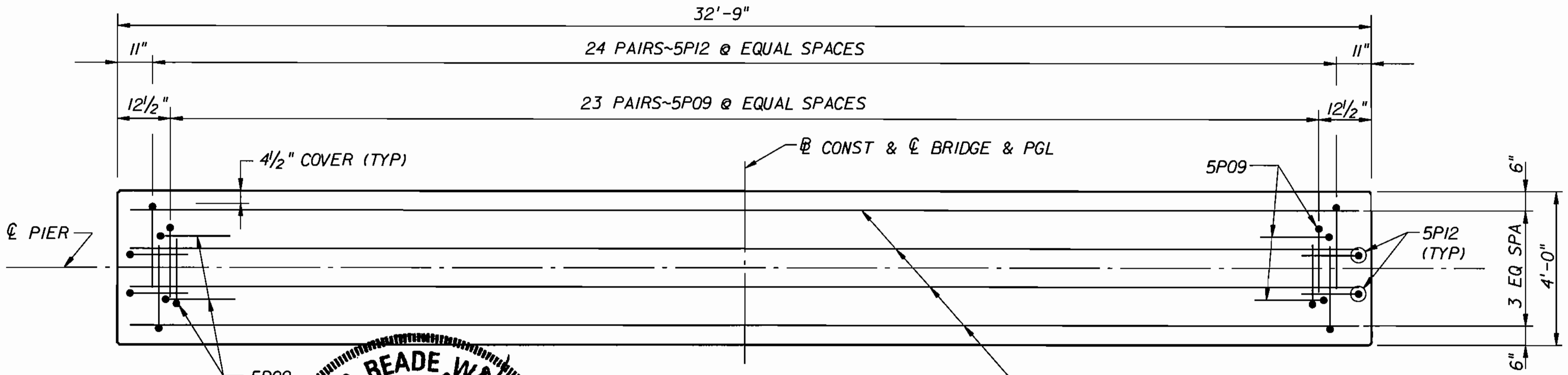
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: PIERS 2 - 5 PLAN AND ELEVATION

PROJECT NAME: PERMANENT BRIDGE
BRIDGE OF LIONS REHABILITATION

SHEET NO. B-45

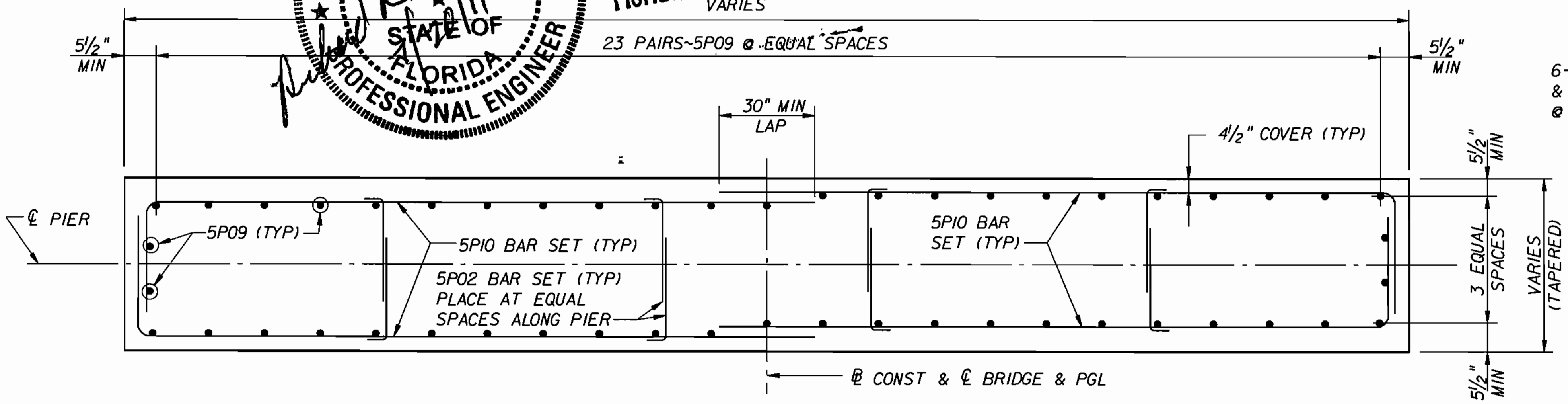
*DGN$SPECIFICATION*******
DD-MMM-YYYY HH:MM



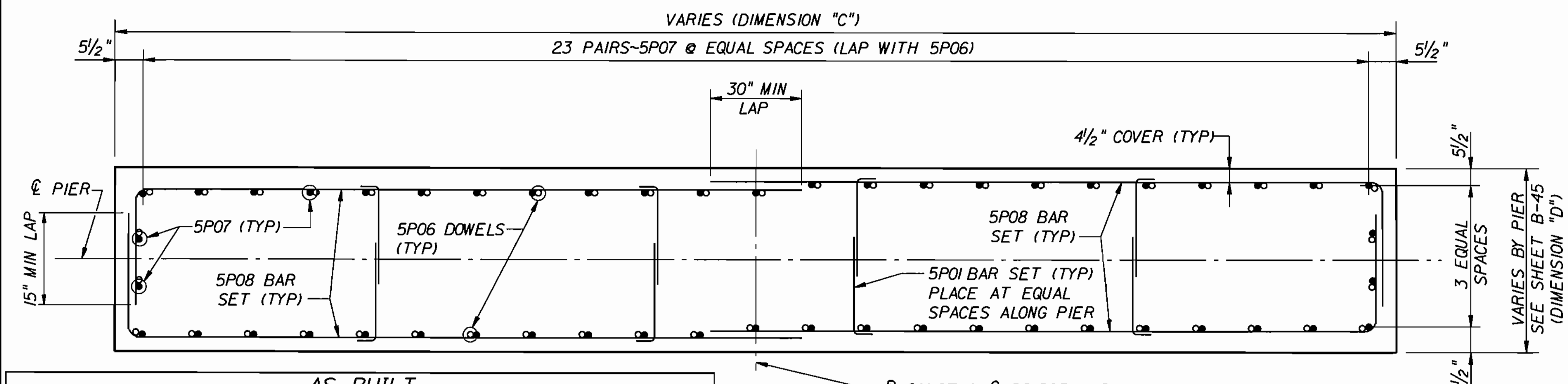
PLAN VIEW A-A OF CAPITAL

RICHARD READE WALLACE
LICENSE
No. 24066
STATE OF FLORIDA
PROFESSIONAL ENGINEER

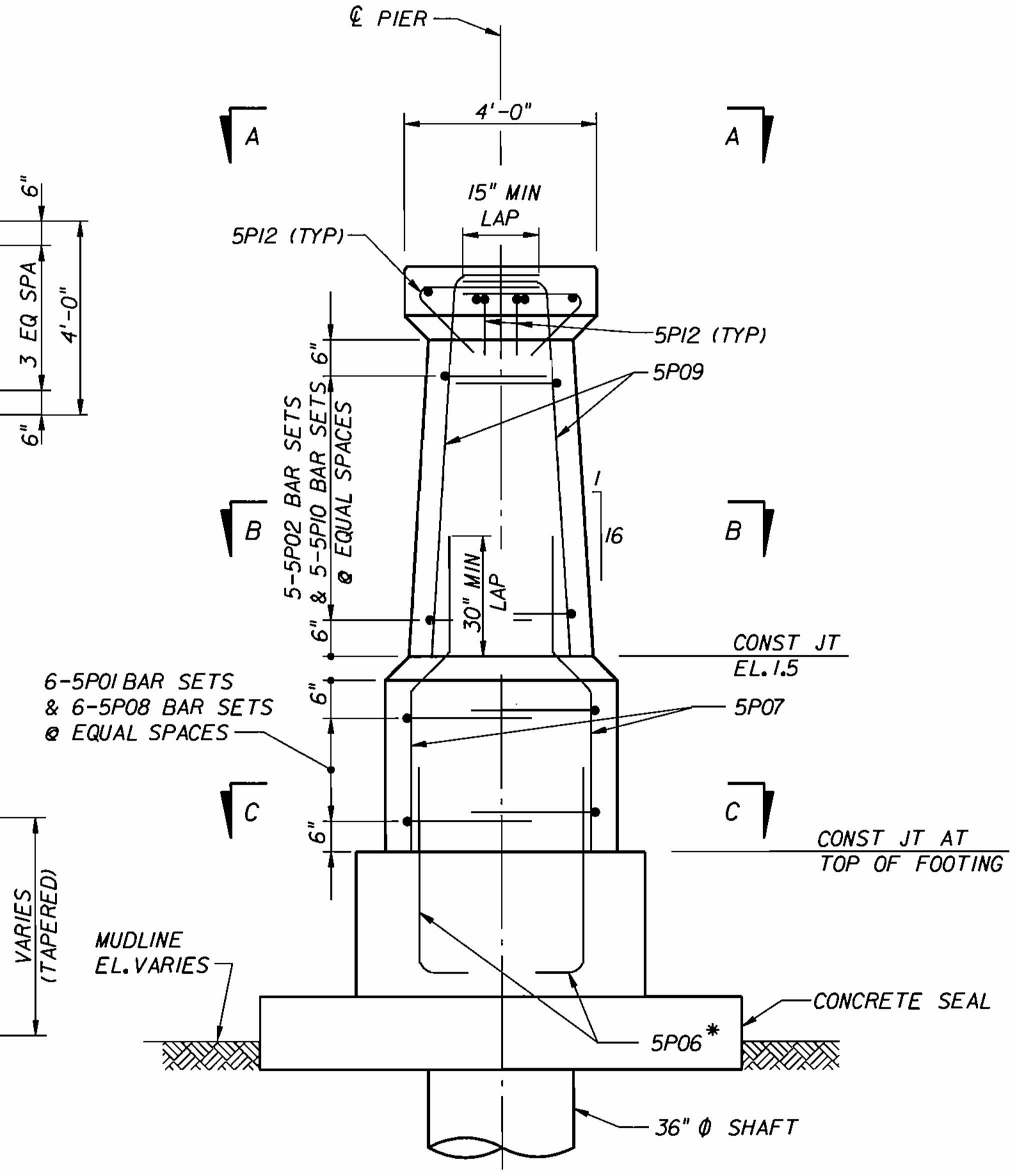
RICHARD R. WALLACE
Florida PE No. 24066



SECTION B-B



SECTION C-C



ELEVATION

* ORIENTATION OF 5P06 BARS MAY BE ADJUSTED TO AVOID CONFLICTS WITH DRILLED SHAFT BARS.

NOTES:

1. FOOTING SHOWN FOR REFERENCE PURPOSES ONLY. SEE SHEET B-48 FOR FOOTING DETAILS AND REINFORCEMENT.
2. AS A COUNTERMEASURE TO SPECIFICATION SECTION 400-9.5, THE CONTRACTOR SHALL NOT PERMIT BAY WATER TO CONTAMINATE THE SURFACE OF ANY CONSTRUCTION JOINTS PLACED BETWEEN ELEVATION -4.00 AND ELEVATION +6.55. JOINT SURFACES SHALL BE DRY OR FLOODED WITH FRESH WATER UNTIL THE COLUMN PORTION ABOVE CONSTRUCTION JOINT IS PLACED AND CURED FOR AT LEAST 72 HOURS.
3. BARS 5P09, 5P11, AND 5P12 MAY BE SHIFTED SLIGHTLY TO AVOID CONFLICTS WITH ANCHOR BOLTS AS NECESSARY.

BRIDGE NO. 780074

AS-BUILT

THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.

AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC. AB

IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER. AB

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	PAW	11-02
CHECKED BY	JPT	06-03
DESIGNED BY	PAW	11-02
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Peter A. Winkler, Florida PE No. 56720

FLORIDA DEPARTMENT OF TRANSPORTATION

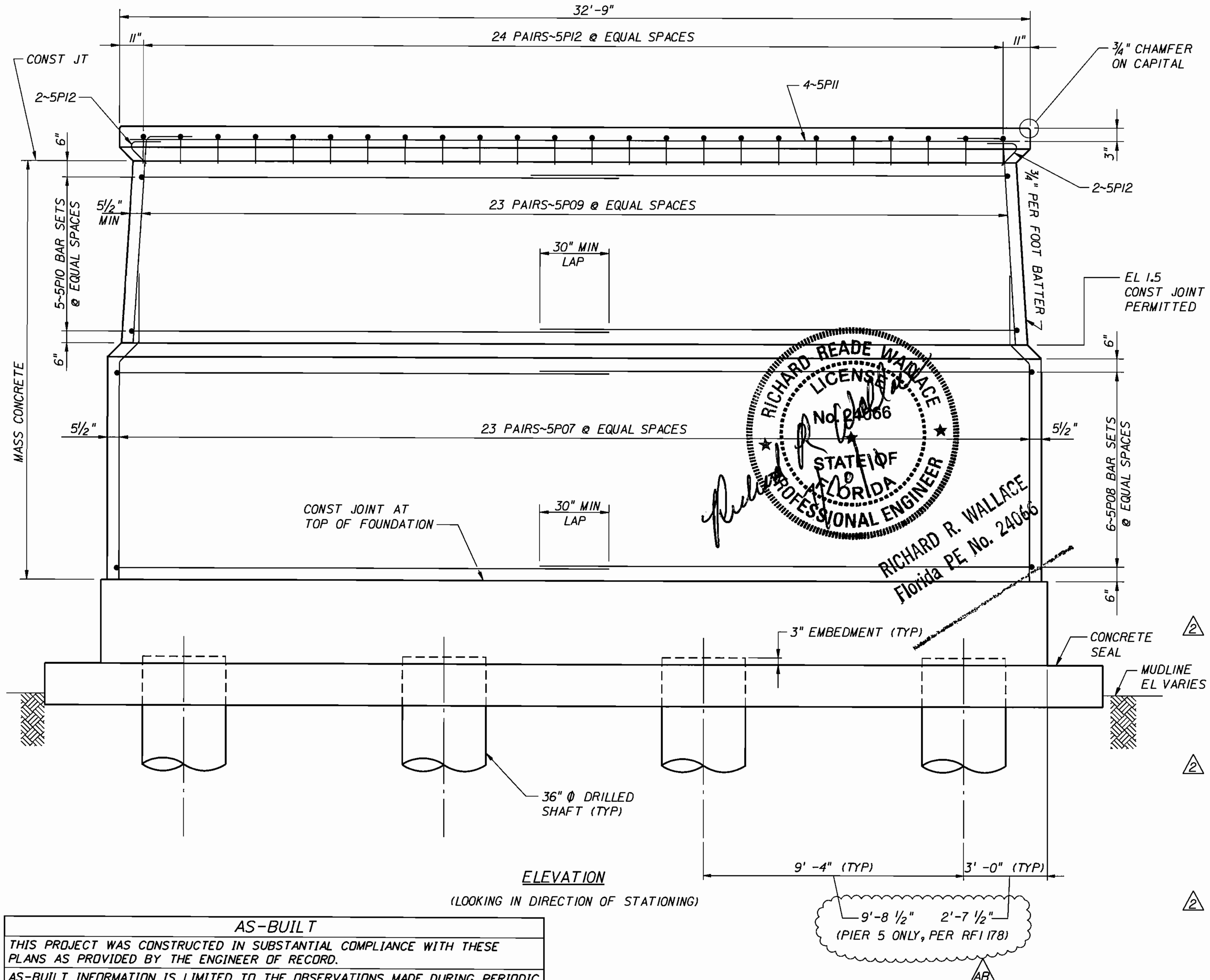
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: REINFORCEMENT DETAILS PIERS 2 - 5 (1 OF 3)

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. B-46

•DGN$SPECIFICATION•••••• •DD-MMM-YYYY HH:MM•



ELEVATION

(LOOKING IN DIRECTION OF STATIONING)

NOTES:

1. FOOTING SHOWN FOR REFERENCE PURPOSES ONLY. SEE SHEET B-48 FOR FOOTING DETAILS AND REINFORCEMENT.

2. AS A COUNTERMEASURE TO SPECIFICATION SECTION 400-9.5, THE CONTRACTOR SHALL NOT PERMIT BAY WATER TO CONTAMINATE THE SURFACE OF ANY CONSTRUCTION JOINTS PLACED BETWEEN ELEVATION -4.00 AND ELEVATION +6.55. JOINT SURFACES SHALL BE DRY OR FLOODED WITH FRESH WATER UNTIL THE COLUMN PORTION ABOVE CONSTRUCTION JOINT IS PLACED AND CURED FOR AT LEAST 72 HOURS.

ESTIMATED QUANTITIES FOR PIER 2

ITEM	UNIT	QUANTITY
Class V Concrete (Substructure)	C.Y.	18.89
Class V Concrete (Microsilica-Substructure)	C.Y.	5.7
Class V Concrete (Mass) (Microsilica-Substructure)	C.Y.	47.5
Class III (Seal) Concrete *	C.Y.	20.1
Reinforcing Steel (Substructure)	LB.	3,954

ESTIMATED QUANTITIES FOR PIER 3

ITEM	UNIT	QUANTITY
Class V Concrete (Substructure)	C.Y.	18.89
Class V Concrete (Microsilica-Substructure)	C.Y.	5.7
2 Class V Concrete (Mass) (Microsilica-Substructure)	C.Y.	64.5
Class III (Seal) Concrete *	C.Y.	20.1
Reinforcing Steel (Substructure)	LB.	4,120

ESTIMATED QUANTITIES FOR PIER 4

ITEM	UNIT	QUANTITY
Class V Concrete (Substructure)	C.Y.	18.89
Class V Concrete (Microsilica-Substructure)	C.Y.	5.7
2 Class V Concrete (Mass) (Microsilica-Substructure)	C.Y.	70.9
Class III (Seal) Concrete *	C.Y.	20.1
Reinforcing Steel (Substructure)	LB.	4,179

ESTIMATED QUANTITIES FOR PIER 5

ITEM	UNIT	QUANTITY
Class V Concrete (Substructure)	C.Y.	18.89
Class V Concrete (Microsilica-Substructure)	C.Y.	5.7
2 Class V Concrete (Mass) (Microsilica-Substructure)	C.Y.	77.4
Class III (Seal) Concrete *	C.Y.	20.1
Reinforcing Steel (Substructure)	LB.	4,292

* SEAL QUANTITIES BASED ON 1'-6" THICKNESS

AS-BUILT

THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.

AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.

IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER. AB

BRIDGE NO. 780074

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
4-2-04	BMS	2 REVISIONS TO ESTIMATED QUANTITIES FOR PIERS 3, 4, AND 5			
7-20-11	RRW	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	PAW	11-02
CHECKED BY	JPT	06-03
DESIGNED BY	PAW	11-02
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Peter A. Winkler, Florida PE No. 56720

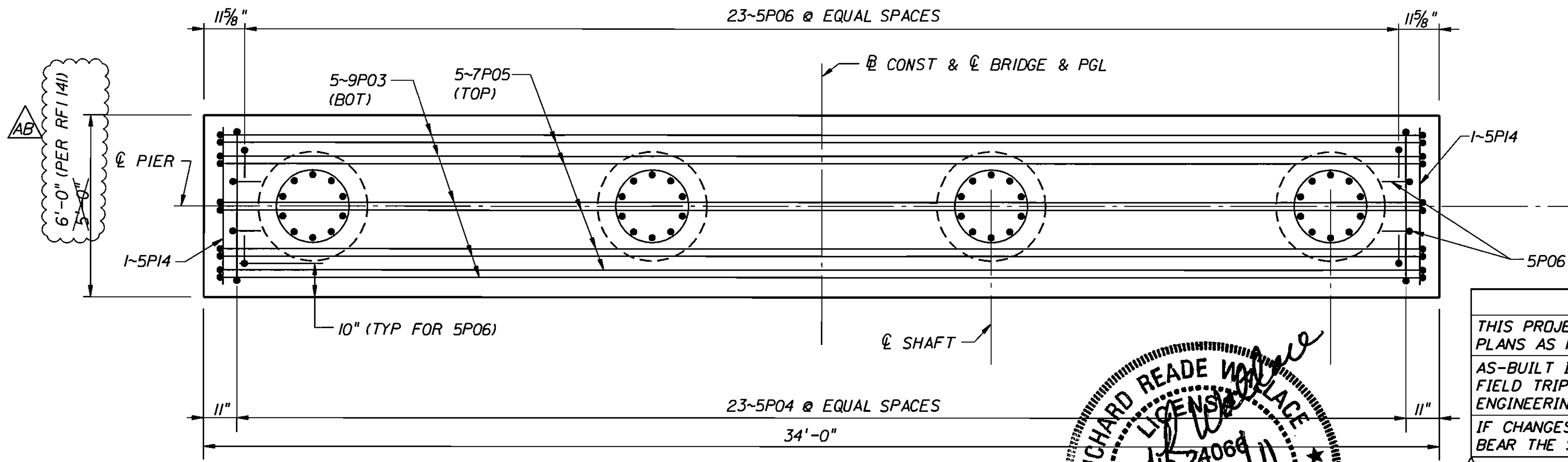
FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: REINFORCEMENT DETAILS PIERS 2 - 5 (2 OF 3)

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. B-47



PLAN

AS-BUILT

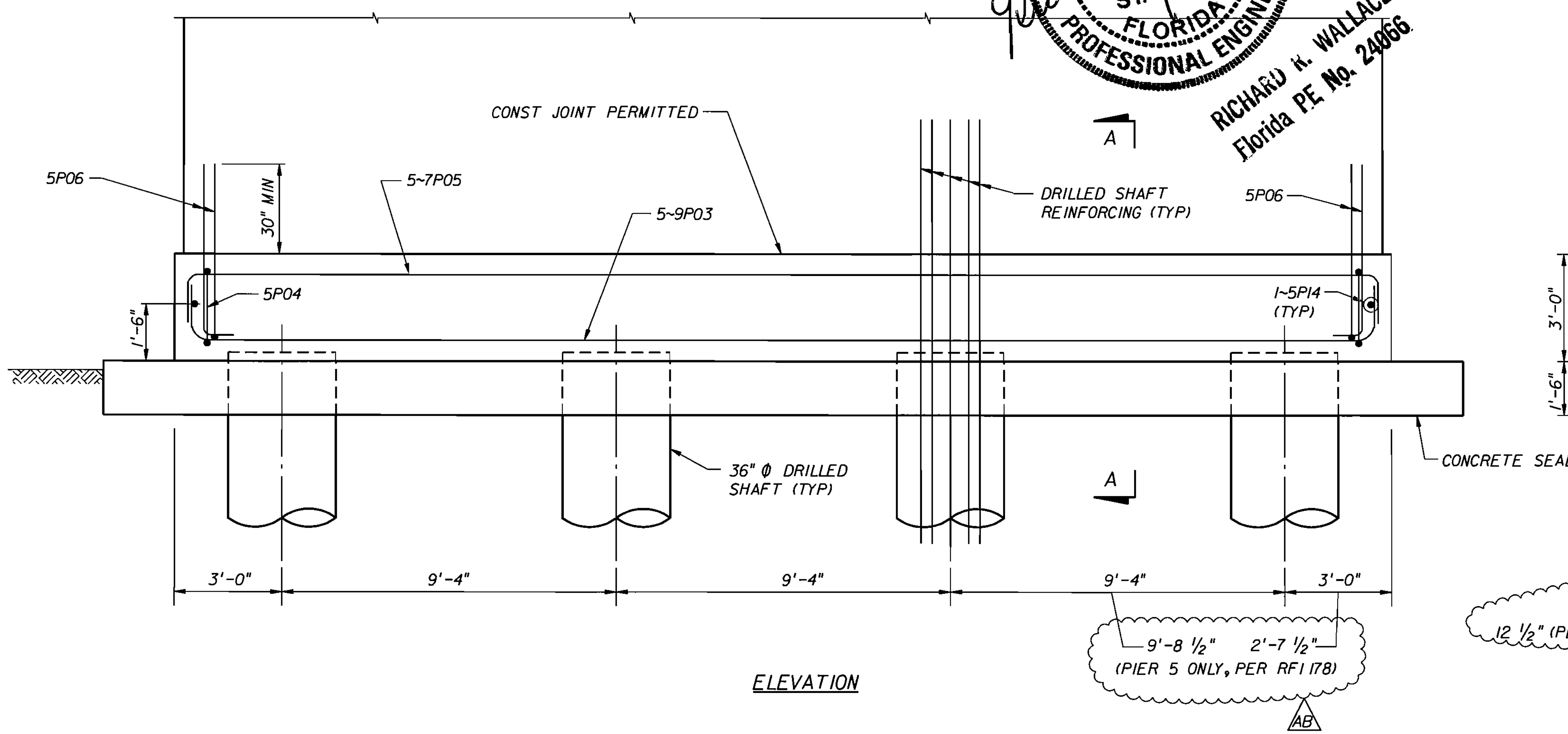
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.

AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.

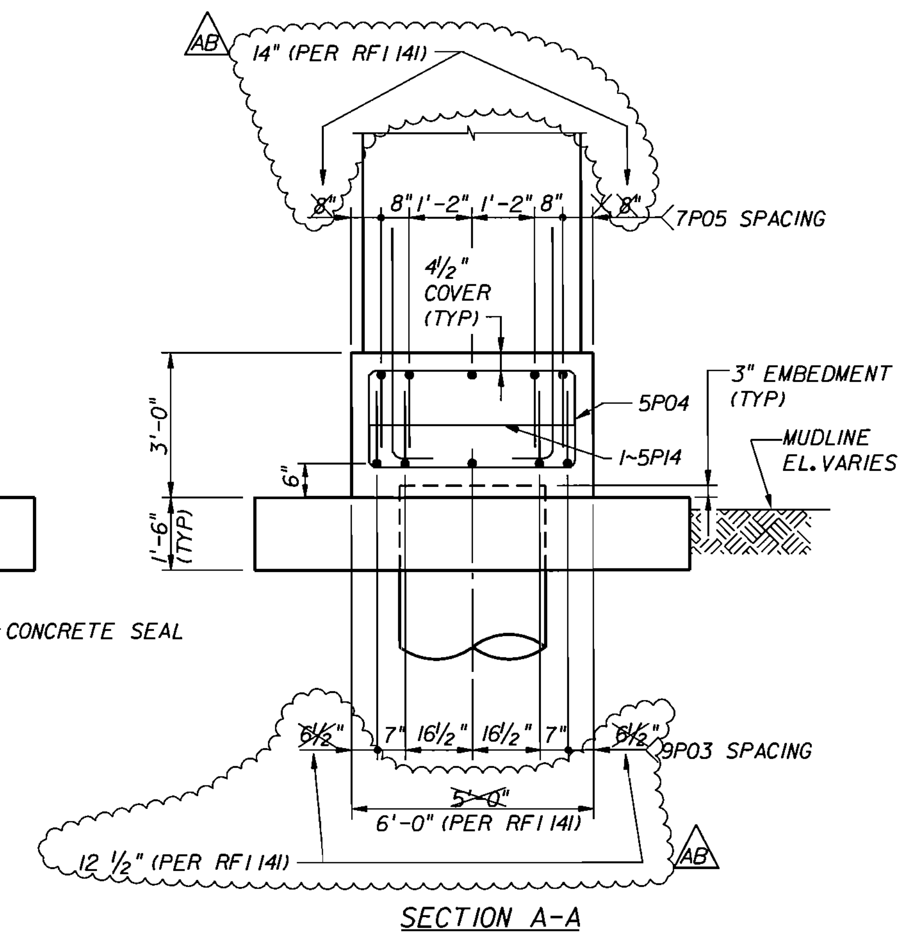
IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.

RICHARD READE WALLACE
LICENSE
No. 24066
7/20/11
STATE OF
FLORIDA
PROFESSIONAL ENGINEER

RICHARD H. WALLACE
Florida PE No. 24066



ELEVATION



SECTION A-A

BRIDGE NO. 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	PAW	11-02
CHECKED BY	JPT	06-03
DESIGNED BY	PAW	11-02
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Peter A. Winkler, Florida PE No. 56720

FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

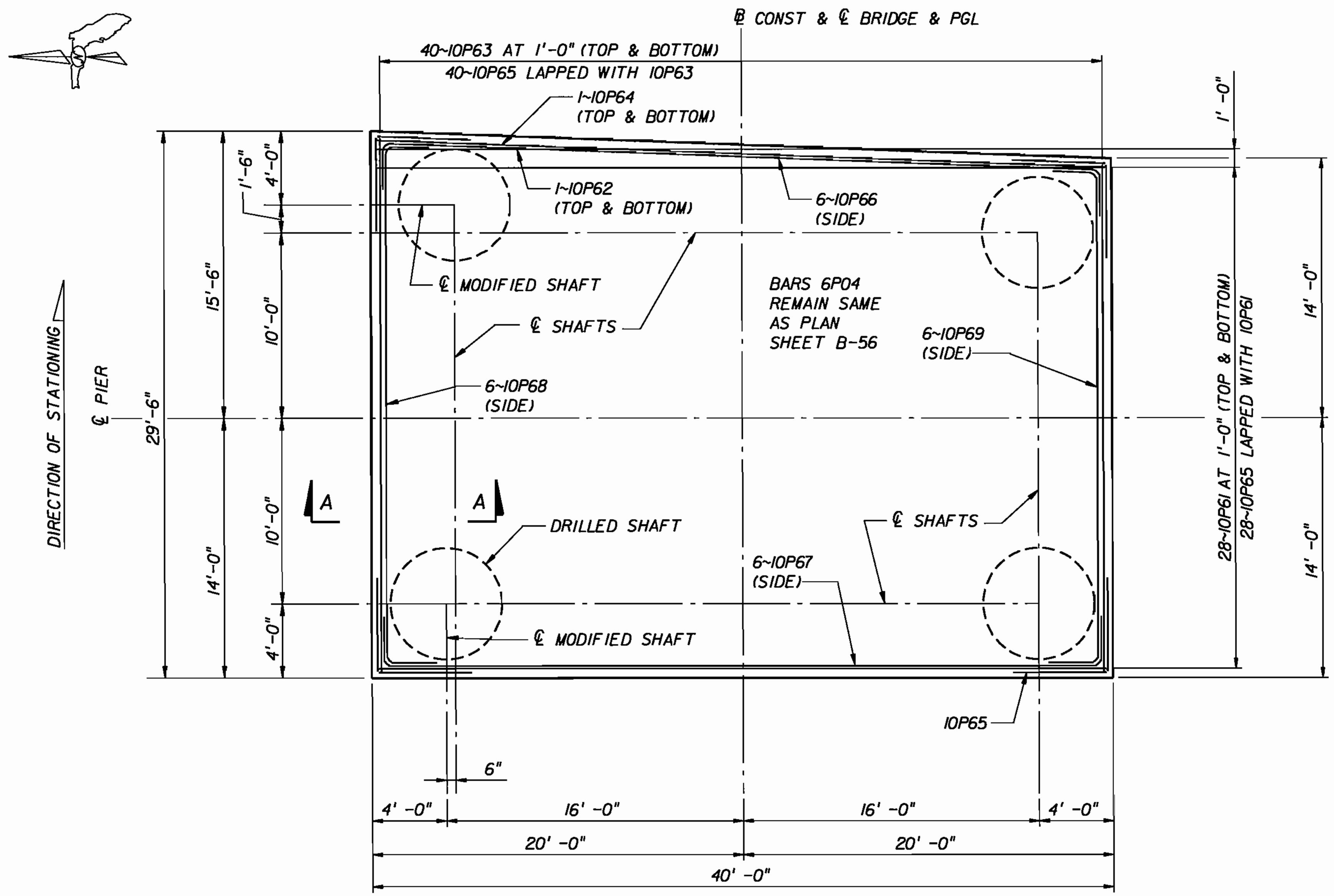
SHEET TITLE: REINFORCEMENT DETAILS
PIERS 2 - 5 (3 OF 3)

PROJECT NAME: PERMANENT BRIDGE
BRIDGE OF LIONS REHABILITATION

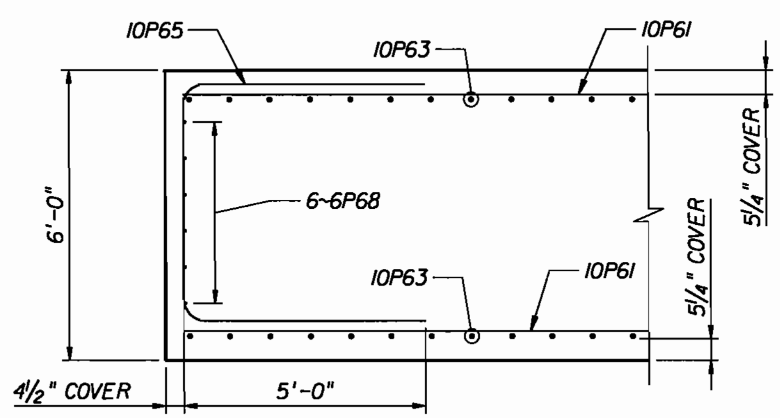
SHEET NO. B-48

•DGN$SPECIFICATION•••••••

•DD-MMM-YYYY HH:MM•



PLAN - PIER 12



SECTION A-A

RICHARD READE WALLACE
LICENSE
No. 24066
STATE OF FLORIDA
PROFESSIONAL ENGINEER
7/20/11
RICHARD R. WALLACE
Florida PE No. 24066

AS-BUILT
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.
IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.

AB

BRIDGE NO. 780074

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT - NEW SHEET (PER RFI 147)			

	NAMES	DATES
DRAWN BY	BMS	01-07
CHECKED BY	RRW	01-07
DESIGNED BY		
CHECKED BY		
APPROVED BY		

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620

FLORIDA DEPARTMENT OF TRANSPORTATION

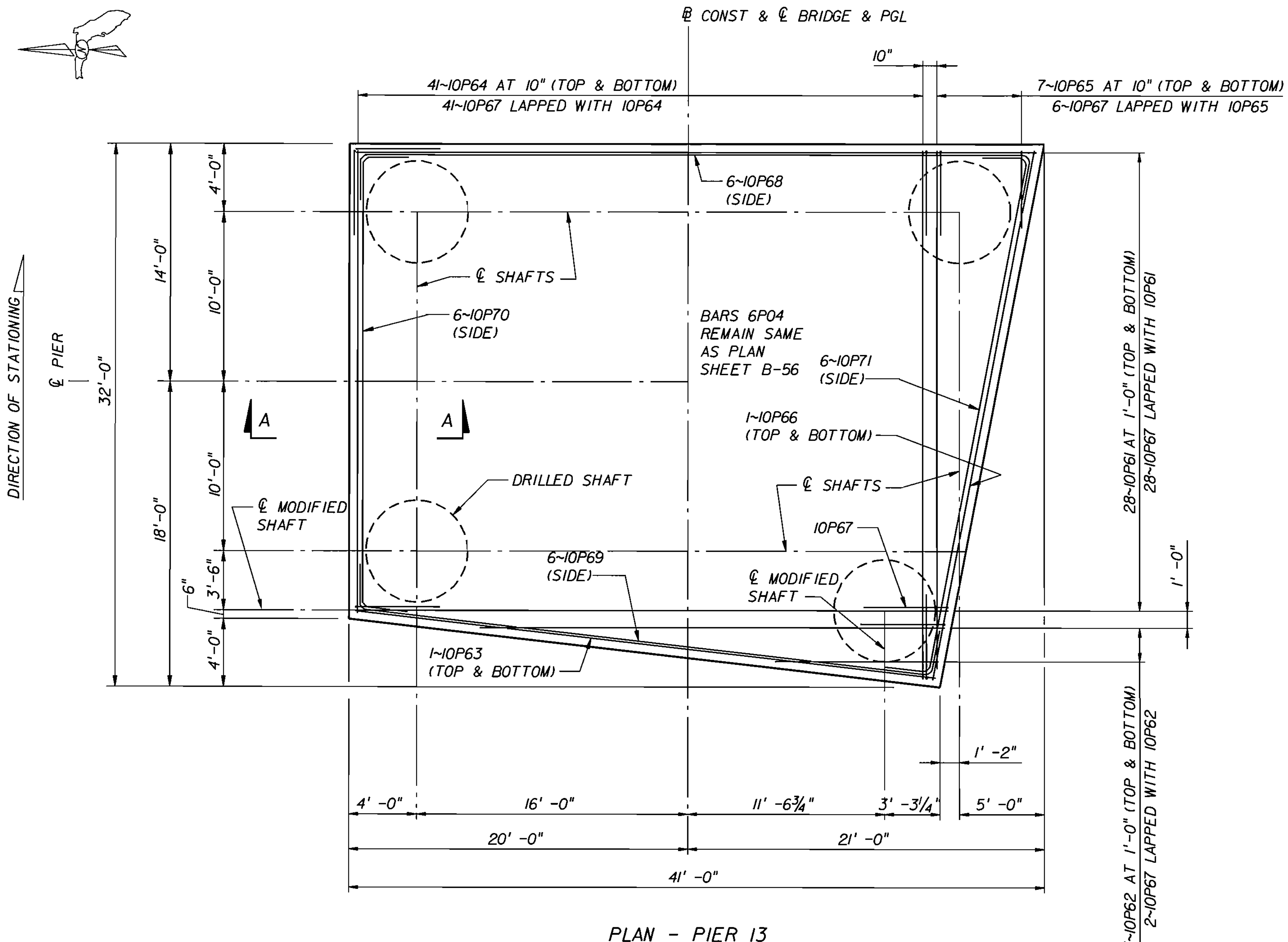
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: FOOTING REINFORCEMENT DETAILS MODIFIED PIER 12

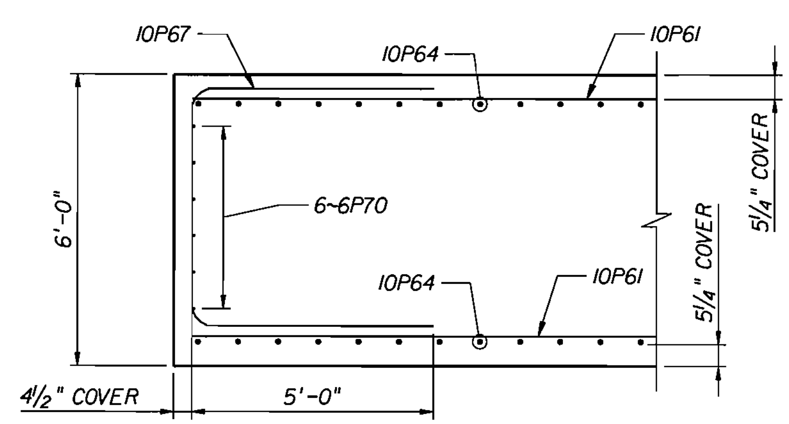
PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. B-56A

•DGN$SPECIFICATION•••••• •DD-MMM-YYYY HH:MM•



PLAN - PIER 13



SECTION A-A

AS-BUILT
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.
IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.

AB

RICHARD READE WALLACE LICENSE No. 24066 STATE OF FLORIDA PROFESSIONAL ENGINEER

RICHARD R. WALLACE
Florida PE No. 24066

BRIDGE NO. 780074

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT - NEW SHEET (PER RFI'S 144, 147, & 148)			

	NAMES	DATES
DRAWN BY	BMS	12-06
CHECKED BY	RRW	12-06
DESIGNED BY		
CHECKED BY		
APPROVED BY		

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620

FLORIDA DEPARTMENT OF TRANSPORTATION

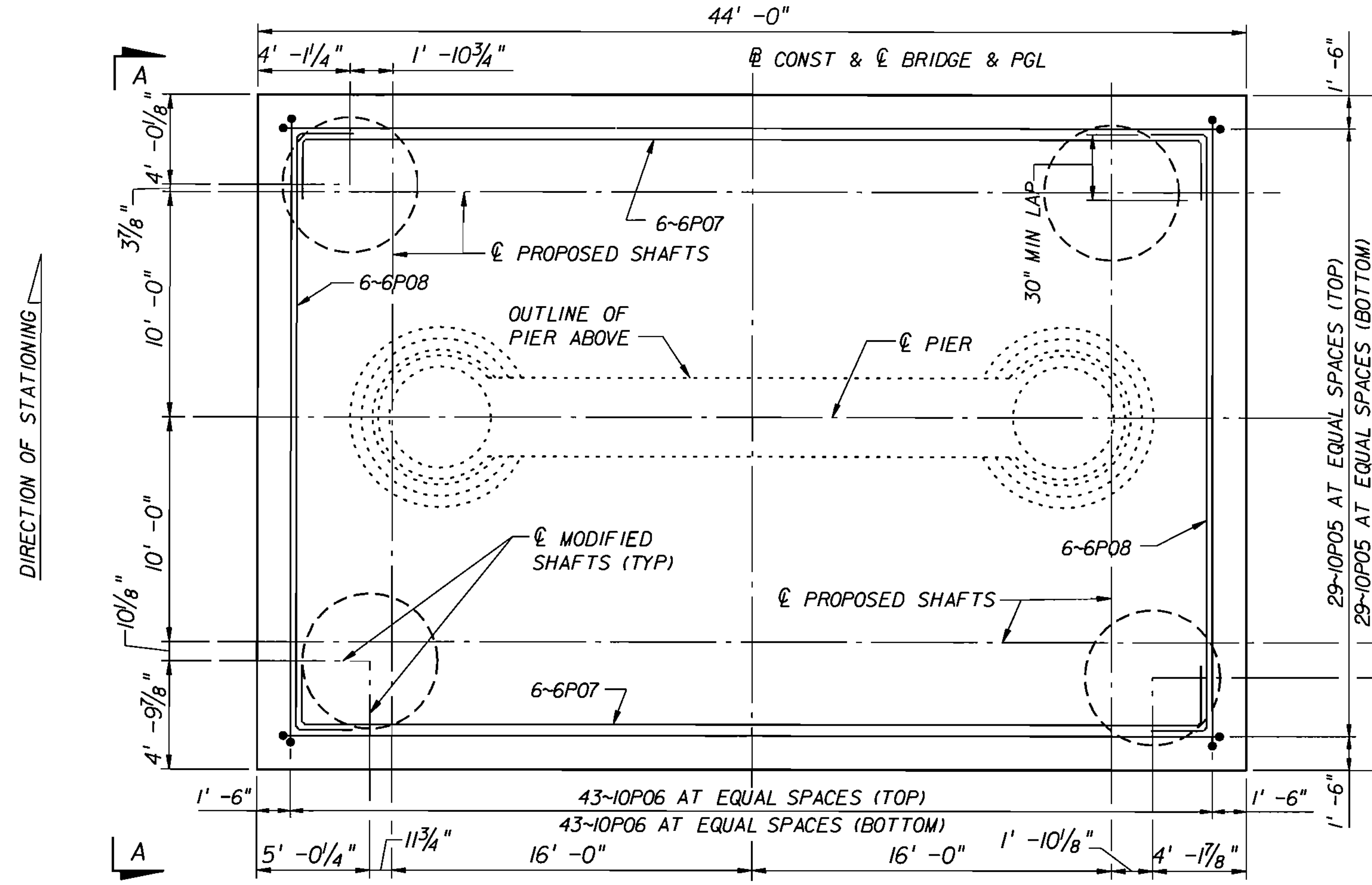
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: FOOTING REINFORCEMENT DETAILS MODIFIED PIER 13

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. B-56B

•DGN$SPECIFICATION•••••••
•DD-MMM-YYYY HH:MM•



PLAN - PIER 17

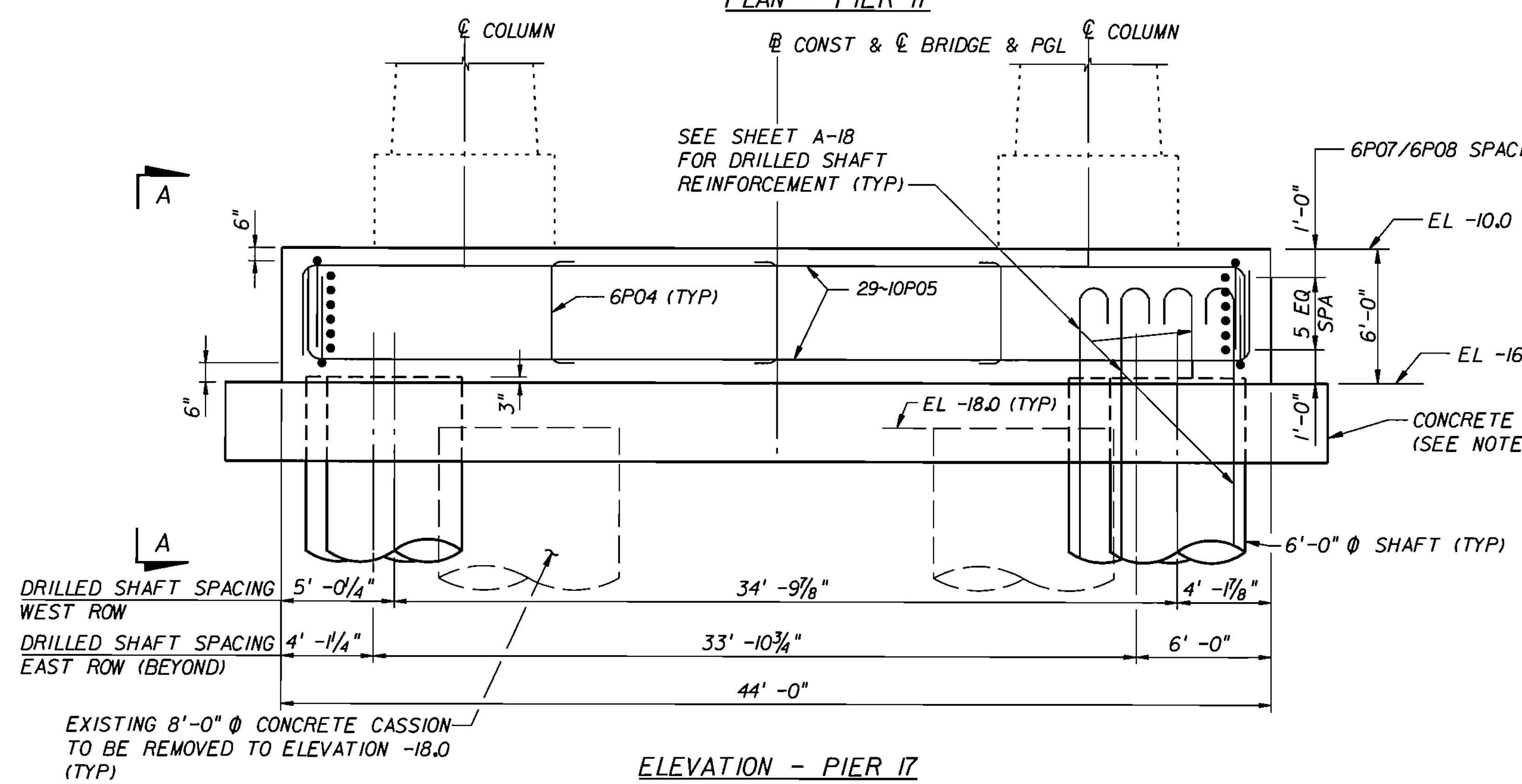
NOTES:

1. DETAILS SHOWN THIS SHEET DEPICT THE MODIFIED SUBMERGED FOUR SHAFT PIER FOOTING USED TO SUPPORT THE PIER COLUMN AT PIER 17.
2. 10P06 AND 10P05 BARS MAY BE SHIFTED SLIGHTLY IN THE VICINITY OF THE DRILLED SHAFTS AND PIER COLUMNS TO AVOID CONFLICTS WITH THE SHAFT VERTICAL REINFORCEMENT.
3. PROVISIONS FOR MASS CONCRETE SHALL APPLY TO CONCRETE FOR PIERS 12-14 AND 17-19 FOOTINGS.
4. NON-REINFORCED, CLASS III (SEAL) CONCRETE SEALS SHALL BE 3'-6" THICK FOR PIERS 12-14 AND 17-19. CONTRACTOR MAY PROVIDE ALTERNATE DESIGN FOR SEALS, SIGNED AND SEALED BY A PROFESSIONAL ENGINEER REGISTERED IN THE STATE OF FLORIDA.

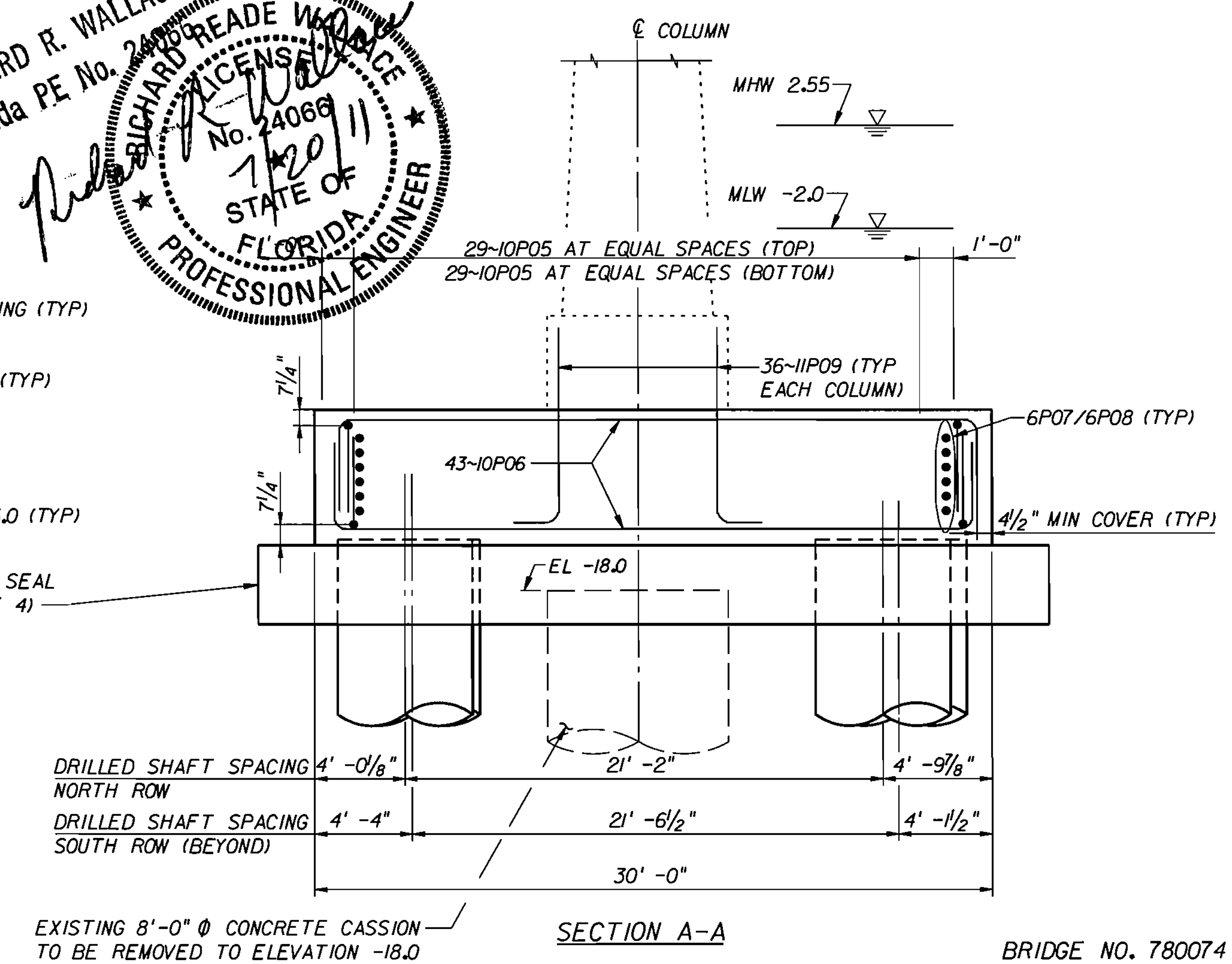
AS-BUILT
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.
IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.

AB

RICHARD R. WALLACE
Florida PE No. 24066
RICHARD READE WALLACE LICENSE No. 24066 STATE OF FLORIDA PROFESSIONAL ENGINEER
7/20/11



ELEVATION - PIER 17



SECTION A-A

BRIDGE NO. 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT - NEW SHEET (PER RFI'S 187, 188, & 190)			

	NAMES	DATES
DRAWN BY	PAW	05-03
CHECKED BY	GBL	06-03
DESIGNED BY	GBL	03-03
CHECKED BY	PAW	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Peter A. Winkler, Florida PE No. 56720

FLORIDA DEPARTMENT OF TRANSPORTATION

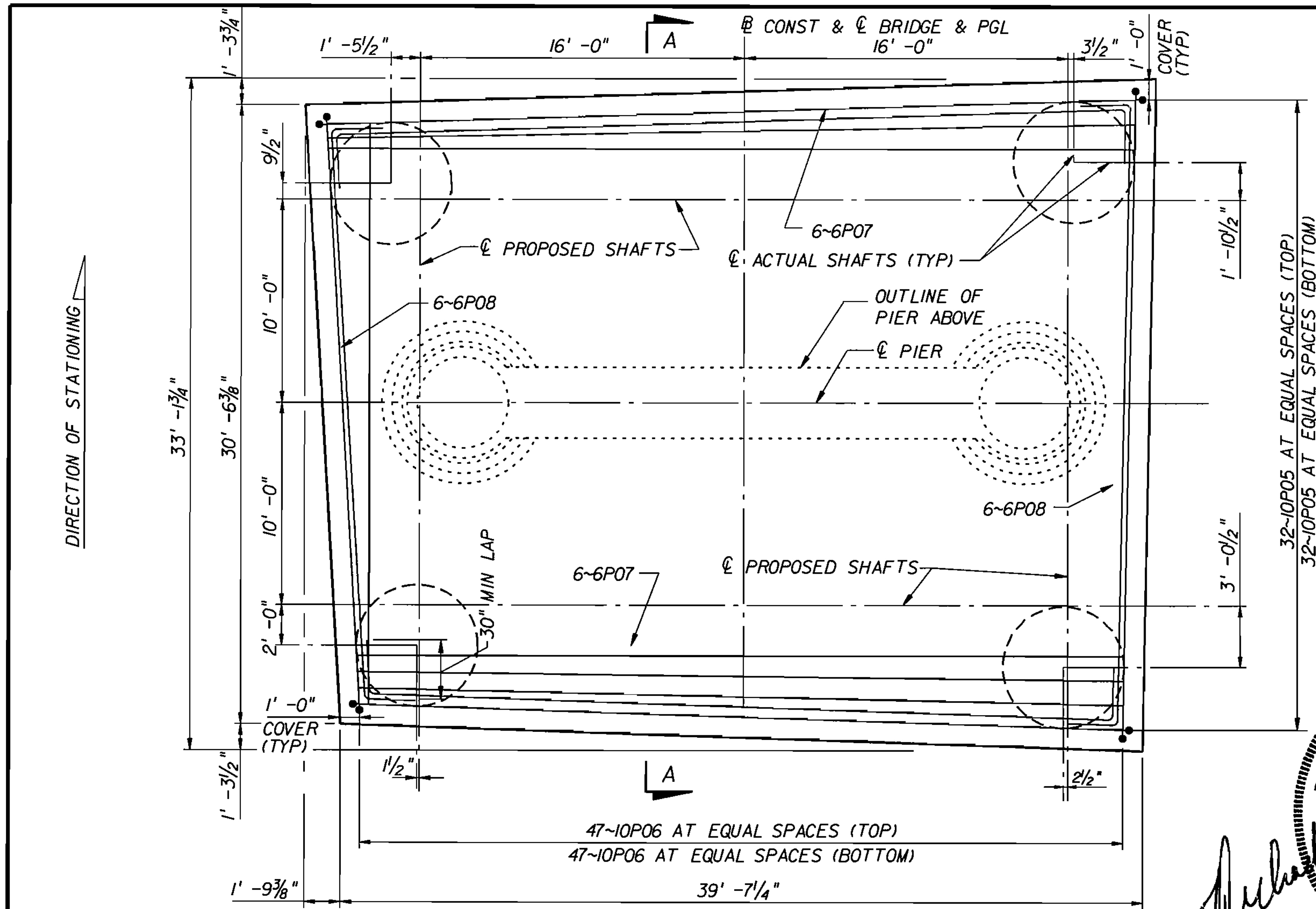
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: FOOTING REINFORCEMENT DETAILS MODIFIED PIER 17

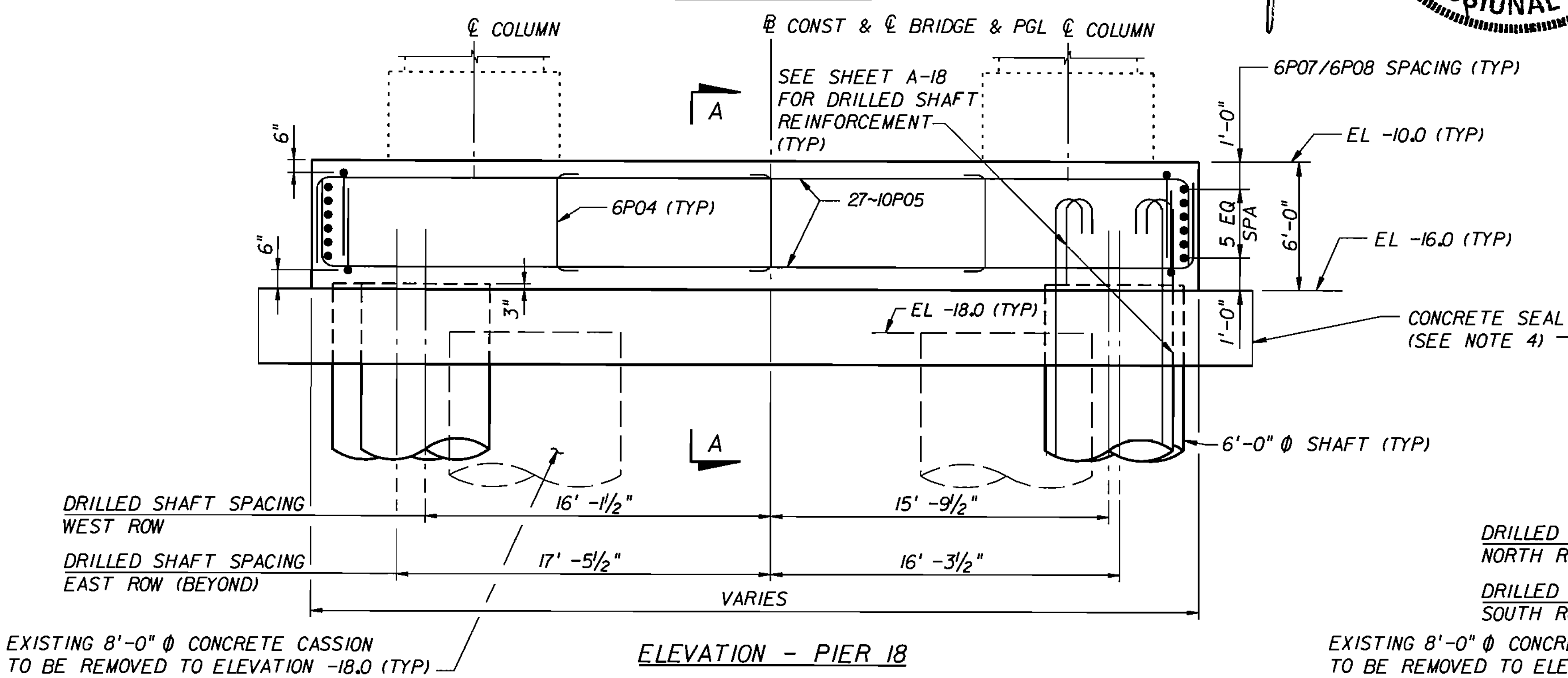
PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. B-56C

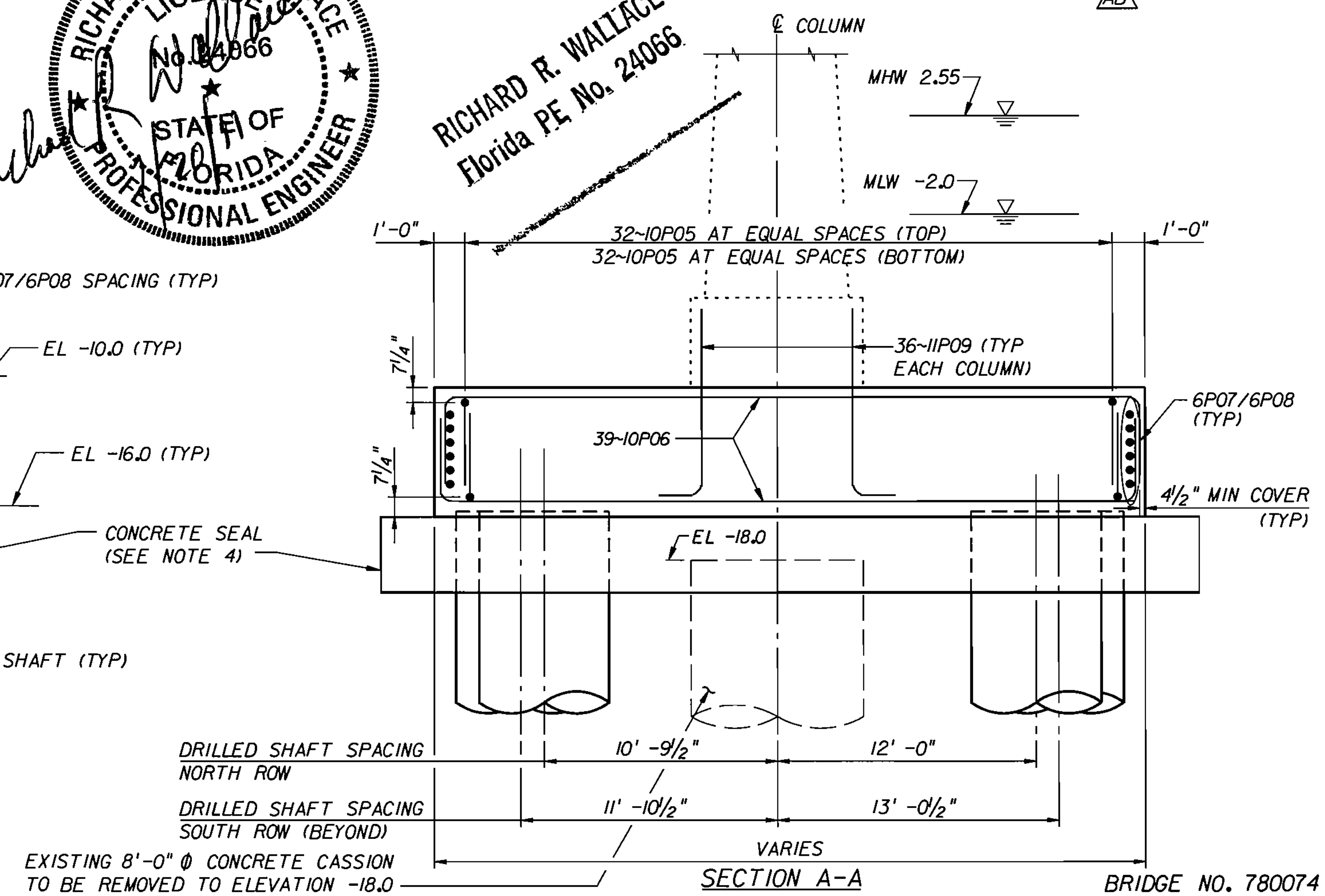
•DD-MMM-YYYY HH:MM• •DGN$SPECIFICATION•••••••



PLAN - PIER 18



ELEVATION - PIER 18



SECTION A-A

NOTES:

1. DETAILS SHOWN THIS SHEET DEPICT THE TYPICAL SUBMERGED FOUR SHAFT PIER FOOTING USED TO SUPPORT PIER 18.
2. 10P06 AND 10P05 BARS MAY BE SHIFTED SLIGHTLY IN THE VICINITY OF THE DRILLED SHAFTS AND PIER COLUMNS TO AVOID CONFLICTS WITH THE SHAFT VERTICAL REINFORCEMENT.
3. PROVISIONS FOR MASS CONCRETE SHALL APPLY TO CONCRETE FOR PIERS 12-14 AND 17-19 FOOTINGS.
4. NON-REINFORCED, CLASS III (SEAL) CONCRETE SEALS SHALL BE 3'-6" THICK FOR PIERS 12-14 AND 17-19. CONTRACTOR MAY PROVIDE ALTERNATE DESIGN FOR SEALS, SIGNED AND SEALED BY A PROFESSIONAL ENGINEER REGISTERED IN THE STATE OF FLORIDA.

AS-BUILT
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.
IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.

AB

RICHARD READE WALLACE LICENSE No. 24066 STATE OF FLORIDA PROFESSIONAL ENGINEER

RICHARD R. WALLACE
Florida PE No. 24066

BRIDGE NO. 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT - NEW SHEET (PER RFI'S 175, 179, 181, 184, & 287)			

	NAMES	DATES
DRAWN BY	PAW	05-03
CHECKED BY	GBL	06-03
DESIGNED BY	GBL	03-03
CHECKED BY	PAW	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Peter A. Winkler, Florida PE No. 56720

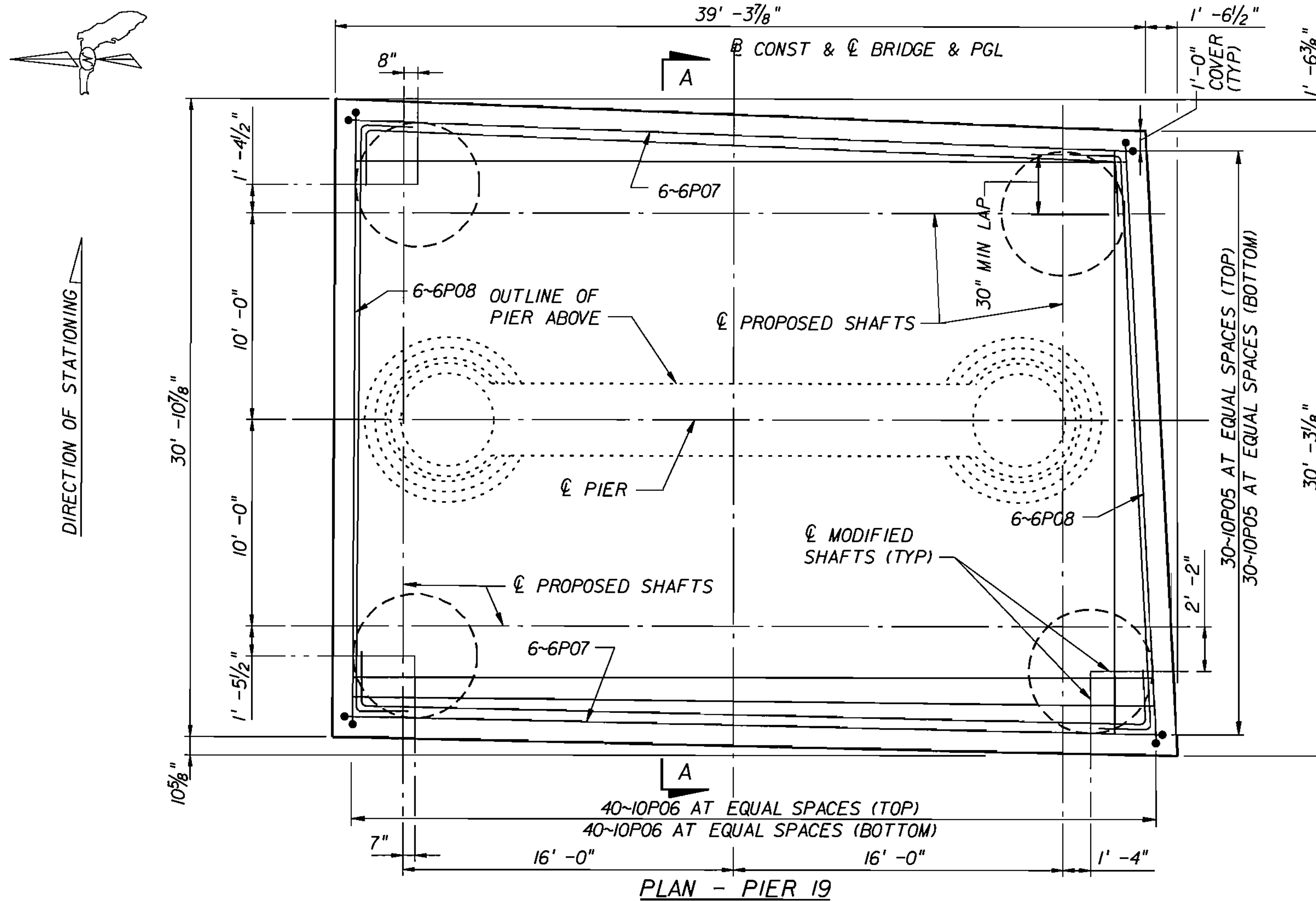
FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: FOOTING REINFORCEMENT DETAILS MODIFIED PIER 18

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. B-56D



PLAN - PIER 19

RICHARD READE WALLACE LICENSE No. 24066 STATE OF FLORIDA PROFESSIONAL ENGINEER

RICHARD R. WALLACE
Florida PE No. 24066

NOTES:

1. DETAILS SHOWN THIS SHEET DEPICT THE TYPICAL SUBMERGED FOUR SHAFT PIER FOOTING USED TO SUPPORT PIER 19.
2. 10P06 AND 10P05 BARS MAY BE SHIFTED SLIGHTLY IN THE VICINITY OF THE DRILLED SHAFTS AND PIER COLUMNS TO AVOID CONFLICTS WITH THE SHAFT VERTICAL REINFORCEMENT.
3. PROVISIONS FOR MASS CONCRETE SHALL APPLY TO CONCRETE FOR PIERS 12-14 AND 17-19 FOOTINGS.
4. NON-REINFORCED, CLASS III (SEAL) CONCRETE SEALS SHALL BE 3'-6" THICK FOR PIERS 12-14 AND 17-19. CONTRACTOR MAY PROVIDE ALTERNATE DESIGN FOR SEALS, SIGNED AND SEALED BY A PROFESSIONAL ENGINEER REGISTERED IN THE STATE OF FLORIDA.

AS-BUILT
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
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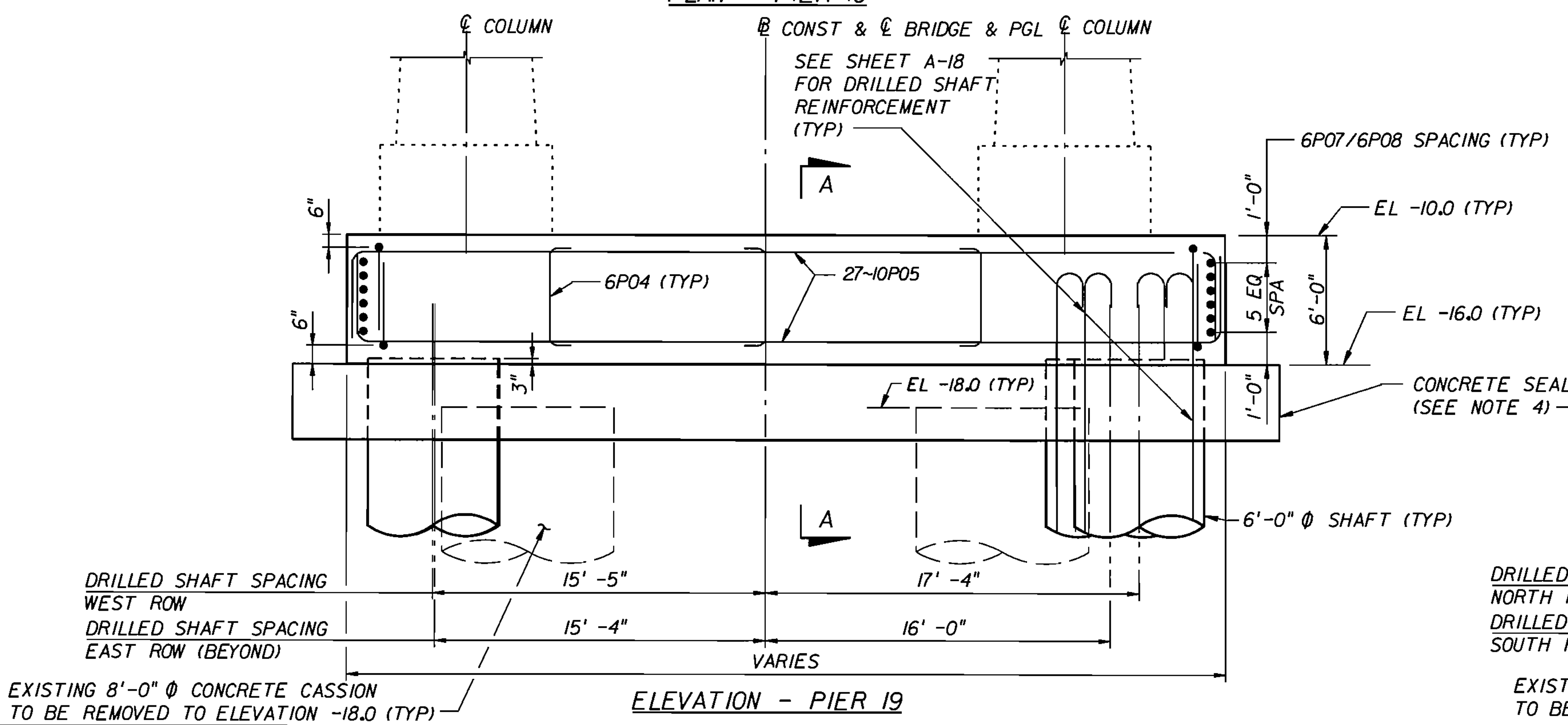
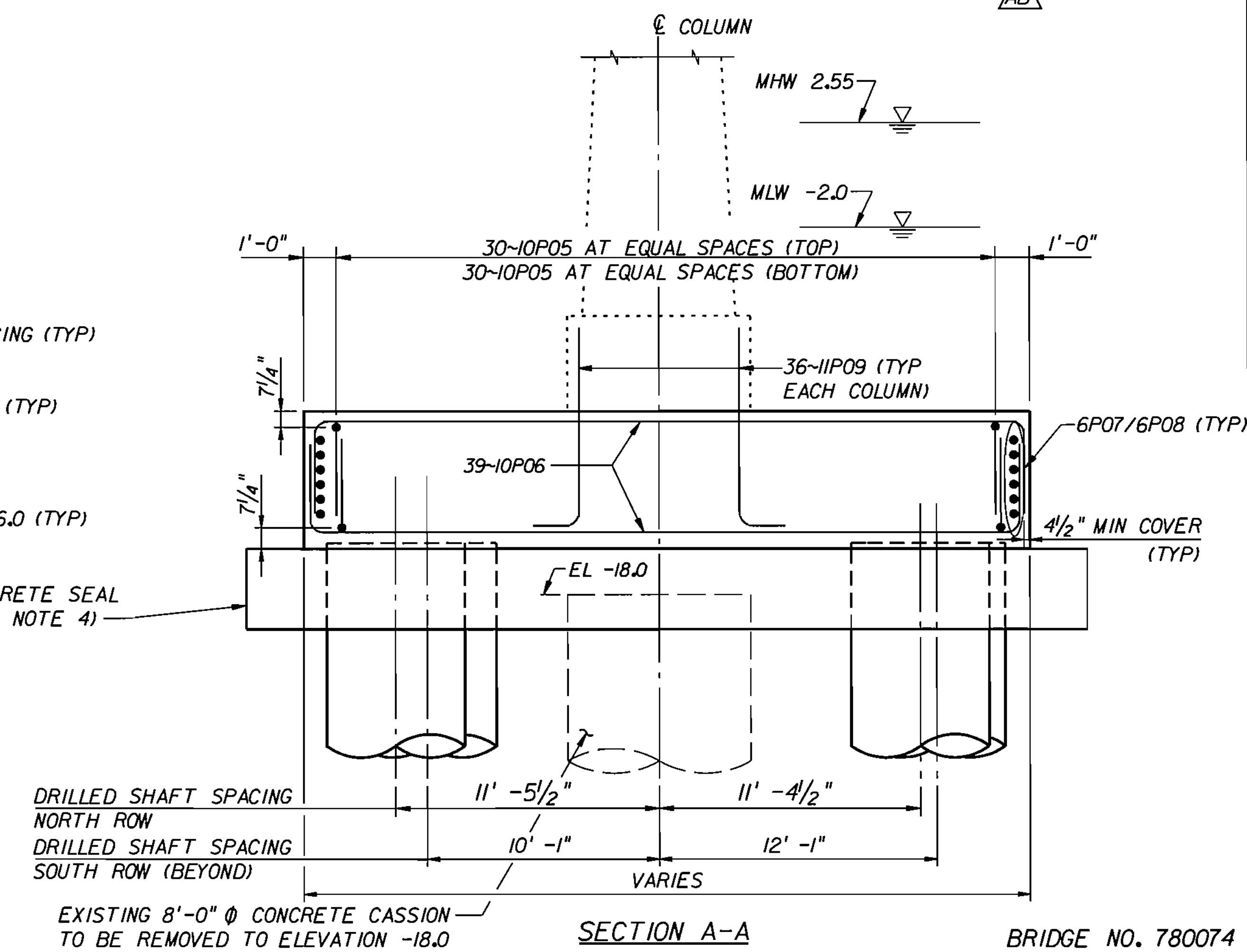
IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.

AB



ELEVATION - PIER 19



SECTION A-A

BRIDGE NO. 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	PAW	05-03
CHECKED BY	GBL	06-03
DESIGNED BY	GBL	03-03
CHECKED BY	PAW	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Peter A. Winkler, Florida PE No. 56720

FLORIDA DEPARTMENT OF TRANSPORTATION

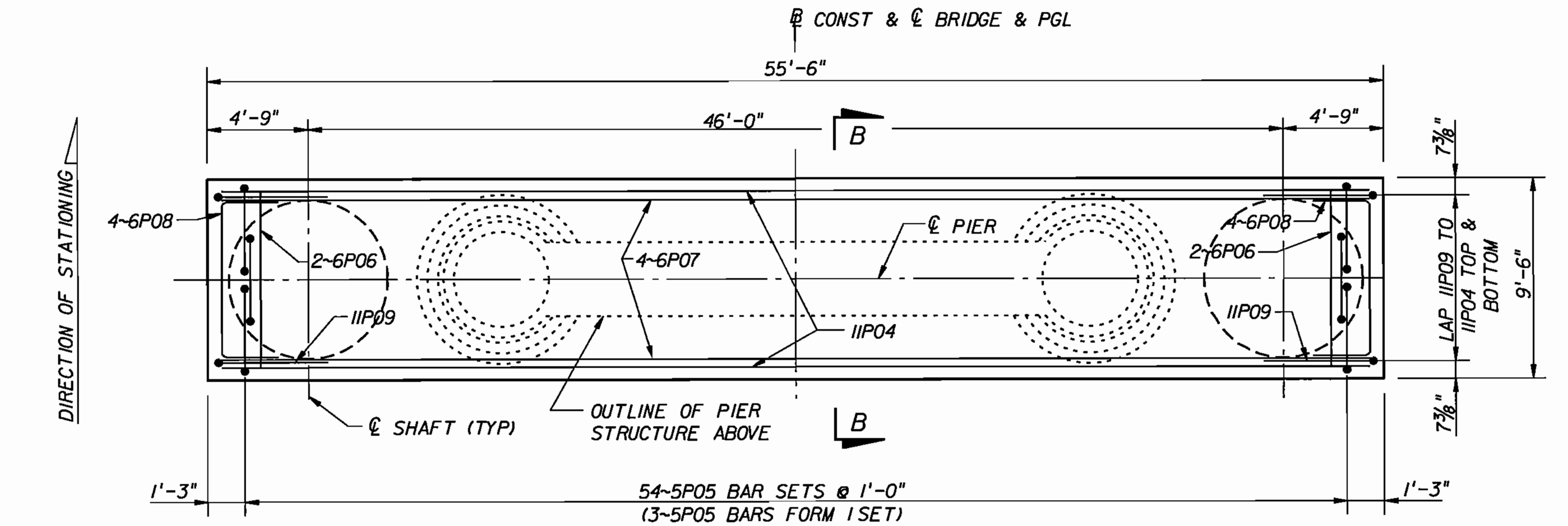
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: FOOTING REINFORCEMENT DETAILS MODIFIED PIER 19

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. B-56E

DGN$SPECIFICATION
DD-MMM-YYYY HH:MM



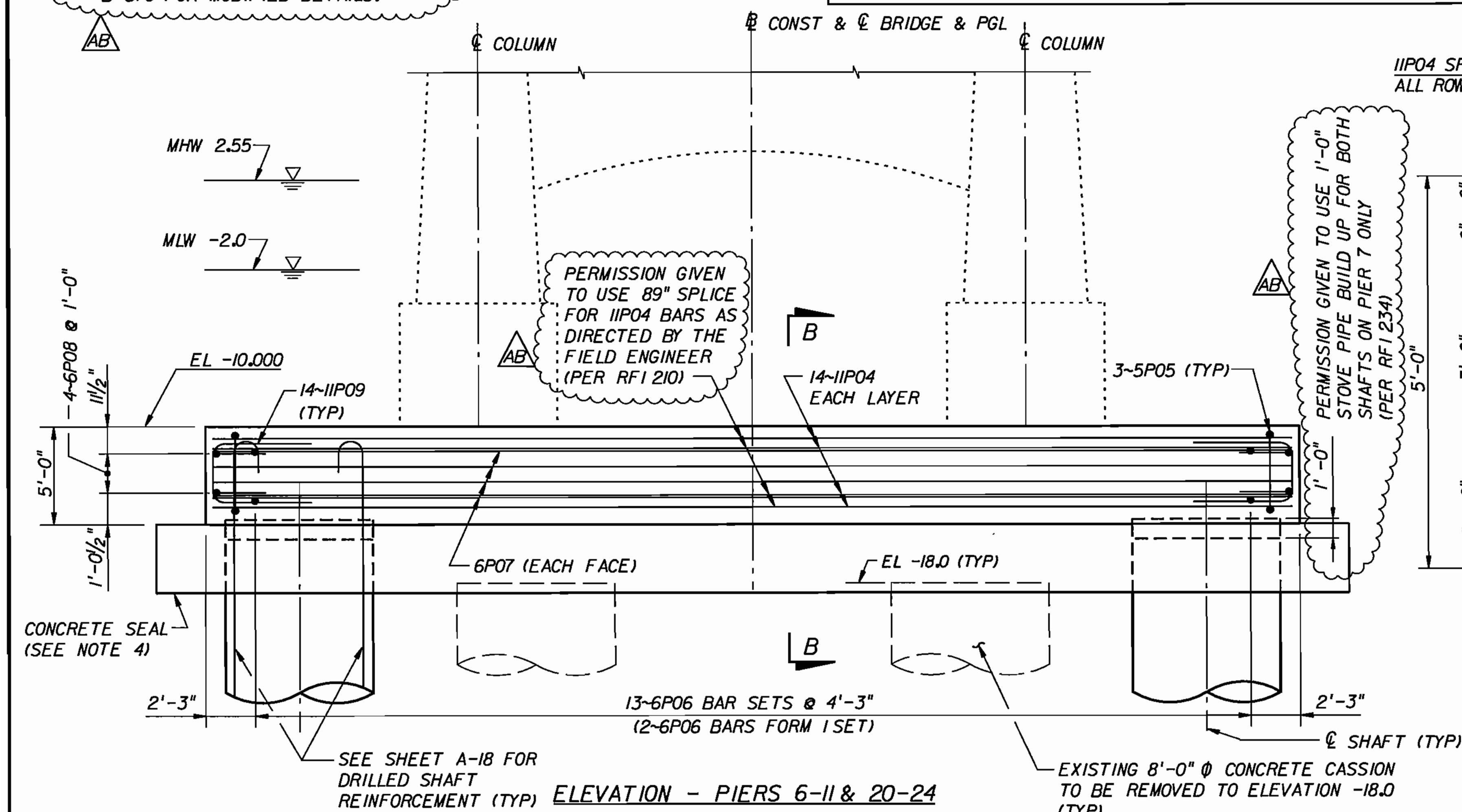
PLAN - PIERS 6-11 & 20-24

NOTES:

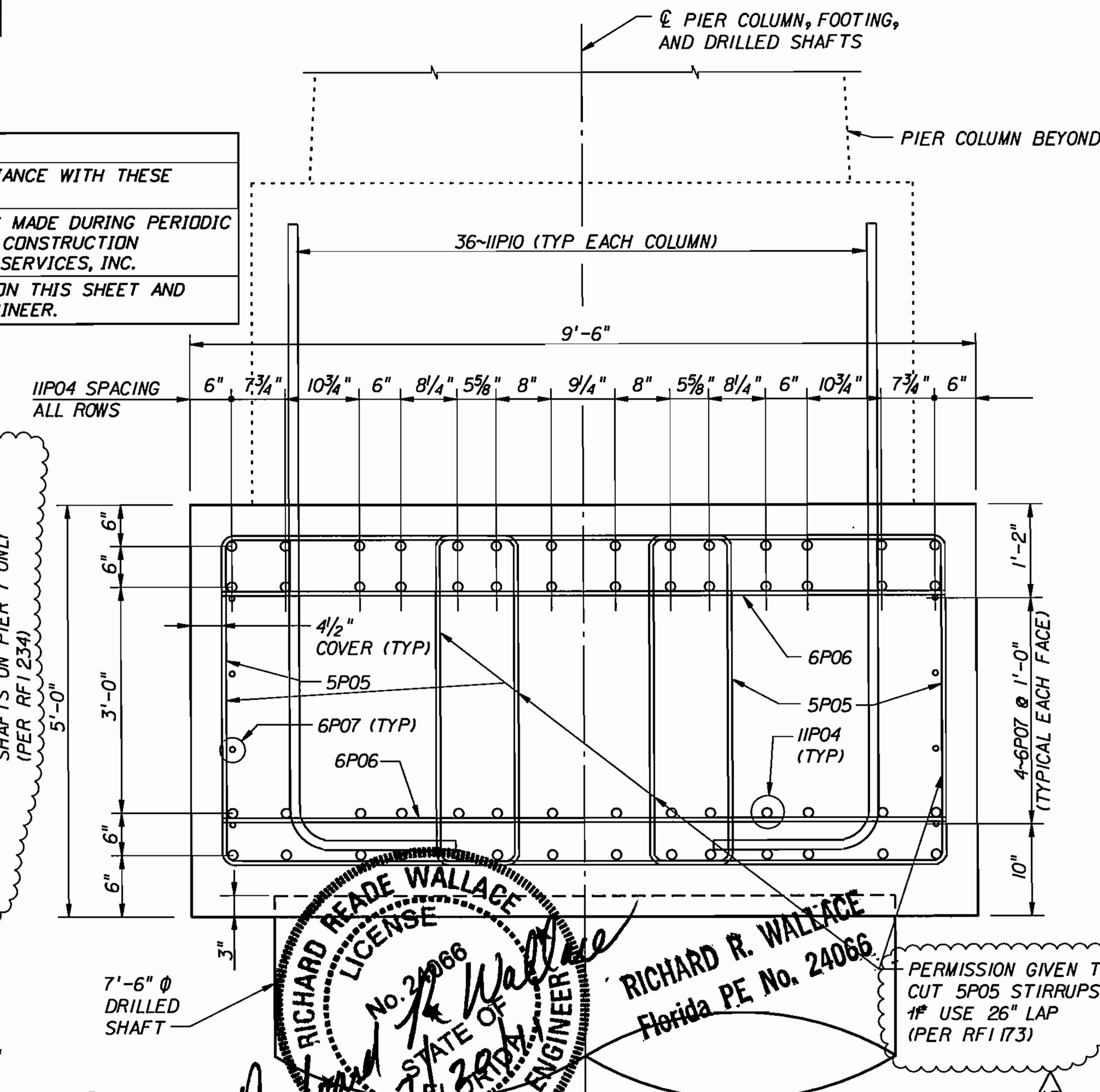
1. THE INFORMATION PRESENTED ON THIS SHEET DEPICTS THE TYPICAL SUBMERGED TWO SHAFT PIER FOOTING USED TO SUPPORT THE PIER COLUMNS.
2. 5P05, 11P04, AND 6P06 BARS MAY BE SHIFTED SLIGHTLY IN VICINITY OF DRILLED SHAFTS AND PIER COLUMNS TO AVOID CONFLICTS WITH REINFORCEMENT IN THOSE ELEMENTS.
3. PROVISIONS FOR MASS CONCRETE SHALL APPLY TO CONCRETE COLUMNS AND FOOTINGS FOR PIERS 6-10 AND 20-24.
4. NON-REINFORCED, CLASS III (SEAL) CONCRETE SEALS SHALL BE 3'-6" THICK FOR PIERS 6-11 AND 20-24. CONTRACTOR MAY PROVIDE ALTERNATE DESIGN FOR SEALS, SIGNED AND SEALED BY A PROFESSIONAL ENGINEER REGISTERED IN THE STATE OF FLORIDA.

AS-BUILT
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.
IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.

NOTE: FOOTING DIMENSIONS & SHAFT SPACING FOR PIERS 20, 21, & 23 HAVE BEEN REVISED. SEE SHEETS B-57A THRU B-57C FOR MODIFIED DETAILS.



ELEVATION - PIERS 6-11 & 20-24



SECTION B-B
(NOT TO SCALE)
(SEAL NOT SHOWN FOR CLARITY)

BRIDGE NO. 780074

•DGN$SPECIFICATION•••••

•DD-MMM-YYYY HH:MM•

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	PAW	03-03
CHECKED BY	GBL	06-03
DESIGNED BY	PAW	03-03
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Peter A. Winkler, Florida PE No. 56720

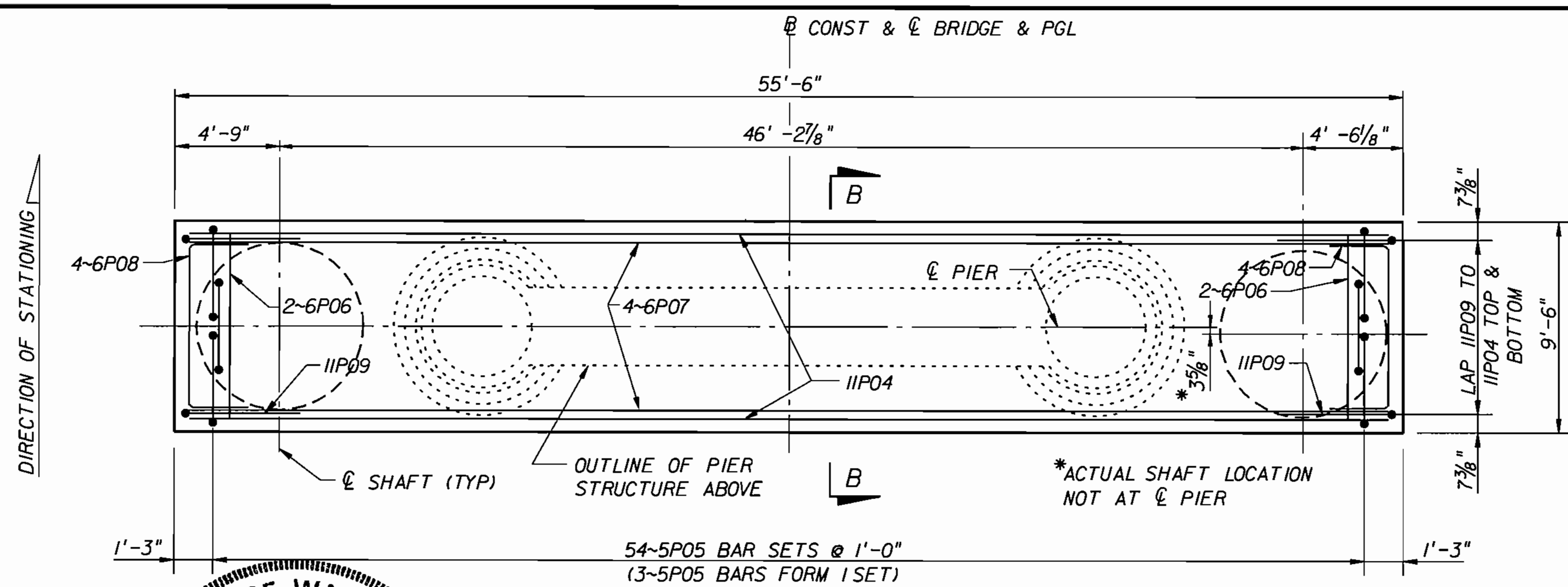
FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

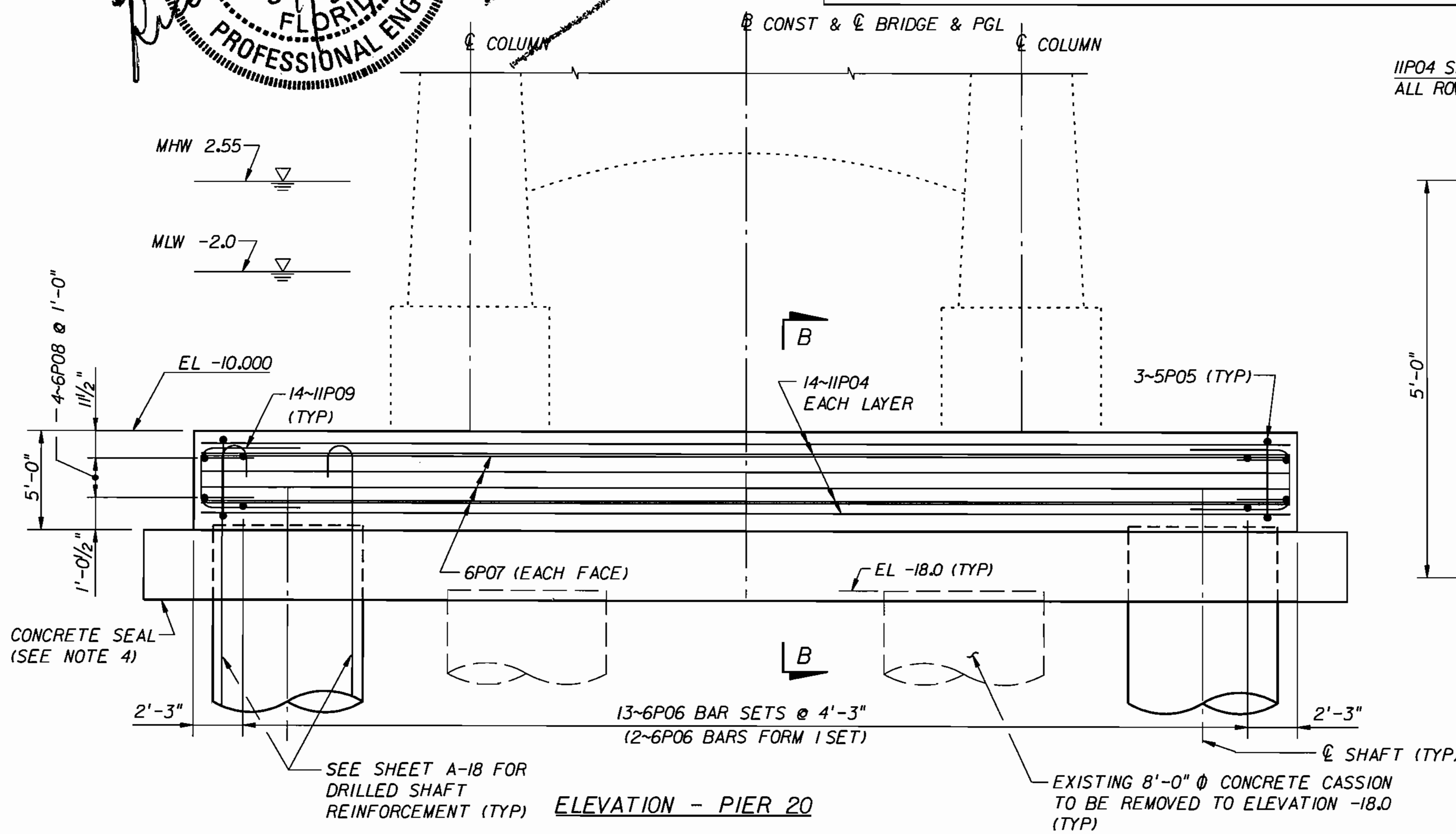
SHEET TITLE: FOOTING REINFORCEMENT DETAILS PIERS 6-11, 20-24

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

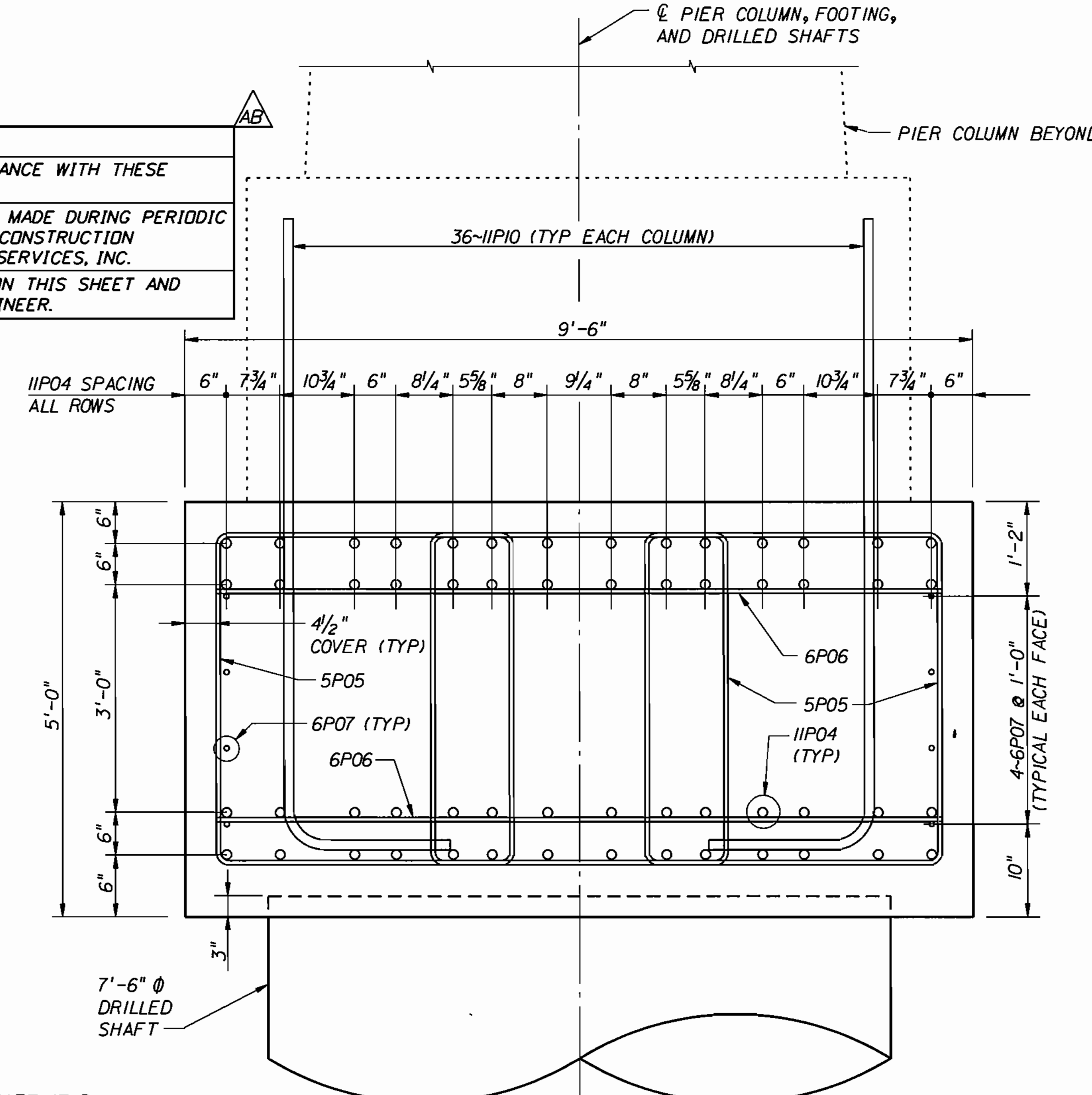
SHEET NO. B-57



PLAN - PIER 20



ELEVATION - PIER 20



SECTION B-B
(NOT TO SCALE)
(SEAL NOT SHOWN FOR CLARITY)

NOTES:

1. THE INFORMATION PRESENTED ON THIS SHEET DEPICTS THE MODIFIED SUBMERGED TWO SHAFT PIER FOOTING USED TO SUPPORT THE PIER COLUMNS AT PIER 20.
2. 5P05, 11P04, AND 6P06 BARS MAY BE SHIFTED SLIGHTLY IN VICINITY OF DRILLED SHAFTS AND PIER COLUMNS TO AVOID CONFLICTS WITH REINFORCEMENT IN THOSE ELEMENTS.
3. PROVISIONS FOR MASS CONCRETE SHALL APPLY TO CONCRETE COLUMNS AND FOOTINGS FOR PIERS 6-10 AND 20-24.
4. NON-REINFORCED, CLASS III (SEAL) CONCRETE SEALS SHALL BE 3'-6" THICK FOR PIERS 6-11 AND 20-24. CONTRACTOR MAY PROVIDE ALTERNATE DESIGN FOR SEALS, SIGNED AND SEALED BY A PROFESSIONAL ENGINEER REGISTERED IN THE STATE OF FLORIDA.

AB

AS-BUILT
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.
IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.

RICHARD READE WALLACE LICENSE No. 24066 STATE OF FLORIDA PROFESSIONAL ENGINEER

RICHARD R. WALLACE Florida PE No. 24066

Richard R Wallace 7/20/11

BRIDGE NO. 780074

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT - NEW SHEET (PER RFI 158)			

	NAMES	DATES
DRAWN BY	PAW	03-03
CHECKED BY	GBL	06-03
DESIGNED BY	PAW	03-03
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Peter A. Winkler, Florida PE No. 56720

FLORIDA DEPARTMENT OF TRANSPORTATION

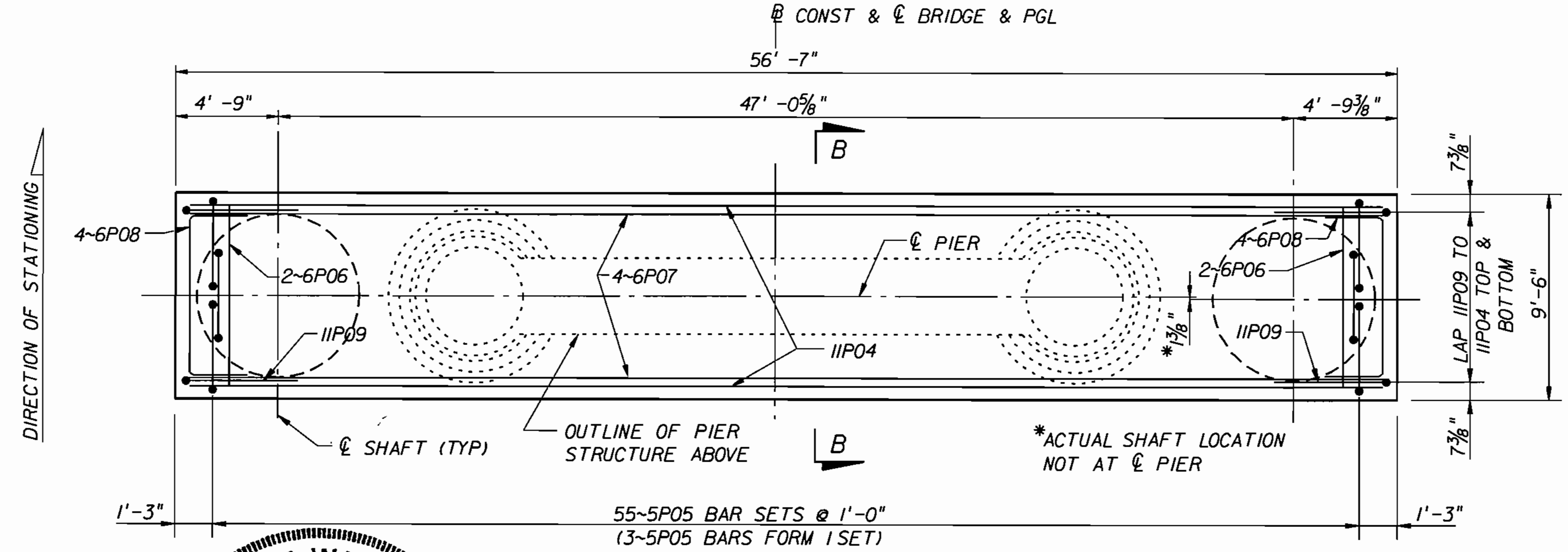
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: FOOTING REINFORCEMENT DETAILS MODIFIED PIER 20

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. B-57A

•DGN$SPECIFICATION•••••• •DD-MMM-YYYY HH:MM•

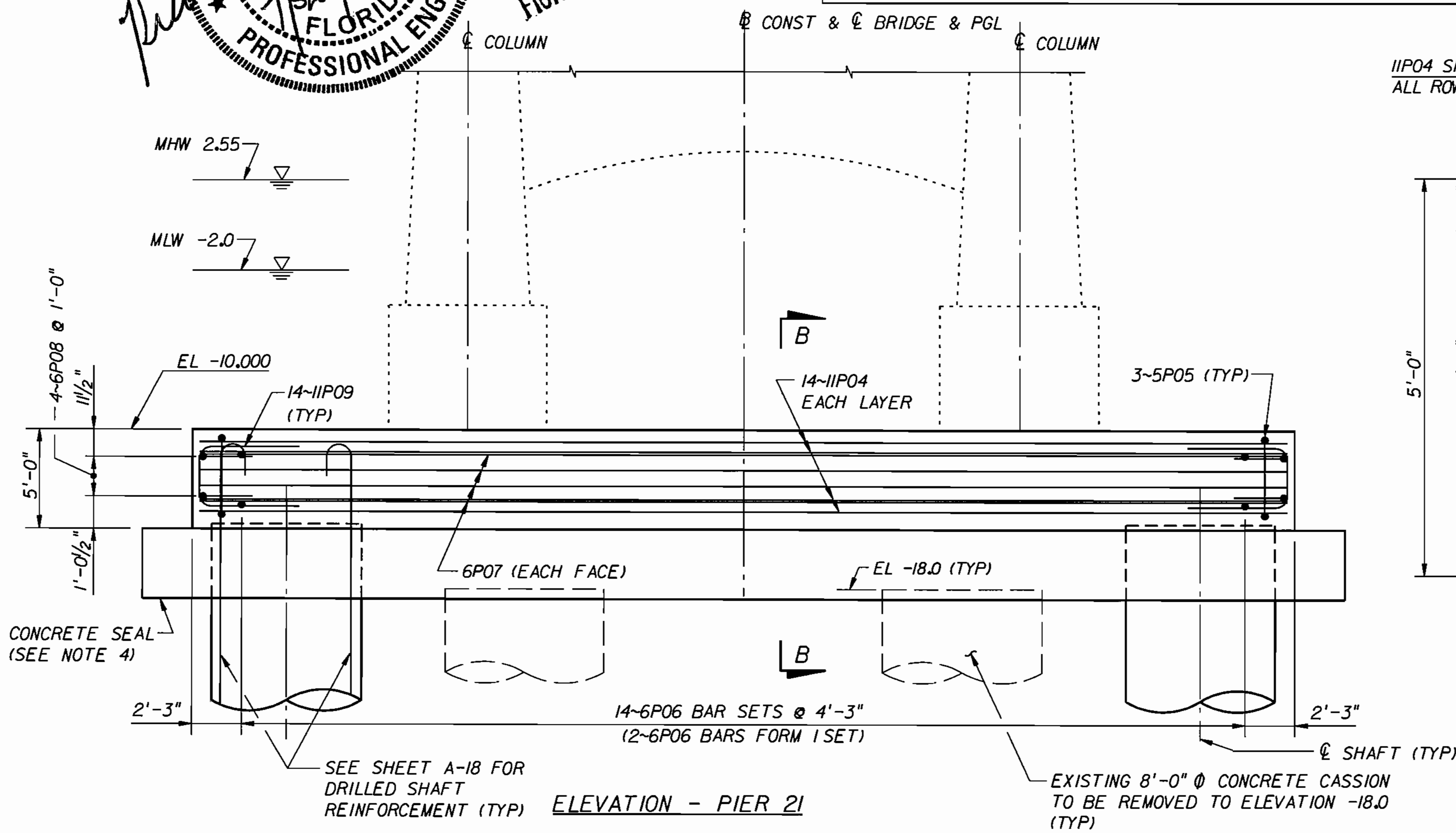


PLAN - PIER 21

RICHARD READE WALLACE
LICENSE
No. 24066
STATE OF
FLORIDA
PROFESSIONAL ENGINEER

RICHARD R. WALLACE
Florida PE No. 24066

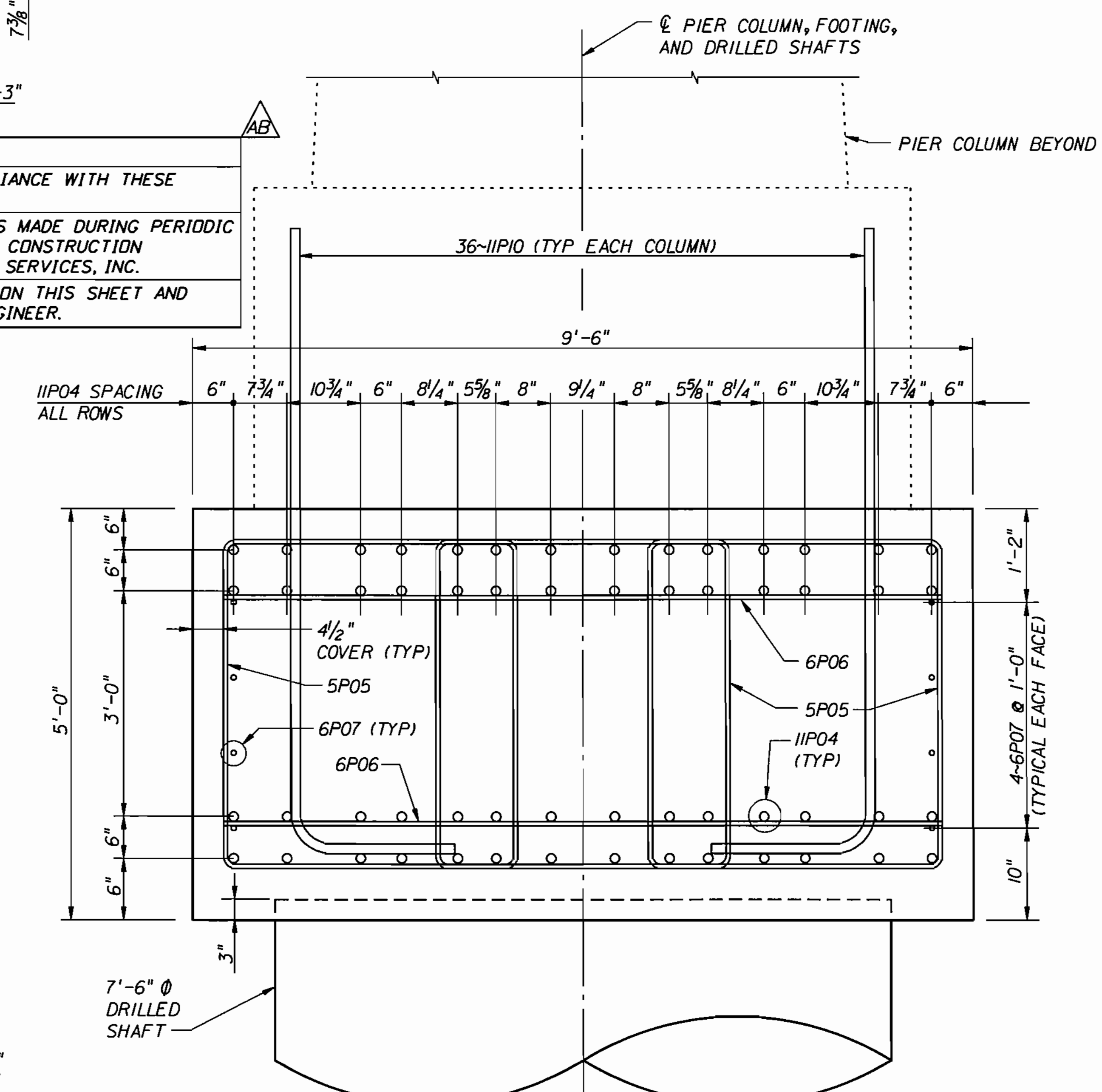
AS-BUILT
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.
IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.



ELEVATION - PIER 21

NOTES:

1. THE INFORMATION PRESENTED ON THIS SHEET DEPICTS THE MODIFIED SUBMERGED TWO SHAFT PIER FOOTING USED TO SUPPORT THE PIER COLUMNS AT PIER 21.
2. 5P05, 11P04, AND 6P06 BARS MAY BE SHIFTED SLIGHTLY IN VICINITY OF DRILLED SHAFTS AND PIER COLUMNS TO AVOID CONFLICTS WITH REINFORCEMENT IN THOSE ELEMENTS.
3. PROVISIONS FOR MASS CONCRETE SHALL APPLY TO CONCRETE COLUMNS AND FOOTINGS FOR PIERS 6-10 AND 20-24.
4. NON-REINFORCED, CLASS III (SEAL) CONCRETE SEALS SHALL BE 3'-6" THICK FOR PIERS 6-11 AND 20-24. CONTRACTOR MAY PROVIDE ALTERNATE DESIGN FOR SEALS, SIGNED AND SEALED BY A PROFESSIONAL ENGINEER REGISTERED IN THE STATE OF FLORIDA.



SECTION B-B
(NOT TO SCALE)
(SEAL NOT SHOWN FOR CLARITY)

BRIDGE NO. 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT - NEW SHEET (PER RFI 159)			

	NAMES	DATES
DRAWN BY	PAW	03-03
CHECKED BY	GBL	06-03
DESIGNED BY	PAW	03-03
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Peter A. Winkler, Florida PE No. 56720

FLORIDA DEPARTMENT OF TRANSPORTATION

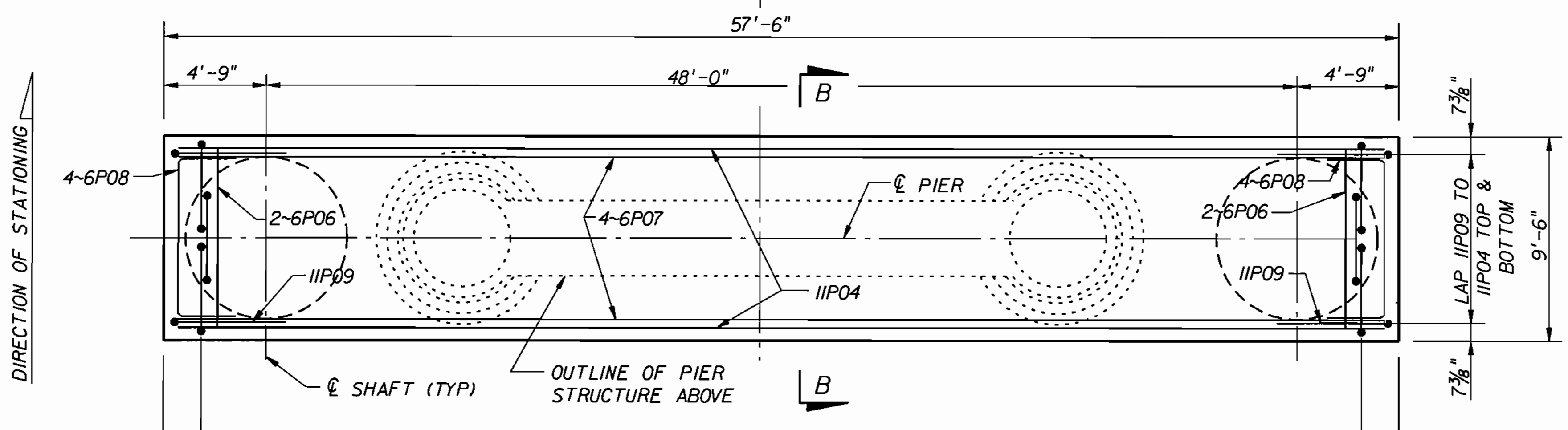
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: FOOTING REINFORCEMENT DETAILS MODIFIED PIER 21

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. B-57B

•DGN$SPECIFICATION•••••••
•DD-MMM-YYYY HH:MM•



PLAN - PIER 23

RICHARD READE WALLACE LICENSE No. 24066 STATE OF FLORIDA PROFESSIONAL ENGINEER

RICHARD R. WALLACE
Florida PE No. 24066

AS-BUILT

AS-BUILT
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
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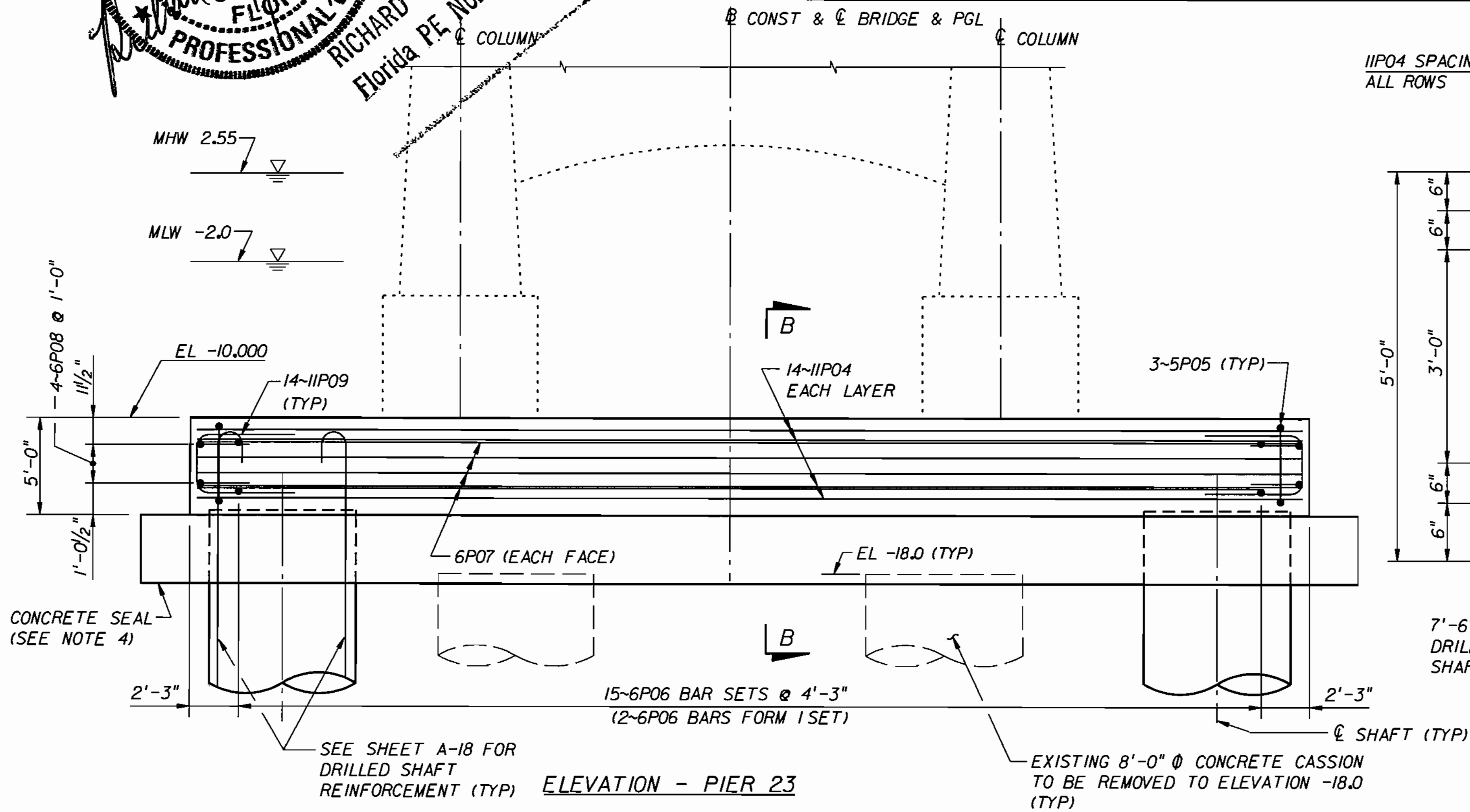
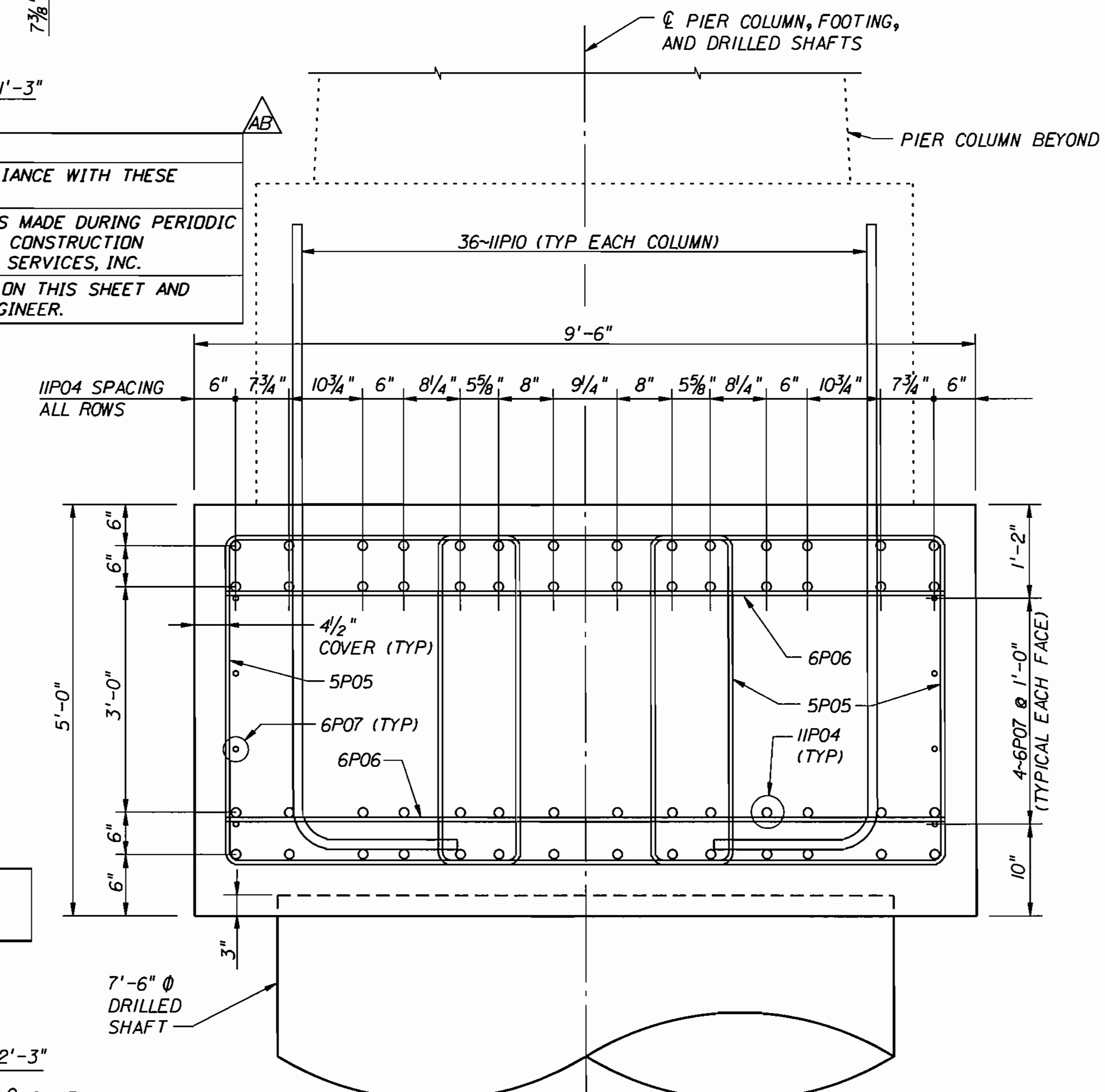


ELEVATION - PIER 23

NOTES:

1. THE INFORMATION PRESENTED ON THIS SHEET DEPICTS THE MODIFIED SUBMERGED TWO SHAFT PIER FOOTING USED TO SUPPORT THE PIER COLUMNS AT PIER 23.
2. 5P05, 11P04, AND 6P06 BARS MAY BE SHIFTED SLIGHTLY IN VICINITY OF DRILLED SHAFTS AND PIER COLUMNS TO AVOID CONFLICTS WITH REINFORCEMENT IN THOSE ELEMENTS.
3. PROVISIONS FOR MASS CONCRETE SHALL APPLY TO CONCRETE COLUMNS AND FOOTINGS FOR PIERS 6-10 AND 20-24.
4. NON-REINFORCED, CLASS III (SEAL) CONCRETE SEALS SHALL BE 3'-6" THICK FOR PIERS 6-11 AND 20-24. CONTRACTOR MAY PROVIDE ALTERNATE DESIGN FOR SEALS, SIGNED AND SEALED BY A PROFESSIONAL ENGINEER REGISTERED IN THE STATE OF FLORIDA.



SECTION B-B
(NOT TO SCALE)
(SEAL NOT SHOWN FOR CLARITY)

BRIDGE NO. 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
7-20-11	RRW	AB AS-BUILT - NEW SHEET (PER DOCUMENT DATED 1-12-2007)			

	NAMES	DATES
DRAWN BY	PAW	03-03
CHECKED BY	GBL	06-03
DESIGNED BY	PAW	03-03
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Peter A. Winkler, Florida PE No. 56720

FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: FOOTING REINFORCEMENT DETAILS
MODIFIED PIER 23

PROJECT NAME: PERMANENT BRIDGE
BRIDGE OF LIONS REHABILITATION

SHEET NO. B-57C

NOTES:

1. FOR STANDARD PENETRATION TEST (SPT) BORING LOCATIONS SEE SHEETS B-10 AND B-27.

2. FOR DRILLED SHAFT DATA AND DETAILS SEE SHEET B-29.

3. REMOVAL AND RESTORATION OF PARTIAL EXISTING PIER WALL AND FOOTING ARE ANTICIPATED FOR INSTALLATION OF DRILLED SHAFTS 4 & 5 AT PIER 15 AND 1 & 2 AT PIER 16. SEE SHEET BP12 FOR DETAILS.

LEGEND:

- 5'-0" ⌀ CONCRETE COLUMN WITH ½" THICK STEEL CASING ON 8'-0" DRILLED SHAFT.
- 8'-0' ⌀ DRILLED SHAFT WITH 1" THICK STEEL CASING
- (#) DRILLED SHAFT NUMBER

GARY ALAN KLEVENS LICENSE No 47187 STATE OF FLORIDA PROFESSIONAL ENGINEER 07/19/11

AS-BUILT

THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.

AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.

BRIDGE NO. 780074

7/19/2011 3:36:58 PM $FILEABBREV$

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
07/20/11	JH	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	CLP	08/03
CHECKED BY	RWM	08/03
DESIGNED BY	JH	08/03
CHECKED BY	RWM	08/03
APPROVED BY	G.A. KLEVENS P.E.	

ENGINEER OF RECORD:
LICHTENSTEIN CONSULTING ENGINEERS, INC.
2700 W. Cypress Creek Rd., Suite D-140
Fort Lauderdale, Florida 33309
Authorization No. 00006065
G. Alan Klevens, P.E.
P.E. License No. 47187

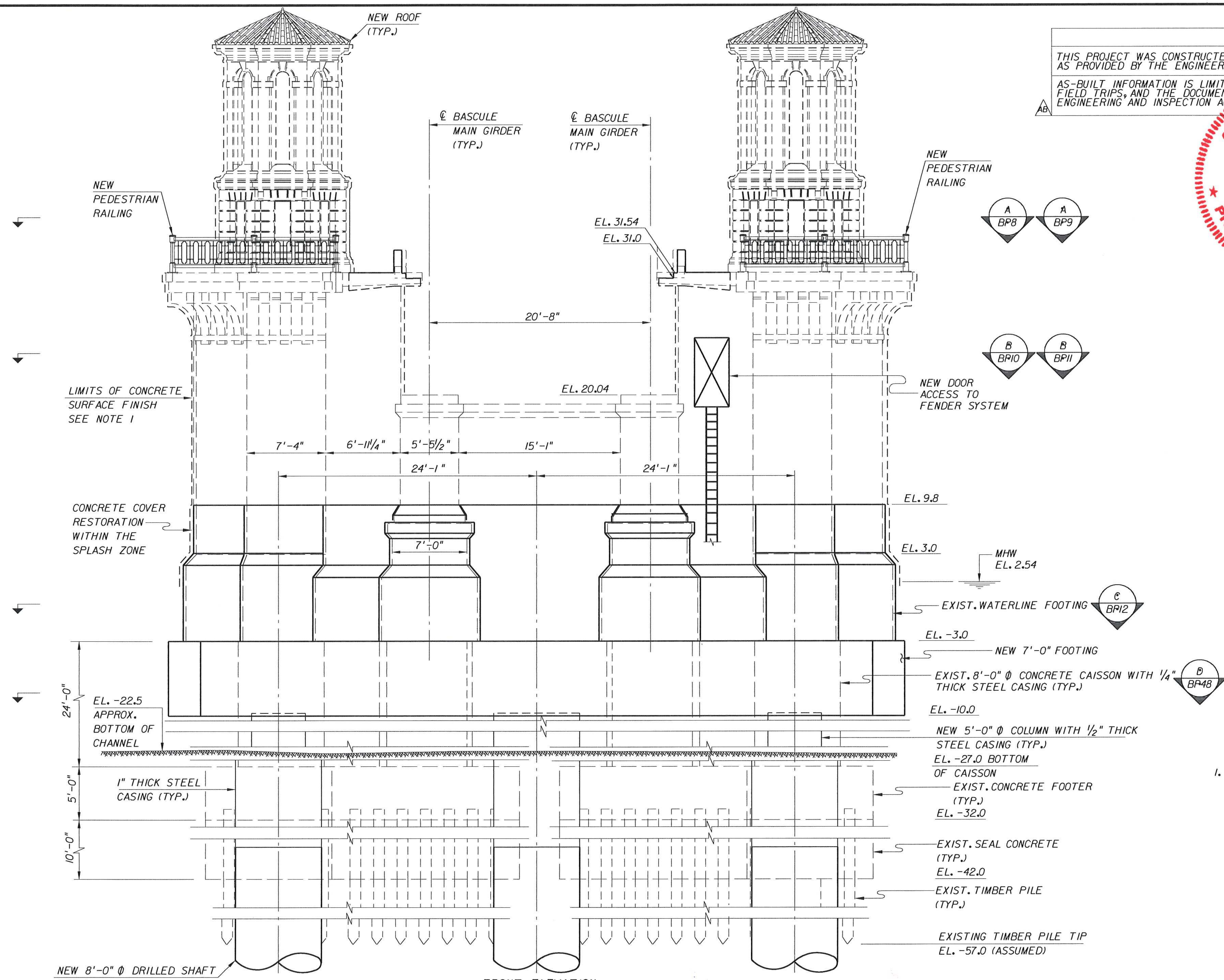
FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: FOUNDATION LAYOUT FOR PIERS 15 AND 16

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. BP3



FRONT ELEVATION

AS-BUILT

THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.

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GARY ALAN KLEVENS LICENSE No 47187 STATE OF FLORIDA PROFESSIONAL ENGINEER 07/20/11

NOTES:

1. APPLY CONCRETE SURFACE FINISH TO ALL EXTERIOR SURFACES ABOVE THE MEAN HIGH WATER LINE AS SEEN IN ANY ELEVATION AND AT THE DIRECTION OF THE ENGINEER.

LEGEND:

——— NEW

– – – EXISTING

BRIDGE NO. 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
07/20/11	JH	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	CLP	08/03
CHECKED BY	RWM	08/03
DESIGNED BY	JH	08/03
CHECKED BY	RWM	08/03
APPROVED BY	G.A. KLEVENS P.E.	

ENGINEER OF RECORD:
LICHTENSTEIN CONSULTING ENGINEERS, INC.
2700 W. Cypress Creek Rd., Suite D-140
Fort Lauderdale, Florida 33309
Authorization No. 00006065
G. Alan Klevens, P.E.
P.E. License No. 47187

FLORIDA DEPARTMENT OF TRANSPORTATION

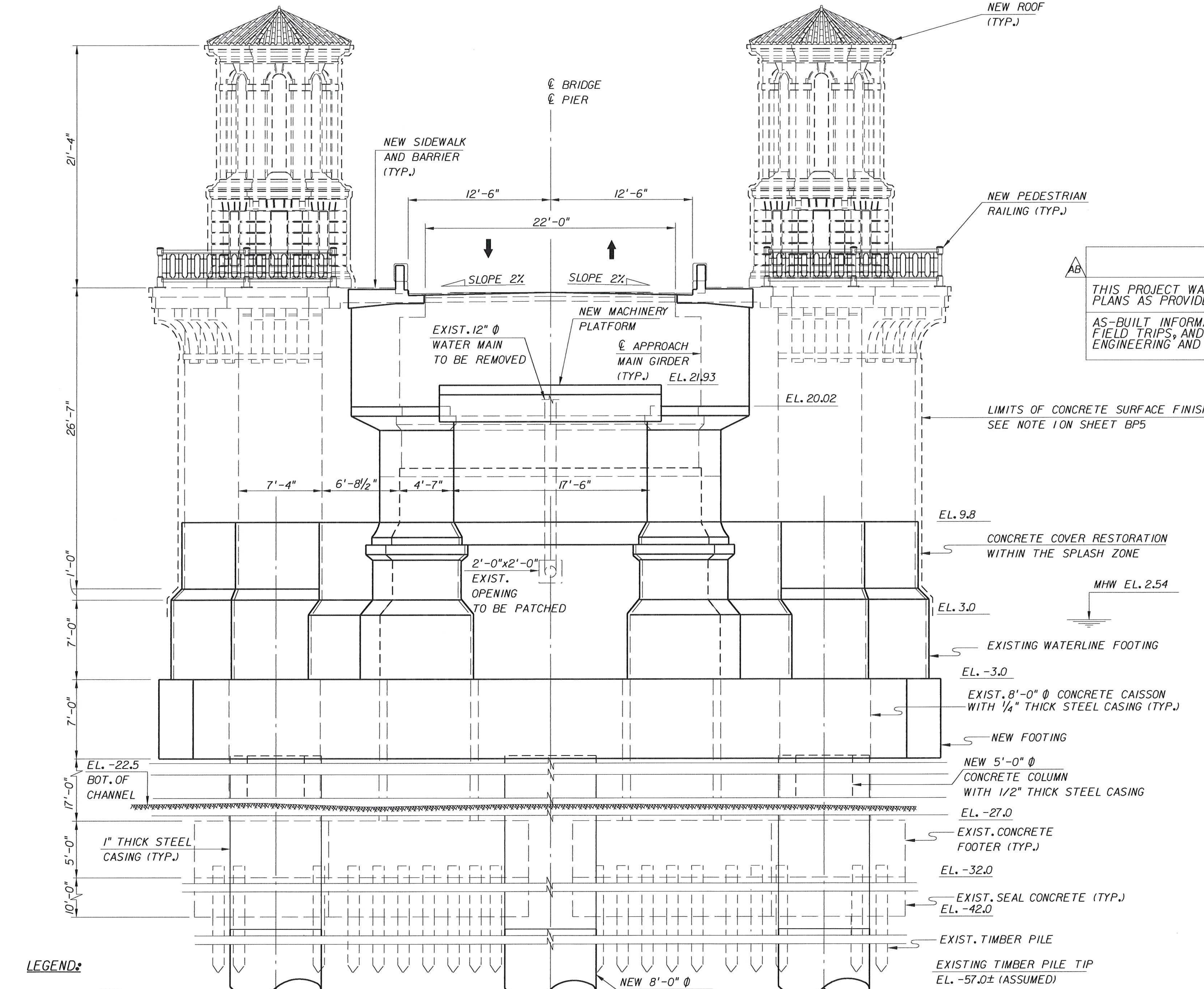
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: FRONT ELEVATION OF BASCULE PIERS

PROJECT NAME: PERMANENT BRIDGE
BRIDGE OF LIONS REHABILITATION

SHEET NO. BP5

7/19/2011 3:40:33 PM $FILEABBREV$



LEGEND:

——— NEW

– – – EXISTING

REAR ELEVATION

(SUPERSTRUCTURE NOT SHOWN FOR CLARITY)

AB	AS-BUILT
	THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
	AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.

GARY ALAN KLEVENS LICENSE No. 47187 STATE OF FLORIDA PROFESSIONAL ENGINEER 7/19/2011

BRIDGE NO. 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
07/20/11	JH	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	CLP	08/03
CHECKED BY	RWM	08/03
DESIGNED BY	JH	08/03
CHECKED BY	RWM	08/03
APPROVED BY	G.A. KLEVENS P.E.	

ENGINEER OF RECORD:
LICHTENSTEIN CONSULTING ENGINEERS, INC.
2700 W. Cypress Creek Rd., Suite D-140
Fort Lauderdale, Florida 33309
Authorization No. 00006065
G. Alan Klevens, P.E.
P.E. License No. 47187

FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: REAR ELEVATION OF BASCULE PIERS

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. BP6

7/19/2011 3:41:19 PM $FILEABBREV$

AS-BUILT

THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.

AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.

GARY ALAN KLEVENS LICENSE No. 47187 STATE OF FLORIDA PROFESSIONAL ENGINEER 7/20/11

SIDE ELEVATIONS

(SUPERSTRUCTURE NOT SHOWN FOR CLARITY)

BRIDGE NO. 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
07/20/11	JH	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	CLP	08/03
CHECKED BY	RWM	08/03
DESIGNED BY	JH	08/03
CHECKED BY	RWM	08/03
APPROVED BY	G.A. KLEVENS P.E.	

ENGINEER OF RECORD:
LICHTENSTEIN CONSULTING ENGINEERS, INC.
2700 W. Cypress Creek Rd., Suite D-140
Fort Lauderdale, Florida 33309
Authorization No. 00006065
G. Alan Klevens, P.E.
P.E. License No. 47187

FLORIDA DEPARTMENT OF TRANSPORTATION

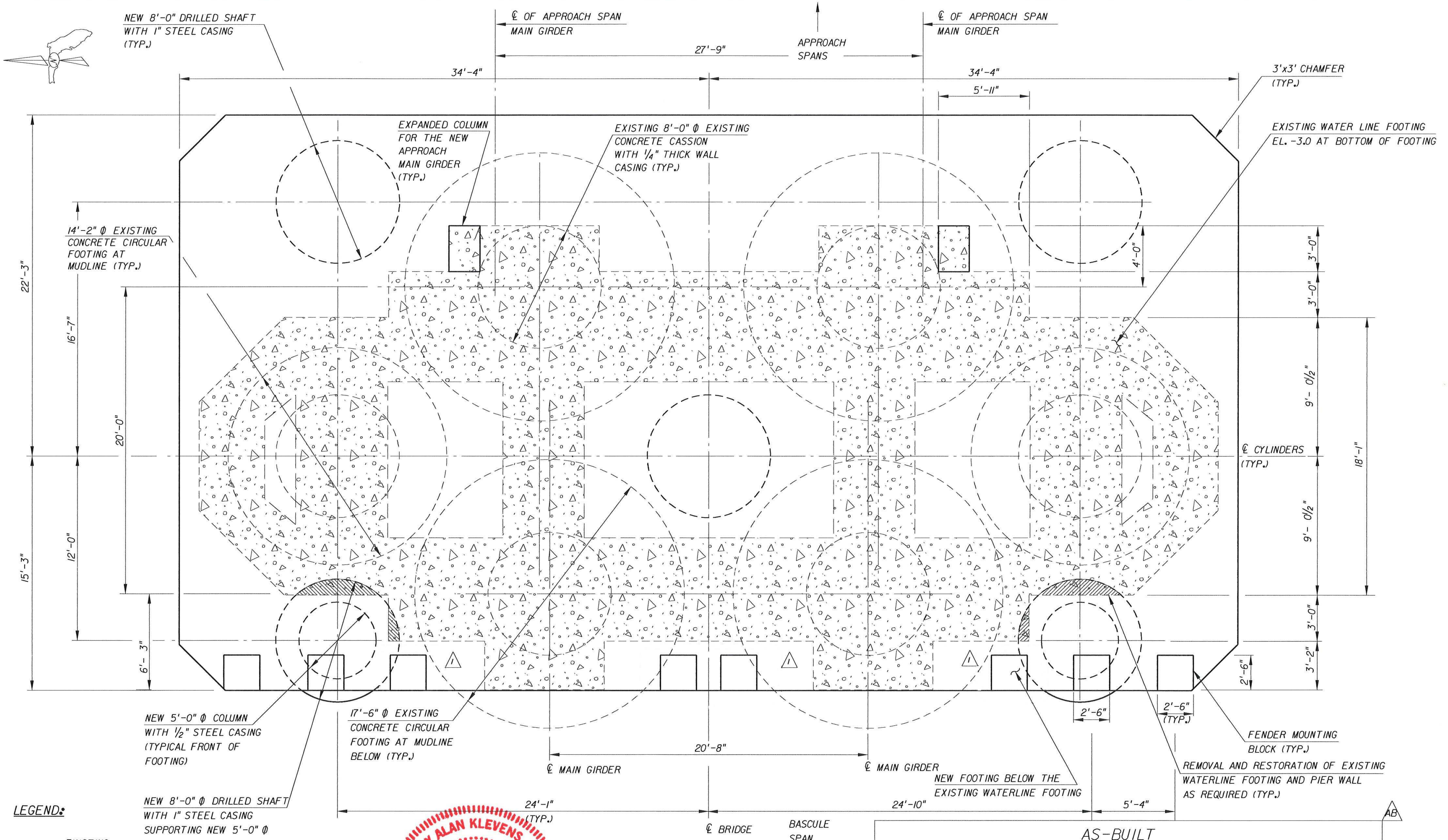
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: SIDE ELEVATIONS OF BASCULE PIERS

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. BP7

7/19/2011 3:41:43 PM $FILEABBREV$



LEGEND:

- — — — EXISTING
- ——— PROPOSED
- · — — — PROPOSED (HIDDEN)

SECTION ℄ / BP5

(EAST PIER SHOWN, WEST PIER OPPOSITE HAND)
(EXISTING TIMBER PILES UNDER CIRCULAR FOOTINGS NOT SHOWN)

AB

AS-BUILT

THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.

AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.

GARY ALAN KLEVENS
LICENSE
No. 47187
STATE OF FLORIDA
PROFESSIONAL ENGINEER
07/20/11

BRIDGE NO. 780074

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
03/19/04	RWM	1 ADDED FENDER MOUNTING BLOCKS			
07/20/11	JH	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	CLP	08/03
CHECKED BY	RWM	08/03
DESIGNED BY	JH	08/03
CHECKED BY	RWM	08/03
APPROVED BY	G.A. KLEVENS P.E.	

ENGINEER OF RECORD:
LICHTENSTEIN CONSULTING ENGINEERS, INC.
2700 W. Cypress Creek Rd., Suite D-140
Fort Lauderdale, Florida 33309
Authorization No. 00006065
G. Alan Klevens, P.E.
P.E. License No. 47187

FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: BASCULE PIER NEW WATERLINE FOOTING

PROJECT NAME: PERMANENT BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. BP12

7/19/2011 4:05:56 PM $FILEABBREV$

PILE DATA TABLE

PIER OR BENT NO.	PILE SIZE (IN)	REQ'D* DRV. RESIST (TONS)	TENSION CAPACITY (TONS)	MIN** TIP ELEV (FT)	TEST PILE LENGTH (FT)	REQ'D JET ELEV (FT)	REQ'D *** PREFORM ELEV (FT)	FACT'D DES LOAD (TONS)	DOWN DRAG (TONS)	TOTAL SCOUR RESIST (TONS)	NET SCOUR RESIST (TONS)	LONG-TERM SCOUR ELEV (FT)	20-YEAR SCOUR ELEV (FT)	φ	PILE CUT-OFF ELEVATION
INSTALLATION CRITERIA								DESIGN CRITERIA							
BENT 4	24	169	56	-65	85	N/A	-64 △3	113	N/A	4.8	4.8	N/A	-9.13	0.7	1.939
BENT 5	24	162	56	-72	N/A	N/A	N/A	113	N/A	0	0	N/A	-16.77	0.7	4.354
BENT 6	24	167	58	-75	105	N/A	N/A	113	N/A	3.3	3.3	N/A	-19.90	0.7	6.769
BENT 7	24	167	58	-80	N/A	N/A	N/A	113	N/A	3.3	3.3	N/A	-24.01	0.7	9.184
BENT 8	24	163	49	-77	100	N/A	N/A	113	N/A	1.1	1.1	N/A	-21.11	0.7	11.563
BENT 9	24	163	49	-78	N/A	N/A	N/A	113	N/A	1.1	1.1	N/A	-22.24	0.7	14.133
BENT 10	24	165	59	-81	120	N/A	N/A	113	N/A	1.9	1.9	N/A	-25.54	0.7	16.697
BENT 11	24	165	59	-84	N/A	N/A	N/A	113	N/A	1.9	1.9	N/A	-28.85	0.7	19.155
BENT 12	24	170	55	-88	130	N/A	N/A	113	N/A	5.8	5.8	N/A	-32.15	0.7	21.611
BENT 13	24	170	55	-86	N/A	N/A	N/A	113	N/A	5.8	5.8	N/A	-30.45	0.7	23.727
BENT 14	24	167	56	-84	130	N/A	N/A	113	N/A	3.9	3.9	N/A	-28.40	0.7	24.186
BENT 17	24	175	55	-90	135	N/A	N/A	113	N/A	9.1	9.1	N/A	-34.44	0.7	24.147
BENT 18	24	165	59	-87	N/A	N/A	N/A	113	N/A	1.9	1.9	N/A	-31.91	0.7	23.680
BENT 19	24	170	44	-88	110	N/A	N/A	113	N/A	5.5	5.5	N/A	-32.86	0.7	21.589
BENT 20	24	170	44	-87	N/A	N/A	N/A	113	N/A	5.5	5.5	N/A	-31.57	0.7	19.133
BENT 21	24	170	57	-87	120	N/A	-65 △3	113	N/A	5.4	5.4	N/A	-31.39	0.7	16.578
BENT 22	24	170	57	-89	N/A	N/A	-65 △3	113	N/A	5.4	5.4	N/A	-33.78	0.7	14.023
BENT 23	24	170	57	-82	115	N/A	-65 △3	113	N/A	5.4	5.4	N/A	-26.77	0.7	11.468
BENT 24	24	176	58	-76	N/A	N/A	N/A	113	N/A	9.9	9.9	N/A	-20.33	0.7	9.012
BENT 25	24	168	58	-69	110	N/A	N/A	113	N/A	4.1	4.1	N/A	-13.09	0.7	6.550

$$\text{* REQUIRED DRIVING RESISTANCE} = \frac{\text{FACTORED DESIGN LOAD + NET SCOUR + DOWNDRAG}}{\phi}$$

** MINIMUM TIP ELEVATION IN REQUIRED FOR LATERAL STABILITY.

△3 *** SEE NOTE 14.

DRILLED SHAFT DATA TABLE

PIER OR BENT NO.	SHAFT SIZE (IN)	TIP ELEV (FT)	MIN TIP ELEV (FT)	MIN ROCK SOCKET LENGTH (FT)	FACT'D DESIGN LOAD (TONS)	DOWN DRAG (TONS)	LONG-TERM SCOUR ELEV (FT)	20-YEAR SCOUR ELEV (FT)	φ	TOP OF SHAFT ELEVATION: DRILLED SHAFT NO. 1 (FT)	DRILLED SHAFT NO. 2 (FT)	DRILLED SHAFT NO. 3 (FT)
INSTALLATION CRITERIA					DESIGN CRITERIA							
END BENT 1	36	-72	-72	N/A	180	N/A	N/A	1.62	0.55	4.442	4.202	3.962
BENT 2	36	-72	-72	N/A	180	N/A	N/A	0.29	0.55	5.160	4.920	4.680
BENT 3	36	-72	-72	N/A	180	N/A	N/A	-9.61	0.55	6.194	5.954	5.714
BENT 26	36	-91	-84	N/A	192	N/A	N/A	-12.27	0.55	10.799	10.733	10.667
BENT 27	36	-91	-84	N/A	192	N/A	N/A	-10.70	0.55	9.399	9.333	9.267
END BENT 28	36	-91	-84	N/A	192	N/A	N/A	-6.27	0.55	7.997	8.177	8.357

NOTES:

1. FOR BENTS 4-14 AND 17-25, ALL PILES SHALL BE 24" SQUARE PRESTRESSED CONCRETE PILES.
2. FOR PRESTRESSED CONCRETE PILE DETAILS, SEE SHEETS A-14 THRU A-16.
3. SEE SHEET CL-6 FOR BENT 15 AND 16 PILE DATA TABLE.
4. A DYNAMIC LOAD TEST SHALL BE PERFORMED ON ALL TEST PILES AS PER SECTION 455 OF THE SPECIFICATIONS. THE DYNAMIC LOAD TEST SHALL BE INCLUDED IN THE CONTRACT UNIT PRICE FOR BID ITEM 455-137 (DYNAMIC TEST LOAD).
5. THE PENETRATION REQUIREMENTS SHALL BE ACHIEVED AS PER SECTION 455-5.8 OF THE FDOT 455 SPECIFICATIONS.
6. THE CONTRACTOR SHALL BE RESPONSIBLE TO VERIFY LOCATIONS OF ALL UTILITIES PRIOR TO DRIVING PILES AND INSTALLING DRILLED SHAFTS. FOR ADDITIONAL UTILITY INFORMATION, SEE UTILITY ADJUSTMENT SHEETS.
7. THE CONTRACTOR SHOULD ANTICIPATE THAT SET-CHECKS AND RE-DRIVES MAY BE REQUIRED TO REACH THE REQUIRED DRIVING RESISTANCE. PAYMENT FOR SET-CHECKS AND RE-DRIVES AND THE TIME INVOLVED IN WAITING TO PERFORM THEM SHALL BE MADE AS PROVIDED IN SECTION 455-5.10.4.
8. JETTING WILL NOT BE PERMITTED FOR THIS PROJECT.
9. REFER TO SPECIFICATION SECTION 455-2 (AND THE TECHNICAL SPECIAL PROVISIONS, SECTION 455-2.11, OSTERBERG CELL LOAD TESTING OF DRILLED SHAFTS (TEMPORARY BRIDGE) PREPARED FOR THIS PROJECT FOR SHAFT TESTING PROCEDURES. THE COST OF ALL TEST EQUIPMENT, LABOR, AND MATERIALS SHALL BE INCLUDED IN PAY ITEM 455-101-1 "LOAD TEST (OSTERBERG CELL) (LESS THAN FIVE CELLS)".
10. "NON-PRODUCTION" TEST SHAFTS SHALL BE CONSTRUCTED AND REINFORCED TO THE SAME SPECIFICATIONS AS THE PRODUCTION SHAFTS, WITH THE EXCEPTIONS NOTED IN THE TECHNICAL SPECIAL PROVISIONS FOR OSTERBERG CELL LOAD TESTING OF DRILLED SHAFTS (TEMPORARY BRIDGE), SECTION 455-2.11 STATIC COMPRESSION LOAD TESTS, PREPARED FOR THIS PROJECT. THE REQUIREMENTS OF SPECIFICATION 455-18 TEST HOLES SHALL APPLY, UNO.
11. THE CONTRACTOR SHALL DRILL ONE PILOT HOLE (STANDARD PENETRATION TEST [SPT] BORING) AT ONE DRILLED SHAFT LOCATION PER FOUNDATION UNIT, AT LEAST 14 DAYS BEFORE THE EXCAVATION OF THE FOUNDATION UNIT SHAFTS BEGIN. THE COST SHALL BE INCLUDED WITH THE RESPECTIVE DRILLED SHAFT. DETAILED REQUIREMENTS OF THE PILOT HOLE PROGRAM (INCLUDING THE LOCATIONS AT WHICH PILOT HOLES ARE TO BE DRILLED, THE MINIMUM BORING TERMINATION ELEVATIONS, AND THE LOWEST ELEVATION AT WHICH SPT SAMPLING SHALL COMMENCE), ARE PRESENTED IN THE TECHNICAL SPECIAL PROVISIONS, SECTION 455-15.6, DRILLED SHAFT PILOT HOLES (PERMANENT BRIDGE) PREPARED FOR THIS PROJECT.
12. TIP ELEVATION SHOWN IN THE TABLE IS THE ELEVATION TO WHICH THE SHAFT SHALL BE CONSTRUCTED UNLESS LOAD TEST DATA OR OTHER GEOTECHNICAL DATA OBTAINED DURING CONSTRUCTION ALLOWS THE ENGINEER TO AUTHORIZE A DIFFERENT TIP ELEVATION. HOWEVER, MINIMUM TIP ELEVATION IS THE HIGHEST ELEVATION THAT THE SHAFT TIP SHALL BE CONSTRUCTED TO.
13. FOR ADDITIONAL NOTES ON DRILLED SHAFT INSTALLATION, SEE SHEET B-29 & B-29A FOR THE PERMANENT BRIDGE.
14. △3 THE CONTRACTOR SHALL USE SPECIAL EQUIPMENT AND/OR METHODS (I.E., CORE BARRELS, ROCK AUGERS, PUNCHES, DRILL BITS, ETC.) AS NEEDED, TO FACILITATE PREFORMING. FURTHERMORE, HARD LAYERS MAY BE ENCOUNTERED ABOVE THE REQUIRED MINIMUM TIP ELEVATIONS (AND PREFORMING MAY BE NECESSARY TO ACHIEVE THE REQUIRED MINIMUM TIP ELEVATIONS) AT PILE BENT LOCATIONS OTHER THAN THOSE WHICH HAVE REQUIRED PREFORM ELEVATIONS LISTED IN THE PILE DATA TABLE.

NOT APPLICABLE - BRIDGE REMOVED

BRIDGE NO. TEMP 780074

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
4-9-04	JRH	△3 ADDED PREFORM ELEVATIONS & NOTE 14			

	NAMES	DATES
DRAWN BY	JKP	04-03
CHECKED BY	GBL	04-03
DESIGNED BY	GBL	04-03
CHECKED BY	RGW	04-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Robert G. Woodruff, Florida PE No. 57099

FLORIDA DEPARTMENT OF TRANSPORTATION

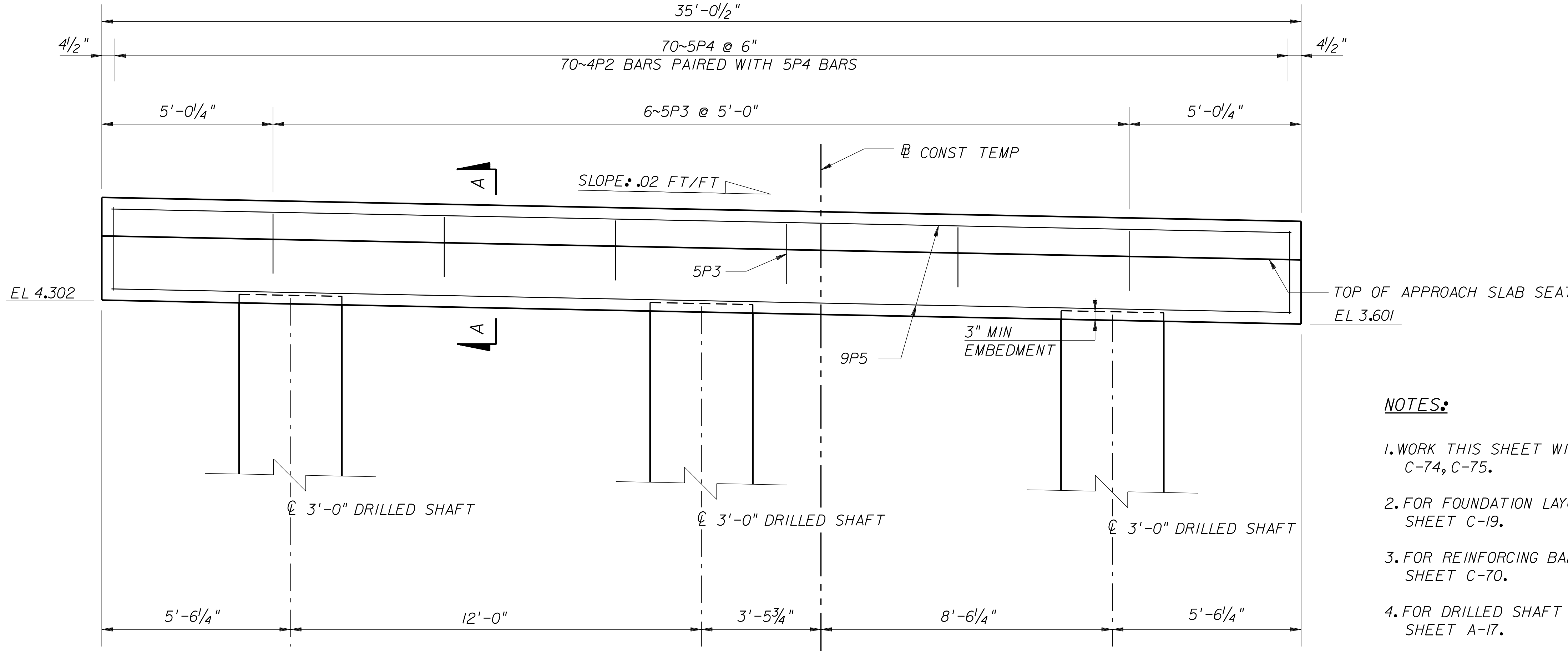
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: PILE DATA AND SHAFT DATA

PROJECT NAME: TEMPORARY BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. C-24

DD-MMM-YYYY HH:MM DGN$SPECIFICATION

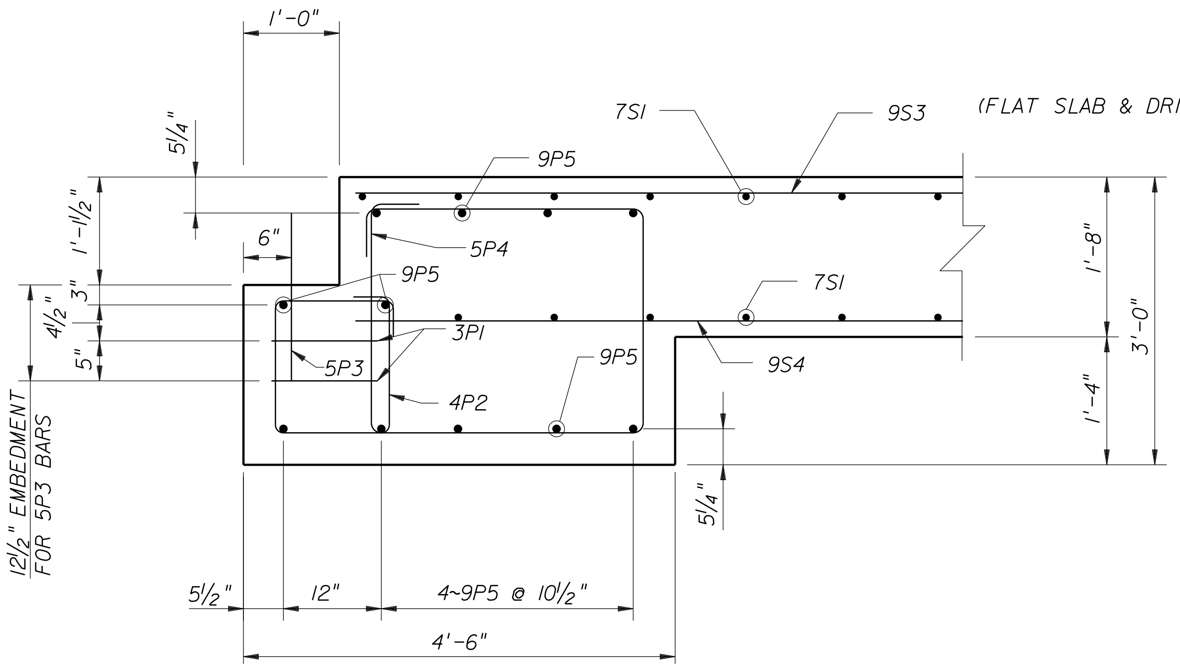


ELEVATION OF END BENT 1

(FLAT SLAB & DRILLED SHAFT REINFORCEMENT OMITTED FOR CLARITY)
(LOOKING AHEAD STATION)

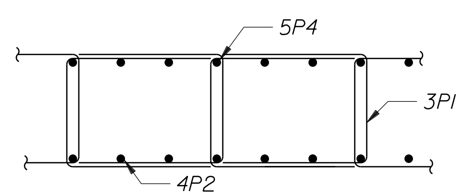
NOTES:

1. WORK THIS SHEET WITH C-42 AND C-74, C-75.
2. FOR FOUNDATION LAYOUT SEE SHEET C-19.
3. FOR REINFORCING BAR LIST SEE SHEET C-70.
4. FOR DRILLED SHAFT DETAILS SEE SHEET A-17.



SECTION A-A

(BETWEEN SHAFTS)



DETAIL OF 3P1 PLACEMENT

ESTIMATED QUANTITIES		
ITEM	UNIT	QUANTITY
CLASS II (BRIDGE DECK) CONCRETE (SUPERSTRUCTURE)	CY	8.5
REINFORCING STEEL (SUPERSTRUCTURE)	LBS	2550
3'-0" DRILLED SHAFT	*	*

* SEE SUMMARY OF BRIDGE PAY ITEMS

NOT APPLICABLE - BRIDGE REMOVED

BRIDGE NO. TEMP 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

	NAMES	DATES
DRAWN BY	JKP	02-03
CHECKED BY	RGW	03-03
DESIGNED BY	JKP	02-03
CHECKED BY	RGW	02-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
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904-256-2500 FL Cert. No. EB0005620
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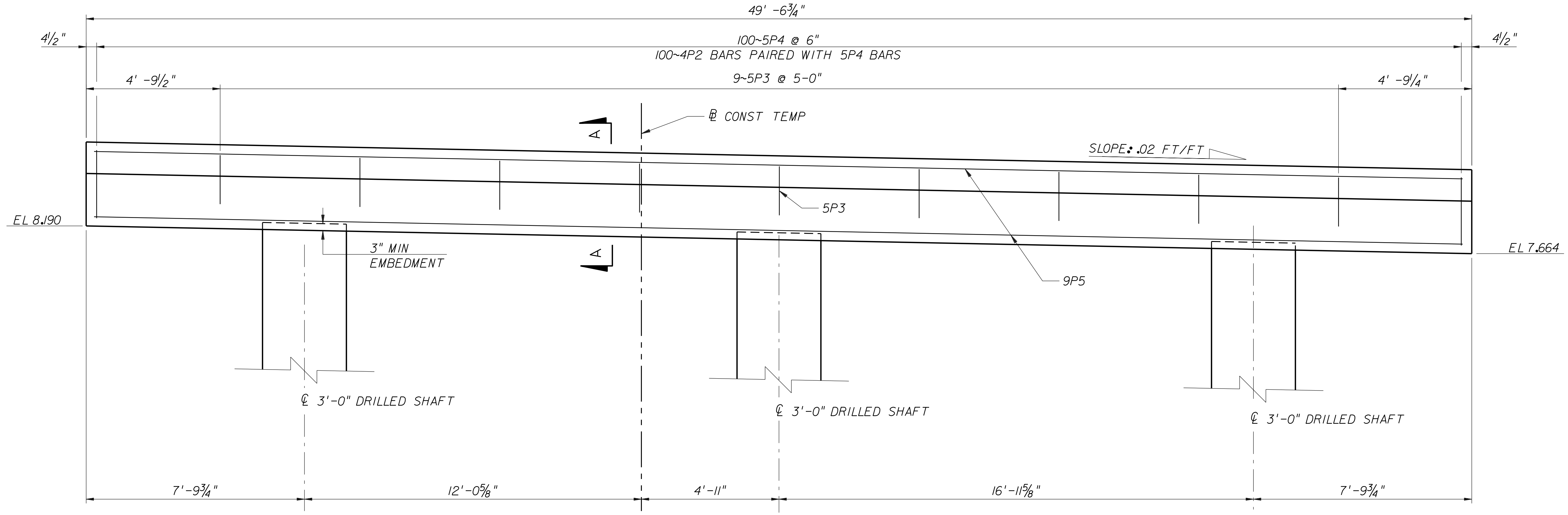
FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: END BENT 1

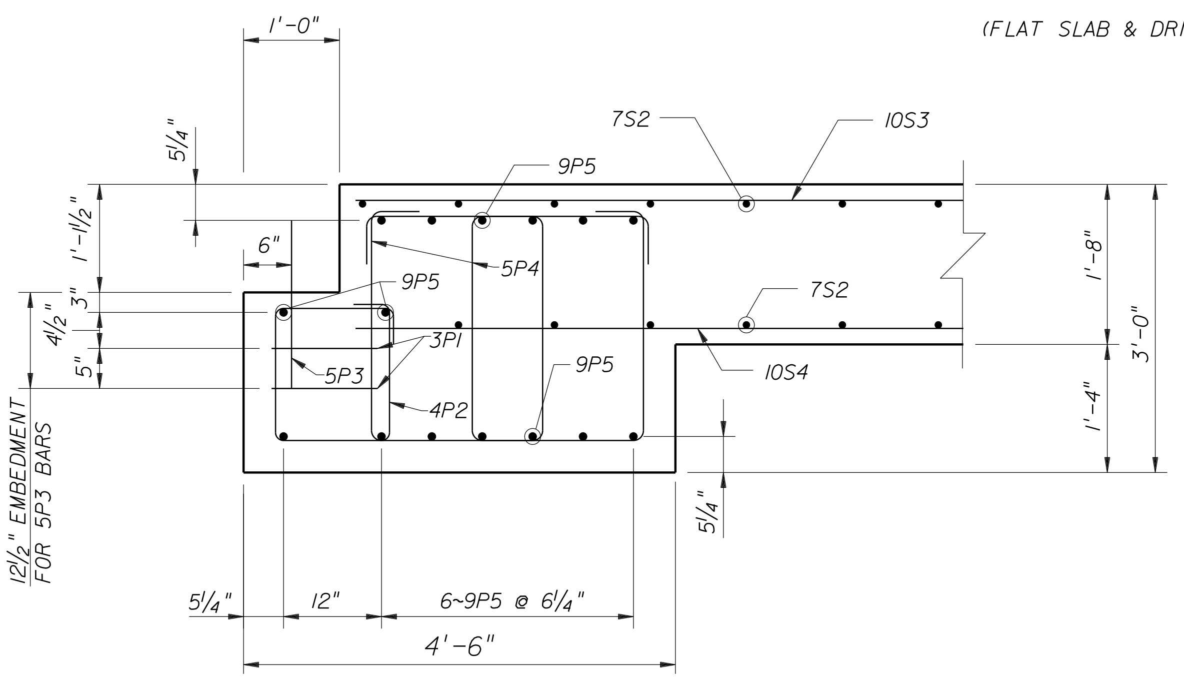
PROJECT NAME: TEMPORARY BRIDGE BRIDGE OF LIONS REHABILITATION

SHEET NO. C-25



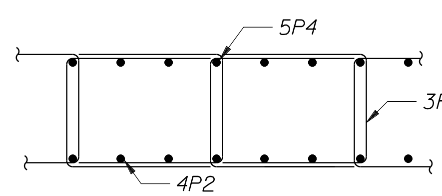
ELEVATION OF END BENT 28

(DIMENSIONS ALONG SKEW)
(FLAT SLAB & DRILLED SHAFT REINFORCEMENT OMITTED FOR CLARITY)
(LOOKING BACK STATION)



SECTION A-A

(BETWEEN SHAFTS)



DETAIL OF 3P1 PLACEMENT

ESTIMATED QUANTITIES		
ITEM	UNIT	QUANTITY
CLASS II (BRIDGE DECK) CONCRETE (SUPERSTRUCTURE)	CY	12.1
REINFORCING STEEL (SUPERSTRUCTURE)	LBS	5033
3'-0" DRILLED SHAFT	*	*

* SEE SUMMARY OF BRIDGE PAY ITEMS

NOTES:

1. WORK THIS SHEET WITH C-52 AND C-74, C-75.
2. FOR FOUNDATION LAYOUT SEE SHEET C-23.
3. FOR REINFORCING BAR LIST SEE SHEET C-70.
4. FOR DRILLED SHAFT DETAILS SEE SHEET A-17.

NOT APPLICABLE - BRIDGE REMOVED

BRIDGE NO. TEMP 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

	NAMES	DATES
DRAWN BY	JKP	02-03
CHECKED BY	RGW	03-03
DESIGNED BY	JKP	02-03
CHECKED BY	RGW	02-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Robert G. Woodruff, Florida PE No. 57099

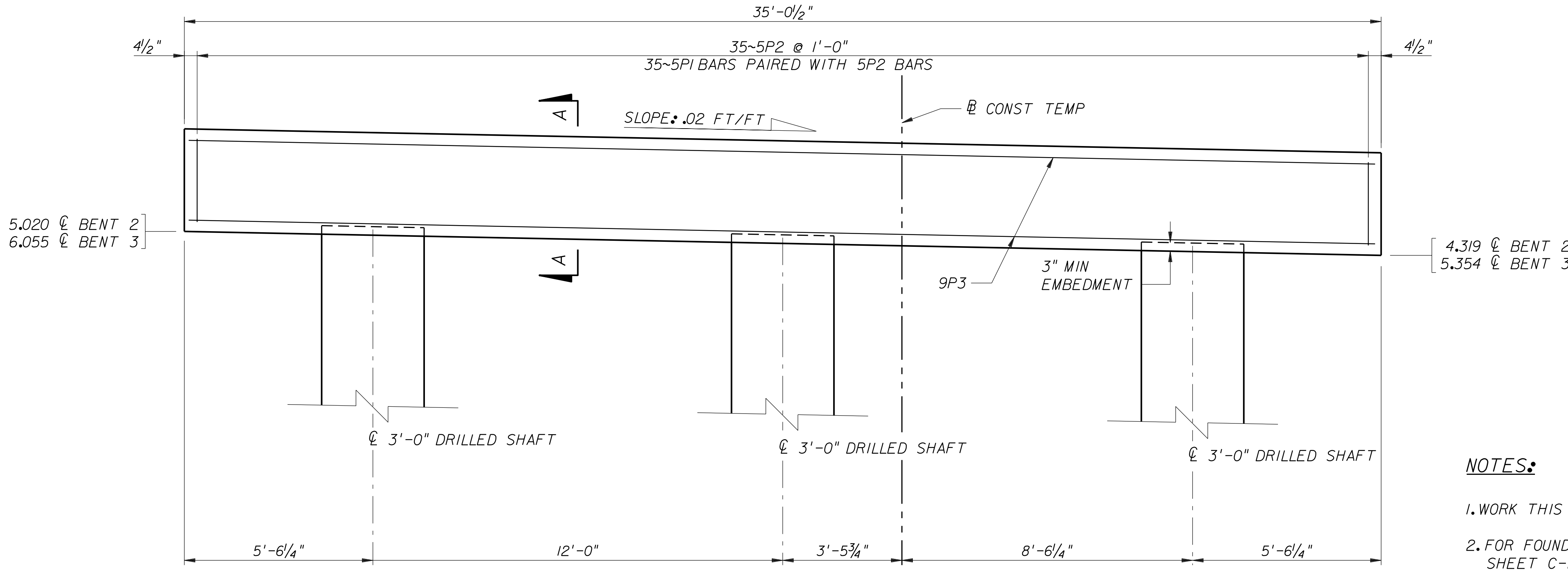
FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: END BENT 28

PROJECT NAME: TEMPORARY BRIDGE
BRIDGE OF LIONS REHABILITATION

SHEET NO. C-26

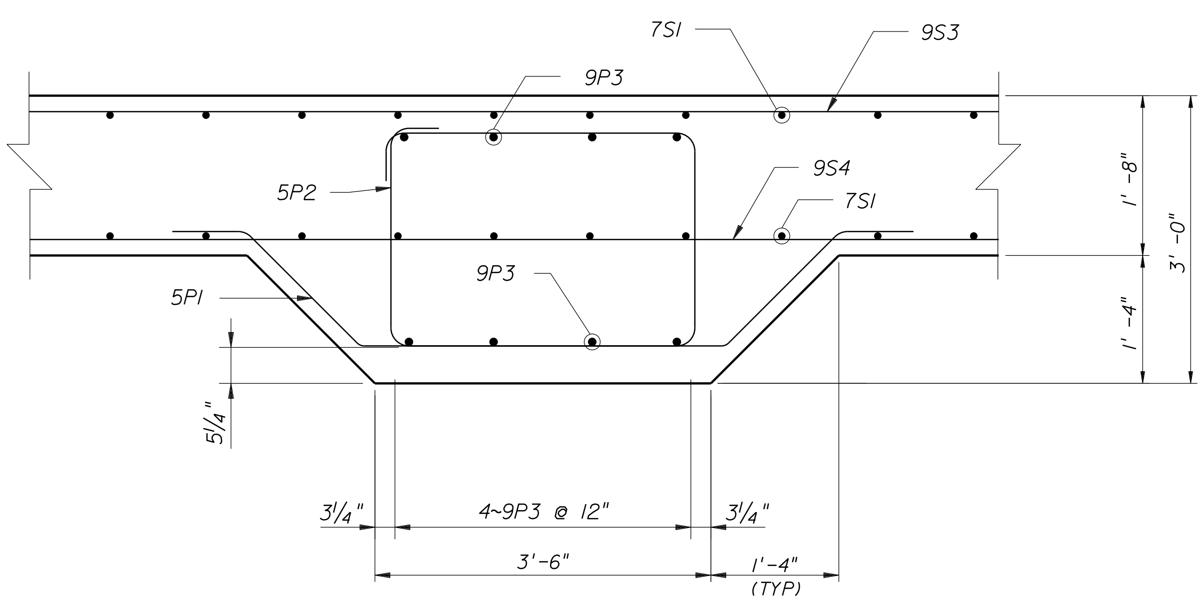


SECTION THROUGH PIERS 2 & 3

(FLAT SLAB & DRILLED SHAFT REINFORCEMENT OMITTED FOR CLARITY)
(LOOKING AHEAD STATION)

NOTES:

1. WORK THIS SHEET WITH C-42.
2. FOR FOUNDATION LAYOUT SEE SHEET C-19.
3. FOR REINFORCING BAR LIST SEE SHEET C-70.
4. FOR DRILLED SHAFT DETAILS SEE SHEET A-17.



SECTION AA

(BETWEEN SHAFTS)

ESTIMATED QUANTITIES		
ITEM	UNIT	QUANTITY
CLASS II (BRIDGE DECK) CONCRETE (SUPERSTRUCTURE)	CY	16.7
REINFORCING STEEL (SUPERSTRUCTURE)	LBS	4360
3'-0" DRILLED SHAFT	*	*

* SEE SUMMARY OF BRIDGE PAY ITEMS

NOT APPLICABLE - BRIDGE REMOVED

BRIDGE NO. TEMP 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

	NAMES	DATES
DRAWN BY	JKP	03-03
CHECKED BY	RGW	03-03
DESIGNED BY	JKP	02-03
CHECKED BY	RGW	02-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Robert G. Woodruff, Florida PE No. 57099

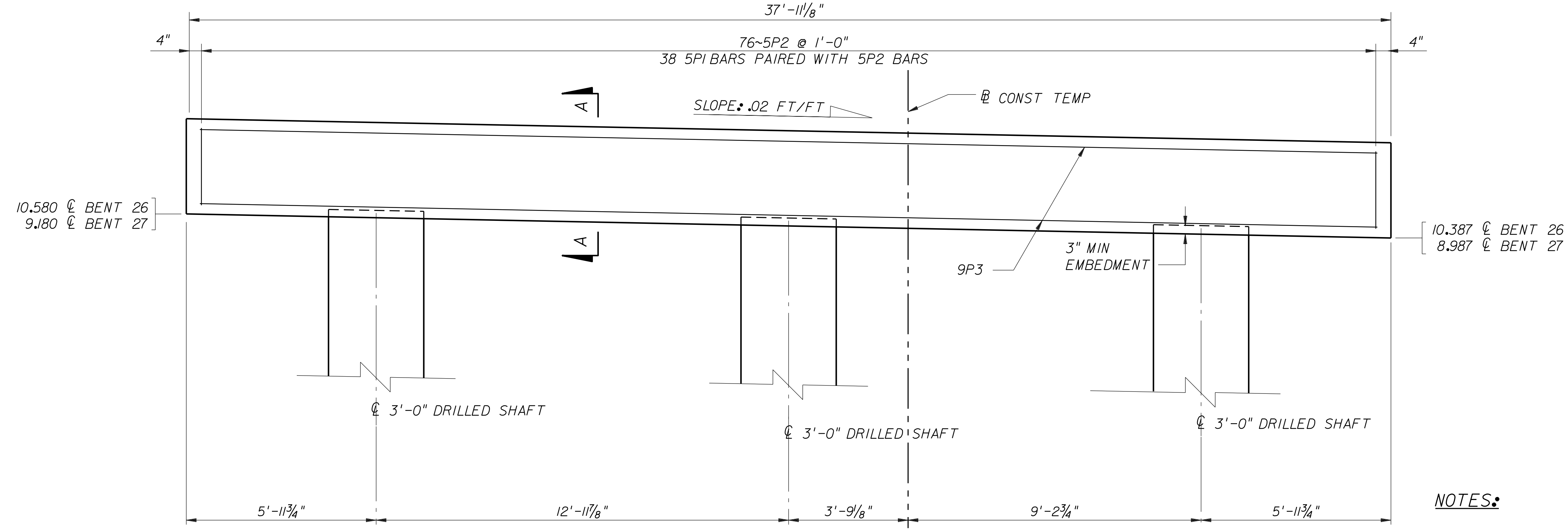
FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: FLAT SLAB INTERGRAL BENTS
BENTS 2-3

PROJECT NAME: TEMPORARY BRIDGE
BRIDGE OF LIONS REHABILITATION

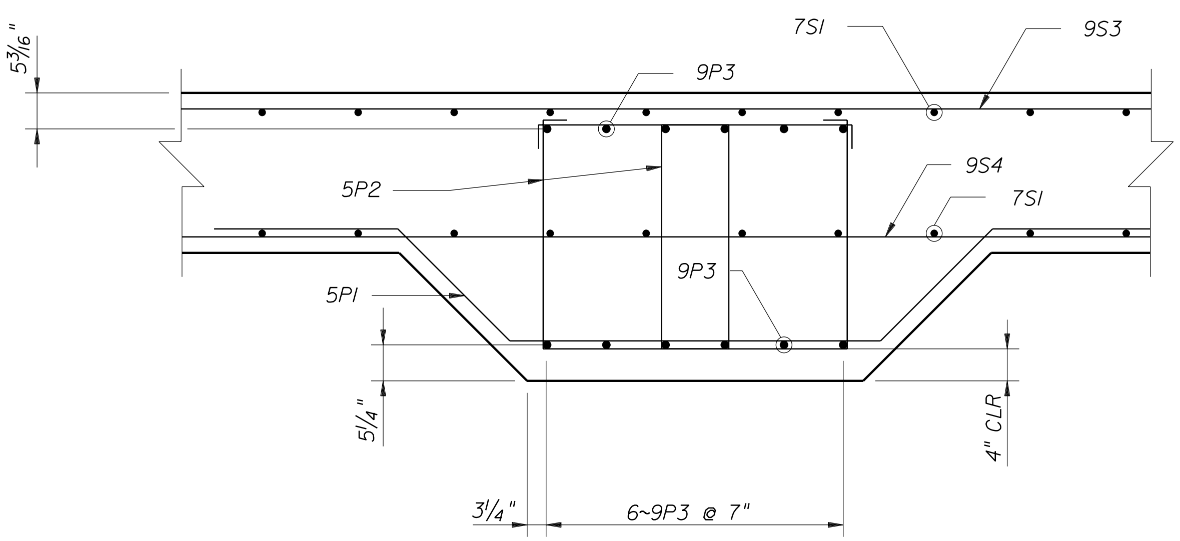
SHEET NO. C-27



SECTION THROUGH PIERS 26 & 27
(DIMENSION ALONG SKEW)
(FLAT SLAB & DRILLED SHAFT REINFORCEMENT OMITTED FOR CLARITY)
(LOOKING AHEAD STATION)

NOTES:

1. WORK THIS SHEET WITH C-52.
2. FOR FOUNDATION LAYOUT SEE SHEET C-23.
3. FOR REINFORCING BAR LIST SEE SHEET C-70.
4. FOR DRILLED SHAFT DETAILS SEE SHEET A-17.



SECTION A-A
(BETWEEN SHAFTS)

ESTIMATED QUANTITIES		
ITEM	UNIT	QUANTITY
CLASS II (BRIDGE DECK) CONCRETE (SUPERSTRUCTURE)	CY	18.1
REINFORCING STEEL (SUPERSTRUCTURE)	LBS	5552
3'-0" DRILLED SHAFT	*	*

* SEE SUMMARY OF BRIDGE PAY ITEMS

NOT APPLICABLE - BRIDGE REMOVED

BRIDGE NO. TEMP 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

	NAMES	DATES
DRAWN BY	JKP	03-03
CHECKED BY	RGW	03-03
DESIGNED BY	JKP	02-03
CHECKED BY	RGW	02-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Robert G. Woodruff, Florida PE No. 57099

FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: FLAT SLAB INTERGRAL BENTS
BENTS 26-27

PROJECT NAME: TEMPORARY BRIDGE
BRIDGE OF LIONS REHABILITATION

SHEET NO. C-28

ESTIMATED QUANTITIES		
ITEM	UNIT	QUANTITY 6
DRILLED SHAFT (36" Ø)	FT	6219 AB
DRILLED SHAFT (60" Ø)	FT	51 *
DRILLED SHAFT (72" Ø) **	FT	2217
DRILLED SHAFT (90" Ø) **	FT	2080
DRILLED SHAFT (96" Ø) **	FT	1202
PERMANENT CASING (60" Ø)	FT	82
PERMANENT CASING (96" Ø)	FT	330
LOAD TEST (1 @ 72" Ø SHAFT AND 1 @ 90" Ø SHAFT)	EA	2 6
UNCLASSIFIED SHAFT EXCAVATION (36" Ø)	FT	6207 AB
UNCLASSIFIED SHAFT EXCAVATION (72" Ø)	FT	1946
UNCLASSIFIED SHAFT EXCAVATION (90" Ø)	FT	1973
UNCLASSIFIED SHAFT EXCAVATION (96" Ø)	FT	1125

NOTES:

1. WORK THIS SHEET WITH SHEET B-29A.
2. TIP ELEVATION SHOWN IN THE TABLE IS THE ELEVATION TO WHICH THE SHAFT SHALL BE CONSTRUCTED UNLESS LOAD TEST DATA OR OTHER GEOTECHNICAL DATA OBTAINED DURING CONSTRUCTION ALLOWS THE ENGINEER TO AUTHORIZE A DIFFERENT TIP ELEVATION. HOWEVER, MINIMUM TIP ELEVATION IS THE HIGHEST ELEVATION THAT THE SHAFT TIP SHALL BE CONSTRUCTED TO. MINIMUM TIP ELEVATION IS BASED ON LATERAL LOAD RESISTANCE REQUIRED FOR VESSEL IMPACT WITH 50% OF 100 YR. PREDICTED SCOUR DEPTH.
3. REFER TO SPECIFICATION SECTION 455-2 (AND THE TECHNICAL SPECIAL PROVISIONS, SECTION 455-2J1, OSTERBERG CELL LOAD TESTING OF DRILLED SHAFTS (PERMANENT BRIDGE) PREPARED FOR THIS PROJECT FOR SHAFT TESTING PROCEDURES. THE COST OF ALL TEST EQUIPMENT, LABOR, AND MATERIALS SHALL BE INCLUDED IN PAY ITEM 455-101-1 "LOAD TEST (OSTERBERG CELL) (LESS THAN FIVE CELLS)".
4. "NON-PRODUCTION" TEST SHAFTS SHALL BE CONSTRUCTED AND REINFORCED TO THE SAME SPECIFICATIONS AS THE PRODUCTION SHAFTS, WITH THE EXCEPTIONS NOTED IN THE TECHNICAL SPECIAL PROVISIONS FOR OSTERBERG CELL LOAD TESTING OF DRILLED SHAFTS (PERMANENT BRIDGE), SECTION 455-2J1 STATIC COMPRESSION LOAD TESTS, PREPARED FOR THIS PROJECT. THE REQUIREMENTS OF SPECIFICATION 455-18 TEST HOLES SHALL APPLY, UNO.
5. ALL DRILLED SHAFTS (TEST AND PRODUCTION) SHALL HAVE CROSS-HOLE SONIC LOGGING (CSL) ACCESS TUBES INSTALLED IN ACCORDANCE WITH THE TECHNICAL SPECIAL PROVISIONS FOR CSL TESTING OF DRILLED SHAFTS PREPARED FOR THIS PROJECT. SEE SHEETS A-17 THRU A-20 FOR LOCATIONS AND REQUIREMENTS. IN ADDITION, THE CONTRACTOR SHALL MAKE ALL NECESSARY ACCOMMODATIONS FOR THE CEI TO FACILITATE INSPECTION AND CSL TESTING OF ALL DRILLED SHAFTS. THE COST FOR ALL MATERIALS AND LABOR ASSOCIATED WITH THE INSTALLATION AND TESTING OF THE CSL SYSTEM SHALL BE CONSIDERED INCIDENTAL TO, AND INCLUDED IN THE COST OF THE DRILLED SHAFT, PER LINEAR FOOT.
6. QUANTITIES IN THE TABLE INCLUDE TEST SHAFT LENGTHS.

PIER 13 SHAFT 1, MIN. TIP ELEV. = -104 AB

ACTUAL TIP DATA COLLECTED 8/06 THRU 12/06 AB

DRILLED SHAFT INSTALLATION TABLE											
	INSTALLATION CRITERIA				DESIGN CRITERIA						
PIER OR BENT NO.	SHAFT SIZE Ø (IN)	TIP ELEV. (FT)	MIN. TIP ELEV. (FT)	MIN. ROCK SOCKET LENGTH (FT)	FACTORED DESIGN LOAD (TONS)	DOWN DRAG (TONS)	LONG-TERM SCOUR ELEV. (FT)	100 YR. SCOUR ELEV. (FT)	Ø	TOP OF SHAFT ELEV. (FT)	
WEST ABUTMENT 1	36	-80	-80 ~~N/A~~	N/A	115	N/A	N/A	-41.2	0.55	-4.25	
PIER 2	36	-88	-85 ~~N/A~~	N/A	163	N/A	N/A	-23.2	0.55	-7.75	
PIER 3	36	-88	-85 ~~N/A~~	N/A	163	N/A	N/A	-25.0	0.55	-9.75	
PIER 4	36	-88	-85 ~~N/A~~	AB N/A	163	N/A	N/A	-26.4	0.55	-9.75	
PIER 5	36	-88	-85 ~~N/A~~	N/A	163	N/A	N/A	-26.9	0.55	-9.75	
PIER 6	90	-101	-100	N/A	569	N/A	N/A	-34.4	0.55	-14.75	
PIER 7	90	-108	-100	N/A	569	N/A	N/A	-37.3	0.55	-14.75	
PIER 8	90	-100	-100	N/A	569	N/A	N/A	-36.8	0.55	-14.75	
PIER 9	90	-100	-100	N/A	569	N/A	N/A	-37.9	0.55	-14.75	
PIER 10	90	-109	-100 AB	N/A	569	N/A	N/A	-39.2	0.55	-14.75	
PIER 11	90	-110	-105 ~~≥110~~	N/A	569	N/A	N/A	-43.9	0.55	-14.75	
PIER 12	72	-109	-100 ~~≥105~~	N/A	395	N/A	N/A	-48.5	0.55	-15.75	
PIER 13	72	-105	-100 ~~≥105~~	N/A	395	N/A	N/A	-47.8	0.55	-15.75	
PIER 14	72	-106	-100 ~~≥105~~	N/A	395	N/A	N/A	-50.3	0.55	-15.75	
PIER 15 SHAFTS 1, 2, & 3	96	-135	-135	N/A	910	N/A	N/A	-46.1	0.55	-9.75	
PIER 15 SHAFTS 4 & 5*	96	-135	-135	N/A	910	N/A	N/A	-46.1	0.55	-22.5	
PIER 16 SHAFTS 1 & 2*	96	-135	-135	N/A	910	N/A	N/A	-50.3	0.55	-22.5	
PIER 16 SHAFTS 3, 4, & 5	96	-135	-135	N/A	910	N/A	N/A	-50.3	0.55	-9.75	
PIER 17	72	-105	-105 AB	N/A	395	N/A	N/A	-56.9	0.55	-15.75	2
PIER 18	72	-105	-100 ~~≥105~~	N/A	395	N/A	N/A	-50.5	0.55	-15.75	
PIER 19	72	-105	-100 ~~≥105~~	N/A	395	N/A	N/A	-48.3	0.55	-15.75	
PIER 20	90	-110	-105 ~~≥110~~	N/A	569	N/A	N/A	-45.2	0.55	-14.75	
PIER 21	90	-110	-105 ~~≥110~~	N/A	569	N/A	N/A	-44.8	0.55	-14.75	
PIER 22	90	-110	-110 AB	N/A	569	N/A	N/A	-51.6	0.55	-14.75	
PIER 23	90	-110	-105 ~~≥110~~	N/A	569	N/A	N/A	-42.0	0.55	-14.75	
PIER 24	90	-103	-95 ~~≥100~~	N/A	569	N/A	N/A	-32.3	0.55	-14.75	
EAST ABUTMENT 25	36	-97	-97 ~~N/A~~	N/A	205	N/A	N/A	-37.8	0.55	-8.75	
EAST SEAWALL	36	-75	-75 ~~N/A~~	N/A	135	N/A	N/A	-14.0	0.55	-8.75	

* DUE TO HORIZONTAL CHANNEL CLEARANCE CONFLICTS, A 5'-0" Ø CONCRETE COLUMN SHALL BE INSTALLED FROM ELEVATION -22.5 TO ELEVATION -9.75 AT PIER 15 SHAFTS 4 & 5 AND AT PIER 16 SHAFTS 1 & 2. (SEE SHEET A-19 FOR DETAILS). THE 5'-0" Ø CONCRETE COLUMN SHALL BE CONSTRUCTED IN ACCORDANCE WITH SECTION 455 AND PAID FOR UNDER PAY ITEM 455-88-6, DRILLED SHAFT (60" Ø).

** FOR ALL DRILLED SHAFTS 72" IN DIAMETER AND LARGER, THE CONTRACTOR SHALL FOLLOW ALL PROVISIONS OF THE SPECIFICATIONS FOR PLACEMENT AND MONITORING OF MASS CONCRETE. (SEE NOTE 7 ON B-29A)

AS-BUILT
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.
IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER.

AB

RICHARD READE WALLACE
LICENSE
No. 24066
STATE OF FLORIDA
PROFESSIONAL ENGINEER

RICHARD R. WALLACE
Florida PE No. 24066

BRIDGE NO. 780074

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
4-2-04	BMS	2 REVISED ESTIMATED QUANTITIES & DRILLED SHAFT TABLE	7-20-11	RRW	AB AS-BUILT
12-04-06	BMS	6 REVISED QUANTITIES			

	NAMES	DATES
DRAWN BY	GBL	11-03
CHECKED BY	JPT	11-03
DESIGNED BY	PAW	02-03
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
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904-256-2500 FL Cert. No. EB0005620
Bryan M. Sturm, Florida PE No. 64066

LICHTENSTEIN CONSULTING ENGINEERS, INC.
2700 W. Cypress Creek Rd., Suite D-140
Fort Lauderdale, Florida 33309
Authorization No. 00006065
G. Alan Klevens, P.E.
P.E. License No. 47187

FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: DRILLED SHAFT DATA (1 OF 2)

PROJECT NAME: PERMANENT BRIDGE
BRIDGE OF LIONS REHABILITATION

SHEET NO. B-29

DRILLED SHAFT INSTALLATION NOTES:

1. WORK THIS SHEET WITH SHEET B-29.

2. THE CONTRACTOR SHALL DRILL ONE PILOT HOLE (STANDARD PENETRATION TEST [SPT] BORING) AT ONE DRILLED SHAFT LOCATION PER FOUNDATION UNIT, AT LEAST 14 DAYS BEFORE THE EXCAVATION OF THE FOUNDATION UNIT SHAFTS BEGIN. THE COST SHALL BE INCLUDED WITH THE RESPECTIVE DRILLED SHAFT. DETAILED REQUIREMENTS OF THE PILOT HOLE PROGRAM (INCLUDING THE LOCATIONS AT WHICH PILOT HOLES ARE TO BE DRILLED, THE MINIMUM BORING TERMINATION ELEVATIONS, AND THE LOWEST ELEVATION AT WHICH SPT SAMPLING SHALL COMMENCE), ARE PRESENTED IN THE TECHNICAL SPECIAL PROVISIONS, SECTION 455-15.6, DRILLED SHAFT PILOT HOLES (PERMANENT BRIDGE) PREPARED FOR THIS PROJECT.

3. PERMANENT DRILLED SHAFTS FOR FOUNDATIONS SHALL NOT BE CONSTRUCTED UNTIL THE ENGINEER EVALUATES THE SPT BORING DATA, SAMPLES, AND RELEVANT OSTERBERG CELL LOAD TEST RESULTS. THE CONTRACTOR SHALL ANTICIPATE 14 DAYS FOR RESULTS FROM THE TIME THE ENGINEER RECEIVES THE SPT BORING DATA AND SAMPLES, OR FROM THE TIME THE MOST RECENT RESULTS FROM THE OSTERBERG CELL LOAD TEST ARE OBTAINED, WHICHEVER IS LATER.

4. LAYERS OF VERY HARD MATERIALS (SUCH AS COQUINA, CEMENTED SOILS/SHELL, BOULDERS, MAN-MADE OBSTRUCTIONS, AND LIMESTONE ROCK) MAY BE ENCOUNTERED AT THIS SITE. SUCH MATERIALS MAY MAKE SHAFT EXCAVATIONS AND/OR CASING INSTALLATION DIFFICULT. THE CONTRACTOR SHALL EXPECT TO ENCOUNTER THESE TYPES OF MATERIALS AT ALL SHAFT LOCATIONS AND SHALL USE SPECIALIZED EQUIPMENT AND/OR PROCEDURES, AS NECESSARY, TO FACILITATE SHAFT EXCAVATION AND/OR CASING INSTALLATION. THE CASING TIPS SHALL BE REINFORCED, AS NECESSARY, AND THE CASING THICKNESS SHALL BE ADEQUATE TO PREVENT CASING DAMAGE/DEFORMATION DURING INSTALLATION THROUGH HARD LAYERS.

△1 5. THE DESIGN FOR THE THICKNESS OF ALL FOUNDATION SEALS FOR APPROACH PIERS AND ABUTMENTS, AS SHOWN IN THE PLANS, IS BASED ON A TOP OF WATER ELEVATION OF +2.55 (MEAN HIGH WATER) AND ASSUMES CLEAN CONTACT SURFACES BETWEEN THE SEAL AND SHEET PILE COFFERDAM AND DRILLED SHAFTS. PLAN QUANTITIES ARE BASED ON THE THICKNESS SHOWN AND HAVING A PLAN AREA EQUAL TO THE AREA OF THE FOUNDATION, PLUS 2'-6" AROUND THE PERIMETER OF THE FOUNDATION. THE PLAN THICKNESS SHALL BE CONSIDERED TO BE THE REQUIRED SEAL THICKNESS, UNLESS THE CONTRACTOR PROVIDES AN ALTERNATE DESIGN FOR THE SEALS AND/OR METHOD OF PROVIDING A "DRY HOLE". SHOP DRAWINGS AND DESIGN CALCULATIONS FOR ANY ALTERNATIVE TO THE SEALS SHOWN, SHALL BE SUBMITTED, SIGNED AND SEALED BY A FLORIDA REGISTERED PROFESSIONAL ENGINEER, FOR REVIEW BY THE ENGINEER. THE COST OF ALL LABOR, MATERIALS AND EQUIPMENT REQUIRED TO CONSTRUCT AND MAINTAIN THE SEALS DURING CONSTRUCTION OF THE SUBSTRUCTURE COMPONENTS, SHALL BE INCLUDED IN THE COST OF PAY ITEM 400-3-20, CLASS III CONCRETE (SEAL), PER CUBIC YARD. PAYMENT SHALL BE BASED ON PLAN QUANTITIES, EVEN IF THE CONTRACTOR ELECTS TO USE AN ALTERNATIVE DESIGN FOR DEWATERING OF THE FOUNDATION AREAS, OR USES QUANTITIES OF MATERIAL WHICH DIFFER FROM THE PLAN QUANTITIES. IF THE CONTRACTOR ELECTS TO USE A PERMANENT SEAL HAVING GREATER CROSS-SECTIONAL AREA (THICKNESS x WIDTH) THAN SHOWN IN THE PLANS, HE SHALL BE RESPONSIBLE FOR THE COST OF ANY ADDITIONAL HYDRAULIC MODELING AND ANALYSIS NEEDED TO ASSESS SCOUR IMPACTS.

6. AT THE ENGINEER'S DISCRETION, SHAFTS MAY BE INSPECTED PRIOR TO PLACEMENT OF CONCRETE USING METHODS AND SHAFT INSPECTION DEVICE (SID) THAT COMPLY WITH SECTION 455-15.11.3. CONTRACTOR SHALL PROVIDE SAFE ACCESS FOR INSPECTION PERSONNEL AND PROVIDE ASSISTANCE WHEN REQUESTED BY THE ENGINEER.

7. CONCRETE FOR ALL DRILLED SHAFTS, 72" DIAMETER AND LARGER, SHALL BE CONSIDERED TO BE MASS CONCRETE AND SHALL COMPLY WITH THE PROVISIONS OF SPECIFICATION SECTION 346-3.3, AS EXPANDED BY SPECIAL PROVISION 346-3.3.1. NO CONCRETE SHALL BE PLACED FOR DRILLED SHAFTS AT PIERS 6 THRU 24 UNTIL THE MASS CONCRETE MIX DESIGN AND MASS CONCRETE MONITORING PLAN FOR THE DRILLED SHAFTS HAVE BEEN APPROVED BY THE ENGINEER AND IMPLEMENTED. THE MASS CONCRETE MONITORING PLAN SHALL BE SPECIFIC FOR EACH OF THE SHAFT DIAMETERS, AND SHALL CONSIDER THE EFFECT OF SOIL INSULATION FOR THE PORTION OF THE SHAFTS BELOW THE MUDLINE VS THE NON-INSULATED PORTION OF THE SHAFTS IN WATER. IF INTERNAL COOLING TUBES ARE TO BE PROVIDED FOR CIRCULATION OF WATER DURING CURING, THE CONTRACTOR SHALL WAIT UNTIL INITIAL SET OF THE CONCRETE HAS OCURRED BEFORE COOLING WATER IS PUMPED INTO AND CIRCULATED IN THE TUBES.

8. COFFERDAMS SHALL BE REQUIRED FOR THE PERMANENT BRIDGE AT ALL PIERS, ABUTMENTS AND SEAWALLS, AS NECESSARY TO CONSTRUCT THE SUBSTRUCTURE COMPONENTS "IN THE DRY". THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE DESIGN OF ALL COFFERDAMS, SHORING AND BRACING. SHOP DRAWINGS AND DESIGN CALCULATIONS FOR THE COFFERDAMS, SHALL BE SUBMITTED TO THE ENGINEER, AND SHALL BE SIGNED AND SEALED BY A FLORIDA REGISTERED PROFESSIONAL ENGINEER. THE COST OF THE COFFERDAMS SHALL INCLUDE THE DESIGN AND INSTALLATION, INCLUDING ALL LABOR, MATERIALS AND EQUIPMENT REQUIRED TO CONSTRUCT AND MAINTAIN THE COFFERDAMS DURING CONSTRUCTION OF THE SUBSTRUCTURE COMPONENTS. COFFERDAMS SHALL BE PAID FOR AS "EACH", UNDER THE FOLLOWING PAY ITEM NUMBERS & DESCRIPTIONS:

400-95	COFFERDAM, PIER	(21 EACH)
400-95-1	COFFERDAM, BASCULE PIER	(2 EACH)
400-95-3	COFFERDAM, SPECIAL (ABUTMENTS & SEAWALLS)	(2 EACH)

9. NO SHAFT CONSTRUCTION SHALL BEGIN WITHIN 30 FEET OF ANOTHER SHAFT WHICH IS UNDER CONSTRUCTION OR ANOTHER SHAFT WHICH HAS BEEN COMPLETED WITHIN THE PREVIOUS 12 HOURS.

10. ALL DRILLED SHAFTS ON THIS PROJECT SHALL BE CONSTRUCTED USING WET METHODS WITH MINERAL SLURRY. DESANDING EQUIPMENT WITH DRILLING TANKS SHALL BE PROVIDED TO CLEAN THE SLURRY FOR REUSE. THE DESANDING EQUIPMENT SUPPLIED BY THE CONTRACTOR SHALL BE EQUIPPED WITH SUITABLE SCREENS AND CYCLONE SEPARATORS AND SHALL HAVE ADEQUATE CAPACITY TO DESAND THE VOLUMES OF SLURRY REQUIRED FOR THE PROJECT. NO SLURRY OR CONTAMINATED WATER SHALL BE DISCHARGED INTO THE RIVER. NO CASINGS SHALL PERMANENTLY PENETRATE BELOW THE 100-YR SCOUR ELEVATION REPORTED IN THE DRILLED SHAFT INSTALLATION TABLE ON SHEET B-29, WITHOUT THE ENGINEER'S APPROVAL.

AB ADDITIONAL CLARIFICATION ON MAXIMUM TIP ELEVATION (RFI 005 - 1/7/2005)

11. ALL COSTS ASSOCIATED WITH THE MATERIALS FOR AND INSTALLATION OF THE STEEL CASINGS FOR THE DRILLED SHAFT INSTALLATION FOR THE ABUTMENTS, SEAWALL, AND APPROACH SPAN PIERS SHALL BE CONSIDERED INCIDENTAL TO, AND INCLUDED WITH, THE COST OF THE DRILLED SHAFT CONCRETE.

12. SPECIAL PRECAUTIONS SHALL BE TAKEN BY THE CONTRACTOR TO PREVENT DISCHARGE OF SLURRY OR EXCAVATED MATERIALS INTO THE WATERWAY DURING THE EXCAVATION OPERATIONS. A RECEPTICAL SHALL BE PROVIDED AROUND THE TOP OF THE DRILLED SHAFT CASING TO COLLECT AND CONTAIN MATERIALS EXCAVATED FROM THE DRILLED SHAFTS FOR TRANSPORT BY BARGE OR CONDUIT, AND DISPOSED OF SATISFACTORILY IN APPROVED OFF-SITE AREAS PROVIDED BY THE CONTRACTOR.

13. CARE SHALL BE TAKEN BY THE CONTRACTOR TO PREVENT DISCHARGE OF SLURRY OR CONCRETE INTO THE WATERWAY DURING THE CONCRETING OPERATION. THE CONTRACTOR SHALL PROVIDE A RECEPTACLE AROUND THE TOP OF THE DRILLED SHAFT CASING TO COLLECT ALL SLURRY AND/OR CONCRETE DISCHARGED FROM THE SHAFT DURING THE CONCRETING OPERATION.

14. TURBIDITY BARRIERS (CURTAINS) SHALL BE USED DURING DRILLED SHAFT EXCAVATION IN WATER UNLESS SHEET PILE COFFERDAMS ARE IN PLACE AND ADEQUATELY ISOLATE TURBIDTY FROM THE RIVER.

15. THE CONTRACTOR SHOULD NOTE THAT THE POTENTIOMETRIC SURFACE OF THE FLORIDAN AQUIFER IS AT ELEVATION +20.0 FEET. THE APPROXIMATE ELEVATION OF THE TOP OF THE FLORIDIAN AQUIFER IS ELEVATION -175 (MSL).

16. FOUNDATION SEALS SHALL BE REQUIRED AT ALL PIERS, ABUTMENTS AND SEAWALLS. ADDITIONALLY, THE CONTRACTOR IS ADVISED THAT THE EXTRACTION OF TIMBER PILING AT THE EXISTING ABUTMENTS MAY CAUSE INCREASED GROUNDWATER FLOW, POSSIBLY RESULTING IN THE NEED TO BACKFILL PILE HOLES WITH SAND OR CLAY SLURRY TO SLOW WATER FLOW INTO THE EXCAVATION. DEWATERING OF THE EXCAVATIONS, INCLUDING ALL PROVISIONS FOR MANAGEMENT OF GROUNDWATER SHALL BE INCLUDED IN THE COST OF THE COFFERDAMS FOR EACH SUBSTRUCTURE UNIT.

RICHARD R. WALLACE
Florida PE No. 24066

RICHARD READE WALLACE LICENSE No. 24066 STATE OF FLORIDA PROFESSIONAL ENGINEER

AS-BUILT
THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD.
AS-BUILT INFORMATION IS LIMITED TO THE OBSERVATIONS MADE DURING PERIODIC FIELD TRIPS, AND THE DOCUMENTATION PROVIDED BY THE CONSTRUCTION ENGINEERING AND INSPECTION AGENT, URS CONSTRUCTION SERVICES, INC.
IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED ON THIS SHEET AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER. AB

BRIDGE NO. 780074

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
3-19-04	JKP	△1 MODIFIED NOTE 5.			
7-20-11	RRW	AB AS-BUILT			

	NAMES	DATES
DRAWN BY	GBL	11-03
CHECKED BY	JPT	11-03
DESIGNED BY	PAW	02-03
CHECKED BY	GBL	03-03
APPROVED BY	D.T. SWEENEY	

ENGINEER OF RECORD:
RS&H
Reynolds, Smith and Hills, Inc.
10748 Deerwood Park Blvd. South
Jacksonville, Florida 32256-0597
904-256-2500 FL Cert. No. EB0005620
Jeffrey P. Toussant, Florida PE No. 36367

FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR A1A	ST. JOHNS	210255-1-52-01

SHEET TITLE: DRILLED SHAFT DATA (2 OF 2)

PROJECT NAME: PERMANENT BRIDGE
BRIDGE OF LIONS REHABILITATION

SHEET NO. B-29A

DD-MMM-YYYY HH:MM
DGN$SPECIFICATION

COMPONENTS OF CONTRACT PLANS INCLUDE

ROADWAY PLANS
SIGNING AND PAVEMENT MARKING PLANS
SIGNALIZATION PLANS
LIGHTING PLANS
STRUCTURE PLANS
A DETAILED INDEX APPEARS ON THE KEY SHEET OF EACH COMPONENT

INDEX OF ROADWAY PLANS

SHEET NO.	SHEET DESCRIPTION
1 - 1A 18	KEY SHEETS
2 - 4A 1	SUMMARY OF PAY ITEMS
5 - 12	TYPICAL SECTIONS
13 - 14	SUMMARY OF QUANTITIES
15 - 18	SUMMARY OF DRAINAGE STRUCTURES
19 - 20	OPTIONAL MATERIALS TABULATION
21	SURVEY LAYOUT
22 - 24	PROJECT LAYOUT
25	CURVE DATA TABLE
26 - 28	REFERENCE POINTS
29 - 41	ROADWAY PLAN
42	BRIDGE APPROACH DETAIL
43 - 54	ROADWAY PROFILE
55 - 56	CURB RETURN PROFILES
57 - 61	PARKING LOT SITE PLANS
62	DRAINAGE DETAILS
62A	DRAINAGE STRUCTURE MODIFICATION 29
63 - 66, 66A, 67 - 97	DRAINAGE STRUCTURES
98	CROSS SECTION PATTERN SHEET
99	ROADWAY SOIL SURVEY
100 - 133	CROSS SECTIONS
134 - 160, 160A, 160B, 161, 162, 162B, 163-165 14	TRAFFIC CONTROL PLANS
166 - 190	UTILITY ADJUSTMENTS
191	SWPPP

GOVERNING STANDARDS AND SPECIFICATIONS: FLORIDA DEPARTMENT OF TRANSPORTATION, ROADWAY AND TRAFFIC DESIGN STANDARDS DATED JANUARY 2000, AND STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION DATED 2000, AS AMMENDED BY CONTRACT DOCUMENTS.

Computation Book CB#1 - Bridge Items

REVISIONS

SEE SHEET 1A. 18

Computation Book CB#2 - Roadway Items

Shop Drawing Index - See Sheet 1A

Fieldbooks & Binders

FB01	Miscellaneous Roadway Items	B17	Signing & Pavement Markings
FB02	Miscellaneous Roadway Items	B17A	Temporary Striping
FB03	Miscellaneous Roadway Items	B18	Lighting Items
FB06	Miscellaneous Roadway Items	B25	Additional Length of Drilled Shafts
FB07	Sodding (Temp)	B26	Sod & Finish Soil Layer
FBO8	Ground Cover & Shrubs	B27	Earthwork
FBO9	Miscellaneous Bridge Items	FBC1	Miscellaneous Roadway Items
B01	MOT Items & Field Office		
B02	Grassing Items		
B03	Striping (Temporary)		
B04	Off Duty Police Officer		
B08a	Pipe Storm Sewer Culvert		
B09	Disputes Review Board & Partnering		
B10	Trainee Manhours		
B13	Riprap (Bank & Shore) and Bedding Stone		
B16	Signalization		

As Built Data Only
William R. Adams, III
3/13/2006

William R. Adams III, P.E.

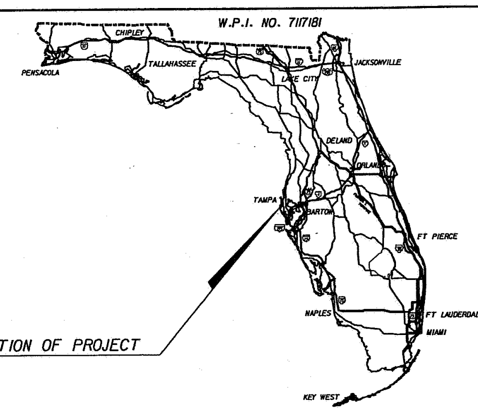
P.E. No.: 45794

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

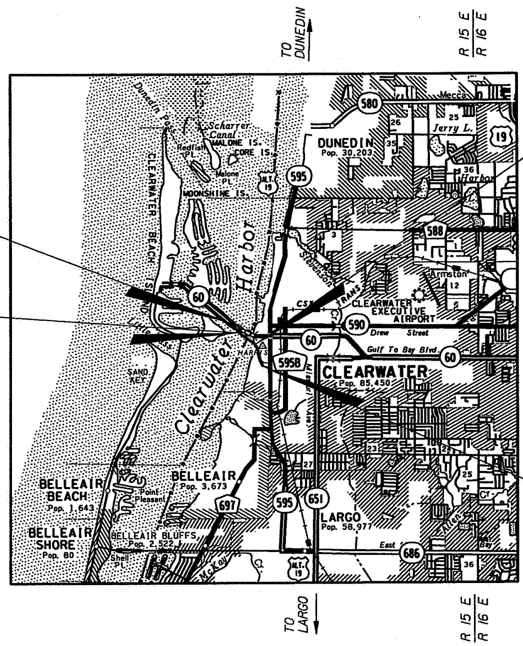
FINAL AS-BUILT PLANS
~~CONTRACT PLANS~~

FINANCIAL PROJECT ID 257093-1-52-01
STATE PROJECT NO. 15220-3599
(FEDERAL FUNDS)
PINELLAS COUNTY (15220)
STATE ROAD NO. 60 (MEMORIAL CAUSEWAY)
BRIDGE REPLACEMENT

W.P.I. NO. 7117181



LOCATION OF PROJECT



BEGIN PROJECT
STA 17+65.80
℄ CONST SR 60
(MEMORIAL CAUSEWAY)
MP = 1.206

BEGIN BRIDGE
STA 27+33.00

T 28 S
T 29 S

END BRIDGE
STA 50+73.00

T 28 S
T 29 S

TO SAFETY HARBOR

TO TAMPA

END PROJECT
STA 65+43.35
℄ CONST SR 60
(CHESTNUT ST.)
MP = 2.111

1 MILE
1" = 3000'

ROADWAY SHOP DRAWINGS TO BE SUBMITTED TO:
DAVID B. MONEY
HDR
2202 N. WESTSHORE BOULEVARD
SUITE 250
TAMPA, FLORIDA 33607-5711

PLANS PREPARED BY :

ENGINEER OF RECORD:
HDR Employee-owned
License No. 4213
HDR Engineering, Inc.
2202 N. West Shore Boulevard
Suite 250
Tampa, FL 33607-5711
(813) 282-2300
www.hdrinc.com

VENDOR NO: F-470-680-568-006

DAVID B. MONEY
HDR
2202 N. WESTSHORE BOULEVARD
SUITE 250
TAMPA, FLORIDA 33607-5711

NOTE: THIS PROJECT TO BE LET TO CONTRACT WITH FINANCIAL PROJECT ID 257093-1-56-01 (UTILITY PLAN) AND 257093-1-56-02 (UTILITY PLAN)

NOTE: THE SCALE OF THESE PLANS MAY HAVE CHANGED DUE TO REPRODUCTION.

REVISIONS 5, 6, 7, 9, 10, 12, 16, 18, 19, 21, 22, 24, 25, 28, 30

ENGINEER OF RECORD:
PARSONS
Kerrie L. Barloga, P.E.
Florida License No. 41813
Certificate of Authorization No. 1838
3500 Financial Plaza, Suite 300
Tallahassee, Fl. 32312-5998
(850)422-0000 Fax (850)422-3373

Contractors: PCL Civil Constructors Inc.
CEI: Parsons Brinckerhoff Construction Services
Sr. Project Engineer: William R. Adams III, P.E.
Project Engineer: Butch Weber, P.E.
District Secretary: Don Skelton, P.E.
Date Work Started: February 27, 2002
Date Work Final Accepted: February 14, 2006

LENGTH OF PROJECT	LINEAR FT.	MILES
ROADWAY	2437.55	0.461
BRIDGES	2340.00	0.443
NET LENGTH OF PROJ.	4777.55	0.904
EXCEPTIONS		
GROSS LENGTH OF PROJ.	4777.55	0.904

FDOT PROJECT MANAGER : JAMAL NAGAMIA

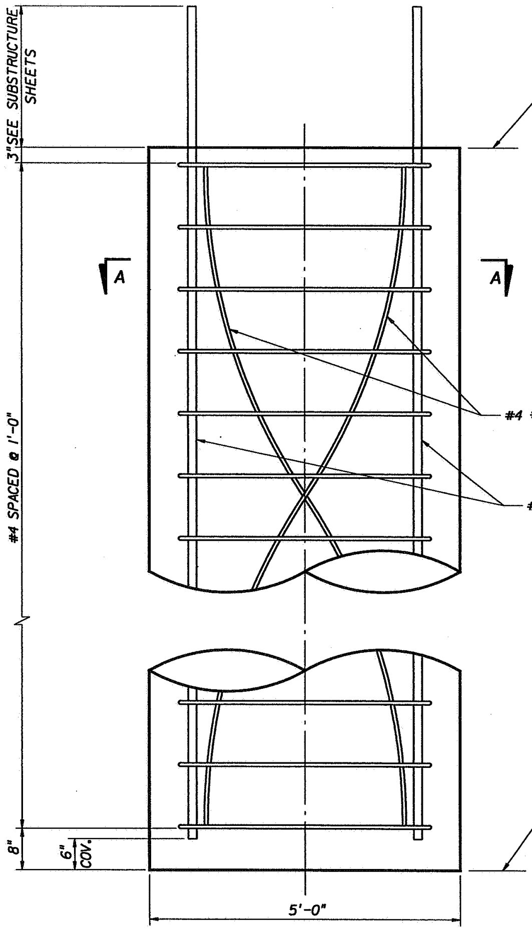
KEY SHEET REVISIONS

DATE	BY	DESCRIPTION
6/25/01	DBM	1 REVISED INDEX TO SHOW SHEET 4A
7/8/02	DBM	1 REVISED INDEX TO SHOW SHEETS 160A, 160B
12/9/02	DBM	14 ADDED SHEET 162B
05/13/03	EBS	18 MOVED REVISIONS 1 THRU 17 TO SHEET 1A.

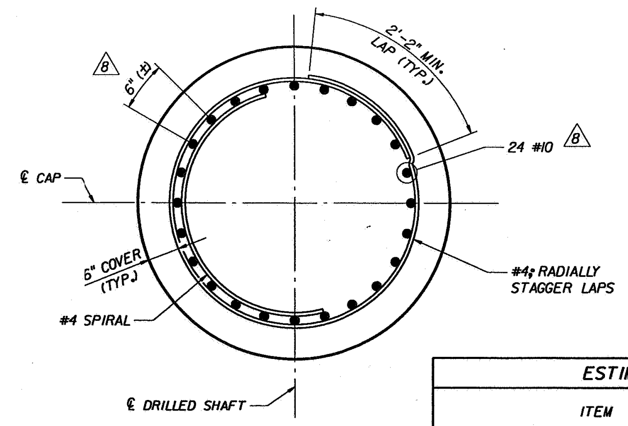
ROADWAY PLANS
ENGINEER OF RECORD: DAVID B MONEY, P.E.
P.E. NO.: 47022

Douglas L Butler
2/7/06

FISCAL YEAR	SHEET NO.
02	1



ELEVATION-60" Ø DRILLED SHAFT



SECTION A-A

ESTIMATED QUANTITIES			
ITEM	UNIT	QUANTITY	
		BRIDGE	RAMP
DRILLED SHAFT	L.F.	~~4761~~	83
UNCLASSIFIED SHAFT EXCAVATION	L.F.	~~4,435~~	85
CORE (SHAFT EXCAVATION)	L.F.	~~440~~	10

5

NOTES:

1. DRILLED SHAFTS SHALL BE PAID FOR AT THE CONTRACT UNIT PRICE PER LINEAR FOOT FOR DRILLED SHAFTS UNDER BID ITEM NO. 455-88-6. COMPENSATION FOR ADDITIONAL COST FOR MATERIALS AND CONSTRUCTION OF DRILLED SHAFTS SHALL BE PAID FOR UNDER THE APPROPRIATE BID ITEMS. SEE SUMMARY OF QUANTITIES AND SPECIFICATIONS. 1
2. CONCRETE SHALL BE POURED CONTINUOUSLY FROM TIP ELEVATION TO SHAFT HEAD ELEVATION OF DRILLED SHAFT.
3. THE CONTRACTOR SHALL SUBMIT TO THE ENGINEER PLANS AND CONSTRUCTION PROCEDURES FOR REVIEW AND APPROVAL AT LEAST 30 DAYS PRIOR TO BEGINNING CONSTRUCTION.
4. 8 A LENGTH OF LAP SPLICE IN #10 BARS: 6'-0".

* FIELD BEND TWO #4 BARS AT THE RATE OF ONE COMPLETE REVOLUTION PER 20 FEET OF SHAFT INSIDE VERTICAL REINFORCING. TIE AT INTERSECTION WITH VERTICAL BARS. USE MIN. LAP SPLICE 1'-0" AS NECESSARY.

** SEE SHEETS C-18 & C-20 FOR PLACEMENT OF DOWELS.

KERRIE LEE BARLOGA
LICENSE
NO. 41013
PROFESSIONAL ENGINEER
Kerri Barloga
4/25/05
Revision 5 Only

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
06/01	TV	1 REVISED BID ITEM NUMBER			
06/02	TV	8 REVISED REINFORCING AND NOTE			
5/03/02	VAZ	5 REVISIONS FOR SUBSTRUCTURE VECP			

	NAMES	DATES
DRAWN BY	JSR	6/00
CHECKED BY	DET	6/00
DESIGNED BY	TV	6/00
CHECKED BY	PRS	6/00
APPROVED BY	TV	

ENGINEER OF RECORD:
HDR Engineering, Inc.
2202 N. West Shore Boulevard Suite 250
Tampa, FL 33607-5755
(813) 282-2300
www.hdrinc.com
License No. 4213

FLORIDA DEPARTMENT OF TRANSPORTATION

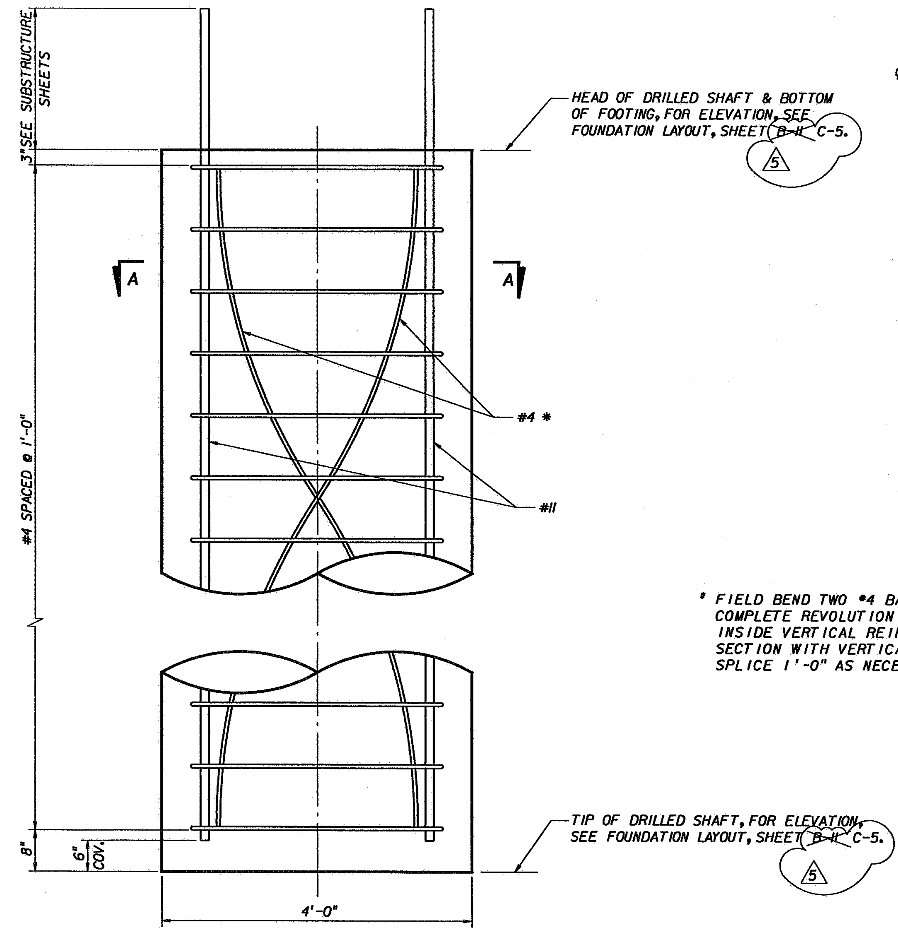
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
60	PINELLAS	257093-1-52-01

SHEET TITLE: 60" Ø DRILLED SHAFTS

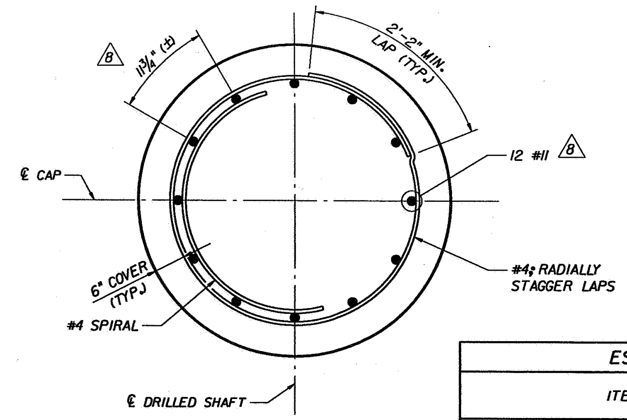
PROJECT NAME: MEMORIAL CAUSEWAY BRIDGE

SHEET NO. A-7

$USERNAME$ $$$$$$$$DGNSPECIFICATION$$$$$$$$
$$$$$$$$SYTIME$$$$$$$$$TIME$



ELEVATION-48" Ø DRILLED SHAFT



SECTION A-A

ESTIMATED QUANTITIES			
ITEM	UNIT	QUANTITY	
		BRIDGE	RAMP
DRILLED SHAFT	L.F.	~~538~~	173
UNCLASSIFIED SHAFT EXCAVATION	L.F.	~~388~~	189
CORE (SHAFT EXCAVATION)	L.F.	~~40~~	20

5

* FIELD BEND TWO #4 BARS AT THE RATE OF ONE COMPLETE REVOLUTION PER 20 FEET OF SHAFT INSIDE VERTICAL REINFORCING. TIE AT INTERSECTION WITH VERTICAL BARS. USE MIN. LAP SPLICE 1'-0" AS NECESSARY.

NOTES:

1. DRILLED SHAFTS SHALL BE PAID FOR AT THE CONTRACT UNIT PRICE PER LINEAR FOOT FOR DRILLED SHAFTS UNDER BID ITEM NO. 455-88-5. COMPENSATION FOR ADDITIONAL COST FOR MATERIALS AND CONSTRUCTION OF DRILLED SHAFTS SHALL BE PAID FOR UNDER THE APPROPRIATE BID ITEMS. SEE SUMMARY OF QUANTITIES AND SPECIFICATIONS. 1
2. CONCRETE SHALL BE POURED CONTINUOUSLY FROM TIP ELEVATION TO SHAFT HEAD ELEVATION OF DRILLED SHAFT.
3. THE CONTRACTOR SHALL SUBMIT TO THE ENGINEER PLANS AND CONSTRUCTION PROCEDURES FOR REVIEW AND APPROVAL AT LEAST 30 DAYS PRIOR TO BEGINNING CONSTRUCTION.
4. LENGTH OF LAP SPLICE IN #11 BARS: 7'-0"
5. (5) WORK THIS SHEET WITH SHEETS C-6 THRU C-8 AS NOTED ON THOSE SHEETS.

Kerri Barloga
4/25/05
Revision 5 only

A Ahoa
4-25-05

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
06/01	TV	1 REVISED BID ITEM NUMBER			
06/02	TV	8 REVISED REINFORCING AND NOTE			
5/03/02	VAZ	5 REVISIONS FOR SUBSTRUCTURE VECP			

	NAMES	DATES
DRAWN BY	SDS	
CHECKED BY	DET	
DESIGNED BY	TV	
CHECKED BY	PRS	
APPROVED BY	TV	

ENGINEER OF RECORD:
HDR Employee-owned
HDR Engineering, Inc.
2202 N. West Shore Boulevard
Suite 250
Tampa, FL 33607-5755
(813) 282-2300
www.hdrinc.com
License No. 4213

FLORIDA DEPARTMENT OF TRANSPORTATION

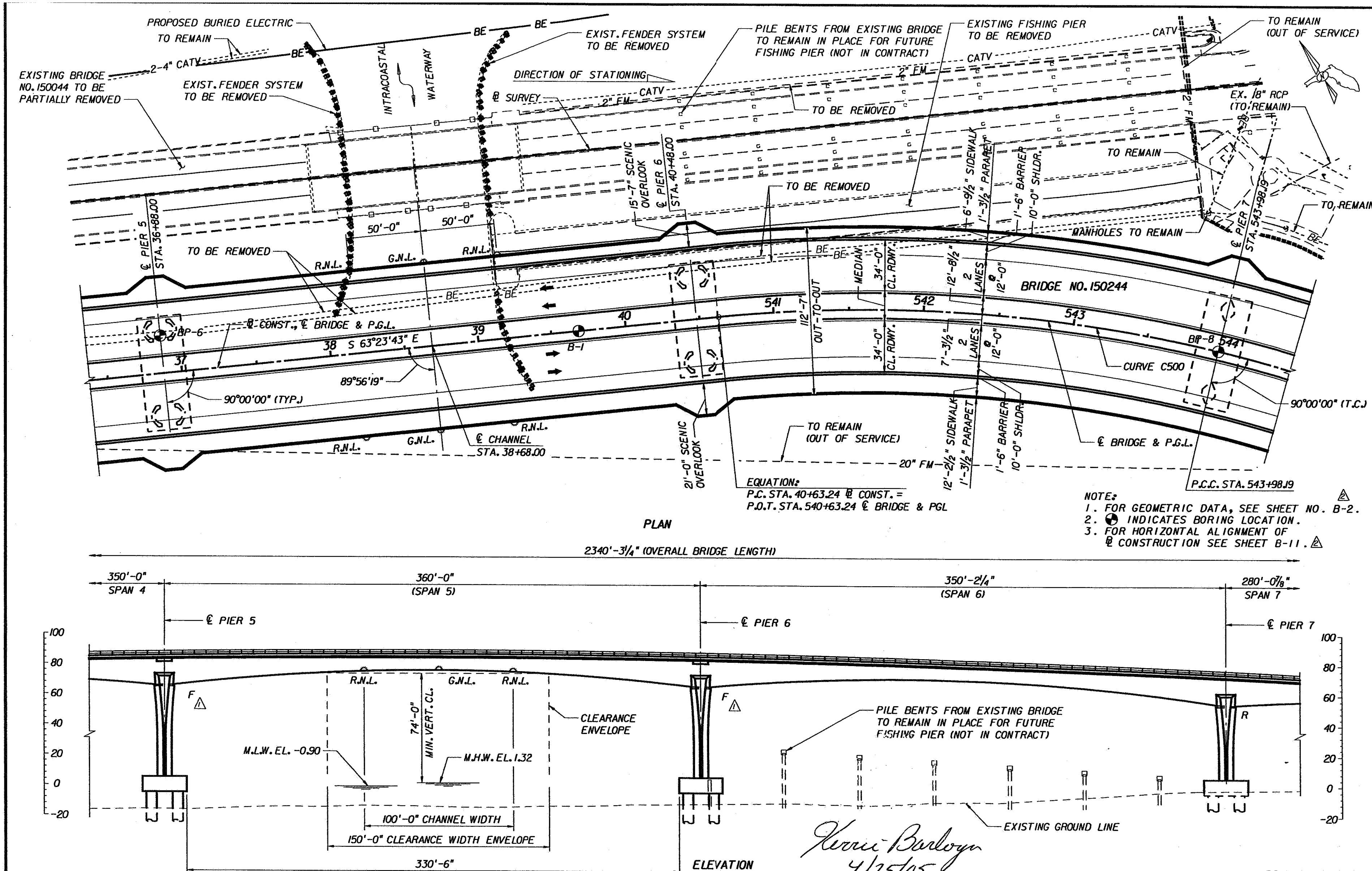
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
60	PINELLAS	257093-1-52-01

SHEET TITLE: 48" Ø DRILLED SHAFTS

PROJECT NAME: MEMORIAL CAUSEWAY BRIDGE

SHEET NO. A-8

$$$$$$$SYTIME$$$$$$$$$$TIME$ $USERNAME$ $$$$$$$$DGNSPECIFICATION$$$$$$$



NOTE:
1. FOR GEOMETRIC DATA, SEE SHEET NO. B-2.
2. ⊕ INDICATES BORING LOCATION.
3. FOR HORIZONTAL ALIGNMENT OF ℄ CONSTRUCTION SEE SHEET B-11.

Kerrie Barlow
4/25/05

BRIDGE NO. 150244

REVISIONS		
DATE	BY	DESCRIPTION
12/14/04	CWN	1 Revised as noted.
3/15/05	CAB	2 Revised sheet reference.

	NAMES	DATES
DRAWN BY	CAB	11-15-04
CHECKED BY	CWN	11-15-04
DESIGNED BY	YG	11-15-04
CHECKED BY	KLB	11-15-04
APPROVED BY	Phil Hartsfield, P.E.	

ENGINEER OF RECORD:
PARSONS
Yves Gauthier, P.E.
Florida License No. 36086
Certificate of Authorization No. 838
3500 Financial Plaza, Suite 300
Tallahassee, FL 32312-5998
(850)422-0000 Fax (850)422-3373

LOGO:
PCL CONSTRUCTION LEADERS
PARSONS

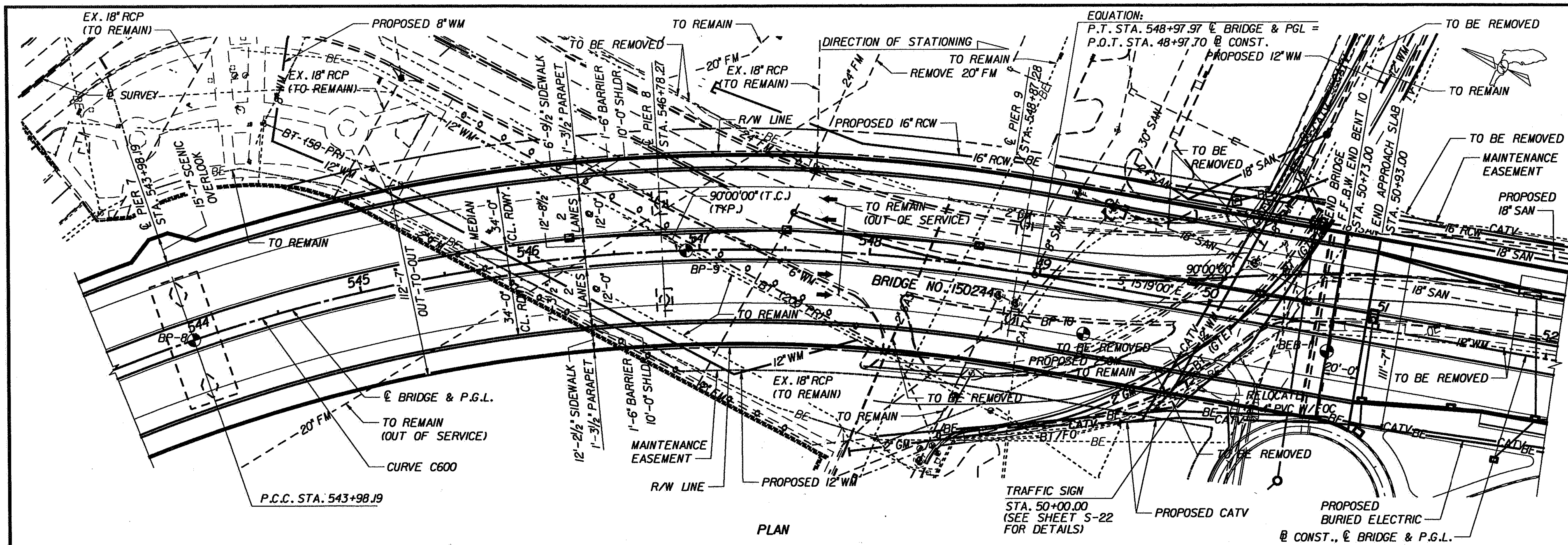
FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
60	PINELLAS	257093-1-52-01

SHEET TITLE: PLAN AND ELEVATION FOR ℄ BRIDGE (SHEET 1 OF 2)

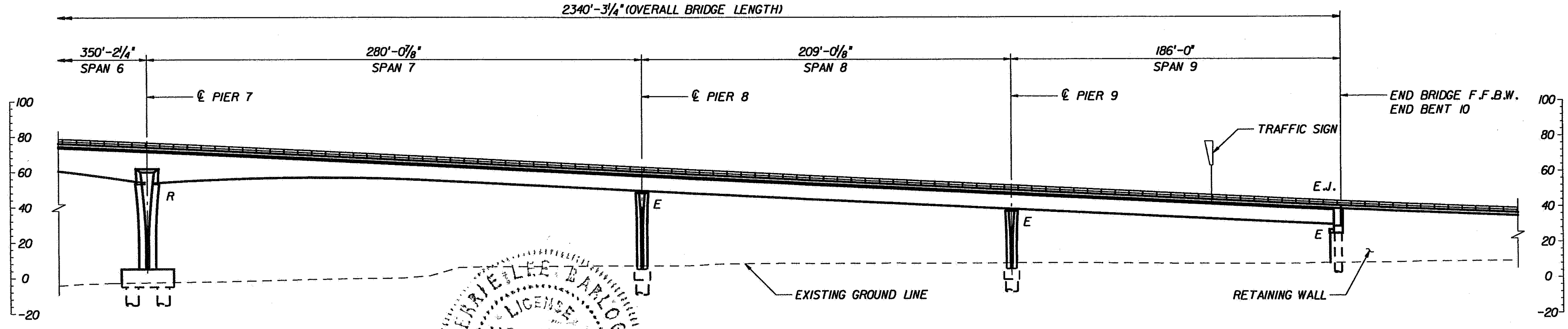
PROJECT NAME: MEMORIAL CAUSEWAY BRIDGE

SHEET NO. B-13



PLAN

NOTE:
1. FOR NOTES, SEE SHEET B-13.
2. FOR HORIZONTAL ALIGNMENT OF ℄ CONSTRUCTION SEE SHEET NO. B-12.



ELEVATION

KERRIE LEE BARLOGA LICENSE NO. 41813 FLORIDA 4/25/05

BRIDGE NO. 150244

REVISIONS		
Date	By	Description

	Names	Dates
Drawn by	JMH	03/03
Checked by	SGS	03/03
Designed by	JL	03/03
Checked by	KLB	3/15/05
Approved by	Yves Gauthier, P.E.	

ENGINEER OF RECORD:
PARSONS
Kerrie L. Barloga, P.E.
Florida License No. 41813
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(850)422-0000 Fax (850)422-3373

LOGO:
PCL CONSTRUCTION LEADERS
PARSONS

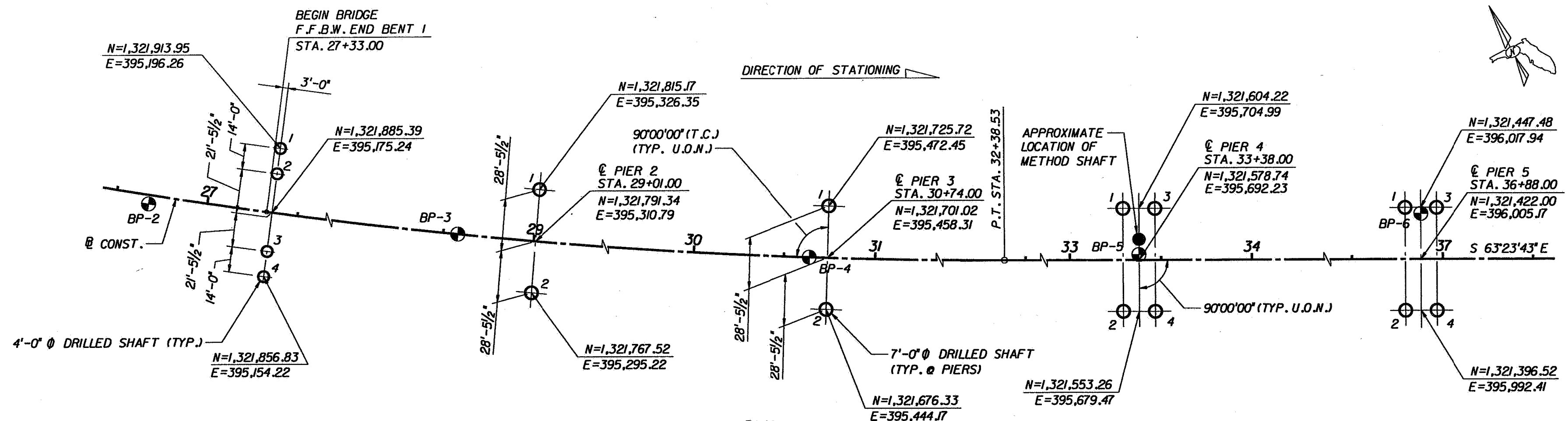
FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
60	PINELLAS	257093-1-52-01

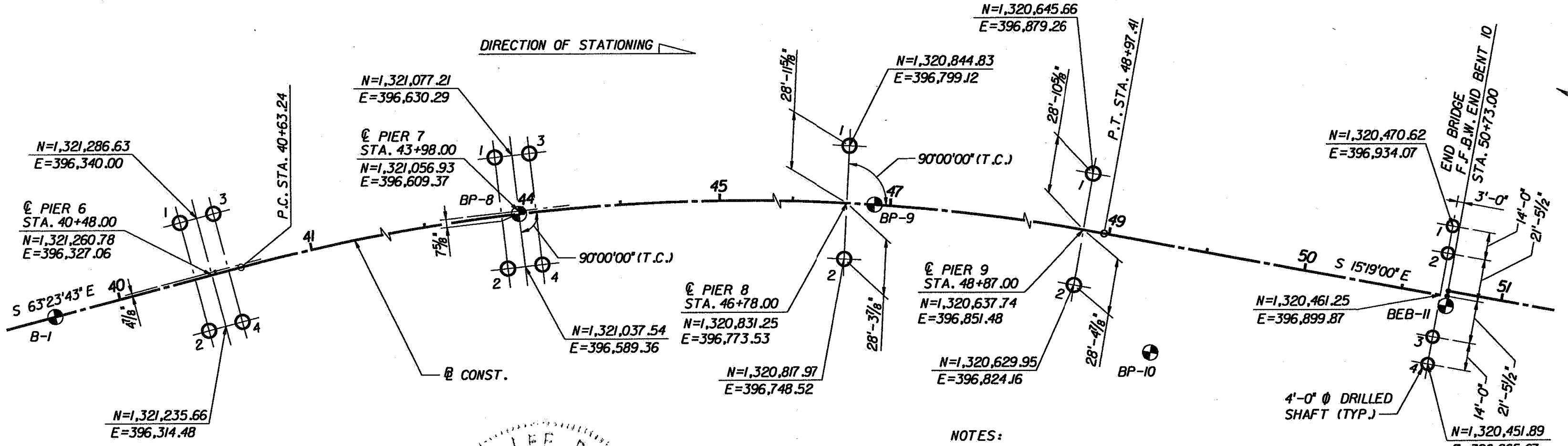
SHEET TITLE: PLAN AND ELEVATION FOR ℄ BRIDGE (SHEET 2 OF 2)

PROJECT NAME: MEMORIAL CAUSEWAY BRIDGE

SHEET NO. B-14



PLAN



PLAN

NOTES:

1. FOR HORIZONTAL CURVE DATA, SEE SHEET B-1.
2. FOR DRILLED SHAFT DATA, SEE SHEET B-17.
3. FOR PIERS 4, 5, 6 & 7 LAYOUT DIMENSIONS, SEE SHEET B-17.
4. ⊕ INDICATES BORING LOCATION.
5. FOR UTILITY LOCATIONS, SEE SHEET B-16.

KERRIE LEE BARLOGA LICENSE NO. 41813 PROFESSIONAL ENGINEER
Kerrie Barloga 4/25/05

BRIDGE NO. 150244

REVISIONS				Names	Dates
Date	By	Description	Drawn by	JMH	03/02
			Checked by	SGS	03/02
			Designed by	JL	03/02
			Checked by	KLB	3/15/05
			Approved by	Yves Gauthier, P.E.	

ENGINEER OF RECORD: PARSONS
Kerrie L. Barloga, P.E.
Florida License No. 41813
Certificate of Authorization No. 1838
3500 Financial Plaza, Suite 300
Tallahassee, Fl. 32312-5998
(850)422-0000 Fax (850)422-3373

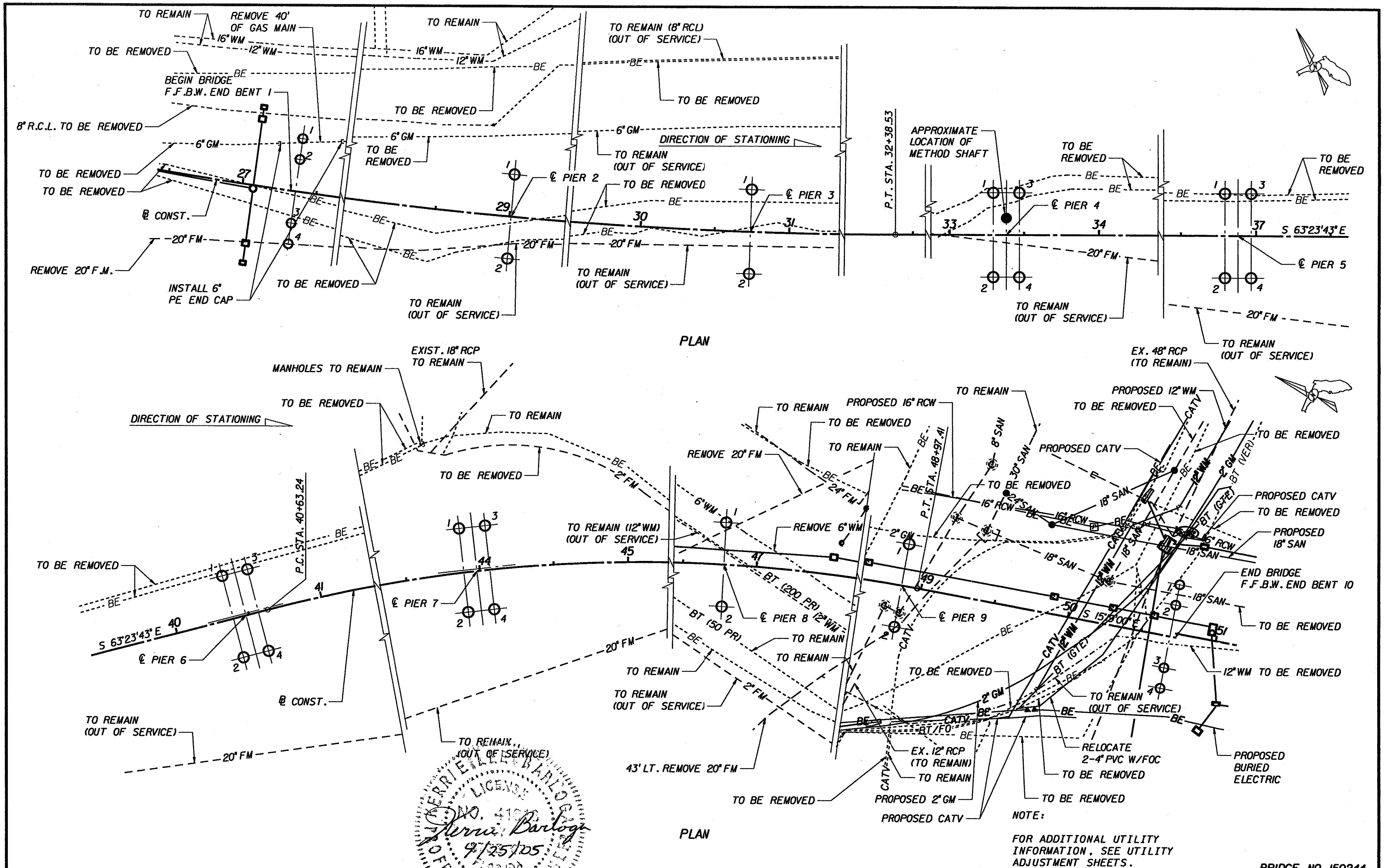
LOGO: PCL CONSTRUCTION LEADERS PARSONS

FLORIDA DEPARTMENT OF TRANSPORTATION		
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
60	PINELLAS	257093-1-52-01

SHEET TITLE: FOUNDATION LAYOUT (SHEET 1 OF 3)

PROJECT NAME: MEMORIAL CAUSEWAY BRIDGE

SHEET NO. B-15



NOTE:

FOR ADDITIONAL UTILITY INFORMATION, SEE UTILITY ADJUSTMENT SHEETS.

BRIDGE NO. 150244

REVISIONS		
Date	By	Description

	Names	Dates
Drawn by	JMH	03/02
Checked by	SGS	03/02
Designed by	JL	03/02
Checked by	KLB	3/15/05
Approved by	Yves Gauthier, P.E.	

ENGINEER OF RECORD:
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Florida License No. 41919
Certificate of Authorization No. 1838
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Tallahassee, Fl. 32312-5998
(850)422-0000 Fax (850)422-3373

LOGO:
PCL CONSTRUCTION LEADERS
PARSONS

FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
60	PINELLAS	257093-1-52-01

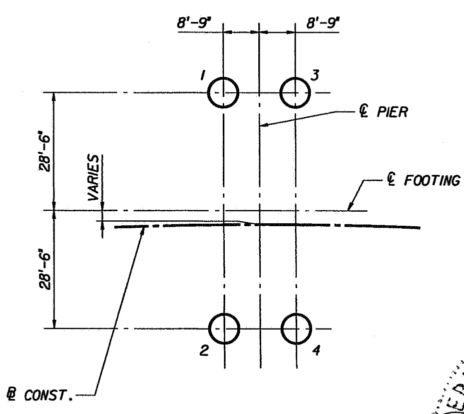
SHEET TITLE: FOUNDATION LAYOUT (SHEET 2 OF 3)

PROJECT NAME: MEMORIAL CAUSEWAY BRIDGE

SHEET NO. B-16

DRILLED SHAFT DATA TABLE							
INSTALLATION CRITERIA					DESIGN CRITERIA		
PIER OR BENT	SHAFT SIZE (IN.)	TIP ELEV. (FT.)	MIN. TIP ELEV. (FT.)	MIN. UNCASED ROCK SOCKET LENGTH (FT.)	FACTORED DESIGN LOAD (KIPS)	DOWNDRAG (KIPS)	APPROX. 100 YR. SCOUR ELEV. (FT.)
E.B. 1	48	-	-10	14	1686	260	N/A
PIER 2	84	-	-49	26	6109	N/A	N/A
PIER 3	84	-	-53	28	6003	N/A	N/A
PIER 4	84	-	-53	48	12460	N/A	-25
PIER 5	84	-	-58	44	9074	N/A	-32
PIER 6	84	-	-68	47	9098	N/A	-36
PIER 7	84	-	-58	60	12450	N/A	-30
PIER 8	84	-	-48	36	6767	N/A	N/A
PIER 9	84	-	-43	28	6761	N/A	N/A
E.B. 10	48	-	-10	13	1686	240	N/A

TOP OF DRILLED SHAFT ELEVATIONS	
PIER OR BENT	ELEVATION
E.B. 1	29.74
PIER 2	-0.90
PIER 3	-0.90
PIER 4	-2.90
PIER 5	-2.90
PIER 6	-2.90
PIER 7	-2.90
PIER 8	-0.90
PIER 9	-0.90
E.B. 10	24.73



PIERS 4, 5, 6 AND 7

DRILLED SHAFT INSTALLATION NOTES:

1. ONE 84 INCH DIAMETER METHOD SHAFT SHALL BE CONSTRUCTED AT THE SITE. THE METHOD SHAFT SHALL BE CONSTRUCTED TO THE DEPTH SPECIFIED BY THE ENGINEER AT A NON-PRODUCTION SHAFT LOCATION AS SHOWN ON THE FOUNDATION LAYOUT SHEET.

2. TIP ELEVATION - THE ELEVATION TO WHICH THE SHAFT SHALL BE CONSTRUCTED WILL BE DETERMINED FROM LOAD TEST DATA, ROCK CORES, OR OTHER GEOTECHNICAL INFORMATION OBTAINED DURING CONSTRUCTION. THIS INFORMATION WILL BE PROVIDED BY THE CONTRACTOR FOR REVIEW AND APPROVAL BY THE ENGINEER.

3. MIN. TIP ELEVATION - THE HIGHEST ELEVATION THAT THE SHAFT TIP MAY BE CONSTRUCTED IF ADJUSTMENTS ARE MADE TO THE TIP ELEVATIONS SPECIFIED.

4. SPECIAL PRECAUTIONS SHALL BE TAKEN BY THE CONTRACTOR TO PREVENT DISCHARGE OF SLURRY OR EXCAVATED MATERIALS INTO THE INTRACOASTAL WATERWAY DURING THE EXCAVATION PROCEDURES. MATERIALS EXCAVATED FROM THE DRILLED SHAFTS SHALL BE COLLECTED, LOADED ON A BARGE AND DISPOSED OF AT AN OFFSITE LOCATION PROVIDED BY THE CONTRACTOR AND APPROVED BY THE ENGINEER.

5. SPECIAL PRECAUTIONS SHALL BE TAKEN BY THE CONTRACTOR TO PREVENT DISCHARGE OF SLURRY OR CONCRETE INTO THE INTRACOASTAL WATERWAY DURING THE CONCRETE OPERATIONS. THE CONTRACTOR SHALL PROVIDE A SUITABLE RECEPTACLE AROUND THE TOP OF THE CASING TO COLLECT SLURRY AND/OR CONCRETE DISCHARGED FROM THE SHAFT DURING CONCRETE OPERATIONS.

6. SLURRY AS DESCRIBED IN STANDARD SPECIFICATION 455-15.11.5 WILL INCLUDE NATURAL SLURRY (SLURRY FORMED FROM WATER AND NATURAL MATERIAL IN THE HOLE) FOR THE PURPOSES OF THE TIME OF EXCAVATION.

7. THE SLUMP LOST TEST DESCRIBED IN STANDARD SPECIFICATION 346-3.2 SHALL BE PERFORMED AT THE PLANT.

8. PILOT HOLES SHALL BE PERFORMED PRIOR TO SHAFT EXCAVATION. THE PILOT HOLES SHALL BE SPT SAMPLED AT 2.5 FOOT INTERVALS FROM THE TOP OF THE ROCK LAYER OR FROM THE SCOUR ELEVATION TO A MINIMUM OF 5 FEET BELOW THE TIP ELEVATION SHOWN IN THE DRILLED SHAFT DATA TABLE. WHEN SCOUR IS ANTICIPATED, PILOT HOLES SHALL BE SPT SAMPLED FROM THE SCOUR ELEVATION. ANY ROCK ENCOUNTERED ABOVE THE SCOUR ELEVATION SHALL NOT BE USED WHEN DETERMINING THE SOCKET LENGTH. ROCK CORING SHALL BE PERFORMED AT ONE PILOT HOLE LOCATION PER INTERIOR PIER AS DIRECTED BY THE ENGINEER

9. THE ROCK SOCKET SHALL BE DEFINED AS THE UNCASED PORTION OF THE SHAFT BELOW THE SCOUR ELEVATION.

10. MINIMUM TIP ELEVATIONS PROVIDED IN THE DRILLED SHAFT DATA TABLES ARE PROVIDED TO MAINTAIN LATERAL STABILITY.

11. CROSS HOLE SONIC LOGGING SHALL BE CONDUCTED IN ACCORDANCE WITH THE TECHNICAL SPECIAL PROVISION AT PRODUCTION SHAFTS LOCATED WITHIN PIERS 2 THROUGH 9.

12. SOCKET MATERIALS INCLUDE HIGHLY WEATHERED TO SLIGHTLY WEATHERED LIMESTONES AND INDURATED CLAYS.

NOTE:

FOR HORIZONTAL CURVE DATA, SEE SHEET B-1.

Kerrie Barloga
4/25/05

BRIDGE NO. 150244

REVISIONS		
Date	By	Description

	Names	Dates
Drawn by	JMH	03/02
Checked by	SGS	03/02
Designed by	JL	03/02
Checked by	KLB	3/15/05
Approved by	Yves Gauthier, P.E.	

ENGINEER OF RECORD:
PARSONS
Kerrie L. Barloga, P.E.
Florida License No. 41813
Certificate of Authorization No. 1838
3500 Financial Plaza, Suite 300
Tallahassee, Fl. 32312-5998
(850)422-0000 Fax (850)422-3373

LOGO: PCL CONSTRUCTION LEADERS PARSONS

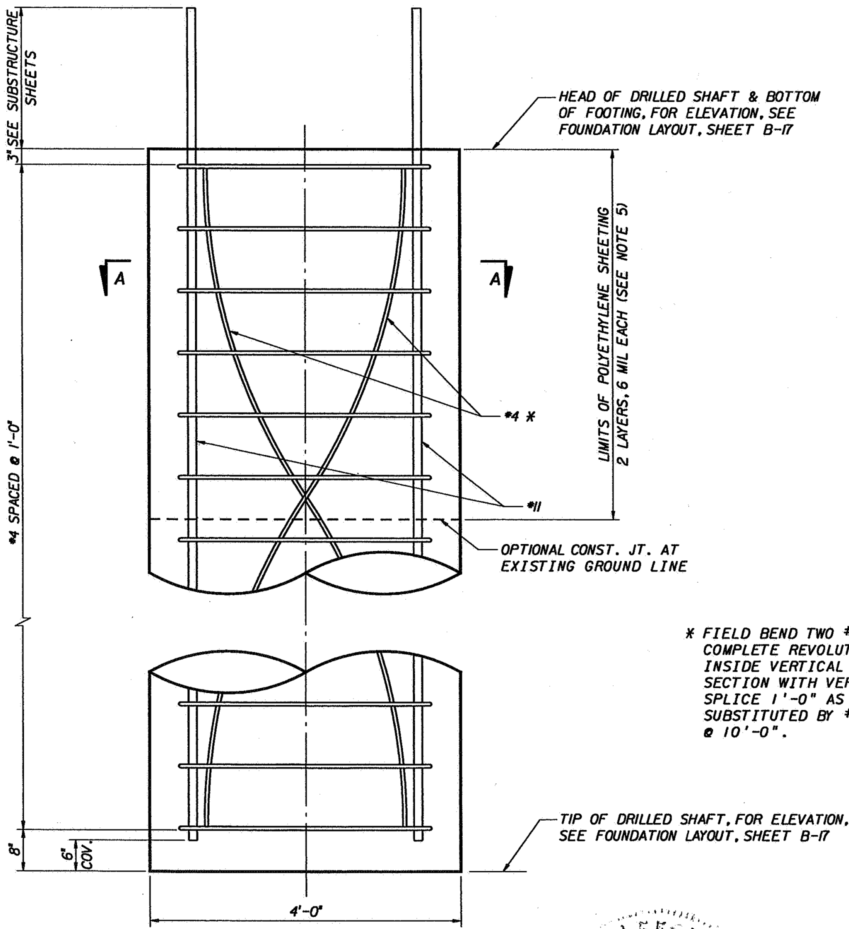
FLORIDA DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
60	PINELLAS	257093-1-52-01

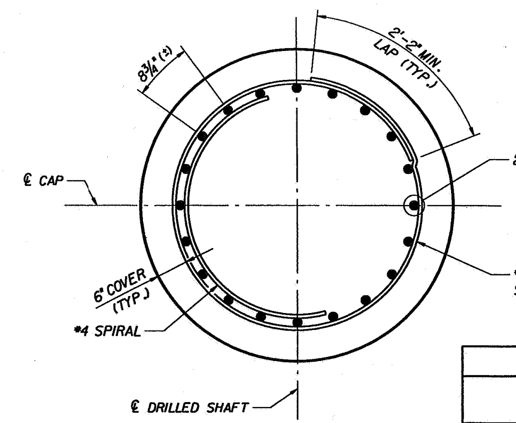
SHEET TITLE: FOUNDATION LAYOUT (SHEET 3 OF 3)

PROJECT NAME: MEMORIAL CAUSEWAY BRIDGE

SHEET NO. B-17



ELEVATION-48"Ø DRILLED SHAFT



SECTION A-A

✱ FIELD BEND TWO #4 BARS AT THE RATE OF ONE COMPLETE REVOLUTION PER 20 FEET OF SHAFT INSIDE VERTICAL REINFORCING. TIE AT INTERSECTION WITH VERTICAL BARS. USE MIN. LAP SPLICE 1'-0" AS NECESSARY. THIS MAY BE SUBSTITUTED BY #8 INTERNAL HOOPS SPACED @ 10'-0".

ESTIMATED QUANTITIES			
ITEM	UNIT	QUANTITY	
		BRIDGE	RAMP
DRILLED SHAFT	L.F.	538	173
UNCLASSIFIED SHAFT EXCAVATION	L.F.	388	189
CORE (SHAFT EXCAVATION)	L.F.	40	20

NOTES:

1. DRILLED SHAFTS SHALL BE PAID FOR AT THE CONTRACT UNIT PRICE PER LINEAR FOOT FOR DRILLED SHAFTS UNDER BID ITEM NO. 455-88-5. COMPENSATION FOR ADDITIONAL COST FOR MATERIALS AND CONSTRUCTION OF DRILLED SHAFTS SHALL BE PAID FOR UNDER THE APPROPRIATE BID ITEMS. SEE SUMMARY OF QUANTITIES AND SPECIFICATIONS.
2. CONCRETE SHALL BE POURED CONTINUOUSLY FROM TIP ELEVATION TO SHAFT HEAD ELEVATION OF DRILLED SHAFT.
3. THE CONTRACTOR SHALL SUBMIT TO THE ENGINEER PLANS AND CONSTRUCTION PROCEDURES FOR REVIEW AND APPROVAL AT LEAST 30 DAYS PRIOR TO BEGINNING CONSTRUCTION.
4. LENGTH OF LAP SPLICE IN #11 BARS: 7'-0".
5. IN ACCORDANCE WITH SECTION 459 OF THE STANDARD SPECIFICATIONS.

KERRIE LEE BARLOGA
Kerrie Barloga
4/25/05

BRIDGE NO. 150244

REVISIONS		
Date	By	Description

	Names	Dates
Drawn by	JMH	03/02
Checked by	SGS	03/02
Designed by	VAZ	03/02
Checked by	KLB	3/15/05
Approved by	Yves Gauthier, P.E.	

ENGINEER OF RECORD:
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Kerrie L. Barloga, P.E.
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LOGO:
PCL
CONSTRUCTION LEADERS
PARSONS

FLORIDA DEPARTMENT OF TRANSPORTATION

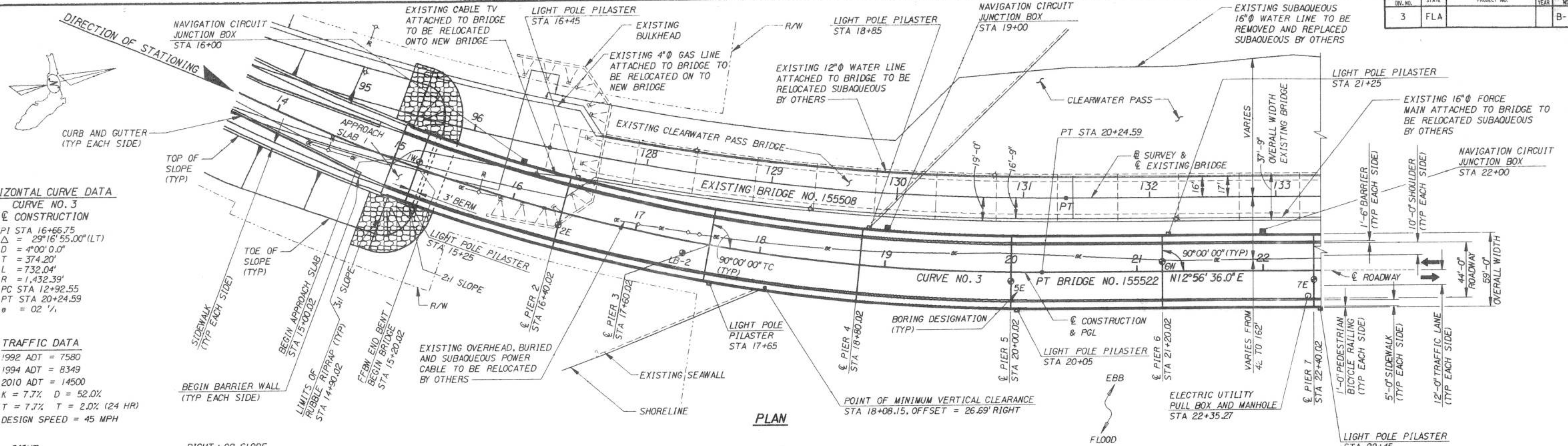
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
60	PINELLAS	257093-1-52-01

SHEET TITLE: 48"Ø DRILLED SHAFTS

PROJECT NAME: MEMORIAL CAUSEWAY BRIDGE

SHEET NO. B-18

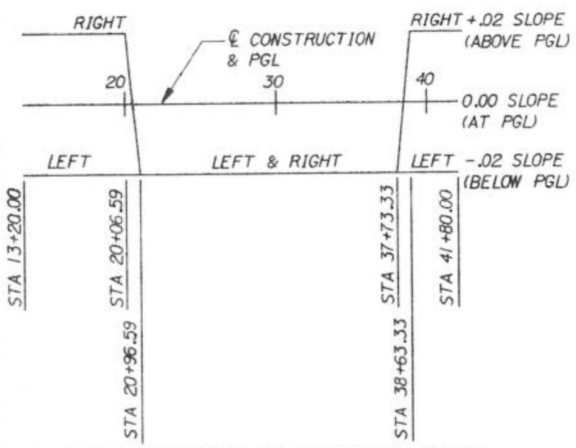
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA			B-11



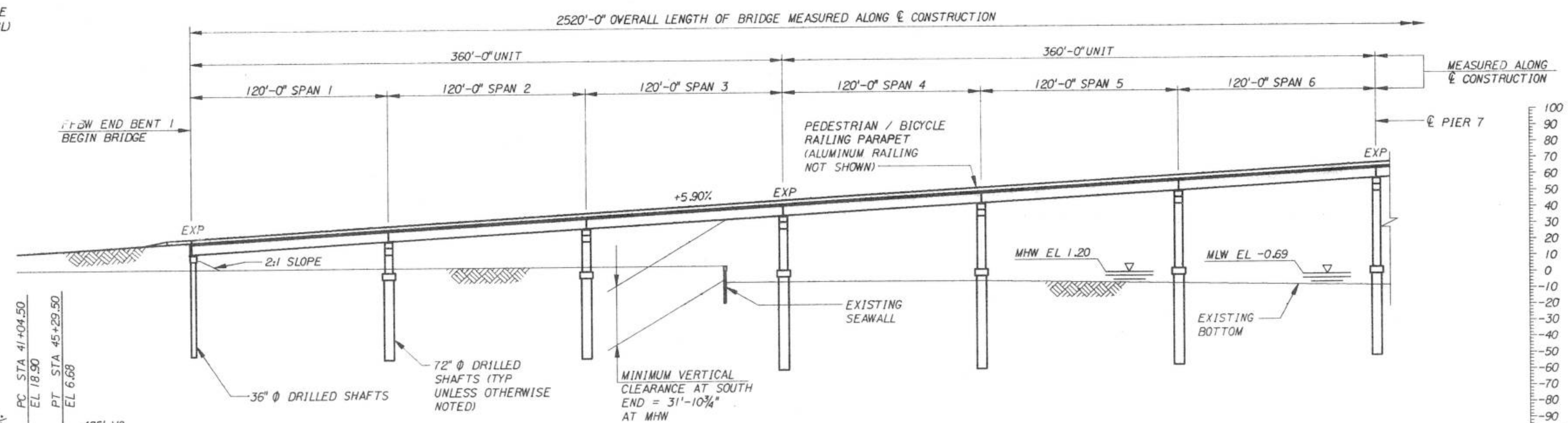
HORIZONTAL CURVE DATA
CURVE NO. 3
℄ CONSTRUCTION
PI STA 16+66.75
Δ = 29°16' 55.00"(LT)
D = 4°00' 0.0"
T = 374.20'
L = 732.04'
R = 1,432.39'
PC STA 12+92.55
PT STA 20+24.59
e = 02 '/,

TRAFFIC DATA
1992 ADT = 7580
1994 ADT = 8349
2010 ADT = 14500
K = 7.7% D = 52.0%
T = 7.7% T = 2.0% (24 HR)
DESIGN SPEED = 45 MPH

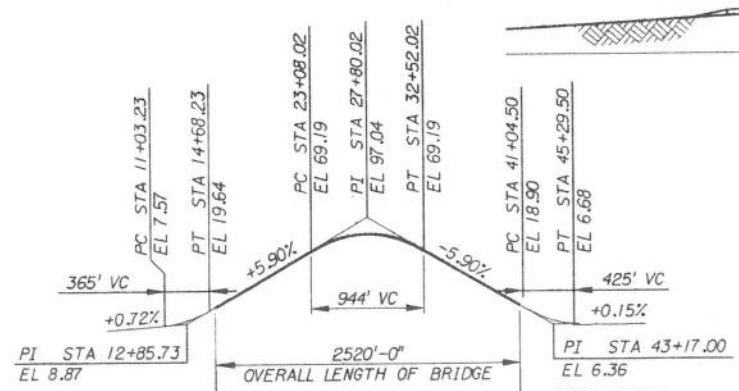
NOTE: REFER TO STATE PROJECTS NO. 15140-6518 AND 15140-6519 FOR ADDITIONAL UTILITY CONDUITS TO BE INSTALLED WITH BRIDGE.



SUPERELEVATION TRANSITION DATA



ELEVATION
LOOKING WEST
(EXISTING BRIDGE AND OVERHEAD LIGHTING NOT SHOWN FOR CLARITY)



VERTICAL CURVE DATA

NOTES:
1. ELEVATIONS ARE NATIONAL GEODETIC VERTICAL DATUM OF 1929 (NGVD '29).
2. BEARING DESIGNATION: EXP = EXPANSION JOINT IN DECK SLAB.
3. FFBW INDICATES FRONT FACE OF BACKWALL.
4. MHW IS DESIGN HIGH WATER.
5. ⊕ INDICATES A SOIL BORING LOCATION.
6. EXISTING BRIDGE NO. 155508 IS TO BE REMOVED (88,176 SF). SEE BRIDGE DEMOLITION PLANS.

BRIDGE NO. 155522

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	KBD/RLL	12-93
Checked by	DBT	12-93
Designed by	KBD	12-93
Checked by	RJH	12-93
Approved by	T. C. WHITE	

ENGINEER OF RECORD:
DAVID VOLKERT & ASSOCIATES, INC.
3409 West Lemon Street, Suite 1
Tampa, Florida 33609

LOGO: DAVID VOLKERT & ASSOCIATES, INC.

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

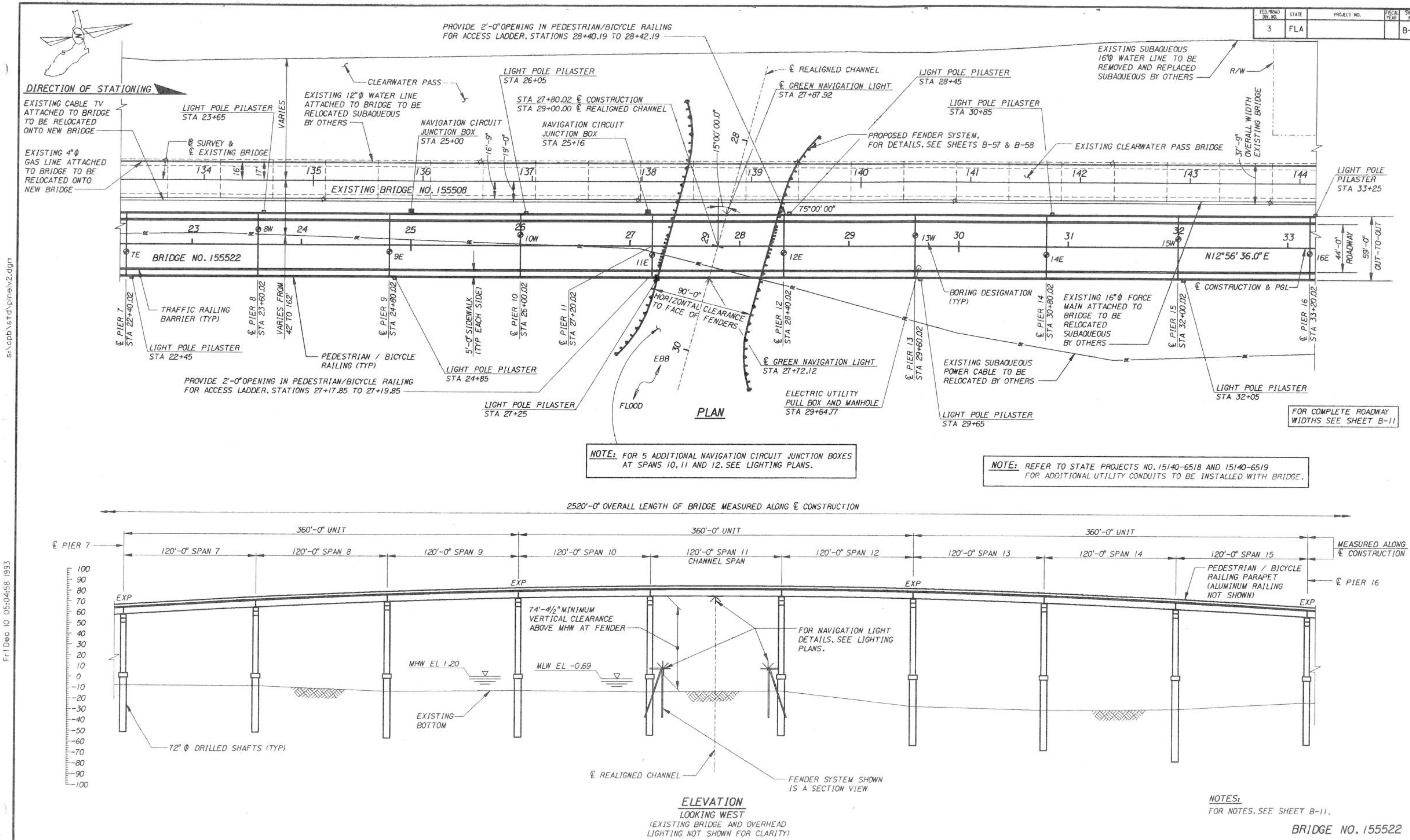
ROAD NO.	COUNTY	PROJECT NO.
	PINELLAS	15140-3518

SHEET TITLE: PLAN AND ELEVATION, SPANS 1 THRU 6

Drawing No.

PROJECT NAME: CLEARWATER PASS BRIDGE

Index No.



REVISIONS							Names	Dates
Date	By	Description	Date	By	Description	Drawn by	KBD/RLL	12-93
						Checked by	DBT	12-93
						Designed by	KBD	12-93
						Checked by	RJH	12-93
						Approved by	T. C. WHITE	

ENGINEER OF RECORD:
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SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
——	PINELLAS	15140-3518

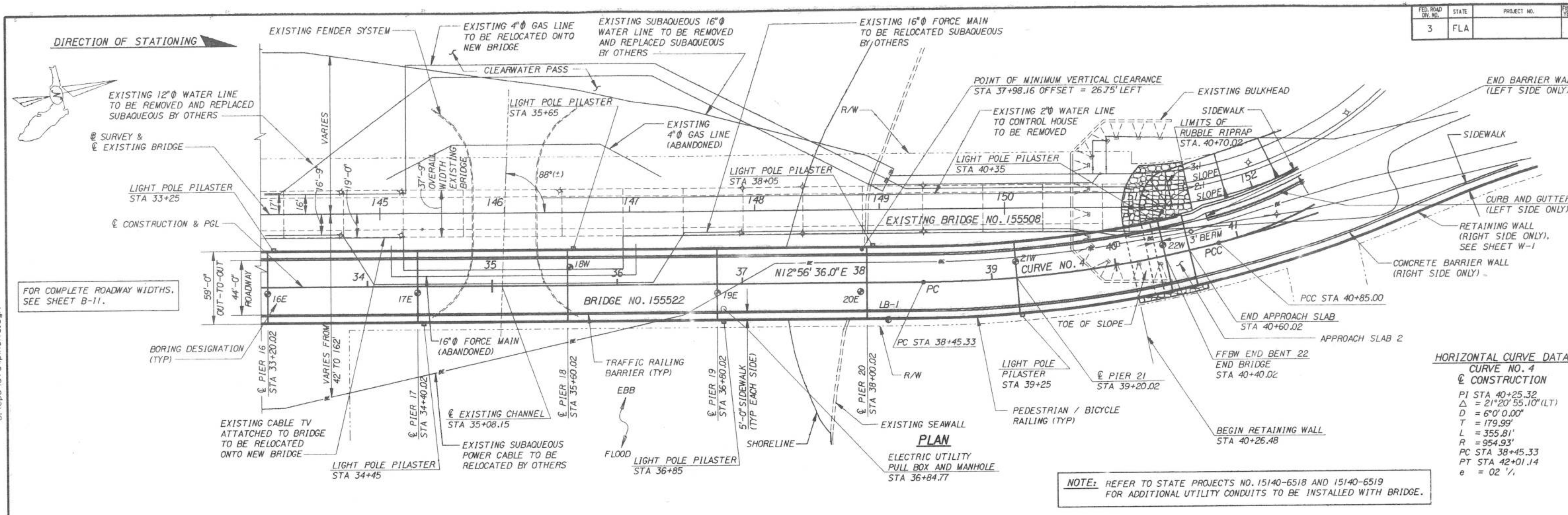
SHEET TITLE: PLAN AND ELEVATION, SPANS 7 THRU 15

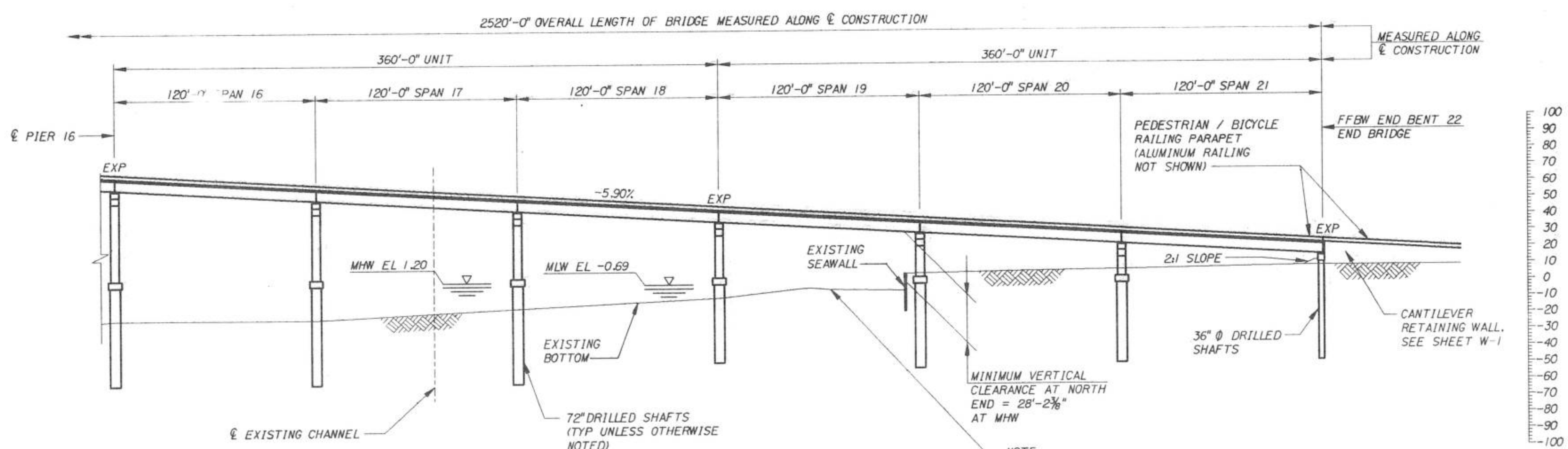
PROJECT NAME: CLEARWATER PASS BRIDGE

Drawing No.

Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA			B-13





ELEVATION
LOOKING WEST
(EXISTING BRIDGE AND OVERHEAD LIGHTING NOT SHOWN FOR CLARITY)

NOTES:
FOR NOTES, SEE SHEET B-11.

BRIDGE NO. 155522

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Fri Dec 10 05:08:04 1993

REVISIONS						
Date	By	Description	Date	By	Description	

	Names	Dates
Drawn by	KBD/RLL	12-93
Checked by	DBT	12-93
Designed by	KBD	12-93
Checked by	RJH	12-93
Approved by	T. C. WHITE	

ENGINEER OF RECORD:
DAVID VOLKERT & ASSOCIATES, INC.
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SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

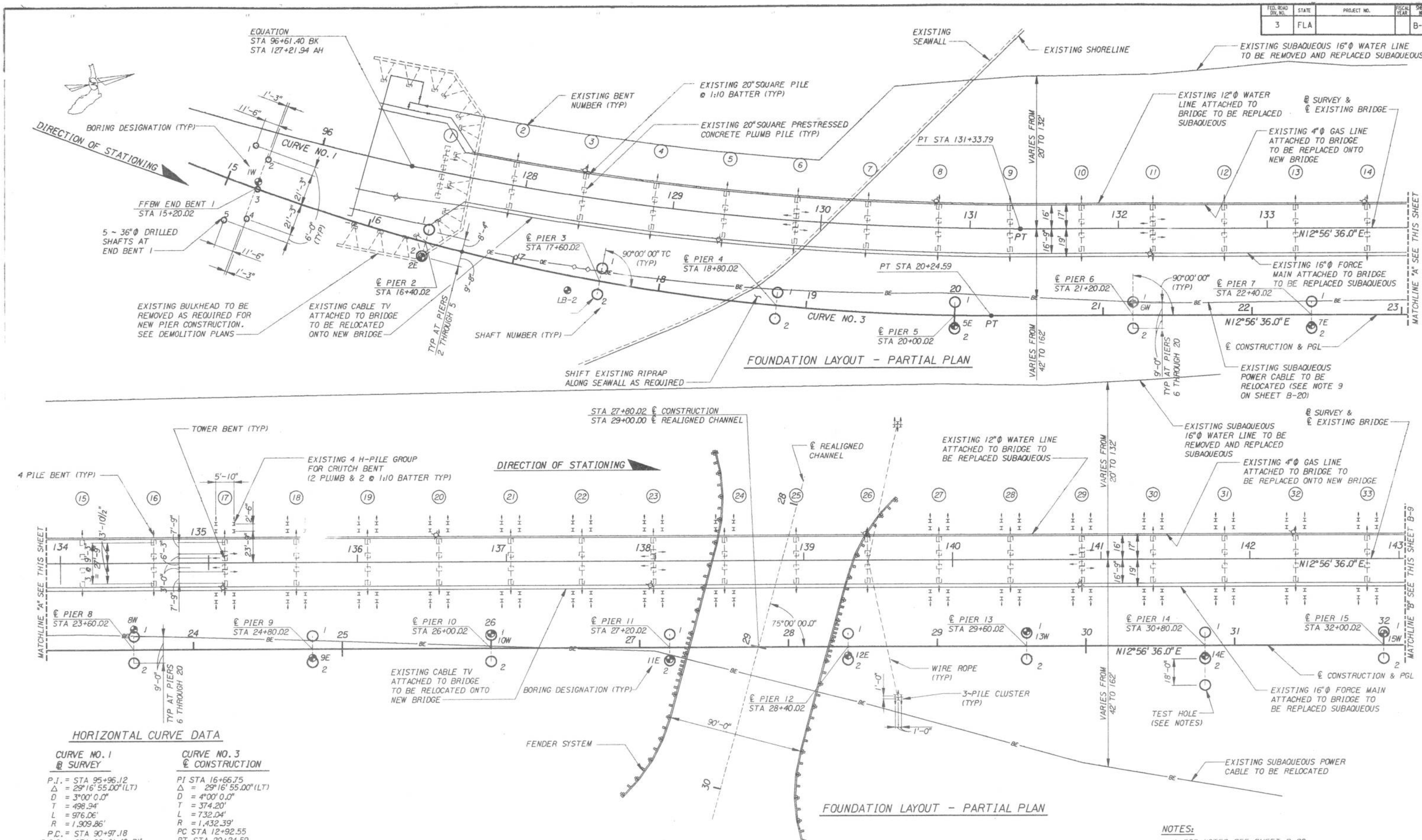
ROAD NO.	COUNTY	PROJECT NO.
	PINELLAS	15140-3518

SHEET TITLE: PLAN AND ELEVATION, SPANS 16 THRU 21

PROJECT NAME: CLEARWATER PASS BRIDGE

Drawing No.

Index No.



HORIZONTAL CURVE DATA

CURVE NO. 1 ℞ SURVEY	CURVE NO. 3 ℄ CONSTRUCTION
P.I. = STA 95+96.12	PI STA 16+66.75
Δ = 29°16' 55.00" (LT)	Δ = 29°16' 55.00" (LT)
D = 3°00' 0.0"	D = 4°00' 0.0"
T = 498.94'	T = 374.20'
L = 976.06'	L = 732.04'
R = 1,909.86'	R = 1,432.39'
P.C. = STA 90+97.18	PC STA 12+92.55
P.O.C. = STA 96+61.40 BK = STA 127+21.94 AH	PT STA 20+24.59
P.T. = STA 131+33.79	e = 02 %

NOTES:

1. FOR NOTES. SEE SHEET B-20.
2. FOR STATION OF EXISTING BENTS. SEE SHEET B-20.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA			B-19

REVISIONS

	Names	Dates
Drawn by	KBD/RLL	12-93
Checked by	DBT	12-93
Designed by	EWB	12-93
Checked by	KBD	12-93
Approved by	T. C. WHITE	

ENGINEER OF RECORD:
DAVID VOLKERT & ASSOCIATES, INC.
3409 West Lemon Street, Suite 1
Tampa, Florida 33609

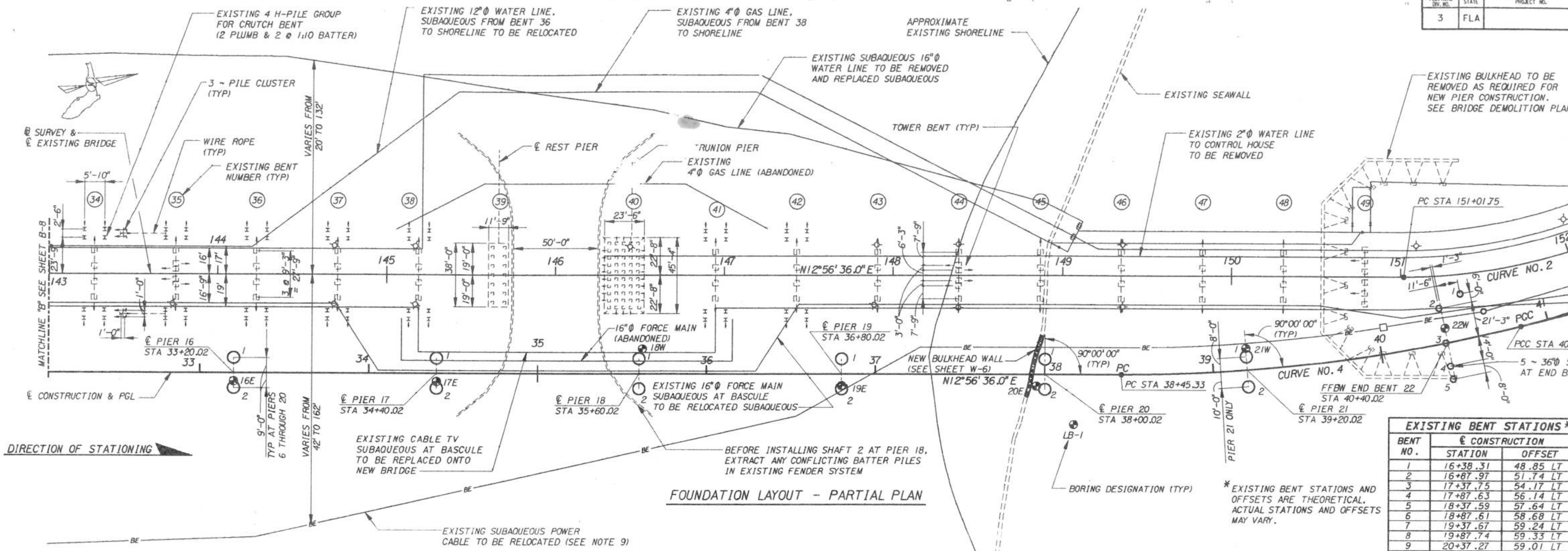
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
	PINELLAS	15140-3518

SHEET TITLE: FOUNDATION LAYOUT, SHEET 1 OF 2

PROJECT NAME: CLEARWATER PASS BRIDGE

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA			B-20



FOUNDATION LAYOUT - PARTIAL PLAN

*EXISTING BENT STATIONS AND OFFSETS ARE THEORETICAL. ACTUAL STATIONS AND OFFSETS MAY VARY.

EXISTING BENT STATIONS *		
BENT NO.	℄ CONSTRUCTION STATION	℄ CONSTRUCTION OFFSET
1	16+38.31	48.85 LT
2	16+87.97	51.74 LT
3	17+37.75	54.17 LT
4	17+87.63	56.14 LT
5	18+37.59	57.64 LT
6	18+87.61	58.68 LT
7	19+37.67	59.24 LT
8	19+87.74	59.33 LT
9	20+37.27	59.01 LT
10	20+85.27	59.00 LT
11	21+33.27	
12	21+81.27	
13	22+29.27	
14	22+77.27	
15	23+25.27	
16	23+73.27	
17	24+21.27	
18	24+69.27	
19	25+17.27	
20	25+65.27	
21	26+13.27	
22	26+61.27	
23	27+09.27	
24	27+57.27	
25	28+05.27	
26	28+53.27	
27	29+01.27	
28	29+49.27	
29	29+97.27	
30	30+45.27	
31	30+93.27	
32	31+41.27	
33	31+89.27	
34	32+37.27	
35	32+85.27	
36	33+33.27	
37	33+81.27	
38	34+29.27	
39	34+77.27	
40	35+56.35	
41	36+05.06	
42	36+53.06	
43	37+01.06	
44	37+49.06	
45	37+97.06	
46	38+45.06	59.00 LT
47	38+96.15	57.73 LT
48	39+46.98	53.90 LT
49	39+97.23	47.54 LT

NOTES:

1. THE CONTRACTOR SHALL CUT A PILOT HOLE AT EACH SHAFT LOCATION. OR AS OTHERWISE DIRECTED BY THE ENGINEER, AND THE RESULTS SHALL BE SUBMITTED TO THE ENGINEER IN ACCORDANCE WITH THE TECHNICAL SPECIAL PROVISIONS.
2. PAYMENT FOR PILOT HOLES IS INCLUDED IN BID ITEM NO. 455-111. CORE (SHAFT EXCAVATION).
3. DRILLED SHAFTS SHALL BE INSTALLED TO ACHIEVE THE AUTHORIZED MINIMUM TIP ELEVATIONS. AUTHORIZED MINIMUM TIP ELEVATIONS WILL BE PROVIDED TO THE CONTRACTOR BY THE DEPARTMENT SUBSEQUENT TO THE PILOT HOLE WORK IN ACCORDANCE WITH THE TECHNICAL SPECIAL PROVISIONS. DRILLED SHAFTS SHALL BE EXTENDED DEEPER IF SO DIRECTED BY THE ENGINEER.
4. THE EXPECTED TIP ELEVATIONS, AS INDICATED IN THE TABLE ON THIS SHEET, AND THE ASSOCIATED ESTIMATED QUANTITIES PRESENTED ON SHEETS B-21 AND B-22 ARE PRESENTED FOR THE ESTABLISHMENT OF CONTRACT UNIT PRICES. ALTHOUGH THE EXPECTED TIP ELEVATIONS SHOWN ARE BELIEVED TO BE REPRESENTATIVE FOR THE PROJECT, THE AUTHORIZED MINIMUM TIP ELEVATIONS MAY BE OF GREATER OR LESSER DEPTH.
5. ALL SHAFTS SHALL BE PLUMB.
6. CARE MUST BE TAKEN TO PREVENT CONTAMINATION OF THE SURROUNDING WATER BY DRILL CUTTINGS AND WASHINGS. MATERIALS SHALL BE DISPOSED OF OFF-SITE. APPROPRIATE FLOATING TURBIDITY BARRIERS SHALL BE PROVIDED. PAYMENT FOR FLOATING TURBIDITY BARRIERS INCLUDED IN ROADWAY ITEMS.
7. EXCAVATE A 72" Ø TEST HOLE TO AN ELEVATION OF -76.0' ADJACENT TO PIER 14. IN THE POSITION INDICATED IN THE FOUNDATION LAYOUT PLAN, OR AS OTHERWISE DIRECTED BY THE ENGINEER. FOR A TEST HOLE LOCATED WITHIN THE WATERWAY, THAT PORTION OF THE HOLE ABOVE THE PREDICTED SCOUR ELEVATION SHALL BE FILLED WITH SOIL CHARACTERISTICLY SIMILAR TO THE WATERWAY BOTTOM.
8. ⊕ INDICATES A SOIL BORING LOCATION.
9. SUBAQUEOUS POWER CABLE WILL BE TEMPORARILY SHIFTED BY OTHERS TO CLEAR FOUNDATIONS AS REQUIRED.
10. TEMPORARY OR PERMANENT CASING MAY BE REQUIRED. THERE IS NO SEPARATE PAY ITEM FOR CASING AND ALL COSTS SHALL BE INCLUDED IN THE PRICE OF THE DRILLED SHAFTS.

DRILLED SHAFT INSTALLATION TABLE					
BENT OR PIER	SHAFT SIZE	DESIGN LOAD	EXPECTED TIP ELEVATION	PREDICTED SCOUR ELEVATION	TOP ELEVATION
1	36"Ø	350 TONS	-47.0	-----	VARIES *
2	72"Ø	860 TONS	-50.0	-----	0.500
3		870 TONS	-50.0	-----	-1.500
4		880 TONS	-58.0	-12.4	-1.690
5		890 TONS	-58.0	-17.9	
6		910 TONS	-56.0	-25.5	
7		920 TONS	-51.0	-25.5	
8		950 TONS	-57.0	-30.8	
9		980 TONS	-56.0	-23.9	
10		1000 TONS	-55.0	-26.2	
11		1010 TONS	-53.0	-25.7	
12		1010 TONS	-53.0	-25.7	
13		1000 TONS	-62.0	-33.5	
14		980 TONS	-66.0	-33.5	
15		950 TONS	-76.0	-33.1	
16		920 TONS	-60.0	-33.1	
17		910 TONS	-60.0	-33.1	
18		890 TONS	-60.0	-33.1	
19		870 TONS	-48.0	-20.2	-1.690
20		870 TONS	-52.0	-----	-3.000
21	72"Ø	870 TONS	-49.0	-----	-5.000
22	36"Ø	350 TONS	-48.0	-----	VARIES *

* SEE INDIVIDUAL END BENT DRAWINGS

PREDICTED SCOUR ELEVATIONS APPLY TO BOTH THE 100 YEAR AND THE 500 YEAR EVENT.

HORIZONTAL CURVE DATA

CURVE NO. 2
℄ SURVEY

PI STA 151+35.02
Δ = 12°15'13.00"(LT)
D = 18°28'57.03"
T = 33.28'
L = 66.30'
R = 310.00'
PC STA 151+01.75
PT STA 151+68.05

CURVE NO. 4
℄ CONSTRUCTION

PI STA 40+25.32
Δ = 21°20'55.10"(LT)
D = 6°0'0.00"
T = 179.99'
L = 355.81'
R = 954.93'
PC STA 38+45.33
PT STA 42+01.14
e = 02 %

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	KBD/RLL	12-93
Checked by	DBT	12-93
Designed by	EWB	12-93
Checked by	KBD	12-93
Approved by	T. C. WHITE	

ENGINEER OF RECORD:
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3409 West Lemon Street, Suite 1
Tampa, Florida 33609

LOGO: DAVID VOLKERT & ASSOCIATES, INC.

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
	PINELLAS	15140-3518

SHEET TITLE: FOUNDATION LAYOUT, SHEET 2 OF 2

PROJECT NAME: CLEARWATER PASS BRIDGE

Drawing No.

Index No.

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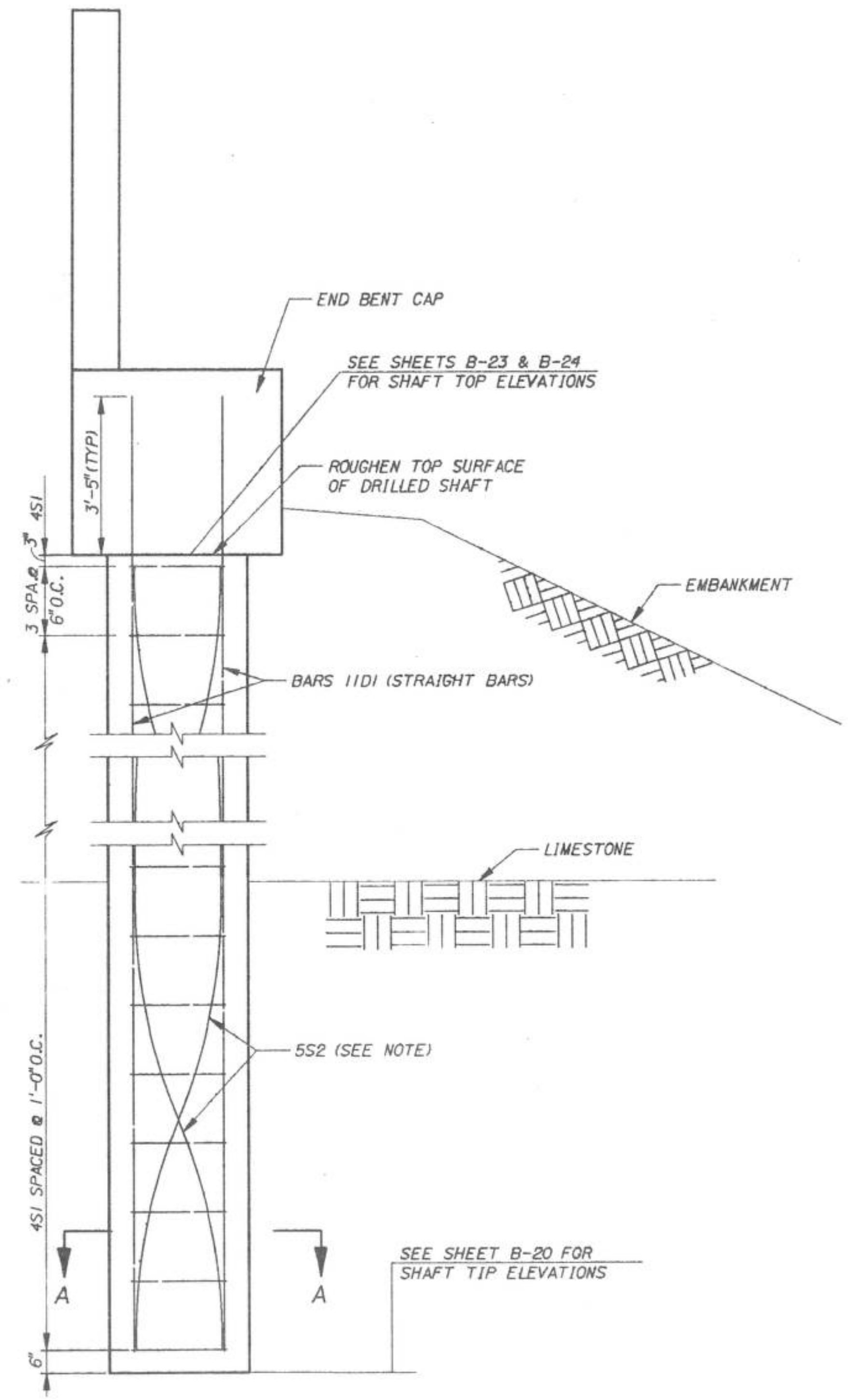
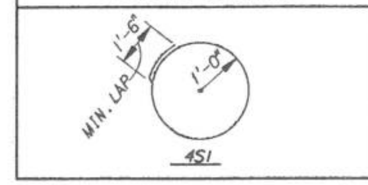
Fri Dec 10 05:55:10 1993

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA			B-21



ELEVATION AT END BENT

BENDING DIAGRAMS



NOTE:
BEND TWO 5S2 #5 BARS AT THE RATE OF ONE COMPLETE REVOLUTION PER 20' OF SHAFT. PLACE TWO SETS OF BARS INSIDE VERTICAL REINFORCING STARTING OPPOSITE EACH OTHER. EACH SET HAS TWO BARS LAP SPLICED 2'-0". TIE AT INTERSECTION WITH VERTICAL BARS.

BILL OF REINFORCEMENT

BENT	SHAFT	4S1 (SEE DIAGRAM) #4 BAR		5S2 (SEE DIAGRAM) #5 BAR		11D1 (STRAIGHT) #11 BAR	
		NO. REQ'D	LENGTH	NO. REQ'D	LENGTH	NO. REQ'D	LENGTH
1	1	60	7'-10"	2	58'-9"	8	60'-6"
	2	60	7'-10"	2	58'-8"	8	60'-5"
	3	60	7'-10"	2	59'-2"	8	60'-11"
	4	61	7'-10"	2	59'-8"	8	61'-4"
	5	61	7'-10"	2	59'-9"	8	61'-5"
22	1	61	7'-10"	2	59'-9"	8	61'-5"
	2	61	7'-10"	2	59'-10"	8	61'-6"
	3	61	7'-10"	2	60'-3"	8	61'-11"
	4	61	7'-10"	2	60'-6"	8	62'-2"
	5	62	7'-10"	2	60'-8"	8	62'-4"

NOTE: QUANTITIES ARE GIVEN PER BENT.

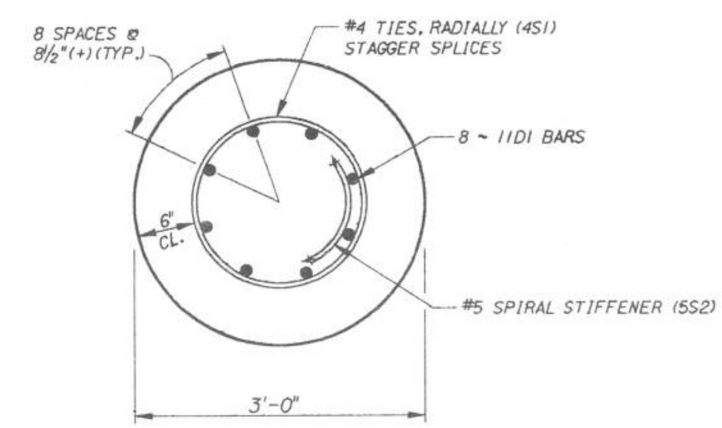
ESTIMATED QUANTITIES

ITEM	UNIT	QUANTITY
DRILLED SHAFT (36" DIAMETER)	LF	585
CORE	LF	595
UNCLASSIFIED SHAFT EXCAVATION (36" DIAMETER)	LF	535
UNCLASSIFIED SHAFT EXCAVATION (EXTRA DEPTH)	LF	20

36" Ø DRILLED SHAFT QUANTITIES PER LF

ITEM	UNIT	QUANTITY
CLASS IV CONCRETE (DRILLED SHAFT)	CY	0.3
* REINFORCING STEEL	LB	52

* INCLUDES ALLOWANCE FOR 11D1 BARS EXTENDING INTO CAP.



SECTION A-A

NOTES:

1. SLURRY TANKS ADEQUATE TO CONTAIN ALL FLUIDS SHALL BE USED IF SLURRY IS EMPLOYED IN THE DRILLING PROCESS.
2. DRILLED SHAFTS SHALL BE PAID FOR AT THE CONTRACT UNIT PRICE PER LINEAR FOOT FOR DRILLED SHAFTS (36" Ø) UNDER ITEM NO. 455-88-3. COMPENSATION FOR ADDITIONAL COST FOR MATERIALS AND CONSTRUCTION OF DRILLED SHAFTS SHALL BE PAID FOR UNDER APPROPRIATE ITEMS. SEE SUMMARY OF QUANTITIES AND SPECIFICATIONS.
3. CONCRETE FOR DRILLED SHAFTS SHALL BE CLASS IV (DRILLED SHAFT).
4. FOR ADDITIONAL NOTES, SEE SHEET B-20.

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	RLL	12-93
Checked by	KBD	12-93
Designed by	EWB	12-93
Checked by	KBD	12-93
Approved by	T. C. WHITE	

ENGINEER OF RECORD:
DAVID VOLKERT & ASSOCIATES, INC.
3409 West Lemon Street, Suite 1
Tampa, Florida 33609

LOGO: DAVID VOLKERT & ASSOCIATES, INC.

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
	PINELLAS	15140-3518

SHEET TITLE: 36 " Ø DRILLED SHAFTS

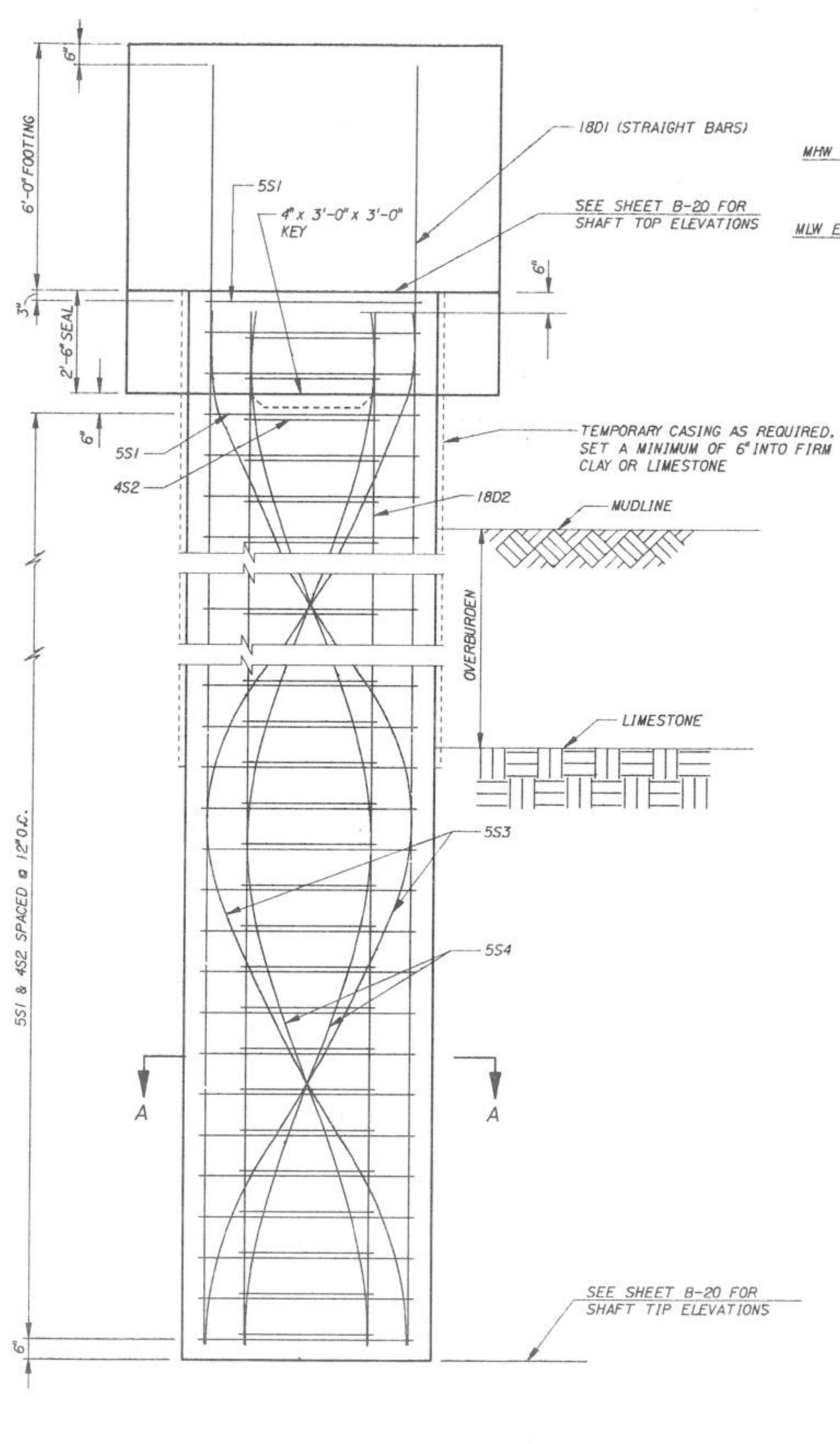
PROJECT NAME: CLEARWATER PASS BRIDGE

Drawing No.

Index No.

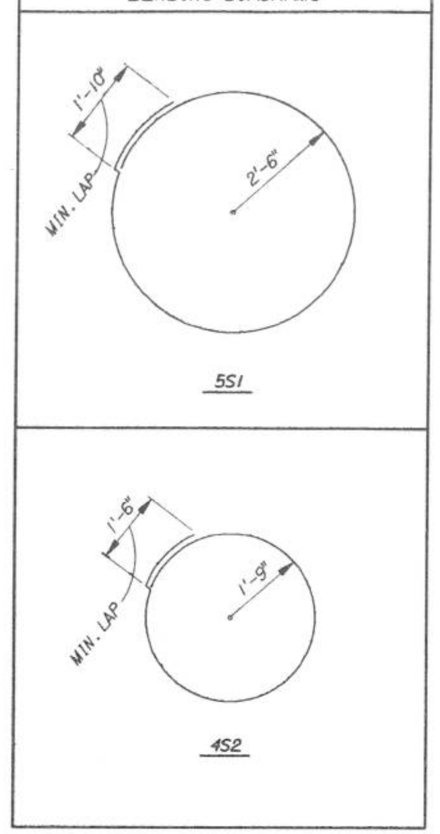
s:\cpb\std\shaft72.dgn

Fri Dec 10 06:01:45 1993

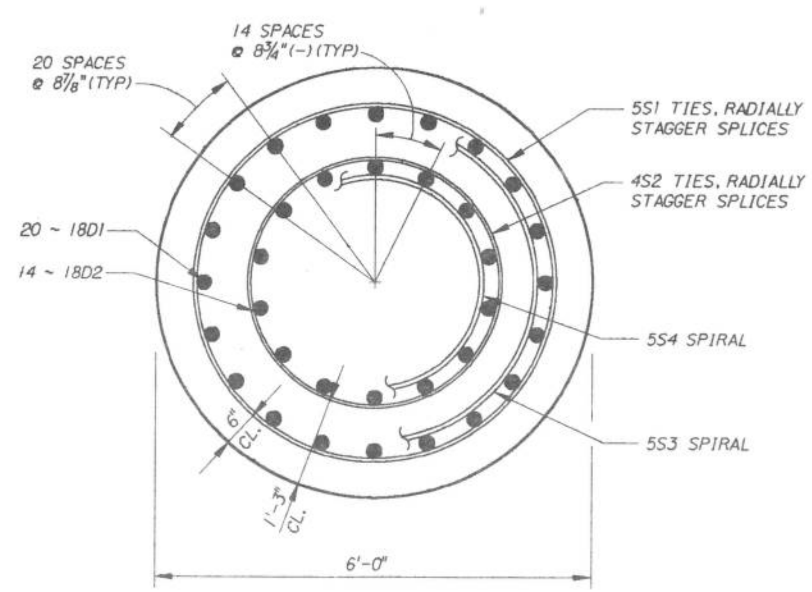


ELEVATION AT PIER

BENDING DIAGRAMS



NOTE:
BEND 5S3 & 5S4 BARS AT THE RATE OF ONE COMPLETE REVOLUTION PER 30' OF SHAFT. PLACE TWO 5S3 OR 5S4 BARS INSIDE VERTICAL REINFORCING STARTING OPPOSITE EACH OTHER. LAP SPLICE 2'-1" AS NECESSARY. TIE AT INTERSECTIONS WITH VERTICAL BARS. LENGTHS IN BILL OF REINFORCEMENT DO NOT INCLUDE LAP SPLICES OF 5S3 AND 5S4 BARS.



SECTION A-A

BILL OF REINFORCEMENT

PIER	5S1 (SEE DIAGRAM) #5 BAR		4S2 (SEE DIAGRAM) #4 BAR		5S3 (SEE NOTE) #5 BAR		5S4 (SEE NOTE) #5 BAR		18D1 (STRAIGHT) #18 BAR		18D2 (STRAIGHT) #18 BAR	
	NO. REQ'D	LENGTH	NO. REQ'D	LENGTH	NO. REQ'D	LENGTH	NO. REQ'D	LENGTH	NO. REQ'D	LENGTH	NO. REQ'D	LENGTH
2	102	17'- 7"	100	12'- 6"	4	54'- 9"	4	52'- 0"	40	55'- 6"	28	49'- 6"
3	98	17'- 7"	96	12'- 6"	4	52'- 6"	4	49'-10"	40	53'- 6"	28	47'- 6"
4	114	17'- 7"	112	12'- 6"	4	61'- 2"	4	58'- 1"	40	61'- 4"	28	55'- 4"
5	114	17'- 7"	112	12'- 6"	4	61'- 2"	4	58'- 1"	40	61'- 4"	28	55'- 4"
6	110	17'- 7"	108	12'- 6"	4	59'- 0"	4	56'- 0"	40	59'- 4"	28	53'- 4"
7	100	17'- 7"	98	12'- 6"	4	53'- 5"	4	50'- 9"	40	54'- 4"	28	48'- 4"
8	112	17'- 7"	110	12'- 6"	4	60'- 1"	4	57'- 1"	40	60'- 4"	28	54'- 4"
9	110	17'- 7"	108	12'- 6"	4	59'- 0"	4	56'- 0"	40	59'- 4"	28	53'- 4"
10	108	17'- 7"	106	12'- 6"	4	57'-10"	4	54'-11"	40	58'- 4"	28	52'- 4"
11	104	17'- 7"	102	12'- 6"	4	55'- 8"	4	52'-10"	40	56'- 4"	28	50'- 4"
12	104	17'- 7"	102	12'- 6"	4	55'- 8"	4	52'-10"	40	56'- 4"	28	50'- 4"
13	122	17'- 7"	120	12'- 6"	4	65'- 7"	4	62'- 3"	40	65'- 4"	28	59'- 4"
14	130	17'- 7"	128	12'- 6"	4	70'- 0"	4	66'- 6"	40	69'- 4"	28	63'- 4"
15	150	17'- 7"	148	12'- 6"	4	81'- 1"	4	77'- 0"	40	79'- 4"	28	73'- 4"
16	118	17'- 7"	116	12'- 6"	4	63'- 5"	4	60'- 2"	40	63'- 4"	28	57'- 4"
17	118	17'- 7"	116	12'- 6"	4	63'- 5"	4	60'- 2"	40	63'- 4"	28	57'- 4"
18	118	17'- 7"	116	12'- 6"	4	63-'5"	4	60'- 2"	40	63'- 4"	28	57'- 4"
19	94	17'- 7"	92	12'- 6"	4	50'- 2"	4	47'- 7"	40	51'- 4"	28	45'- 4"
20	100	17'- 7"	98	12'- 6"	4	53'- 1"	4	50'- 5"	40	54'- 0"	28	48'- 0"
21	90	17'- 7"	88	12'- 6"	4	47'- 7"	4	45'- 2"	40	49'- 0"	28	43'- 0"

NOTE: QUANTITIES ARE GIVEN PER PIER (2 SHAFTS).

ESTIMATED QUANTITIES

ITEM	UNIT	QUANTITY
DRILLED SHAFT (72" DIAMETER)	LF	2188
TEST HOLE (72" DIAMETER)	LF	47
CORE	LF	2274
UNCLASSIFIED SHAFT EXCAVATION (72" DIAMETER)	LF	1794
UNCLASSIFIED SHAFT EXCAVATION (EXTRA DEPTH)	LF	80

72" Ø DRILLED SHAFT QUANTITIES PER LF

ITEM	UNIT	QUANTITY
CLASS IV CONCRETE (DRILLED SHAFT)	CY	1.0
* REINFORCING STEEL	LB	515

* INCLUDES ALLOWANCE FOR 18D1 BARS EXTENDING INTO CAP.

NOTES:

1. SLURRY TANKS ADEQUATE TO CONTAIN ALL FLUIDS SHALL BE USED IF SLURRY IS EMPLOYED IN THE DRILLING PROCESS.
2. DRILLED SHAFTS SHALL BE PAID FOR AT THE CONTRACT UNIT PRICE PER LINEAR FOOT FOR DRILLED SHAFTS (72"Ø) UNDER ITEM NO. 455-88-7. COMPENSATION FOR ADDITIONAL COST FOR MATERIALS AND CONSTRUCTION OF DRILLED SHAFTS SHALL BE PAID FOR UNDER APPROPRIATE ITEMS. SEE SUMMARY OF QUANTITIES AND SPECIFICATIONS.
3. CONCRETE FOR DRILLED SHAFTS SHALL BE CLASS IV (DRILLED SHAFT).
4. FOR ADDITIONAL NOTES, SEE SHEET B-20.
5. IF OVERALL LENGTH OF 18D1 OR 18D2 BARS EXCEEDS 60', ONE MECHANICAL REINFORCING STEEL SPLICE PER BAR MAY BE USED INSTEAD OF A FULL LENGTH BAR. STAGGER SPLICES IN ADJACENT BARS IN THE SAME RING OF REINFORCING 8'-0" MINIMUM.
6. MECHANICAL REINFORCING STEEL SPLICES SHALL DEVELOP IN TENSION OR COMPRESSION AT LEAST 125 PERCENT OF THE YIELD STRENGTH OF THE BAR. SHOP DRAWINGS SHALL BE PROVIDED FOR THE MECHANICAL SPLICES, INCLUDING TEST REPORTS FROM A LABORATORY CERTIFIED BY THE FDOT. WHEN REQUESTED BY THE ENGINEER, UP TO TWO FIELD SPLICES OUT OF EACH 100, OR PORTION THEROF, TO BE PLACED IN THE WORK AND CHOSEN AT RANDOM BY THE ENGINEER, SHALL BE SUBMITTED BY THE CONTRACTOR TO THE ENGINEER TO BE TESTED TO 125 PERCENT OF THE YIELD STRENGTH OF THE BAR.

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	RLL	12-93
Checked by	KBD	12-93
Designed by	EWB	12-93
Checked by	KBD	12-93
Approved by	T. C. WHITE	

ENGINEER OF RECORD:
DAVID VOLKERT & ASSOCIATES, INC.
3409 West Lemon Street, Suite 1
Tampa, Florida 33609

LOGO: DAVID VOLKERT & ASSOCIATES, INC.

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
	PINELLAS	15140-3518

SHEET TITLE: 72" Ø DRILLED SHAFTS

Drawing No.

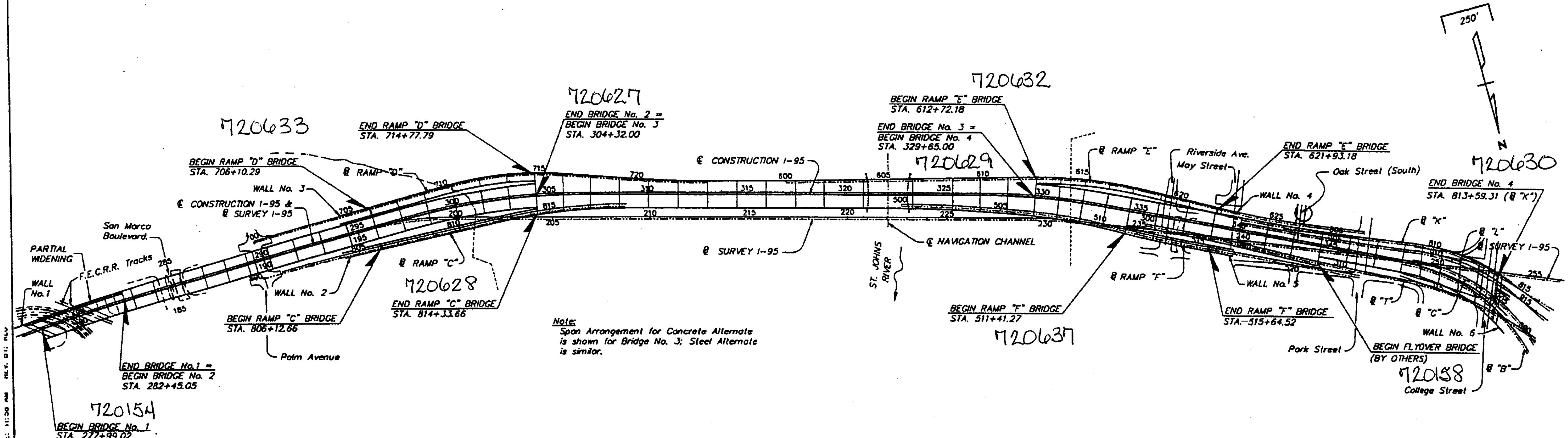
PROJECT NAME: CLEARWATER PASS BRIDGE

Index No.

3 FLA. 7

FULLER WARREN BRIDGE REPLACEMENT
STRUCTURAL PLANS

PROJECT NO. 72020-3485



THE FISCAL YEAR FOR THIS ENTIRE PROJECT HAS BEEN REVISED TO 1996.

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	RLG	10-93
Checked by	SLW	1-94
Designed by	RDS	10-93
Checked by	SLW	11-93
Approved by	D.W. Browning	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

SHEET TITLE: KEY MAP

PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

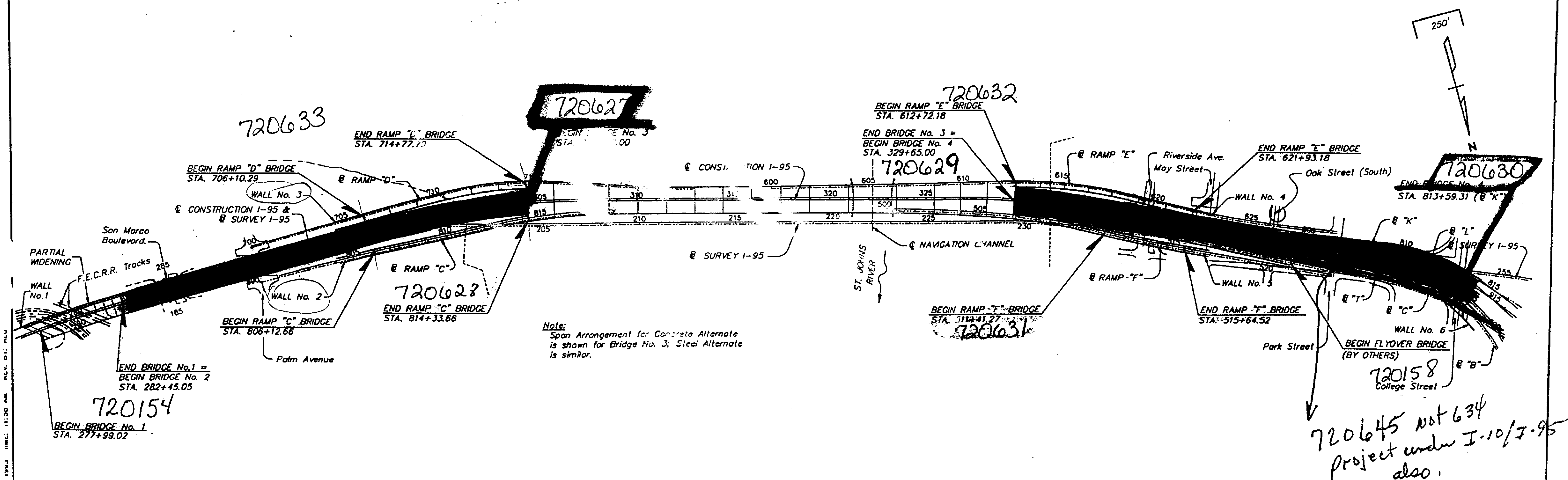
Drawing No.

Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	
3	FLA.	72020-3485	

FULLER WARREN BRIDGE REPLACEMENT
STRUCTURAL PLANS

PROJECT NO. 72020-3485



THE FISCAL YEAR FOR THIS ENTIRE PROJECT HAS BEEN REVISED TO 1996.

REVISIONS

	Names	Dates
Drawn by	RLG	10-93
Checked by	SLW	1-94
Designed by	RDS	10-93
Checked by	SLW	11-93
Approved by	D.W. Browning	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

SHEET TITLE: KEY MAP

PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

INDEX OF ROADWAY PLANS

SHEET NO.	SHEET DESCRIPTION
1	KEY SHEET
1A - 1H	SUMMARY OF PAY ITEMS
2 - 5	SUPPLEMENTAL DRAINAGE MAP
- 11	TYPICAL SECTIONS
- 13	SUMMARY OF QUANTITIES
- 19	SUMMARY OF DRAINAGE STRUCTURES
20 - 21	PROJECT LAYOUT/GENERAL NOTES
22	REFERENCE TIES
23 - 41	ROADWAY PLANS(1"=20')
42 - 69	ROADWAY PROFILES
70 - 71	GEOMETRIC LAYOUT
72 - 83	GEOMETRIC PLANS(1"=40')
84	CURVE & COORDINATE DATA
85 - 86	RAMP DETAILS
87 - 133	DRAINAGE STRUCTURES
134 - 135	POND DETAILS
136 - 137	DRAINAGE DETAILS
138 - 141	PAVEMENT JOINT DETAILS
142 - 143	CROSS SECTION PATTERN
144 - 155	SOIL BORING LOCATIONS
156 - 215	ROADWAY CROSS SECTIONS
216	TCP GENERAL NOTES
217 - 218	TCP PHASE NOTES
219	TCP QUANTITIES
220	MOT VARIABLE MESSAGES
221 - 313	TCP PLANS
314 - 349	DETOUR PLANS AND DETAILS
350 - 368	UTILITY ADJUSTMENT PLANS
369 - 387	CLEARING AND GRUBBING PLANS
388-406	STORMWATER POLLUTION PREVENTION PLANS
407-408	APPROACH SLABS
409-415	INTERIM STANDARDS

THESE PLANS HAVE BEEN PREPARED IN ACCORDANCE WITH AND ARE GOVERNED BY THE STATE OF FLORIDA, DEPARTMENT OF TRANSPORTATION, ROADWAY AND TRAFFIC DESIGN STANDARDS (BOOKLET DATED JANUARY, 1994).

REVISIONS

ROADWAY PLANS SHT. NOS. 1,13 & 382 (10/04/95)
SIGNING & MARKING PLANS SHT. NOS. S-1, S-22 & S-23 (10/04/95)
SIGNALIZATION PLANS SHT. NOS. T-1, T-2, T-4, T-7, T-11 & T-15 (10/04/95)
LIGHTING PLANS SHT. NOS. L-1, L-3 & L-22 (10/04/95)
GRAVITY SEWER & FORCE MAIN PLANS SHT. NOS. U1, U2 & U3 (10/04/95)
STRUCTURAL PLANS SHT. NOS. A8 & W7 (10/04/95)
ROADWAY PLANS SHT. NOS. 13, 216, 219, 315, 357 (11/09/95)
STRUCTURAL PLANS SHT. NOS. A47, A48 & A49 (11/09/95)
ADDED STATE PROJECT NO. 72020-6485 (11/09/95)
ROADWAY PLANS SHT. NOS. 1, (ADDED SHT 1A - 1H) (11/17/95)
ROADWAY PLANS SHT. NOS. 1, 1H, 357 (12/11/95)
CHANGED STATE PROJECT NO. 72020-6485 TO 72020-6526 (12/11/95)
ROADWAY PLANS SHT. NOS. 28 (2/5/96)
STRUCTURAL PLANS SHT. NOS. A4, A9, A10, A47, A49, C2, H1, W4, W11, W13 (2/5/96)
ROADWAY PLANS SHT. NOS. 1A, 1C, 1D, 1E, 1F, 216 (2/15/96)
ROADWAY PLANS SHT. NOS. 1A, 1B, 1D, 1E, 1F (2/20/96)
STRUCTURAL PLANS SHT. NOS. A47 (2/20/96)
ROADWAY PLANS SHT. NOS. 1, 34, 35, 112, 115, 118 (05/17/97)
STRUCTURAL PLANS SHT. NOS. A31A, D28, D29, D30, D31, D35, D36, D50, D51, F27, F28, F29, F36, F37, F38, F39, F40, F41, F44, F45, F65, K8 (05/17/97)
ADDED TEMPORARY SIGNALIZATION AND SIGNING PLANS (7/11/97)
REVISED TEMPORARY SIGNALIZATION AND SIGNING PLAN SHEETS 2-5, 8, 12, 17, 20-23, 27,-31, 34, AND 36 (8/27/97)
REVISED BEAM REINFORCING - SHEETS D81, D82, D83 AND D84 (2/6/98)
INCLUDED M2 BARS - SHEETS D81 AND D84 (3/20/98)
REVISED FENDER SYSTEM - SHEETS A17, A18, AND A19 (4/3/98)
REVISED BAR LISTS - SHEETS D20, D89-D96, F30 AND F108-F118 (4/28/98)
REVISED STRAND PATTERN TYPE - SHEET J24 (11/11/98)
REVISED SUPERSTURCTURE REINFORCING - SHEETS F70, F71, F72, F81 AND F123 (3/5/99)
REVISED SURVEILLANCE SYSTEM - SHEETS L1, L2, L3, L5-L13 -- ELIMINATED SHEETS P1-P11 (3/31/99)
STRUCTURAL PLANS SHEET NO. D85
19 STRUCTURAL PLAN SHEET NOS. C50, C51, C52, D14 & H20 (7/22/99)

1 ROADWAY PLANS
SIGNING AND MARKING PLANS
SIGNALIZATION PLANS
LIGHTING PLANS
INTERSTATE SURVEILLANCE AND CONTROL SYSTEM
2 STRUCTURAL PLANS
A DETAILED INDEX APPEARS ON THE KEY SHEET OF EACH COMPONENT SET OF PLANS

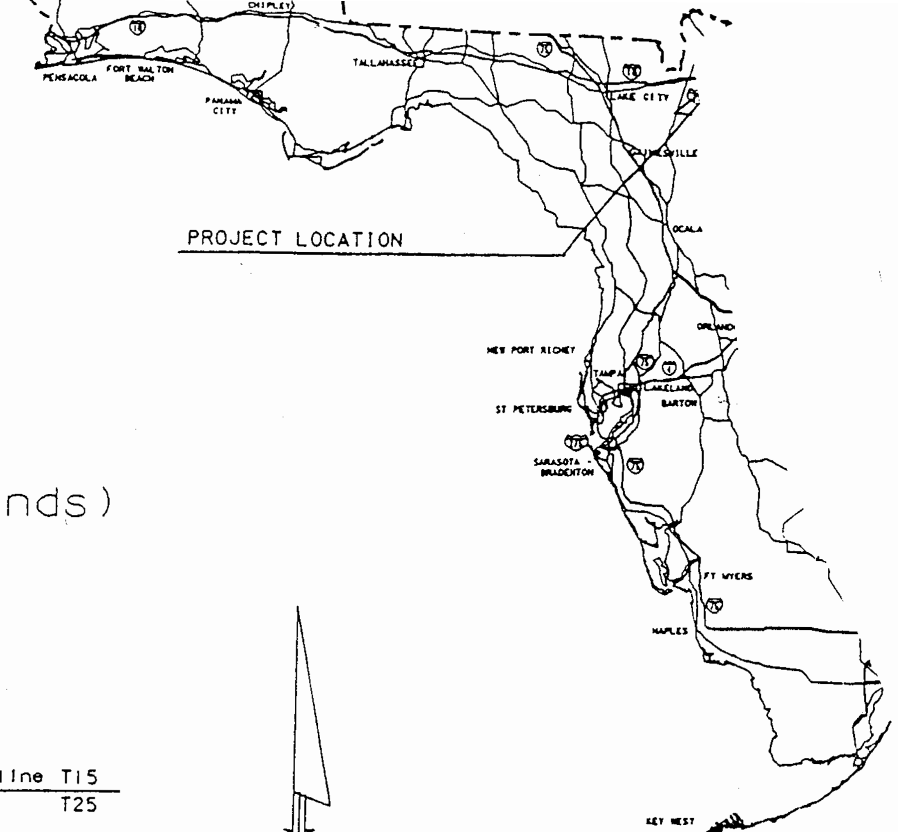
STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

PLANS OF PROPOSED

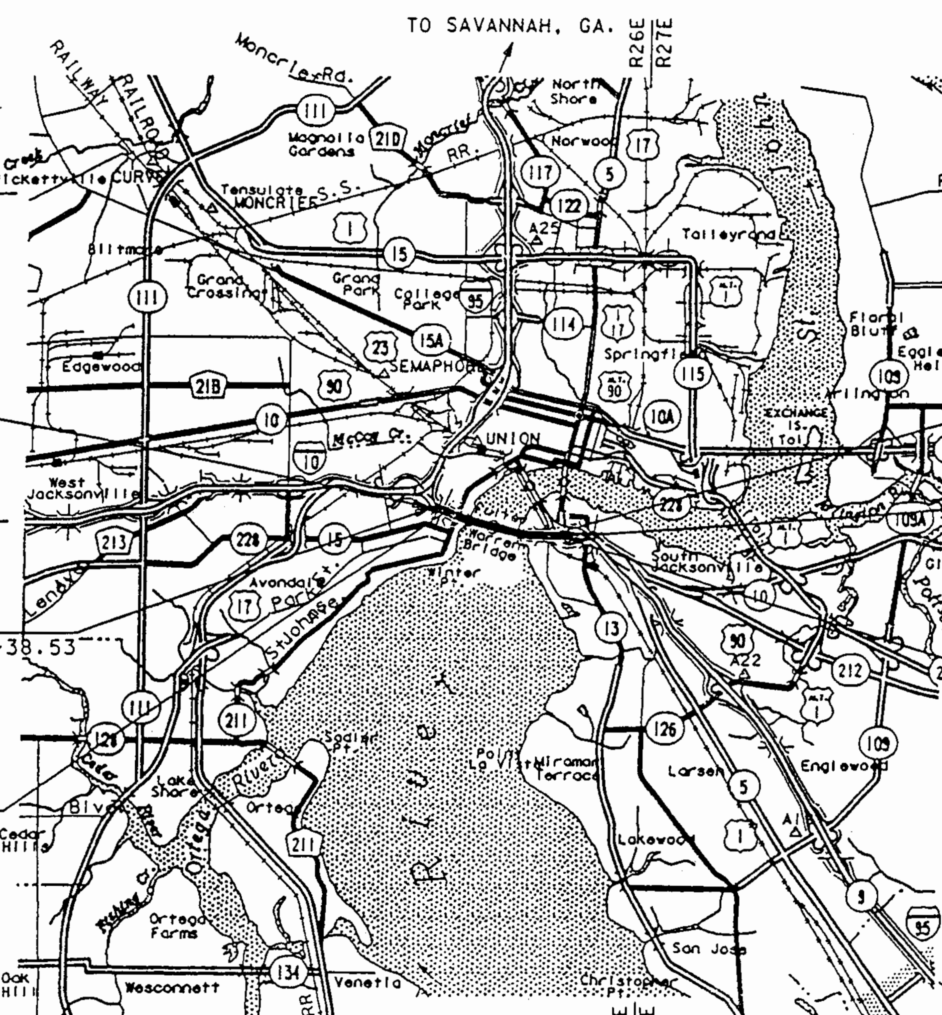
STATE HIGHWAY

FULLER WARREN BRIDGE

STATE PROJECT NO. 72020-3485 (Federal Funds)



PROJECT LOCATION



1 MILE

END PROJECT
℄ B STA. 105+38.53

BEGIN PROJECT
STA. 267+07

TO TALLAHASSEE

STA. 164+00.00, B SURVEY I-95 =
STA. 264+00.00, ℄ CONST. I-95

END BRIDGE
℄ B STA. 105+38.53

BEGIN BRIDGE
STA. 277+99.02

STA. 348+50 ℄ CONST. I-95 BK=
STA. 100+00 ℄ B AHD, 11' RT.

TO ST. AUGUSTINE

PROJECT LENGTH MEASURED ALONG ℄ CONST. I-95 AND ℄ B

ROADWAY PLANS ENGINEER OF RECORD
RODGER P. SCHMIDT, P.E.

DRAINAGE PLANS ENGINEER OF RECORD
LYNN A. KENDRICK, P.E.

PLANS PREPARED BY :
HNTB
5850 T.G. LEE BOULEVARD
SUITE 600
ORLANDO, FLORIDA 32822
(407) 859-8380

RECEIVED
JUL 28 1999
PBCS-FULLER W...

3 4 THIS PROJECT TO BE LET TO CONTRACT WITH STATE PROJECT NO.72020-6526 GRAVITY SEWER & FORCE MAIN RELOCATION PLANS

ATTENTION IS DIRECTED TO THE FACT THAT THESE PLANS MAY HAVE BEEN REDUCED IN SIZE BY REPRODUCTION. THIS MUST BE CONSIDERED WHEN OBTAINING SCALED DATA.

GOVERNING SPECIFICATIONS. STATE OF FLORIDA, DEPARTMENT OF TRANSPORTATION, STANDARD SPECIFICATIONS, DATED 1991 SUPPLEMENTS AND SPECIAL PROVISIONS THERETO IF NOTED IN THE CONTRACT SPECIFICATIONS FOR THIS PROJECT.

LENGTH OF PROJECT

	LINEAR FT.	MILES
ROADWAY	1092.02	0.207
BRIDGES	7589.51	1.437
* NET LENGTH OF PROJ.	8681.53	1.644
EXCEPTIONS		
GROSS LENGTH OF PROJ.	8681.53	1.644

REVISIONS		
DATE	BY	DESCRIPTION
10-04-95	LAK	1 REVISED NO. OF SHEETS
11-09-95	LAK	2 ADDED NOTE DELETED UTILITY PLANS, ADDED UTILITY PLANS TO 72020-6485
11-17-95	LAK	3 DELETED SUM. OF PAY ITEMS FROM CONTRACT PLANS ADD SUMMARY OF PAY ITEMS TO ROADWAY PLANS
12-11-95	LAK	4 REVISED PROJECT NO.

ROADWAY PLANS APPROVED BY: RODGER P. SCHMIDT, P.E.
DATE: 6/29/95
P.E. NO.: 40234

ROADWAY PLANS APPROVED BY: LYNN KENDRICK, P.E.
DATE: 9/22/95
P.E. NO.: 22919

FDOT PROJECT MANAGER : EUGENE HAGAN JR. P.E.

2132651520I

3 | FLA

A2

GENERAL SHEETS

A11

- A1 Key Map
- A2 Index of Sheets(1)
- A3 Index of Sheets(2)
- A4 General Notes (1)
- 5 General Notes (2)
- A6 Plan & Elevation Data (1)
- A7 Plan & Elevation Data (2)
- A8 Miscellaneous Details
- A9 Composite Neoprene Bearing Pads (1)
- A10 Composite Neoprene Bearing Pads (2)
- A11 Expansion Joints
- A12 Modular Expansion Joints (1)
- A13 Modular Expansion Joints Data
- A14 Light Pole Pilaster
- A15 Navigational Light System Details
- A16 ~~NOT USED~~
- A17 Plan of Fender System
- A17A Elevations of Fender System
- A18 Connection Details for Fender System - 1
- A18A Connection Details for Fender System - 2
- A19 Connection Details for Fender System - 3
- A20 ~~Traffic Railing Barrier~~
- A21 Standard Bar Bending Details
- A22 Prestressed Concrete Piles (1)
- A23 Prestressed Concrete Piles (2)
- A24 Drilled Shaft Details (1)
- A25 Drilled Shaft Details (2)
- A26 Jacking Details (1)
- A27 Jacking Details (2)
- A28 Bridge Drain Layout
- A29 Bridge Drainage Details (1)
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- A31 Bridge Drainage Details (3)
- A31A Scupper Schedule
- A32 Bridge Hydraulic Recommendation Sheet
- A33 Legend for Generalized Subsurface Profiles
- A34 Report of Core Borings (1)
- A35 Report of Core Borings (2)
- A36 Report of Core Borings (3)
- A37 Report of Core Borings (4)
- A38 Report of Core Borings (5)
- A39 Report of Core Borings (6)
- A40 Report of Core Borings (7)
- A41 Cantilever Retaining Wall (Index 800)
- A42 Case III 6 Ft. To 15 Ft. Height (Index 810)
- A43 Attenuator Survey (1)
- A44 Attenuator Survey (2)
- A45 Attenuator Survey (3)
- A46 Attenuator Survey (4)
- A47 Foundation Load Test Program Requirements (1)
- A48 Foundation Load Test Program Requirements (2)
- A49 Foundation Load Test Program Requirements (3)

720629

26

BRIDGE NO. 1 - PARTIAL WIDENING

720154

- B1 Plan and Elevation
- B2 Foundation Layout
- B3 End Bent No. 2E
- B4 End Bent Details
- B5 Pier Nos. 8E,9E,10E or 11E
- B6 Pier No. 12E
- B7 Pier No. 13E and 14E
- B8 Miscellaneous Pier Details
- B9 Construction Data
- B10 Superstructure Plan (1)
- B11 Superstructure Plan (2)
- B12 Superstructure Section
- B13 Superstructure Details (1)
- B14 Superstructure Details (2)
- B15 Framing Plan
- B16 AASHTO Type II Beams
- B17 Typical Notes & Details for AASHTO Type II Prestressed Beams
- B18 Girder Details (1)
- B19 Girder Details (2)
- B20 Rolled Beam Details
- B21 Camber Diagram
- B22 Reinforcing Bar List (1)
- B23 Reinforcing Bar List (2)

BRIDGE NO. 2

720627

- C1 Plan and Elevation (1)
- C2 Plan and Elevation (2)
- C3 Foundation Layout (1)
- C4 Foundation Layout (2)
- C5 Pier Cap Plan, Piers 2 & 3
- C6 Pier Cap Plan, Piers 4-15
- C7 Pier Cap Plan, Piers 16 & 17
- C8 Pier Cap Details, Pier 2
- C9 Pier Cap Sections, Pier No. 2
- C10 Pier Cap Details, Piers 3-15
- C11 Center Pier Cap Details, Piers 6,7 & 8
- C12 Pier Cap Details, Piers 16 & 17
- C13 Pedestal Details
- C14 Pier Column Details
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- C16 Pier Elevations, Pier 2
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- C18 Pier Elevations, Piers 16 & 17
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- C21 Construction Data (3)
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- C23 Construction Data (5)
- C24 Construction Data (6)
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- C26 Framing Plan (2)
- C27 Framing Plan (3)
- C28 Framing Plan (4)
- C29 Framing Plan (5)
- C30 Framing Plan (6)
- C31 Framing Plan (7)
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- C34 Superstructure Plan (3)
- C35 Superstructure Plan (4)
- C36 Superstructure Plan (5)
- C37 Superstructure Plan (6)
- C38 Superstructure Plan (7)
- C39 Superstructure Plan (8)
- C40 Superstructure Section (1)
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REVISIONS

Date	By	Description	Date	By	Description
5/8/02	JAL	26 Revised Titles and added Sheets A17A & A18A			

	Names	Dates
Drawn by	DCB	10-93
Checked by	SLW	1-94
Designed by	RDS	10-93
Checked by	SLW	1-94
Approved by	D. W. Browning	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

SHEET TITLE: INDEX OF SHEETS (1)

Drawing No.

PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

Index No.

NAME: G:\JOBS\1841\...\CHPROD\FENDER REVISION\BRDO1B.DWG DATE: MAY 8, 2002 TIME: 4:00 PM REV. BY: JAL

DIV. NO.			YEAR	NO.
3	FLA.		95	A3

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720631

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7206~~34~~ 45

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I-95 S.B. DETOUR OVER COLLEGE STREET

720158
JTA Bridge
Widened
temp.-then
to be
deleted

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EXISTING BRIDGE PLANS

EB1 thru EB39

REVISIONS					
Date	By	Description	Date	By	Description
5/15/00	MDC	20 INDEX REVISED			

	Names	Dates
Drawn by	SCG	1-95
Checked by	SLW	1-94
Designed by	RDS	10-93
Checked by	SLW	1-94
Approved by	D. W. Browning	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL
M. Cauthen
5/15/00

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

SHEET TITLE: INDEX OF SHEETS (2)
PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER
Drawing No.
Index No.

NAME: M:\1641\...20\BRD01C.DWG DATE: MAY 03, 2000 TIME: 9:49 AM REV. BY: RLG

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.		95	A24

BBCI:

1. Construct end bent drill shafts in two stages, first stage construct drill shafts to natural ground using approved construction methods, second stage extend to grade rebar cages using mechanical splices, form and four drill shafts to grade.

2. Cut drill shafts vertical rebar approximately 6 each (3 each on opposite sides) for the removal of zipper casing (when the casing is below the top of the rebar cage). Extend rebar cage to grade with mechanical splices after casing removal.

HNTB -

1) We have no objection to BBCI constructing the end bent drill shafts in two stages as described in their 1/31/97 letter to you. We suggest that design data from the manufacturer for the mechanical splice be provided to PBCS for review.

2) Cutting vertical bars in the drilled shafts in order to remove the zipper casing is undesirable, but we have no objection to doing it provided the cut reinforcing is mechanically spliced and extended to the planned length after the casing is removed.

Miscellaneous Changes

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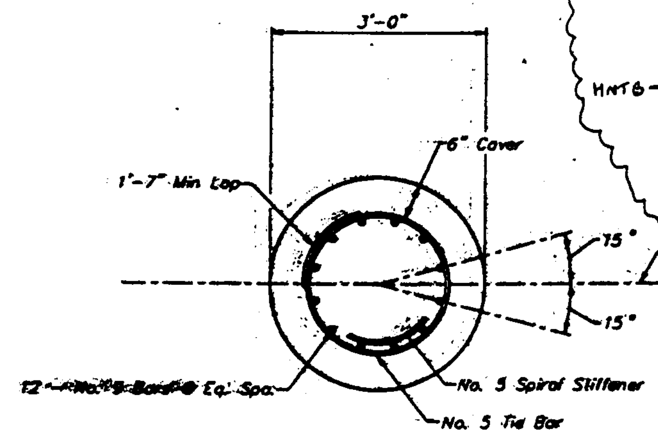
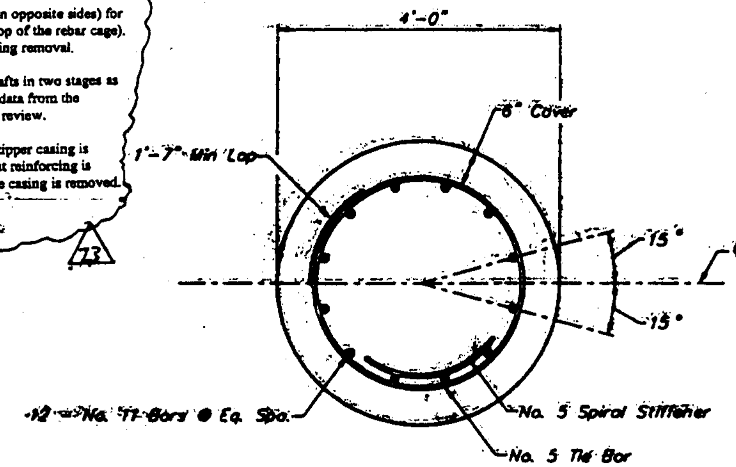


SECTION A-A



SECTION B-B

NOTES

1. Reinforcing Steel shall be Grade 60.
2. Concrete for Drilled Shafts shall be Class IV (Drilled Shaft), f'c = 4,000 p.s.i.
3. Drilled Shafts shall be paid for at the Contract Unit Price per linear foot for Drilled Shafts under Item No. 455-88-3 (36"⌀), 455-88-5 (48"⌀), or 455-88-7 (72"⌀). See Summary of Quantities and Specifications.
4. The Contractor shall submit plans, showing method of Constructing the Drilled Shaft Foundations, to the Engineer for Approval.
5. For Shaft Elevations, see Drilled Shaft Installation Data on the Foundation Layout Sheet.
6. Bar Extensions for Embedment into Footing are to be included in the cost of Drilled Shaft Item, but are not included in Quantities Per Lin. Ft. shown below.

36"⌀ DRILLED SHAFT QUANTITIES PER LIN. FT.

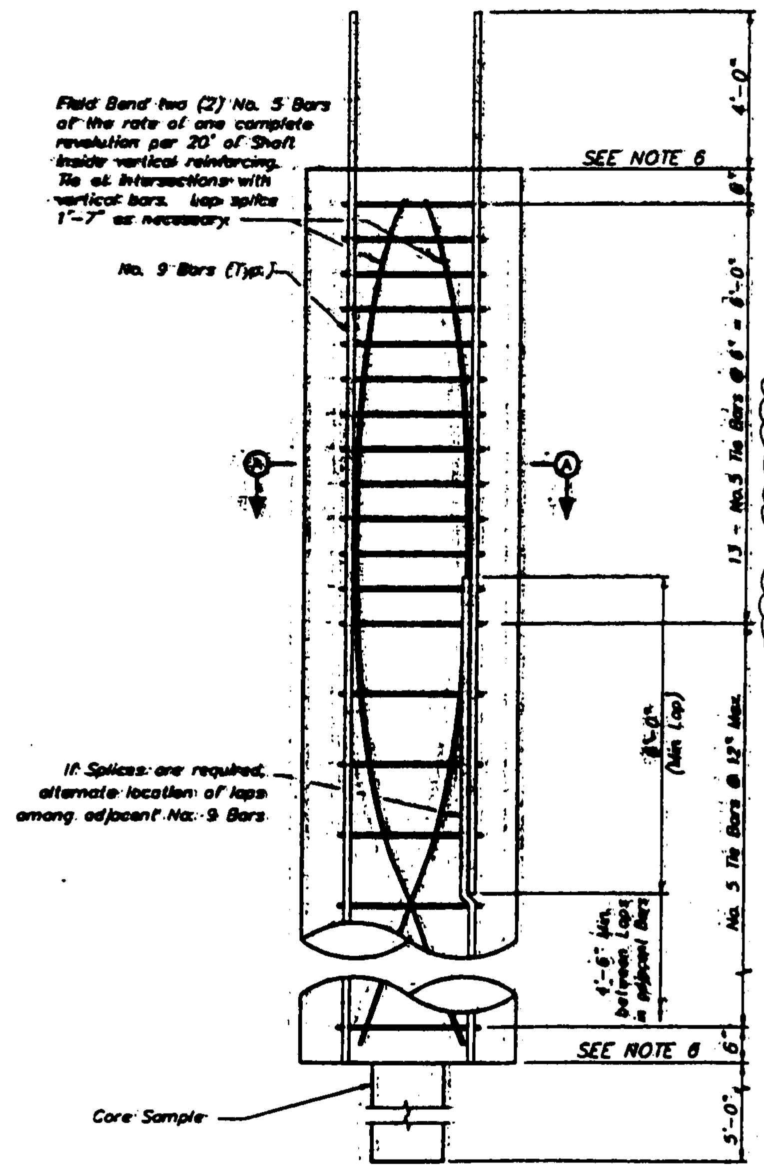
Class IV Concrete = 0.26 Cu Yds./Lin. Ft.
Reinforcing Steel = 58.2 Lbs./Lin. Ft. for top 7 ft.
51.2 Lbs./Lin. Ft. for Remainder of shaft

48"⌀ DRILLED SHAFT QUANTITIES PER LIN. FT.

Class IV Concrete = 0.47 Cu.Yds./Lin. Ft.
Reinforcing Steel = 87.2 Lbs./Lin. Ft. for top 7 ft.
77.4 Lbs./Lin. Ft. for remainder of shaft

72"⌀ DRILLED SHAFT QUANTITIES PER LIN. FT.

Class IV Concrete = 1.05 Cu.Yds./Lin. Ft.
Reinforcing Steel (Type A) = 184.4 Lbs./Lin. Ft. for top 7 ft.
169.0 Lbs./Lin. Ft. for remainder of shaft
Reinforcing Steel (Type B) = 311.4 Lbs./Lin. Ft. for top 7 ft.
296.0 Lbs./Lin. Ft. for remainder of shaft



ELEVATION - 36" DIAMETER DRILLED SHAFT

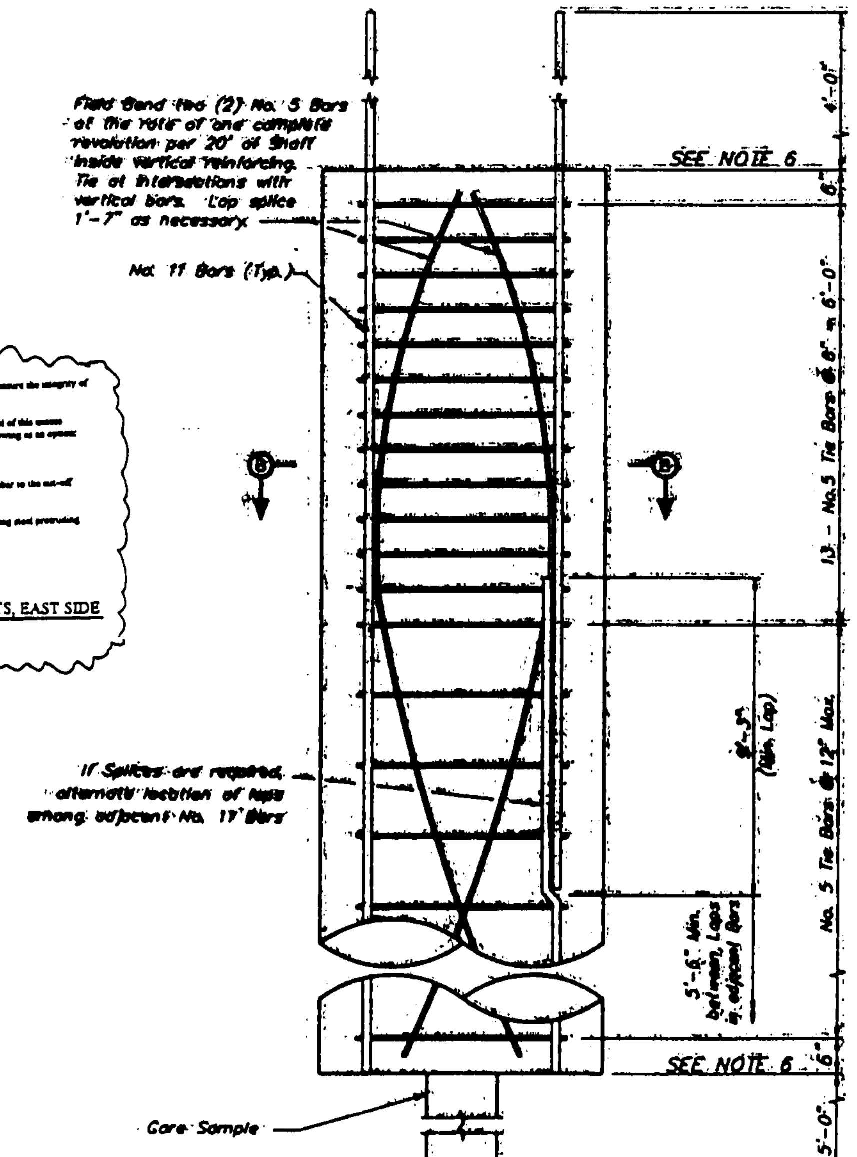
The 36 inch drilled shafts on the east side of the river were poured high to ensure the integrity of the concrete at the cut-off elevation of the shaft.

In an attempt to minimize potential damage to the shafts during the removal of this excess concrete, Balfour Beatty Construction, Inc. would like to propose the following as an option:

1. Cut the shaft off 1.0 feet above the cut-off elevation.
2. Remove and clean out the remaining concrete around the rebar to the cut-off elevation.
3. Mechanically splice a 3.0 feet piece of rebar to the reinforcing steel protruding from the drilled shaft.
4. Form the footing as originally planned.

REINFORCING STEEL - 36" DRILLED SHAFTS, EAST SIDE

23



ELEVATION - 48" DIAMETER DRILLED SHAFT

To repair small underwater concrete spalls on the 72 inch drilled shafts in the river Balfour Beatty Construction, Inc. proposes the following procedure:

1. Throughly clean the affected area with high pressure air or water.
2. Chip and remove any loose material to achieve a clean rough surface.
3. Place either the product "Plug-Crete" or "Dam-It" into the afftected area (literature and product data attached). The material has a fast set time and will be hand placed by a diver.
4. These products require no special curing procedures.

23

The 36 and 48 inch diameter drilled shafts on land were poured to a height above the cut off elevation to insure the integrity of the shaft concrete.

Balfour Beatty Construction, Inc. proposes to use the same method that is currently being utilized on the 36 inch drilled shafts located on bridge #2 for the remaining 36 inch and 48 inch drilled shafts located on land. This method is as follows:

1. Cut the shaft off one foot above the cut-off elevation.
2. Remove and clean out the remaining concrete around the reinforcing steel to the cut-off elevation.
3. Mechanically splice a three foot piece of rebar to the top of the reinforcing steel that extends out of the drilled shaft using an approved mechanical coupling device.
4. Form and pour the footing as originally planned.

REINFORCING STEEL - LAND DRILLED SHAFTS

23

NAME: M:\16414\GENERAL\BRD298.DWG DATE: AUG 31, 1995 TIME: 6:58 AM REV. BY: RLG

REVISIONS

Date	By	Description	Date	By	Description
7-15-01	SH	23 AS-BUILT			

	Names	Dates
Drawn by	LJS	1-94
Checked by	DMW	5-95
Designed by	HDR	1-94
Checked by	DMM	2-94
Approved by	D. W. Browning	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

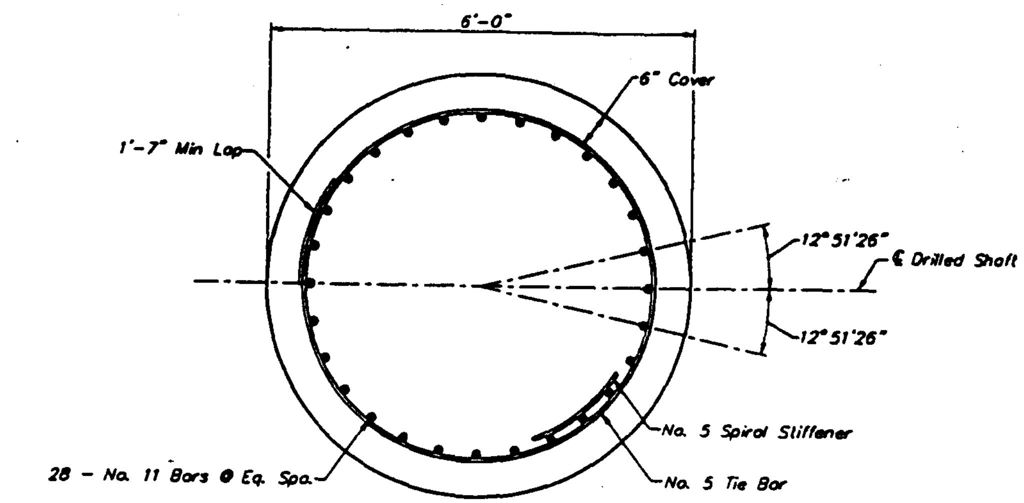
SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

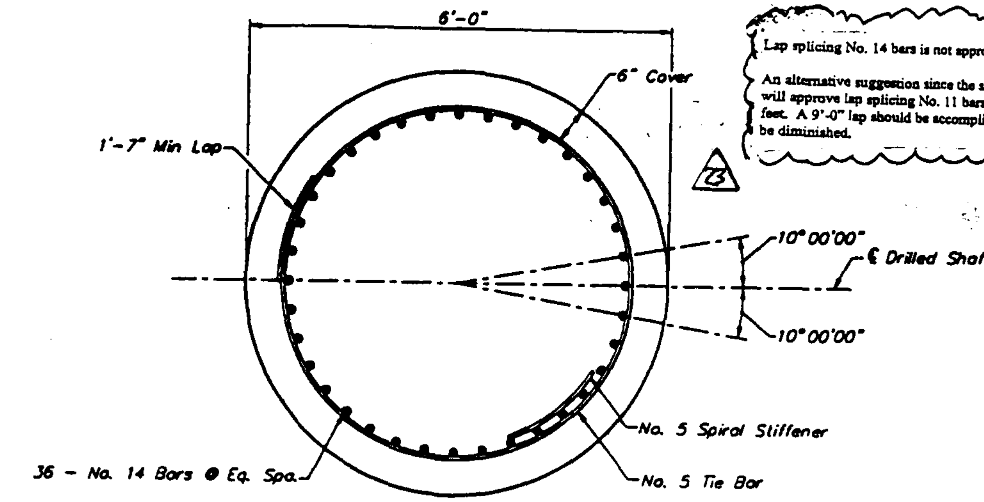
ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

SHEET TITLE: DRILLED SHAFT DETAILS (1)

PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER



SECTION C-C

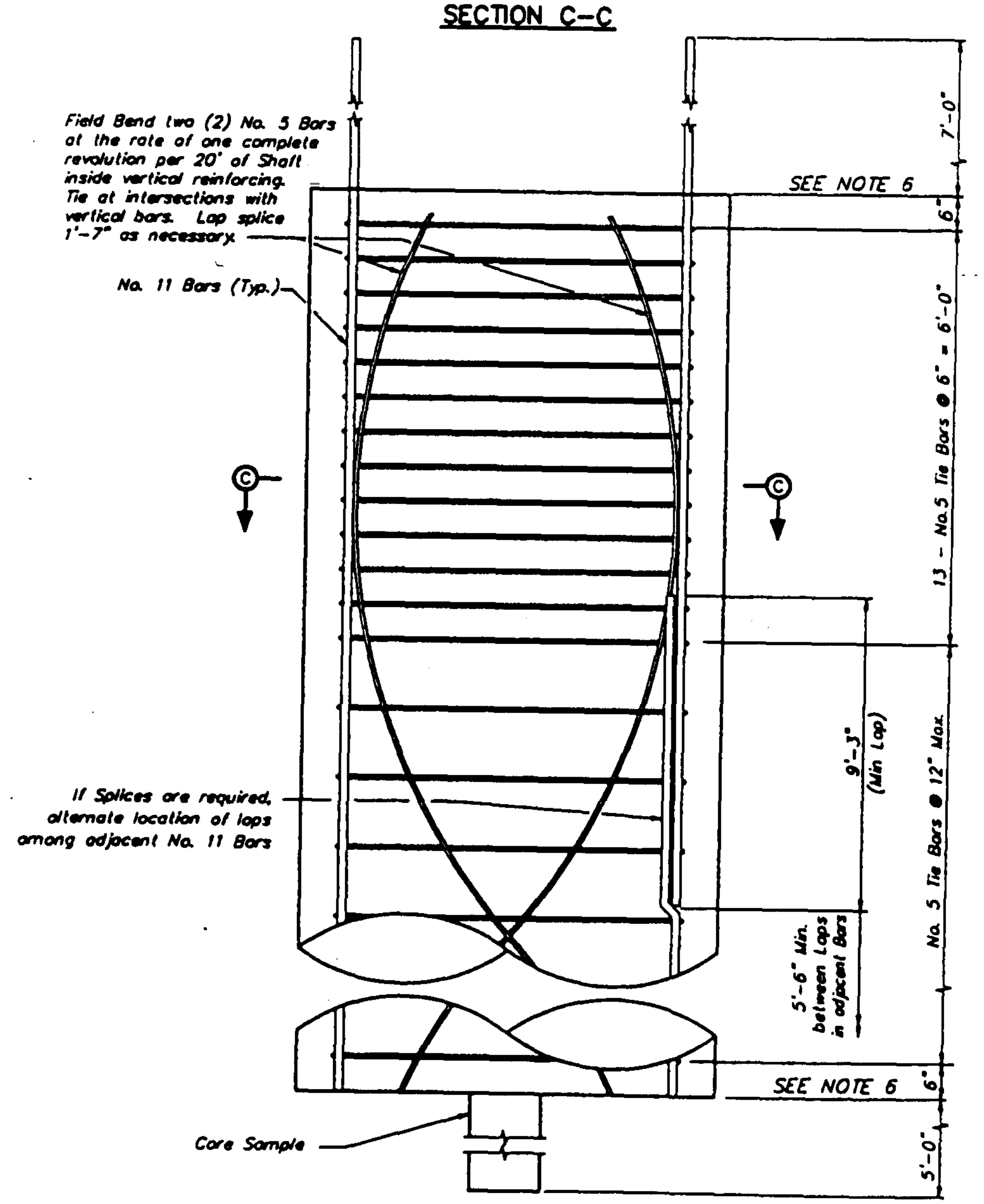


SECTION D-D

Lap splicing No. 14 bars is not approved.

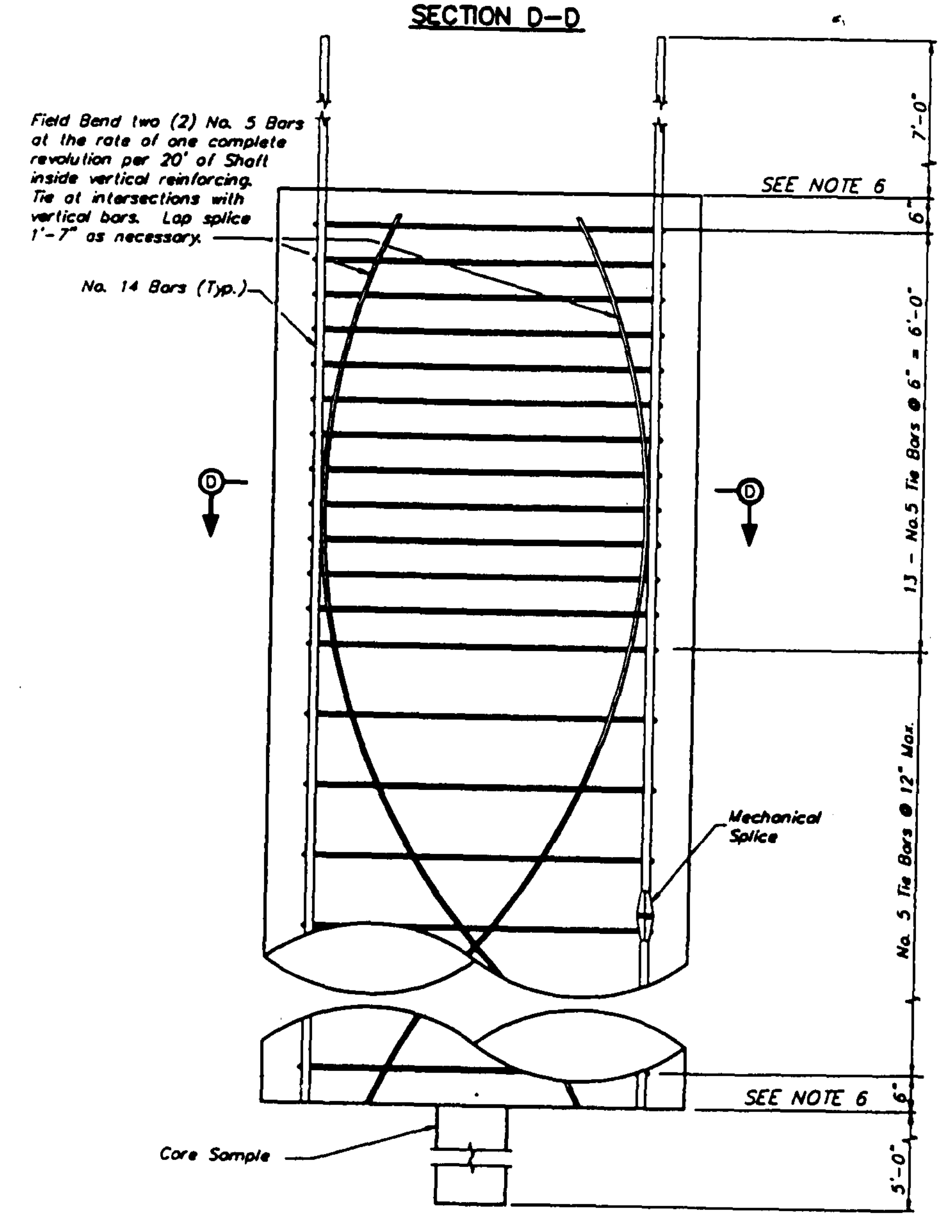
An alternative suggestion since the shaft is only six feet longer than it's original length we will approve lap splicing No. 11 bars to the bottom of the cage to extend it the additional six feet. A 9'-0" lap should be accomplished to the inside so clear distance between bars will not be diminished.

23



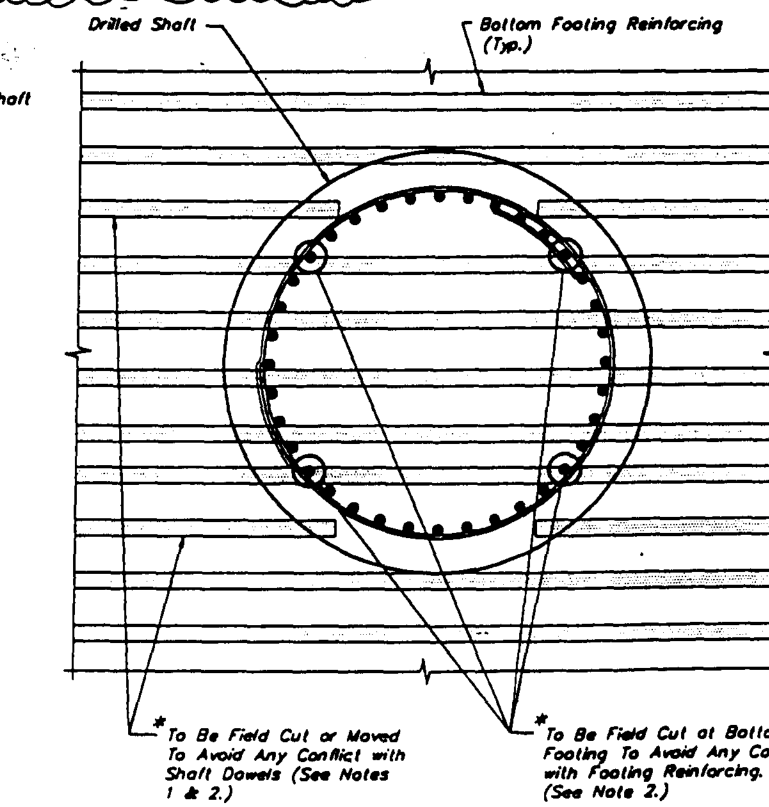
ELEVATION - 72" DIAMETER DRILLED SHAFT

Type A



ELEVATION - 72" DIAMETER DRILLED SHAFT

Type B



PLAN

(TYPICAL SHAFT/FOOTING REINFORCING INTERACTION)

Below procedure Note 3 used at following locations
Pier 28 - Shaft 9
Pier 30 Shaft 1, 2, 16

23

NOTES

(1.) The Contractor Shall Adjust Footing Reinforcement where possible to avoid conflicts with the Drilled Shaft Reinforcing. Minimum Bar Spacing shall be three (3) times Bar Diameter and Shall Not Exceed Reinforcing Spacing Shown on Pier Footing Reinforcing by 1".

(2.) The Contractor Shall Be Permitted to Field Cut Footing or Drilled Shaft Bars only as indicated on This Sheet. No Adjacent Footing Bars or Drilled Shaft Bars may be cut. No more than four (4) Drilled Shaft Bars may be cut in any given shaft.

* To Be Field Cut and only if there is an Unavoidable Conflict Between Shaft and Footing Reinforcing and Field Adjusment Is Not An Option.

For Additional Notes see Sheet A24.

(3.) Drilled shaft Repair Procedure - Approved March 23, 1998
Bridge No. 3
(1) Several of the drilled shafts appear to have irregular surfaces
(a) Install the shaft cut-off box over the shaft to be repaired.
(b) Seal and dewater the box to an elevation lower than affected area
(c) Chip and clean the affected area to obtain flat even surface
(d) Place "stay form" expanded metal around shaft to repair.
(e) Place class IV drilled shaft concrete - dewater for 24 hours
(f) Remove the box and continue soffit slab [illegible]

23

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.		95	A25

REVISIONS

Date	By	Description	Date	By	Description
7-15-01	SH	23 AS-BUILT			

	Names	Dates
Drawn by	LJS	1-94
Checked by	DMW	5-95
Designed by	HDR	1-94
Checked by	DMM	2-94
Approved by	D. W. Browning	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

SHEET TITLE: DRILLED SHAFT DETAILS (2)

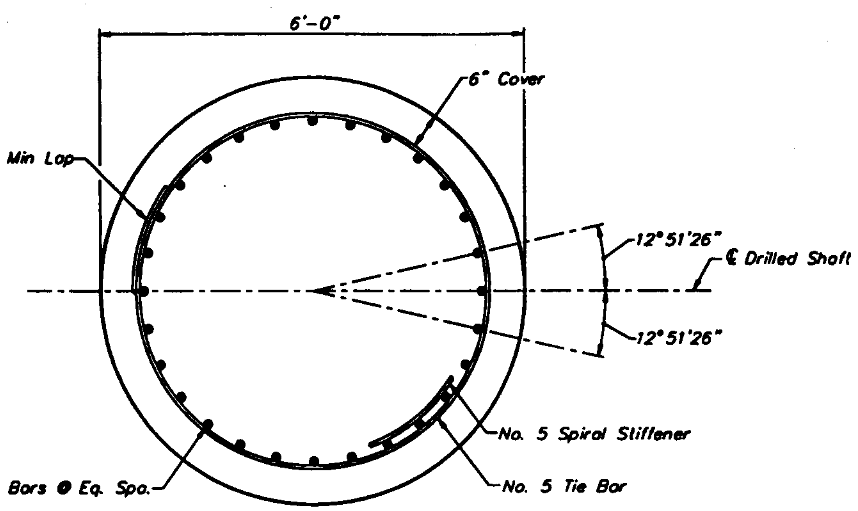
PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

Drawing No.

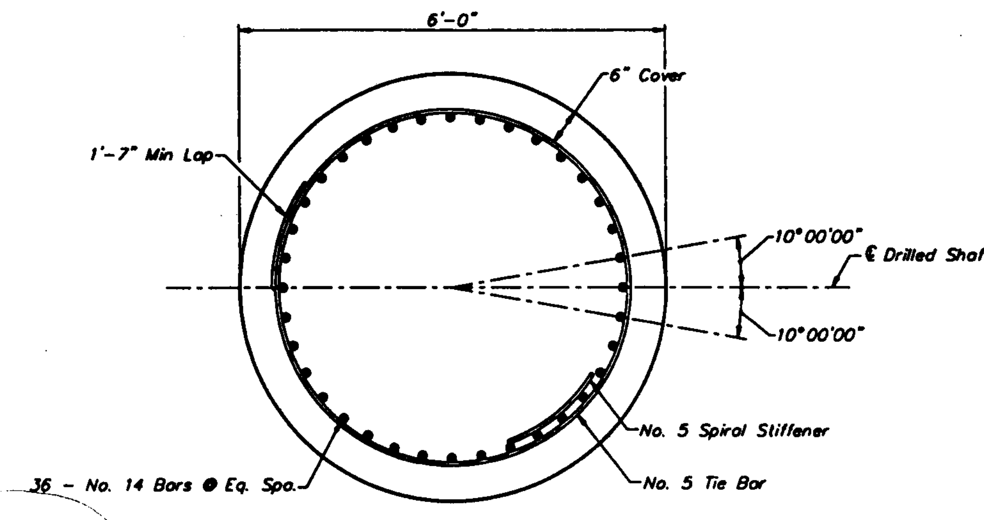
Index No.

NAME: M:\18416\GENERAL\0029C.DWG DATE: AUG 31, 1995 TIME: 7:30 AM REV. BY: RLG

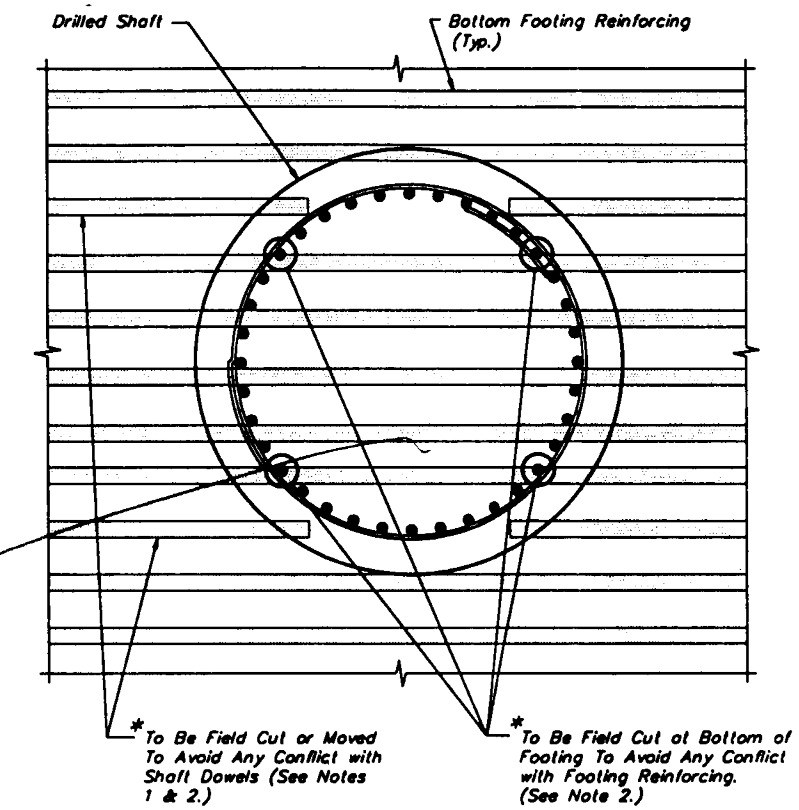
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.		95	A25



SECTION C-C

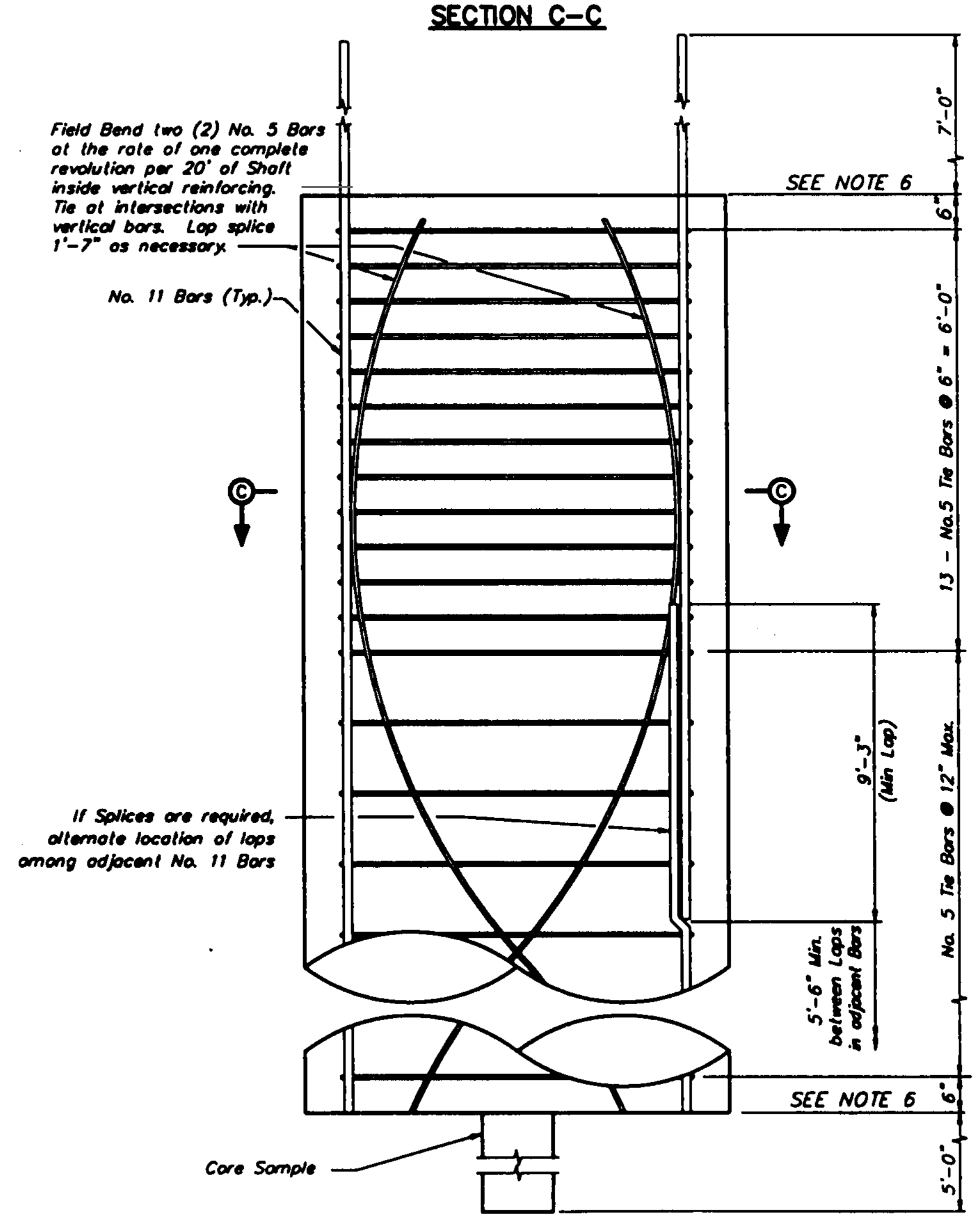


SECTION D-D



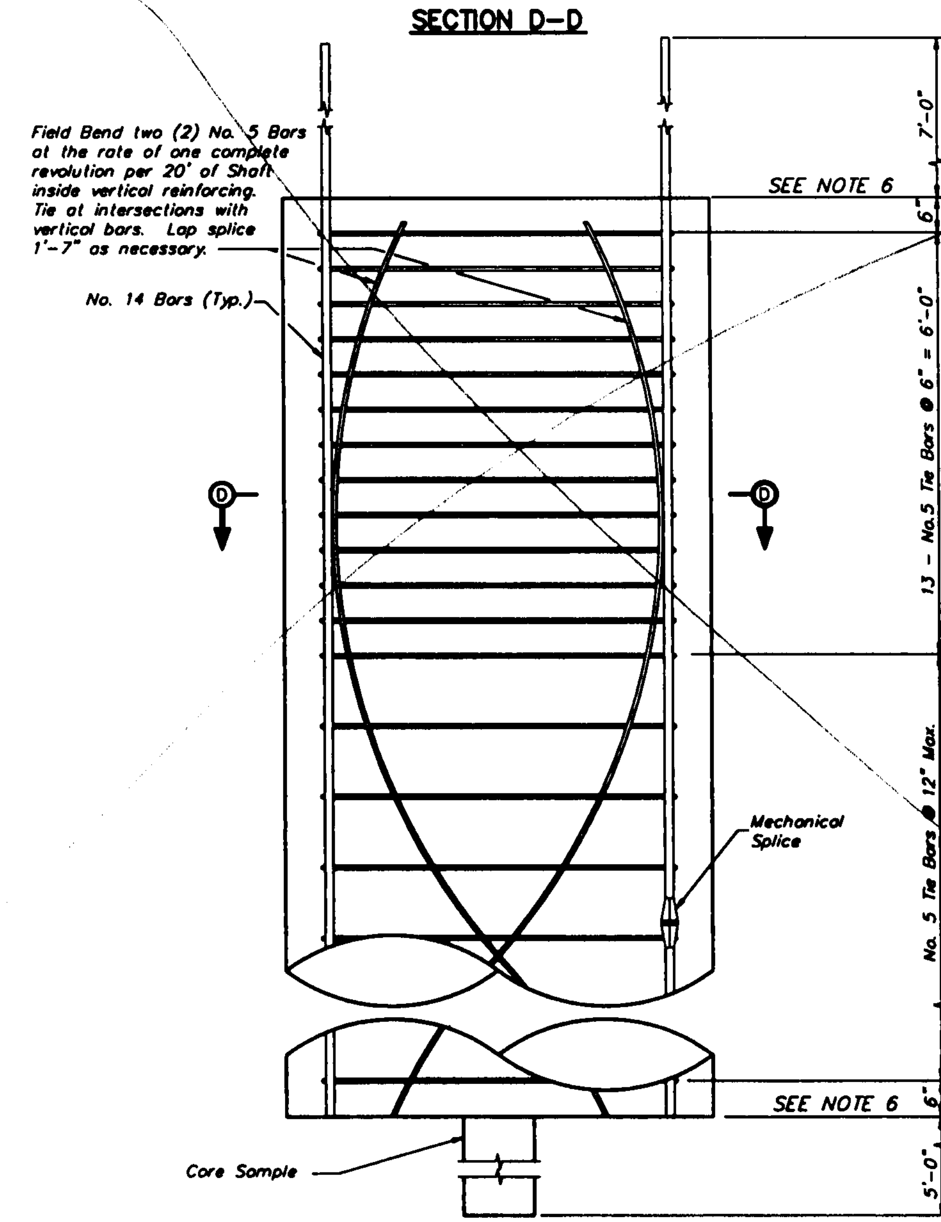
PLAN

(TYPICAL SHAFT/FOOTING REINFORCING INTERACTION)



ELEVATION - 72" DIAMETER DRILLED SHAFT

Type A



ELEVATION - 72" DIAMETER DRILLED SHAFT

Type B

NOTES

(1.) The Contractor Shall Adjust Footing Reinforcement where possible to avoid conflicts with the Drilled Shaft Reinforcing. Minimum Bar Spacing shall be three (3) times Bar Diameter and Shall Not Exceed Reinforcing Spacing Shown on Pier Footing Reinforcing by 1".

(2.) The Contractor Shall Be Permitted to Field Cut Footing or Drilled Shaft Bars only as indicated on This Sheet. No Adjacent Footing Bars or Drilled Shaft Bars may be cut. No more than four (4) Drilled Shaft Bars may be cut in any given shaft.

* To Be Field Cut and only if there is an Unavoidable Conflict Between Shaft and Footing Reinforcing and Field Adjustment Is Not An Option.

For Additional Notes see Sheet A24.

NAME: ...BRD29C.DWG DATE: AUG 31, 1995 TIME: 7:50 AM REV. BY: RLG

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	LJS	1-94
Checked by	DMW	5-95
Designed by	HDR	1-94
Checked by	DMM	2-94
Approved by	D. W. Browning	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

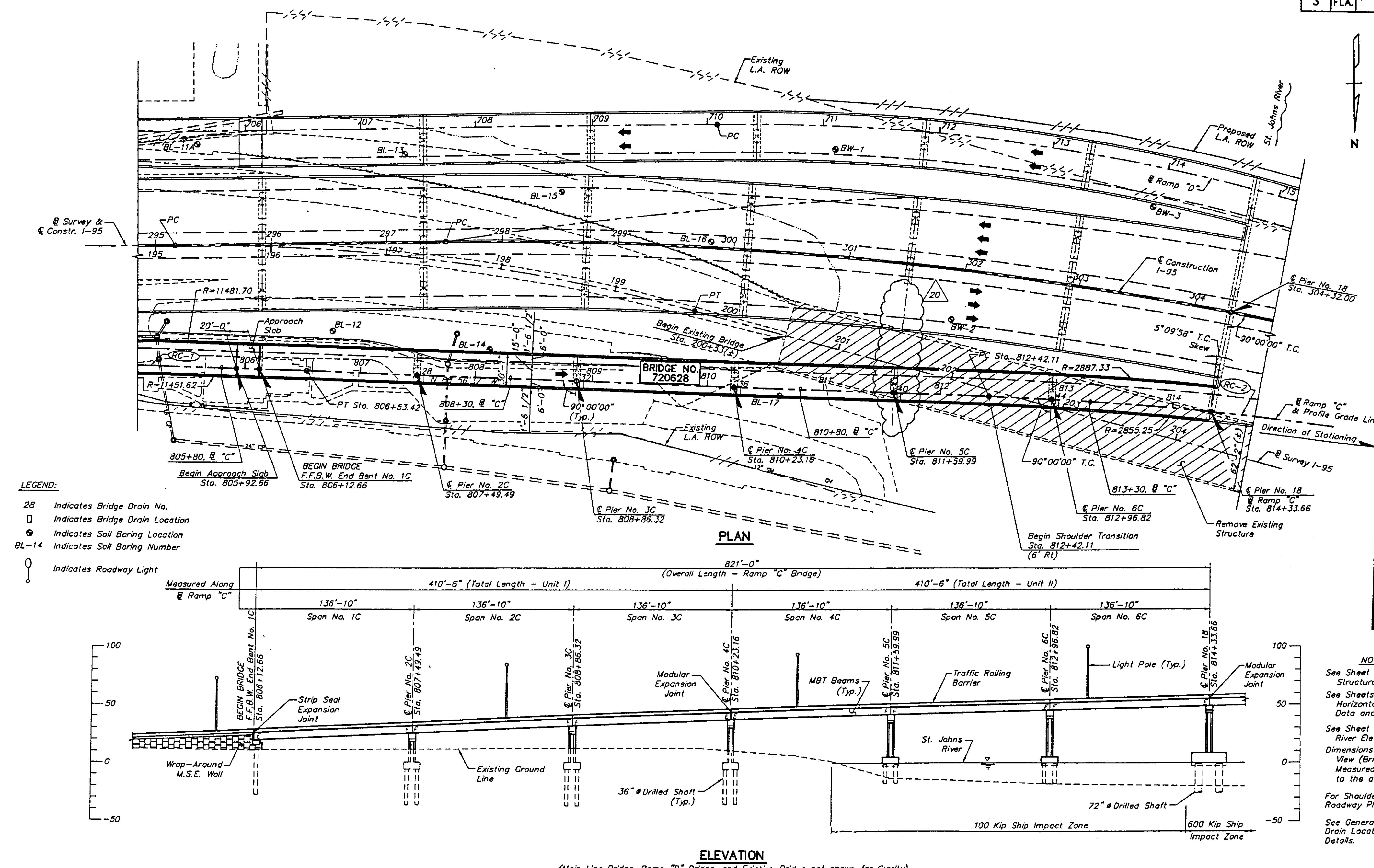
SHEET TITLE: DRILLED SHAFT DETAILS (2)

PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

Drawing No.

Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.		95	G1



[redacted]

NOTES:

See Sheet A4 & A5 for Structural Notes

See Sheets A6 & A7 for Horizontal & Vertical Curve Data and Traffic Data.

See Sheet A7 for St. Johns River Elevations.

Dimensions shown on Plan View (Bridge Width, etc.) are Measured radial/normal to the associated €/€.

For Shoulder Transition see Roadway Plans.

See General Sheets for Bridge Drain Locations and Drainage Details.

BRIDGE NO. 720628

REVISIONS					
Date	By	Description	Date	By	Description
5/15/00	MDC	20 TEMPORARY PIERS REMOVED			

	Names	Dates
Drawn by	BDV/CSD	8-93
Checked by	LDP	11-93
Designed by	MJL/HDR	9-93
Checked by	DMM	12-93
Approved by	S.M. JARRETT	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL: M. Caulfield 5-15-00

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020 - 3485

SHEET TITLE: PLAN AND ELEVATION RAMP "C" BRIDGE

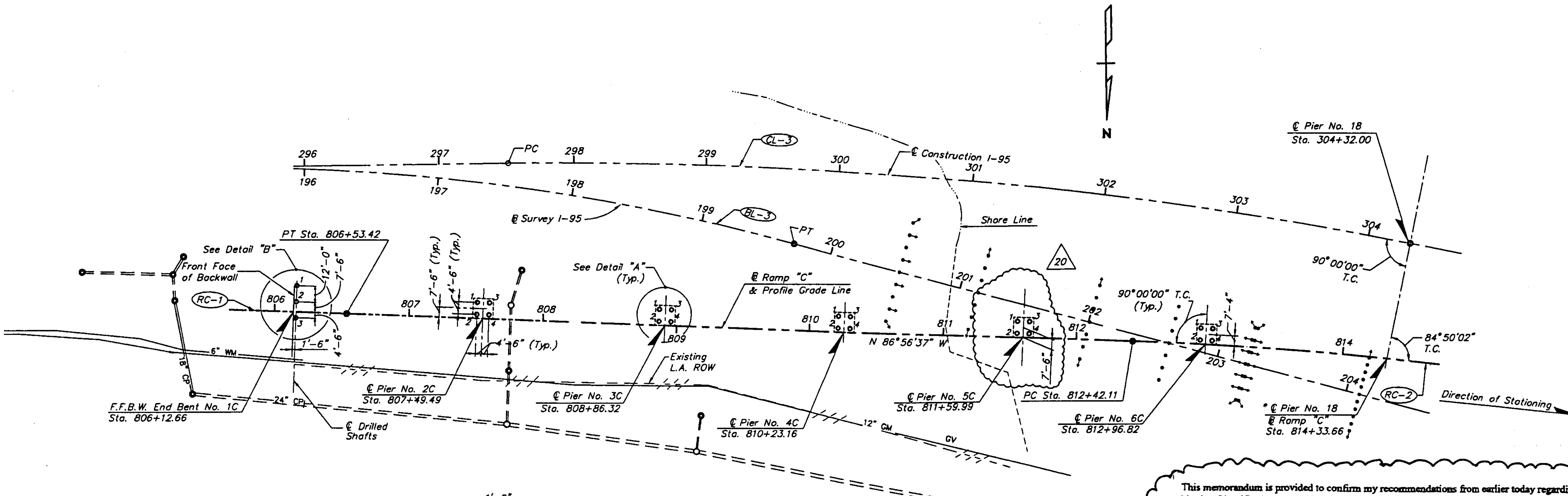
PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

Drawing No.

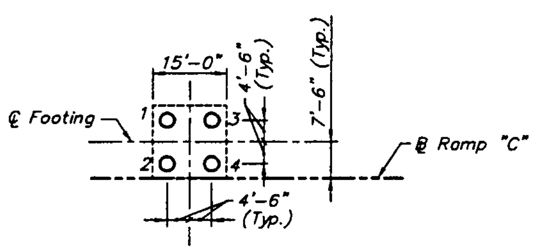
Index No.

NAME: M:\16414\REVISION 20\DWG-G01.DWG DATE: MAY 3, 2000 TIME: 1:56 PM REV. BY: RLG

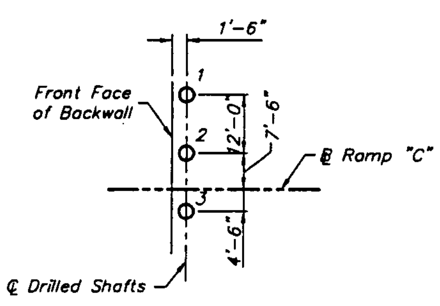
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.		95	G2



PLAN



DETAIL "A"



DETAIL "B"

To: Pom Chakkaphak, P.E. PBC:
From: Ray Castelli, P.E.
Date: March 9, 2001
Subject: Fuller Warren Bridge
36-inch Diameter Drilled Shafts
Pier 6C, Shaft No. 2

23

This memorandum is provided to confirm my recommendations from earlier today regarding Shaft No. 2 at Pier 6C. As reported by Chuck Iley, the initial installation of this shaft could not be completed because the drilling tools could advance no closer than 5 feet to the casing tip due to suspected deformation of the casing. At the time this problem was encountered, the casing tip was at elevation -61.02.

Considering the inability to advance the drill tools to the bottom of the casing, it will be necessary to backfill the excavation, extract the existing casing and replace it with another casing. Since it is very likely that the replacement casing will be slightly offset from the original casing alignment, we must assume a reduced friction resistance within the depth of the original casing. To compensate for this reduced friction, it is recommended that the shaft penetration be increased an additional 7 feet to a revised tip elevation at -68. With the exception of the revised shaft tip elevation, the procedures for installation of this shaft should be the same as those for the original shaft, and the final casing tip elevation -42 remains unchanged.

Please contact me if you have any questions regarding the above recommendations, or if further problems are encountered at this shaft.

ACTUAL AS-BUILT SEE SHEET No. G2A

DRILLED SHAFT INSTALLATION TABLE

END BENT or PIER NO.	SHAFT DIAMETER (INCHES)	DESIGN LOAD (TONS)	MINIMUM TIP ELEVATION (FT)	100 YEAR SCOUR ELEVATION (FT)
1C	36	250	−35	
2C	36	400	−35	
3C	36	400	−35	
4C	36	400	−35	
5C	36	450	−48	−36
6C	36	450	−56	−42
18	*	*	*	*

20

* For Foundation Layout and Drilled Shaft Installation Data for Pier No. 18, See Sheet D4 or E4.

ACTUAL AS-BUILT SEE SHEET No. G2A

TABLE OF DRILLED SHAFT TOP ELEVATIONS

SHAFT NO.	END BENT NO. 1C	PIER NO. 2C THRU PIER 4C	PIER NO. 5C	PIER NO. 6C
1.	14.15	−4.25	−0.83	−0.83
2.	13.91	−4.25	−0.83	−0.83
3.	13.68	−4.25	−0.83	−0.83
4.	−	−4.25	−0.83	−0.83

20

LEGEND

- O Proposed Drilled Shaft
- Existing Jacketed Steel Pile with Batter Direction

NOTES

1. For Horizontal Curve Data see Sheet A7.
2. Plan location of Drilled Shafts is given at Bottom of Caps.
3. All Drilled Shafts at the End Bent shall be installed Prior to the Installation of MSE Walls.
4. Drilled Shafts shall be installed to the Minimum Tip Elevations shown in the Drilled Shaft Installation Table.
5. For Drilled Shaft Reinf. Details and associated notes see Sheet A24.

20

6. The portion of End Bent drilled shafts exposed above the existing ground shall be wrapped with 6 mil. thickness polyethylene plastic film. See General Notes, Sheet A4.
7. The Foundations shall be constructed in the Phases outlined in the Sequence of Construction drawings.

DWG DATE: MAY 10, 2000 TIME: 12:19 PM REV. BY: RLC

NAME: M:\16414\RAMPC\

REVISIONS							
Date	By		Description	Date	By		Description
5/15/00	MDC	20	TEMPORARY PIERS REMOVED				
6/1/01	SH	23	AS-BUILT				

	Names	Dates
Drawn by	SCG/CSD	10−93
Checked by	TQT	7−94
Designed by	TQT/RMS	7−94
Checked by	LDP/TQT	7−94

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL
M. Castelli 5-15-00

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020 − 3485

SHEET TITLE: FOUNDATION LAYOUT RAMP "C" BRIDGE

PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

Drawing No.

Index No.

3	FLA		95	G2-A

AS-BUILT

DRILLED SHAFT PAY QUANTITIES - RAMP "C" BRIDGE

		DESIGN CONDITIONS								AS-BUILT CONDITIONS							
PIER NO.	SHAFT NO.	SHAFT SIZE	AXIAL LOAD	100-YR SCOUR EL.	CUT-OFF ELEV.	PLAN TIP ELEV.	AUTH TI ELEV.	EXTRA DEPTH	SHAFT LENGTH	FINISH DATE	GROUND ELEV.	AUTH TI ELEV.	ACTU TI ELEV.	UNCLAS EXCAV.	EXTRA DEPTH	SHAFT LENGTH	COMMENTS
		in	tons	ft	ft	ft	ft	ft	ft		ft	ft	ft	ft	ft	ft	
1C	1	36	250	N/A	14.15	-35.0	-36.0	1.0	50.15	4/17/00	11	-36	-36.43	46.00	1.00	50.15	
1C	2	36	250	N/A	13.91	-35.0	-35.0	0.0	48.91	5/2/00	11	-41	-41.26	46.00	6.00	54.91	
1C	3	36	250	N/A	13.68	-35.0	-35.0	0.0	48.68	5/4/00	11	-35	-35.08	46.00	0.00	48.68	
2C	1	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	4/12/00	-4.75	-39	-39.27	30.25	4.00	34.75	
2C	2	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	4/17/00	-4.75	-39	-39.18	30.25	4.00	34.75	
2C	3	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	4/13/00	-4.75	-39	-39.00	30.25	4.00	34.75	
2C	4	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	4/11/00	-4.75	-39	-42.18	30.25	4.00	34.75	
3C	1	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	5/16/00	-4.75	-39	-48.16	30.25	4.00	34.75	
3C	2	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	5/18/00	-4.75	-39	-39.36	30.25	4.00	34.75	
3C	3	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	5/15/00	-4.75	-39	-39.1	30.25	4.00	34.75	
3C	4	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	5/19/00	-4.75	-39	-39.02	30.25	4.00	34.75	
4C	1	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	5/15/00	-4.75	-39	-40.4	30.25	4.00	34.75	
4C	2	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	5/16/00	-4.75	-39	-35.59	30.25	4.00	34.75	
4C	3	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	5/17/00	-4.75	-39	-39.02	30.25	4.00	34.75	
4C	4	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	5/18/00	-4.75	-39	-39.15	30.25	4.00	34.75	
5C	1	36	250	-36	-0.83	-41.0	-43.0	2.0	42.17	3/22/01	-6.25	-64.00	-68.68	34.75	23.00	63.17	By Ray
5C	2	36	250	-36	-0.83	-41.0	-43.0	2.0	42.17	4/2/01	-6.25	-64.00	-64.63	34.75	23.00	63.17	By Ray
5C	3	36	250	-36	-0.83	-41.0	-43.0	2.0	42.17	3/24/01	-6.25	-64.00	-64.07	34.75	23.00	63.17	By Ray
5C	4	36	250	-36	-0.83	-41.0	-43.0	2.0	42.17	4/5/01	-6.25	-64.00	-65.25	34.75	23.00	63.17	By Ray
5C	5	36	450	-36	-0.83	-48.0	-48.0	0.0	47.17			-56.00					Deleted
5C	6	36	450	-36	-0.83	-48.0	-48.0	0.0	47.17			-56.00					Deleted
5C	7	36	450	-36	-0.83	-48.0	-48.0	0.0	47.17			-56.00					Deleted
5C	8	36	450	-36	-0.83	-48.0	-48.0	0.0	47.17			-56.00					Deleted
5C	9	36	250	-36	-0.83	-41.0	-43.0	2.0	42.17			-50.00					Deleted
5C	10	36	250	-36	-0.83	-41.0	-43.0	2.0	42.17			-50.00					Deleted
5C	11	36	250	-36	-0.83	-41.0	-43.0	2.0	42.17			-50.00					Deleted
5C	12	36	250	-36	-0.83	-41.0	-43.0	2.0	42.17			-50.00					Deleted
6C	1	36	450	-42	-0.83	-56.0	-61.0	5.0	60.17	3/2/01	-18.5	-61.0	-61.01	37.50	5.00	60.17	
6C	2	36	450	-42	-0.83	-56.0	-61.0	5.0	60.17	3/8/01	-18.5	-68.0	-68.82	37.50	12.00	67.17	By Ray
6C	3	36	450	-42	-0.83	-56.0	-61.0	5.0	60.17	3/7/01	-18.5	-61.0	-61.14	37.50	5.00	60.17	
6C	4	36	450	-42	-0.83	-56.0	-61.0	5.0	60.17	3/13/01	-18.5	-61.0	-61.27	37.50	5.00	60.17	
TOTAL	31							85.0	1331.5					790.00	174.00	1071.10	

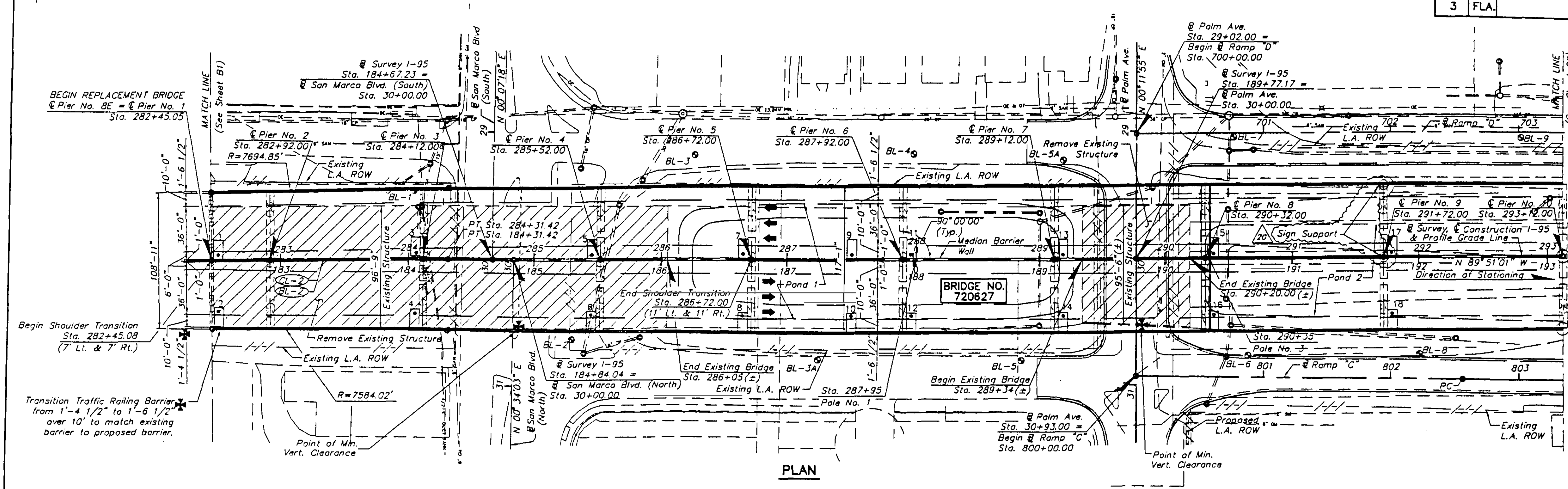
1246.5

23

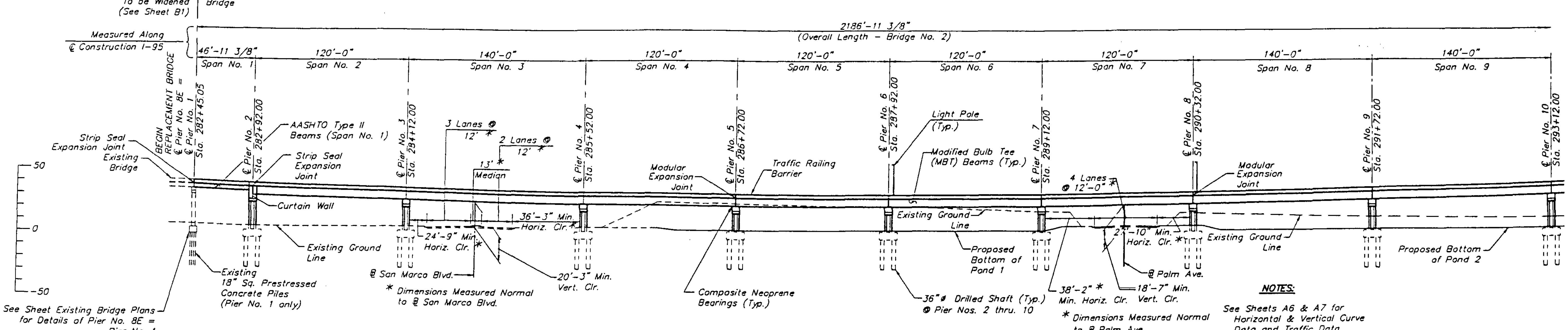
REVISIONS					
Date	By	Description	Date	By	Description
7-21-01	SH	23 AS-BUILT			

SHEET TITLE: FOUNDATION LAYOUT RAMP "C" BRIDGE	Drawing No.
PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER	Index No.

ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.		95	C1



PLAN



ELEVATION
(Existing Bridges not shown for Clarity)

LEGEND:

Indicates (2) 400 Watt High Pressure Sodium Luminaire

13 —— Indicates Bridge Drain Number

Indicates Bridge Drain Location

Indicates Soil Boring Location

NOTE:
See General Sheets for Bridge Drain Locations and Drainage Details.

NOTES:
See Sheets A6 & A7 for Horizontal & Vertical Curve Data and Traffic Data.
Dimensions shown on Plan View (Bridge Width, etc.) are Measured radial/normal to the associated ℄ / ℄.

BRIDGE NO. 720627

REVISIONS

Date	By	Description	Date	By	Description
5/15/00	MDC	20 ADD SIGN SUPPORT AT PIER 9			

	Names	Dates
Drawn by	SPJ	8-94
Checked by	PLS	8-94
Designed by	MJL/HDR	9-93
Checked by	DMM	12-93
Approved by	M. D. Caulfield	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL: M. Caulfield 5/15/00

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

SHEET TITLE: PLAN AND ELEVATION (1) BRIDGE NO. 2

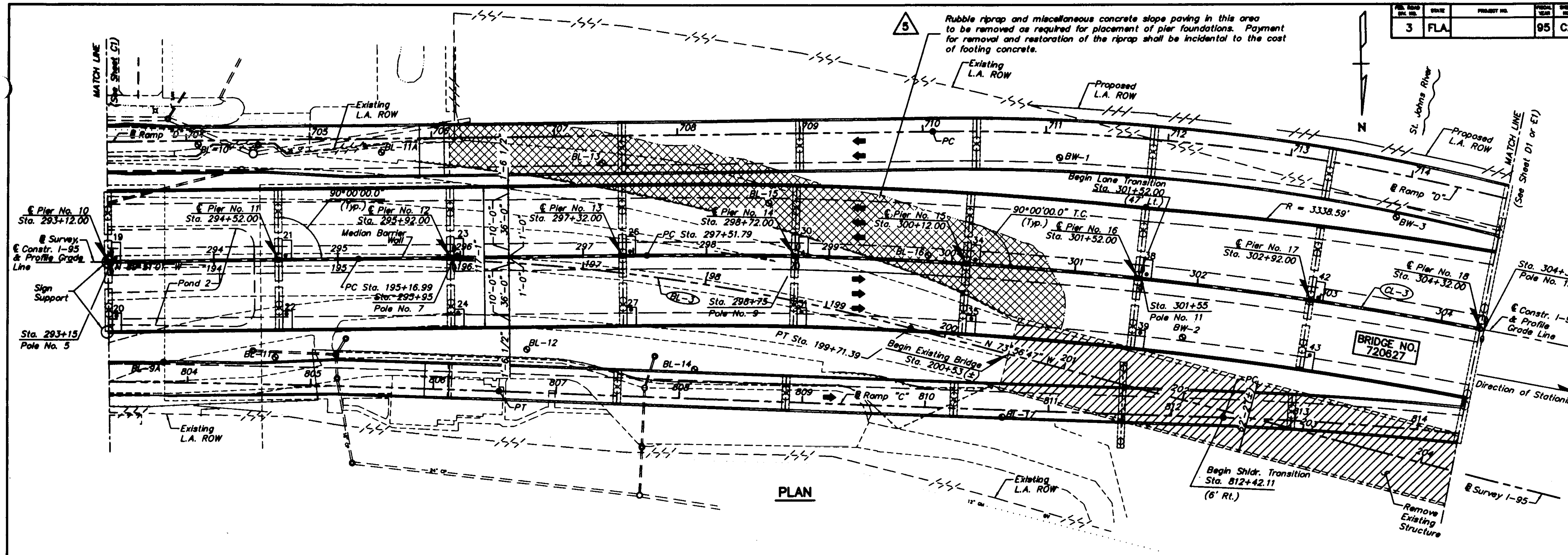
PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

Drawing No.

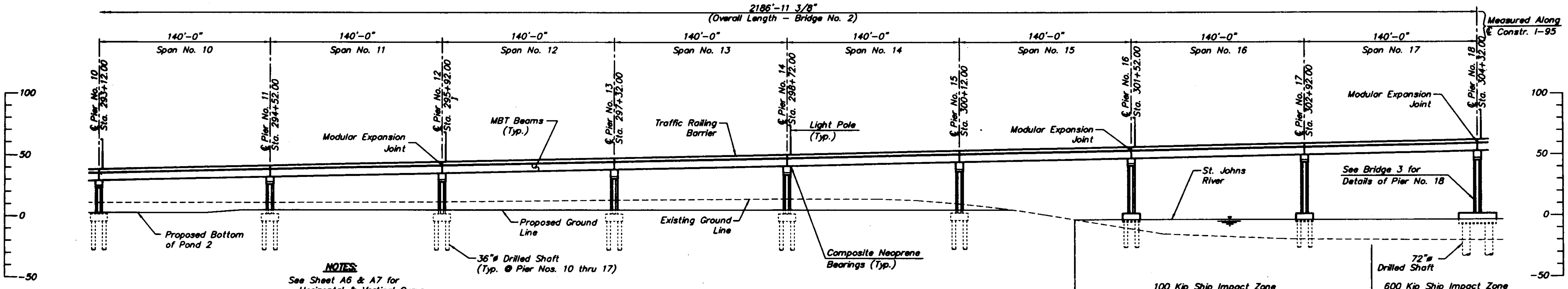
Index No.

NAME: M:\16414\REVISION20\2BRD02A.DWG DATE: MAY 3, 2000 TIME: 10:53 AM REV. BY: RLG

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.		95	C2



PLAN



ELEVATION

(Existing Bridge and Ramp Bridges not shown for Clarity)

NOTES:

See Sheet A6 & A7 for Horizontal & Vertical Curve Data and Traffic Data.

See Sheet A7 for Elevation of St. Johns River

Dimensions shown on Plan View (Bridge Width, etc.) are Measured radial/normal to the associated ℄ / ℄.

⊙ Indicates Soil Boring Location.

NOTE:

See General Sheets for Bridge Drain Locations and Drainage Details.

LEGEND:

- Indicates (2) 400 Watt High Pressure Sodium Luminaire
- 27 — Indicates Bridge Drain Number
- Indicates Bridge Drain Location
- ⊙ Indicates Soil Boring Location

BRIDGE NO. 720627

REVISIONS

Date	By	Description	Date	By	Description
2/5/96	DMW	5 Added note identifying rubble riprap.			

	Names	Dates
Drawn by	BDV	7-93
Checked by	LDP	11-93
Designed by	MJL/HDR	9-93
Checked by	DMM	12-93
Approved by	M. D. Caulfield	

HNTB HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

SHEET TITLE: PLAN AND ELEVATION (2) BRIDGE NO. 2

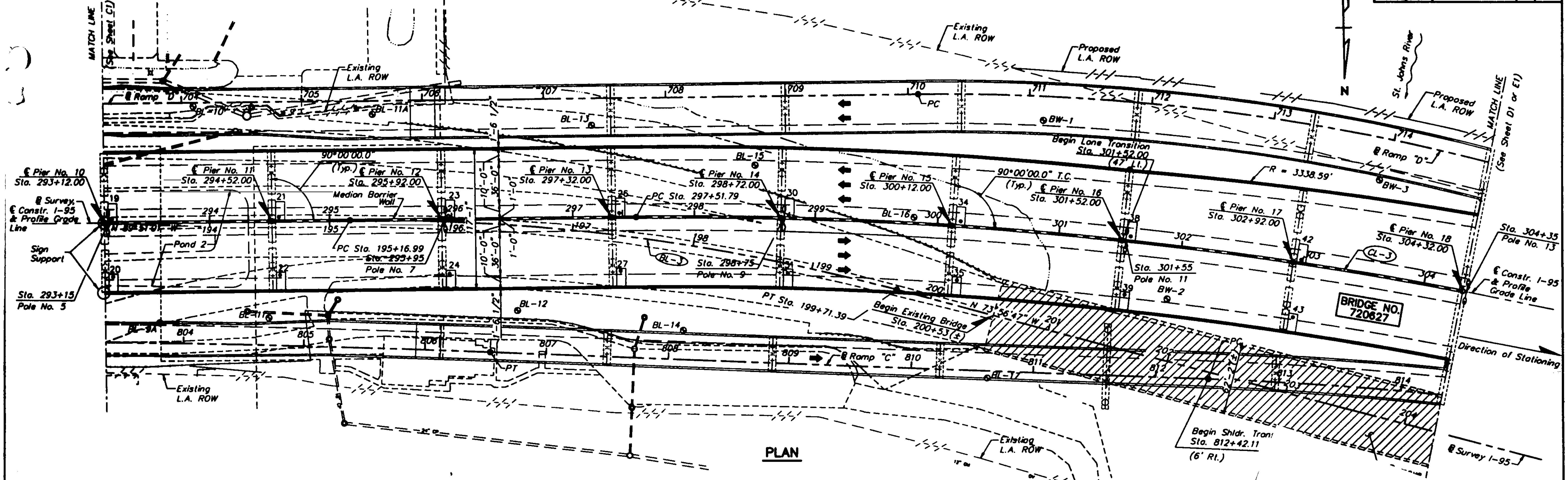
PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

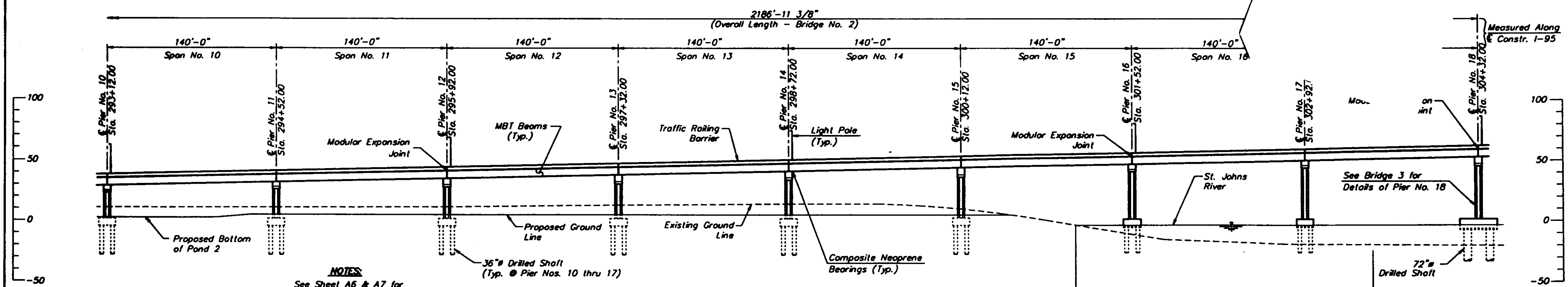
Drawing No.

Index No.

NAME: ... DATE: FEB 07, 1996 TIME: 12:43 PM REV. BY: RLG

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.		95	C2





ELEVATION
(Existing Bridge and Ramp Bridges not shown for Clarity)

NOTES:
See Sheet A6 & A7 for Horizontal & Vertical Curve Data and Traffic Data.
See Sheet A7 for Elevation of St. Johns River
Dimensions shown on Plan View (Bridge Width,etc.) are Measured radial/normal to the associated ℄ / ℄.
⊗ indicates Soil Boring Location.

NOTE:
See General Sheets for Bridge Drain Locations and Drainage Details.

LEGEND:
Indicates (2) 400 Watt High Pressure Sodium Luminaire
27 — Indicates Bridge Drain Number
Indicates Bridge Drain Location
⊗ Indicates Soil Boring Location

BRIDGE NO. 720627

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	BDV	7-93
Checked by	LDP	11-93
Designed by	MJL/HDR	9-93
Checked by	DMM	12-93
Approved by	M. D. Coulfield	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

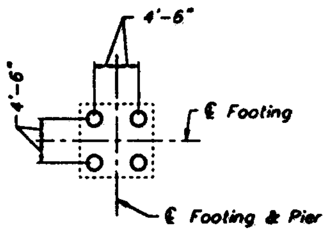
SHEET TITLE: PLAN AND ELEVATION (2) BRIDGE NO. 2

PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

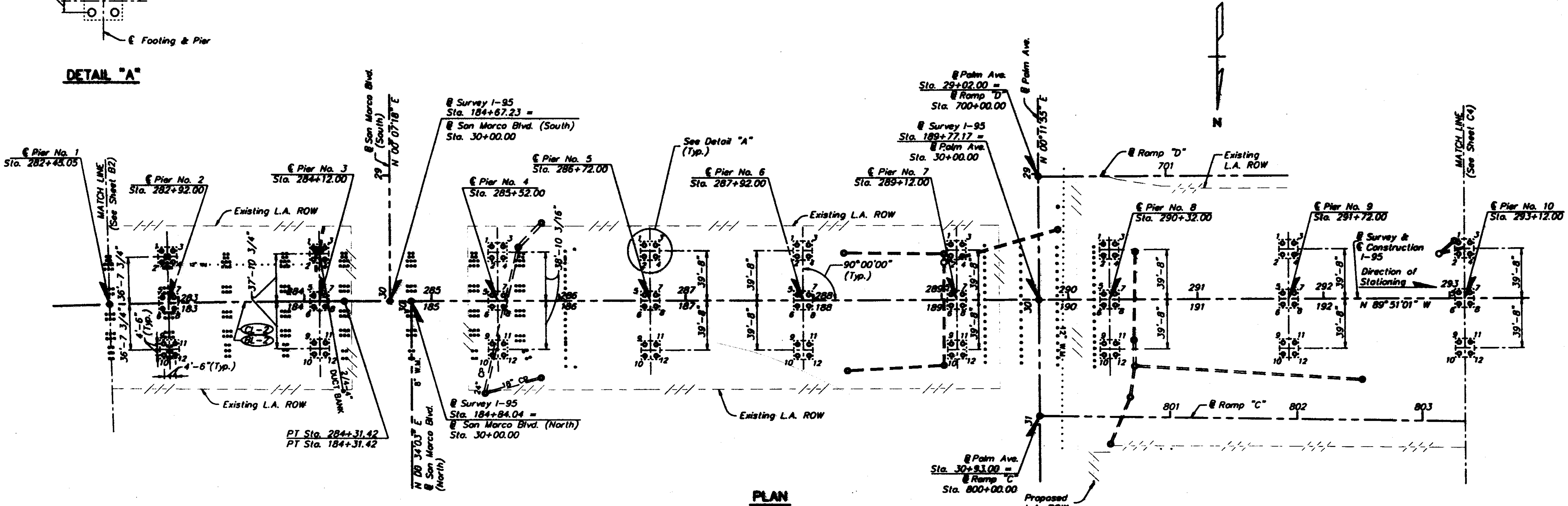
Drawing No.

Index No.

3	FLA.			95	C3



DETAIL "A"



PLAN

LEGEND

○ Proposed Drilled Shaft

⟵○ Existing Pile with Batter Direction

DRILLED SHAFT INSTALLATION TABLE

PIER No.	SHAFT DIAMETER (INCHES)	DESIGN LOAD (TONS)	MINIMUM TIP ELEVATION (FT)	100 YR. SCOUR ELEVATION (FT)
2	36	450	-35	
3	36			
4	36			
5	36			
6	36			
7	36			
8	36			
9	36			
10	36			
11	36			
12	36			
13	36			
14	36	450	-35	
15	36	550	-37	
16	36	550	-51	-36
17	36	550	-59	-42
18	*	*	*	*

*For Foundation Layout Installation Data for Pier No. 18, see Sheet D4 or E4.

TABLE OF DRILLED SHAFT TOP ELEVATIONS

SHAFT No.	PIER 2 THRU 14	PIER 15	PIERS 16 & 17
1	-4.250	-0.830	-0.830
2			
3			
4		-0.830	
5		-4.250	
6			
7			
8			
9			
10			
11			
12	-4.250	-4.250	-0.830

NOTES

1. For Horizontal Curve Data see Sheet A7.
2. Plan location of Drilled Shafts is given at Bottom of Caps.
3. Drilled Shafts shall be installed to the Minimum Tip Elevations Shown in the Drilled Shaft Installation Table.
4. Existing Piles at Pier No. 2 shall be completely extracted.
5. For Drilled Shaft reinforcing details and associated notes, see Sheet A24.
6. The foundations shall be constructed in the Phases outlined in the Sequence of Construction drawings.

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	SPJ	4-94
Checked by	PLS	7-94
Designed by	MJL/HDR	9-93
Checked by	DMM	12-93
Approved by	M. D. Coulfield	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

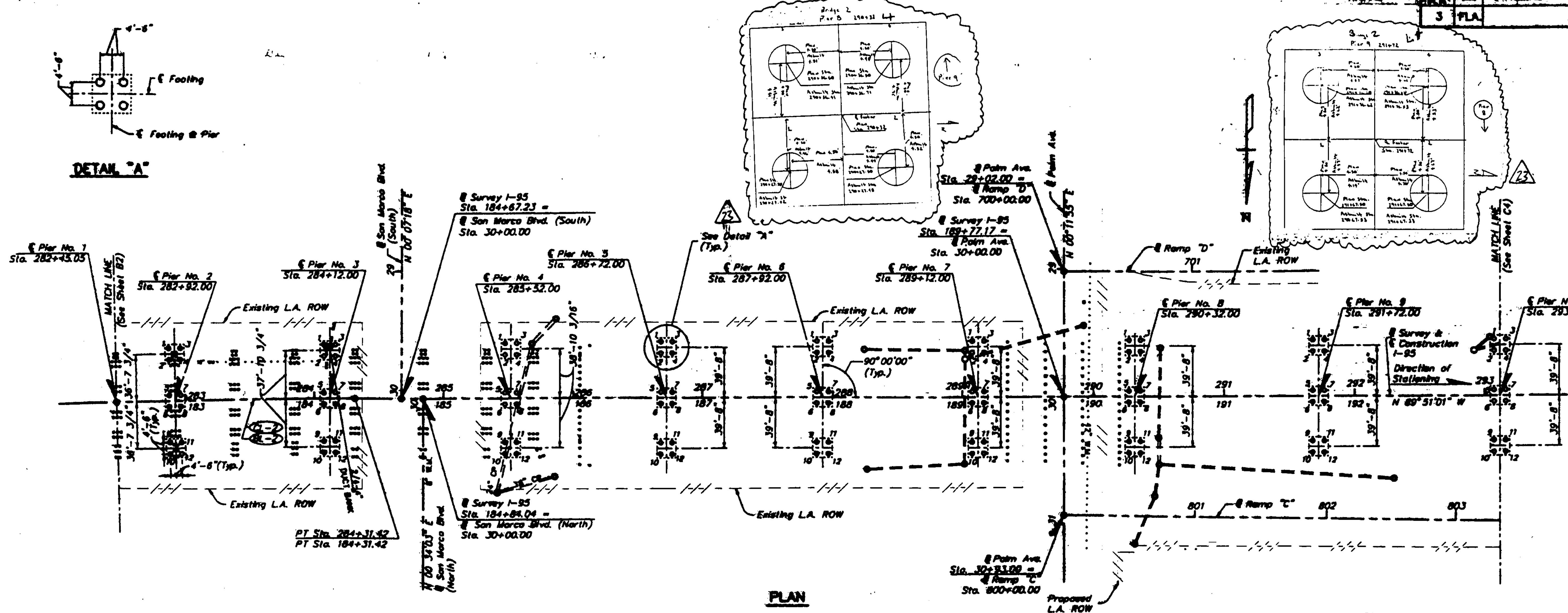
SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

SHEET TITLE: FOUNDATION LAYOUT (1)
BRIDGE NO. 2

PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER



DRILLED SHAFT INSTALLATION TABLE				
PIER No.	SHAFT DIAMETER (INCHES)	DESIGN LOAD (TONS)	MINIMUM TIP ELEVATION (FT)	100 YR. SCOUR ELEVATION (FT)
2	36	450	-35	
3	36			
4	36			
5	36			
6	36			
7	36			
8	36			
9	36			
10	36			
11	36			
12	36			
13	36			
14	36	450	-35	
15	36	550	-37	
16	36	550	-51	-36
17	36	550	-59	-42
18	*	*	*	*

*For Foundation Layout Installation Data for Pier No. 18, see Sheet D4 or E4.

TABLE OF DRILLED SHAFT TOP ELEVATIONS			
SHAFT No.	PIER 2 THRU 14	PIER 15	PIERS 16 & 17
1	-4.250	-0.830	-0.830
2			
3			
4		-0.830	
5		-4.250	
6			
7			
8			
9			
10			
11			
12	-4.250	-4.250	-0.830

NOTES

1. For Horizontal Curve Data see Sheet A7.
2. Plan location of Drilled Shafts is given at Bottom of Caps.
3. Drilled Shafts shall be installed to the Minimum Tip Elevations Shown in the Drilled Shaft Installation Table.
4. Existing Piles at Pier No. 2 shall be completely extracted.
5. For Drilled Shaft reinforcing details and associated notes, see Sheet A24.
6. The foundations shall be constructed in the Phases outlined in the Sequence of Construction drawings.

This is in response to your Letter No. PB-114-97 dated, April 9th, 1997, regarding ~~the above mentioned subject~~. Your request to cast the footing of Pier 4 South in Bridge No. 2 on four drilled shafts which were cast 10-inches north of the plan location is approved. The footing and the column dowels will be placed in the correct plan location.

13

REVISIONS			
Date	By	Description	
7-15-[illegible]	SH	13 AS-BUILT	

	Names	Dates
Drawn by	SPV	4-94
Checked by	PLS	7-94
Designed by	MJL/HDR	9-93
Checked by	DMM	12-93
Approved by	M. D. Caulfield	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

SHEET TITLE: FOUNDATION LAYOUT (1)
BRIDGE NO. 2

PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

FULLER WARREN BRIDGE

DRILLED SHAFT PAY QUANTITIES - BRIDGE NO. 2

		DESIGN CONDITIONS								AS-BUILT CONDITIONS							
PIER NO.	SHAFT NO.	SHAFT SIZE	AXIAL LOAD	100-YR SCOUR EL	CUT-OFF ELEV.	PLAN TIP ELEV.	AUTH TIP ELEV.	EXTRA DEPTH	SHAFT LENGTH	FINISH DATE	GROUND ELEV.	AUTH TIP ELEV.	ACTU TIP ELEV.	UNCLAS EXCAV.	EXTRA DEPTH	SHAFT LENGTH	COMMENTS
		in	tons	ft	ft	ft	ft	ft	ft		ft	ft	ft	ft	ft	ft	
2	1	36	450	N/A	-4.25	-35	-35	0	30.75	02/18/97	-4.75	-35.00	-35.00	30.25	0.00	30.75	
2	2	36	450	N/A	-4.25	-35	-35	0	30.75	02/14/97	-4.75	-35.00	-35.16	30.25	0.00	30.75	
2	3	36	450	N/A	-4.25	-35	-35	0	30.75	02/13/97	-4.75	-35.00	-35.10	30.25	0.00	30.75	
2	4	36	450	N/A	-4.25	-35	-35	0	30.75	02/19/97	-4.75	-35.00	-35.16	30.25	0.00	30.75	
2	5	36	450	N/A	-4.25	-35	-35	0	30.75	07/08/00	-4.75	-35.00	-35.00	30.25	0.00	30.75	
2	6	36	450	N/A	-4.25	-35	-35	0	30.75	07/10/00	-4.75	-35.00	-35.13	30.25	0.00	30.75	
2	7	36	450	N/A	-4.25	-35	-35	0	30.75	07/07/00	-4.75	-35.00	-35.80	30.25	0.00	30.75	
2	8	36	450	N/A	-4.25	-35	-35	0	30.75	07/11/00	-4.75	-35.00	-36.23	30.25	0.00	30.75	
2	9	36	450	N/A	-4.25	-35	-35	0	30.75	04/07/01	-4.75	-35.00	-35.15	30.25	0.00	30.75	
2	10	36	450	N/A	-4.25	-35	-35	0	30.75	04/13/01	-4.75	-35.00	-35.42	30.25	0.00	30.75	
2	11	36	450	N/A	-4.25	-35	-35	0	30.75	04/10/01	-4.75	-35.00	-35.37	30.25	0.00	30.75	
2	12	36	450	N/A	-4.25	-35	-35	0	30.75	04/16/01	-4.75	-35.00	-35.20	30.25	0.00	30.75	
3	1	36	450	N/A	-4.25	-35	-35	0	30.75	02/14/97	-4.75	-35.00	-35.10	30.25	0.00	30.75	
3	2	36	450	N/A	-4.25	-35	-35	0	30.75	02/19/97	-4.75	-35.00	-35.00	30.25	0.00	30.75	
3	3	36	450	N/A	-4.25	-35	-35	0	30.75	02/18/97	-4.75	-35.00	-35.00	30.25	0.00	30.75	
3	4	36	450	N/A	-4.25	-35	-35	0	30.75	02/13/97	-4.75	-35.00	-35.03	30.25	0.00	30.75	
3	5	36	450	N/A	-4.25	-35	-35	0	30.75	07/14/00	-4.75	-35.00	-35.39	30.25	0.00	30.75	
3	6	36	450	N/A	-4.25	-35	-35	0	30.75	07/14/00	-4.75	-35.00	-35.23	30.25	0.00	30.75	
3	7	36	450	N/A	-4.25	-35	-35	0	30.75	07/15/00	-4.75	-35.00	-35.40	30.25	0.00	30.75	
3	8	36	450	N/A	-4.25	-35	-35	0	30.75	07/12/00	-4.75	-35.00	-35.18	30.25	0.00	30.75	
3	9	36	450	N/A	-4.25	-35	-35	0	30.75	04/09/01	-4.75	-35.00	-35.34	30.25	0.00	30.75	
3	10	36	450	N/A	-4.25	-35	-35	0	30.75	04/10/01	-4.75	-35.00	-35.26	30.25	0.00	30.75	
3	11	36	450	N/A	-4.25	-35	-35	0	30.75	04/13/01	-4.75	-35.00	-35.16	30.25	0.00	30.75	
3	12	36	450	N/A	-4.25	-35	-35	0	30.75	04/11/01	-4.75	-35.00	-36.59	30.25	0.00	30.75	
4	1	36	450	N/A	-4.25	-35	-35	0	30.75	02/05/97	-4.75	-35.00	-35.04	30.25	0.00	30.75	
4	2	36	450	N/A	-4.25	-35	-35	0	30.75	02/06/97	-4.75	-35.00	-35.00	30.25	0.00	30.75	
4	3	36	450	N/A	-4.25	-35	-35	0	30.75	02/07/97	-4.75	-35.00	-35.10	30.25	0.00	30.75	
4	4	36	450	N/A	-4.25	-35	-35	0	30.75	02/04/97	-4.75	-35.00	-35.00	30.25	0.00	30.75	
4	5	36	450	N/A	-4.25	-35	-35	0	30.75	08/17/00	-4.75	-35.00	-35.44	30.25	0.00	30.75	
4	6	36	450	N/A	-4.25	-35	-35	0	30.75	08/16/00	-4.75	-38.00	-38.23	30.25	3.00	33.75	
4	7	36	450	N/A	-4.25	-35	-35	0	30.75	08/18/00	-4.75	-35.00	-35.37	30.25	0.00	30.75	
4	8	36	450	N/A	-4.25	-35	-35	0	30.75	08/19/00	-4.75	-35.00	-36.47	30.25	0.00	30.75	
4	9	36	450	N/A	-4.25	-35	-35	0	30.75	04/17/01	-4.75	-35.00	-35.20	30.25	0.00	30.75	
4	10	36	450	N/A	-4.25	-35	-35	0	30.75	04/19/01	-4.75	-35.00	-35.07	30.25	0.00	30.75	
4	11	36	450	N/A	-4.25	-35	-35	0	30.75	04/20/01	-4.75	-35.00	-35.05	30.25	0.00	30.75	
4	12	36	450	N/A	-4.25	-35	-35	0	30.75	04/23/01	-4.75	-35.00	-35.14	30.25	0.00	30.75	
5	1	36	450	N/A	-4.25	-35	-35	0	30.75	01/28/97	-4.75	-35.00	-35.27	30.25	0.00	30.75	
5	2	36	450	N/A	-4.25	-35	-35	0	30.75	01/27/97	-4.75	-35.00	-35.72	30.25	0.00	30.75	
5	3	36	450	N/A	-4.25	-35	-35	0	30.75	01/22/97	-4.75	-35.00	-35.20	30.25	0.00	30.75	
5	4	36	450	N/A	-4.25	-35	-35	0	30.75	01/24/97	-4.75	-35.00	-35.10	30.25	0.00	30.75	
5	5	36	450	N/A	-4.25	-35	-35	0	30.75	08/11/00	-4.75	-35.00	-35.00	30.25	0.00	30.75	
5	6	36	450	N/A	-4.25	-35	-35	0	30.75	08/08/00	-4.75	-35.00	-35.52	30.25	0.00	30.75	
5	7	36	450	N/A	-4.25	-35	-35	0	30.75	08/09/00	-4.75	-35.00	-35.78	30.25	0.00	30.75	
5	8	36	450	N/A	-4.25	-35	-35	0	30.75	08/05/00	-4.75	-35.00	-35.48	30.25	0.00	30.75	
5	9	36	450	N/A	-4.25	-35	-35	0	30.75	04/20/01	-4.75	-35.00	-35.33	30.25	0.00	30.75	
5	10	36	450	N/A	-4.25	-35	-35	0	30.75	04/23/01	-4.75	-35.00	-35.19	30.25	0.00	30.75	
5	11	36	450	N/A	-4.25	-35	-35	0	30.75	04/17/01	-4.75	-35.00	-35.02	30.25	0.00	30.75	
5	12	36	450	N/A	-4.25	-35	-35	0	30.75	04/19/01	-4.75	-35.00	-35.20	30.25	0.00	30.75	

		DESIGN CONDITIONS								AS-BUILT CONDITIONS							
PIER NO.	SHAFT NO.	SHAFT SIZE	AXIAL LOAD	100-YR SCOUR EL	CUT-OFF ELEV.	PLAN TIP ELEV.	AUTH TIP ELEV.	EXTRA DEPTH	SHAFT LENGTH	FINISH DATE	GROUND ELEV.	AUTH TIP ELEV.	ACTU TIP ELEV.	UNCLAS EXCAV.	EXTRA DEPTH	SHAFT LENGTH	COMMENTS
		in	tons	ft	ft	ft	ft	ft	ft		ft	ft	ft	ft	ft	ft	
6	1	36	450	N/A	-4.25	-35	-35	0	30.75	2/3/97	-4.75	-35.00	-35.20	30.25	0	30.75	
6	2	36	450	N/A	-4.25	-35	-35	0	30.75	1/22/97	-4.75	-35.00	-35.20	30.25	0	30.75	
6	3	36	450	N/A	-4.25	-35	-35	0	30.75	1/21/97	-4.75	-35.00	-35.15	30.25	0	30.75	
6	4	36	450	N/A	-4.25	-35	-35	0	30.75	1/24/97	-4.75	-35.00	-35.24	30.25	0	30.75	
6	5	36	450	N/A	-4.25	-35	-35	0	30.75	7/29/00	-4.75	-35.00	-35.47	30.25	0	30.75	
6	6	36	450	N/A	-4.25	-35	-35	0	30.75	7/31/00	-4.75	-35.00	-35.24	30.25	0	30.75	
6	7	36	450	N/A	-4.25	-35	-35	0	30.75	7/20/00	-4.75	-35.00	-37.1	30.25	0	30.75	
6	8	36	450	N/A	-4.25	-35	-35	0	30.75	7/28/00	-4.75	-35.00	-35	30.25	0	30.75	ify with log estimated elevations.
6	9	36	450	N/A	-4.25	-35	-35	0	30.75	4/24/01	-4.75	-35.00	-35.23	30.25	0	30.75	
6	10	36	450	N/A	-4.25	-35	-35	0	30.75	4/25/01	-4.75	-37.50	-37.51	30.25	2.5	33.25	
6	11	36	450	N/A	-4.25	-35	-35	0	30.75	4/27/01	-4.75	-36.00	-36.02	30.25	1	31.75	
6	12	36	450	N/A	-4.25	-35	-35	0	30.75	4/30/01	-4.75	-35.00	-35.24	30.25	0	30.75	
7	1	36	450	N/A	-4.25	-35	-35	0	30.75	1/21/97	-4.75	-38.00	-38.55	30.25	3.00	33.75	Extra Auth. RJC
7	2	36	450	N/A	-4.25	-35	-35	0	30.75	2/3/97	-4.75	-35.00	-35.02	30.25	0.00	30.75	
7	3	36	450	N/A	-4.25	-35	-35	0	30.75	1/27/97	-4.75	-42.00	-42.30	30.25	7.00	37.75	7' per criteria
7	4	36	450	N/A	-4.25	-35	-35	0	30.75	1/23/97	-4.75	-37.00	-38.18	30.25	2.00	32.75	Extra Auth. RJC
7	5	36	450	N/A	-4.25	-35	-35	0	30.75	7/19/00	-4.75	-35.00	-35.49	30.25	0.00	30.75	
7	6	36	450	N/A	-4.25	-35	-35	0	30.75	7/27/00	-4.75	-35.00	-35.19	30.25	0.00	30.75	
7	7	36	450	N/A	-4.25	-35	-35	0	30.75	8/1/00	-4.75	-35.00	-36.67	30.25	0.00	30.75	
7	8	36	450	N/A	-4.25	-35	-35	0	30.75	8/3/00	-4.75	-35.00	-35.90	30.25	0.00	30.75	
7	9	36	450	N/A	-4.25	-35	-35	0	30.75	4/24/01	-4.75	-43.00	-43.31	30.25	8.00	38.75	
7	10	36	450	N/A	-4.25	-35	-35	0	30.75	4/30/01	-4.75	-42.00	-42.50	30.25	7.00	37.75	
7	11	36	450	N/A	-4.25	-35	-35	0	30.75	4/24/01	-4.75	-41.78	-41.00	30.25	6.78	37.53	
7	12	36	450	N/A	-4.25	-35	-35	0	30.75	5/25/01	-4.75	-41.60	-42.12	30.25	6.50	37.25	
8	1	36	450	N/A	-4.25	-35	-43	8	38.75	1/9/97	-4.75	-53.00	-53.46	30.25	18.00	48.75	Extra Auth. RJC
8	2	36	450	N/A	-4.25	-35	-43	8	38.75	1/13/97	-4.75	-53.00	-53.3	30.25	18.00	48.75	Extra Auth. RJC
8	3	36	450	N/A	-4.25	-35	-43	8	38.75	1/16/97	-4.75	-43.00	-43.06	30.25	8.00	38.75	Extra Auth. RJC
8	4	36	450	N/A	-4.25	-35	-43	8	38.75	1/14/97	-4.75	-45.00	-45.07	30.25	10.00	40.75	Extra Auth. RJC
8	5	36	450	N/A	-4.25	-35	-35	0	30.75	8/22/00	-4.75	-42.00	-42.47	30.25	7.00	37.75	
8	6	36	450	N/A	-4.25	-35	-35	0	30.75	8/25/00	-4.75	-43.00	-43.74	30.25	8.00	38.75	
8	7	36	450	N/A	-4.25	-35	-35	0	30.75	8/23/00	-4.75	-35.00	-38.6	30.25	0.00	30.75	
8	8	36	450	N/A	-4.25	-35	-35	0	30.75	8/28/00	-4.75	-42.00	-42.27	30.25	7.00	37.75	
8	9	36	450	N/A	-4.25	-35	-35	0	30.75	5/2/01	-4.75	-35.00	-39.53	30.25	0.00	30.75	
8	10	36	450	N/A	-4.25	-35	-35	0	30.75	5/1/01	-4.75	-35.00	-39.2	30.25	0.00	30.75	
8	11	36	450	N/A	-4.25	-35	-35	0	30.75	5/3/01	-4.75	-35.00	-37.4	30.25	0.00	30.75	
8	12	36	450	N/A	-4.25	-35	-35	0	30.75	5/4/01	-4.75	-41	-41.37	30.25	6	36.75	
9	1	36	450	N/A	-4.25	-35	-43	8	38.75	1/16/97	-4.75	-43.00	-43.07	30.25	8.00	38.75	
9	2	36	450	N/A	-4.25	-35	-43	8	38.75	1/14/97	-4.75	-43.00	-45.10	30.25	8.00	38.75	
9	3	36	450	N/A	-4.25	-35	-43	8	38.75	1/15/97	-4.75	-45.00	-45.40	30.25	10.00	40.75	Extra Auth. RJC
9	4	36	450	N/A	-4.25	-35	-43	8	38.75	1/17/97	-4.75	-43.00	-43.50	30.25	8.00	38.75	
9	5	36	450	N/A	-4.25	-35	-36	1	31.75	8/30/00	-4.75	-36	-36.88	30.25	1	31.75	
9	6	36	450	N/A	-4.25	-35	-36	1	31.75	9/1/00	-4.75	-37.5	-37.56	30.25	2.5	33.25	
9	7	36	450	N/A	-4.25	-35	-36	1	31.75	8/29/00	-4.75	-36	-37.87	30.25	1	31.75	
9	8	36	450	N/A	-4.25	-35	-36	1	31.75	8/31/00	-4.75	-37	-37.68	30.25	2	32.75	
9	9	36	450	N/A	-4.25	-35	-36	1	31.75	5/4/01	-4.75	-36	-39.55	30.25	1	31.75	
9	10	36	450	N/A	-4.25	-35	-36	1	31.75	5/7/01	-4.75	-36	-40.08	30.25	1	31.75	
9	11	36	450	N/A	-4.25	-35	-36	1	31.75	5/2/01	-4.75	-36	-40.48	30.25	1	31.75	
9	12	36	450	N/A	-4.25	-35	-36	1	31.75	5/3/01	-4.75	-36	-40.37	30.25	1	31.75	
								72	1548					1452.00	170.28	1646.28	

AS-BUILT

REVISIONS
Date	By	Description	Date	By	Description
7-20-01	SH	3 AS-BUILT			

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO. 9 | COUNTY DUVAL | PROJECT NO. 72020-3485

SHEET TITLE: FOUNDATION LAYOUT (2) BRIDGE NO. 2 AS-BUILT

PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

3 | FLA | 95 | C4-A

FULLER WARREN BRIDGE

DRILLED SHAFT PAY QUANTITIES - BRIDGE NO. 2

		DESIGN CONDITIONS								AS-BUILT CONDITIONS							
PIER NO.	SHAFT NO.	SHAFT SIZE	AXIAL LOAD	100-YR SCOUR EL	CUT-OFF ELEV.	PLAN TIP ELEV.	AUTH TIP ELEV.	EXTRA DEPTH	SHAFT LENGTH	FINISH DATE	GROUND ELEV.	AUTH TIP ELEV.	ACTU TIP ELEV.	UNCLAS EXCAV.	EXTRA DEPTH	SHAFT LENGTH	COMMENTS
		in	tons	ft	ft	ft	ft	ft	ft		ft	ft	ft	ft	ft	ft	
10	1	36	450	N/A	-4.25	-35	-38	3	33.75	1/10/97	-4.75	-38.00	-38.08	30.25	3.00	33.75	
10	2	36	450	N/A	-4.25	-35	-38	3	33.75	12/11/96	-4.75	-39.30	39.60	30.25	4.30	35.05	Auth. by RJC
10	3	36	450	N/A	-4.25	-35	-38	3	33.75	1/15/97	-4.75	-38.00	-38.20	30.25	3.00	33.75	
10	4	36	450	N/A	-4.25	-35	-38	3	33.75	1/7/97	-4.75	-38.00	-38.02	30.25	3.00	33.75	
10	5	36	450	N/A	-4.25	-35	-38	3	33.75	9/8/00	-4.75	-38.00	-47.68	30.25	3.00	33.75	
10	6	36	450	N/A	-4.25	-35	-38	3	33.75	9/12/00	-4.75	-38.00	-45.35	30.25	3.00	33.75	
10	7	36	450	N/A	-4.25	-35	-38	3	33.75	9/11/00	-4.75	-38.00	-45.49	30.25	3.00	33.75	
10	8	36	450	N/A	-4.25	-35	-38	3	33.75	9/13/00	-4.75	-38.00	-44.80	30.25	3.00	33.75	
10	9	36	450	N/A	-4.25	-35	-38	3	33.75	5/11/01	-4.75	-38.00	-38.75	30.25	3.00	33.75	
10	10	36	450	N/A	-4.25	-35	-38	3	33.75	5/10/01	-4.75	-38.00	-39.42	30.25	3.00	33.75	
10	11	36	450	N/A	-4.25	-35	-38	3	33.75	5/16/01	-4.75	-38.00	-40.50	30.25	3.00	33.75	
10	12	36	450	N/A	-4.25	-35	-38	3	33.75	5/21/01	-4.75	-38.00	-41.01	30.25	3.00	33.75	
11	1	36	450	N/A	-4.25	-35	-42	7	37.75	12/12/96	-4.75	-42.00	-43.45	30.25	7.00	37.75	
11	2	36	450	N/A	-4.25	-35	-42	7	37.75	3/6/97	-4.75	-43.00	-43.20	30.25	8.00	38.75	Extra 1' per RJC
11	3	36	450	N/A	-4.25	-35	-42	7	37.75	3/5/97	-4.75	-42.00	-42.00	30.25	7.00	37.75	
11	4	36	450	N/A	-4.25	-35	-42	7	37.75	3/4/97	-4.75	-53.00	-53.20	30.25	18.00	48.75	Extra 11' per RJC
11	5	36	450	N/A	-4.25	-35	-42	7	37.75	10/9/00	-4.75	-42.00	-46.92	30.25	7.00	37.75	
11	6	36	450	N/A	-4.25	-35	-42	7	37.75	10/12/00	-4.75	-42.00	-46.74	30.25	7.00	37.75	
11	7	36	450	N/A	-4.25	-35	-42	7	37.75	10/5/00	-4.75	-42.00	-48.27	30.25	7.00	37.75	
11	8	36	450	N/A	-4.25	-35	-42	7	37.75	10/10/00	-4.75	-42.00	-45.67	30.25	7.00	37.75	
11	9	36	450	N/A	-4.25	-35	-42	7	37.75	5/24/01	-4.75	-42.00	-44.00	30.25	7.00	37.75	
11	10	36	450	N/A	-4.25	-35	-42	7	37.75	5/30/01	-4.75	-42.00	-44.30	30.25	7.00	37.75	
11	11	36	450	N/A	-4.25	-35	-42	7	37.75	6/1/01	-4.75	-42.00	-44.17	30.25	7.00	37.75	
11	12	36	450	N/A	-4.25	-35	-42	7	37.75	6/4/01	-4.75	-42.00	-44.34	30.25	7.00	37.75	
12	1	36	450	N/A	-4.25	-35	-40	5	35.75	12/4/96	-4.75	-40.00	-46.10	30.25	5.00	35.75	
12	2	36	450	N/A	-4.25	-35	-40	5	35.75	12/5/96	-4.75	-40.00	-40.00	30.25	5.00	35.75	
12	3	36	450	N/A	-4.25	-35	-40	5	35.75	12/3/96	-4.75	-42.67	-43.25	30.25	7.67	38.42	Auth. by RJC
12	4	36	450	N/A	-4.25	-35	-40	5	35.75	11/25/96	-4.75	-40.00	-41.10	30.25	5.00	35.75	
12	5	36	450	N/A	-4.25	-35	-40	5	35.75	10/13/00	-4.75	-40.00	-41.96	30.25	5.00	35.75	
12	6	36	450	N/A	-4.25	-35	-40	5	35.75	10/19/00	-4.75	-40.00	-41.61	30.25	5.00	35.75	
12	7	36	450	N/A	-4.25	-35	-40	5	35.75	10/17/00	-4.75	-40.00	-40.99	30.25	5.00	35.75	
12	8	36	450	N/A	-4.25	-35	-40	5	35.75	10/21/00	-4.75	-40.00	-40.18	30.25	5.00	35.75	
12	9	36	450	N/A	-4.25	-35	-40	5	35.75	6/5/01	-4.75	-40.00	-47.65	30.25	5.00	35.75	
12	10	36	450	N/A	-4.25	-35	-40	5	35.75	6/6/01	-4.75	-43.00	-47.58	30.25	8.00	38.75	Auth. by RJC
12	11	36	450	N/A	-4.25	-35	-40	5	35.75	6/7/01	-4.75	-42.00	-47.49	30.25	7.00	37.75	
12	12	36	450	N/A	-4.25	-35	-40	5	35.75	6/8/01	-4.75	-40.00	-47.63	30.25	5.00	35.75	
13	1	36	450	N/A	-4.25	-35	-42	7	37.75	10/18/96	-4.75	-42.00	-42.70	30.25	7.00	37.75	
13	2	36	450	N/A	-4.25	-35	-42	7	37.75	10/22/96	-4.75	-42.00	-42.19	30.25	7.00	37.75	
13	3	36	450	N/A	-4.25	-35	-42	7	37.75	10/24/96	-4.75	-42.00	-42.18	30.25	7.00	37.75	
13	4	36	450	N/A	-4.25	-35	-42	7	37.75	10/25/96	-4.75	-42.00	-43.40	30.25	7.00	37.75	
13	5	36	450	N/A	-4.25	-35	-42	7	37.75	10/24/00	-4.75	-42.00	-43.03	30.25	7.00	37.75	
13	6	36	450	N/A	-4.25	-35	-42	7	37.75	10/26/00	-4.75	-42.00	-42.04	30.25	7.00	37.75	
13	7	36	450	N/A	-4.25	-35	-42	7	37.75	10/25/00	-4.75	-43.00	-43.02	30.25	8.00	38.75	Auth. by RJC
13	8	36	450	N/A	-4.25	-35	-42	7	37.75	10/27/00	-4.75	-42.00	-42.11	30.25	7.00	37.75	
13	9	36	450	N/A	-4.25	-35	-42	7	37.75	6/12/01	-4.75	-42.00	-46.34	30.25	7.00	37.75	
13	10	36	450	N/A	-4.25	-35	-42	7	37.75	6/19/01	-4.75	-42.00	-43.85	30.25	7.00	37.75	
13	11	36	450	N/A	-4.25	-35	-42	7	37.75	6/14/01	-4.75	-49.00	-49.38	30.25	14.00	44.75	Auth. by RJC
13	12	36	450	N/A	-4.25	-35	-42	7	37.75	6/15/01	-4.75	-42.00	-43.90	30.25	7.00	37.75	

		DESIGN CONDITIONS								AS-BUILT CONDITIONS							
PIER NO.	SHAFT NO.	SHAFT SIZE	AXIAL LOAD	100-YR SCOUR EL	CUT-OFF ELEV.	PLAN TIP ELEV.	AUTH TIP ELEV.	EXTRA DEPTH	SHAFT LENGTH	FINISH DATE	GROUND ELEV.	AUTH TIP ELEV.	ACTU TIP ELEV.	UNCLAS EXCAV.	EXTRA DEPTH	SHAFT LENGTH	COMMENTS
		in	tons	ft	ft	ft	ft	ft	ft		ft	ft	ft	ft	ft	ft	
14	1	36	450	N/A	2.25	-35	-41	6	43.25	10/31/96	1.75	-41.00	-41.26	36.75	6.00	43.25	
14	2	36	450	N/A	2.25	-35	-41	6	43.25	10/30/96	1.75	-41.00	-41.05	36.75	6.00	43.25	
14	3	36	450	N/A	2.25	-35	-41	6	43.25	11/4/96	1.75	-41.00	-41.07	36.75	6.00	43.25	
14	4	36	450	N/A	2.25	-35	-41	6	43.25	11/2/96	1.75	-41.00	-41.02	36.75	6.00	43.25	
14	5	36	450	N/A	2.25	-35	-41	6	43.25	10/31/00	1.75	-41.00	-46.38	36.75	6.00	43.25	
14	6	36	450	N/A	2.25	-35	-41	6	43.25	11/1/00	1.75	-47.50	-48.41	36.75	12.50	49.75	Per Ray Castelli
14	7	36	450	N/A	2.25	-35	-41	6	43.25	10/30/00	1.75	-41.00	-47.96	36.75	6.00	43.25	
14	8	36	450	N/A	2.25	-35	-41	6	43.25	11/2/00	1.75	-41.00	-41.09	36.75	6.00	43.25	
14	9	36	450	N/A	2.25	-35	-41	6	43.25	6/18/01	1.75	-42.00	-43.76	36.75	7.00	44.25	Per Ray Castelli
14	10	36	450	N/A	2.25	-35	-41	6	43.25	6/25/01	1.75	-50.00	-55.56	36.75	15.00	52.25	Per Ray Castelli
14	11	36	450	N/A	2.25	-35	-41	6	43.25	7/11/01	1.75	-41.00	-48.80	36.75	6.00	43.25	
14	12	36	450	N/A	2.25	-35	-41	6	43.25	7/2/01	1.75	-41.00	-48.69	36.75	6.00	43.25	
15	1	36	550	N/A	-0.83	-37	-42	5	41.17	3/26/99	-8	-55.00	-69.70	29.00	18.00	54.17	
15	2	36	550	N/A	-0.83	-37	-42	5	41.17	3/12/99	-1	-57.00	-59.30	36.00	20.00	56.17	Per Ray Castelli
15	3	36	550	N/A	-0.83	-37	-42	5	41.17	3/26/99	-8	-55.00	-69.70	29.00	18.00	54.17	
15	4	36	550	N/A	-0.83	-37	-42	5	41.17	3/23/99	-4	-55.00	-90.15	33.00	18.00	54.17	
15	5	36	550	N/A	-4.25	-37	-42	5	37.75	10/15/96	-4.75	-42.00	-42.22	32.25	5.00	37.75	
15	6	36	550	N/A	-4.25	-37	-42	5	37.75	10/12/96	-4.75	-42.00	-42.23	32.25	5.00	37.75	
15	7	36	550	N/A	-4.25	-37	-42	5	37.75	11/18/96	-4.75	-42.00	-41.92	32.25	4.92	37.67	Bottom Above Auth Tip
15	8	36	550	N/A	-4.25	-37	-42	5	37.75	11/19/96	-4.75	-42.00	-42.00	32.25	5.00	37.75	
15	9	36	550	N/A	-4.25	-37	-42	5	37.75	7/2/01	-4.75	-48.00	-52.55	32.25	11.00	43.75	Per Ray Castelli
15	10	36	550	N/A	-4.25	-37	-42	5	37.75	6/25/01	-4.75	-55.00	-55.56	32.25	18.00	50.75	Per Ray Castelli
15	11	36	550	N/A	-4.25	-37	-42	5	37.75	6/21/01	-4.75	-42.00	-46.07	32.25	5.00	37.75	
15	12	36	550	N/A	-4.25	-37	-42	5	37.75	7/11/01	-4.75	-42.00	-53.04	32.25	5.00	37.75	
16	1	36	550	N/A	-0.83	-51	-54	3	53.17	2/15/99	-17.5	-54.00	-72.15	33.50	3.00	53.17	See PBDN 7130
16	2	36	550	N/A	-0.83	-51	-54	3	53.17	2/18/99	-18.5	-54.00	-71.00	32.50	3.00	53.17	See PBDN 7130
16	3	36	550	N/A	-0.83	-51	-54	3	53.17	2/24/99	-16.5	-54.00	-72.65	34.50	3.00	53.17	See PBDN 7130
16	4	36	550	N/A	-0.83	-51	-54	3	53.17	2/22/99	-14	-54.00	-71.20	37.00	3.00	53.17	See PBDN 7130
16	5	36	550	N/A	-0.83	-51	-54	3	53.17	2/16/99	-16.5	-54.00	-70.75	34.50	3.00	53.17	See PBDN 7130
16	6	36	550	N/A	-0.83	-51	-54	3	53.17	2/19/99	-14	-54.00	-71.50	37.00	3.00	53.17	See PBDN 7130
16	7	36	550	N/A	-0.83	-51	-54	3	53.17	2/24/99	-16	-54.00	-70.53	35.00	3.00	53.17	See PBDN 7130
16	8	36	550	N/A	-0.83	-51	-54	3	53.17	2/22/99	-13.5	-54.00	-71.60	37.50	3.00	53.17	See PBDN 7130
16	9	36	550	N/A	-0.83	-51	-54	3	53.17	2/23/99	-13	-54.00	-72.05	38.00	3.00	53.17	See PBDN 7130
16	10	36	550	N/A	-0.83	-51	-54	3	53.17	2/19/99	-11.5	-54.00	-71.70	39.50	3.00	53.17	See PBDN 7130
16	11	36	550	N/A	-0.83	-51	-54	3	53.17	2/26/99	-16	-54.00	-70.51	35.00	3.00	53.17	See PBDN 7130
16	12	36	550	N/A	-0.83	-51	-54	3	53.17	3/1/99	-16	-54.00	-71.29	35.00	3.00	53.17	See PBDN 7130
17	1	36	550	N/A	-0.83	-59	-63	4	62.17	12/14/98	-19.5	-63.00	-76.10	39.50	4.00	62.17	Additional depth required
17	2	36	550	N/A	-0.83	-59	-63	4	62.17	12/29/98	-19.5	-63.00	-99.30	39.50	4.00	62.17	due to contractor
17	3	36	550	N/A	-0.83	-59	-63	4	62.17	1/9/99	-19.5	-63.00	-93.10	39.50	4.00	62.17	diffiulties. Authorized
17	4	36	550	N/A	-0.83	-59	-63	4	62.17	1/15/99	-19.5	-63.00	-71.78	39.50	4.00	62.17	tip elevation used to
17	5	36	550	N/A	-0.83	-59	-63	4	62.17	12/30/98	-19.5	-63.00	-96.15	39.50	4.00	62.17	calculate shaft lengths.
17	6	36	550	N/A	-0.83	-59	-63	4	62.17	1/20/99	-19.5	-63.00	-73.82	39.50	4.00	62.17	
17	7	36	550	N/A	-0.83	-59	-63	4	62.17	1/9/99	-19.5	-63.00	-96.30	39.50	4.00	62.17	
17	8	36	550	N/A	-0.83	-59	-63	4	62.17	1/18/99	-19.5	-63.00	-74.32	39.50	4.00	62.17	
17	9	36	550	N/A	-0.83	-59	-63	4	62.17	1/21/99	-19.5	-63.00	-73.70	39.50	4.00	62.17	
17	10	36	550	N/A	-0.83	-59	-63	4	62.17	1/4/99	-20	-63.00	-76.30	39.00	4.00	62.17	
17	11	36	550	N/A	-0.83	-59	-63	4	62.17	1/15/99	-19.5	-63.00	-90.57	39.50	4.00	62.17	
17	12	36	550	N/A	-0.83	-59	-63	4	62.17	1/18/99	-19.5	-63.00	-72.59	39.50	4.00	62.17	
TOTAL	192								7133.76					6084.50	771.67	7353.43	

AS-BUILT

AS-BUILT

REVISIONS

Date	By	Description
7-21-01	SH	AS-BUILT

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

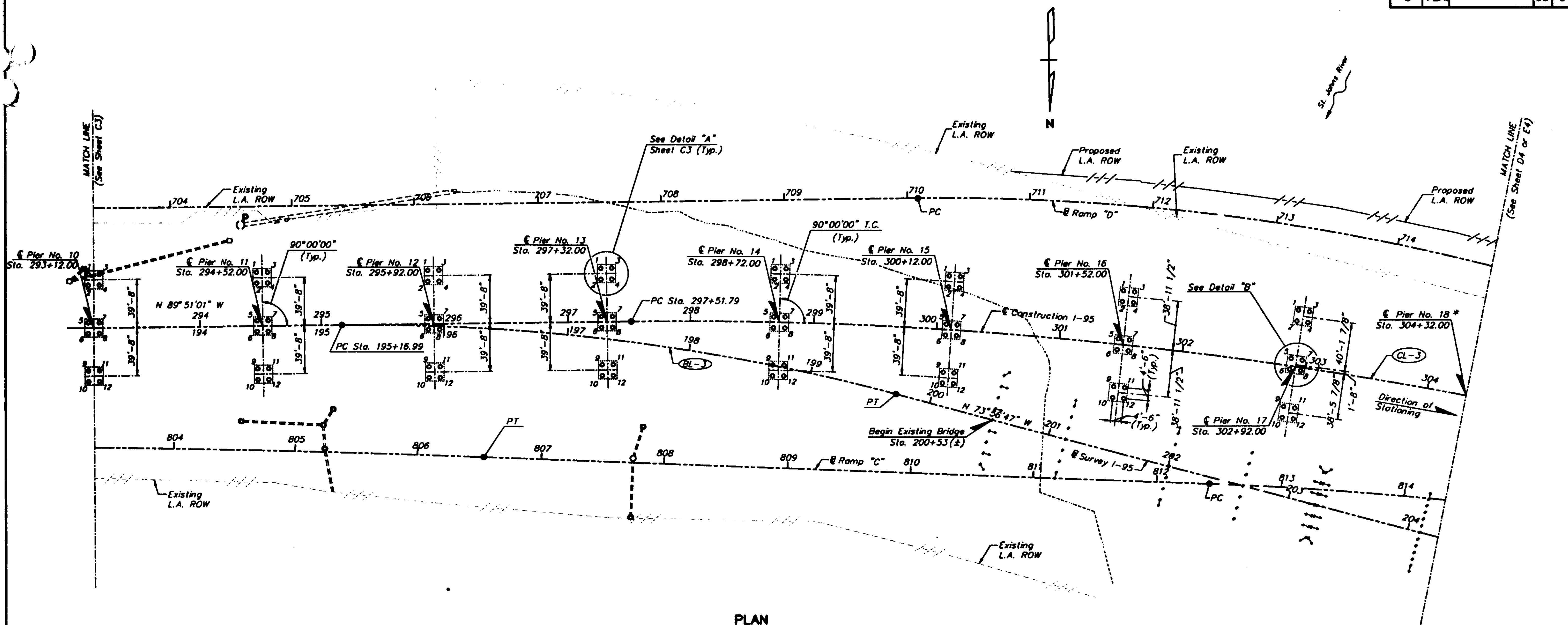
ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

SHEET TITLE: FOUNDATION LAYOUT (2) BRIDGE NO. 2

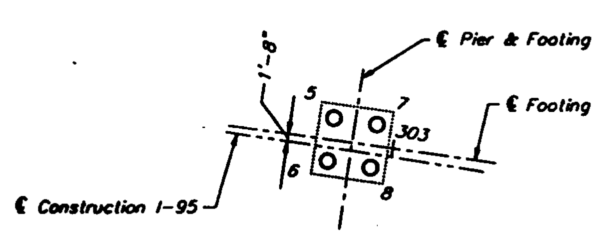
PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

FISCAL YEAR	STATE	SHEET NO.
3	FLA	95 C4-B

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.		95	C4



PLAN



DETAIL "B"

NOTES

1. For Horizontal Curve Data see Sheet A7.
2. Plan location of Drilled Shafts is given at Bottom of Caps.
3. Drilled Shafts shall be installed to the Minimum Tip Elevations Shown in the Drilled Shaft Installation Table.
4. For Drilled Shaft Installation Table, See Sheet C3.
5. For Drilled Shaft reinforcing details and associated notes, see Sheet A24.
* 6. For Foundation layout and installation data for Pier 18, see Sheet D4 or E4.
7. The foundations shall be constructed in the Phases outlined in the Sequence of Construction drawings.

LEGEND

- O Proposed Drilled Shaft
- Existing Pile with Batter Direction
- Existing Jacketed Steel Pile with Batter Direction

REVISIONS

	Names	Dates
Drawn by	SPJr.	4-94
Checked by	PLS	7-94
Designed by	MJL/HDR	9-93
Checked by	DMM	12-93
Approved by	M. D. Coulfield	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

SHEET TITLE: FOUNDATION LAYOUT (2) BRIDGE NO. 2

PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

Drawing No.

Index No.

A2

3 FLA

GENERAL SHEETS

All

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- A2 Index of Sheets(1)
- A3 Index of Sheets(2)
- A4 General Notes (1)
- A5 General Notes (2)
- A6 Plan & Elevation Data (1)
- A7 Plan & Elevation Data (2)
- A8 Miscellaneous Details
- A9 Composite Neoprene Bearing Pads (1)
- A10 Composite Neoprene Bearing Pads (2)
- A11 Expansion Joints
- A12 Modular Expansion Joints (1)
- A13 Modular Expansion Joints Data
- A14 Light Pole Pilaster
- A15 Navigational Light System Details
- ~~A16 NOT USED~~
- A17 Plan of Fender System
- A17A Elevations of Fender System
- A18 Connection Details for Fender System - 1
- A18A Connection Details for Fender System - 2
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- A21 Standard Bar Bending Details
- A22 Prestressed Concrete Piles (1)
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- A24 Drilled Shaft Details (1)
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- A33 Legend for Generalized Subsurface Profiles
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- A41 Cantilever Retaining Wall (Index 800)
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- A43 Attenuator Survey (1)
- A44 Attenuator Survey (2)
- A45 Attenuator Survey (3)
- A46 Attenuator Survey (4)
- A47 Foundation Load Test Program Requirements (1)
- A48 Foundation Load Test Program Requirements (2)
- A49 Foundation Load Test Program Requirements (3)

720629

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BRIDGE NO. 1 - PARTIAL WIDENING

720154

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- B3 End Bent No. 2E
- B4 End Bent Details
- B5 Pier Nos. 8E,9E,10E or 11E
- B6 Pier No. 12E
- B7 Pier No. 13E and 14E
- B8 Miscellaneous Pier Details
- B9 Construction Data
- B10 Superstructure Plan (1)
- B11 Superstructure Plan (2)
- B12 Superstructure Section
- B13 Superstructure Details (1)
- B14 Superstructure Details (2)
- B15 Framing Plan
- B16 AASHTO Type II Beams
- B17 Typical Notes & Details for AASHTO Type II Prestressed Beams
- B18 Girder Details (1)
- B19 Girder Details (2)
- B20 Rolled Beam Details
- B21 Camber Diagram
- B22 Reinforcing Bar List (1)
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720627

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- C2 Plan and Elevation (2)
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- C4 Foundation Layout (2)
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- C6 Pier Cap Plan, Piers 4-15
- C7 Pier Cap Plan, Piers 16 & 17
- C8 Pier Cap Details, Pier 2
- C9 Pier Cap Sections, Pier No. 2
- C10 Pier Cap Details, Piers 3-15
- C11 Center Pier Cap Details, Piers 6,7 & 8
- C12 Pier Cap Details, Piers 16 & 17
- C13 Pedestal Details
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- C31 Framing Plan (7)
- C32 Superstructure Plan (1)
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- C38 Superstructure Plan (7)
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- C47 Modified Bulb Tee Beams (1)
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- C50 Modified Bulb-Tee Beams (3)
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- C56 Reinforcing Bar List (4)
- C57 Reinforcing Bar List (5)
- C58 Reinforcing Bar List (6)
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- C63 Reinforcing Bar List (11)
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- C71 Reinforcing Bar List (19)
- C72 Reinforcing Bar List (20)
- C73 Reinforcing Bar List (21)
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720629

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- D13 Pier Plan (7)
- D14 Pedestal Elevations (1)
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- D18 Pier Elevations (3)
- D19 Pier Elevation Variables
- D20 Pier Cap Details (1)
- D21 Pier Cap Details (2)
- D22 Pier Cap Details (3)
- D23 NOT USED
- D24 Pier Cap Details (4)
- D25 Pier Cap Details (5)
- D26 Tendon Profiles
- D27 Pier Cap Details (6)
- D28 Pier Cap Section (1)
- D29 Pier Cap Section (2)
- D30 Pier Cap Section (3)
- D31 Pier Cap Section (4)
- D32 Pier Cap Section (5)
- D33 Pier Cap Section (6)
- D34 Anchor Zone Details
- D35 Pier Column Details (1)
- D36 Pier Column Details (2)
- D37 Pier Column Details (3)
- D38 Pier Column Details (4)
- D39 Pier Footing Details (1)
- D40 Pier Footing Details (2)
- D41 Pier Footing Details (3)
- D42 Pier Footing Details (4)
- D43 Construction Data (1)
- D44 Construction Data (2)
- D45 Construction Data (3)
- D46 Construction Data (4)
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- D48 Construction Data (6)
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- D53 Superstructure Plan (4)
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- D78 Typical Notes and Details for Modified Bulb Tee Beams
- D79 Post Tensioning Notes
- D80 Tendon Profile Spans 28 - 30
- D81 Post-Tensioned MBT Beams, End Segment Details
- D82 Post-Tensioned MBT Beams, Drop-in Segment Details
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- D84 Post-Tensioned MBT Beams, Bending Bar Details
- D85 Erection Sequence
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- E6 Foundation Layout (3)
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- E8 Piers 20S & 21S (Plan & Elevation)
- E9 Piers 22S & 23S (Plan & Elevation)
- E10 Piers 24S & 25S (Plan & Elevation)
- E11 Piers 26S & 27S (Plan & Elevation)
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- E13 Pier 30S (Plan & Elevation)
- E14 Pedestal Details (1)
- E15 Pedestal Details (2)
- E16 Pier Cap Details-Pier 18 (1)
- E17 Pier Cap Details-Pier 18 (2)
- E18 Pier Cap Details-Piers 19 & 29 (1)
- E19 Pier Cap Details-Piers 19 & 29 (2)
- E20 Pier Cap Details-Piers 20 & 28
- E21 Pier Cap Details-Piers 21, 26 & 27
- E22 Pier Cap Details-Piers 22, 23, 24 & 25
- E23 Pier Cap Details-Pier 30 (1)
- E24 Pier Cap Details-Pier 30 (2)
- E25 Anchor Zone Details (1)
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- E28 Pier Cap Sections-P21, P22 & P23
- E29 Pier Cap Sections-P24, P25, P26 & P27
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- E55 Superstructure Plan (Span 21S)
- E56 Superstructure Plan (Span 21N)
- E57 Superstructure Plan (Span 22S)
- E58 Superstructure Plan (Span 22N)
- E59 Superstructure Plan (Span 23S)
- E60 Superstructure Plan (Span 23N)
- E61 Superstructure Plan (Span 24S)
- E62 Superstructure Plan (Span 24N)
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- E64 Superstructure Plan (Span 25N)
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- E68 Superstructure Plan (Span 27N)
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- E118 Reinforcing Bar List (4)
- E119 Reinforcing Bar List (5)
- E120 Reinforcing Bar List (6)
- E121 Reinforcing Bar List (7)
- E122 Reinforcing Bar List (8)
- E123 Reinforcing Bar List (9)
- E124 Reinforcing Bar List (10)

REVISIONS

Date	By	Description	Date	By	Description
5/8/02	JAL	26 Revised Titles and added Sheets A17A & A18A			

	Names	Dates
Drawn by	DCB	10-93
Checked by	SLW	1-94
Designed by	RDS	10-93
Checked by	SLW	1-94
Approved by	D. W. Browning	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

SHEET TITLE: INDEX OF SHEETS (1)

PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

Drawing No.

Index No.

NAME: G:\JOBS\1841\...\CHPROD\FENDER REVISION\BRD01B.DWG DATE: MAY 8, 2002 TIME: 4:00 PM REV. BY: JAL

BBCI:

1. Construct end bent drill shafts in two stages, first stage construct drill shafts to natural ground using approved construction methods, second stage extend to grade rebar cages using mechanical splices, form and four drill shafts to grade.

2. Cut drill shafts vertical rebar approximately 6 each (3 each on opposite sides) for the removal of zipper casing (when the casing is below the top of the rebar cage). Extend rebar cage to grade with mechanical splices after casing removal.

HNTB -

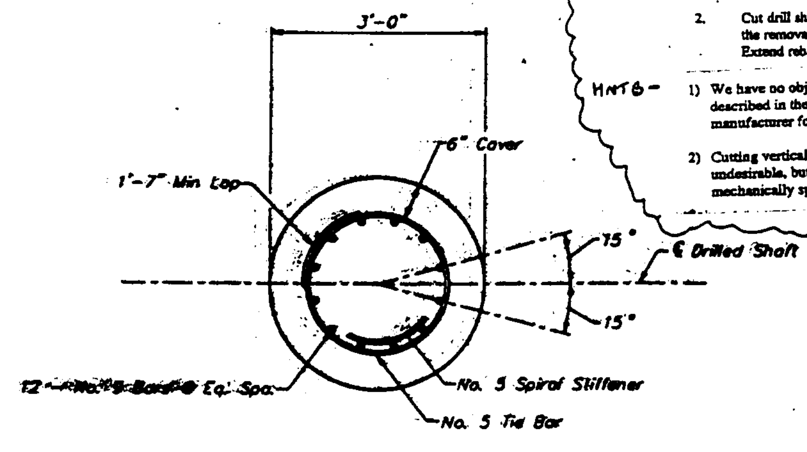
1) We have no objection to BBCI constructing the end bent drill shafts in two stages as described in their 1/31/97 letter to you. We suggest that design data from the manufacturer for the mechanical splice be provided to PBCS for review.

2) Cutting vertical bars in the drilled shafts in order to remove the zipper casing is undesirable, but we have no objection to doing it provided the cut reinforcing is mechanically spliced and extended to the planned length after the casing is removed.

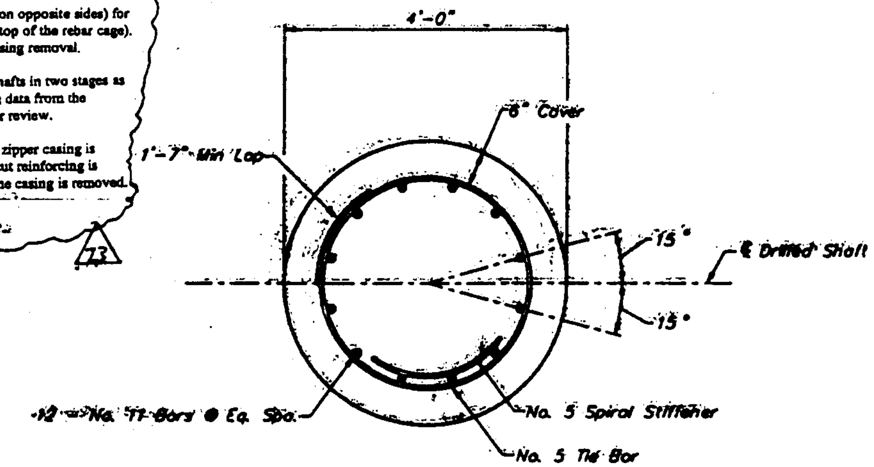
Miscellaneous Changes

23

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.		95	A24



SECTION A-A



SECTION B-B

NOTES

1. Reinforcing Steel shall be Grade 60.
2. Concrete for Drilled Shafts shall be Class IV (Drilled Shaft), f'c = 4,000 p.s.i.
3. Drilled Shafts shall be paid for at the Contract Unit Price per linear foot for Drilled Shafts under Item No. 455-88-3 (36"ø), 455-88-5 (48"ø), or 455-88-7 (72"ø). See Summary of Quantities and Specifications.
4. The Contractor shall submit plans, showing method of Constructing the Drilled Shaft Foundations, to the Engineer for Approval.
5. For Shaft Elevations, see Drilled Shaft Installation Data on the Foundation Layout Sheet.
6. Bar Extensions for Embedment into Footing are to be included in the cost of Drilled Shaft Item, but are not included in Quantities Per Lin. Ft. shown below.

36"ø DRILLED SHAFT QUANTITIES PER LIN. FT.

Class IV Concrete = 0.26 Cu Yds./Lin. Ft.
Reinforcing Steel = 58.2 Lbs./Lin. Ft. for top 7 ft.
51.2 Lbs./Lin. Ft. for Remainder of shaft

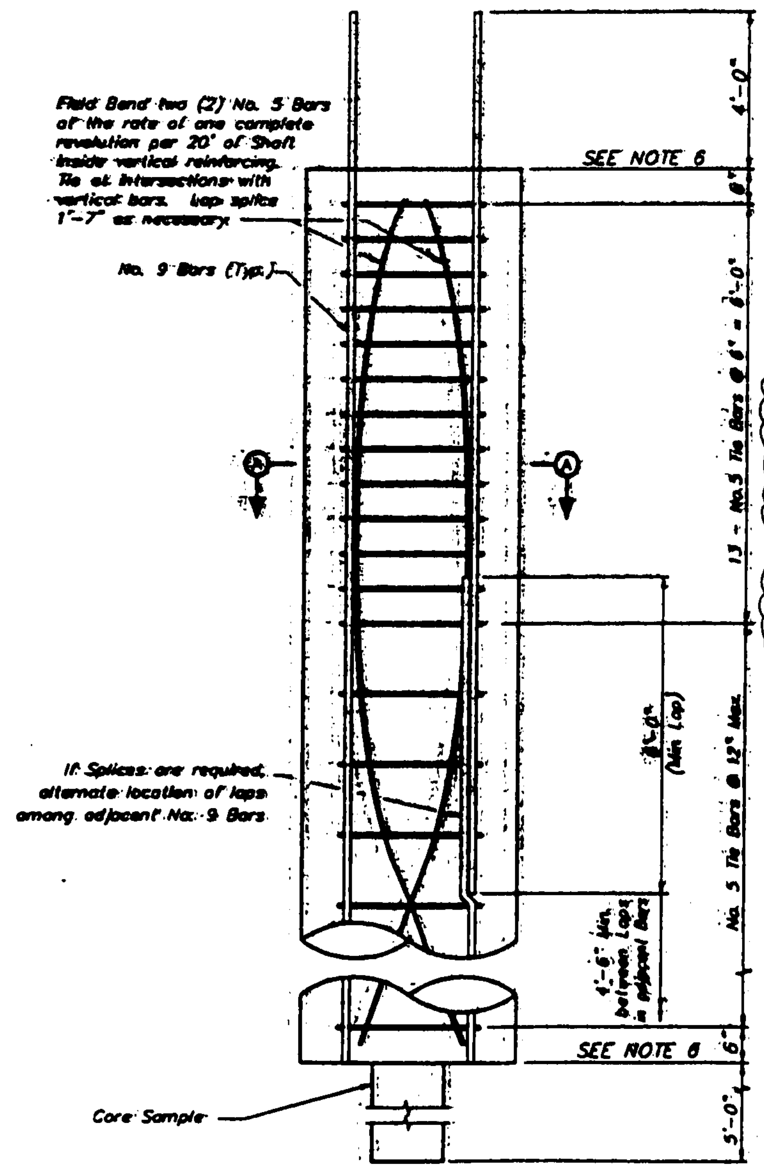
48"ø DRILLED SHAFT QUANTITIES PER LIN. FT.

Class IV Concrete = 0.47 Cu.Yds./Lin. Ft.
Reinforcing Steel = 87.2 Lbs./Lin. Ft. for top 7 ft.
77.4 Lbs./Lin. Ft. for remainder of shaft

72"ø DRILLED SHAFT QUANTITIES PER LIN. FT.

Class IV Concrete = 1.05 Cu.Yds./Lin. Ft.
Reinforcing Steel = 184.4 Lbs./Lin. Ft. for top 7 ft.
(Type A) 169.0 Lbs./Lin. Ft. for remainder of shaft

Reinforcing Steel = 311.4 Lbs./Lin. Ft. for top 7 ft.
(Type B) 296.0 Lbs./Lin. Ft. for remainder of shaft



ELEVATION - 36" DIAMETER DRILLED SHAFT

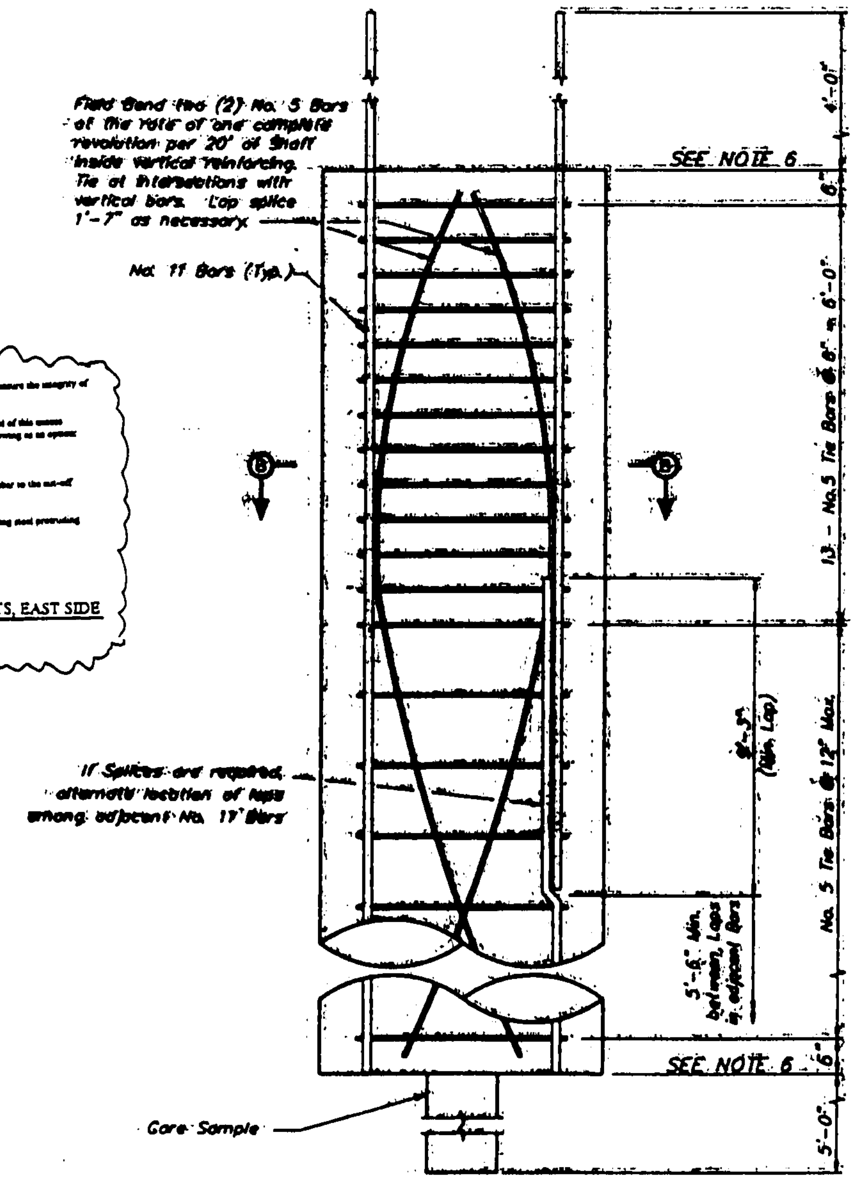
The 36 inch drilled shafts on the east side of the river were poured high to ensure the integrity of the concrete at the cut-off elevation of the shaft.

In an attempt to minimize potential damage to the shafts during the removal of this excess concrete, Balfour Beatty Construction, Inc. would like to propose the following as an option:

1. Cut the shaft off 1.0 feet above the cut-off elevation.
2. Remove and clean out the remaining concrete around the rebar to the cut-off elevation.
3. Mechanically splice a 3.0 feet piece of rebar to the reinforcing steel protruding from the drilled shaft.
4. Form the footing as originally planned.

REINFORCING STEEL - 36" DRILLED SHAFTS, EAST SIDE

23



ELEVATION - 48" DIAMETER DRILLED SHAFT

To repair small underwater concrete spalls on the 72 inch drilled shafts in the river Balfour Beatty Construction, Inc. proposes the following procedure:

1. Throughly clean the affected area with high pressure air or water.
2. Chip and remove any loose material to achieve a clean rough surface.
3. Place either the product "Plug-Crete" or "Dam-It" into the afftected area (literature and product data attached). The material has a fast set time and will be hand placed by a diver.
4. These products require no special curing procedures.

23

The 36 and 48 inch diameter drilled shafts on land were poured to a height above the cut off elevation to insure the integrity of the shaft concrete.

Balfour Beatty Construction, Inc. proposes to use the same method that is currently being utilized on the 36 inch drilled shafts located on bridge #2 for the remaining 36 inch and 48 inch drilled shafts located on land. This method is as follows:

1. Cut the shaft off one foot above the cut-off elevation.
2. Remove and clean out the remaining concrete around the reinforcing steel to the cut-off elevation.
3. Mechanically splice a three foot piece of rebar to the top of the reinforcing steel that extends out of the drilled shaft using an approved mechanical coupling device.
4. Form and pour the footing as originally planned.

REINFORCING STEEL - LAND DRILLED SHAFTS

23

NAME: M:\16414\GENERAL\BRD298.DWG DATE: AUG 31, 1995 TIME: 6:58 AM REV. BY: RLG

REVISIONS					
Date	By	Description	Date	By	Description
7-15-01	SH	23 AS-BUILT			

	Names	Dates
Drawn by	LJS	1-94
Checked by	DMW	5-95
Designed by	HDR	1-94
Checked by	DMM	2-94
Approved by	D. K. Browning	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

SHEET TITLE: DRILLED SHAFT DETAILS (1)

PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

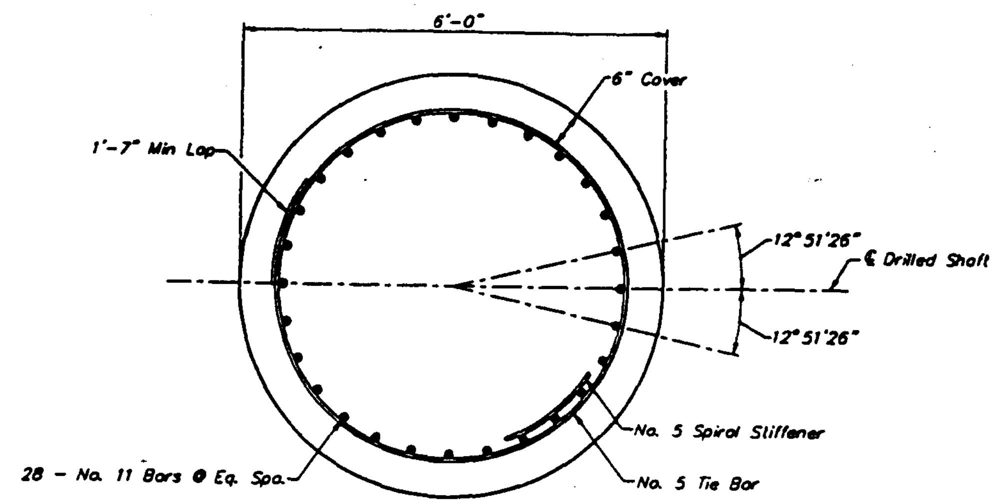
Drawing No.

Index No.

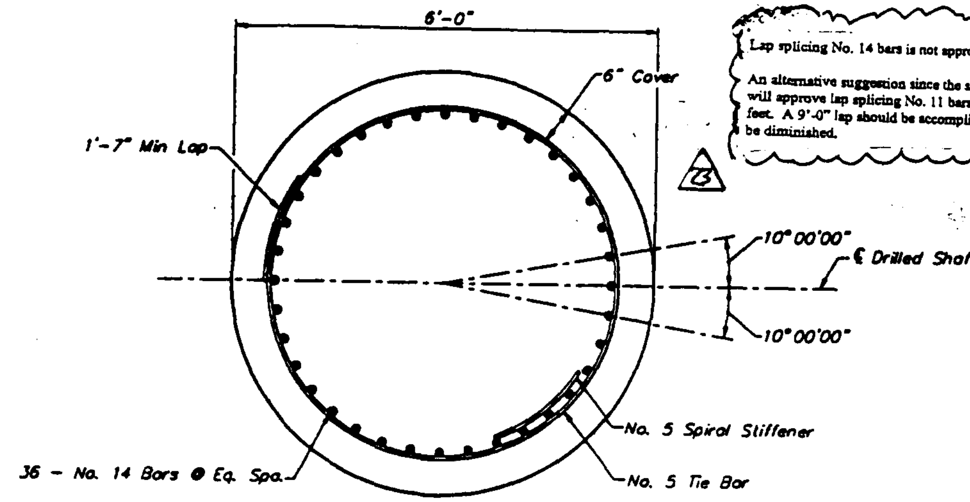
Fed. Road Div. No.	State	Project No.	Fiscal Year	Sheet No.
3	FLA.		95	A25

Lap splicing No. 14 bars is not approved.

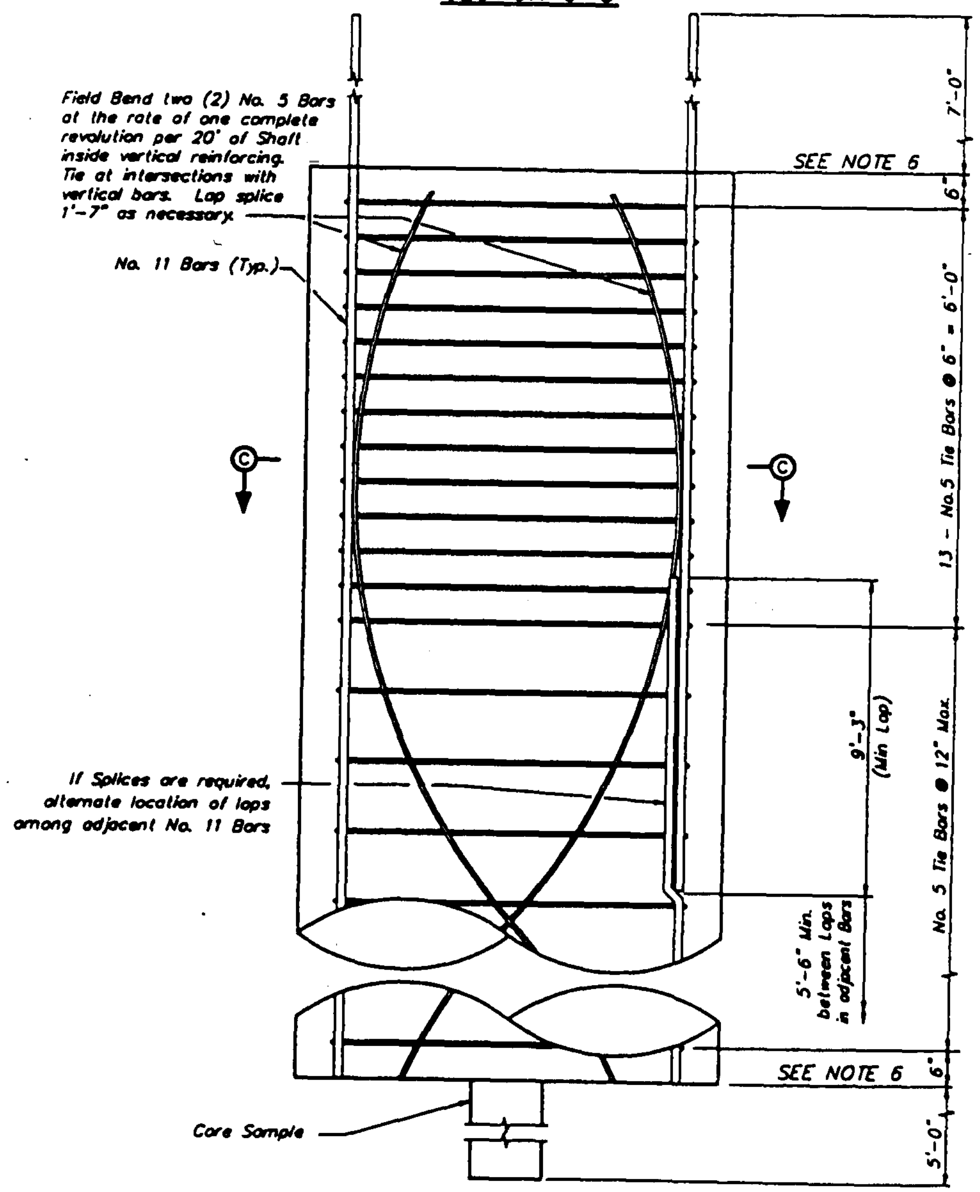
An alternative suggestion since the shaft is only six feet longer than it's original length we will approve lap splicing No. 11 bars to the bottom of the cage to extend it the additional six feet. A 9'-0" lap should be accomplished to the inside so clear distance between bars will not be diminished.



SECTION C-C

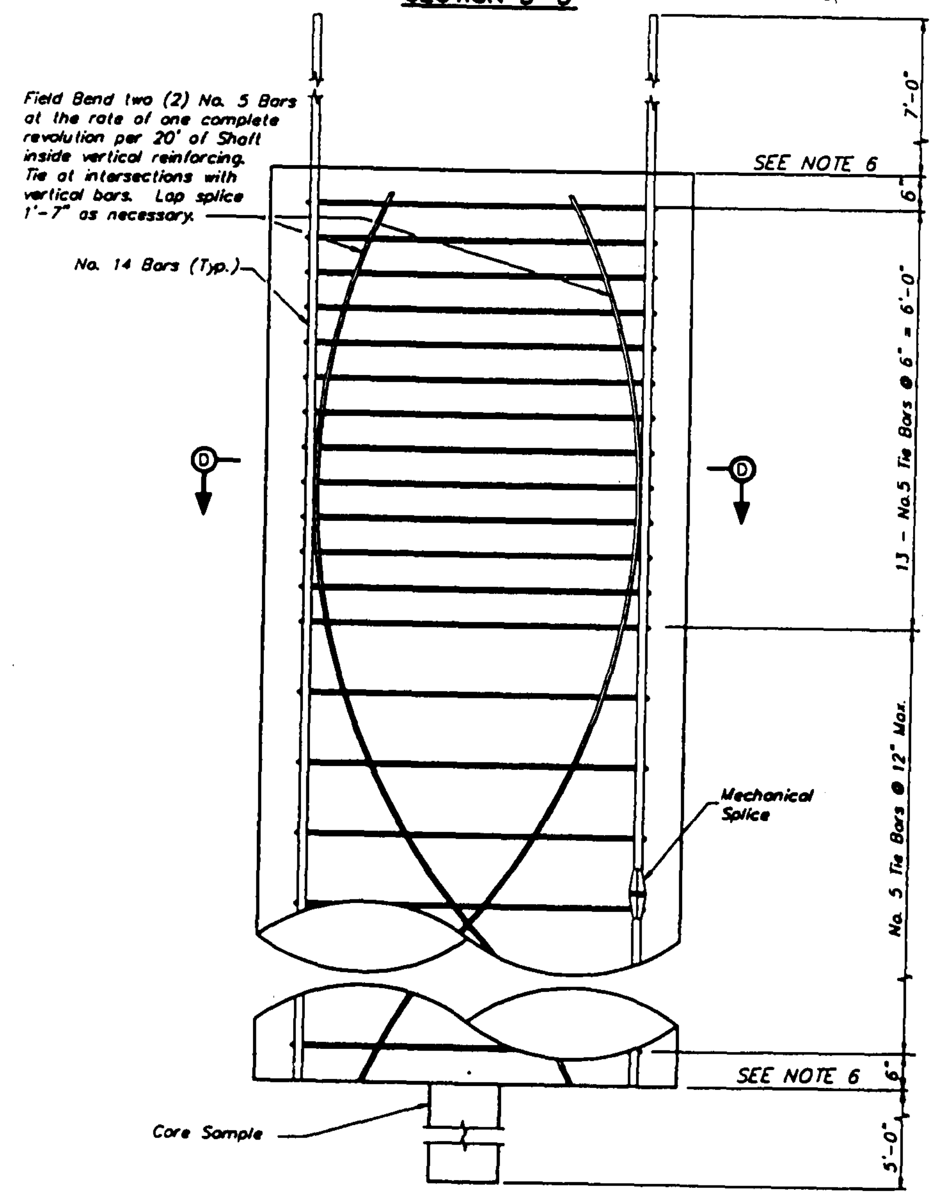


SECTION D-D



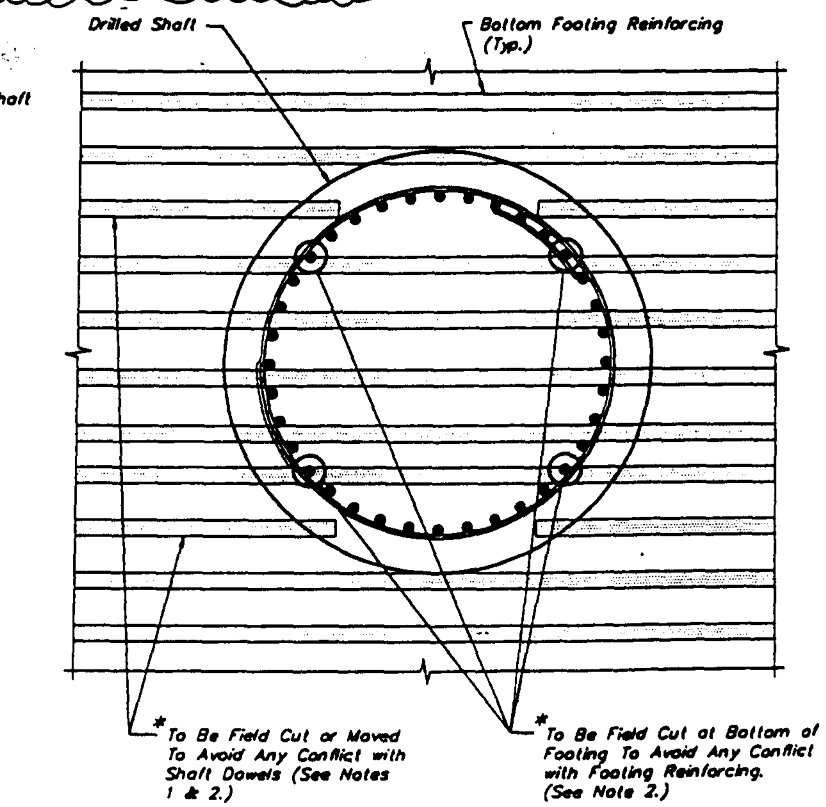
ELEVATION - 72" DIAMETER DRILLED SHAFT

Type A



ELEVATION - 72" DIAMETER DRILLED SHAFT

Type B



PLAN

(TYPICAL SHAFT/FOOTING REINFORCING INTERACTION)

Below procedure Note 3 used at following locations
Pier 28 - Shaft 9
Pier 30 Shaft 1, 2, 16

NOTES

(1.) The Contractor Shall Adjust Footing Reinforcement where possible to avoid conflicts with the Drilled Shaft Reinforcing. Minimum Bar Spacing shall be three (3) times Bar Diameter and Shall Not Exceed Reinforcing Spacing Shown on Pier Footing Reinforcing by 1".

(2.) The Contractor Shall Be Permitted to Field Cut Footing or Drilled Shaft Bars only as indicated on This Sheet. No Adjacent Footing Bars or Drilled Shaft Bars may be cut. No more than four (4) Drilled Shaft Bars may be cut in any given shaft.

* To Be Field Cut and only if there is an Unavoidable Conflict Between Shaft and Footing Reinforcing and Field Adjustment Is Not An Option.

For Additional Notes see Sheet A24.

(3.) Drilled shaft Repair Procedure - Approved March 23, 1998
Bridge No. 3
(1) Several of the drilled shafts appear to have irregular surfaces
(a) Install the shaft cut-off box over the shaft to be repaired.
(b) Seal and dewater the box to an elevation lower than affected area
(c) Chip and clean the affected area to obtain flat even surface
(d) Place "stay form" expanded metal around shaft to repair.
(e) Place class IV drilled shaft concrete - dewater for 24 hours
(f) Remove the box and continue soffit slab [illegible]

REVISIONS					
Date	By	Description	Date	By	Description
7-15-01	SH	23 AS-BUILT			

	Names	Dates
Drawn by	LJS	1-94
Checked by	DMW	5-95
Designed by	HDR	1-94
Checked by	DMM	2-94
Approved by	D. W. Browning	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

Road No.	County	Project No.
9	DUVAL	72020-3485

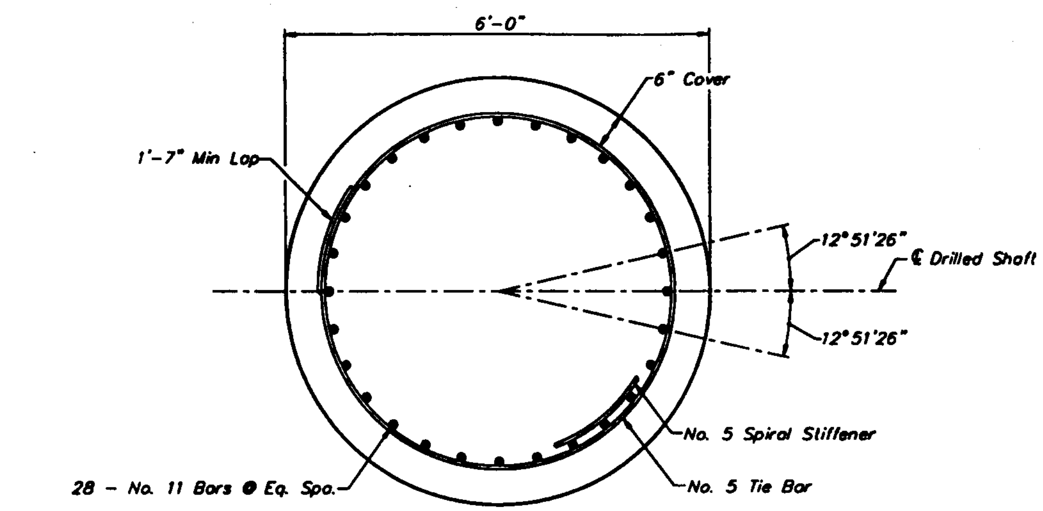
Sheet Title: DRILLED SHAFT DETAILS (2)

Project Name: I-95 OVER THE ST. JOHNS RIVER

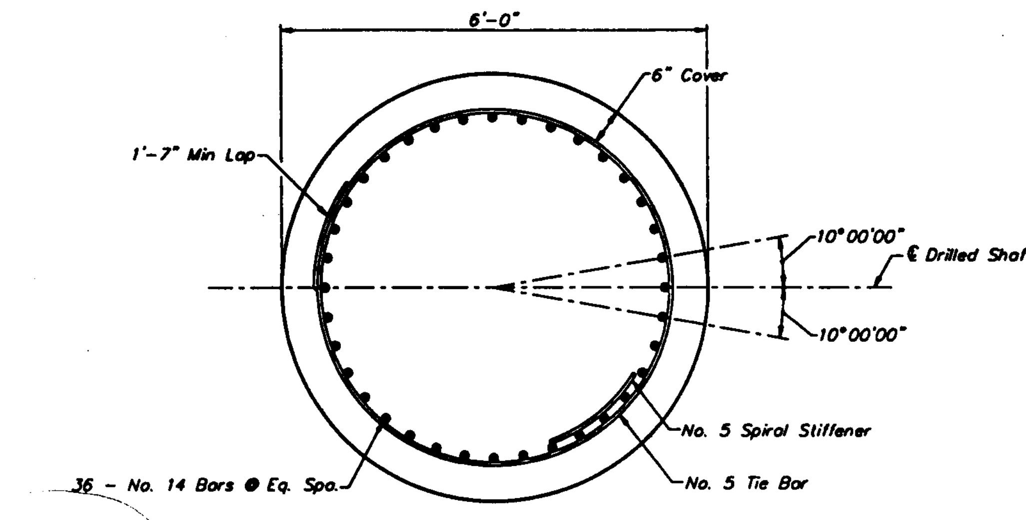
Index No.

NAME: M:\18414\GENERAL\...029C.DWG DATE: AUG 31, 1995 TIME: 7:50 AM REV. BY: RLG

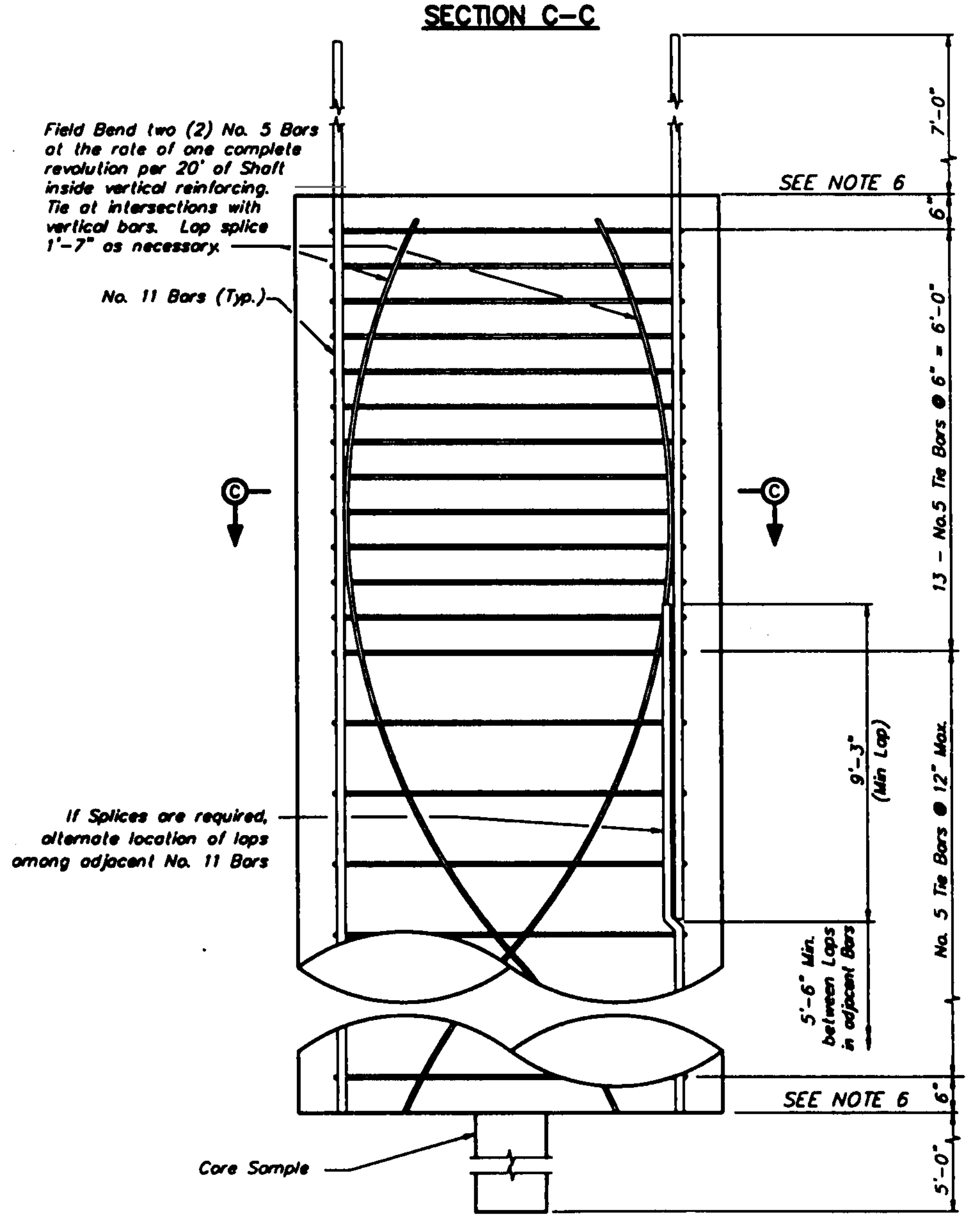
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.		95	A25



SECTION C-C

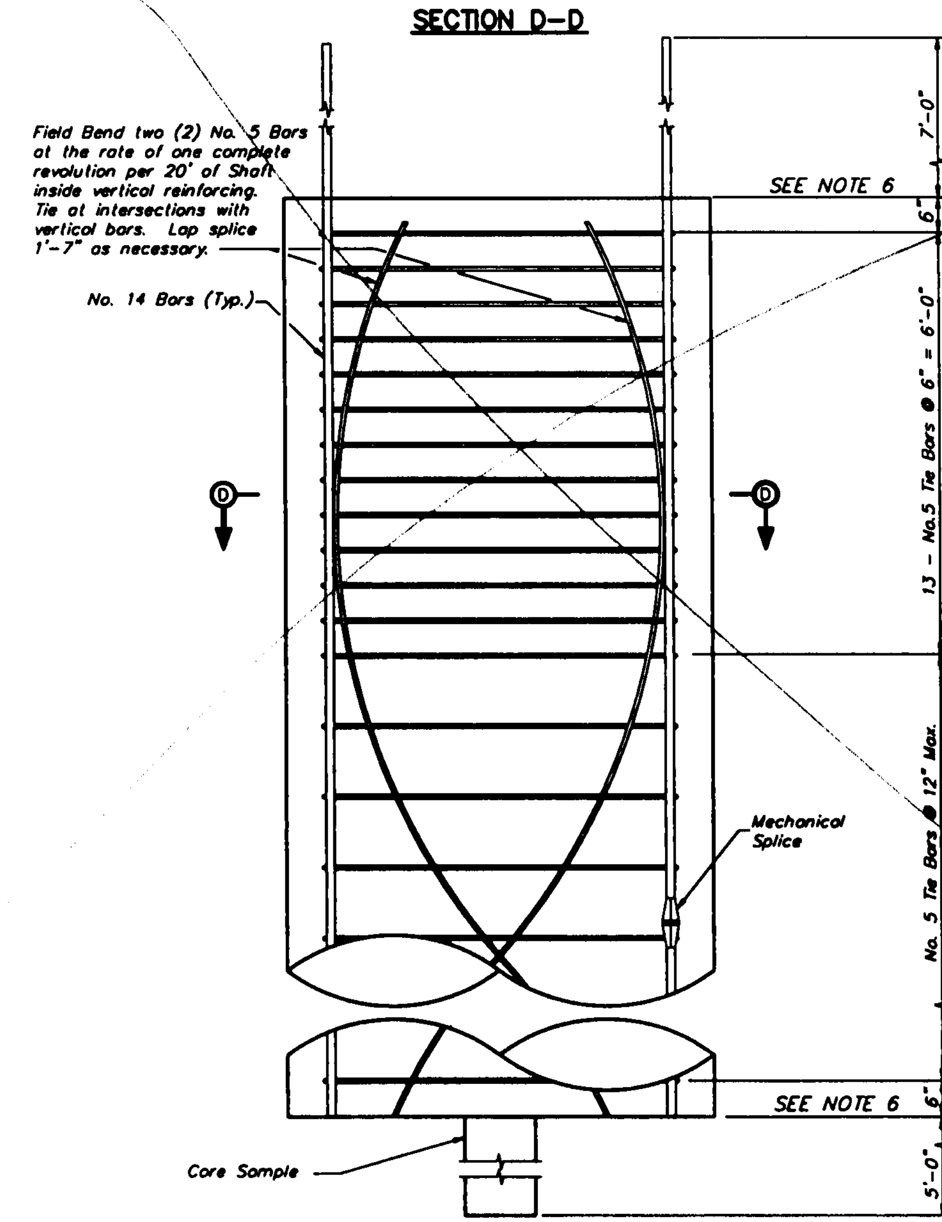


SECTION D-D



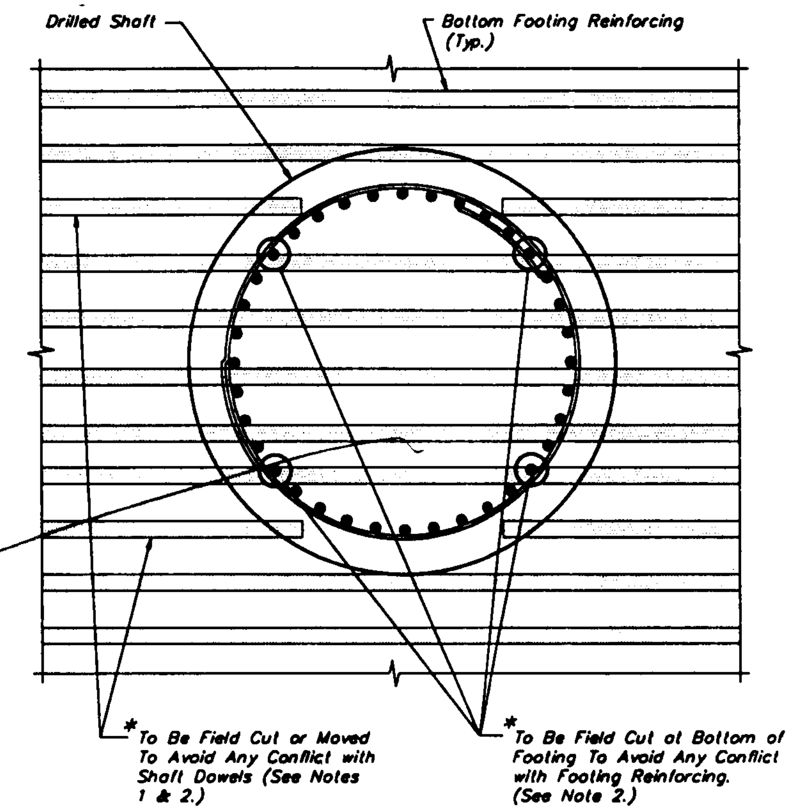
ELEVATION - 72" DIAMETER DRILLED SHAFT

Type A



ELEVATION - 72" DIAMETER DRILLED SHAFT

Type B



PLAN

(TYPICAL SHAFT/FOOTING REINFORCING INTERACTION)

NOTES

(1.) The Contractor Shall Adjust Footing Reinforcement where possible to avoid conflicts with the Drilled Shaft Reinforcing. Minimum Bar Spacing shall be three (3) times Bar Diameter and Shall Not Exceed Reinforcing Spacing Shown on Pier Footing Reinforcing by 1".

(2.) The Contractor Shall Be Permitted to Field Cut Footing or Drilled Shaft Bars only as indicated on This Sheet. No Adjacent Footing Bars or Drilled Shaft Bars may be cut. No more than four (4) Drilled Shaft Bars may be cut in any given shaft.

* To Be Field Cut and only if there is an Unavoidable Conflict Between Shaft and Footing Reinforcing and Field Adjustment Is Not An Option.

For Additional Notes see Sheet A24.

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	LJS	1-94
Checked by	DMW	5-95
Designed by	HDR	1-94
Checked by	DMM	2-94
Approved by	D. W. Browning	

HNTB

HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION

STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

SHEET TITLE: DRILLED SHAFT DETAILS (2)

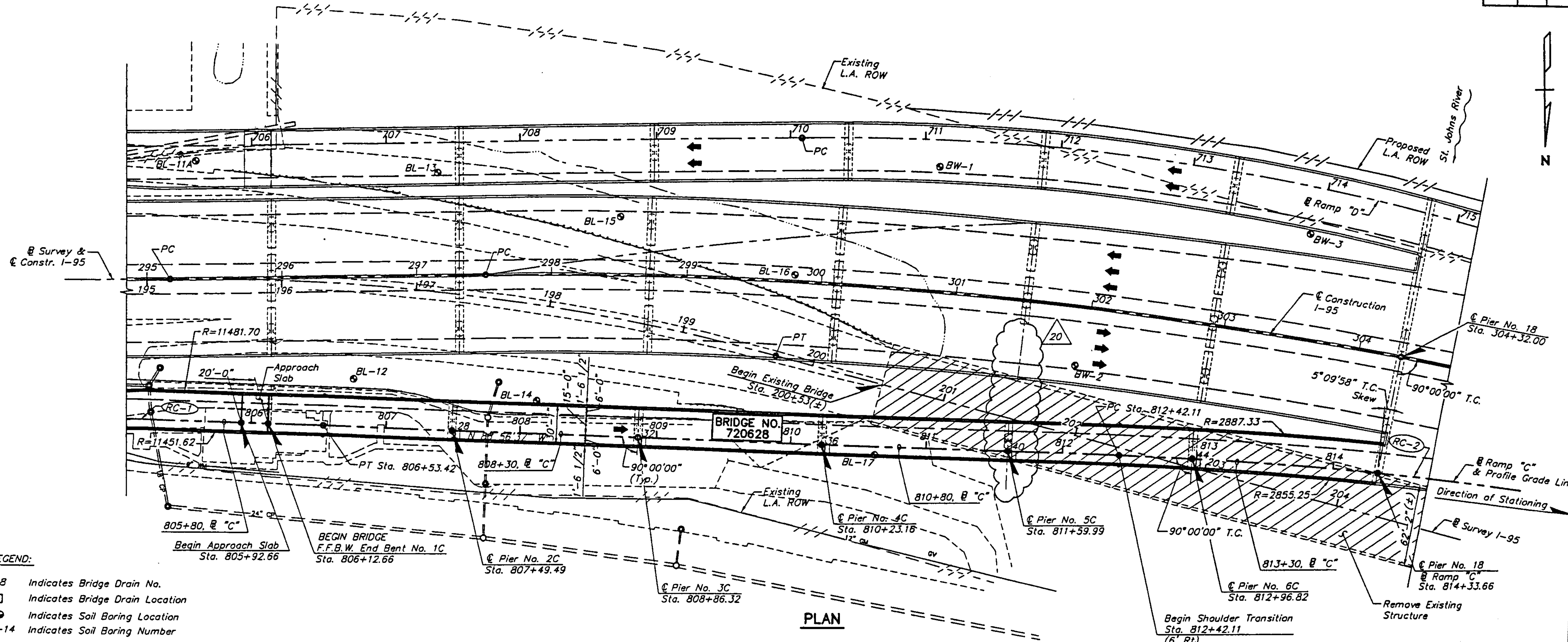
PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

Drawing No.

Index No.

NAME: ...3414\CE...\BRD29C.DWG DATE: AUG 31, 1995 TIME: 7:50 AM REV. BY: RLG

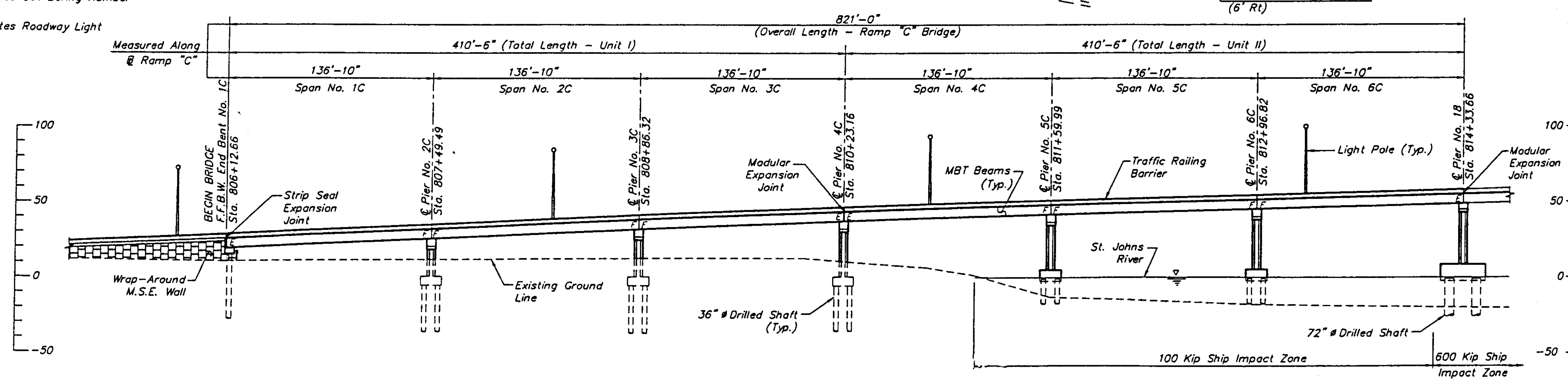
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.		95	G1



PLAN

LEGEND:

28 Indicates Bridge Drain No.
Indicates Bridge Drain Location
Indicates Soil Boring Location
BL-14 Indicates Soil Boring Number
Indicates Roadway Light



ELEVATION

(Main Line Bridge, Ramp "D" Bridge, and Existing Bridge not shown for Clarity)

NOTES:

See Sheet A4 & A5 for Structural Notes

See Sheets A6 & A7 for Horizontal & Vertical Curve Data and Traffic Data.

See Sheet A7 for St. Johns River Elevations.

Dimensions shown on Plan View (Bridge Width,etc.) are Measured radial/normal to the associated @/@.

For Shoulder Transition see Roadway Plans.

See General Sheets for Bridge Drain Locations and Drainage Details.

[redacted]

BRIDGE NO. 720628

REVISIONS

Date	By	Description	Date	By	Description
5/15/00	MDC	20 TEMPORARY PIERS REMOVED			

	Names	Dates
Drawn by	BDV/CSD	8-93
Checked by	LDP	11-93
Designed by	MJL/HDR	9-93
Checked by	DMM	12-93
Approved by	S.M. JARRETT	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL: M. Caulfield 5-15-00

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020 – 3485

SHEET TITLE: PLAN AND ELEVATION RAMP "C" BRIDGE

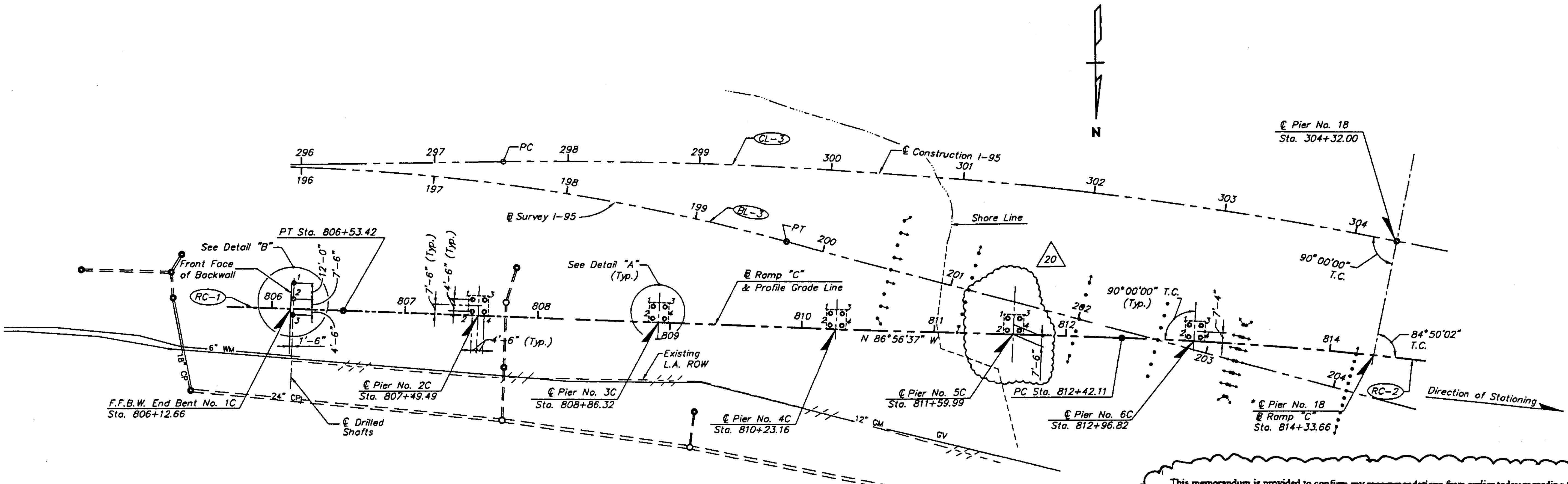
PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

Drawing No.

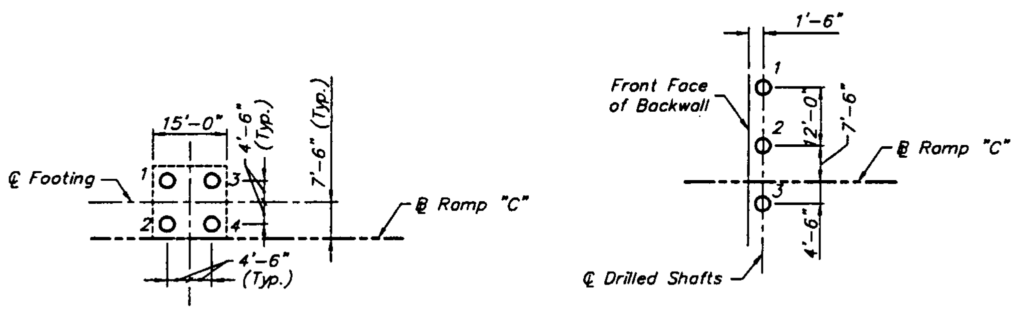
Index No.

NAME: M:\16414\REVISION 20\DWG-G01.DWG DATE: MAY 3, 2000 TIME: 1:56 PM REV. BY: RLG

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.		95	G2



PLAN



DETAIL "A"

DETAIL "B"

To:	Pom Chakkaphak, P.E. PBC
From:	Ray Castelli, P.E.
Date:	March 9, 2001
Subject:	Fuller Warren Bridge 36-inch Diameter Drilled Shafts Pier 6C, Shaft No. 2

This memorandum is provided to confirm my recommendations from earlier today regarding Shaft No. 2 at Pier 6C. As reported by Chuck Iley, the initial installation of this shaft could not be completed because the drilling tools could advance no closer than 5 feet to the casing tip due to suspected deformation of the casing. At the time this problem was encountered, the casing tip was at elevation -61.02.

Considering the inability to advance the drill tools to the bottom of the casing, it will be necessary to backfill the excavation, extract the existing casing and replace it with another casing. Since it is very likely that the replacement casing will be slightly offset from the original casing alignment, we must assume a reduced friction resistance within the depth of the original casing. To compensate for this reduced friction, it is recommended that the shaft penetration be increased an additional 7 feet to a revised tip elevation at -68. With the exception of the revised shaft tip elevation, the procedures for installation of this shaft should be the same as those for the original shaft, and the final casing tip elevation -42 remains unchanged.

Please contact me if you have any questions regarding the above recommendations, or if further problems are encountered at this shaft.

23

ACTUAL AS-BUILT SEE SHEET No. G2A

DRILLED SHAFT INSTALLATION TABLE

END BENT or PIER NO.	SHAFT DIAMETER (INCHES)	DESIGN LOAD (TONS)	MINIMUM TIP ELEVATION (FT)	100 YEAR SCOUR ELEVATION (FT)
1C	36	250	−35	
2C	36	400	−35	
3C	36	400	−35	
4C	36	400	−35	
5C	36	450	−48	−36
6C	36	450	−56	−42
18	*	*	*	*

20

* For Foundation Layout and Drilled Shaft Installation Data for Pier No. 18, See Sheet D4 or E4.

ACTUAL AS-BUILT SEE SHEET No. G2A

TABLE OF DRILLED SHAFT TOP ELEVATIONS

SHAFT NO.	END BENT NO. 1C	PIER NO. 2C THRU PIER 4C	PIER NO. 5C	PIER NO. 6C
1.	14.15	−4.25	−0.83	−0.83
2.	13.91	−4.25	−0.83	−0.83
3.	13.68	−4.25	−0.83	−0.83
4.	−	−4.25	−0.83	−0.83

20

LEGEND

- O Proposed Drilled Shaft
- Existing Jacketed Steel Pile with Batter Direction

NOTES

1. For Horizontal Curve Data see Sheet A7.
2. Plan location of Drilled Shafts is given at Bottom of Caps.
3. All Drilled Shafts at the End Bent shall be installed Prior to the Installation of MSE Walls.
4. Drilled Shafts shall be Installed to the Minimum Tip Elevations shown in the Drilled Shaft Installation Table.
5. For Drilled Shaft Reinf. Details and associated notes see Sheet A24.

20

6. The portion of End Bent drilled shafts exposed above the existing ground shall be wrapped with 6 mil. thickness polyethylene plastic film. See General Notes, Sheet A4.
7. The Foundations shall be constructed in the Phases outlined in the Sequence of Construction drawings.

REVISIONS

Date	By	Description	Date	By	Description
5/15/00	MDC	20 TEMPORARY PIERS REMOVED			
6/4/01	SH	23 AS-BUILT			

	Names	Dates
Drawn by	SCG/CSD	10-93
Checked by	TQT	7-94
Designed by	TQT/RMS	7-94
Checked by	LDP/TQT	7-94

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL M. Castelli 5-15-00

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020 − 3485

SHEET TITLE: FOUNDATION LAYOUT RAMP "C" BRIDGE

PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

Drawing No.

Index No.

DWG DATE: MAY 10, 2000 TIME: 12:19 PM REV. BY: RLG

NAME: M:\16414\RAMPC\

3	FLA		95	G2	A

AS-BUILT

DRILLED SHAFT PAY QUANTITIES - RAMP "C" BRIDGE

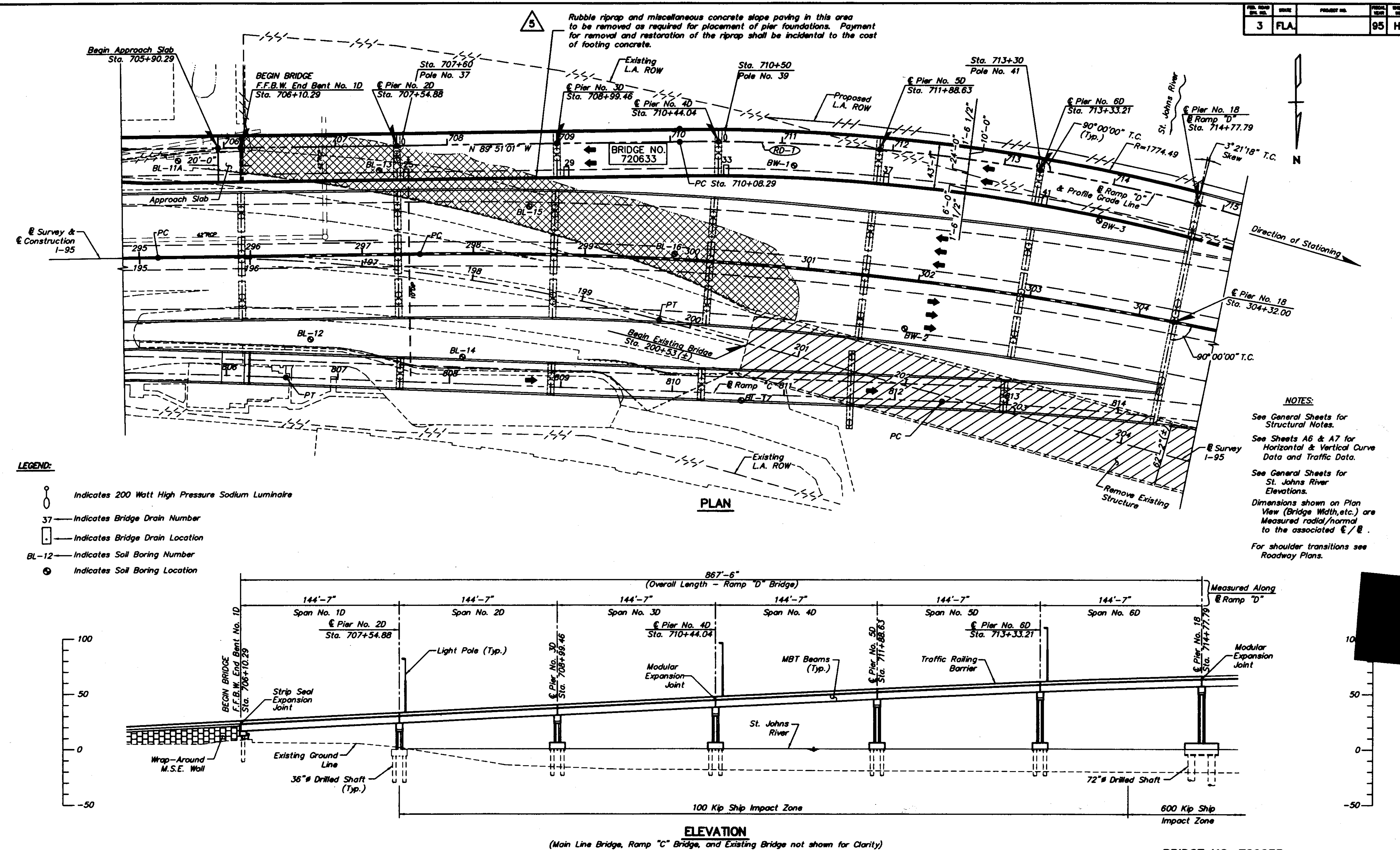
		DESIGN CONDITIONS								AS-BUILT CONDITIONS							
PIER NO.	SHAFT NO.	SHAFT SIZE	AXIAL LOAD	100-YR SCOUR EL.	CUT-OFF ELEV.	PLAN TIP ELEV.	AUTH TI ELEV.	EXTRA DEPTH	SHAFT LENGTH	FINISH DATE	GROUND ELEV.	AUTH TI ELEV.	ACTU TI ELEV.	UNCLAS EXCAV.	EXTRA DEPTH	SHAFT LENGTH	COMMENTS
		in	tons	ft	ft	ft	ft	ft	ft		ft	ft	ft	ft	ft	ft	
1C	1	36	250	N/A	14.15	-35.0	-36.0	1.0	50.15	4/17/00	11	-36	-36.43	46.00	1.00	50.15	
1C	2	36	250	N/A	13.91	-35.0	-35.0	0.0	48.91	5/2/00	11	-41	-41.26	46.00	6.00	54.91	
1C	3	36	250	N/A	13.68	-35.0	-35.0	0.0	48.68	5/4/00	11	-35	-35.08	46.00	0.00	48.68	
2C	1	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	4/12/00	-4.75	-39	-39.27	30.25	4.00	34.75	
2C	2	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	4/17/00	-4.75	-39	-39.18	30.25	4.00	34.75	
2C	3	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	4/13/00	-4.75	-39	-39.00	30.25	4.00	34.75	
2C	4	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	4/11/00	-4.75	-39	-42.18	30.25	4.00	34.75	
3C	1	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	5/16/00	-4.75	-39	-48.16	30.25	4.00	34.75	
3C	2	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	5/18/00	-4.75	-39	-39.36	30.25	4.00	34.75	
3C	3	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	5/15/00	-4.75	-39	-39.1	30.25	4.00	34.75	
3C	4	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	5/19/00	-4.75	-39	-39.02	30.25	4.00	34.75	
4C	1	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	5/15/00	-4.75	-39	-40.4	30.25	4.00	34.75	
4C	2	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	5/16/00	-4.75	-39	-35.59	30.25	4.00	34.75	
4C	3	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	5/17/00	-4.75	-39	-39.02	30.25	4.00	34.75	
4C	4	36	400	N/A	-4.25	-35.0	-39.0	4.0	34.75	5/18/00	-4.75	-39	-39.15	30.25	4.00	34.75	
5C	1	36	250	-36	-0.83	-41.0	-43.0	2.0	42.17	3/22/01	-6.25	-64.00	-68.68	34.75	23.00	63.17	By Ray
5C	2	36	250	-36	-0.83	-41.0	-43.0	2.0	42.17	4/2/01	-6.25	-64.00	-64.63	34.75	23.00	63.17	By Ray
5C	3	36	250	-36	-0.83	-41.0	-43.0	2.0	42.17	3/24/01	-6.25	-64.00	-64.07	34.75	23.00	63.17	By Ray
5C	4	36	250	-36	-0.83	-41.0	-43.0	2.0	42.17	4/5/01	-6.25	-64.00	-65.25	34.75	23.00	63.17	By Ray
5C	5	36	450	-36	-0.83	-48.0	-48.0	0.0	47.17			-56.00					Deleted
5C	6	36	450	-36	-0.83	-48.0	-48.0	0.0	47.17			-56.00					Deleted
5C	7	36	450	-36	-0.83	-48.0	-48.0	0.0	47.17			-56.00					Deleted
5C	8	36	450	-36	-0.83	-48.0	-48.0	0.0	47.17			-56.00					Deleted
5C	9	36	250	-36	-0.83	-41.0	-43.0	2.0	42.17			-50.00					Deleted
5C	10	36	250	-36	-0.83	-41.0	-43.0	2.0	42.17			-50.00					Deleted
5C	11	36	250	-36	-0.83	-41.0	-43.0	2.0	42.17			-50.00					Deleted
5C	12	36	250	-36	-0.83	-41.0	-43.0	2.0	42.17			-50.00					Deleted
6C	1	36	450	-42	-0.83	-56.0	-61.0	5.0	60.17	3/2/01	-18.5	-61.0	-61.01	37.50	5.00	60.17	
6C	2	36	450	-42	-0.83	-56.0	-61.0	5.0	60.17	3/8/01	-18.5	-68.0	-68.82	37.50	12.00	67.17	By Ray
6C	3	36	450	-42	-0.83	-56.0	-61.0	5.0	60.17	3/7/01	-18.5	-61.0	-61.14	37.50	5.00	60.17	
6C	4	36	450	-42	-0.83	-56.0	-61.0	5.0	60.17	3/13/01	-18.5	-61.0	-61.27	37.50	5.00	60.17	
TOTAL	31							85.0	1331.5 1246.5					790.00	174.00	1071.10	

23

REVISIONS

Date	By	Description	Date	By	Description
7-21-01	SH	23 AS-BUILT			

SHEET TITLE: FOUNDATION LAYOUT RAMP "C" BRIDGE	Drawing No.
PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER	Index No.



FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.		95	H1

BRIDGE NO. 720633

REVISIONS

Date	By	Description	Date	By	Description
2/3/95	DMW	5 Added note identifying rubble riprap.			

	Names	Dates
Drawn by	SPJr	8-93
Checked by	PLS	11-93
Designed by	MJL/HDR	9-93
Checked by	DMM	12-93
Approved by	M.D. Caulfield	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

SHEET TITLE: PLAN AND ELEVATION RAMP "D" BRIDGE

PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

Drawing No.

Index No.

NAME: M:\1641\RAMP...J2A.DWG DATE: FEB 07, 1996 TIME: 1:03 PM REV. BY: RLG

HIA

FULLER WARREN BRIDGE

DRILLED SHAFT PAY QUANTITIES - RAMP "D" BRIDGE

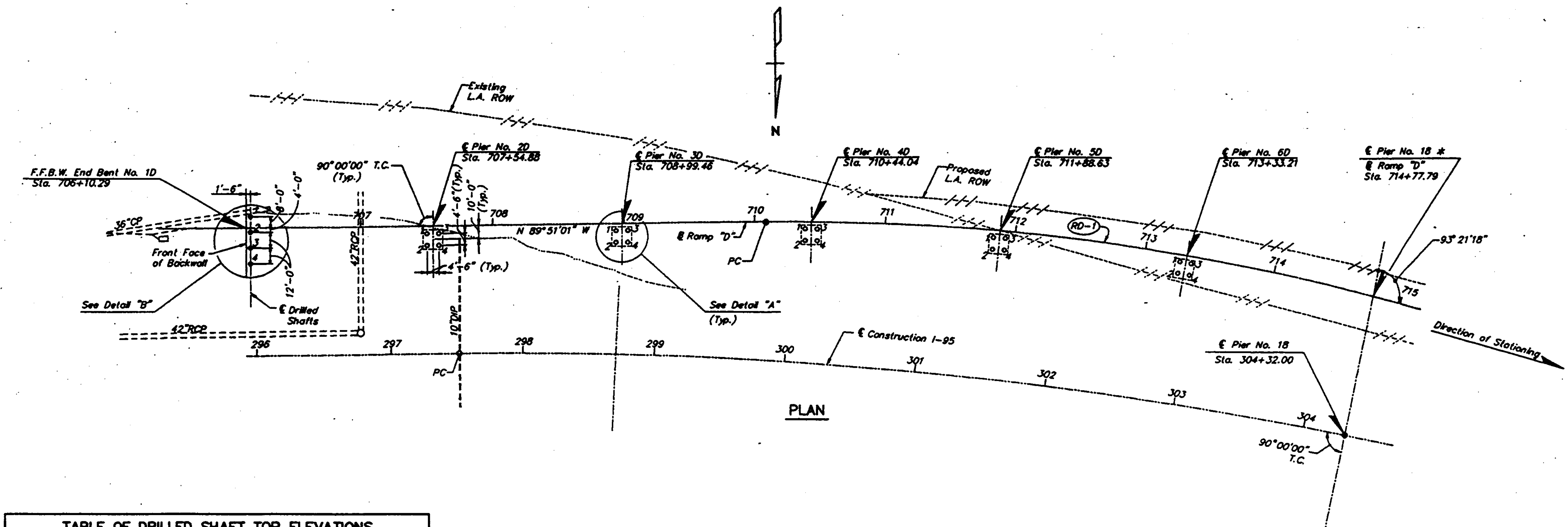
		DESIGN CONDITIONS								AS-BUILT CONDITIONS							
PIER NO.	SHAFT NO.	SHAFT SIZE	AXIAL LOAD	100-YR SCOUR EL.	CUT-OFF ELEV.	PLAN TIP ELEV.	AUTH TIP ELEV.	EXTRA DEPTH	SHAFT LENGTH	FINISH DATE	GROUND ELEV.	AUTH TIP ELEV.	ACTU TIP ELEV.	UNCLAS EXCAV.	EXTRA DEPTH	SHAFT LENGTH	COMMENTS
		in	tons	ft	ft	ft	ft	ft	ft		ft	ft	ft	ft	ft	ft	
EB1D	1	36	450	N/A	14.302	-35	-35	0	49.302	12/5/96	3.87	-35.00	-36.23	38.87	0.00	49.30	
EB1D	2	36	450	N/A	14.062	-35	-35	0	49.062	3/7/97	10.00	-36.00	-36.10	45.00	1.00	50.06	Extra 1' per RJC
EB1D	3	36	450	N/A	13.822	-35	-35	0	48.822	3/10/97	10.00	-39.00	-39.00	45.00	4.00	52.82	Extra 4' per RJC
EB1D	4	36	450	N/A	13.582	-35	-35	0	48.582	3/11/97	10.00	-35.00	-35.00	45.00	0.00	48.58	
2D	1	36	450	N/A	-4.25	-35	-42	7	37.75	11/22/96	-4.75	-42.00	-42.40	30.25	7.00	37.75	
2D	2	36	450	N/A	-4.25	-35	-42	7	37.75	11/26/96	-4.75	-42.00	-42.80	30.25	7.00	37.75	
2D	3	36	450	N/A	-4.25	-35	-42	7	37.75	11/21/96	-4.75	-42.00	-42.10	30.25	7.00	37.75	
2D	4	36	450	N/A	-4.25	-35	-42	7	37.75	12/3/96	-4.75	-42.00	-43.40	30.25	7.00	37.75	
3D	1	36	550	N/A	-0.83	-37	-53	16	52.17	4/12/99	-4.00	-53.00	-71.00	33.00	16.00	52.17	
3D	2	36	550	N/A	-0.83	-37	-53	16	52.17	4/9/99	-4.00	-53.00	-81.50	33.00	16.00	52.17	
3D	3	36	550	N/A	-0.83	-37	-53	16	52.17	4/13/99	-5.50	-53.00	-71.10	31.50	16.00	52.17	
3D	4	36	550	N/A	-0.83	-37	-53	16	52.17	4/7/99	-4.00	-53.00	-70.10	33.00	16.00	52.17	
4D	1	36	550	-36	-0.83	-51	-55	4	54.17	3/30/99	-11.00	-55.00	-69.72	40.00	4.00	54.17	
4D	2	36	550	-36	-0.83	-51	-55	4	54.17	4/2/99	-11.00	-55.00	-69.63	40.00	4.00	54.17	
4D	3	36	550	-36	-0.83	-51	-55	4	54.17	3/31/99	-11.00	-55.00	-69.92	40.00	4.00	54.17	
4D	4	36	550	-36	-0.83	-51	-55	4	54.17	4/1/99	-11.00	-55.00	-70.90	40.00	4.00	54.17	
5D	1	36	550	-36	-0.83	-51	-54	3	53.17	2/8/99	-18.50	-54.00	-94.34	32.50	3.00	53.17	
5D	2	36	550	-36	-0.83	-51	-54	3	53.17	2/26/99	-16.50	-54.00	-70.72	34.50	3.00	53.17	PBDN 7130
5D	3	36	550	-36	-0.83	-51	-54	3	53.17	2/18/99	-17.00	-54.00	-72.40	34.00	3.00	53.17	PBDN 7130
5D	4	36	550	-36	-0.83	-51	-54	3	53.17	3/1/99	-17.00	-54.00	-70.92	34.00	3.00	53.17	PBDN 7130
6D	1	36	550	-42	-0.83	-59	-59	0	58.17	1/20/99	-20.00	-59.00	-72.08	39.00	0.00	58.17	
6D	2	36	550	-42	-0.83	-59	-59	0	58.17	12/28/98	-19.50	-59.00	-99.70	39.50	0.00	58.17	
6D	3	36	550	-42	-0.83	-59	-59	0	58.17	1/25/99	-19.50	-59.00	-74.82	39.50	0.00	58.17	
6D	4	36	550	-42	-0.83	-59	-59	0	58.17	1/13/99	-20.00	-59.00	-71.70	39.00	0.00	58.17	
TOTAL	24							120	1217.5					877.37	125.00	1222.49	

1097.5

23

REVISIONS					
Date	By	Description	Date	By	Description
7-1-?	SA	23 AS-BUILT			

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.		95	H2



PLAN

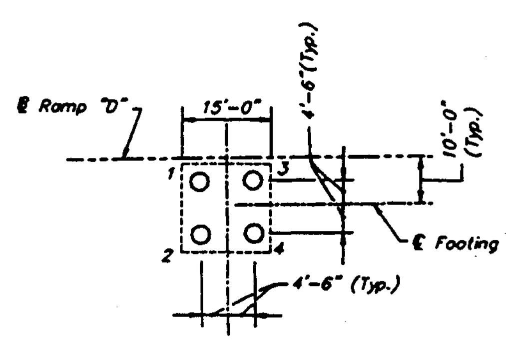
TABLE OF DRILLED SHAFT TOP ELEVATIONS			
SHAFT No.	END BENT NO. 1D	PIER 2D	PIER NOS. 3D THRU 6D
1	14.302	−4.250	−0.830
2	14.062		
3	13.822		
4	13.582	−4.250	−0.830

DRILLED SHAFT INSTALLATION TABLE				
PIER No.	SHAFT DIAMETER (INCHES)	DESIGN LOAD (TONS)	MINIMUM TIP ELEVATION (FT)	100 YEAR SCOUR ELEVATION (FT.)
EB1D	36	450	−35	
2D	36	450	−35	
3D	36	550	−37	
4D	36	550	−51	−36
5D	36	550	−51	−36
6D	36	550	−59	−42
18	*	*	*	*

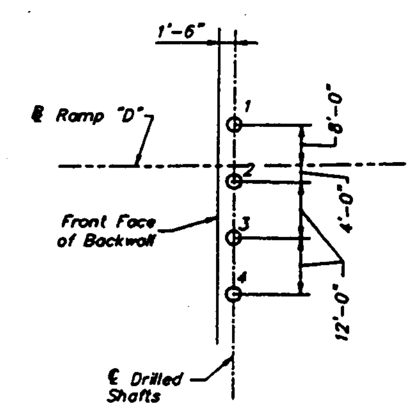
* For Foundation Layout And Installation Data For Pier No. 18, See Sheet D4 or E4.

This is in response to your letter number PB-297-98 dated, August 26th, 1998, in regard to your request to construct a construction joint at elevation 5.50 feet for shaft numbers 2 and 3 at Ramp D, End Bent D1. Your request is approved.

Ramp D, End Bent D1



DETAIL "A"



DETAIL "B"

LEGEND

o Proposed Drilled Shaft.

NOTES

1. For Horizontal Curve Data see Sheet A7.
2. Plan location of Drilled Shafts is given at Bottom of Caps.
3. All Drilled Shafts at the End Bent shall be installed Prior to the installation of MSE Walls.
4. Drilled Shafts shall be installed to the Minimum Tip Elevations shown in the Drilled Shaft Installation Table.
5. For Drilled Shafts reinforcing details and associated notes, See Sheet A24.
6. The portion of End Bent Drilled Shafts exposed above the existing ground shall be wrapped with 6 mil. thickness Polyethylene-Plastic film. See General Notes, Sheets A4.
7. The Foundations shall be constructed in the Phases outlined in the Sequence of Construction drawings.

REVISIONS

Date	By	Description	Date	By	Description
7-15-01	SH	73 AS-BUILT			

	Names	Dates
Drawn by	SCG	10-93
Checked by	LDP	11-93
Designed by	MJL/HDR	9-93
Checked by	DMM	12-93
Approved by	M.D. Caulfield	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

SHEET TITLE: FOUNDATION LAYOUT RAMP "D" BRIDGE

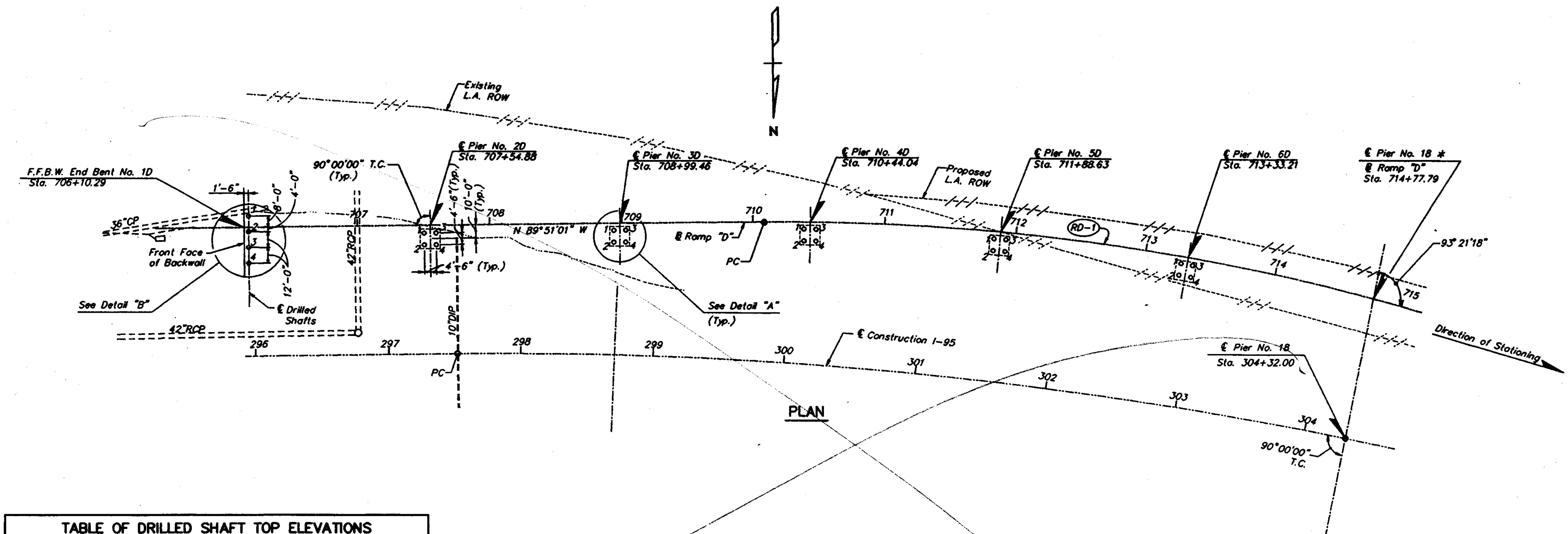
PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

Drawing No.

Index No.

NAME: M:\16414\RAMPD\BRD04A.DWG DATE: SEP 02, 1995 TIME: 10:34 AM REV. BY: RLG

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.		95	H2



PLAN

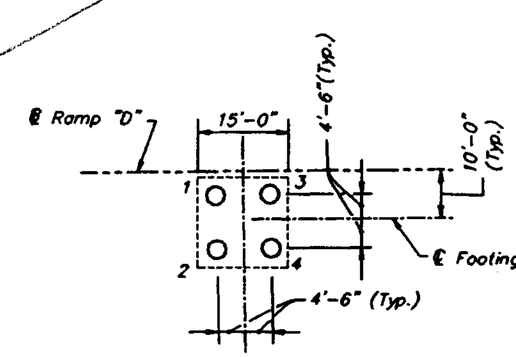
TABLE OF DRILLED SHAFT TOP ELEVATIONS

SHAFT No.	END BENT NO. 1D	PIER 2D	PIER NOS. 3D THRU 6D
1	14.302	−4.250	−0.830
2	14.062		
3	13.822		
4	13.582	−4.250	−0.830

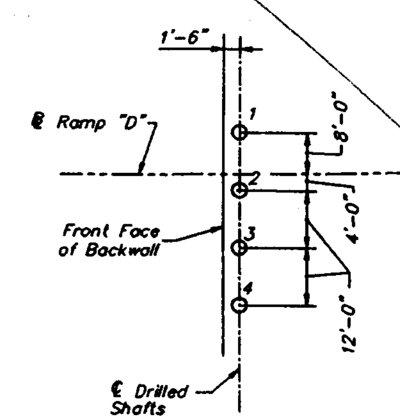
DRILLED SHAFT INSTALLATION TABLE

PIER No.	SHAFT DIAMETER (INCHES)	DESIGN LOAD (TONS)	MINIMUM TIP ELEVATION (FT)	100 YEAR SCOUR ELEVATION (FT.)
EB1D	36	450	−35	
2D	36	450	−35	
3D	36	550	−37	
4D	36	550	−51	−36
5D	36	550	−51	−36
6D	36	550	−59	−42
18	*	*	*	*

* For Foundation Layout And Installation Data For Pier No. 18, See Sheet D4 or E4.



DETAIL "A"



DETAIL "B"

LEGEND

o Proposed Drilled Shaft.

NOTES

1. For Horizontal Curve Data see Sheet A7.
2. Plan location of Drilled Shafts is given at Bottom of Caps.
3. All Drilled Shafts at the End Bent shall be installed Prior to the Installation of MSE Walls.
4. Drilled Shafts shall be Installed to the Minimum Tip Elevations shown in the Drilled Shaft Installation Table.
5. For Drilled Shafts reinforcing details and associated notes, See Sheet A24.
6. The portion of End Bent Drilled Shafts exposed above the existing ground shall be wrapped with 6 mil. thickness Polyethylene Plastic film. See General Notes, Sheets A4.
7. The Foundations shall be constructed in the Phases outlined in the Sequence of Construction drawings.

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	SCG	10-93
Checked by	LDP	11-93
Designed by	MJL/HDR	9-93
Checked by	DMM	12-93
Approved by	M.D. Caulfield	

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
9	DUVAL	72020-3485

SHEET TITLE: FOUNDATION LAYOUT RAMP "D" BRIDGE

PROJECT NAME: I-95 OVER THE ST. JOHNS RIVER

Drawing No.

Index No.

NAME: M:\16414\...\D\DBRD04A.DWG DATE: SEP 02, 1995 TIME: 10:34 AM REV. BY: RLG

W.P.I. NO. 7113770

FISCAL YEAR	SHEET NO.
94	1

THIS CONTRACT PLAN SET INCLUDES:

ROADWAY PLANS
SUMMARY OF PAY ITEMS (07 SHEETS)
SIGNING AND PAVEMENT MARKING PLANS
ROADWAY LIGHTING PLANS
STRUCTURE PLANS
A DETAILED INDEX APPEARS ON THE KEY SHEET OF EACH COMPONENT SET OF PLANS

INDEX OF ROADWAY PLANS

* A Detailed Index of these Sections appears on the Key Sheet for Each Component.

REVISIONS

ROADWAY SHEETS 1,10 (REVISION 2-8-94).
STRUCTURE SHEETS A-4 (REVISION 2-8-94).
C.E.S. UNSTRUNG S.P.N. 10130-3544 & 10130-3501 (REVISION 2-15-94)
C.E.S. - DESIGN GROUP 20 CHANGED PAY ITEMS 583-4, 583-5, ADDED 709-12, DESIGN GROUP 30 DELETED 709-12, CHANGED 711-1, DESIGN GROUP 40 DELETED 510-1 AA, 510-1 AB (REVISION 2-15-94).
ROADWAY SHEETS 8, 9, 14, 17, 24. (REVISION 2-15-94).
SIGNING PAVEMENT MARKING SHEETS S-2, S-3 (REVISION 2-15-94).
LIGHTING SHEETS L-2 ALTERNATE NO. 1, L-2 ALTERNATE NO. 2 (REVISION 2-15-94).
C.E.S. - DESIGN GROUP 20 CHANGED PAY ITEM 109-71 (REVISION 2-24-94).
STRUCTURE SHEETS A-4 THRU A-7, A-7A, B-1, B-2, B-19, B-20, B-22, B-25, B-57, C-1, C-2, C-20, C-21, C-23, C-26, C-67 (REVISION 3-18-94).
ROADWAY 1, 2, 4, 5, 8, 18, 71, 74, 77, 95 (REVISION 3-18-94).
C.E.S. - DESIGN GROUP 10 DELETED 506-2 AA, 506-2 AB. ADDED 459-71 AA, 459-71 AB
C.E.S. - DESIGN GROUP 20 ADDED 102-74-2, 102-76, 102-82, 102-83, 102-92-3, 102-92-4, 102-92-5, 400-1-15, CHANGED 102-81-1, 102-78, 709-12, 709-82-61 (REVISION 3-18-94)
ROADWAY SHEET NO. 1 (REVISION 4-11-94)
C.E.S. - DESIGN GROUP 10 ADDED 110-81 (REVISION 4-11-94)
STRUCTURE SHEETS A-3, A-4, B-85 (REVISION 4-18-94)
C.E.S. - DESIGN GROUP 10 DELETED 110-81, ADDED 110-81 AA, 110-81 AB, 400-149 AA, 400-149 AB, 400-154 AA, 400-154 AB (REVISION 4-18-94).
ROADWAY SHEETS NO. 8,11,16 END PROP. WESTBOUND BRIDGE ADJUSTED (REVISION 9-14-94)
REVISED SHEET NOS. 71,75,76,79,80 FOR DETOUR #1 & ADDED PROFILE SHEET NO. 82A (10-24-94)
REVISED SHEET NOS. 73,74,77,78,81,82 FOR DETOUR #2 & ADDED PROFILE SHEET NO. 82B (11-18-94)
REVISIONS TO OUTFALL DITCH AT MARINE RESERVE PARKING LOT. SHEET NO. 25 (12-18-94)
REVISIONS TO CURVE DATA SHEET FOR COORDINATES. SHEET 12 (01-05-95)
DRAINAGE REVISIONS FOR MARINE RESERVE PARKING LOT. SHEET 9,17,20,23,25 56,57,58. ADDED SHEETS 20A,34A & 34B(1-10-95)
REVISIONS TO FRONTAGE ROAD #1 MILLING & RESURFACING LIMITS. SHEET NOS. 14,15,43 THROUGH 50. (1-13-95)
REVISIONS TO SLOPE FOR RE WALL & MILLING & RESURFACING LIMITS FOR WB ROADWAY. SHEET NO. 3 (1-15-95)
REVISED RIP-RAP LIMITS. SHEET NO. 25 (1-18-95)
REVISED TEMP. PAVT. WIDENING & TEMP. BARRIER WALL. SHEET NO. 73 & 77 (1-18-95)
REVISED VERTICAL CURVE LENGTH. SHEET NO. 19 (1-20-95)
REVISED VERTICAL CURVE GRADES & LENGTH. SHEET NO. 14,15,16,17 (2-8-95)
REVISED LIGHTING PLANS SHEET L-1 THROUGH L-17.(2-13-95)
REVISED BRIDGE SHEET NOS. A-2,B-36,B-39,B-39A,B-40,B-40A,B41,B-41A,B-89,B-90 (3-2-95)
TEMPORARY LIGHTING PLANS SHEET L-17 THROUGH L-20.DELETED TEMP. PAVT. CROSSOVERS FROM SHEET 77(3-18-95)
REVISED SHEET 9. REVISED QUANTITIES FOR DRAINAGE STRUCTURES S-7,S-8,S-9,S-11,S-12,S-17 (5-12-95)
REVISED SHEET 15. REVISED APPROACH SLAB STATION & LOCATION OF DRAINAGE STRUCTURE S-9 (5-12-95)
REVISED SHEET 28. MODIFIED EMBANKMENT SLOPES @ STA 467+35.00, REVISED LENGTHS OF 18" CMP'S FOR DRAINAGE STRUCTURES S-8,S-9. RELOCATED S-9 FROM STA 470+91 TO STA 470+50 (5-12-95)
REVISED SHEET 29 & 31. REVISED LENGTHS OS 18" CMP'S FOR DRAINAGE STR. S-11,S-12. RELOCATED S-17 FROM STA 124+97.50 TO STA 125+00 (5-12-95)
REVISED SHEET 48. ADDED SHLDR. GUTTER @ STA 468+00 (5-12-95)
REVISED SHEET 29: CHANGED GRATE EL. FOR S-10, S-11, S-12. (6-2-95)
REVISED SHEET 30: CHANGED STATION LOCATION OF S-14 & GRATE EL. (6-2-95).REVISED EMBANKMENT AT STA 123+84.61 & REVISED PG EL. AT STA 123+60.00.(7-20-95)
SHEET 33: REVISED LT. DITCH EL. AT STA 128+00 & ADDED MISC. ASPHALT.(7-20-95)
SHEET 31: REVISED GRATE RL. FOR S-15.(7-21-95)
SHEET 77: REVISED TEMP. BARRIER WALL AND ADDED TEMP. PAVT. NEAR DETOUR 3 (8-31-95)
SHEET NOS. 8,12,16,17,23,31,53,54,55,56: MODIFIED FRONTAGE RD. NO.2 AND EXTENDED GUARDRAIL LIMITS (12-95)
SHEET NOS. 8,17,49,50,51,56,57,59,60,77,81,82,& 82C : MODIFIED GUARDRAIL LENGHT, EARTHWORK QUANTITIES, CROSS SECTIONS AND TRAFFIC CONTROL PLANS.
SHEETS 9,25,35,47,62,63: MODIFIED USE OF RIP-RAP

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

FINAL PLANS

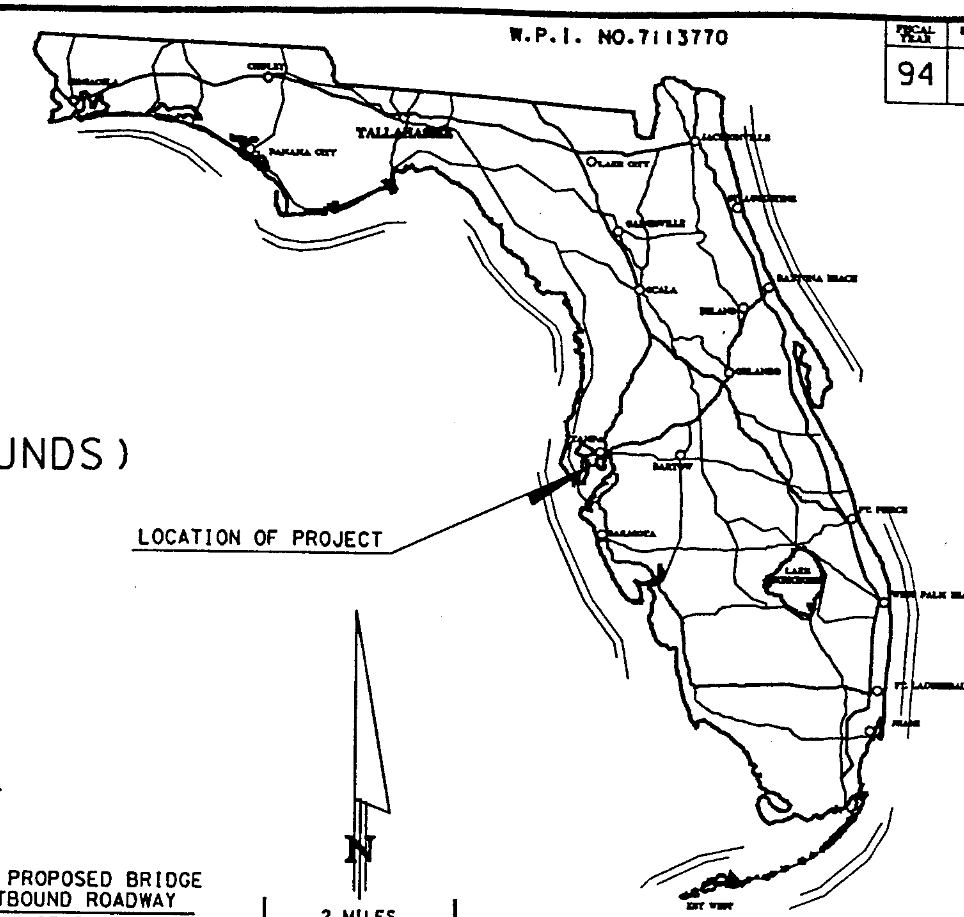
~~PLANS OF PROPOSED~~

STATE HIGHWAY

STATE PROJECT NO. 10130-3544 (FEDERAL FUNDS)

PINELLAS & HILLSBOROUGH COUNTIES

STATE ROAD NO. 600



R 17 E | R 18 E

TO TAMPA

BEGIN PROPOSED BRIDGE WESTBOUND ROADWAY
STA. 44+66.75
℄ WESTBOUND RDWY.

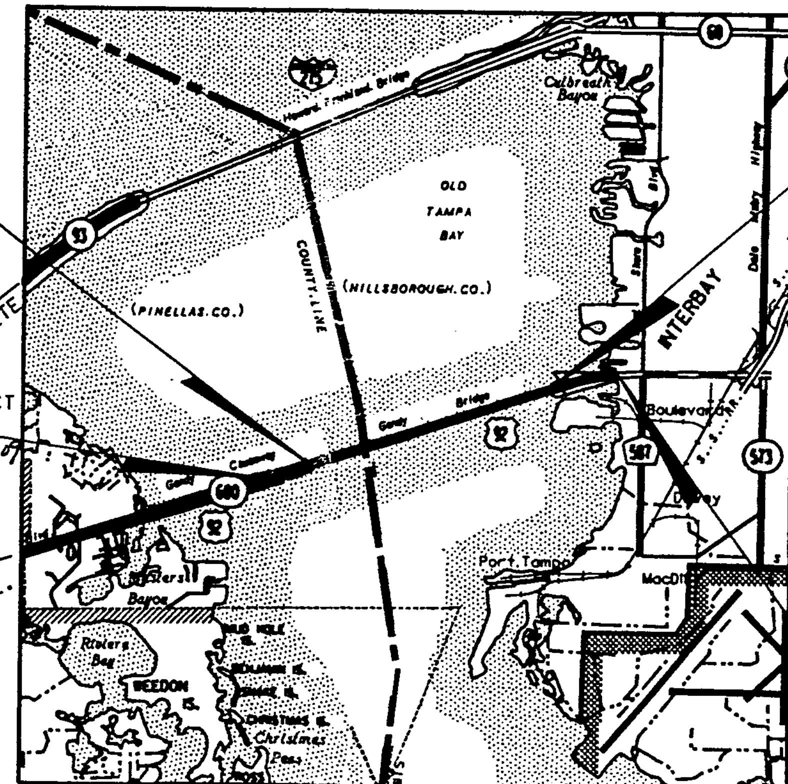
END PROPOSED BRIDGE WESTBOUND ROADWAY
STA. 183+33.25
℄ WESTBOUND RDWY.

T 29 S
T 30 S

TO ST. PETE

BEGIN STATE PROJECT NO. 10130-3544
STA. ~~11+49.64~~ 9+39.67
℄ WESTBOUND RDWY.
M.P. No. 9.849

TO ST. PETE.



N

2 MILES

END STATE PROJECT NO. 10130-3544
STA. 222+65.03
℄ WESTBOUND RDWY.

R 17 E | R 18 E

ROADWAY PLANS ENGINEER OF RECORD

DAVID C. BULLARD
PROFESSIONAL ENGINEER NO. 10087

PLANS PREPARED BY :

BEISWENGER, HOCH & ASSOCIATES, INC.
1408 NORTH WESTSHORE BOULEVARD
TAMPA, FL. 33607 SUITE 900

CONTRACTOR:	Misener Marine Const., Inc.
PRIME CONSULTANT:	PBS&J Construction Serv., Inc.
SUBCONSULTANT:	Williams Earth Sciences, Inc.
DISTRICT SECRETARY:	William McDaniel
RESIDENT ENGINEER:	Marty R. Sanchez
PROJECT MANAGER:	Marc J. Knapp
PROJECT ENGINEER:	Gary M. Granata
DATE WORK STARTED:	August 20, 1994
DATE WORK COMPLETE:	March 28, 1997
DATE WORK ACCEPTED:	MARCH 28, 1997

Bridge 100585

NOTE: LENGTH OF PROJECT BASED ON ℄ WESTBOUND RDWY.STATIONING

LENGTH OF PROJECT

	LINEAR FT.	MILES
ROADWAY	7248.89	1.373
BRIDGES	13866.50	2.626
NET LENGTH OF PROJ.	21115.39	3.999
EXCEPTIONS	--------	------
GROSS LENGTH OF PROJ.	21115.39	3.999

FDOT PROJECT MANAGER : RANDY SANBORN P.E.

REVISIONS

DATE	BY	DESCRIPTION
2-8-94	JJR	REMOVED NOTE, LOWER RIGHT; REVISED NUMBER OF PAY ITEM SHEETS.
3-18-94	JJR	REVISED NUMBER OF PAY ITEM SHEETS.
10-24-94	PNC	ADDED SHEET NO. 82A
11-18-94	PNC	ADDED SHEET NO. 82B
1-10-95	PNC	ADDED SHEET NOS. 20A,34A&34B
1-26-96	NJR	ADDED SHEET NO. 82C

RECEIVED
DEC 24 1998
District 7 Bridge Structures & Facilities

ROADWAY PLANS APPROVED BY:
DATE:
P.E. NO.:

Thu Feb 20 09:10 1997
O: \KEYSBA01.PRF
s: \gandyb\design\keysba01.dgn

GENERAL NOTES - COMMON (CONT'D)

FOUNDATION: Foundations at all Piers are supported on concrete drilled shafts. Foundations at abutments only are supported on 18" square prestressed concrete piles. Drilled Shafts shall be constructed to provide a minimum service load capacity as given in Sheet No. A-5 on the Drill Shaft Installation Table.
End Bent piles shall be driven to provide a minimum service load capacity of 90 Tons (180 Kips), times the appropiate factor of safety in accordance with Section 3J2.2 of the Supplemental Specification 455. △

END BENT CONSTRUCTION: End Bent Piles are to be driven before proprietary wall installation. The exposed portion of these piles, above the existing ground and below the bottom of the end bent cap, shall be wrapped with two independent wrappings of 6 mil. thickness polyethylene plastic, in accordance with Section 459 of the Supplemental Specifications. The polyethylene sheeting in concrete piles shall be paid for at the contract unit price per square yard under Item No. 459-71. △

FOOTING CONSTRUCTION: All pier footing excavation in dry land shall be dewatered prior to placing reinforcement and concrete. Footing concrete shall be placed in the dry. Dewatering shall continuously maintain the water surface a minimum of one foot below the bottom of the footing from one hour before concrete placement commences to six hours after all concrete in the footing has been cast. No separate payment will be made for dewatering.
Seal concrete for footings over water shall be used. Payment shall be made in accordance with article 400-21 of the Standard Specifications.

ANCHOR BOLTS: Shall be ASTM A-307 or ASTM A-709 Gr.36 unless otherwise noted. All bearing plates, anchor bolts, nuts and washer shall be hot-dip galvanized in accordance with Standard Specification 962-7.

DIMENSIONS: All dimensions are measured horizontally and vertically unless otherwise noted.

UTILITIES: See Utility Plans for the location of existing utilities.

CHAMFER: All exposed edges and corners of concrete shall be chamfered 3/4", unless otherwise noted.

ELEVATIONS: All Elevations are referred to National Geodetic Vertical Datum (NGVD) of 1929.

NEOPRENE PADS: Composite neoprene bearing pads as designated in the plans, shall be furnished and installed by the Contractor.

STAY-IN-PLACE DECK FORMS: Stay-In-place deck forms shall not be allowed.

DECK FINISHING: All deck slabs shall be finished and grooved according to the Specification Section 400-15.2.5 Class 4 Floor Finish.

△3 PENETRANT SEALER: All concrete surfaces of substructure over the waterway up to 12 feet above El. 1.50 shall be sealed with a high performance penetrant sealer. Materials and labor for the placement of the penetrant sealer shall be paid for under pay Items 400-149 Penetrant Sealer and 400-154 Cleaning and Sealing Concrete Surfaces.

PIERS DUCTS AND SHEAR KEYS: The caps of piers have to be provided with ducts in size and numbers as shown in the substructure plans for the future symmetrical widening of the bridge. The end sides of each cap will have shear keys according to the plans. For additional information refer to Pier drawings.

BID ITEM NOTES

1. For summary of bridge pay items see print of computer output.
2. Payment for incidental items not specifically covered in the individual bid items shall be included in the contract unit prices for bid items.
3. Bid Item No. 400-7 includes 178 S.Y. of approach slab grooving.
4. Total plan area of approach slabs required is 252 S.Y. Details of approach slabs and payment are included in the roadway plans and Finish Grade Elevation Sheets.
5. The contractor shall bid on the common items and only one of the following alternates:
   a) Florida Bulb-T Beams (FBT) with Drop-In-beams in Channel Unit.
   b) Simple span AASHTO Beams in low approaches and over water except Channel Unit of 3 spans. This unit is the same for both alternates. FBT with Drop-in beams.

△ ~~6. Bid Item 506-2 (Bridge Drainage Piping) includes 1,403 LF (each alternate) of 4"Ø P.V.C. Pipe.~~

△ **NOTE: DEMOLITION OF EXISTING WESTBOUND BRIDGE, RENOVATION OF A PORTION OF THE BRIDGE AS A FISHING PIER AND CONSTRUCTION OF REEFS WILL BE INCLUDED IN A SEPARATE CONTRACT S.P.N. 10130-3501.**

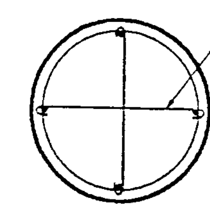
DRILLED SHAFT GENERAL NOTES

△ 1. All drilled shafts shall be built by a suitable method chosen by the contractor and approved by the Engineer.
2. Preliminary drilled shaft tip elevations are shown in Sheet No. A-5. Final drilled elevations may be modified after results of the load test program and other soil investigation and they may vary significantly from those shown in the plans. The contractor shall provide equipment capable of drilling to elevation -80 ft. or deeper.
△ 3. The contractor's attention is called to the fact that some of the drilled shafts for the piers may be installed through part of the removed old bridge substructure as well as the debris thereof. The contractor shall submit to the engineer the sequence of removing or drilling through the piles or any other obstacles for the construction of the drilled shafts. Should removal be required, the contractor will be compensated under pay item number 110-81, Piling Removal and Disposal. Compensation will be made per each for each drilled shaft location where the conflict occurs. Compensation shall include all costs associated with removal as well as standby time for the drilled shaft crew. These costs shall include, but not be limited to, direct cost, field and office overhead costs and acceleration. No additional compensation in dollars or time will be made for drilling through piling or other obstacles.
4. Vertical reinforcement and spiral reinforcement shall be spliced using mechanical couplers capable of developing 125% fy of the bar. Mechanical couplers shall be staggered 2'-0" between vertical bars. Proposed couplers shall be submitted for approval by the engineer.
5. All spiral reinforcement shall begin and end with 1 1/2 closed turns.
6. The spacing of the vertical bars at footing level shall be set with a template.
7. The vertical reinforcement (as a unit) and spiral reinforcement may be adjusted slightly, within specified tolerances, to clear both column and footing reinforcement.
8. For Drilled Shaft Installation Tables for both alternates, see Sheet No. A-5.
△ 9. Estimated casing tip elevations are shown in sheet No. A-5. Final end of casing elevations may be modified depending upon local soil conditions. Permanent casing is not required in this project.
△ 10. The cost of performing Test Holes shall be paid under Items 455-91-5, 455-91-7 and 455-91-16 for shafts of 48", 72" and 84" diameter, respectively. The cost of Unclassified Shaft Excavation shall be paid for under items 455-122-5, 455-122-7 and 455-122-15 for 48", 72" and 84" shaft diameter, respectively. Item 455-111 includes the quantity for a 5' core shaft at the bottom of each drilled shaft excavation.

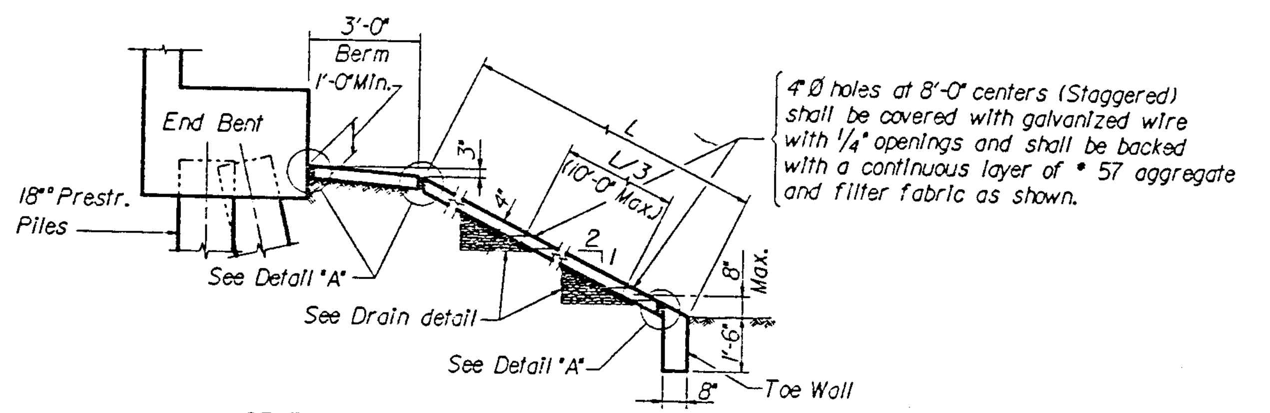
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			A-4

ESTIMATED QUANTITIES △

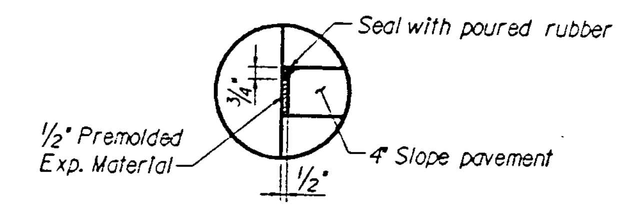
ITEM	UNIT	ALTERNATE 1			ALTERNATE 2		
		48"	72"	84"	48"	72"	84"
Test Holes	LF	50	50	50	50	50	50
Unclass. Shaft Excav.	LF	8766	3744	1644	9711	5616	1644
Core (Shaft Excav.)	LF	1920			2295		



TEMPORARY CAGE STIFFENER DETAILS
N. T. S.

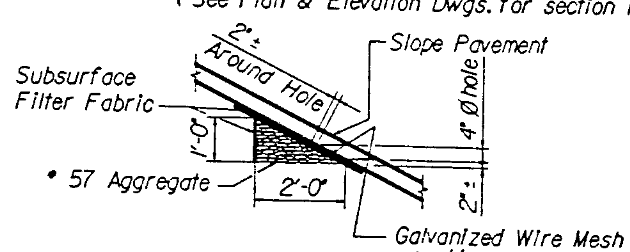


SECTION X - X (NORMAL TO END BENT)
(See Plan & Elevation Dwgs. for section location.)

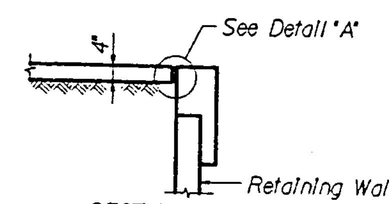


DETAIL "A"

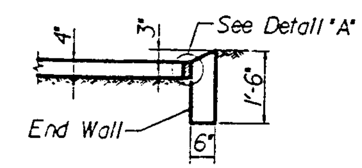
Details of joint at End Bent applies at all contacting surfaces of slope Pavements with End Bents, Proprietary Walls, End Walls, Toe Walls and both edges of Berm.



DRAIN DETAIL



SECTION Y - Y



SECTION Z - Z

SECTION AT LATERAL EDGE OF SLOPE PAVEMENT
(See Plan & Elevation Dwgs. for section location.)

SLOPE PAVEMENT DETAILS

Cost of making holes, wire, filter fabric and aggregate to be included in the cost of Concrete Slope Pavement (4" thick).
Slope pavement shall be tooled with a double 1/4" radius tool at 2 ft± intervals along the slope at 4 ft± across the slope. Construction joints will be permitted at joints as directed by the Engineer.

REVISIONS

	Date	By	Description	Date	By	Description
△	2-8-94	J.O.C.	Add Note for S.P.N. 10130-3501.	△4-8-94	L.M.V.	Omit concrete admixture note and add penetrant sealer note.
△	3-18-94	L.M.V.	Add note for casing, test holes, shaft excavation & omit note for Bridge Drainage Pipe.	△3		
△	4-14-94	L.M.V.	Revise drilled shaft note.			

	Names	Dates
Drawn by	X.M.	1-93
Checked by	L.M.V.	8-93
Designed by	B.O.K.	1-93
Checked by	L.M.V.	8-93
Approved by	S. H. W.	

ENGINEER OF RECORD:
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CONSULTING ENGINEERS & PLANNERS

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 600	PINELLAS/HILLSBOROUGH	10130-3544

SHEET TITLE: GENERAL NOTES - COMMON (2)
PROJECT NAME: WESTBOUND CANDY BRIDGE

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FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			A-5

DRILLED SHAFT INSTALLATION TABLE I - ALTERNATE I

PIER NO.	NO. REQ'D.	DIA.	DESIGN LOAD (KIP)	MIN. TIP EL (FT.)	SCOUR EL (FT.)	QUANTITY L.F.
2	4	48" TYPE I	809	-61	N.A.	238
3			918	-52	-29	202
4			1026	-47	-32	182
5			1026	-48	-34	186
6			1046	-55	-38	214
7			1026	-52	-36	202
8			1026	-41	-28	158
9			1011	-41	-19	158
10			1084	-31	-20	118
11			1015	-42	-27	162
12			1084	-38	-25	146
13			956	-52	-16	202
14			1084	-26	-13	98
15			1015	-32	-17	122
16			1084	-21	-9	78
17			956	-30	-16	114
18			1084	-36	-20	138
19			1015	-22	-10	82
20			1084	-57	-10	222
21			956	-29	-16	110
22			1084	-30	-16	114
23			1015	-41	-9	158
24			1084	-21	-9	78
25			956	-32	-19	122
26			1084	-24	-11	90
27			1015	-42	-10	162
28			1084	-47	-20	182
29			956	-37	-11	142
30			1084	-38	-21	146
31			1015	-35	-13	134
32			1084	-32	-11	122
33			956	-24	-11	90

PIER NO.	NO. REQ'D.	DIA.	DESIGN LOAD (KIP)	MIN. TIP EL (FT.)	SCOUR EL (FT.)	QUANTITY L.F.
34	4	48" TYPE I	1084	-22	-9	82
35			1015	-25	-12	94
36			1084	-33	-19	126
37			956	-35	-21	134
38			1084	-41	-22	158
39			1015	-46	-22	178
40			1084	-45	-22	174
41			956	-57	-21	222
42			1084	-50	-22	194
43			1015	-42	-18	162
44			1084	-38	-21	146
45			956	-44	-21	170
46			1084	-41	-22	158
47			1015	-40	-22	154
48			1084	-44	-20	170
49			956	-56	-22	218
50			1084	-61	-20	238
51			1015	-47	-23	182
52			1084	-46	-23	178
53			956	-47	-23	182
54	4	2.0' 72" TYPE 2	1237	-42	-23	162
55			1168	-46	-18	178
56			1237	-36	-22	138
57			1109	-41	-23	158
58			1237	-32	-19	122
59			1168	-32	-20	122
60	4	72" TYPE 2	1237	-33	-20	118
61			1109	-35	-22	126
82			1237	-46	-30	170
63			1168	-47	-29	174
64			1237	-36	-23	130
65			1109	-39	-25	142

PIER NO.	NO. REQ'D.	DIA.	DESIGN LOAD (KIP)	MIN. TIP EL (FT.)	SCOUR EL (FT.)	QUANTITY L.F.
66	4	72"	1237	-41	-27	150
67			1168	-68	-26	258
68			1237	-45	-31	166
69			1109	-41	-31	150
70	4	84" TYPE I	1353	-65	-32	246
71			1284	-73	-40	278
72			1353	-70	-38	266
73			1154	-67	-42	254
74			1873	-64	-42	242
75			1873	-61	-33	230 ✓
76			1154	-53	-29	198
77			1353	-39	-25	142
78			1284	-39	-25	142
79			1353	-53	-34	198 ✓
80	4	TYPE 2	1109	-59	-28	222
81			1237	-51	-32	190
82			1168	-48	-35	178
83			1237	-50	-36	186
84			1109	-67	-39	254 ✓
85			1237	-53	-35	198
86			1168	-56	-33	210
87			1237	-70	-31	266
88			1109	-61	-32	230
89			1237	-59	-35	222
90	4	48" TYPE I	1015	-57	-30	222
91			1084	-61	-28	238
92			956	-61	-22	238
93			1084	-61	-22	238
94			1015	-68	-20	266
95			1084	-64	-21	250
96			883	-76	N.A.	298
97			809	-76	N.A.	298

TOTAL DRILLED SHAFTS ALTERNATE I

48" 10,040 L.F.
72" 4,620 L.F.
84" 2,196 L.F.

~~TOTAL DRILLED SHAFTS ALTERNATE 2~~

~~48" 11,197 L.F.~~
~~72" 6,930 L.F.~~
~~84" 2,196 L.F.~~

NOTES: 1) For "Drilled Shaft General Notes," see Sheet No. A-4.
2) Design Loads are Service Loads for DL and LL after future symmetrical widening as the maximum axial load per one shaft.

Gandy Bridge Repalcement
State Project No. 10130-3544
DRILLED SHAFT AS-BUILT DATA

PIER #	ACTUAL TIP ELEVATIONS Shaft #1	Shaft #2	Shaft #3	Shaft #4
2W	-49.3	-55.6	-49.0	-49.6
3W	-52.0	-52.5	-58.4	-52.0
4W	-48.0	-47.3	-48.2	-47.6
5W	-56.5	-48.4	-49.3	-49.2
6W	-55.0	-55.4	-55.0	-55.3
7W	-52.3	-52.3	-52.4	-51.8
8W	-41.4	-42.0	-41.5	-42.5
9W	-41.5	-41.4	-41.1	-41.0
10W	-33.1	-32.2	-33.1	-32.2
11W	-42.3	-42.0	-42.0	-42.0
12W	-38.2	-38.1	-38.4	-38.3
13W	-51.8	-52.4	-52.4	-52.1
14W	-27.1	-26.5	-26.0	-26.1
15W	-32.1	-32.3	-33.0	-32.8
16W	-25.5	-25.4	-25.7	-25.7
17W	-30.5	-30.2	-30.1	-30.3
18W	-36.3	-36.2	-36.2	-36.1
19W	-22.4	-22.7	-22.6	-22.5
20W	-30.4	-30.7	-30.1	-30.4
21W	-29.6	-29.2	-29.8	-29.5
22W	-30.2	-30.6	-32.3	-30.1
23W	-41.0	-41.2	-41.3	-41.8
24W	-22.9	-23.5	-23.1	-22.5
25W	-32.6	-33.4	-33.0	-33.0
26W	-24.7	-30.5	-24.4	-24.1
27W	-43.5	-41.5	-41.9	-41.7
28W	-47.2	-47.5	-47.4	-47.6
29W	-42.1	-37.5	-37.4	-37.8
30W	-38.3	-38.4	-38.4	-38.2
31W	-34.9	-35.4	-35.3	-35.1
32W	-37.0	-37.6	-37.0	-32.4
33W	-23.9	-24.0	-24.0	-24.0
34W	-22.5	-22.8	-22.6	-22.6
35W	-27.0	-25.1	-33.0	-25.8
36W	-33.6	-33.4	-35.5	-33.8
37W	-34.8	-46.7	-35.5	-45.7
38W	-41.0	-41.7	-41.3	-47.7
39W	-46.9	-46.3	-45.5	-46.0
40W	-45.1	-45.2	-45.5	-45.2
41W	-57.2	-57.2	-57.4	-57.4
42W	-50.5	-50.0	-50.4	-50.1
43W	-41.8	-42.2	-42.0	-42.0
44W	-38.0	-38.0	-38.2	-38.0
45W	-44.3	-44.5	-44.0	-44.2
46W	-41.0	-41.1	-41.0	-41.2
47W	-40.1	-40.1	-40.2	-40.2
48W	-44.1	-44.1	-44.7	-44.0
49W	-38.0	-31.2	-56.1	-56.7

PIER #	ACTUAL TIP ELEVATIONS Shaft #1	Shaft #2	Shaft #3	Shaft #4
50W	-46.2	-43.4	-40.0	-61.1
51W	-47.0	-47.4	-47.2	-47.6
52W	-46.3	-46.1	-46.3	-47.9
53W	-48.0	-48.2	-47.8	-47.8
54W	-42.0	-42.0	-41.8	-42.0
55W	-58.9	-65.6	-46.0	-46.2
56W	-50.8	-50.1	-41.3	-52.4
57W	-41.0	-41.4	-41.1	-41.0
58W	-32.1	-32.2	-33.8	-32.0
59W	-39.1	-42.0	-38.3	-37.7
60W	-33.3	-33.0	-52.5	-33.3
61W	-35.1	-35.2	-35.2	-35.0
62W	-46.5	-45.5	-44.4	-46.0
63W	-50.7	-47.3	-47.3	-58.4
64W	-44.7	-39.7	-46.1	-36.0
65W	-39.1	-57.4	-39.0	-39.9
66W	-41.6	-41.2	-41.5	-43.2
67W	-52.8	-63.8	-43.0	-64.8
68W	-45.0	-46.8	-68.3	-80.2
69W	-84.7	-85.3	-86.7	-63.8
70W	-66.2	-65.3	-65.7	-65.8
71W	-73.1	-73.8	-73.7	-73.5
72W	-71.0	-69.8	-70.6	-70.4
73W	-67.0	-67.4	-67.5	-67.2
74W	-64.0	-64.1	-64.0	-64.0
75W	-63.6	-62.7	-61.7	-61.4
76W	-54.9	-58.3	-58.3	-54.7
77W	-43.5	-44.7	-40.1	-43.2
78W	-44.4	-41.4	-45.7	-40.8
79W	-53.7	-53.3	-54.0	-54.8
80W	-70.0	-60.0	-58.8	-59.0
81W	-50.7	-57.6	-51.0	-51.0
82W	-48.1	-48.0	-48.2	-48.1
83W	-50.4	-50.7	-50.7	-50.2
84W	-67.0	-67.1	-67.0	-67.7
85W	-53.1	-53.6	-52.7	-53.3
86W	-56.0	-56.8	-56.3	-56.0
87W	-70.3	-70.0	-70.0	-74.6
88W	-61.1	-61.2	-60.9	-72.1
89W	-59.2	-59.8	-58.9	-59.2
90W	-58.6	-58.1	-59.4	-58.9
91W	-62.1	-61.6	-61.6	-67.7
92W	-61.0	-62.0	-62.3	-68.9
93W	-61.8	-61.9	-61.6	-61.3
94W	-68.4	-68.3	-68.4	-68.2
95W	-65.2	-64.7	-65.0	-72.8
96W	-76.1	-76.2	-76.4	-75.8
97W	-76.4	-76.2	-77.1	-76.6

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	G.E.	1-93
Checked by	B.O.K.	1-93
Designed by	B.O.K.	1-93
Checked by	S.S.	1-93
Approved by	S.H.W.	

ENGINEER OF RECORD:
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CONSULTING ENGINEERS & PLANNERS

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 600	PINELLAS/HILLSBOROUGH	10130-3544

SHEET TITLE: DRILLED SHAFT INSTALLATION TABLES

PROJECT NAME: WESTBOUND GANDY BRIDGE

Drawing No.

Index No.

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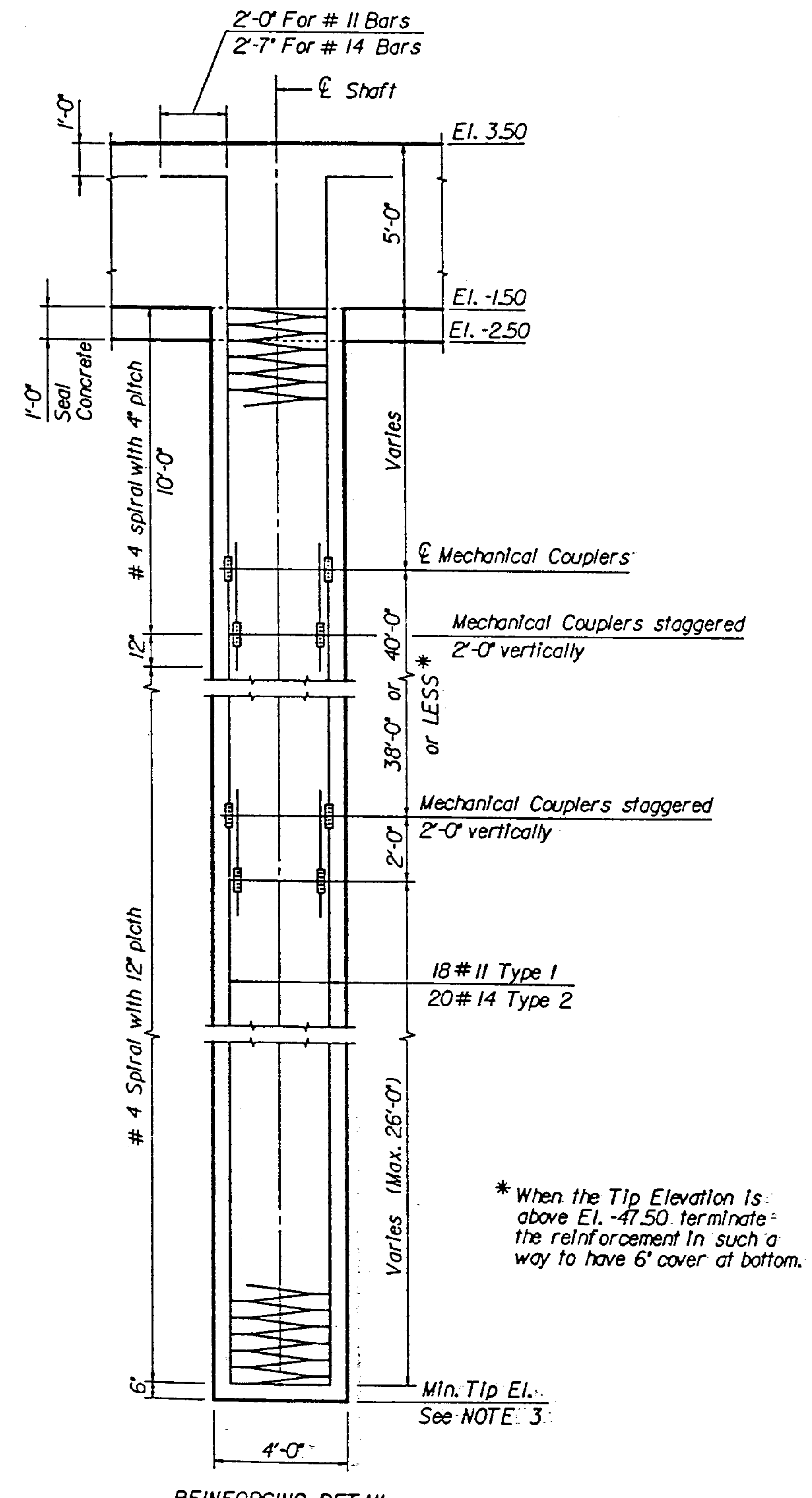
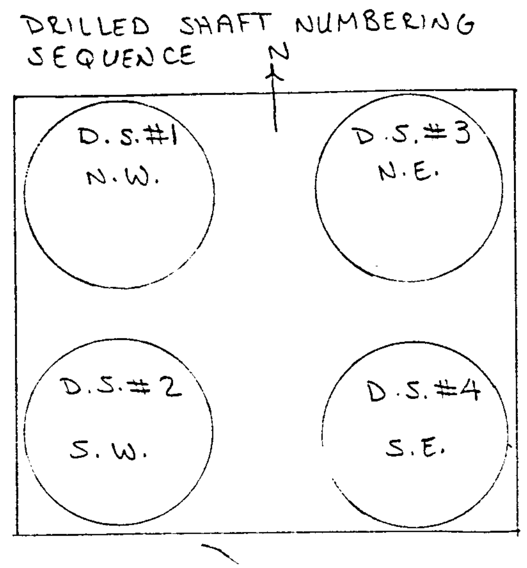
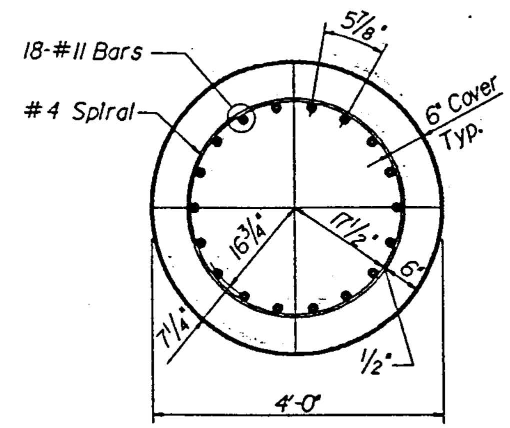
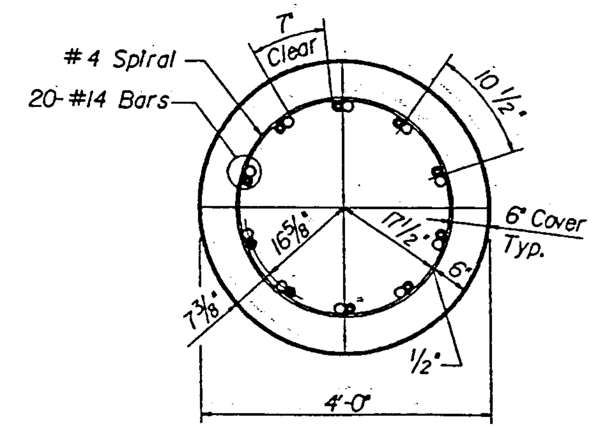


REINFORCING DETAIL
48" DIA. DRILLED SHAFT





TYPE 1
(ALTERNATE 1 ONLY)



TYPE 2
(ALTERNATE 2 ONLY)

DRILLED SHAFT SCHEDULE

ALT.	PIER NOS.	No. of D. S. Dia. & Type/Pier
1	2-W thru 53-W & 90-W thru 97-W	4 - 4' Ø Type 1
	54-W thru 59-W	4 - 6' Ø Type 2
	60-W thru 69-W & 80-W thru 89-W	4 - 6' Ø Type 1
	70-W thru 79-W	4 - 7' Ø Type 1
2	2-W thru 79-W & 128-W thru 138-W	3 - 4' Ø Type 2
	80-W thru 87-W	4 - 6' Ø Type 2
	88-W thru 102-W & 113-W thru 127-W	4 - 6' Ø Type 3
	103-W thru 112-W	4 - 7' Ø Type 1

NOTES: 1) For "Drilled Shaft General Notes", see Sheet No. A-4.
2) For "Drilled Shaft Installation Tables", see Sheet No. A-5.
3) For Minimum Tip Elevation, see Sheet No. A-5.
4) —▭— Denotes mechanical coupler.

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	E.A.P.	4-93
Checked by	B.O.K.	4-93
Designed by	B.O.K.	4-93
Checked by	S.S.	5-93
Approved by	S.H.W.	

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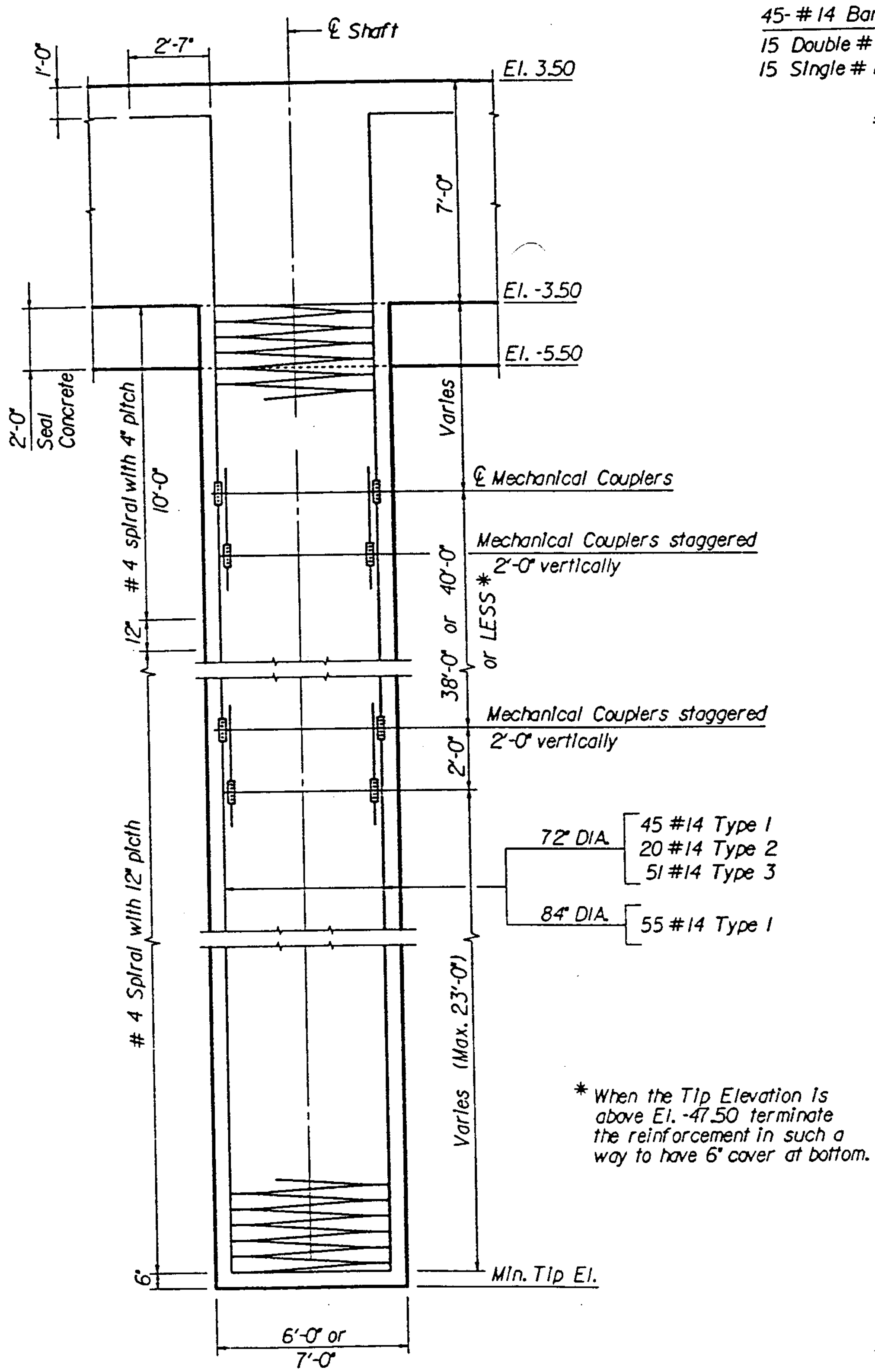
SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

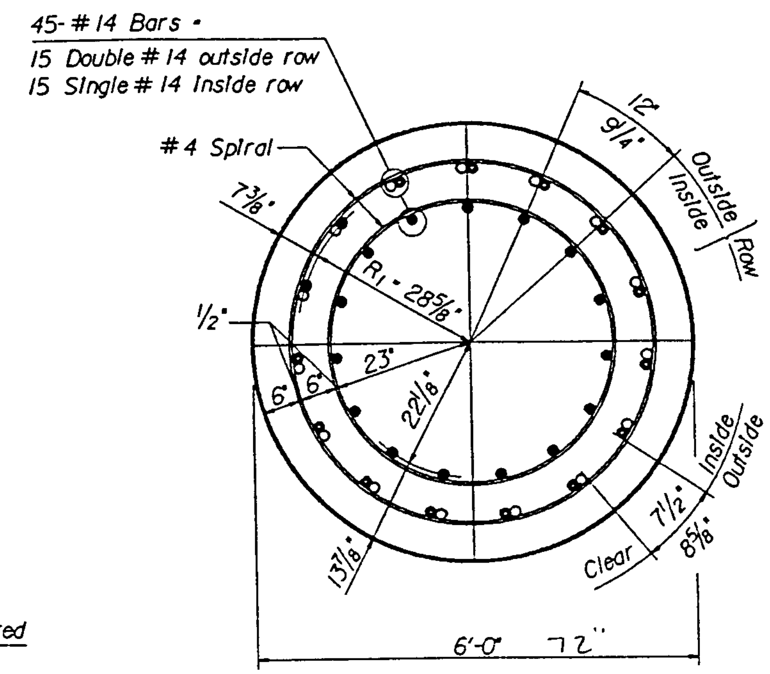
ROAD NO.	COUNTY	PROJECT NO.
S.R. 600	PINELLAS/HILLSBOROUGH	10130-3544

SHEET TITLE: DRILLED SHAFT REINFORCING (1)
PROJECT NAME: WESTBOUND GANDY BRIDGE
Drawing No.
Index No.

FDM ROAD DIST. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			A-7



REINFORCING DETAIL
72" DIA. DRILLED SHAFT &
84" DIA. DRILLED SHAFT



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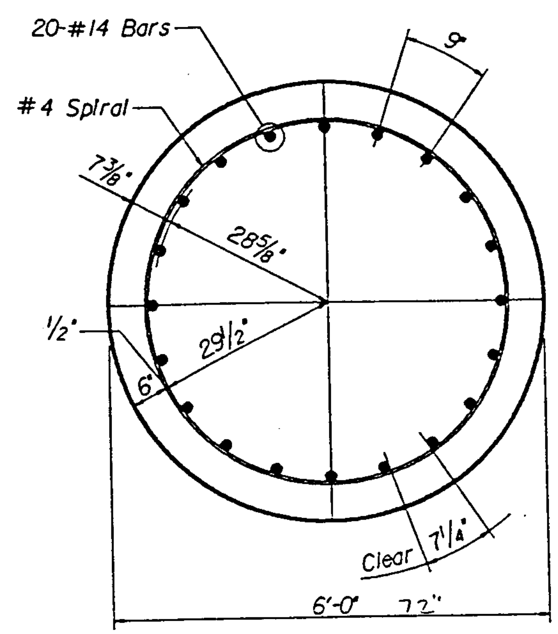
(ALTERNATE 1 ONLY)



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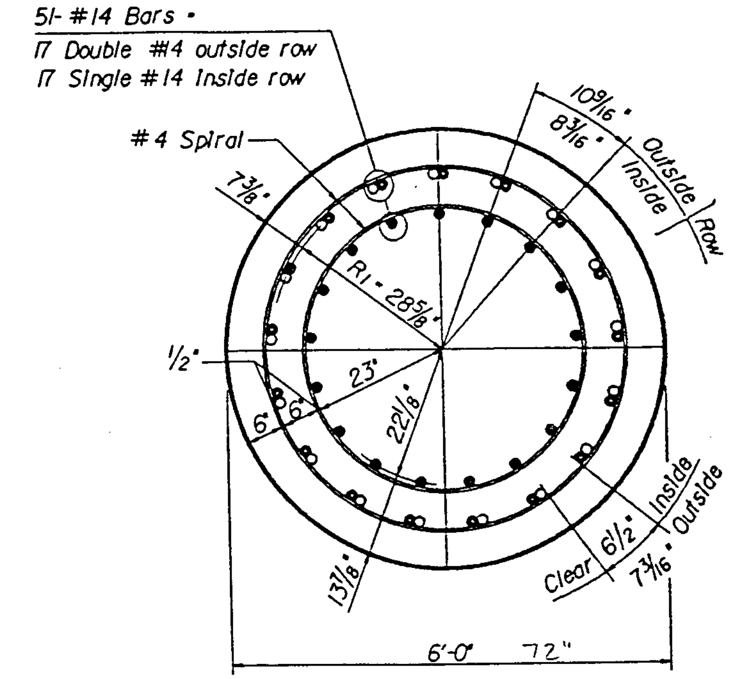
(ALTERNATES 1 & 2)



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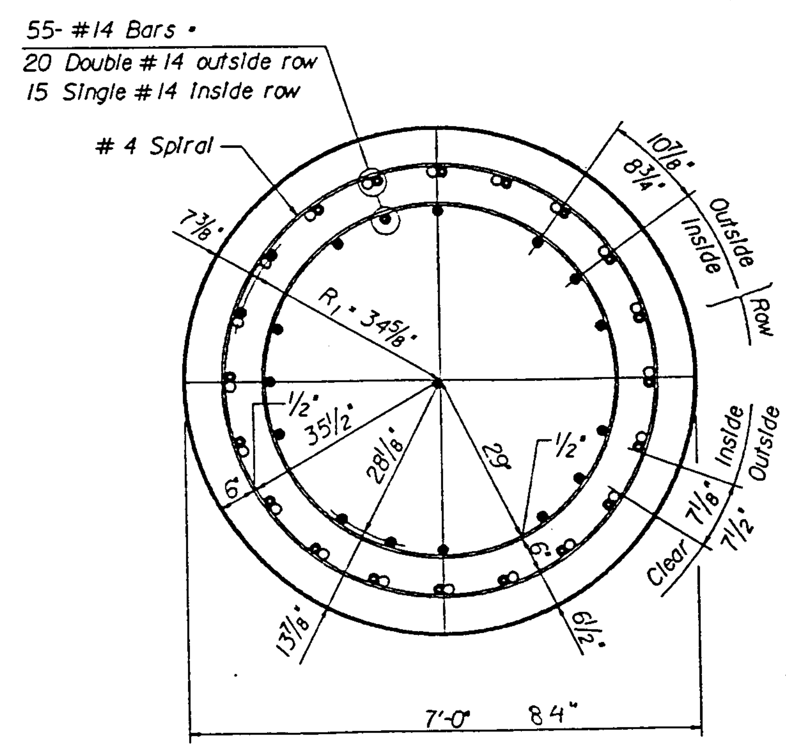
(ALTERNATE 2 ONLY)



TYPE 1
(ALTERNATEs 1 & 2 ONLY)

NOTE: Work this sheet in conjunction with Sheet No. A-6.

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	E.A.P.	4-93
Checked by	B.O.K.	4-93
Designed by	B.O.K.	4-93
Checked by	S.S.	5-93
Approved by	S.H.W.	

ENGINEER OF RECORD:
BEISWENGER, HOCH & ASSOCIATES, INC.
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BEISWENGER, HOCH & ASSOCIATES, INC.
CONSULTING ENGINEERS & PLANNERS

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 600	PINELLAS/HILLSBOROUGH	10130-3544

SHEET TITLE:
DRILLED SHAFT REINFORCING (2)

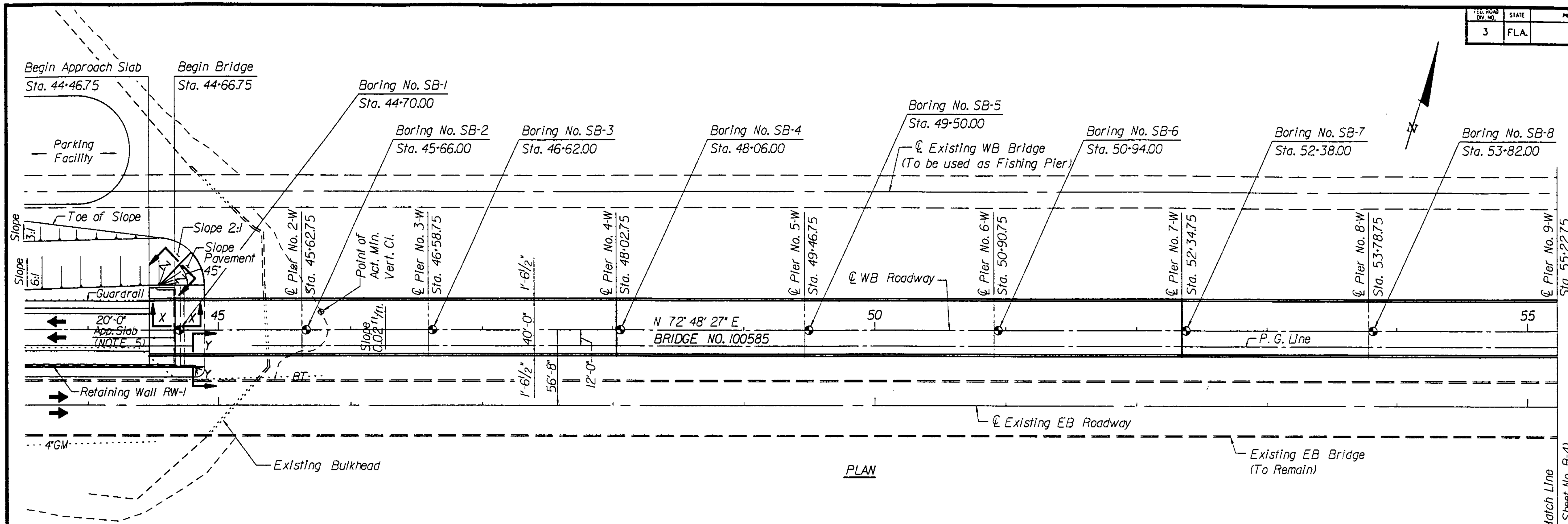
PROJECT NAME:
WESTBOUND GANDY BRIDGE

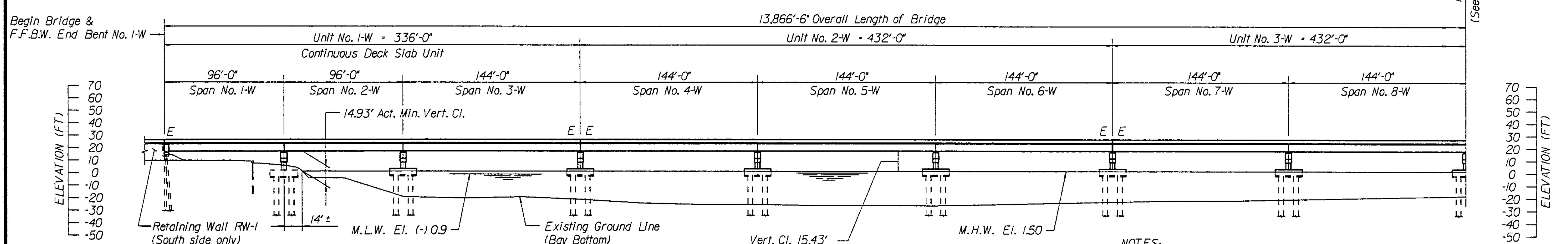
Drawing No.

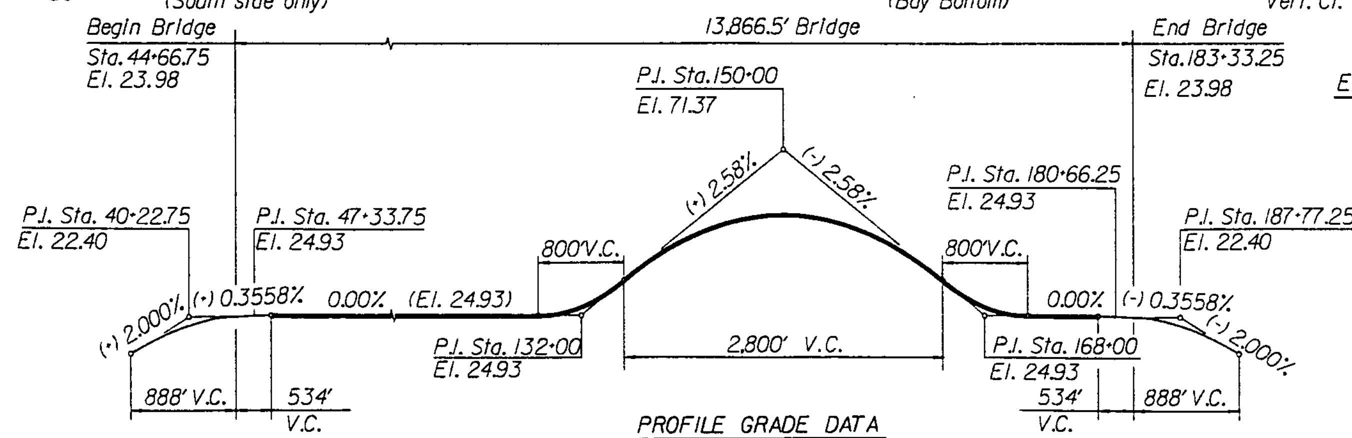
Index No.

S:\BRIDGE\BGANDY\SBDRSH2.DGN S:\BRIDGE\BGANDY\SBDRSH2.PRF 8/29/93 09:39:04

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-3







ELEVATION

LEGEND:

E = Expansion
⊕ = Core Boring
F.F.B.W. = Front Face of Backwall

NOTES:

1) For boring details, see sheets No. A-10 thru A-29.
2) For Sections X-X, Y-Y & Z-Z see sheet No. A-4.
3) For Typical Bridge Cross Sections, see sheet Nos. B-17 & B-18.
4) See Retaining Wall sheets No. W-1 thru W-4 for details of Retaining Walls.
5) For information on the Approach Slab see sheet No. B-43.
6) For location of Light Pole pilasters, see sheet No. B-59.
7) Existing Westbound Gandy Bridge to be removed under contract for State Project No. 10130-3501.
8) The County Line Sign is located at Sta. 87+78.58. For additional information see Signing Plans.

TRAFFIC DATA

1991 ADT = 27,800
1998 ADT = 38,358
2015 ADT = 64,000
K = 8%, D = 55%, T = 4.6%
Design Speed (Bridge) = 70 M.P.H.

FINLEY McNARY/JANSSEN SPAANS
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2825 East 56th Street
Indianapolis, Indiana 46220

REVISIONS

Date	By	Description	Date	By	Description
1/95	H.D.R.	VALUE ENGINEERING CHANGE			

	Names	Dates
Drawn by	CWN	1/95
Checked by	J.L.S.	1/95
Designed by	B.F.H.	1/95
Checked by	H.D.R.	1/95
Approved by	R.CRAIG FINLEY JR.,P.E.	1/95

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FLORIDA DEPARTMENT OF TRANSPORTATION
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ROAD NO.	COUNTY	PROJECT NO.
SR 600	PINELLAS/HILLSBOROUGH	10130-3544

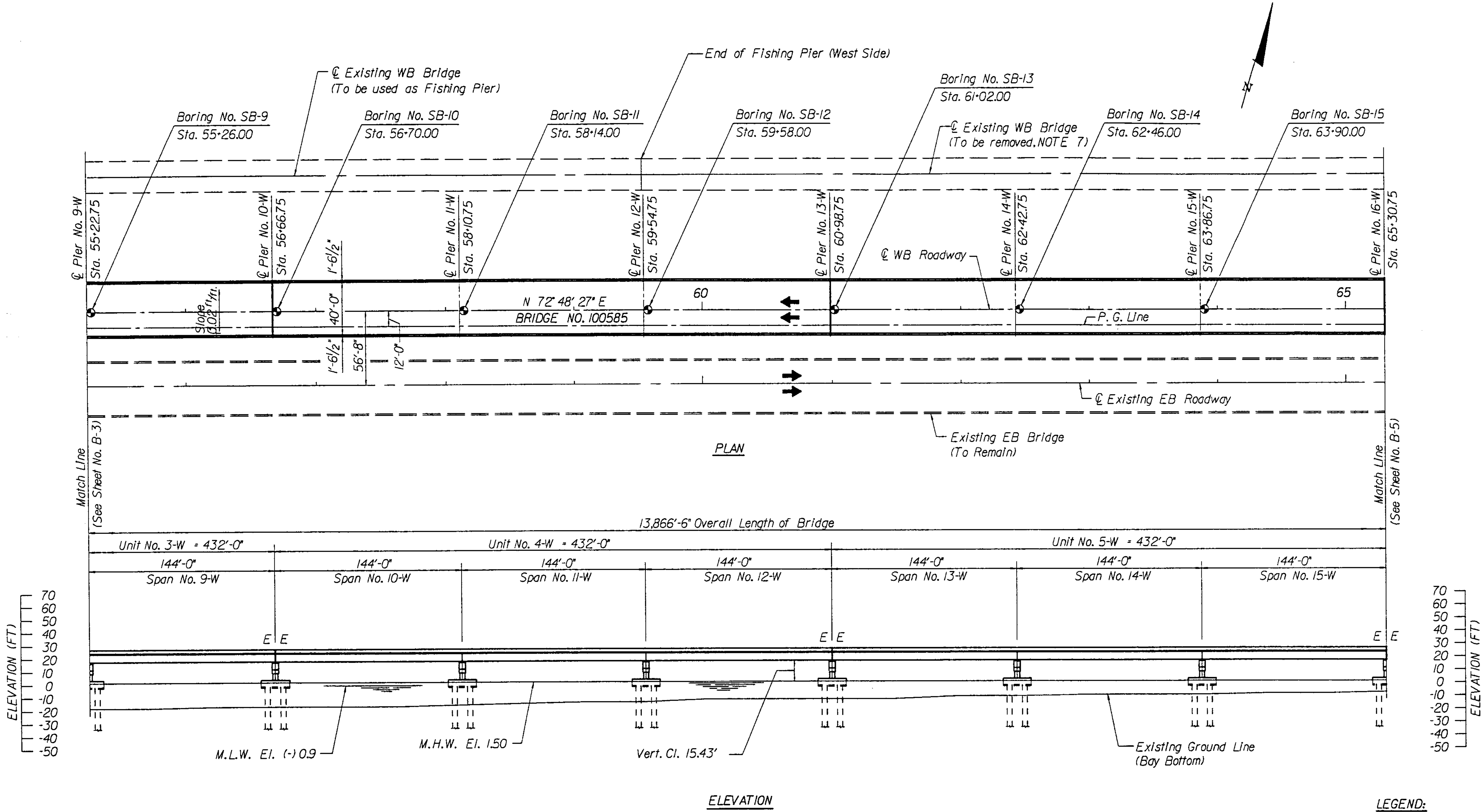
SHEET TITLE: PLAN AND ELEVATION (1)

Drawing No. FM/JS-7

PROJECT NAME: WESTBOUND GANDY BRIDGE
BRIDGE NO. 100585

Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-4



LEGEND:
E = Expansion
● = Core Boring

NOTE: For Plan and Elevation Notes, see Sheet B-3.

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2825 East 56th Street
Indianapolis, Indiana 46220

REVISIONS					
Date	By	Description	Date	By	Description
1/95	H.D.R.	VALUE ENGINEERING CHANGE			

	Names	Dates
Drawn by	C.W.N.	1/95
Checked by	J.L.S.	1/95
Designed by	B.F.H.	1/95
Checked by	H.D.R.	1/95
Approved by	R. CRAIG FINLEY JR., P.E.	

ENGINEER OF RECORD:
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SEAL:
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FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR 600	PINELLAS/HILLSBOROUGH	10130-3544

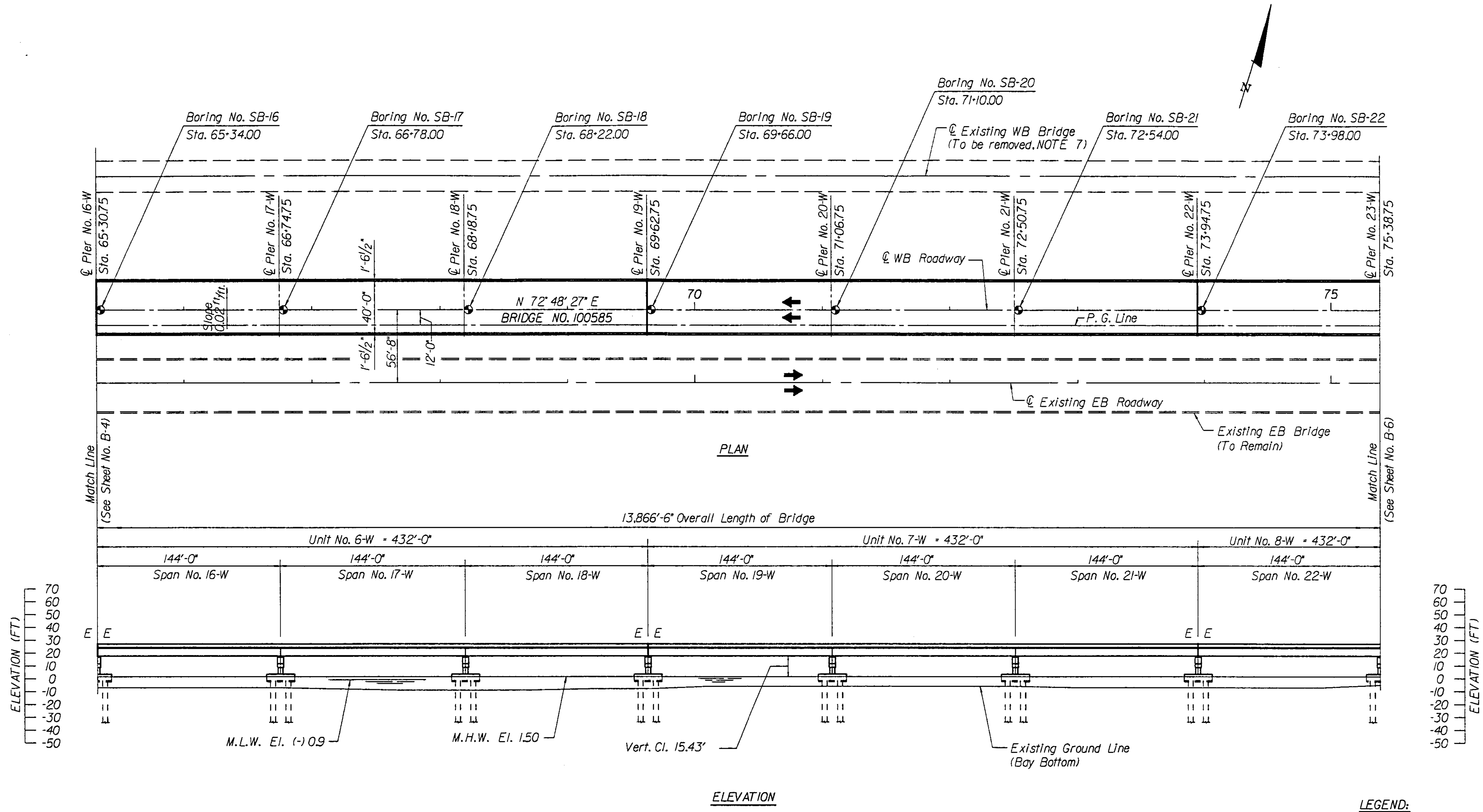
SHEET TITLE: PLAN AND ELEVATION (2)

Drawing No. FM/JS-8

PROJECT NAME: WESTBOUND GANDY BRIDGE

Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-5



LEGEND:
E = Expansion
◉ = Core Boring

NOTE: For Plan and Elevation Notes, see Sheet B-3.

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REVISIONS

Date	By	Description	Date	By	Description
1/95	H.D.R.	VALUE ENGINEERING CHANGE			

	Names	Dates
Drawn by	C.W.N.	1/95
Checked by	J.L.S.	1/95
Designed by	B.F.H.	1/95
Checked by	H.D.R.	1/95
Approved by	R.CRAIG FINLEY JR.,P.E.	

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FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 600	PINELLAS/HILLSBOROUGH	10130-3544

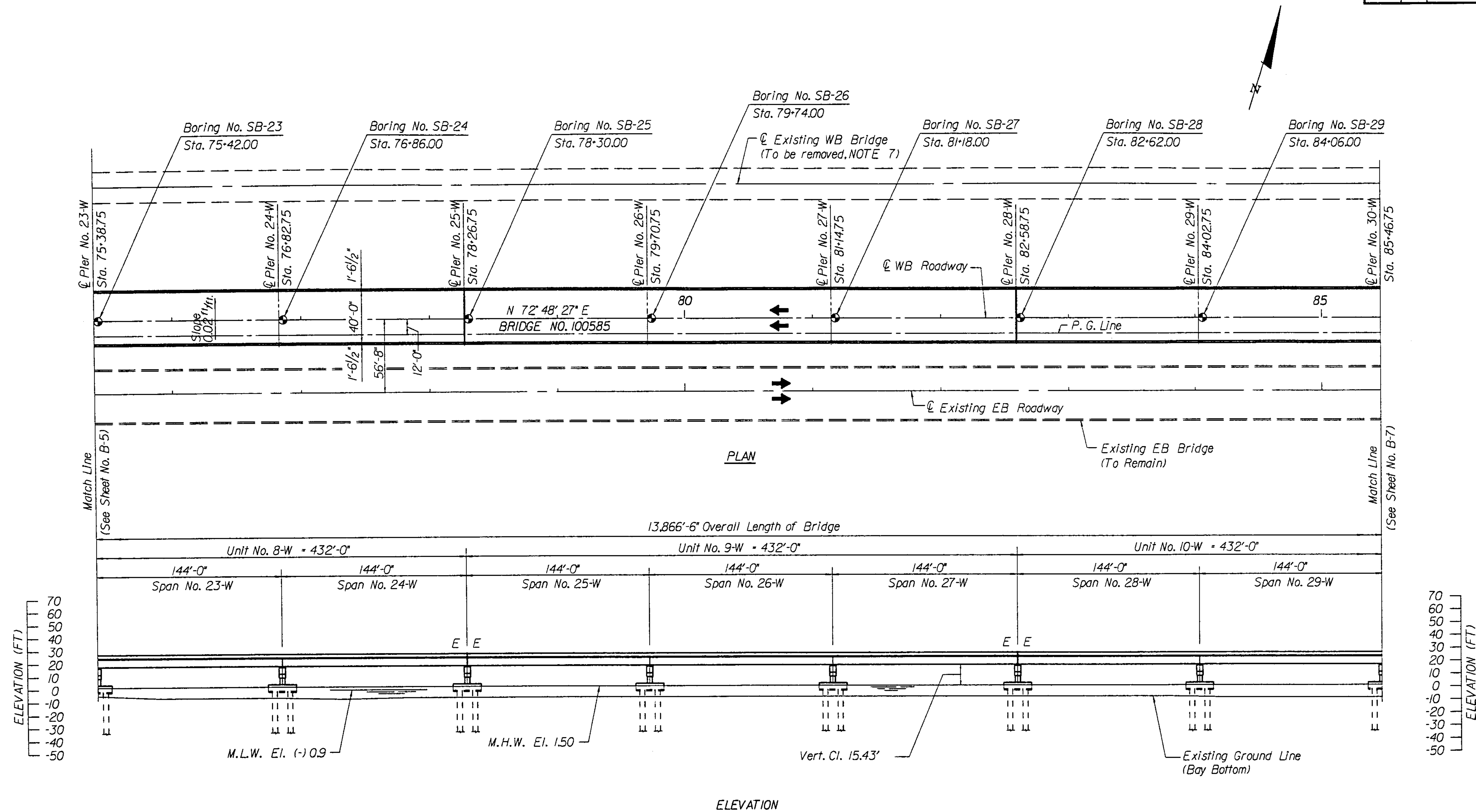
SHEET TITLE:
PLAN AND ELEVATION (3)

Drawing No. FM/JS-9

PROJECT NAME:
WESTBOUND GANDY BRIDGE

Index No.

FED. ROAD DIV NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-6



NOTE: For Plan and Elevation Notes, see Sheet B-3.

LEGEND:
E = Expansion
⊕ = Core Boring

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REVISIONS

Date	By	Description	Date	By	Description
1/95	H.D.R.	VALUE ENGINEERING CHANGE			

	Names	Dates
Drawn by	C.W.N.	1/95
Checked by	J.L.S.	1/95
Designed by	B.F.H.	1/95
Checked by	H.D.R.	1/95
Approved by	R. CRAIG FINLEY JR., P.E.	

ENGINEER OF RECORD:
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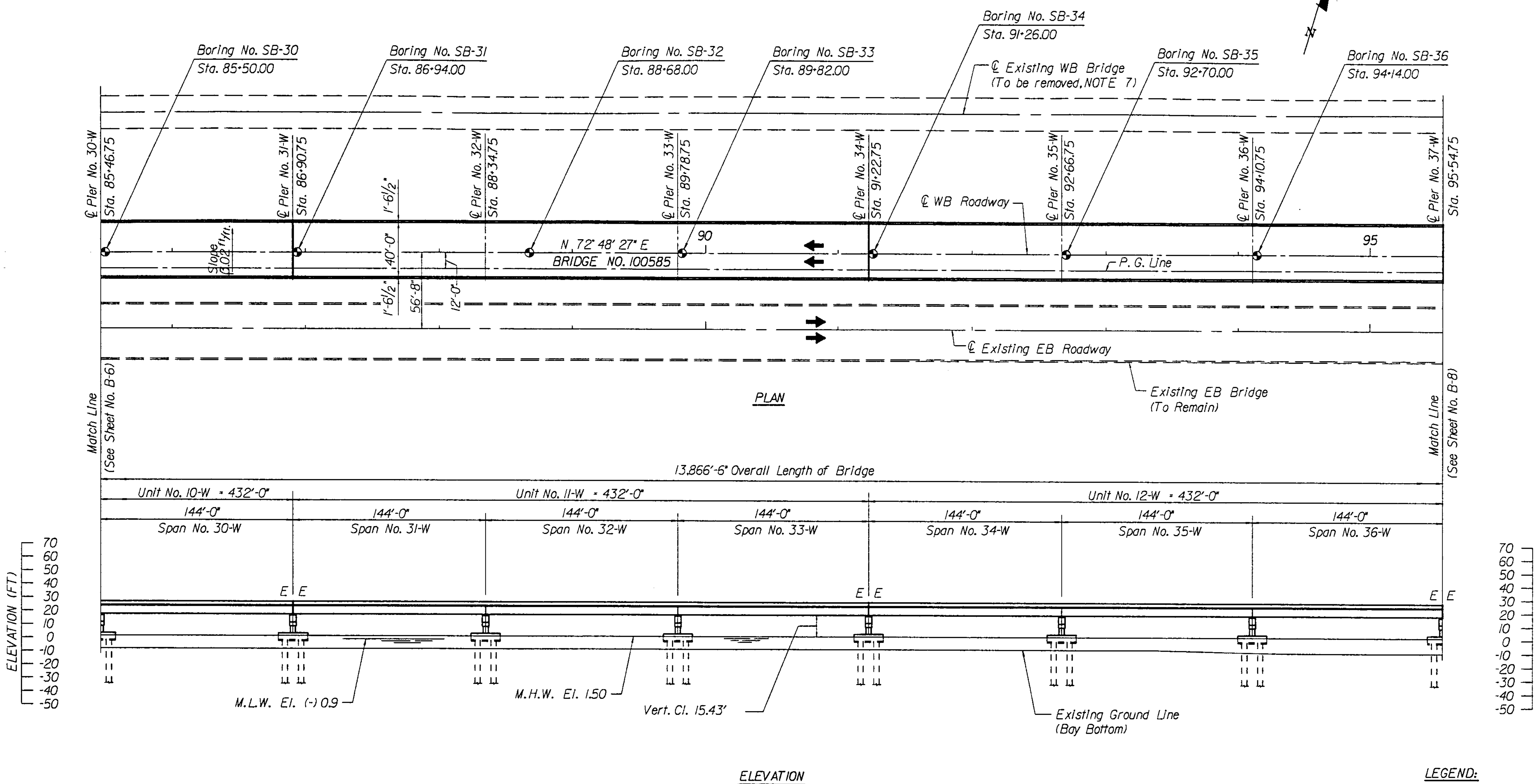
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR 600	PINELLAS/HILLSBOROUGH	10130-3544

SHEET TITLE: PLAN AND ELEVATION (4)
Drawing No. FM/JS-10
PROJECT NAME: WESTBOUND GANDY BRIDGE
Index No.



LEGEND:
E = Expansion
⊕ = Core Boring

NOTE: For Plan and Elevation Notes, see Sheet B-3.

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REVISIONS

Date	By	Description	Date	By	Description
1/95	H.D.R.	VALUE ENGINEERING CHANGE			

	Names	Dates
Drawn by	C.W.N.	1/95
Checked by	J.L.S.	1/95
Designed by	B.F.H.	1/95
Checked by	H.D.R.	1/95
Approved by	R.CRAIG FINLEY JR.,P.E.	

ENGINEER OF RECORD:
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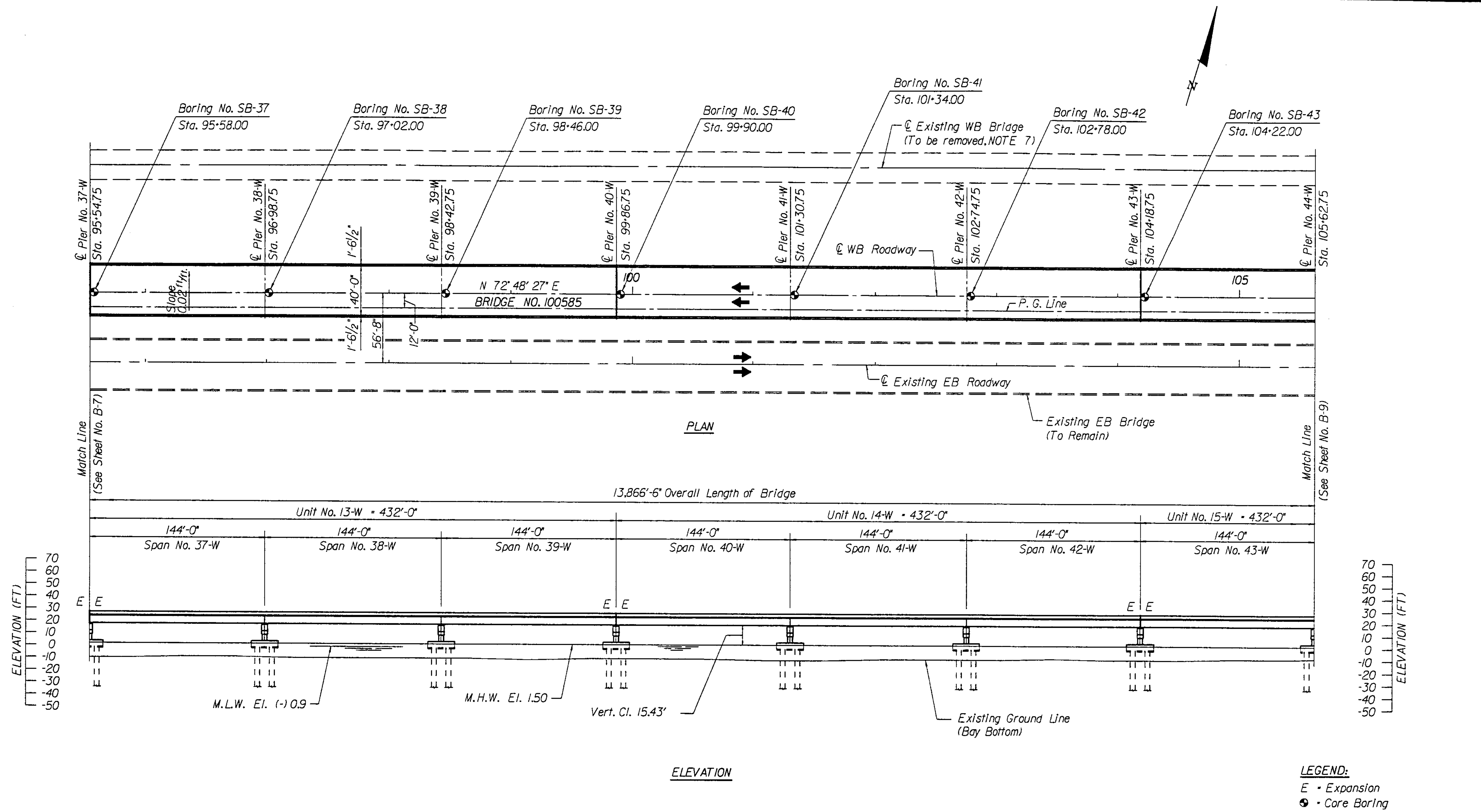
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR 600	PINELLAS/HILLSBOROUGH	10130-3544

SHEET TITLE: PLAN AND ELEVATION (5)
Drawing No. FM/JS-11
PROJECT NAME: WESTBOUND GANDY BRIDGE
Index No.

FLA. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-8



NOTE: For Plan and Elevation Notes, see Sheet B-3.

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Indianapolis, Indiana 46220

REVISIONS

Date	By	Description	Date	By	Description
1/95	H.D.R.	VALUE ENGINEERING CHANGE			

	Names	Dates
Drawn by	C.W.N.	1/95
Checked by	J.L.S.	1/95
Designed by	B.F.H.	1/95
Checked by	H.D.R.	1/95
Approved by	R.CRAIG FINLEY JR.,P.E.	

ENGINEER OF RECORD:
BEISWENGER, HOCH & ASSOCIATES, INC.
1190 N.E. 163rd. Street Suite 203
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CONTRACTOR:
misner marine construction, inc.
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Tampa, Florida 33681-3427

SEAL:
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FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR 600	PINELLAS/HILLSBOROUGH	10130-3544

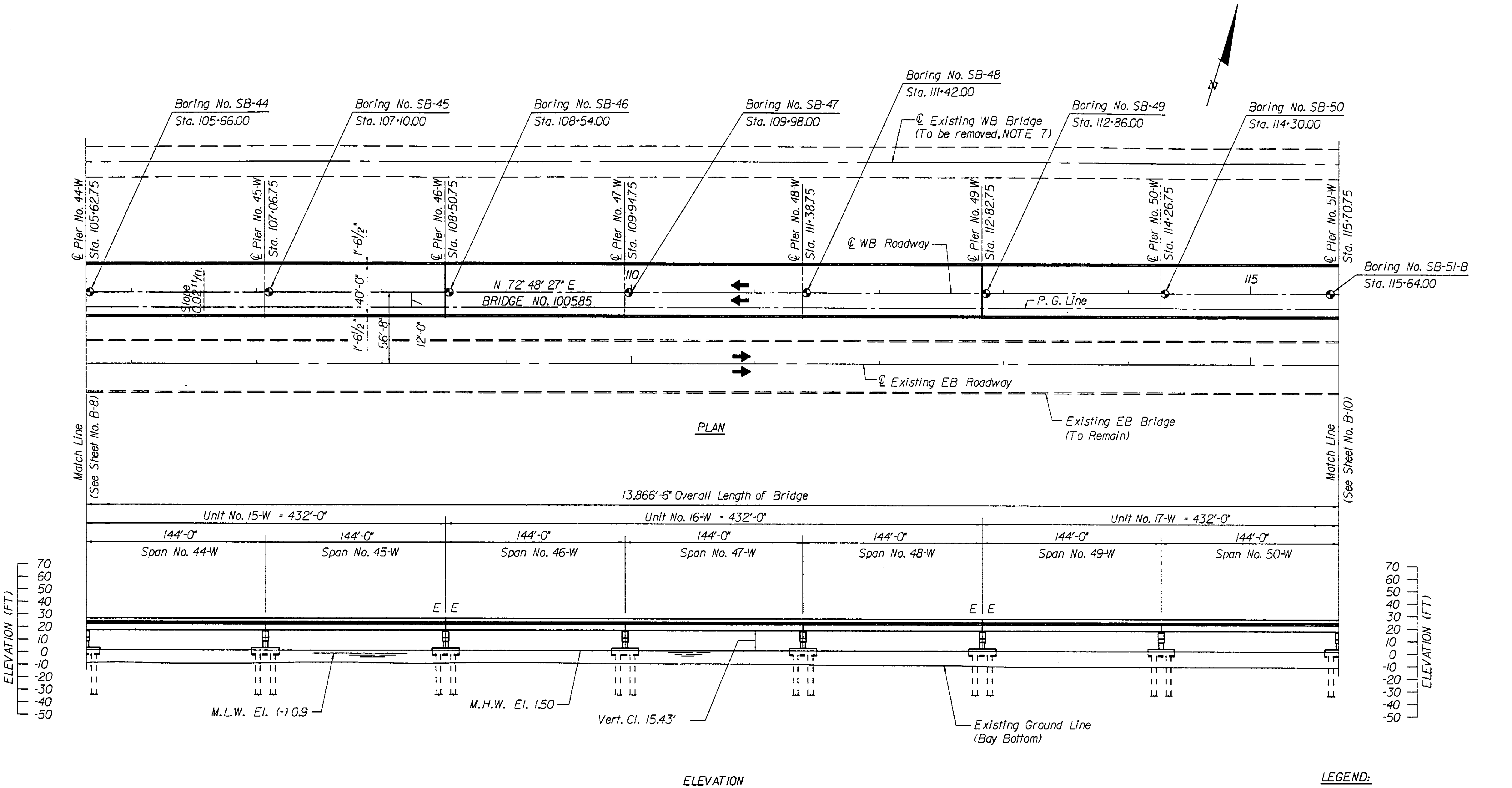
SHEET TITLE: PLAN AND ELEVATION (6)

Drawing No. FM/JS-12

PROJECT NAME: WESTBOUND GANDY BRIDGE

Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-9



LEGEND:
E - Expansion
⊛ - Core Boring

NOTE: For Plan and Elevation Notes, see Sheet B-3.

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Tallahassee, Florida 32312-1721

Janssen & Spaans Engineers, Inc.
2825 East 56th Street
Indianapolis, Indiana 46220

REVISIONS

Date	By	Description	Date	By	Description
1/95	H.D.R.	VALUE ENGINEERING CHANGE			

	Names	Dates
Drawn by	C.W.N.	1/95
Checked by	J.L.S.	1/95
Designed by	B.F.H.	1/95
Checked by	H.D.R.	1/95
Approved by	R.CRAIG FINLEY JR.,P.E.	

ENGINEER OF RECORD:
BEISWENGER, HOCH & ASSOCIATES, INC.
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North Miami Beach, Fl. 33162

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misner marine construction, inc.
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Tampa, Florida 33681-3427

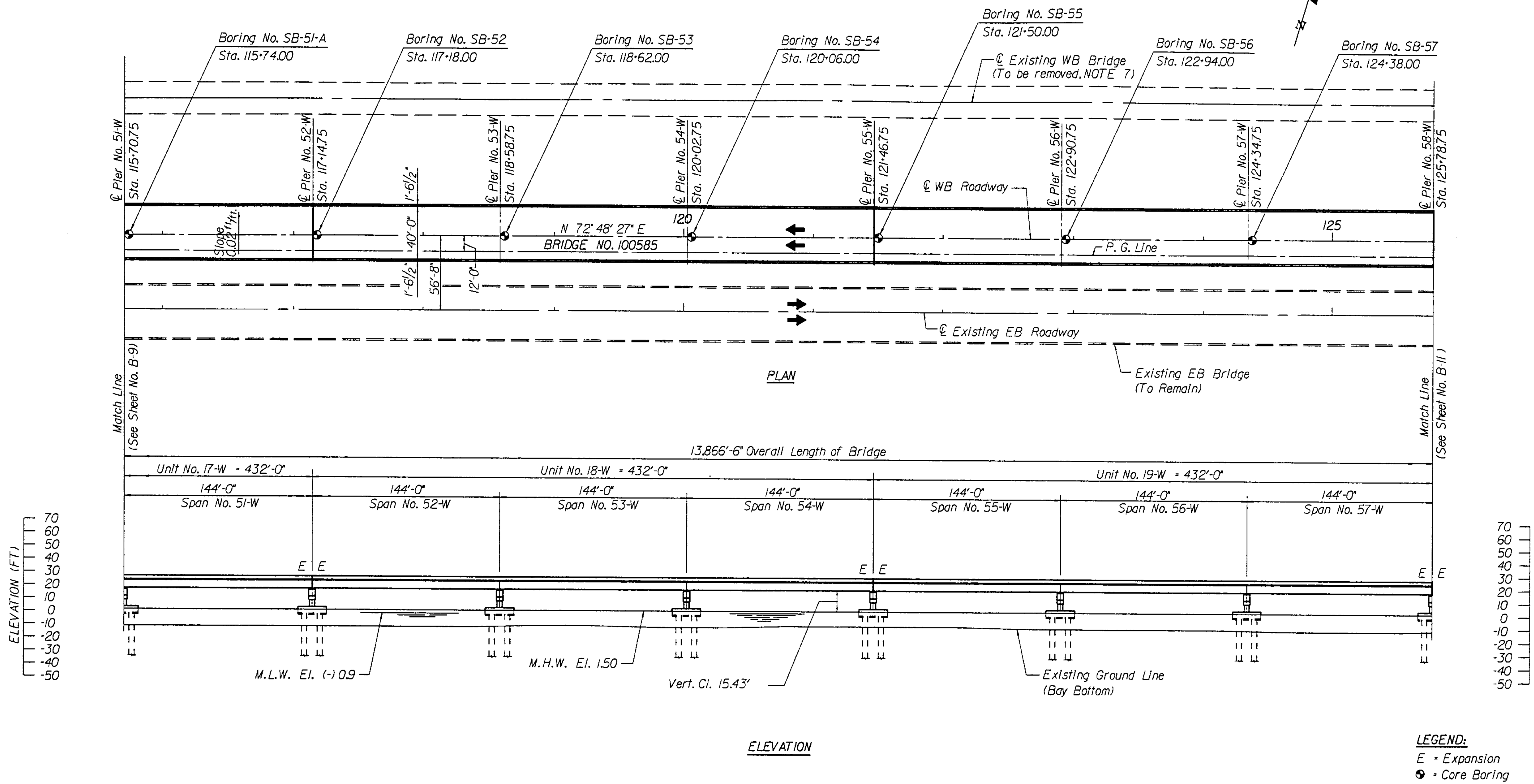
SEAL:
4-19-95

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR 600	PINELLAS/HILLSBOROUGH	10130-3544

SHEET TITLE: PLAN AND ELEVATION (7)
Drawing No. FM/JS-13
PROJECT NAME: WESTBOUND GANDY BRIDGE
Index No.

FLA. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-10



LEGEND:
E = Expansion
⊕ = Core Boring

NOTE: For Plan and Elevation Notes, see Sheet B-3.

FINLEY McNARY/JANSSEN SPAANS
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1391 Timberlane Road Suite 200
Tallahassee, Florida 32312-1721

Janssen & Spaans Engineers, Inc.
2825 East 56th Street
Indianapolis, Indiana 46220

REVISIONS

Date	By	Description	Date	By	Description
1/95	H.D.R.	VALUE ENGINEERING CHANGE			

	Names	Dates
Drawn by	C.W.N.	1/95
Checked by	J.L.S.	1/95
Designed by	B.F.H.	1/95
Checked by	H.D.R.	1/95
Approved by	R.CRAIG FINLEY JR.,P.E.	

ENGINEER OF RECORD:
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SEAL:
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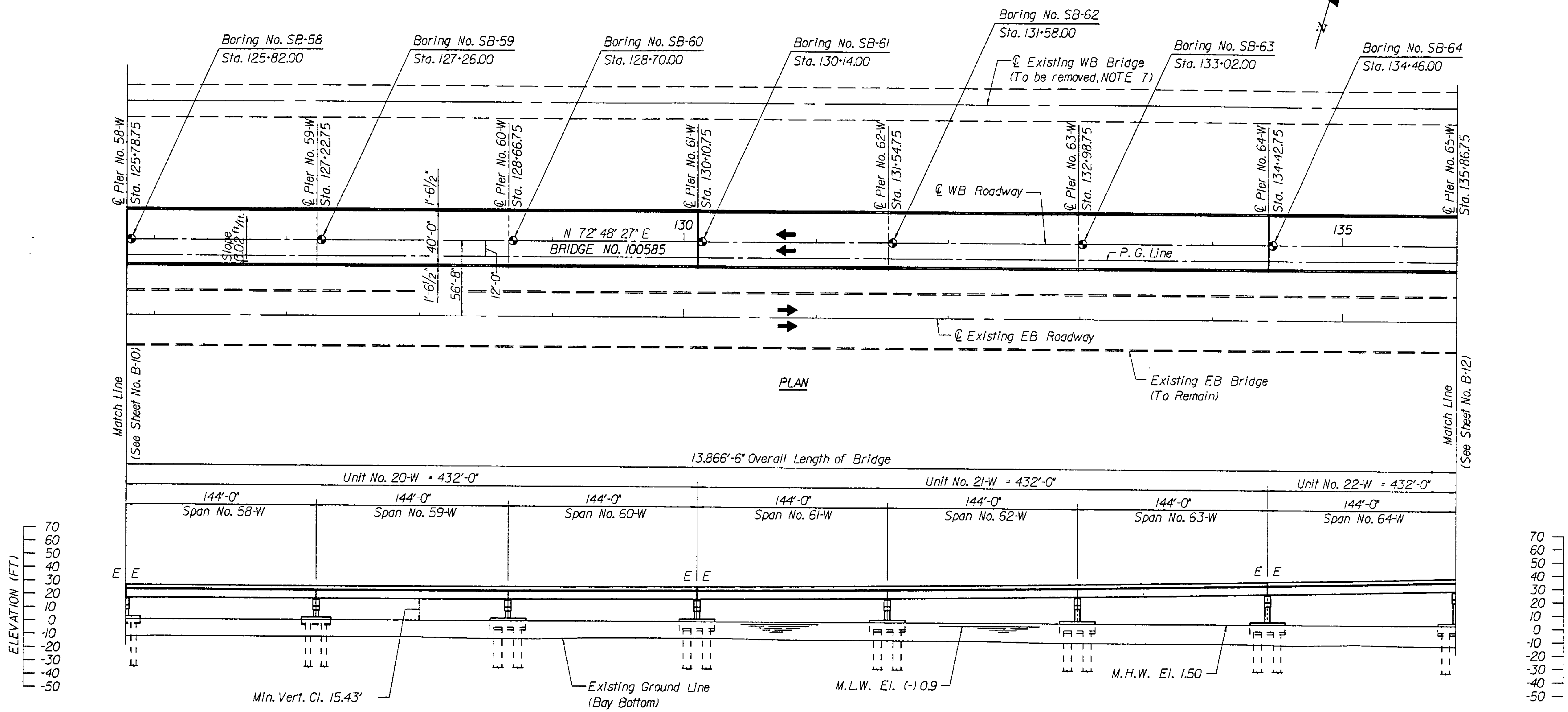
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR 600	PINELLAS/HILLSBOROUGH	10130-3544

SHEET TITLE: PLAN AND ELEVATION (8)
Drawing No. FM/JS-14
PROJECT NAME: WESTBOUND GANDY BRIDGE
Index No.

F.D. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-11



ELEVATION

LEGEND:
E = Expansion
⊕ = Core Boring

NOTE: For Plan and Elevation Notes, see Sheet B-3.

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REVISIONS

Date	By	Description	Date	By	Description
1/95	H.D.R.	VALUE ENGINEERING CHANGE			

	Names	Dates
Drawn by	C.W.N.	1/95
Checked by	J.L.S.	1/95
Designed by	B.F.H.	1/95
Checked by	H.D.R.	1/95
Approved by	R. CRAIG FINLEY JR., P.E.	

ENGINEER OF RECORD:
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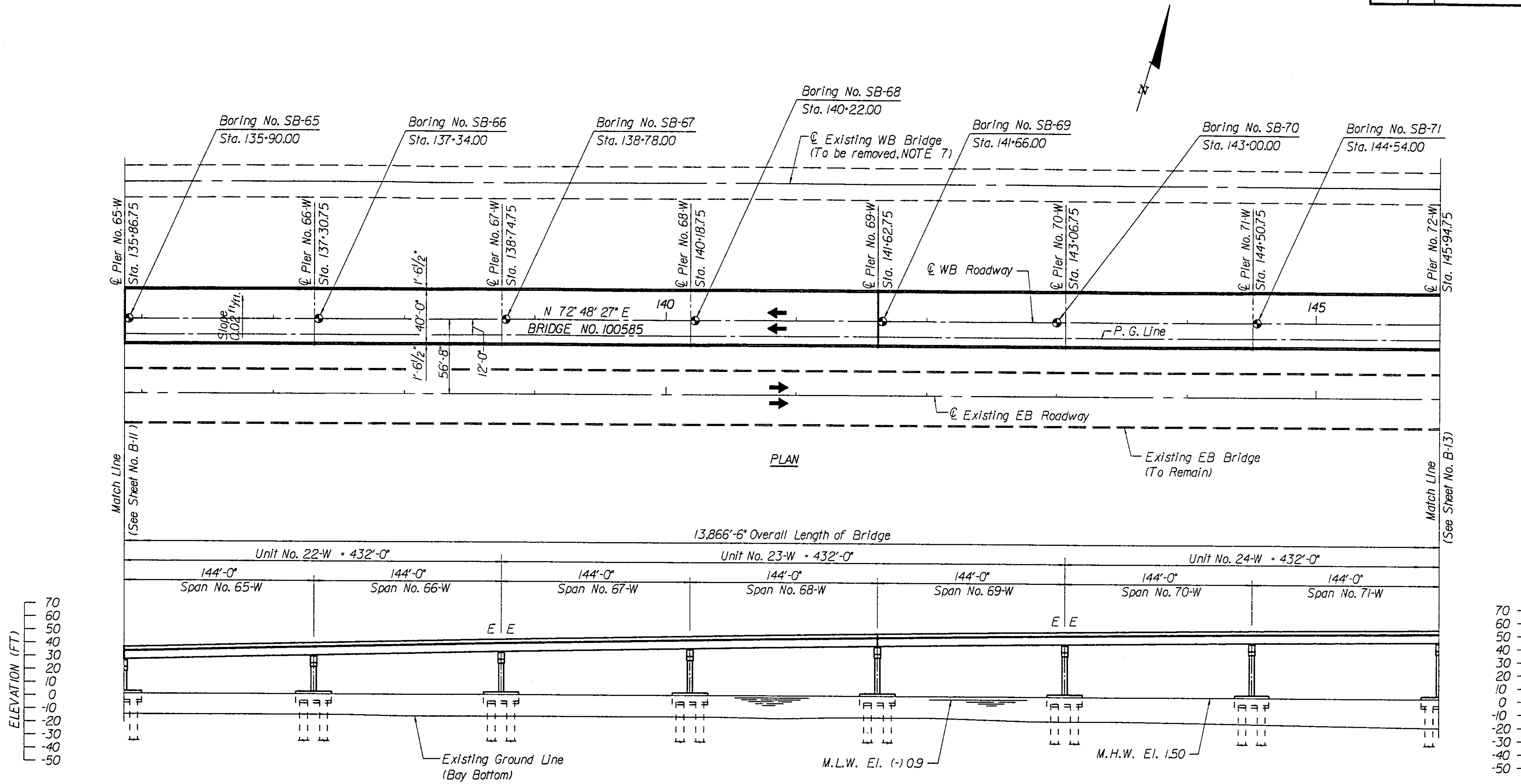
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR 600	PINELLAS/HILLSBOROUGH	10130-3544

SHEET TITLE: PLAN AND ELEVATION (9)
Drawing No. FM/JS-15
PROJECT NAME: WESTBOUND GANDY BRIDGE
Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-12



LEGEND:
E = Expansion
F = Fixed
⊕ = Core Boring

NOTE: For Plan and Elevation Notes, see Sheet B-3.

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	Names	Dates
Drawn by	C.W.H.	1/95
Checked by	J.L.S.	1/95
Designed by	B.F.H.	1/95
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Approved by	R. CRAIG FINLEY JR., P.E.	

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FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR 600	PINELLAS/HILLSBOROUGH	10130-3544

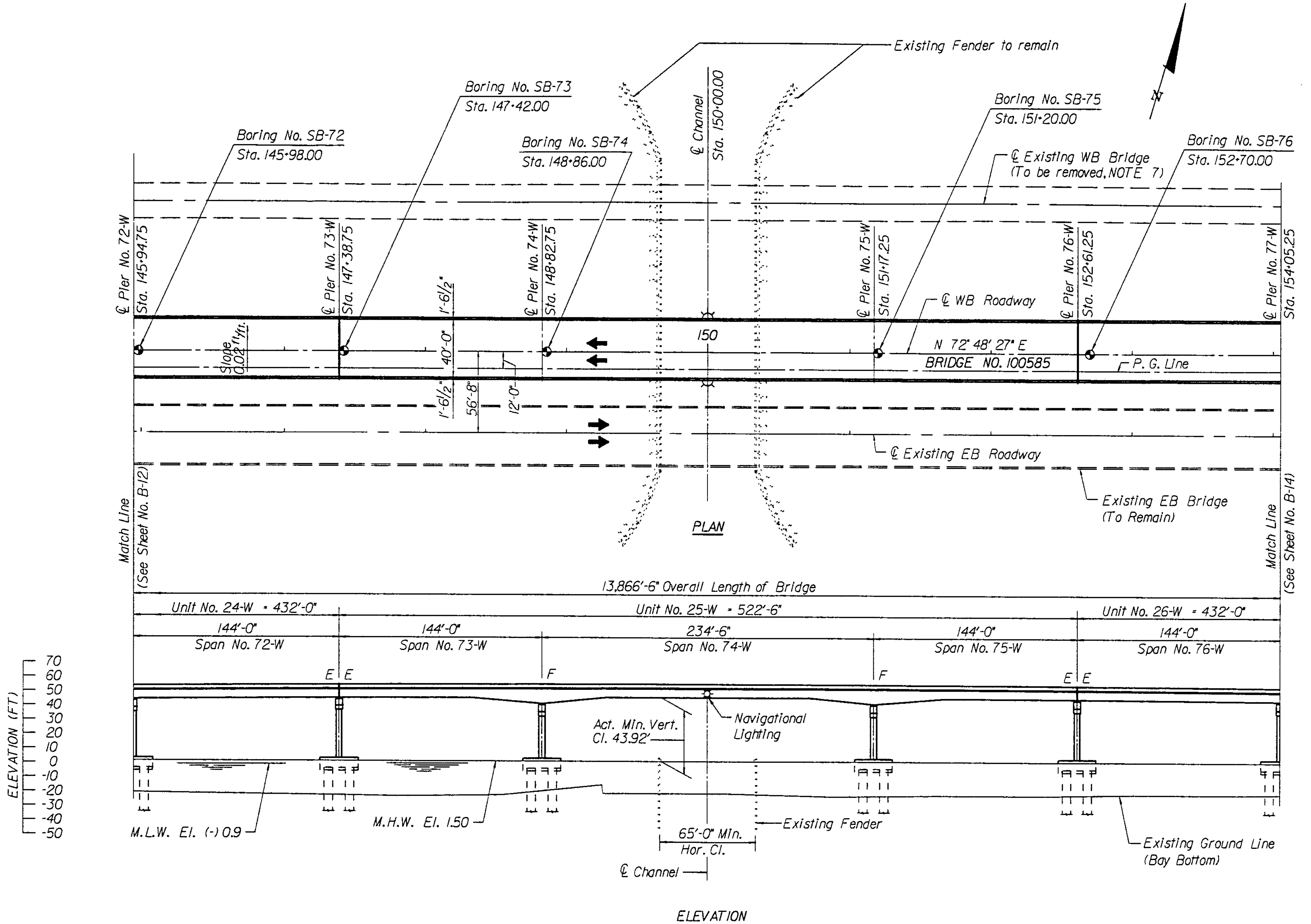
SHEET TITLE: PLAN AND ELEVATION (10)

Drawing No. FM/JS-16

PROJECT NAME: WESTBOUND GANDY BRIDGE

Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-13



NOTE: For Plan and Elevation Notes, see Sheet B-3.

LEGEND:
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F = Fixed
⊕ = Core Boring

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2825 East 56th Street
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REVISIONS

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	Names	Dates
Drawn by	C.W.N.	1/95
Checked by	J.L.S.	1/95
Designed by	B.F.H.	1/95
Checked by	H.D.R.	1/95
Approved by	R.CRAIG FINLEY JR.,P.E.	

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SEAL:
4-19-95

FLORIDA DEPARTMENT OF TRANSPORTATION
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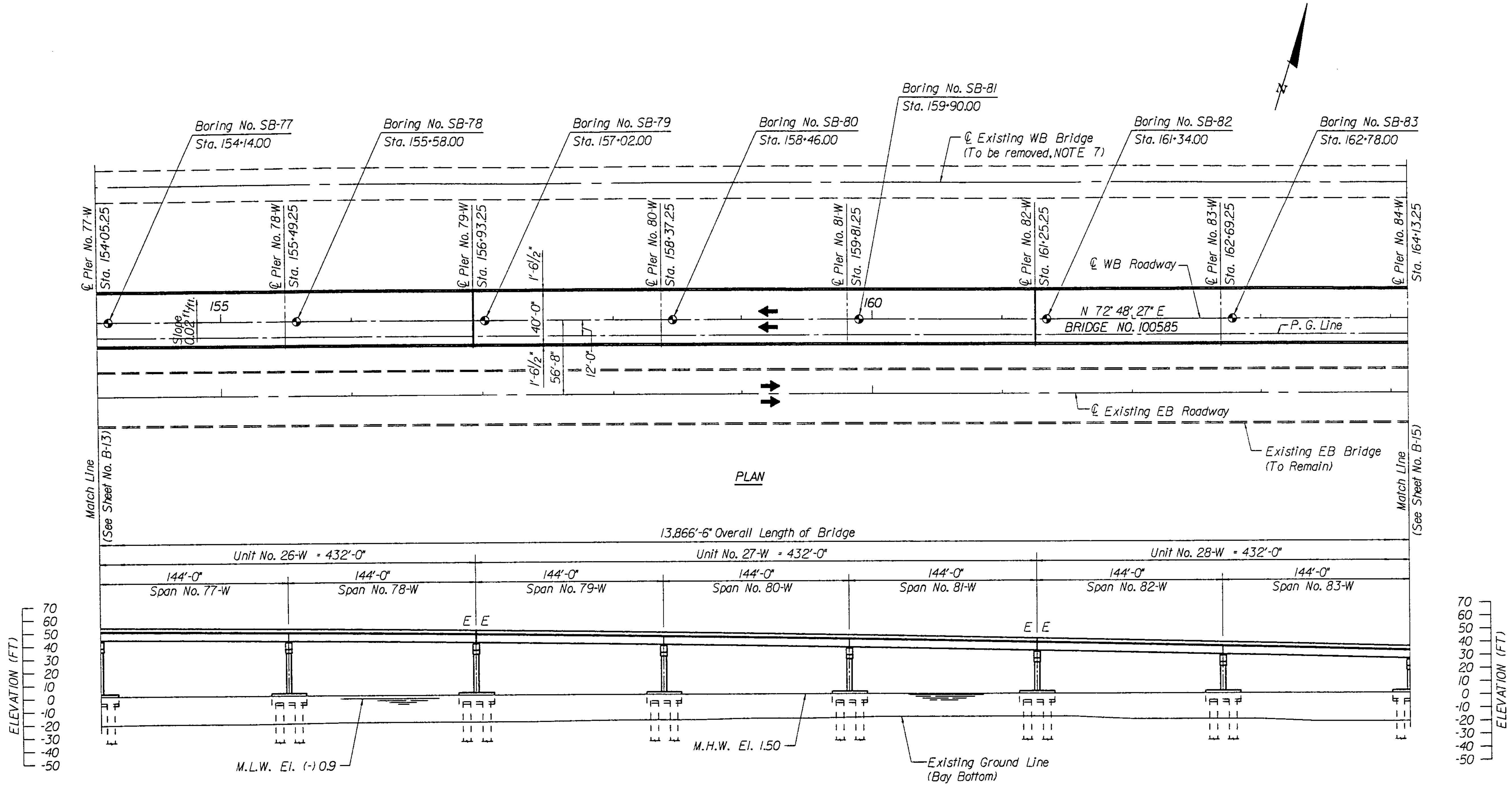
ROAD NO.	COUNTY	PROJECT NO.
SR 600	PINELLAS/HILLSBOROUGH	10130-3544

SHEET TITLE: PLAN AND ELEVATION (11)
Drawing No. FM/JS-17
PROJECT NAME: WESTBOUND GANDY BRIDGE
Index No.

FED. ROAD DIV NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-14



LEGEND:
E = Expansion
⊕ = Core Boring

NOTE: For Plan and Elevation Notes, see Sheet B-3.

FINLEY McNARY/JANSSEN SPAANS
a Joint Venture

Finley McNary Engineers, Inc
1391 Timberlane Road Suite 200
Tallahassee, Florida 32312-1721

Janssen & Spaans Engineers, Inc.
2825 East 56th Street
Indianapolis, Indiana 46220

REVISIONS					
Date	By	Description	Date	By	Description
1/95	H.D.R.	VALUE ENGINEERING CHANGE			

	Names	Dates
Drawn by	C.W.N.	1/95
Checked by	J.L.S.	1/95
Designed by	B.F.H.	1/95
Checked by	H.D.R.	1/95
Approved by	R.CRAIG FINLEY JR.,P.E.	

ENGINEER OF RECORD:
BEISWENGER, HOCH & ASSOCIATES, INC.
1190 N.E. 163rd. Street Suite 203
North Miami Beach, Fl. 33162

CONTRACTOR:
misner marine construction, inc.
5440 West Tyson Avenue
Post Office Box 13427
Tampa, Florida 33681-3427

SEAL:
4-19-95

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR 600	PINELLAS/HILLSBOROUGH	10130-3544

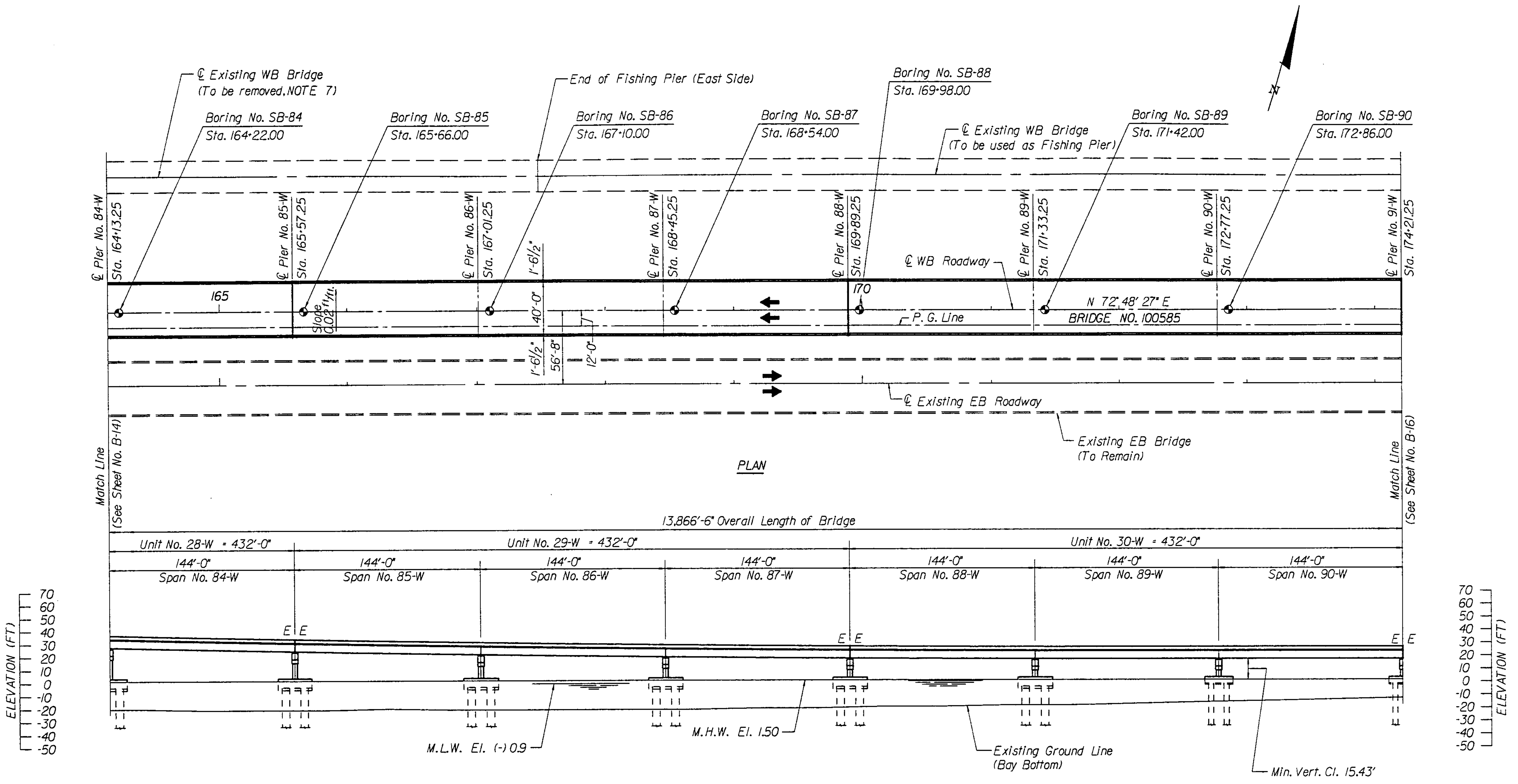
SHEET TITLE: PLAN AND ELEVATION (12)

Drawing No. FM/JS-18

PROJECT NAME: WESTBOUND GANDY BRIDGE

Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-15



ELEVATION

LEGEND:
E - Expansion
⊕ - Core Boring

NOTE: For Plan and Elevation Notes, see Sheet B-3.

FINLEY McNARY/JANSSEN SPAANS
a Joint Venture

Finley McNary Engineers, Inc
1391 Timberlane Road Suite 200
Tallahassee, Florida 32312-1721

Janssen & Spaans Engineers, Inc.
2825 East 56th Street
Indianapolis, Indiana 46220

REVISIONS					
Date	By	Description	Date	By	Description
1/95	H.D.R.	VALUE ENGINEERING CHANGE			

	Names	Dates
Drawn by	C.W.N.	1/95
Checked by	J.L.S.	1/95
Designed by	B.F.H.	1/95
Checked by	H.D.R.	1/95
Approved by	R.CRAIG FINLEY JR.,P.E.	

ENGINEER OF RECORD:
BEISWENGER, HOCH & ASSOCIATES, INC.
1190 N.E. 163rd. Street Suite 203
North Miami Beach, Fl. 33162

CONTRACTOR:
misner marine construction, inc.
5440 West Tyson Avenue
Post Office Box 13427
Tampa, Florida 33681-3427

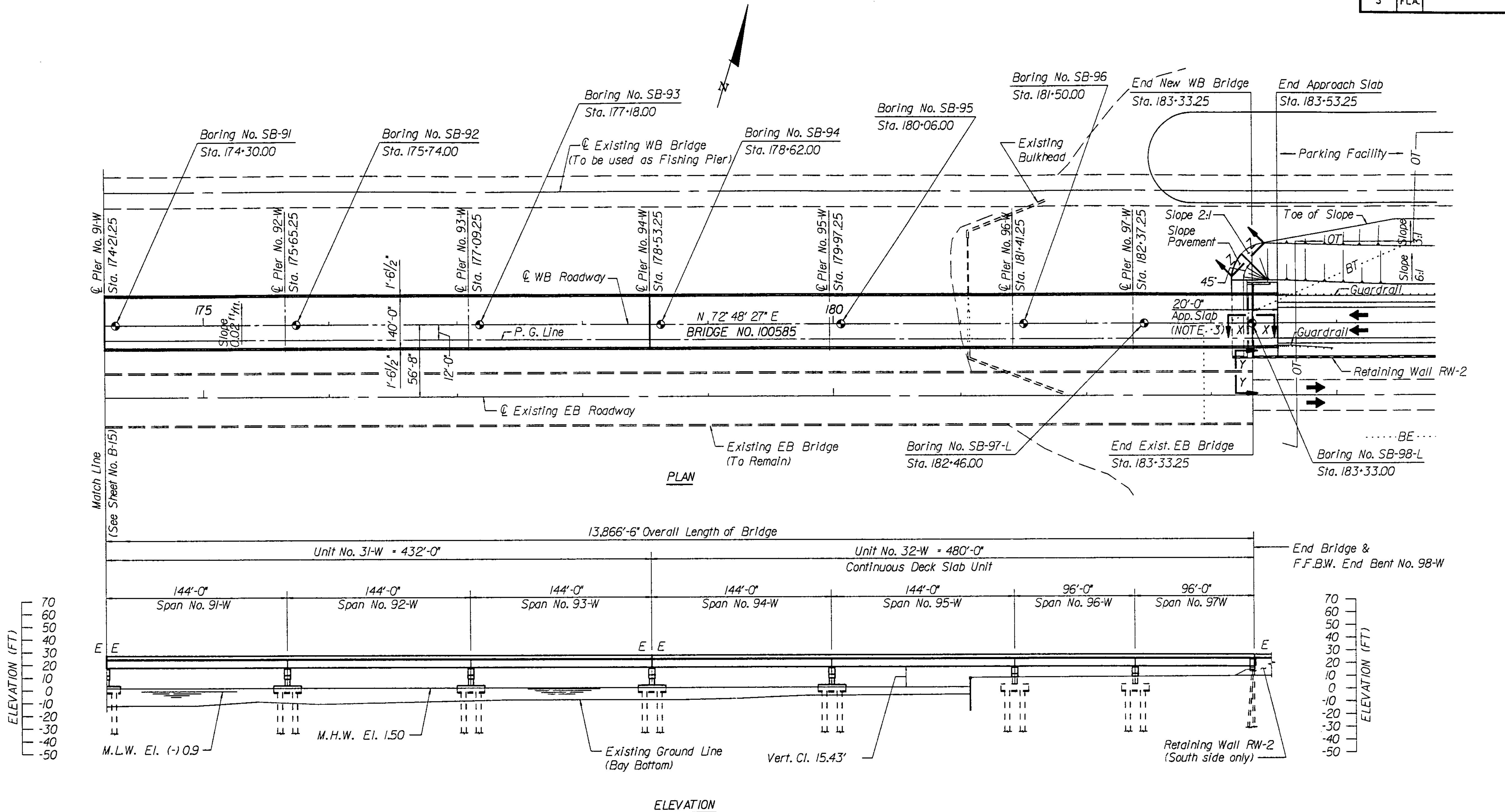
SEAL:
4-19-95

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR 600	PINELLAS/HILLSBOROUGH	10130-3544

SHEET TITLE: PLAN AND ELEVATION (13)
Drawing No. FM/JS-19
PROJECT NAME: WESTBOUND GANDY BRIDGE
Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-16



NOTE: For Plan and Elevation Notes, see Sheet B-3.

LEGEND:
E = Expansion
● = Core Boring
F.F.B.W. = Front Face of Backwall

FINLEY McNARY/JANSSEN SPAANS
a Joint Venture

Finley McNary Engineers, Inc
1391 Timberlane Road Suite 200
Tallahassee, Florida 32312-1721

Janssen & Spaans Engineers, Inc.
2825 East 56th Street
Indianapolis, Indiana 46220

REVISIONS					
Date	By	Description	Date	By	Description
1/95	H.D.R.	VALUE ENGINEERING CHANGE			

	Names	Dates
Drawn by	C.W.N.	1/95
Checked by	J.L.S.	1/95
Designed by	B.F.H.	1/95
Checked by	H.D.R.	1/95
Approved by	R.CRAIG FINLEY JR.,P.E.	

ENGINEER OF RECORD:
BEISWENGER, HOCH & ASSOCIATES, INC.
1190 N.E. 163rd. Street Suite 203
North Miami Beach, Fl. 33162

CONTRACTOR:
misner marine construction, inc.
5440 West Tyson Avenue
Post Office Box 13427
Tampa, Florida 33681-3427

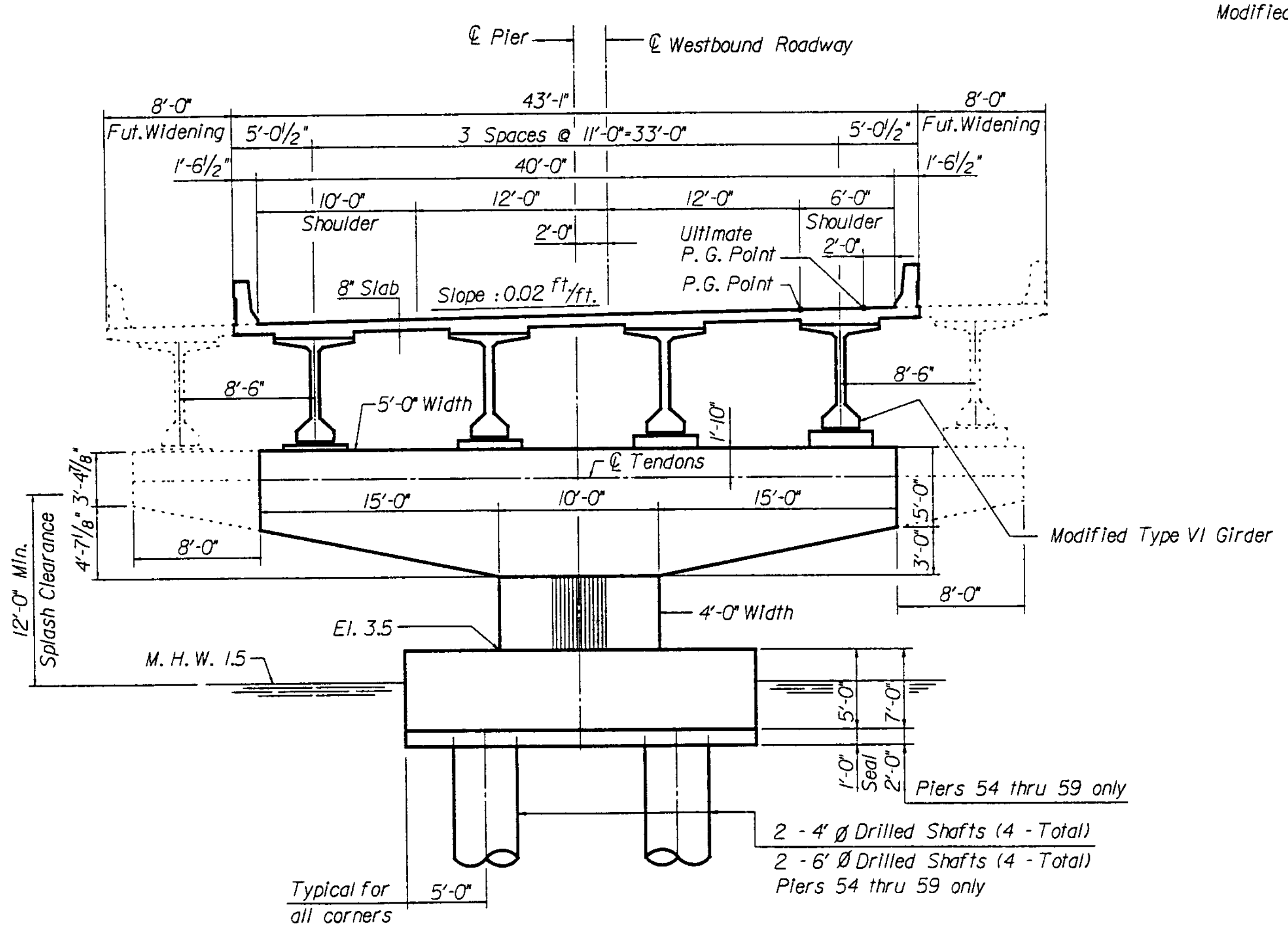
SEAL:
4-19-95

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR 600	PINELLAS/HILLSBOROUGH	10130-3544

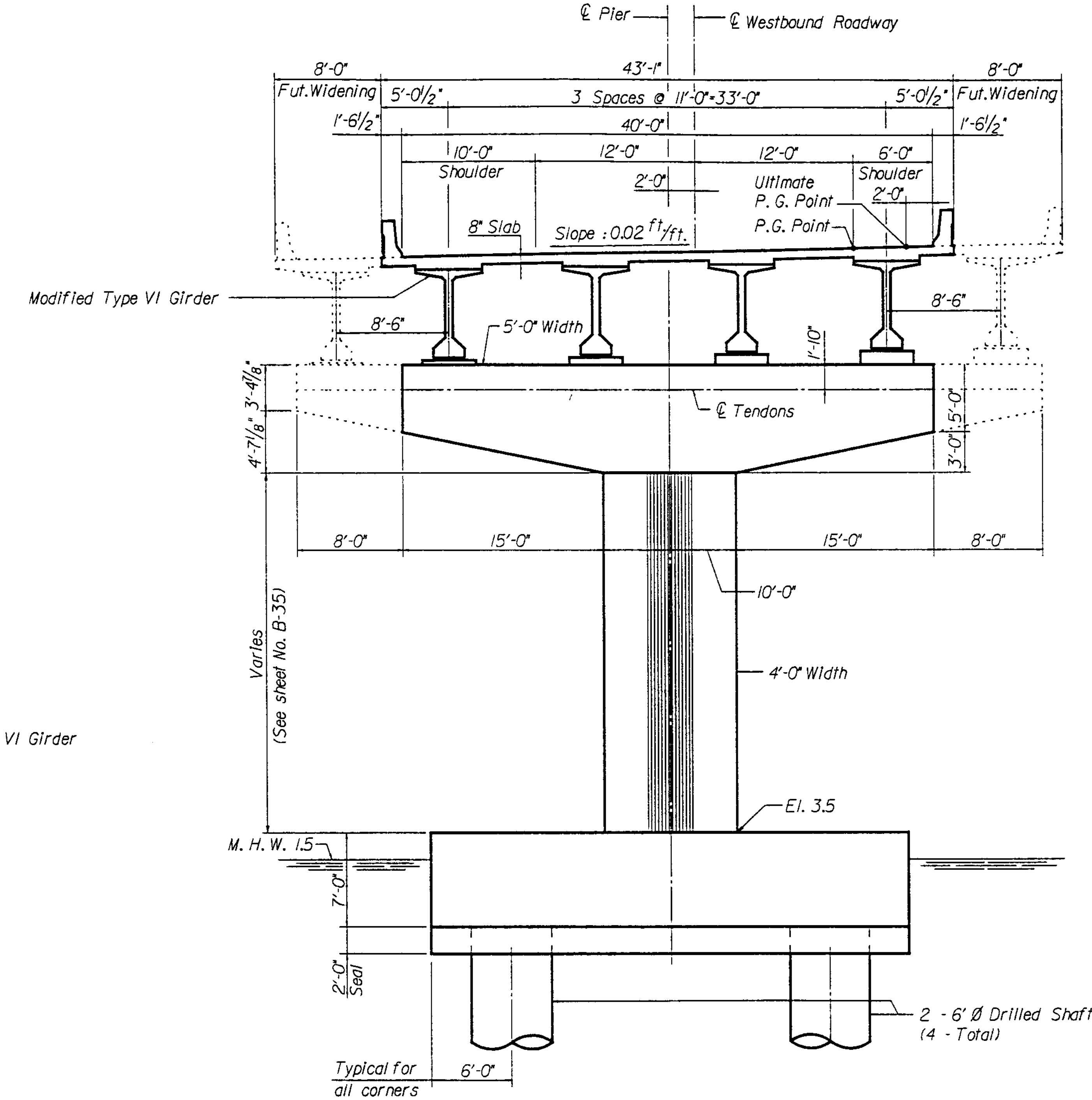
SHEET TITLE: PLAN AND ELEVATION (14)
Drawing No. FM/JS-20
PROJECT NAME: WESTBOUND GANDY BRIDGE
Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-17



LOW-LEVEL PIERS

(Piers 2 through 59 and 90 through 97)



MEDIUM-LEVEL PIERS

(Piers 60 through 69 and 80 through 89)

NOTES: 1) Solid lines denote construction of 40' roadway.
Dashed lines denote future widening to 56' roadway.
2) Future Bridge Widening "NOT IN CONTRACT".
For Pier Ducts and Shear Keys, see sheet No. A-4 and Pier Drawings.

FINLEY McNARY/JANSSEN SPAANS
a Joint Venture

Finley McNary Engineers, Inc
1391 Timberlane Road Suite 200
Tallahassee, Florida 32312-1721

Janssen & Spaans Engineers, Inc.
2825 East 56th Street
Indianapolis, Indiana 46220

REVISIONS

Date	By	Description	Date	By	Description
1/95	H.D.R.	VALUE ENGINEERING CHANGE			

	Names	Dates
Drawn by	J.L.S.	1/95
Checked by	C.W.N.	1/95
Designed by	B.F.H.	1/95
Checked by	H.D.R.	1/95
Approved by	R.CRAIG FINLEY JR.,P.E.	

ENGINEER OF RECORD:
BEISWENGER, HOCH & ASSOCIATES, INC.
1190 N.E. 163rd. Street Suite 203
North Miami Beach, Fl. 33162

CONTRACTOR:
misner marine construction, inc.
5440 West Tyson Avenue
Post Office Box 13427
Tampa, Florida 33681-3427

SEAL:
4-19-95

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR 600	PINELLAS/HILLSBOROUGH	10130-3544

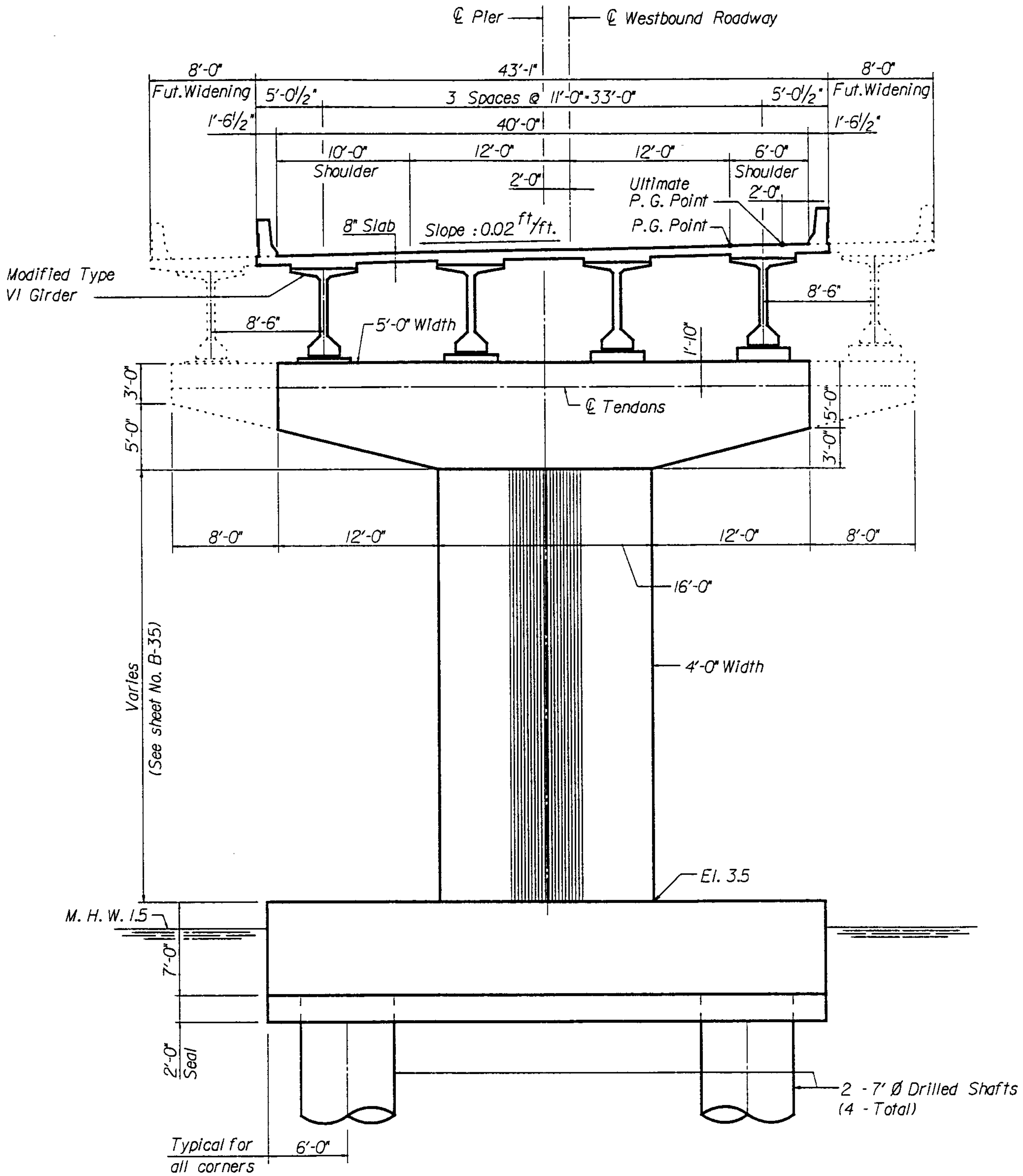
SHEET TITLE: TYPICAL CROSS-SECTIONS (1)

Drawing No. FM/JS-21

PROJECT NAME: WESTBOUND GANDY BRIDGE

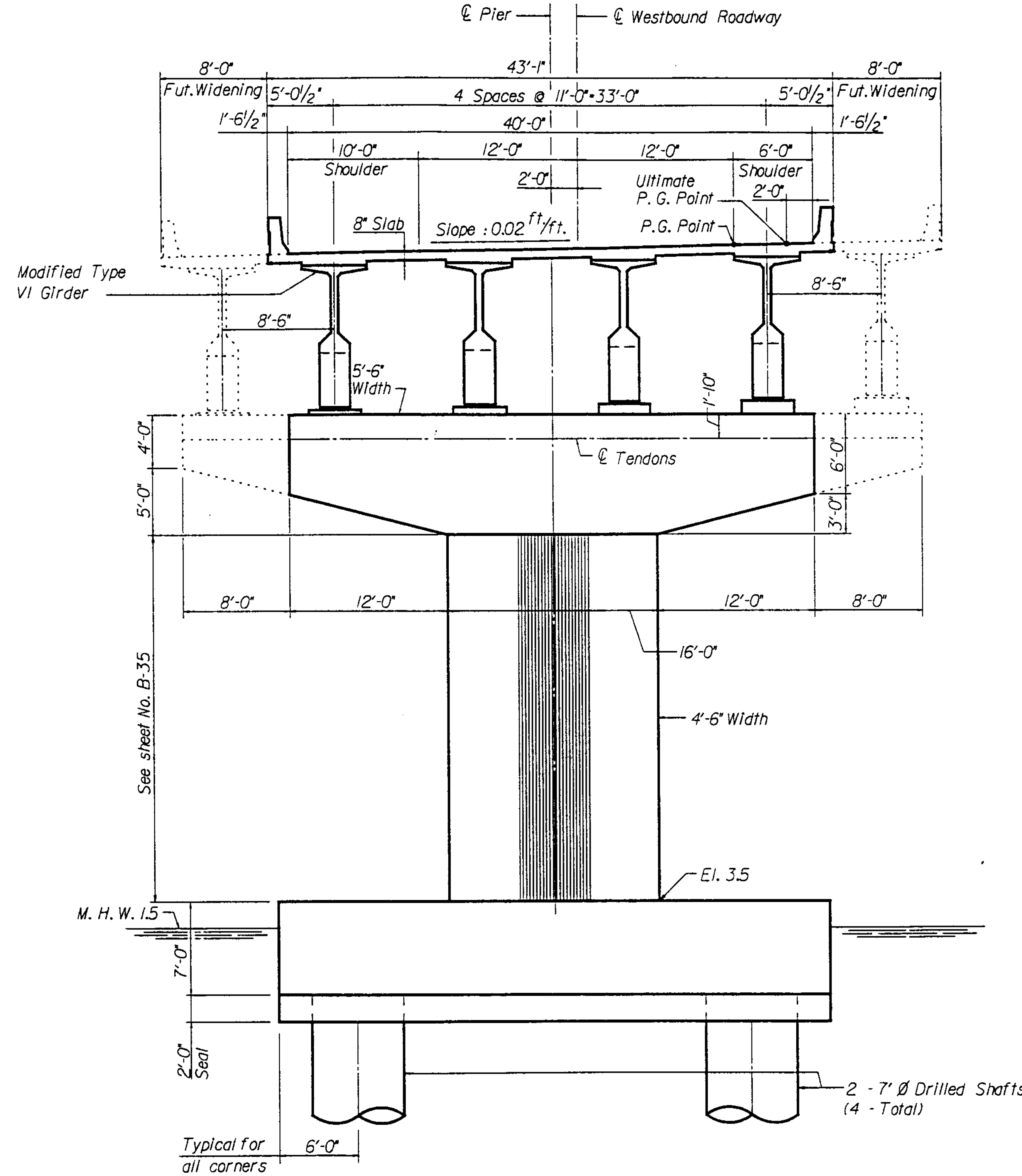
Index No.

FLA. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-18



HIGH-LEVEL PIERS

(Piers 70 through 73 and 76 through 79)



CHANNEL PIERS

(Piers 74 and 75)

NOTES: 1) Solid lines denote construction of 40' roadway.
Dashed lines denote future widening to 56' roadway.
2) Future Bridge Widening "NOT IN CONTRACT".
For Pier Ducts and Shear Keys, see sheet No. A-4 and Pier Drawings.

FINLEY McNARY/JANSSEN SPAANS
a Joint Venture

Finley McNary Engineers, Inc
1391 Timberlane Road Suite 200
Tallahassee, Florida 32312-1721

Janssen & Spaans Engineers, Inc.
2825 East 56th Street
Indianapolis, Indiana 46220

REVISIONS					
Date	By	Description	Date	By	Description
1/95	H.D.R.	VALUE ENGINEERING CHANGE			

	Names	Dates
Drawn by	J.L.S.	1/95
Checked by	C.W.N.	1/95
Designed by	B.F.H	1/95
Checked by	H.D.R.	1/95
Approved by	R.CRAIG FINLEY JR.,P.E.	

ENGINEER OF RECORD:
BEISWENGER, HOCH & ASSOCIATES, INC.
1190 N.E. 163rd. Street Suite 203
North Miami Beach, Fl. 33162

CONTRACTOR:
misner marine construction, inc.
5440 West Tyson Avenue
Post Office Box 13427
Tampa, Florida 33681-3427

SEAL:
4-19-95

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

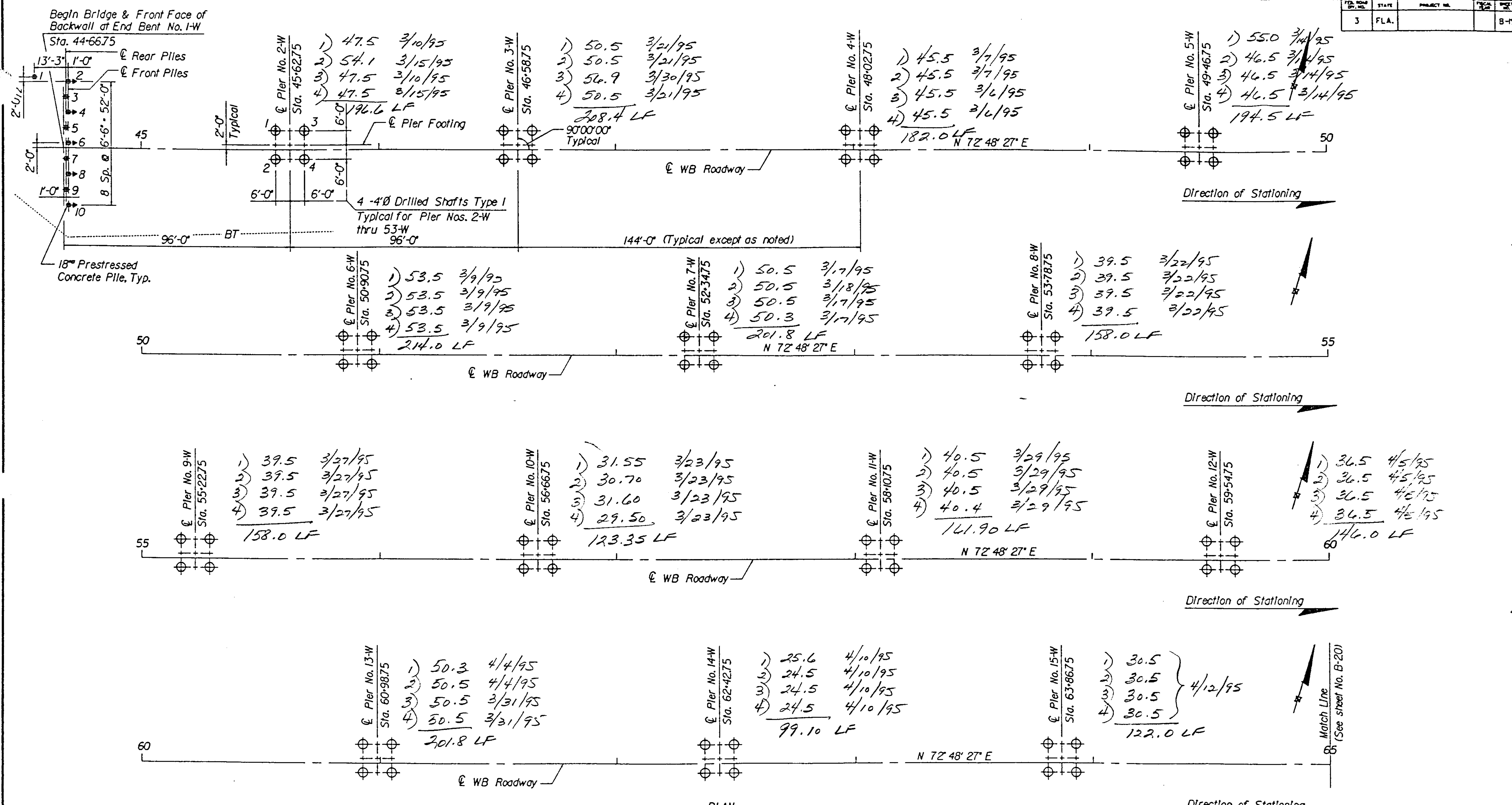
ROAD NO.	COUNTY	PROJECT NO.
SR 600	PINELLAS/HILLSBOROUGH	10130-3544

SHEET TITLE: TYPICAL CROSS-SECTIONS (2)

Drawing No. FM/JS-22

PROJECT NAME: WESTBOUND GANDY BRIDGE

Index No.



PLAN

NOTES FOR FOUNDATION LAYOUT

1. 18" Square Prestressed Concrete Piles for End Bents only. For Pile details see Sheet Nos. A-30 and A-31.
   ⊕ Denotes plumb Pile.
   ⊕→ Denotes Piles battered 2" per foot in the direction shown.
2. Drive one test pile in the position of a permanent plumb pile, a length of 84 ft. The test pile shall be dynamically monitored using the Pile Driver Analyzer.
3. All piers are supported on Concrete Drilled Shafts.
4. See Sheet No. A-3 for general notes about "FOUNDATION".
5. See Sheet No. A-4 for "DRILLED SHAFT GENERAL NOTES".
6. See Sheet No. A-5 for "DRILLED SHAFT INSTALLATION TABLES".
7. Each pile or drilled shaft number is to be prefixed by the respective End Bent or Pier number. Thus Drilled Shaft 1 of Pier No. 2 would be logged as 2-W (1).
8. See Sheet Nos. A-6 & A-7 for details of 4'Ø, 6'Ø and 7'Ø Drilled Shafts.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-19

ALTERNATE I

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	E.A.P.	1-93
Checked by	B.O.K.	9-93
Designed by	S.S.	1-93
Checked by	B.O.K.	9-93
Approved by	S.H.W.	

ENGINEER OF RECORD:
BEISWENGER, HOCH & ASSOCIATES, INC.
1190 N.E. 163rd Street Suite 203
North Miami Beach, FL 33162

BEISWENGER, HOCH & ASSOCIATES, INC.
CONSULTING ENGINEERS & PLANNERS

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 600	PINELLAS/HILLSBOROUGH	10130-3544

SHEET TITLE: FOUNDATION LAYOUT (1)

PROJECT NAME: WESTBOUND GANDY BRIDGE

Drawing No.

Index No.

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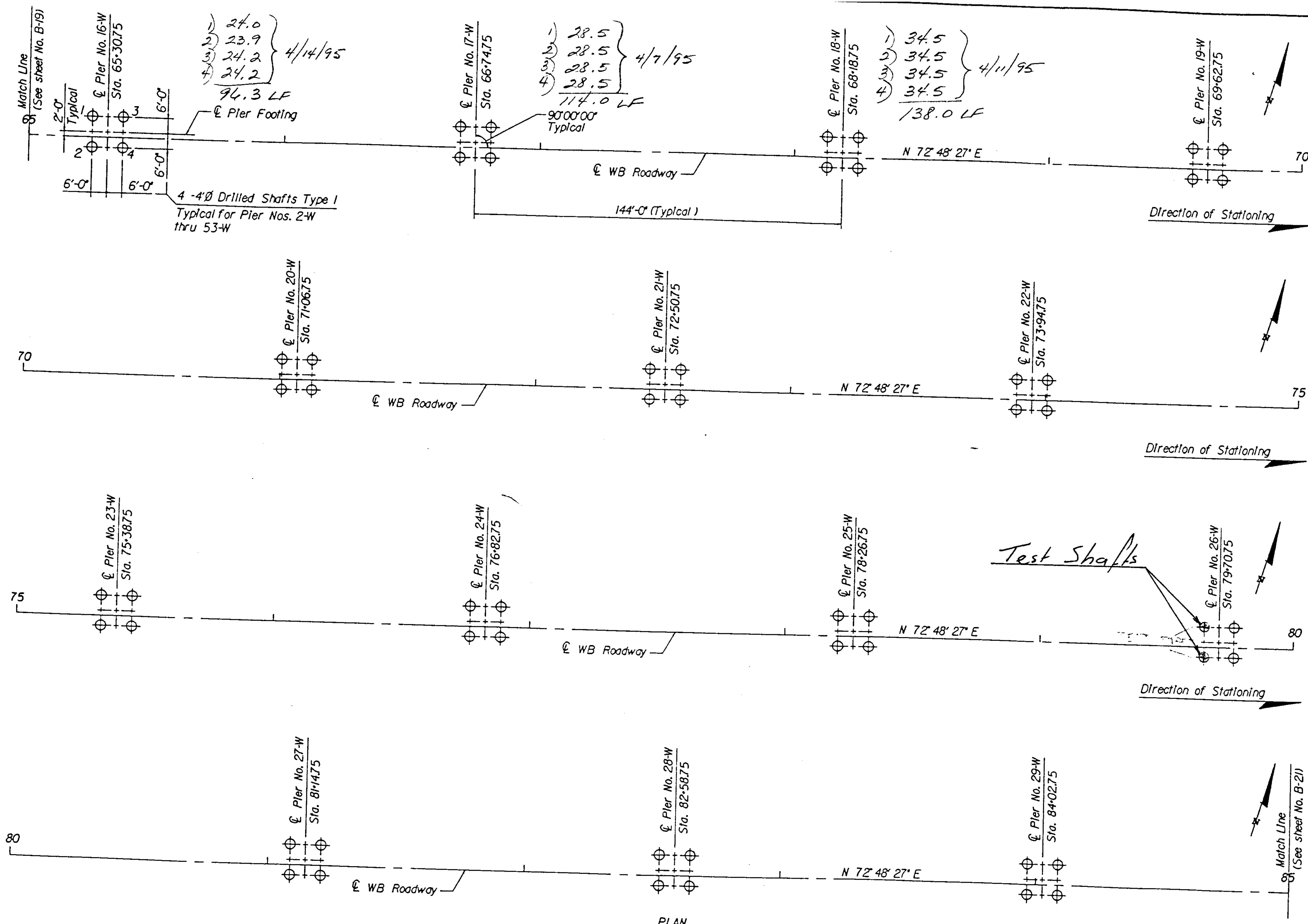
FISCAL YEAR / ROAD DIST. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-20



NOTE: For "NOTES FOR FOUNDATION LAYOUT", see Sheet No. B-19.

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	E.A.P.	1-93
Checked by	B.O.K.	9-93
Designed by	S.S.	1-93
Checked by	B.O.K.	9-93
Approved by	S.H.W.	

ENGINEER OF RECORD:
BEISWENGER, HOCH & ASSOCIATES, INC.
1190 N.E. 163rd Street Suite 203
North Miami Beach, FL 33162

BEISWENGER, HOCH & ASSOCIATES, INC.
CONSULTING ENGINEERS & PLANNERS

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 600	PINELLAS/HILLSBOROUGH	10130-3544

ALTERNATE I

SHEET TITLE: FOUNDATION LAYOUT (2)

Drawing No.

PROJECT NAME: WESTBOUND GANDY BRIDGE

Index No.

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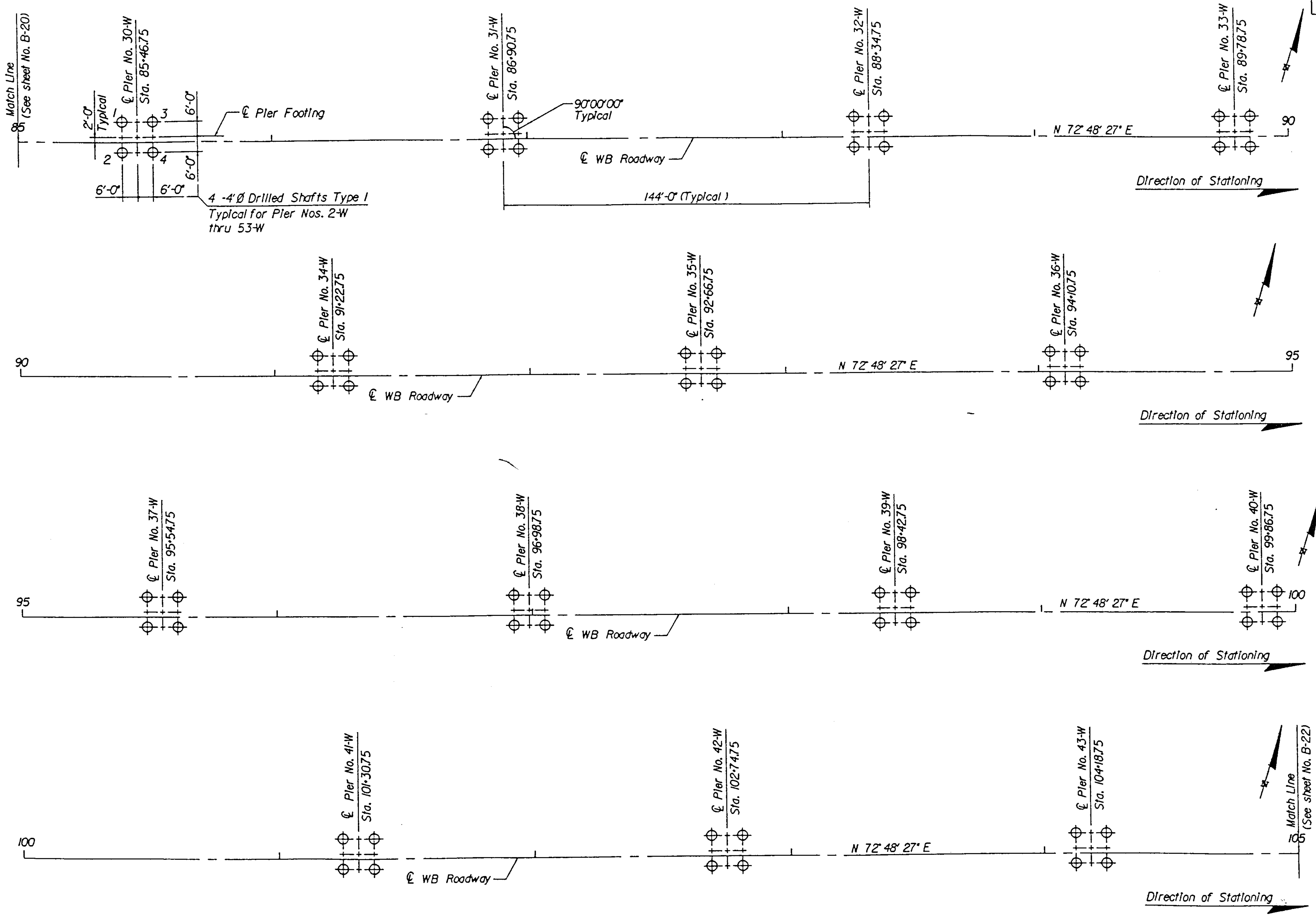
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-21



PLAN

NOTE: For "NOTES FOR FOUNDATION LAYOUT", see Sheet No. B-19.

ALTERNATE I

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	E.A.P.	1-93
Checked by	B.O.K.	9-93
Designed by	S.S.	1-93
Checked by	B.O.K.	9-93
Approved by	S.H.W.	

ENGINEER OF RECORD:
BEISWENGER, HOCH & ASSOCIATES, INC.
1190 N.E. 163rd Street Suite 203
North Miami Beach, FL 33162

BEISWENGER, HOCH & ASSOCIATES, INC.
CONSULTING ENGINEERS & PLANNERS

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 600	PINELLAS/HILLSBOROUGH	10130-3544

SHEET TITLE: FOUNDATION LAYOUT (3)

PROJECT NAME: WESTBOUND GANDY BRIDGE

Drawing No.

Index No.

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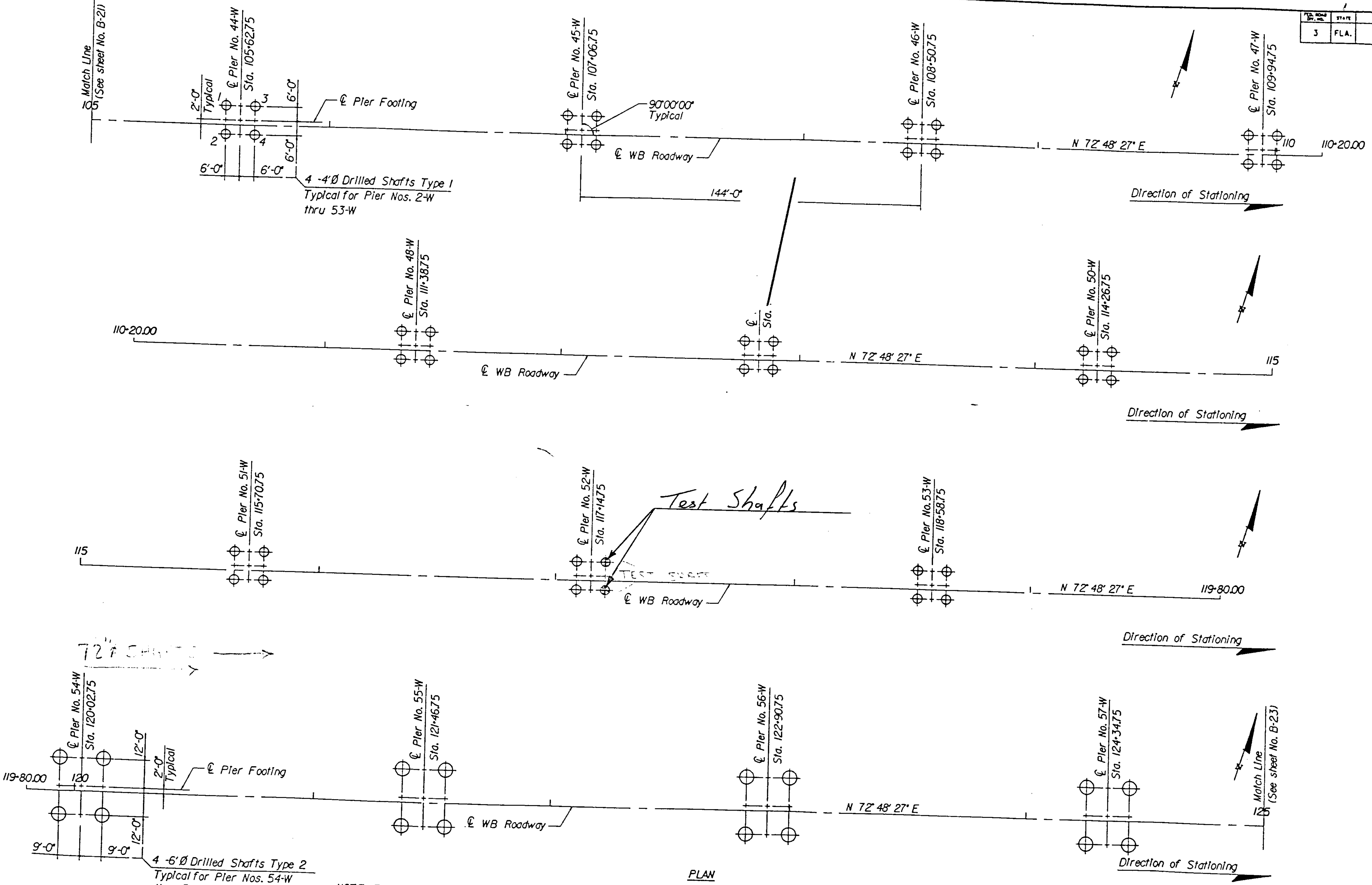
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-22



PLAN

NOTE: For "NOTES FOR FOUNDATION LAYOUT", see Sheet No. B-19.

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	E.A.P.	1-93
Checked by	B.O.K.	9-93
Designed by	S.S.	1-93
Checked by	B.O.K.	9-93
Approved by	S.H.W.	

ENGINEER OF RECORD:
BEISWENGER, HOCH & ASSOCIATES, INC.
1190 N.E. 163rd Street Suite 203
North Miami Beach, FL 33162

BEISWENGER, HOCH & ASSOCIATES, INC.
CONSULTING ENGINEERS & PLANNERS

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 600	PINELLAS/HILLSBOROUGH	10130-3544

ALTERNATE 1

SHEET TITLE: FOUNDATION LAYOUT (4)

PROJECT NAME: WESTBOUND GANDY BRIDGE

Drawing No.

Index No.

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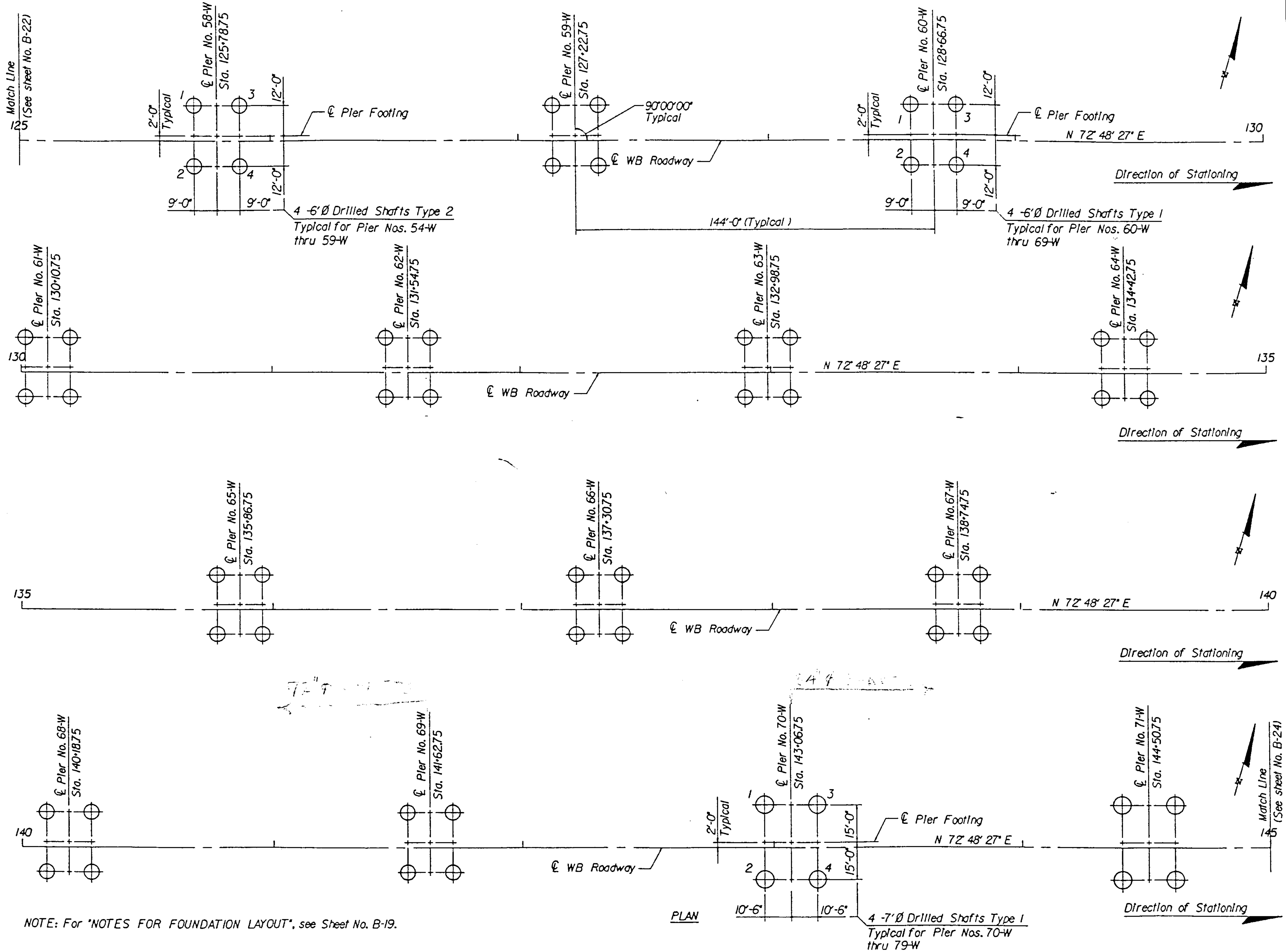
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-23



NOTE: For "NOTES FOR FOUNDATION LAYOUT", see Sheet No. B-19.

ALTERNATE I

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	E.A.P.	1-93
Checked by	B.O.K.	9-93
Designed by	S.S.	1-93
Checked by	B.O.K.	9-93
Approved by	S.H.W.	

ENGINEER OF RECORD:
BEISWENGER, HOCH & ASSOCIATES, INC.
1190 N.E. 163rd Street Suite 203
North Miami Beach, FL 33162

BEISWENGER, HOCH & ASSOCIATES, INC.
CONSULTING ENGINEERS & PLANNERS

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FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 600	PINELLAS/HILLSBOROUGH	10130-3544

SHEET TITLE: FOUNDATION LAYOUT (5)

PROJECT NAME: WESTBOUND GANDY BRIDGE

Drawing No.

Index No.

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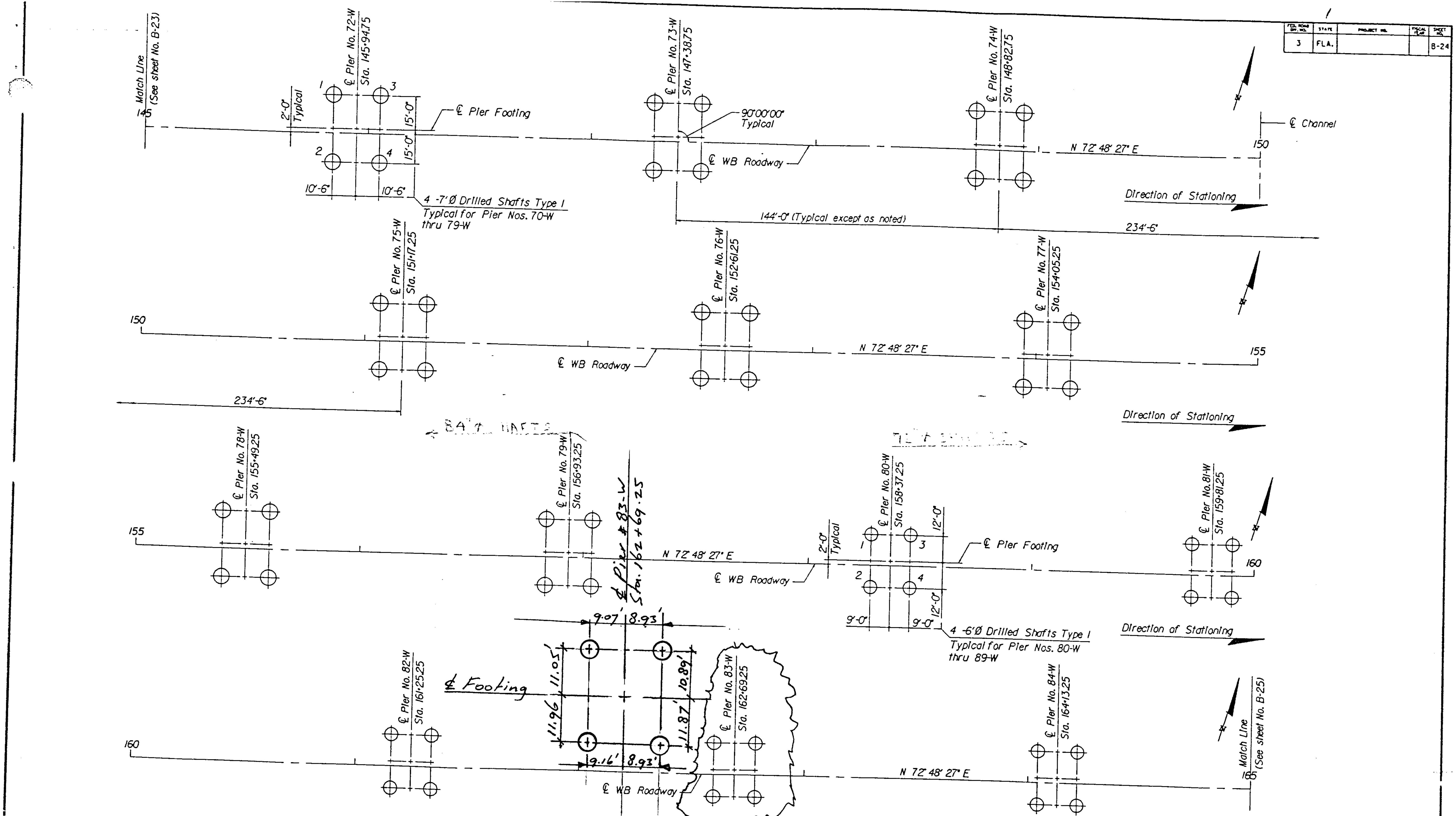

NOTE: For "NOTES FOR FOUNDATION LAYOUT", see Sheet No. B-19.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-24

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	E.A.P.	1-93
Checked by	B.O.K.	9-93
Designed by	S.S.	1-93
Checked by	B.O.K.	9-93
Approved by	S.H.W.	

ENGINEER OF RECORD:
BEISWENGER, HOCH & ASSOCIATES, INC.
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North Miami Beach, FL 33162

BEISWENGER, HOCH & ASSOCIATES, INC.
CONSULTING ENGINEERS & PLANNERS

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 600	PINELLAS/HILLSBOROUGH	10130-3544

ALTERNATE I

SHEET TITLE: FOUNDATION LAYOUT (6)

Drawing No.

PROJECT NAME: WESTBOUND GANDY BRIDGE

Index No.

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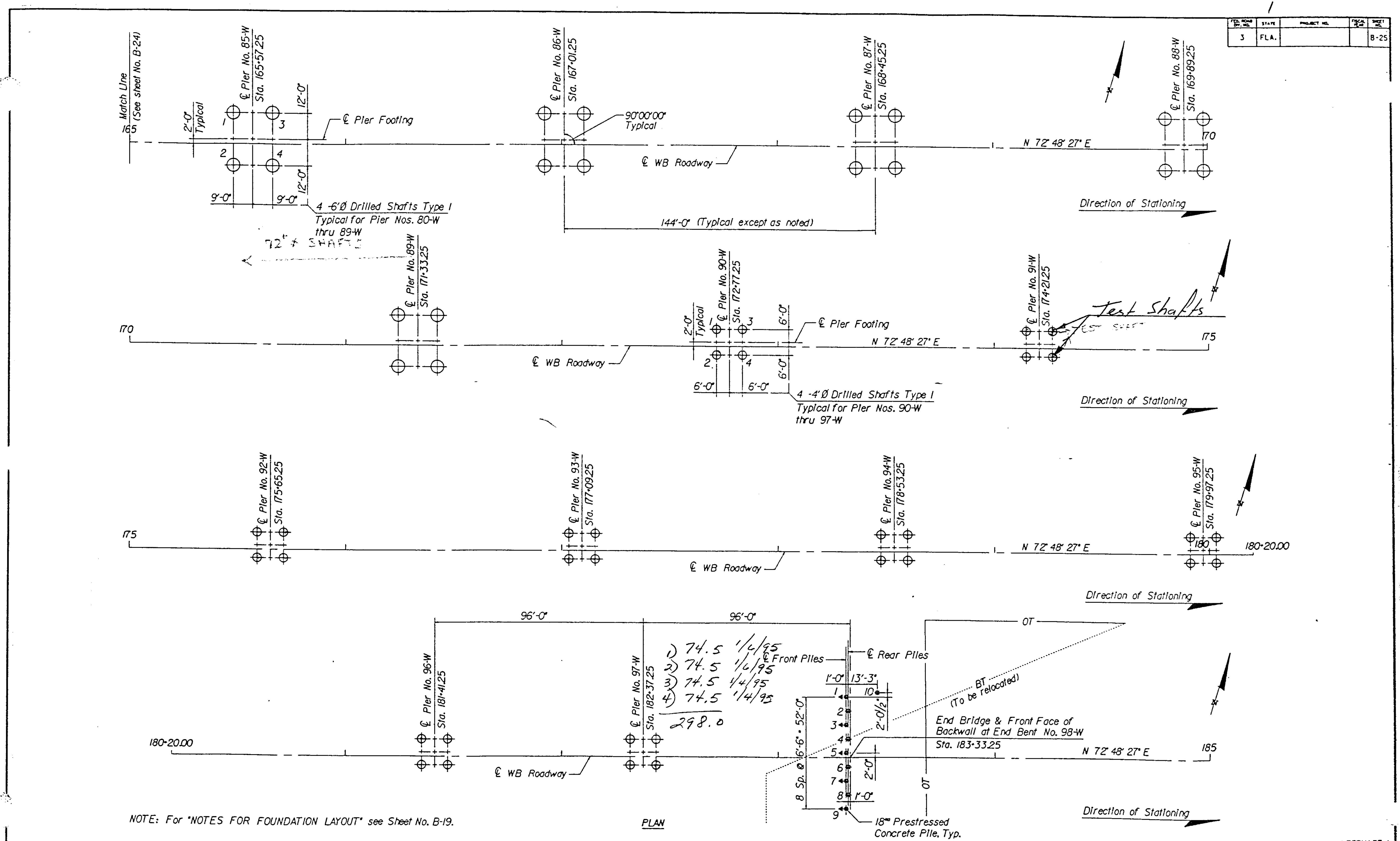
Fed. Road Div. No.	State	Project No.	Fiscal Year	Sheet No.
3	FLA.			B-25



NOTE: For "NOTES FOR FOUNDATION LAYOUT" see Sheet No. B-19.

PLAN

ALTERNATE I

Date	By	Description	Date	By	Description

REVISIONS

	Names	Dates
Drawn by	E.A.P.	1-93
Checked by	B.O.K.	9-93
Designed by	S.S.	1-93
Checked by	B.O.K.	9-93
Approved by	S.H.W.	

ENGINEER OF RECORD:
BEISWENGER, HOCH & ASSOCIATES, INC.
1190 N.E. 163rd Street Suite 203
North Miami Beach, FL 33162

BEISWENGER, HOCH & ASSOCIATES, INC.
CONSULTING ENGINEERS & PLANNERS

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

Road No.	County	Project No.
S.R. 600	PINELLAS/HILLSBOROUGH	10130-3544

Sheet Title: FOUNDATION LAYOUT (7)

Project Name: WESTBOUND GANDY BRIDGE

Drawing No.

Index No.

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W.P.I. NO. 1119232

OCT 30 2003

DESIGN/BUILD TEAM

PCL CIVIL CONSTRUCTORS, INC.

JMI ENGINEERS, INC.

PARSONS

TAMPA BAY ENGINEERING, INC.

PROFESSIONAL SERVICE INDUSTRIES, INC.

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

FINAL "AS BUILT" PLANS

~~DESIGN/BUILD PLANS~~

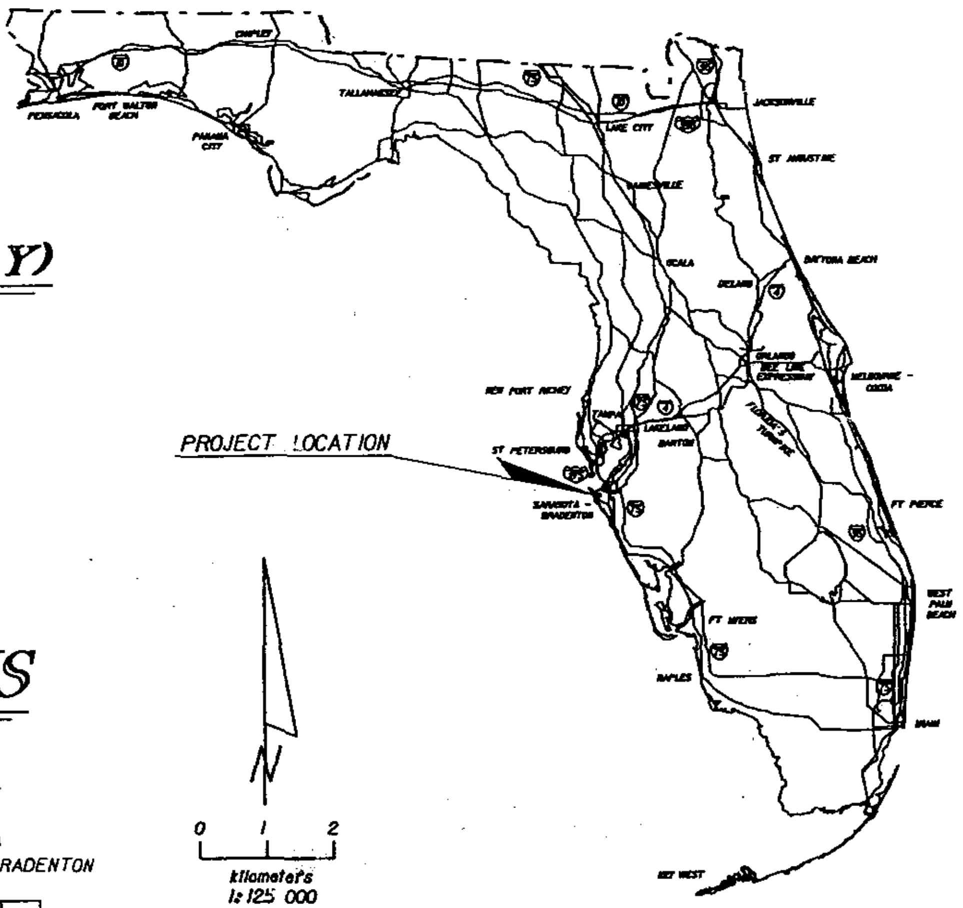
STATE ROAD NO. 789 (RINGLING CAUSEWAY)

FINANCIAL PROJECT ID 197942-1-52-01

STATE PROJECT NO. 17030-3509

(FEDERAL FUNDS)

SARASOTA COUNTY



FINAL STRUCTURES PLANS

Revisions (05/02):
- B-1 THRU B-9
- B-13
- B-15
- B-16
- B-21
- B-22
- B-24
- B-26 THRU B-32
- B-34 THRU B-38
- B-40
- B-41
- B-43 THRU B-55
- B-57
- B-59
- B-61 THRU 70
- B-73
- B-75
- B-77 THRU B-80
- B-82
- B-83
- B-91 THRU B-99
- B-102 THRU B-106
- B-109 THRU B-111
- B-113
- B-115
- B-117
- B-118
- B-120
- B-123 THRU B-127

Revisions (07/02):
- B-3
- B-5
- B-6
- B-7
- B-8
- B-11
- B-29 THRU B-31
- B-40
- B-41
- B-43
- B-53
- B-57
- B-98

Revisions (08/02):
- SW-2, SW-3
- SW-5 THRU SW-8
- SW-11

Revisions (11/02):
- B-107
- ~~B-122A~~

Revisions (01/03):
- B-1, B-2
- B-27A, B-27B
- B-40, B-41
- B-45 THRU B-47U, B-51 THRU B-54
- B-56 THRU B-57A, B-60, B-66
- B-67 THRU B-70E
- B-72A THRU B-72J
- B-74A THRU B-74G
- B-79A, B-79F
- B-80 THRU B-91, B-97A
- B-98 THRU B-99B
- B-93
- B-102
- B-123 THRU B-126
- B-128 THRU B-154
- SW-6

Revisions (02/03):
- SW-2
- SW-3

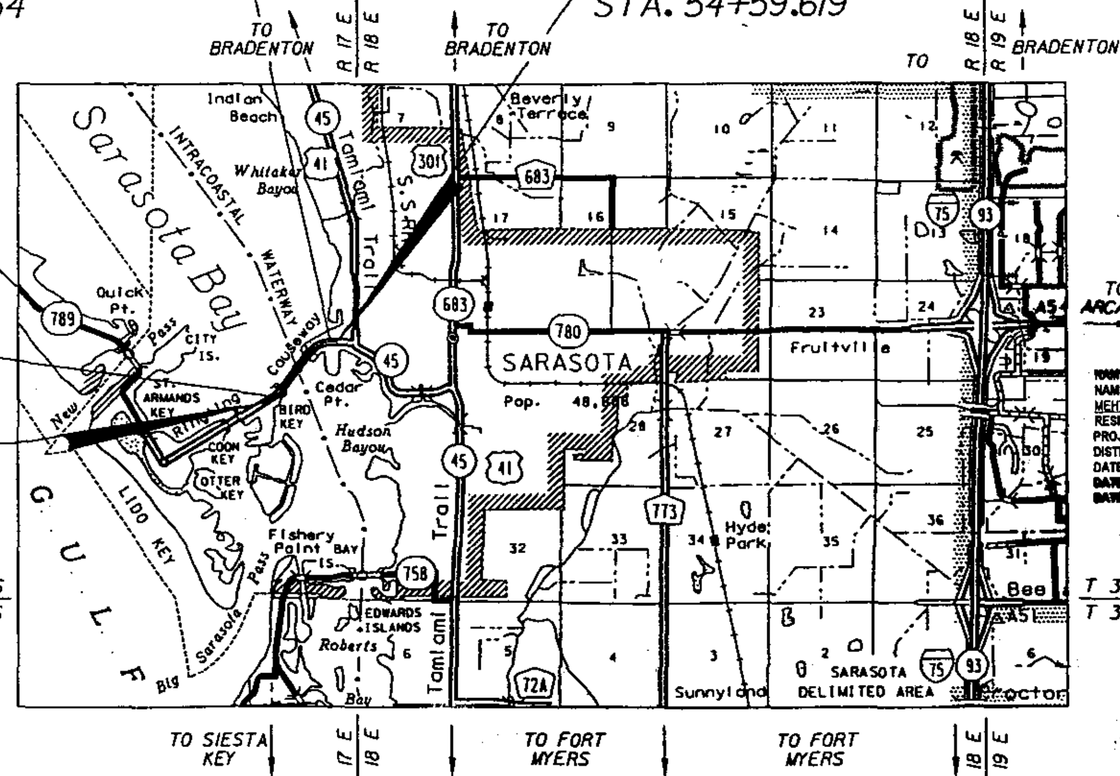
Revisions (03/03):
- B-11
- B-72A THRU B-72G
- B-72J

END BRIDGE NO. 170176
STA. 50+30.834

END PROJECT
STA. 54+59.619

BEGIN BRIDGE
NO. 170176
STA. 40+86.934

BEGIN PROJECT
STA. 38+70.663
(K.P. 1.899)



NAME OF CONTRACTOR: PCL CIVIL CONSTRUCT., INC.
NAME OF CONSULTANTS INVOLVED: RS&H CS, TYLI, MEHTA, & INFRASTRUCTURE
RESIDENT ENGINEER: MARY ELLEN MAURER
PROJECT ADMINISTRATOR: ALBERT ROSENSTEIN
DISTRICT SECRETARY: RICK LANGLEY
DATE WORK STARTED: AUGUST 1, 2001

APPROVED FOR CONSTRUCTION
JMI ENGINEERS, INC.
BY

3/21/03

By 750.00

COMPONENTS OF CONTRACT PLANS SET

- BRIDGE DRAWING PLANS
- LIGHTING PLANS
- SEAWALL PLANS
- ROADWAY PLANS
- SIGNING AND PAVEMENT MARKING PLANS
- RETAINING WALL PLANS
- TEMPORARY WALL PLANS

GOVERNING STANDARDS AND SPECIFICATIONS:
FLORIDA DEPARTMENT OF TRANSPORTATION,
ROADWAY AND TRAFFIC DESIGN STANDARDS
DATED JANUARY 2000, AND
STANDARDS SPECIFICATIONS FOR ROAD AND BRIDGE
CONSTRUCTION DATED 2000,
AS AMENDED BY CONTRACT DOCUMENTS.

1/14/04

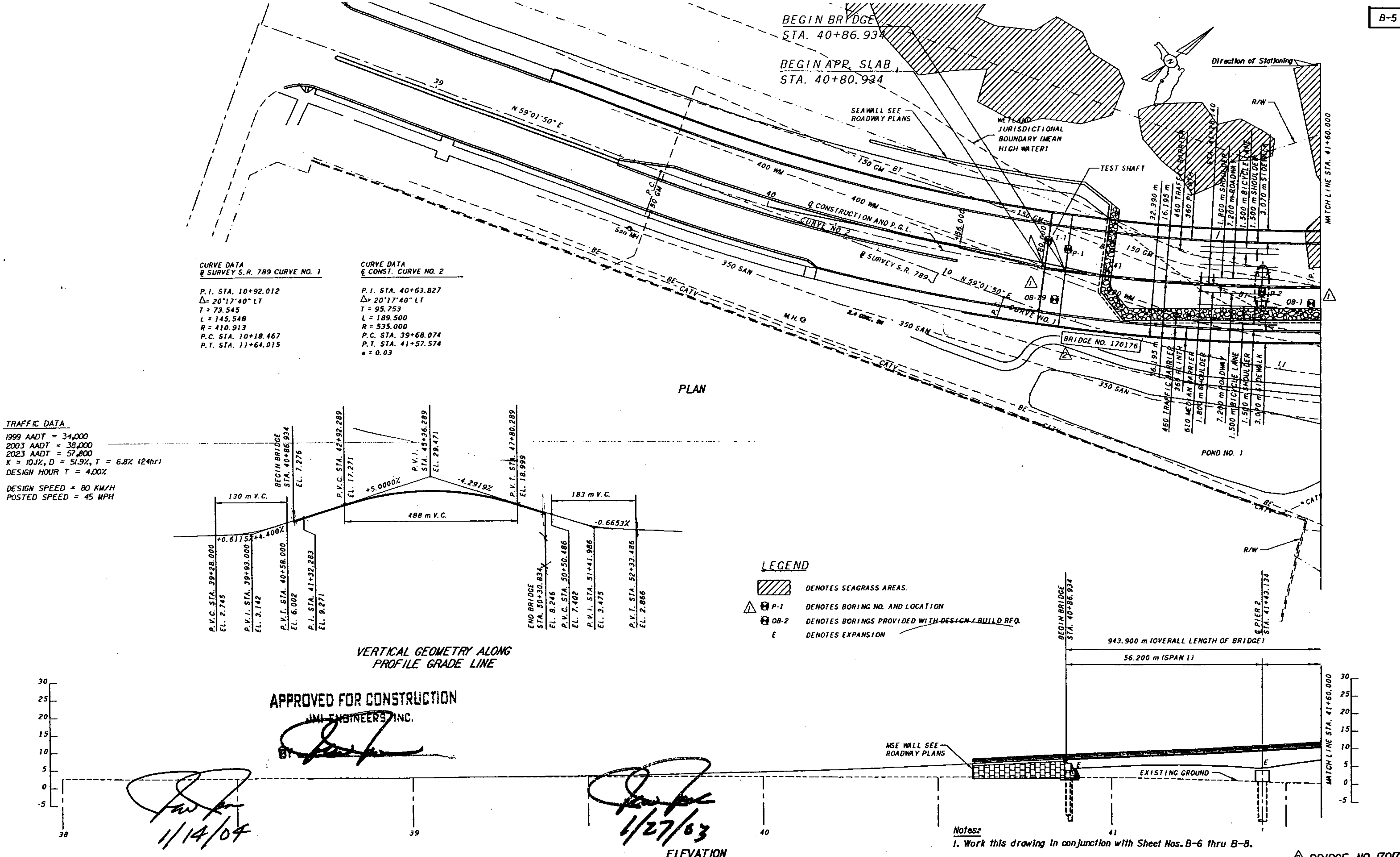
Engineer of Record: John W. Jordan, Jr.
P.E. No.: 50894

JMI Engineers, Inc.
1424 Piedmont Drive East, Suite 100
Tallahassee, FL 32308
License No. 5991

Bridge 170176

PCL CIVIL CONSTRUCTORS, INC.
9900 WEST SAMPLE ROAD, SUITE 203
CORAL SPRINGS, FLORIDA 33065
TEL: (954) 345-1725
FL. LIC. # C011358

SHEET No. 1



Notes:
1. Work this drawing in conjunction with Sheet Nos. B-6 thru B-8.

BRIDGE NO. 170176

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
05/02	EBS	1. As Noted.			
07/02	EBS	2. Revised & Added Bridge Number. Added MSE Wall and Seawall Reference.			

	NAMES	DATES
DRAWN BY	DLH	10/09/00
CHECKED BY	EBS	10/09/00
DESIGNED BY	MF	10/09/00
CHECKED BY	JWJ	10/09/00
APPROVED BY	J.W. JORDAN, P.E.	10/09/00

ENGINEER OF RECORD:
JMI ENGINEERS
JOHN W. JORDAN JR., PE
FLORIDA LICENSE NO. 50854
CERTIFICATE OF AUTHORIZATION NO. 8115
1424 PIEDMONT DRIVE EAST, SUITE 100
TALLAHASSEE, FL 32308 TEL: 850-385-7450

LOGO:
PCL CONSTRUCTION LEADERS
JMI ENGINEERS

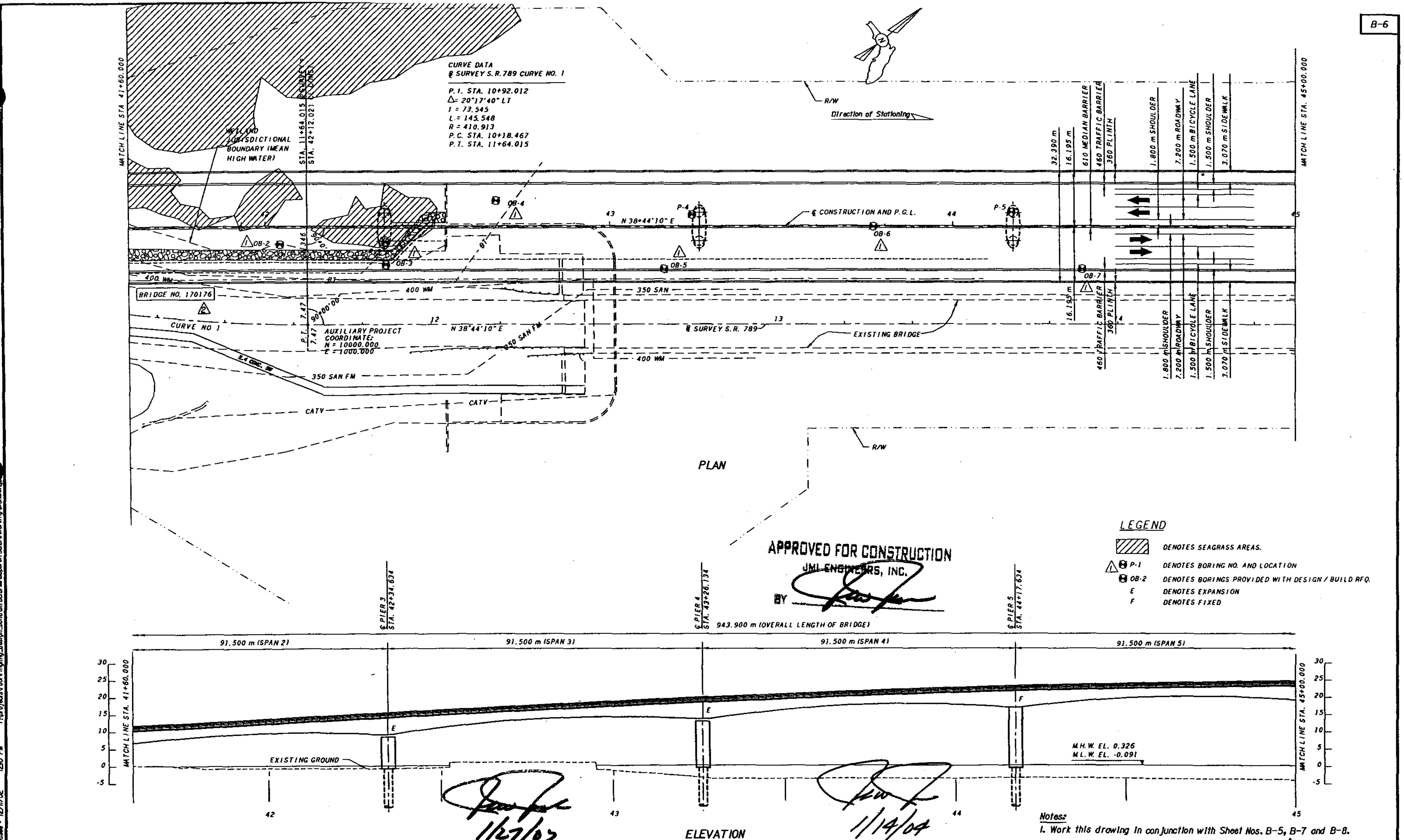
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

SHEET TITLE: GENERAL PLAN AND ELEVATION 1 OF 4

PROJECT NAME: STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY BRIDGE NO. 170176

f:\projects\r\ringling_design_build\structures\superstructure\working\planelev.dgn 12:10 PM JMI Engineers - Tallahassee - 12/11/02



Notes:
1. Work this drawing in conjunction with Sheet Nos. B-5, B-7 and B-8.

BRIDGE NO. 170176

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
05/02	EBS	1. As Noted.			
07/02	EBS	2. Revised & Added Bridge Number.			

	NAMES	DATES
DRAWN BY	DLH	10/09/00
CHECKED BY	EBS	10/09/00
DESIGNED BY	MF	10/09/00
CHECKED BY	JWJ	10/09/00
APPROVED BY	J.W. JORDAN, P.E.	

ENGINEER OF RECORD:
JMI ENGINEERS
JOHN W. JORDAN JR., PE
FLORIDA LICENSE NO. 50894
CERTIFICATE OF AUTHORIZATION NO. 8115
1424 PIEDMONT DRIVE EAST, SUITE 100
TALLAHASSEE, FL 32308 TEL: 850-385-7450

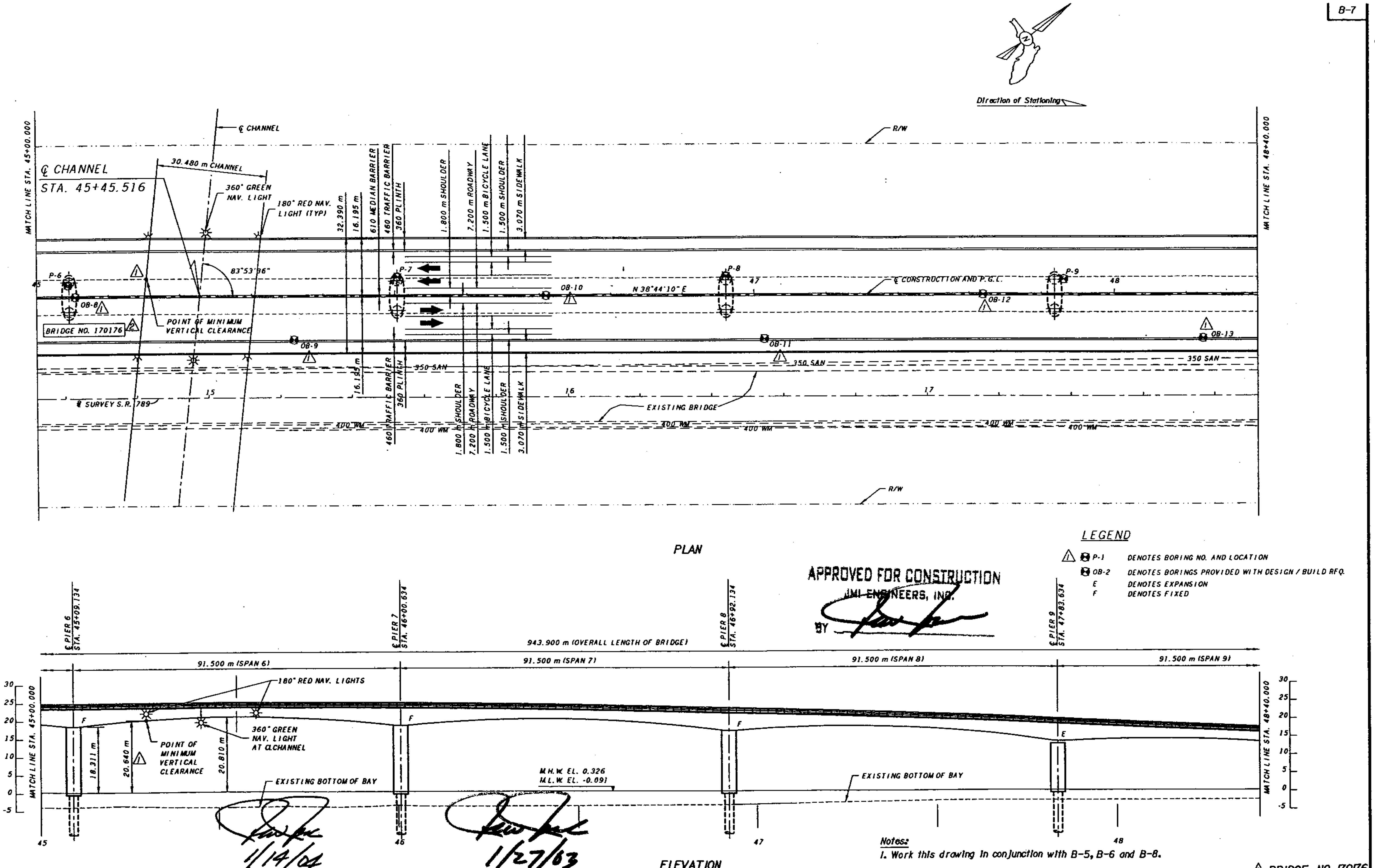
LOGO:
PCL CONSTRUCTION LEADERS
JMI ENGINEERS

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

SHEET TITLE:
GENERAL PLAN AND ELEVATION 2 OF 4

PROJECT NAME:
STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY BRIDGE NO. 170176



APPROVED FOR CONSTRUCTION
JMI ENGINEERS, INC.
BY

1/14/04

1/27/03

Notes:
1. Work this drawing in conjunction with B-5, B-6 and B-8.

△2 BRIDGE NO. 170176

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
05/02	EBS	1. Revised Min. Vert. Clr. location and height and boring callout.			
07/02	EBS	2. Revised & Added Bridge Number.			

	NAMES	DATES
DRAWN BY	DLH	10/09/00
CHECKED BY	EBS	10/09/00
DESIGNED BY	MF	10/09/00
CHECKED BY	JWJ	10/09/00
APPROVED BY	J.W. JORDAN, P.E.	

ENGINEER OF RECORD:
JMI ENGINEERS
JOHN W. JORDAN JR., PE
FLORIDA LICENSE NO. 50894
CERTIFICATE OF AUTHORIZATION NO. 8115
1424 PIEDMONT DRIVE EAST, SUITE 100
TALLAHASSEE, FL 32308 TEL: 850-385-7450

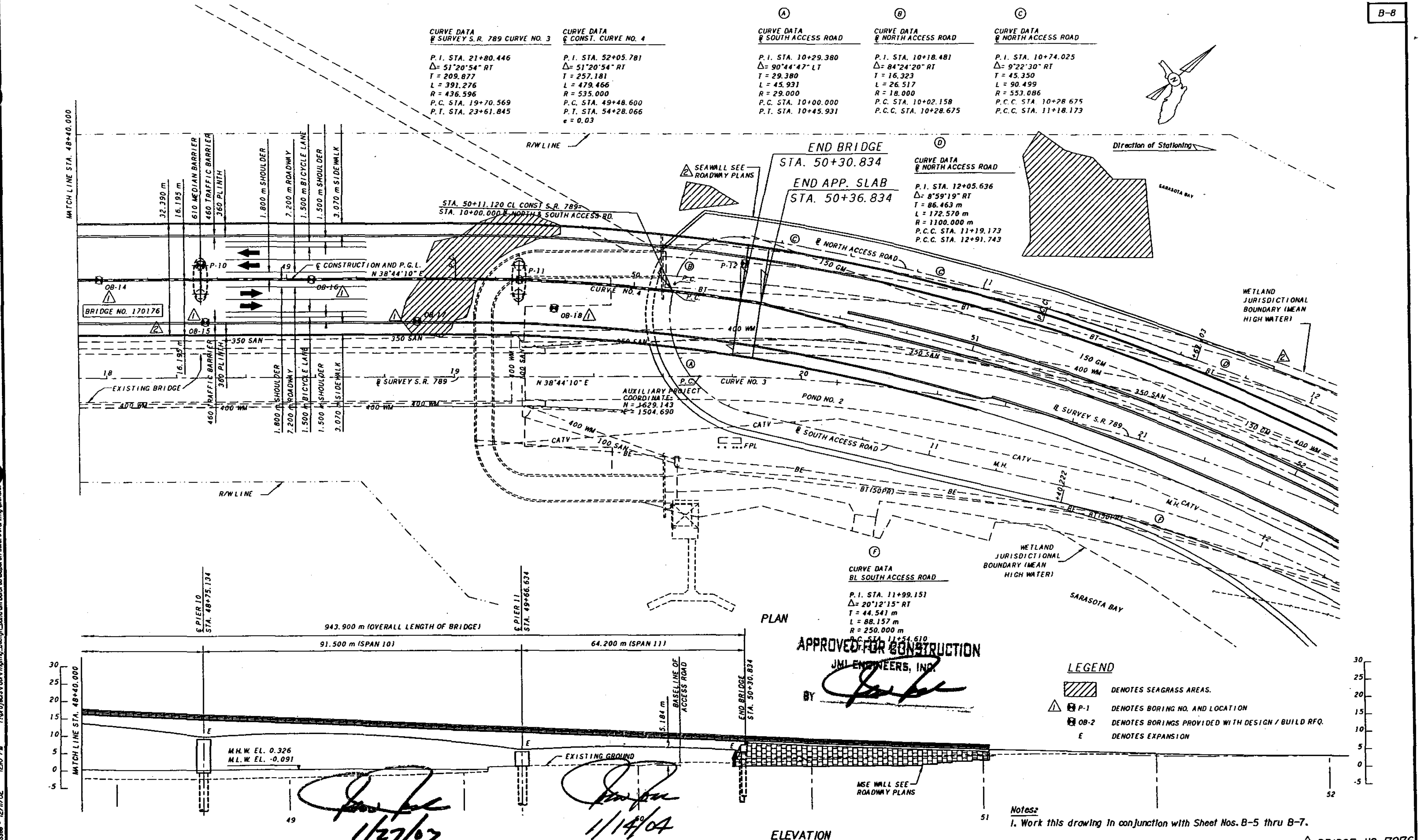
LOGO:
PCL CONSTRUCTION LEADERS
JMI ENGINEERS

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

SHEET TITLE: GENERAL PLAN AND ELEVATION 3 OF 4

PROJECT NAME: STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY △2 BRIDGE NO. 170176



APPROVED FOR CONSTRUCTION
JMI ENGINEERS, INC.
BY

1/27/03

1/14/04

Notes:
1. Work this drawing in conjunction with Sheet Nos. B-5 thru B-7.

BRIDGE NO. 170176

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
05/02	EBS	1. As Noted.			
07/02	EBS	2. Revised & Added Bridge Number. Added MSE Wall and Seawall Reference.			

	NAMES	DATES
DRAWN BY	DLH	10/09/00
CHECKED BY	EBS	10/09/00
DESIGNED BY	MF	10/09/00
CHECKED BY	MWJ	10/09/00
APPROVED BY	J.W. JORDAN, P.E.	

ENGINEER OF RECORD:
JMI ENGINEERS
JOHN W. JORDAN JR., PE
FLORIDA LICENSE NO. 50894
CERTIFICATE OF AUTHORIZATION NO. 8115
1424 PIEDMONT DRIVE EAST, SUITE 100
TALLAHASSEE, FL 32308 TEL: 850-385-7450

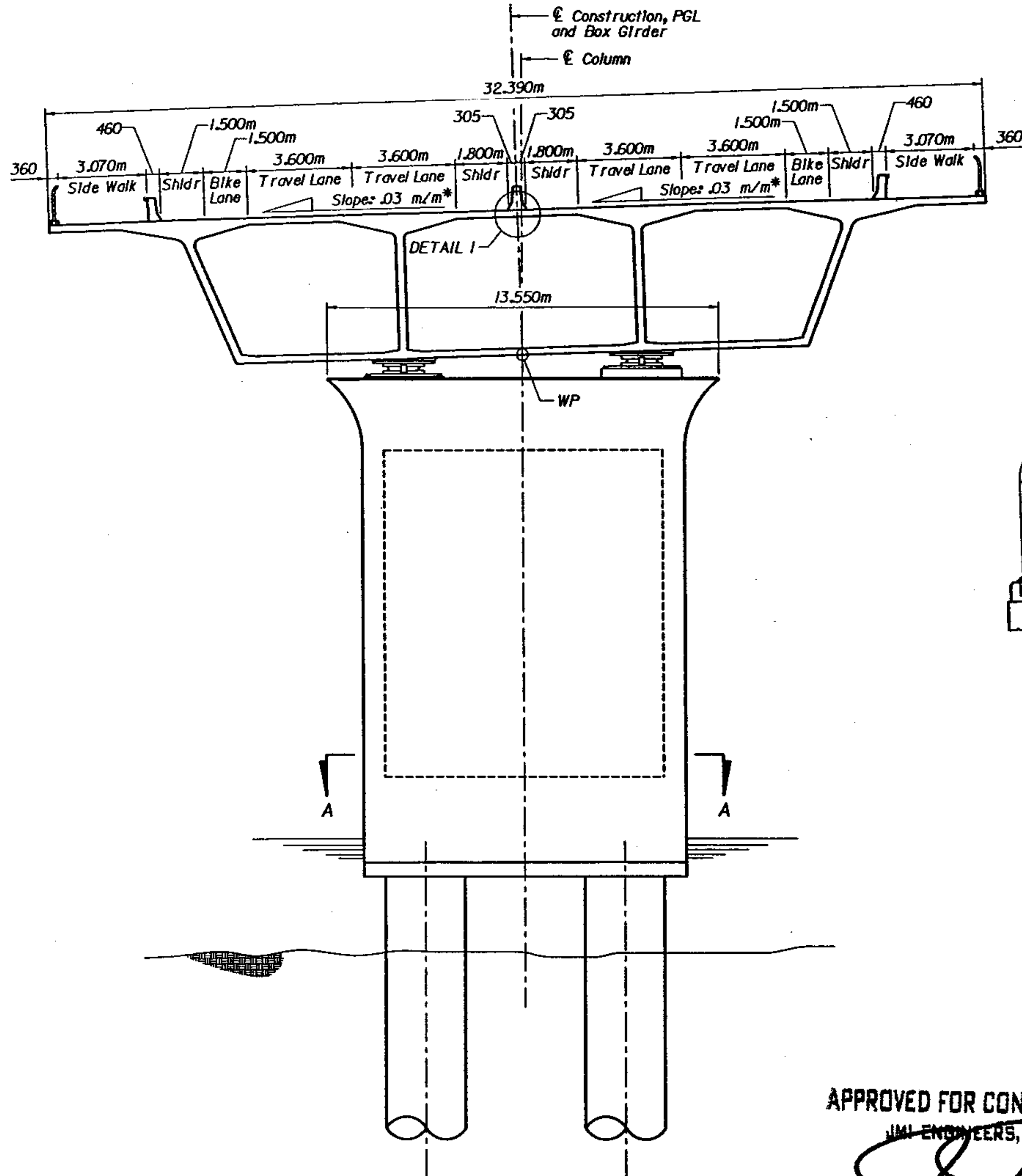
LOGO:
PCL CONSTRUCTION LEADERS
JMI ENGINEERS

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

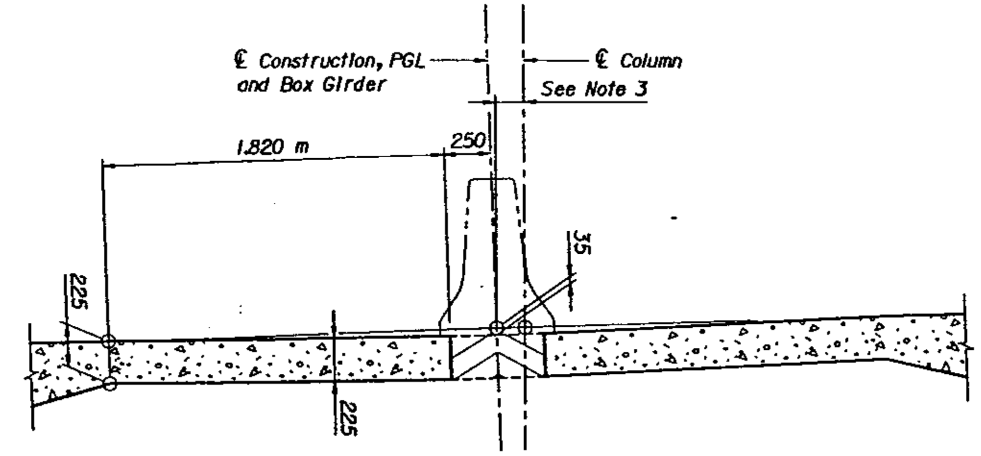
SHEET TITLE:
GENERAL PLAN AND ELEVATION 4 OF 4

PROJECT NAME:
STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY
BRIDGE NO. 170176



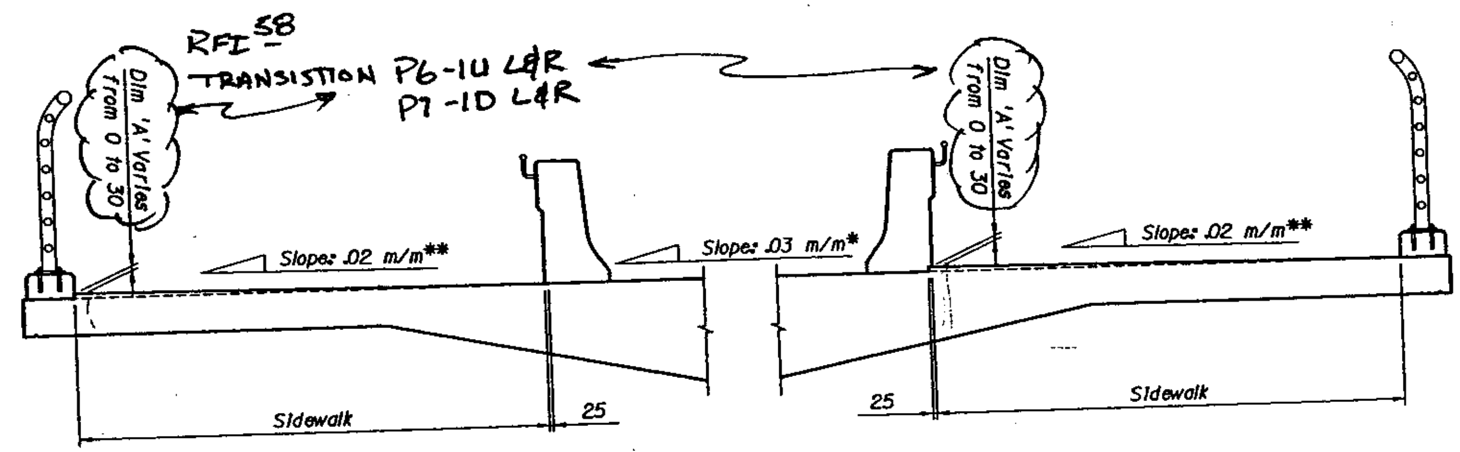
TYPICAL BRIDGE SECTION
STA. 40+86.934 TO STA. 44+69.884 (Looking Upstation)
STA. 46+39.884 TO STA. 50+30.834 (Looking Downstation)

* Slope varies between STA. 44+69.884 to STA. 46+39.884.
** Slope matches travel lanes between STA. 45+08.217 to STA. 46+01.551.

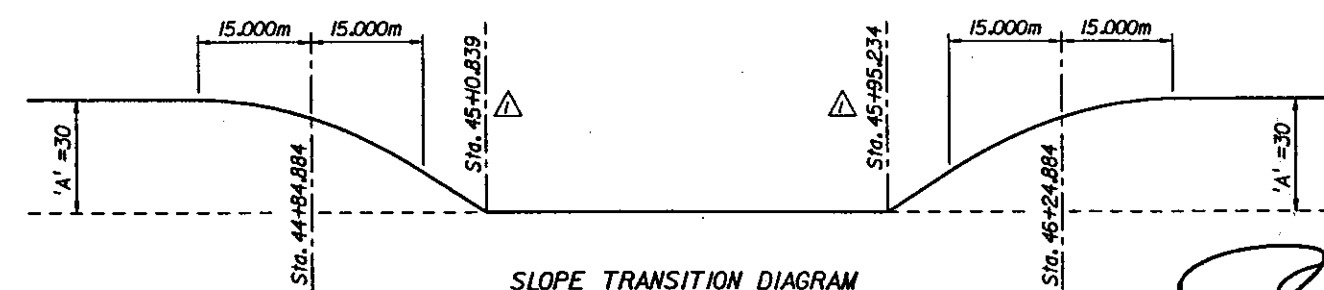


DETAIL I

RFI 58
TRANSITION P6-1U L&R
P7-1D L&R



ELEVATION



SLOPE TRANSITION DIAGRAM

SIDEWALK DETAIL

APPROVED FOR CONSTRUCTION
JMI ENGINEERS, INC.
BY

1/27/03

1/14/04

Notes:
1. Work this drawing in conjunction with Sheet No. B-10.
2. For Foundation Layouts see Sheet Nos. B-15 thru B-21.
3. For Offset Dimension See Sheet Nos. B-15 thru B-21.
4. WP denotes Working Point.

BRIDGE NO. 170176

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
05/02	JMH	1. Revised Sidewalk Detail Sta.			
01/04	JJ	AS BUILTS			

	NAMES	DATES
DRAWN BY	DLH	08/01
CHECKED BY	EBS	08/01
DESIGNED BY	MF	08/01
CHECKED BY	JWJ	08/01
APPROVED BY	J.W. JORDAN, P.E.	

ENGINEER OF RECORD:
JMI ENGINEERS
JOHN W. JORDAN JR., PE
FLORIDA LICENSE NO. 50894
CERTIFICATE OF AUTHORIZATION NO. 6115
1424 PIEDMONT DRIVE EAST, SUITE 100
TALLAHASSEE, FL 32308 TEL: 850-385-7450

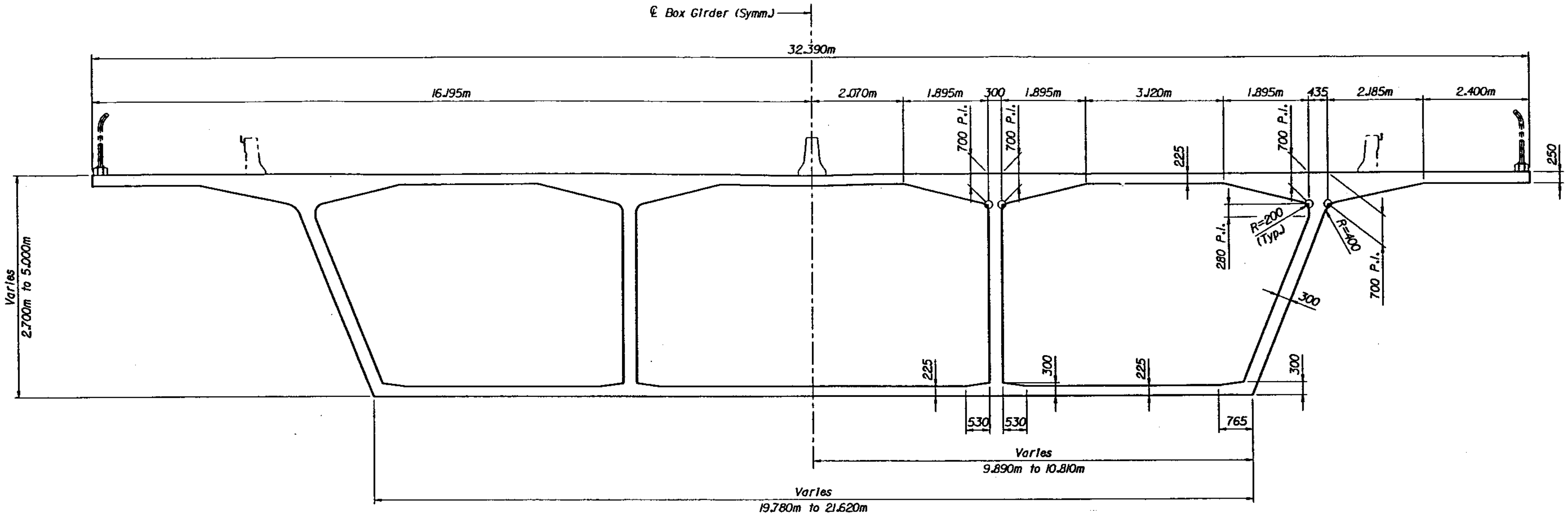
LOGO:
PCL
CONSTRUCTION LEADERS
JMI ENGINEERS

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

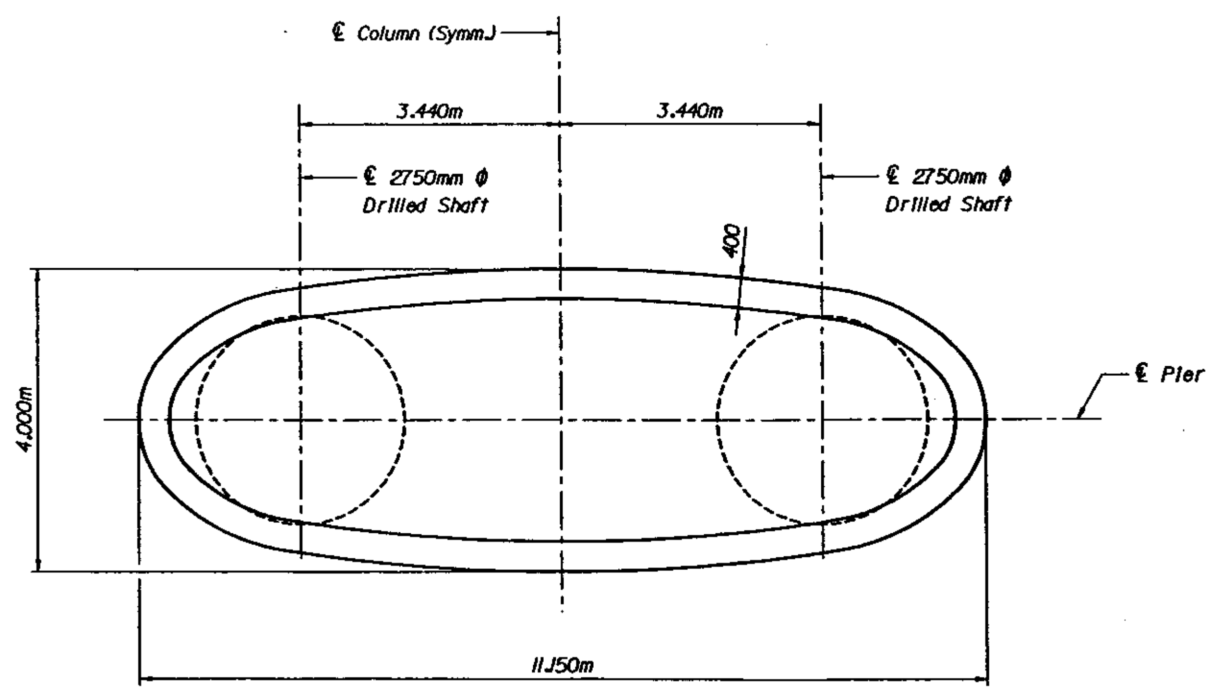
ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

SHEET TITLE:
TYPICAL BRIDGE SECTION 1 OF 2

PROJECT NAME:
STATE ROAD 789 (RINGLING CSWY)
OVER INTRACOASTAL WATERWAY



TYPICAL SUPERSTRUCTURE SECTION



TYPICAL SUBSTRUCTURE SECTION A-A

APPROVED FOR CONSTRUCTION
JMI ENGINEERS, INC.
BY

1/27/03

1/14/04

Notes:
1. Work this drawing in conjunction with Sheet No. B-9.

BRIDGE NO. 170176

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

	NAMES	DATES
DRAWN BY	JMH	09/01
CHECKED BY	EBS	09/01
DESIGNED BY	JL	09/01
CHECKED BY	JWJ	09/01
APPROVED BY	J.W. JORDAN, P.E.	

ENGINEER OF RECORD:
JMI ENGINEERS
JOHN W. JORDAN JR., PE
FLORIDA LICENSE NO. 50894
CERTIFICATE OF AUTHORIZATION NO. 8115
1424 PIEDMONT DRIVE EAST, SUITE 100
TALLAHASSEE, FL 32308 TEL: 850-385-7450

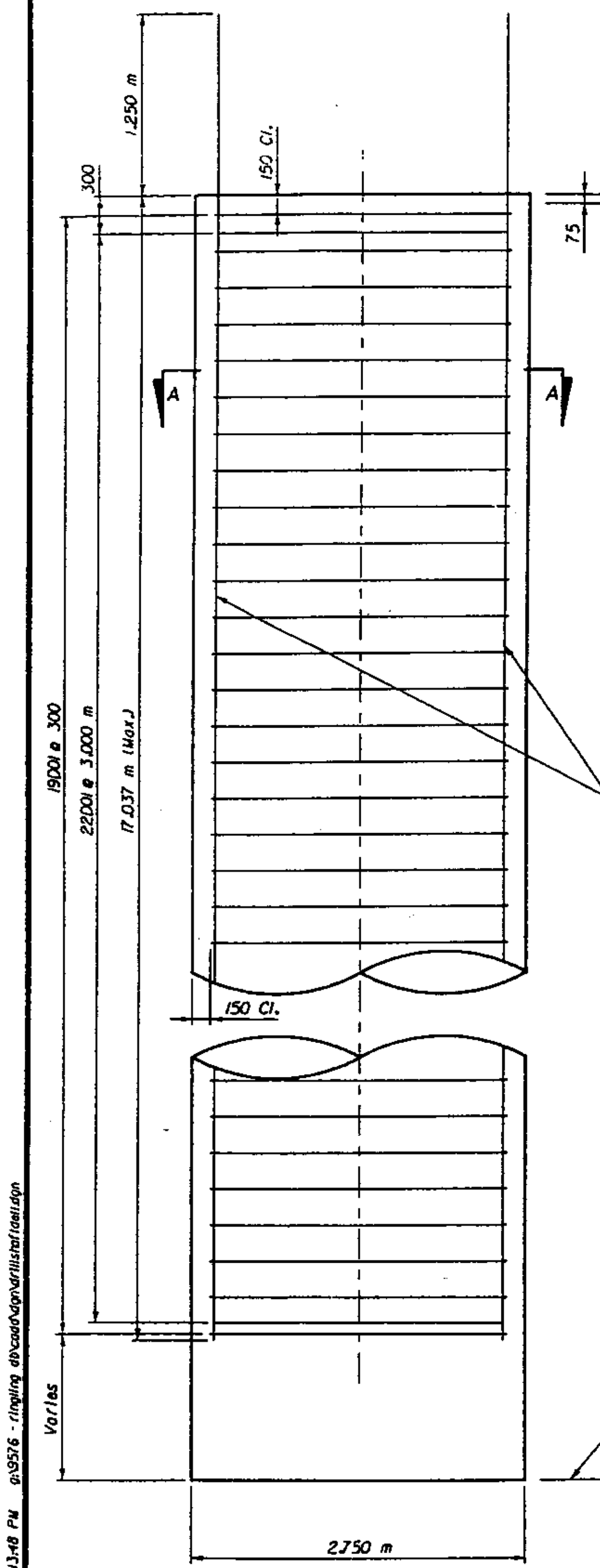
LOGO:
PCL CONSTRUCTION LEADERS
JMI ENGINEERS

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

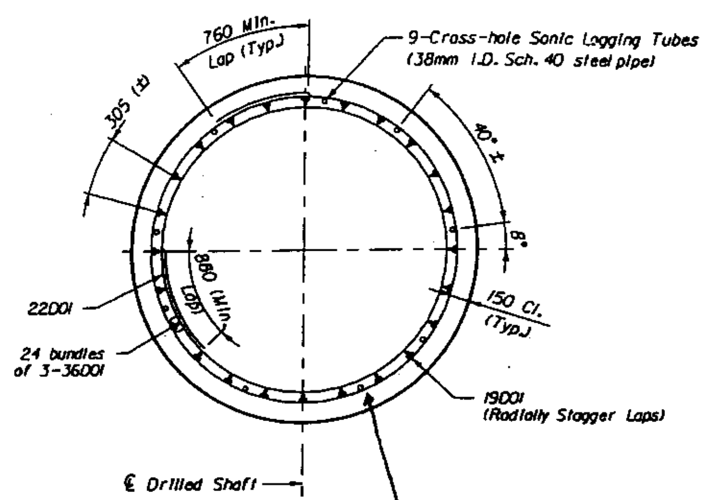
ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

SHEET TITLE: TYPICAL BRIDGE SECTION 2 OF 2

PROJECT NAME: STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY



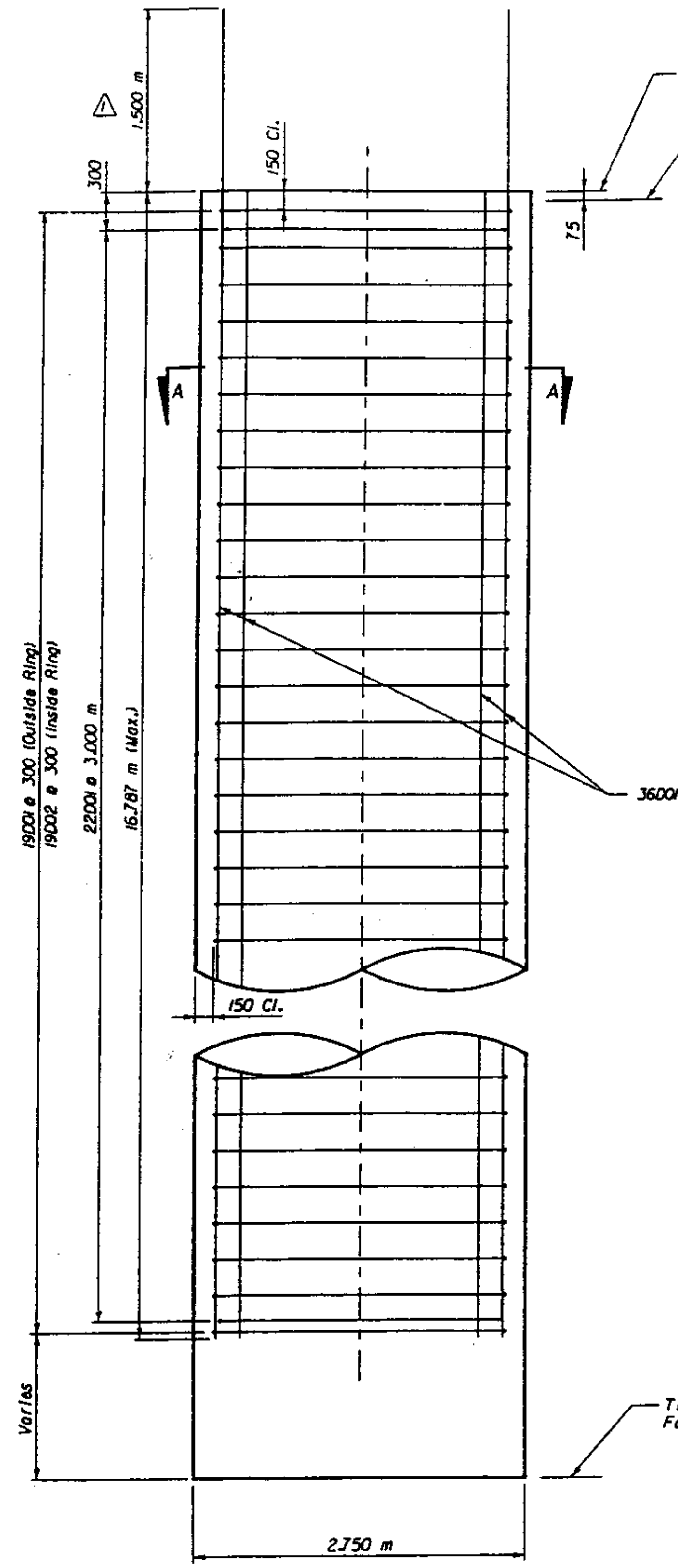
ELEVATION - 2.750m Ø DRILLED SHAFT
(For all Piers except Piers 4 & 9)



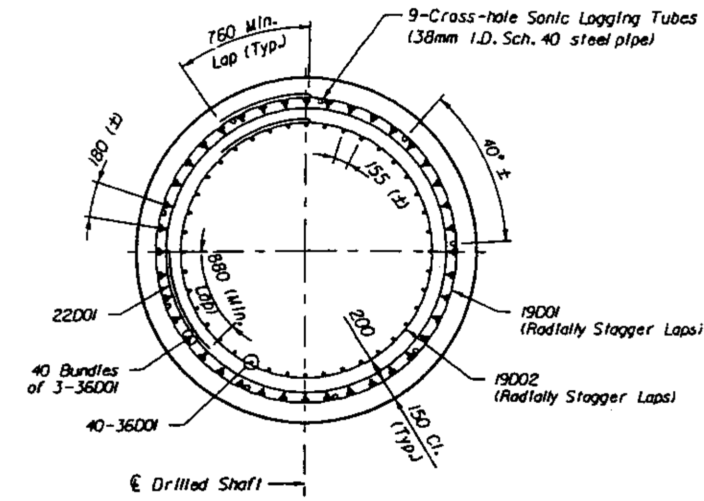
SECTION A-A

NOTE: STEEL IS NOT CONCENTRIC WITHIN TOLERANCE 38mm
@ PIER 4 SHAFTS 1 & 2
PIER 9 SHAFT 2

ESTIMATED QUANTITIES		
Pier No.	Drilled Shaft Length	Quantity
2	17.175 m	2
3	17.675 m	2
5	15.175 m	2
6	17.175 m	2
7	13.675 m	2
8	14.175 m	2
10	14.175 m	2
11	16.475 m	2



ELEVATION - 2.750m Ø DRILLED SHAFT
(For Piers 4 & 9 only)



SECTION A-A

ESTIMATED QUANTITIES		
Pier No.	Drilled Shaft Length	Quantity
4	18.675 m	2
9	14.675 m	2

APPROVED FOR CONSTRUCTION
JMI ENGINEERS, INC.
1/27/03

Notes:

1. Concrete shall be poured continuously from tip elevation to shaft head elevation of drilled shaft.
2. Concrete shall be Class IV, 28 MPa.

Philip M Hartsfield
1/15/04

Philip M Hartsfield
1/27/03

BRIDGE NO. 170176

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
5/21/02	CWN	Added dimension.			
0104	JJ	AS BUILTS			

	NAMES	DATES
DRAWN BY	JLS	11/20/01
CHECKED BY	CWN	11/20/01
DESIGNED BY	DCG	11/20/01
CHECKED BY	PMH	11/20/01
APPROVED BY	Philip M. Hartsfield, P.E.	

ENGINEER OF RECORD: PARSONS

LOGO: PCL CONSTRUCTION LEADERS

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

SHEET TITLE: 2.750m Ø DRILLED SHAFT

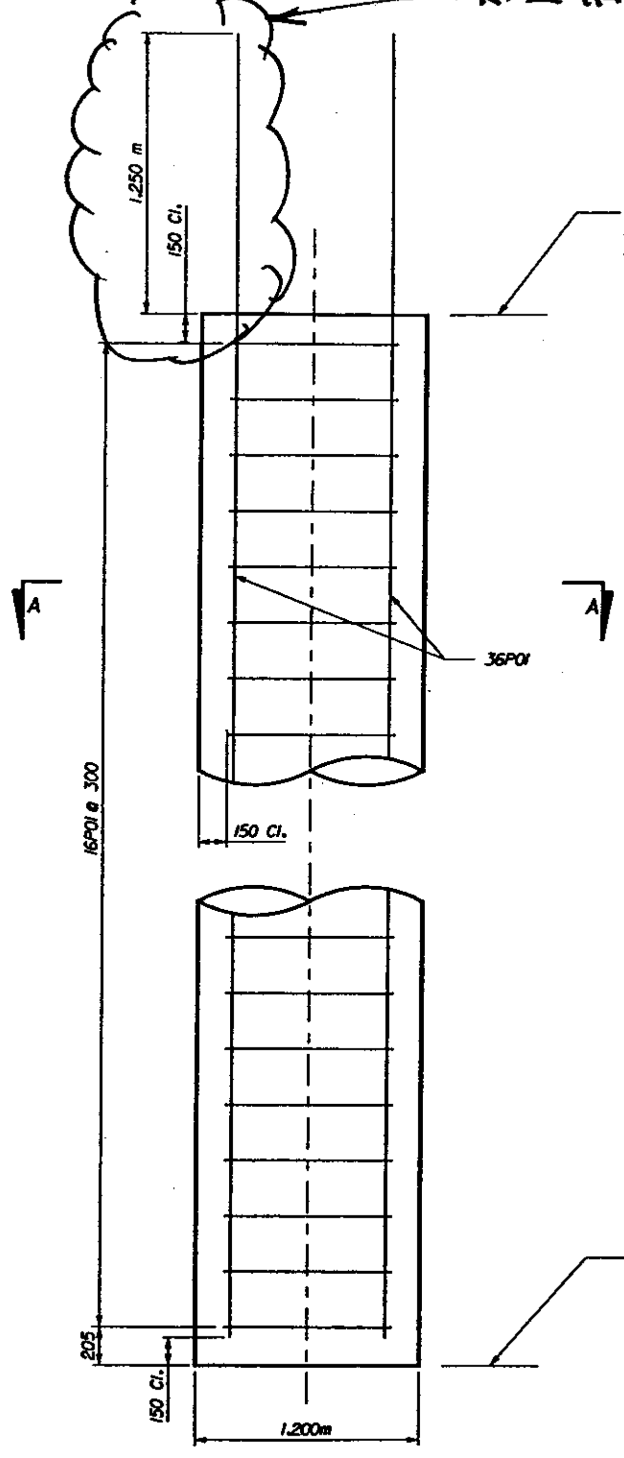
PROJECT NAME: STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY

01/27/2003 04:13:48 PM g:\9576 - ringling dr\cadd\dgn\drillshaftdetl.dgn

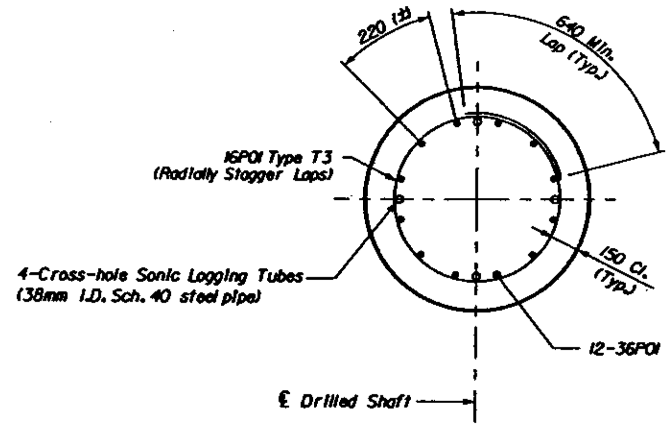
PARSONS

B-14

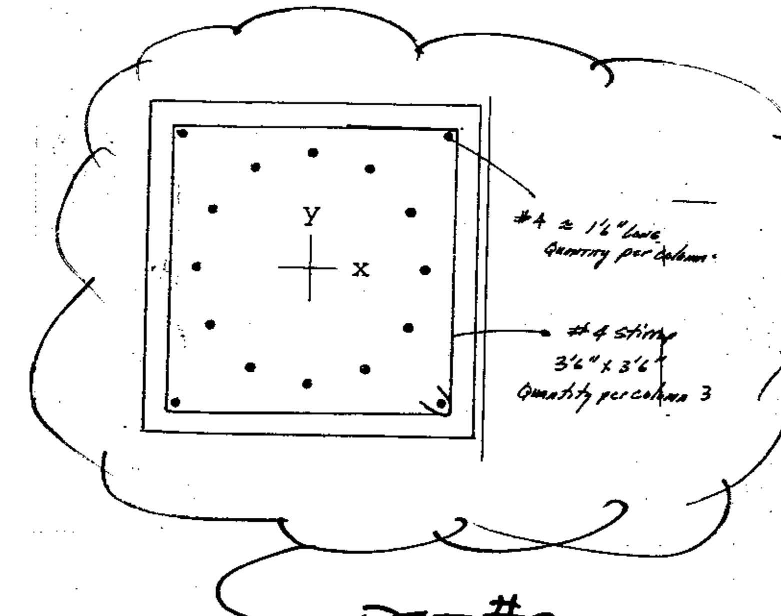
RFI#114 FOR DRILLED SHAFT #1, DEVELOPMENT LENGTH EQUALS 1.070 M.



ELEVATION - 1.200m ϕ DRILLED SHAFT



SECTION A-A



RFI #87
ABUT #1 (PIER #1) TOP 2' FORMED SQUARE, ADD ADD'L. REBAR AS NOTED ABOVE.

ESTIMATED QUANTITIES / END BENT 1		
Drilled Shaft No.	Drilled Shaft Length	Quantity
1	13.991 m	1
2	14.111 m	1
3	14.471 m	1
4	14.591 m	1

ESTIMATED QUANTITIES / END BENT 12		
Drilled Shaft No.	Drilled Shaft Length	Quantity
1	14.060 m	1
2	13.940 m	1
3	13.580 m	1
4	13.460 m	1

APPROVED FOR CONSTRUCTION
JMI ENGINEERS, INC.
BY
1/27/03

Notes:

1. Concrete shall be poured continuously from tip elevation to shaft head elevation of drilled shaft.
2. Concrete shall be Class IV, 28 MPa.

Philip M Hartsfield
1/15/04

Philip M Hart
1/27/03

01/27/2003 04:43:49 PM p19576 - ringling ab\cadd\dgn\drillshaft\shaft2.dgn

BRIDGE NO. 170176

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
01-04	JJ	AS BUILTS			

	NAMES	DATES
DRAWN BY	JLS	11/14/01
CHECKED BY	CWN	11/14/01
DESIGNED BY	DCG	11/14/01
CHECKED BY	PMH	11/14/01
APPROVED BY		

ENGINEER OF RECORD: PARSONS

LOGO: PCL CONSTRUCTION LEADERS

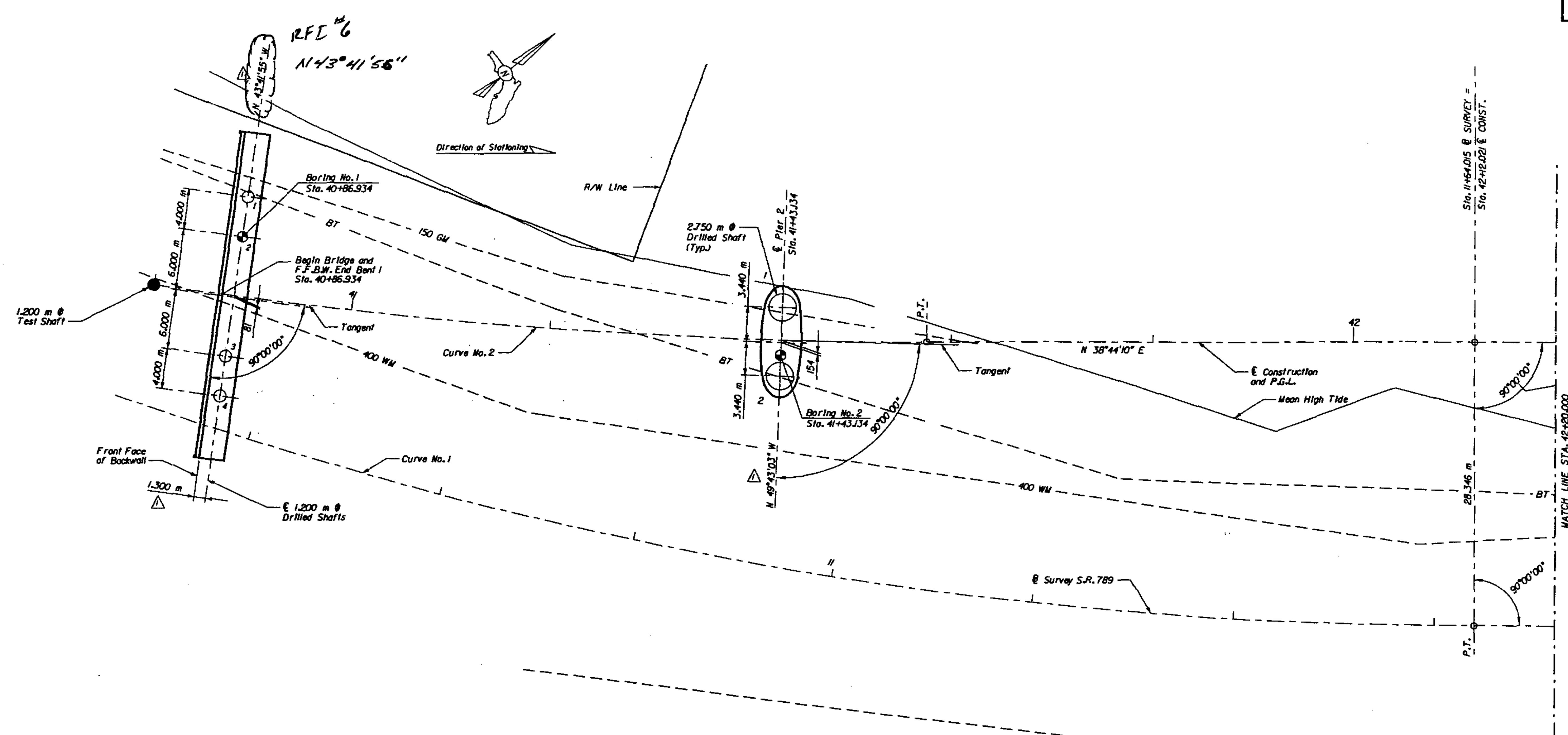
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

SHEET TITLE: 1.200m ϕ DRILLED SHAFT

PROJECT NAME: STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY

PARSONS



LEGEND

- Indicates Pier Outline
- Indicates Seagrass
- Statnamic Load Test
- Pilot Hole Boring / Rock Core

DRILLED SHAFT DATA TABLE

	Installation Criteria					Design Criteria		
Pier or Bent	Shaft Size (m)	No. of Shafts	Antic. Tip Elev. (m)	Min. Tip Elev. (m)	Min. Uncased Rock Socket Length (m)	Factored Design Loads (KN)	Downdrag (KN)	Long-Term/100 yr. Scour Elev. (m)
E.B. 1	1.200	4	-11.6	-11.6	2.1	8900	250	N/A
Pier 2	2.750	2	-17.5	-17.5	8.0	50,000	0	-9.5

TOP OF DRILLED SHAFT ELEVATIONS

Pier or Bent	Drilled Shaft No.	Elevation
E.B. 1	1	2.391
E.B. 1	2	2.511
E.B. 1	3	2.871
E.B. 1	4	2.991
Pier 2	1 & 2	-0.325

Notes:

1. For horizontal curve data see sheets B-5 thru B-8.
2. Minimum Tip Elevations for Piers 2 through 11 are controlled by lateral stability requirements.
3. Prior to construction of the proprietary walls at End Bents 1 and 12, the portion of the end bent shafts exposed above the existing grade shall be wrapped with polyethylene sheeting in accordance with Section 459 of the FDOT Standard Specifications for Road and Bridge Construction (2000) and supplements thereto.

APPROVED FOR CONSTRUCTION
JMI ENGINEERS, INC.
BY 1/27/03

Philip M Hartsfield 1/16/04

Philip M Hartsfield 1/27/03

BRIDGE NO. 170176

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
2/20/02	CAB	Revised as noted.	0104	JJ	AS BUILTS
5/21/02	CWN	Removed proposed bulkhead.			

	NAMES	DATES
DRAWN BY	JLS	11/14/01
CHECKED BY	CWN	11/14/01
DESIGNED BY	DCG	11/14/01
CHECKED BY	PMH	11/14/01
APPROVED BY	Philip M. Hartsfield, P.E.	

ENGINEER OF RECORD: PARSONS

LOGO: PCL CONSTRUCTION LEADERS

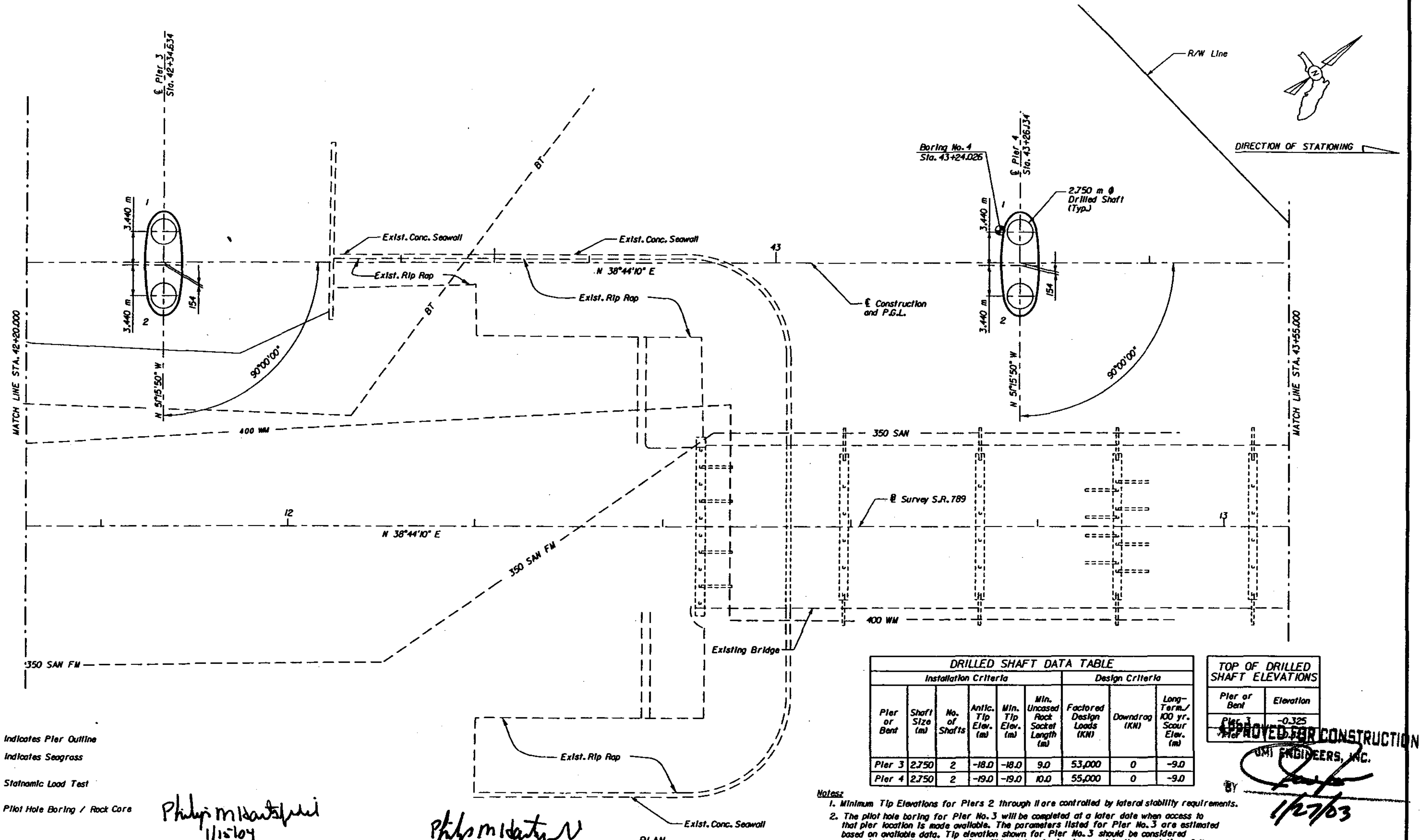
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

SHEET TITLE: FOUNDATION LAYOUT (1 OF 7)

PROJECT NAME: STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY

SHEET NO. B-16



DRILLED SHAFT DATA TABLE								
Installation Criteria						Design Criteria		
Pier or Bent	Shaft Size (m)	No. of Shafts	Antic. Tip Elev. (m)	Min. Tip Elev. (m)	Min. Uncased Rock Socket Length (m)	Factored Design Loads (KN)	Downdrag (KN)	Long-Term./100 yr. Scour Elev. (m)
Pier 3	2.750	2	-18.0	-18.0	9.0	53,000	0	-9.0
Pier 4	2.750	2	-19.0	-19.0	10.0	55,000	0	-9.0

TOP OF DRILLED SHAFT ELEVATIONS	
Pier or Bent	Elevation
Pier 3	-0.325
Pier 4	[illegible]

APPROVED FOR CONSTRUCTION
UMI ENGINEERS, INC.
BY
1/27/03

LEGEND

- Indicates Pier Outline
- Indicates Seagrass
- Statnamic Load Test
- Pilot Hole Boring / Rock Core

Philip M Hartsfield
1/15/04

Philip M Hartsfield
1/27/03

Notes:

1. Minimum Tip Elevations for Piers 2 through 11 are controlled by lateral stability requirements.
2. The pilot hole boring for Pier No. 3 will be completed at a later date when access to that pier location is made available. The parameters listed for Pier No. 3 are estimated based on available data. Tip elevation shown for Pier No. 3 should be considered approximate. A final tip elevation will be determined subsequent to the completion of the Pier No. 3 pilot hole boring. Drilled shaft construction at Pier No. 3 shall not commence until the pilot hole has been completed and a design tip elevation established by the Engineer.

BRIDGE NO. 170176

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
5/21/02	CWN	Removed proposed bulkhead.			

	NAMES	DATES
DRAWN BY	JLS	11/14/01
CHECKED BY	CWN	11/14/01
DESIGNED BY	DCG	11/14/01
CHECKED BY	PMH	11/14/01
APPROVED BY	Philip M. Hartsfield, P.E.	

ENGINEER OF RECORD: PARSONS

LOGO: PCL CONSTRUCTION LEADERS

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

SHEET TITLE: FOUNDATION LAYOUT (2 OF 7)

PROJECT NAME: STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY

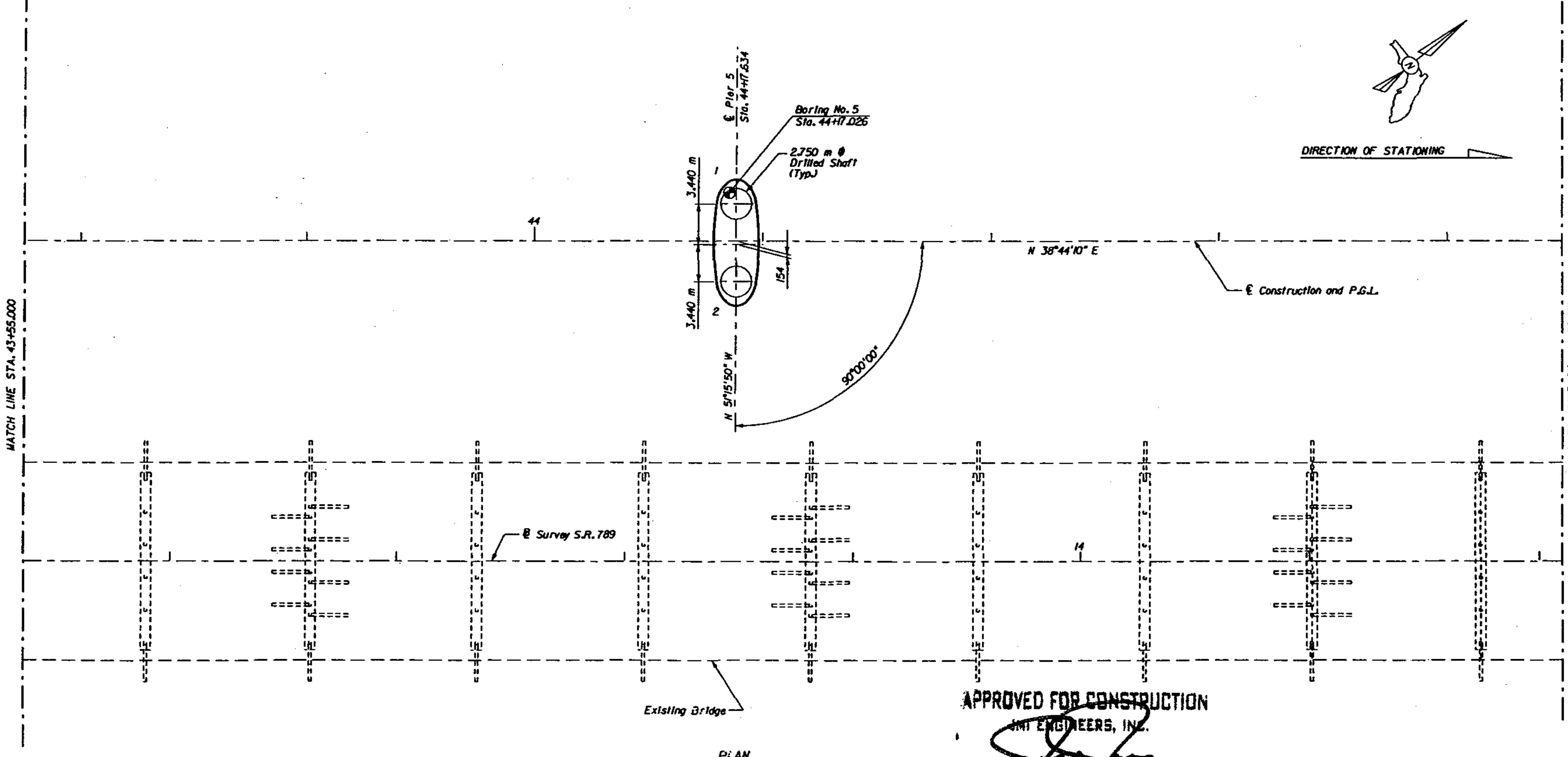
PARSONS - 01/27/2003 04:13:59 PM ptt9576 - ringling dbscad\dgn\foundlay2.dgn

LEGEND

- Indicates Pier Outline
- Indicates Seagrass
- Statnamic Load Test
- Pilot Hole Boring / Rock Core

DRILLED SHAFT DATA TABLE								
Installation Criteria						Design Criteria		
Pier or Bent	Shaft Size (m)	No. of Shafts	Antic. Tip Elev. (m)	Min. Tip Elev. (m)	Min. Uncased Rock Socket Length (m)	Factored Design Loads (KN)	Downdrag (KN)	Long-Term/100 yr. Scour Elev. (m)
Pier 5	2.750	2	-15.5	-15.5	8.0	55,750	0	-7.5

TOP OF DRILLED SHAFT ELEVATIONS	
Pier or Bent	Elevation
Pier 5	-0.325



PLAN

APPROVED FOR CONSTRUCTION
[?] ENGINEERS, INC.
BY
1/27/03

Philip M Hartsfield
1/15/04

Philip M Hartsfield
1/27/03

Notes:

1. Minimum Tip Elevations for Piers 2 through 11 are controlled by lateral stability requirements.

BRIDGE NO. 170176

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

	NAMES	DATES
DRAWN BY	JLS	11/14/01
CHECKED BY	CWN	11/14/01
DESIGNED BY	DCG	11/14/01
CHECKED BY	PMH	11/14/01
APPROVED BY	Philip M. Hartsfield, P.E.	

ENGINEER OF RECORD: PARSONS

LOGO: PCL CONSTRUCTION LEADERS

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

SHEET TITLE: FOUNDATION LAYOUT (3 OF 7)

PROJECT NAME: STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY

PARSONS - 01/27/2003 04:44:00 PM p:\9576 - ringling db\cadd\dgn\foundlay3.dgn

LEGEND

Indicates Pier Outline

Indicates Seagrass

Statnamic Load Test

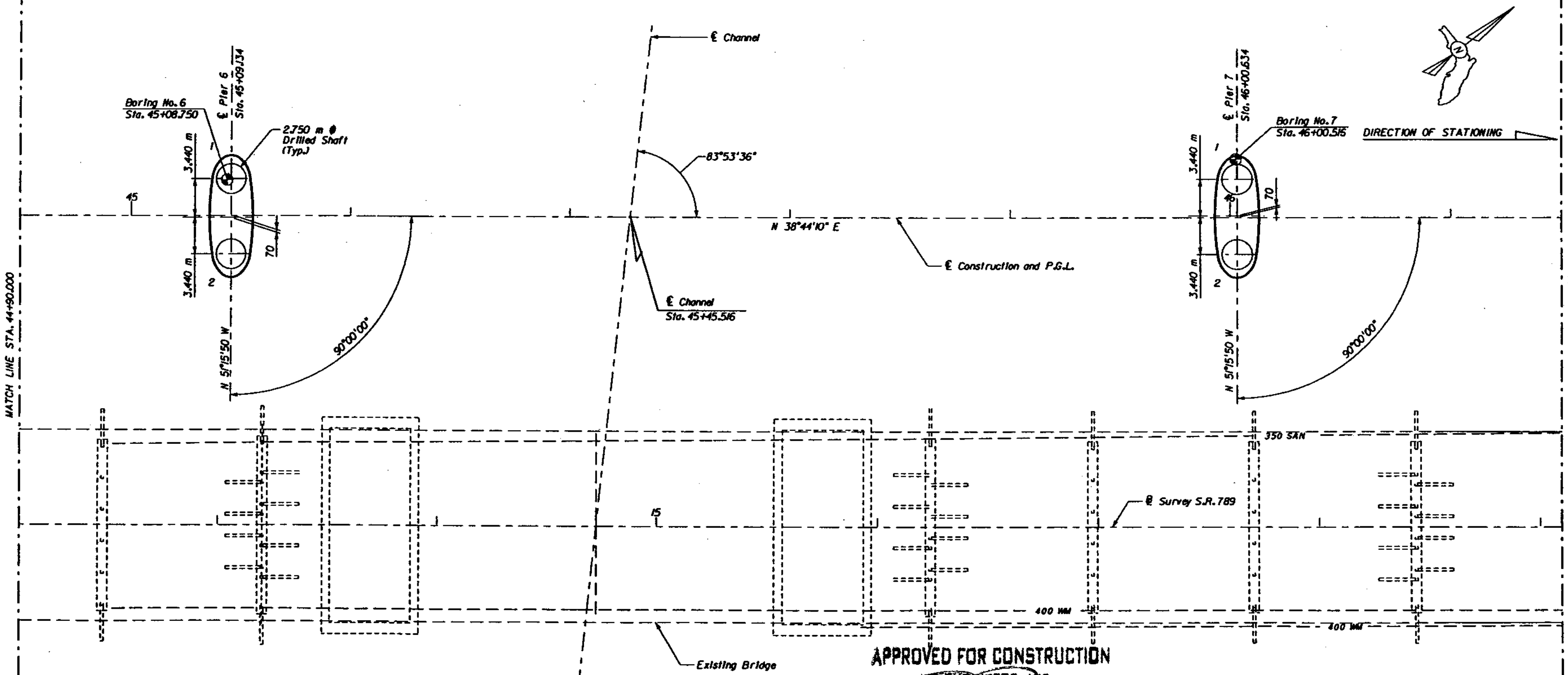
Pilot Hole Boring / Rock Core

DRILLED SHAFT DATA TABLE

Pier or Bent	Installation Criteria: Shaft Size (m)	No. of Shafts	Antic. Tip Elev. (m)	Min. Tip Elev. (m)	Min. Uncased Rock Socket Length (m)	Design Criteria: Factored Design Loads (KN)	Downdrag (KN)	Long-Term/100 yr. Scour Elev. (m)
Pier 6	2.750	2	-17.5	-17.5	8.0	55,750	0	-9.5
Pier 7	2.750	2	-14.0	-14.0	8.0	55,750	0	-6.0

TOP OF DRILLED SHAFT ELEVATIONS

Pier or Bent	Elevation
Pier 6	-0.325
Pier 7	-0.325



PLAN

APPROVED FOR CONSTRUCTION
JMI ENGINEERS, INC.
BY
1/27/03

Philip M Hartsfield
1/15/04

Philip M Hartsfield
1/27/03

Notes:
1. Minimum Tip Elevations for Piers 2 through 11 are controlled by lateral stability requirements.

BRIDGE NO. 170176

REVISIONS DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

	NAMES	DATES
DRAWN BY	JLS	11/14/01
CHECKED BY	CWN	11/14/01
DESIGNED BY	DCG	11/14/01
CHECKED BY	PMH	11/14/01
APPROVED BY	Philip M. Hartsfield, P.E.	

ENGINEER OF RECORD: PARSONS

LOGO: PCL CONSTRUCTION LEADERS

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

SHEET TITLE: FOUNDATION LAYOUT (4 OF 7)

PROJECT NAME: STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY

PARSONS - 01/27/2003 04:41:01 PM p:\9576 - ringling c:\caddr\sgn\foundlay4.dgn

R/W Line

LEGEND

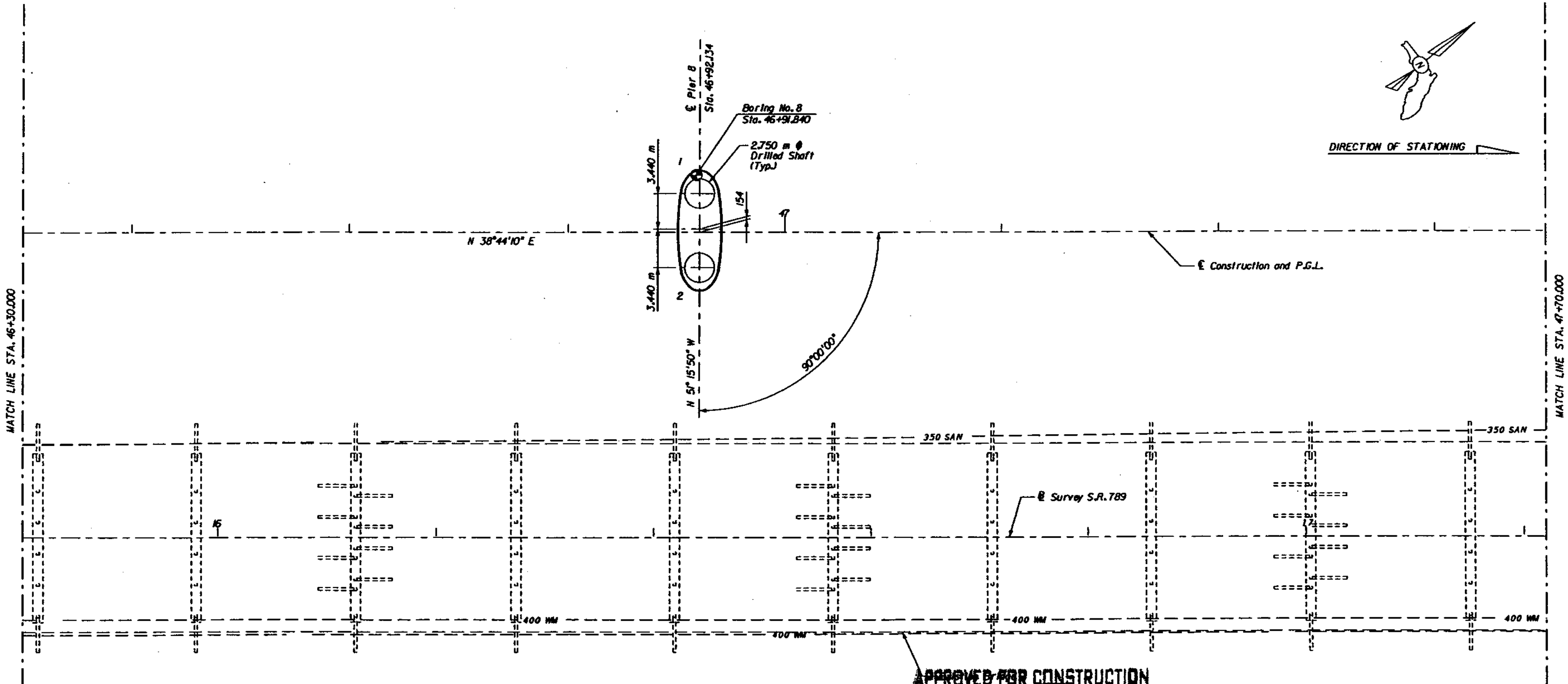
- Indicates Pier Outline
- Indicates Seagrass
- Statnamic Load Test
- Pilot Hole Boring / Rock Core

DRILLED SHAFT DATA TABLE

Installation Criteria						Design Criteria		
Pier or Bent	Shaft Size (m)	No. of Shafts	Antic. Tip Elev. (m)	Min. Tip Elev. (m)	Min. Uncased Rock Socket Length (m)	Factored Design Loads (KN)	Downdrag (KN)	Long-Term/100 yr. Scour Elev. (m)
Pier 8	2.750	2	-14.5	-14.5	8.5	55,750	0	-6.0

TOP OF DRILLED SHAFT ELEVATIONS

Pier or Bent	Elevation
Pier 8	-0.325



PLAN

APPROVED FOR CONSTRUCTION
JMI ENGINEERS, INC.
BY
1/27/03

Philip M Hartsfield
1/15/04

Philip M Hartsfield
1/27/03

Notes:
1. Minimum Tip Elevations for Piers 2 through 11 are controlled by lateral stability requirements.

BRIDGE NO. 170176

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REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

	NAMES	DATES
DRAWN BY	JLS	11/14/01
CHECKED BY	CWN	11/14/01
DESIGNED BY	DCG	11/14/01
CHECKED BY	PMH	11/14/01
APPROVED BY	Philip M. Hartsfield, P.E.	

ENGINEER OF RECORD:
PARSONS

LOGO:
PCL
CONSTRUCTION LEADERS

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

SHEET TITLE:
FOUNDATION LAYOUT (5 OF 7)

PROJECT NAME:
STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY

PARSONS

LEGEND

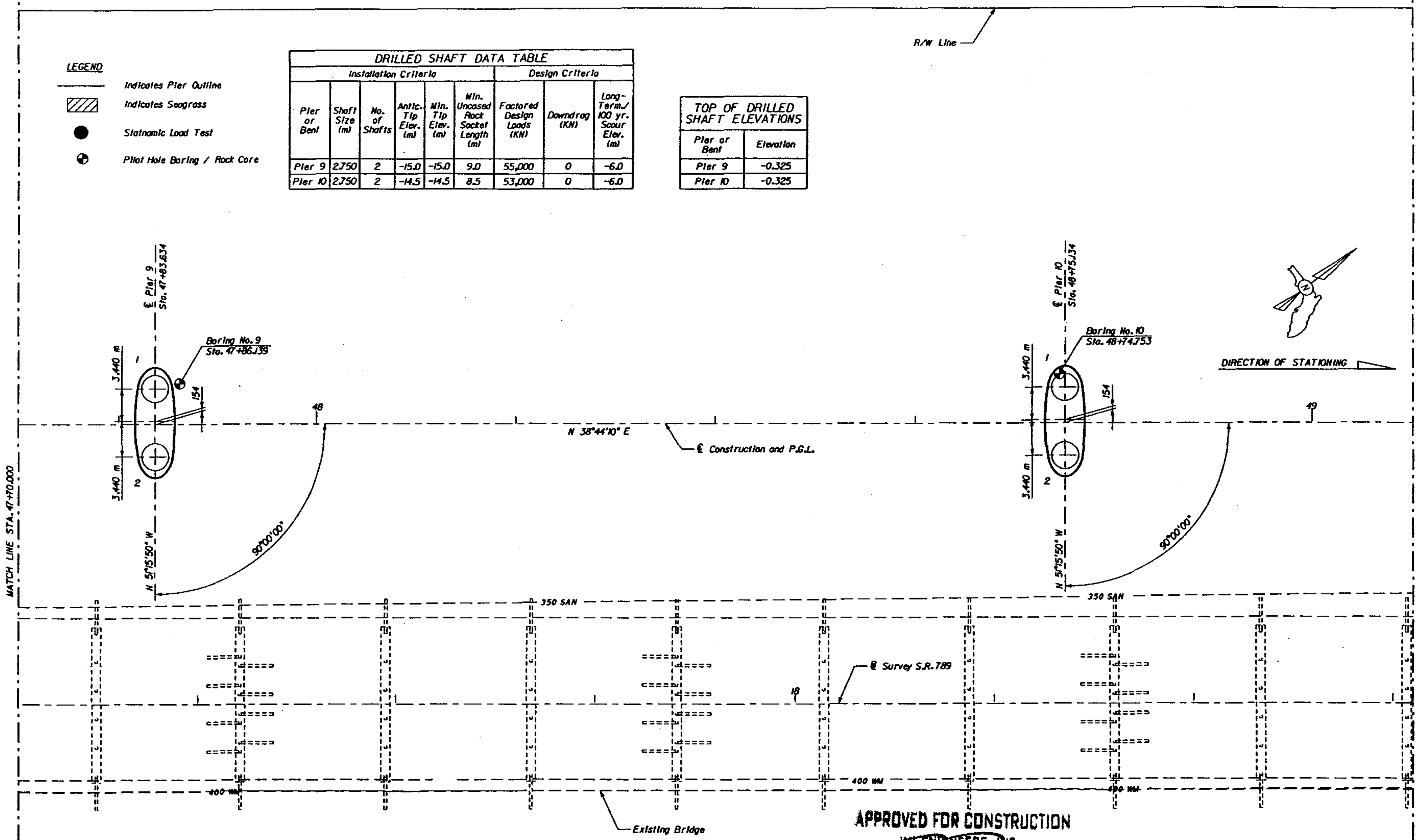
- Indicates Pier Outline
- Indicates Seagrass
- Statnamic Load Test
- Pilot Hole Boring / Rock Core

DRILLED SHAFT DATA TABLE

Installation Criteria						Design Criteria		
Pier or Bent	Shaft Size (m)	No. of Shafts	Antic. Tip Elev. (m)	Min. Tip Elev. (m)	Min. Uncased Rock Socket Length (m)	Factored Design Loads (KN)	Downdrag (KN)	Long-Term./100 yr. Scour Elev. (m)
Pier 9	2.750	2	-15.0	-15.0	9.0	55,000	0	-6.0
Pier 10	2.750	2	-14.5	-14.5	8.5	53,000	0	-6.0

TOP OF DRILLED SHAFT ELEVATIONS

Pier or Bent	Elevation
Pier 9	-0.325
Pier 10	-0.325



PLAN

APPROVED FOR CONSTRUCTION
JMI ENGINEERS, INC.
1/27/03

Philip M Hartsfield 1/15/04

Philip M Hartsfield 1/27/03

Notes:
1. Minimum Tip Elevations for Piers 2 through 11 are controlled by lateral stability requirements.

BRIDGE NO. 170176

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

	NAMES	DATES
DRAWN BY	JLS	11/14/01
CHECKED BY	CWN	11/14/01
DESIGNED BY	DCG	11/14/01
CHECKED BY	PMH	11/14/01
APPROVED BY	Philip M. Hartsfield, P.E.	

ENGINEER OF RECORD: PARSONS

LOGO: PCL CONSTRUCTION LEADERS

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

SHEET TITLE: FOUNDATION LAYOUT (6 OF 7)

PROJECT NAME: STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY

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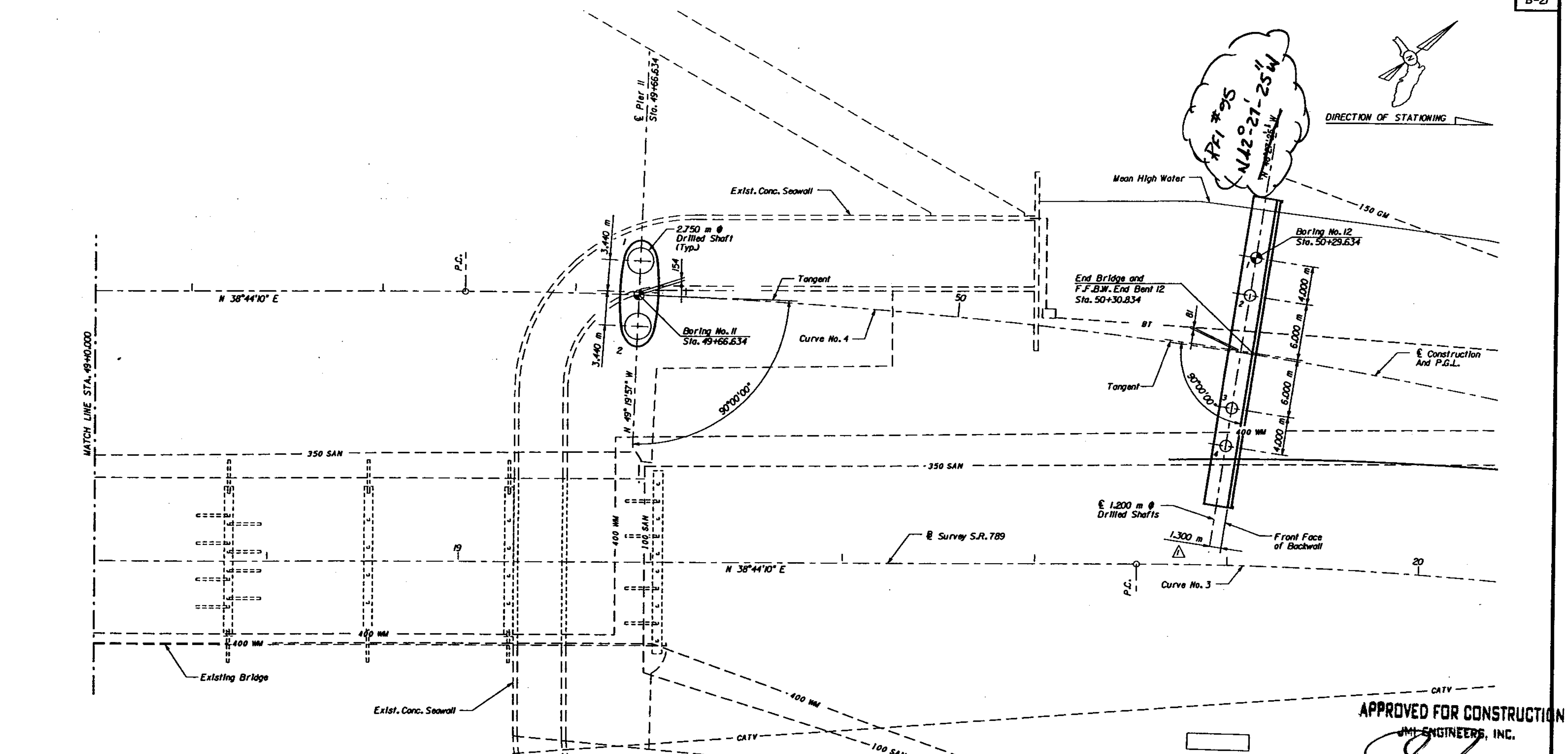
PARSONS



LEGEND

- Indicates Pier Outline
- Indicates Seagrass
- Statnamic Load Test
- Pilot Hole Boring / Rock Core

DRILLED SHAFT DATA TABLE

Installation Criteria						Design Criteria		
Pier or Bent	Shaft Size (m)	No. of Shafts	Antic. Tip Elev. (m)	Min. Tip Elev. (m)	Min. Uncased Rock Socket Length (m)	Factored Design Loads (KN)	Downdrag (KN)	Lon-Term/100 yr. Scour Elev. (m)
Pier 11	2.750	2	-15.5	-15.5	8.0	50,000	0	-7.5
E.B. 12	1.200	4	-10J	-10J	2J	8900	200	N/A

TOP OF DRILLED SHAFT ELEVATIONS

Pier or Bent	Drilled Shaft No.	Elevation
Pier 11	1 & 2	0.975
E.B. 12	1	3.960
E.B. 12	2	3.840
E.B. 12	3	3.480
E.B. 12	4	3.360

Philip M Hartsfield 1/27/03 Philip M Hartsfield 1/15/04

Notes:

1. For horizontal curve data see sheets B-5 thru B-8.
2. Minimum Tip Elevations for Piers 2 through 11 are controlled by lateral stability requirements.
3. Prior to construction of the proprietary walls at End Bents 1 and 12, the portion of the end bent shafts exposed above the existing grade shall be wrapped with polyethylene sheeting in accordance with Section 459 of the FDOT Standard Specifications for Road and Bridge Construction (2000) and supplements thereto.

BRIDGE NO. 170176

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
2/20/02	CAB	Revised as noted.	01/04	JJ	AS BUILTS
5/21/02	CWN	Removed proposed bulkhead.			

	NAMES	DATES
DRAWN BY	JLS	11/14/01
CHECKED BY	CWN	11/14/01
DESIGNED BY	DCG	11/14/01
CHECKED BY	PMH	11/14/01
APPROVED BY	Philip M. Hartsfield, P.E.	

ENGINEER OF RECORD: PARSONS

LOGO: PCL CONSTRUCTION LEADERS

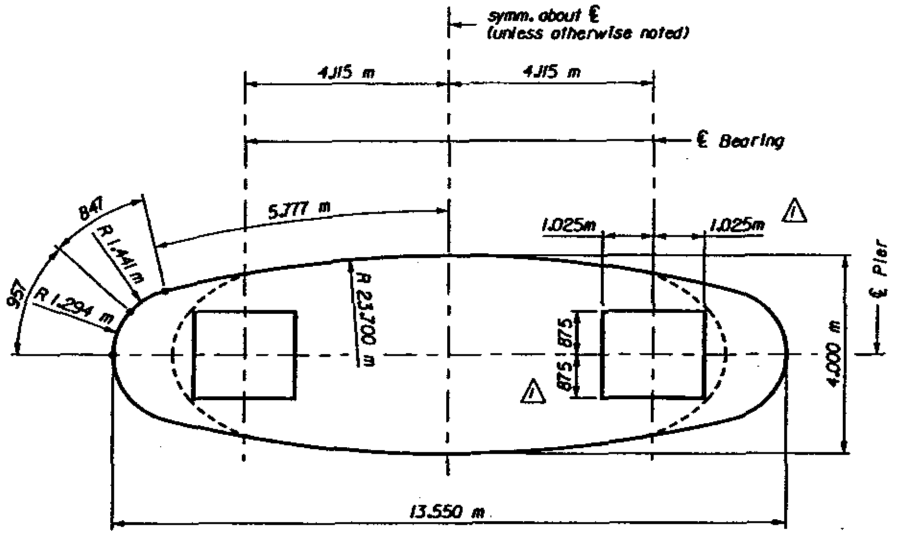
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

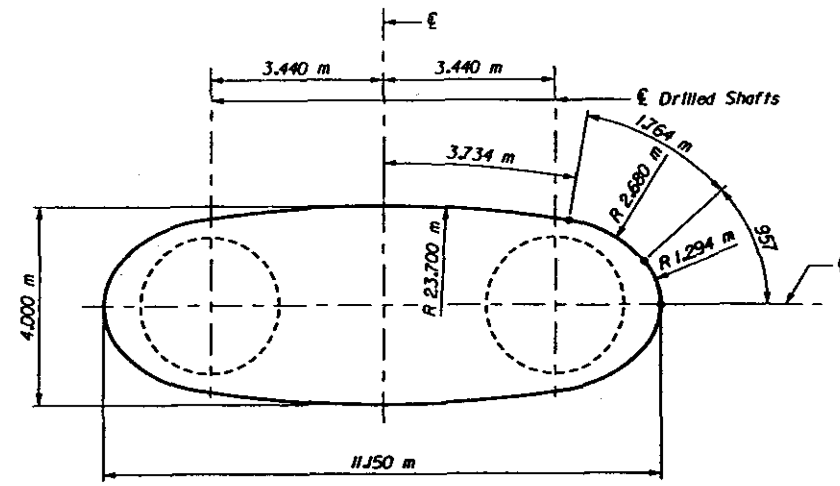
SHEET TITLE: FOUNDATION LAYOUT (7 OF 7)

PROJECT NAME: STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY

SHEET NO. B-28

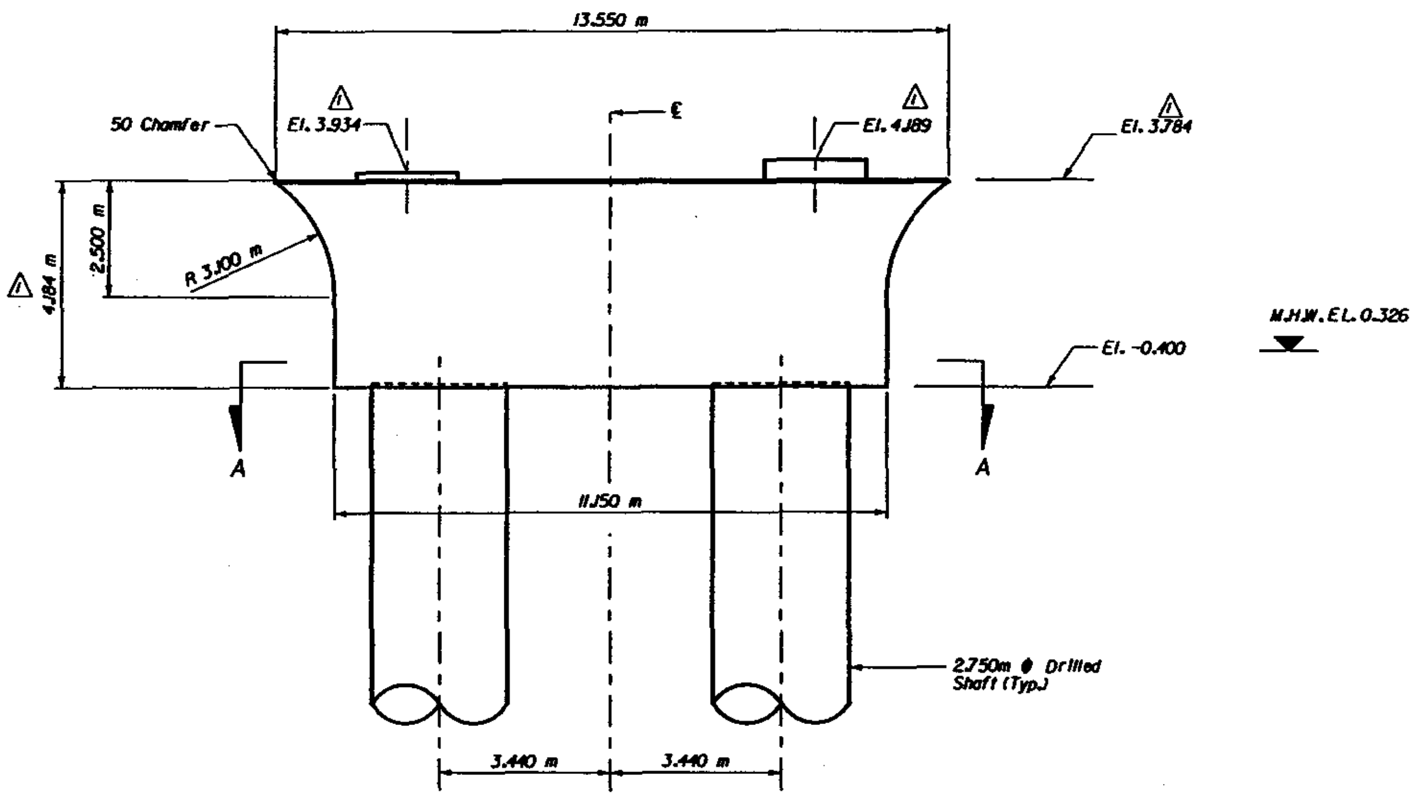


PLAN

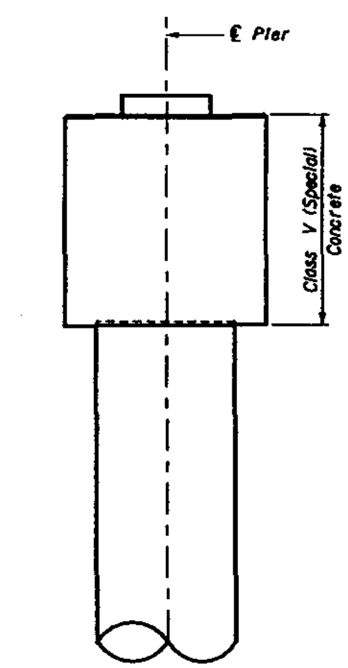


SECTION A-A

ESTIMATED QUANTITIES	
Concrete	Quantity
Class V (Special) (41 MPa)	167.40 Cu. m.



ELEVATION
(LOOKING UP-STATION)



SIDE VIEW

APPROVED FOR CONSTRUCTION
DMI ENGINEERS, INC.
BY
1/27/03

Notes:
1. Concrete Quantity is for pier only and does not include seal or drilled shafts.

Philip M Hartsfield
1/27/03

Philip M Hartsfield
1/15/04

BRIDGE NO. 170176

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
2/20/02	CAB	Revised as noted.			

	NAMES	DATES
DRAWN BY	STP	11/14/01
CHECKED BY	CWN	11/14/01
DESIGNED BY	DCG	11/14/01
CHECKED BY	PMH	11/14/01
APPROVED BY	Philip M. Hartsfield, P.E.	

ENGINEER OF RECORD: PARSONS

LOGO: CONSTRUCTION LEADERS

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

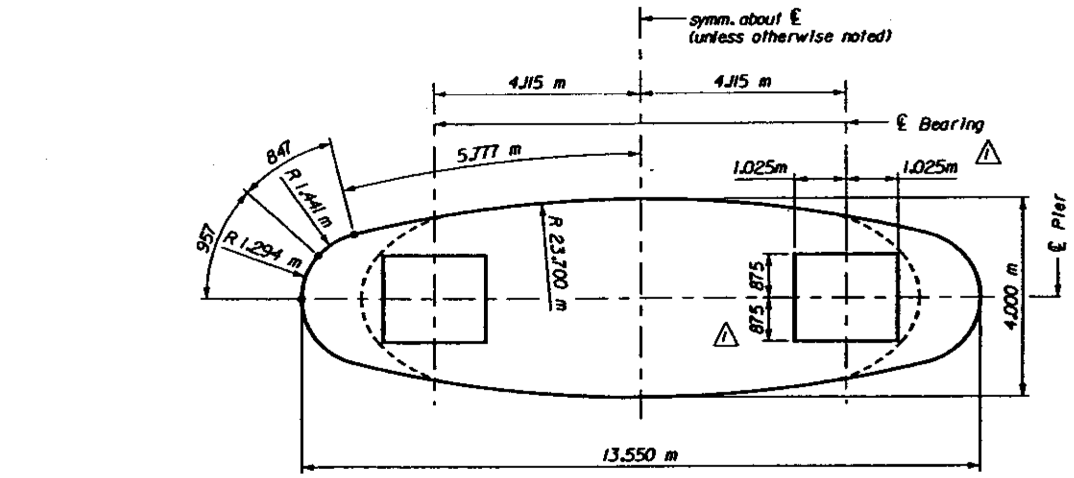
ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

SHEET TITLE: PIER 2 DIMENSIONS

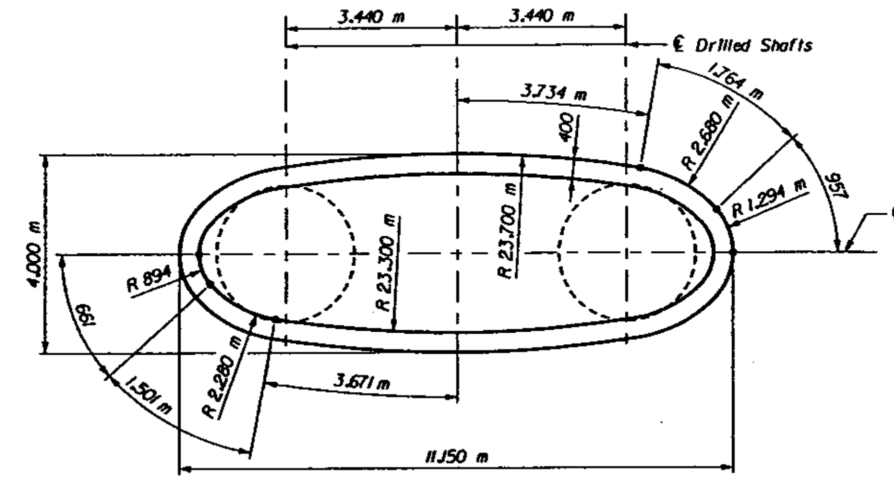
PROJECT NAME: STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY

PARSONS - 01/27/2003 04:44:05 PM gh9576 - ringling dtbcadd\sgn\pier2.dgn

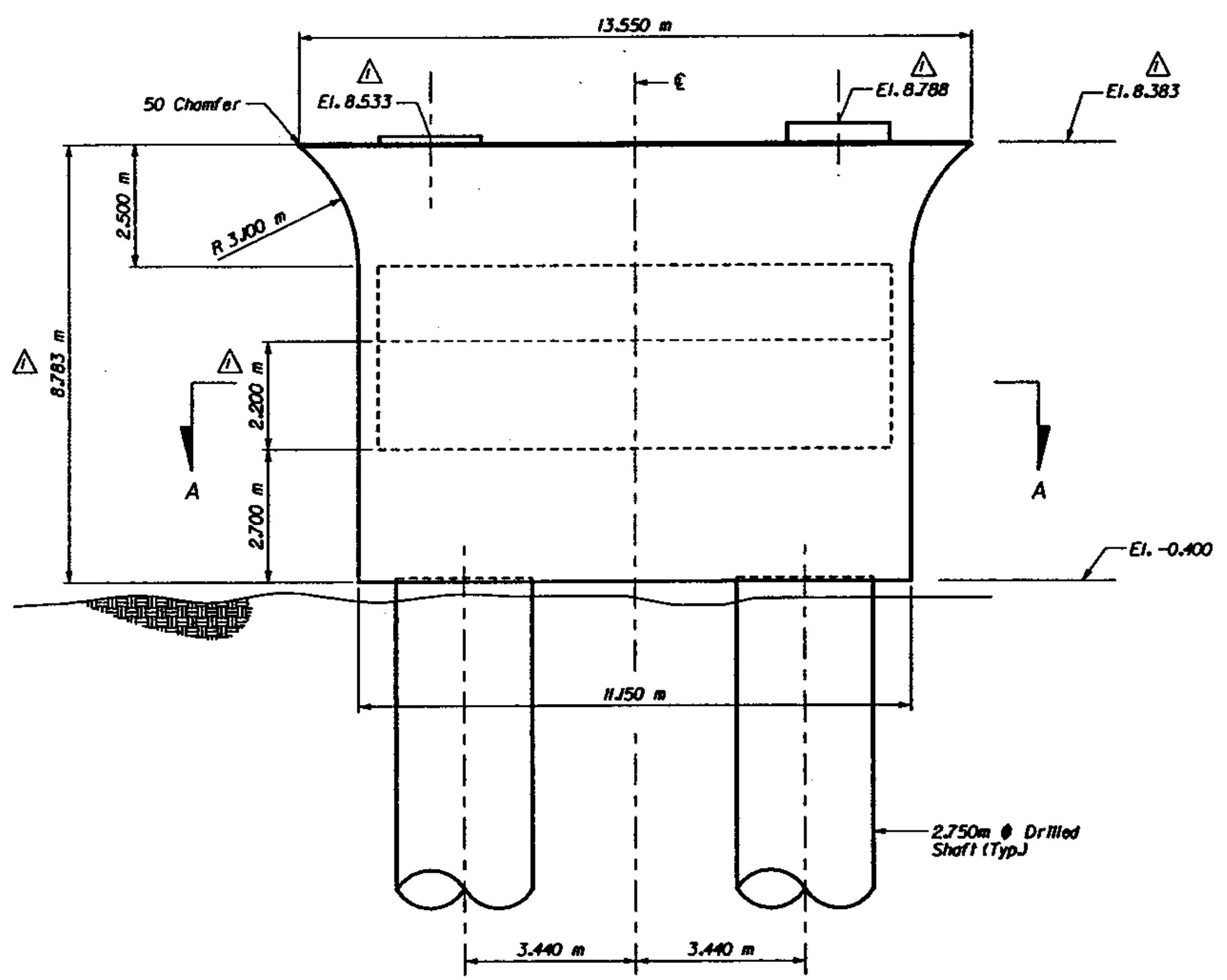
SHEET NO. B-29



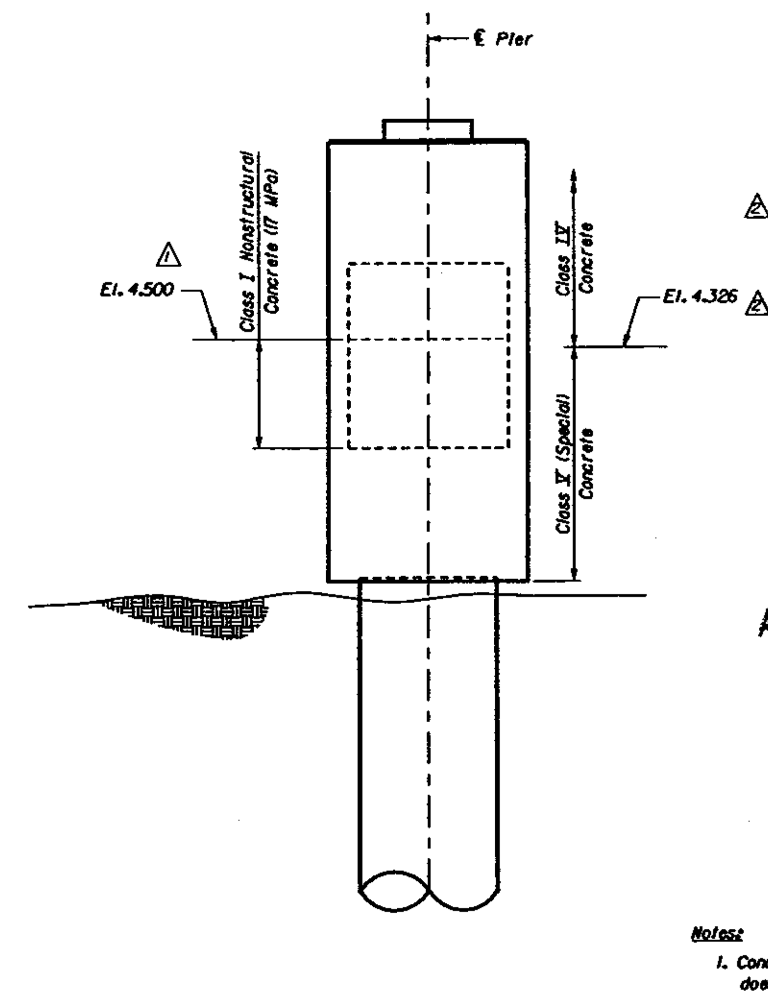
PLAN



SECTION A-A



ELEVATION
(LOOKING UP-STATION)



SIDE VIEW

ESTIMATED QUANTITIES	
Concrete	Quantity
Class I (21 MPa)	6.112 Cu. m.
Class IV (38 MPa)	119.48 Cu. m.
Class V (Special) (41 MPa)	121.43 Cu. m.

APPROVED FOR CONSTRUCTION
JMT ENGINEERS, INC.
BY
1/27/03

Notes:

1. Concrete Quantity is for pier only and does not include seal or drilled shafts.

Philip M Hatsfield
1/27/03

Philip M Hatsfield
1/15/04

BRIDGE NO. 170176

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
2/20/02	CAB	Revised as noted.			
7/18/02	CWN	Revised concrete class, added elevation.			

	NAMES	DATES
DRAWN BY	STP	11/14/01
CHECKED BY	CWN	11/14/01
DESIGNED BY	DCG	11/14/01
CHECKED BY	PMH	11/14/01
APPROVED BY	Philip M. Hartsfield, P.E.	

ENGINEER OF RECORD: PARSONS

LOGO: PCL CONSTRUCTION LEADERS

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

SHEET TITLE: PIER 3 DIMENSIONS

PROJECT NAME: STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY

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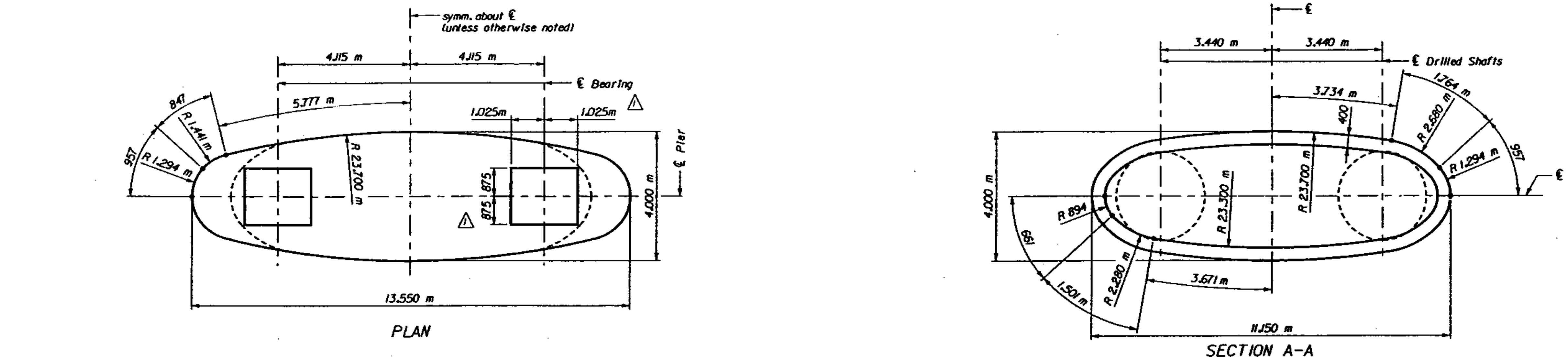
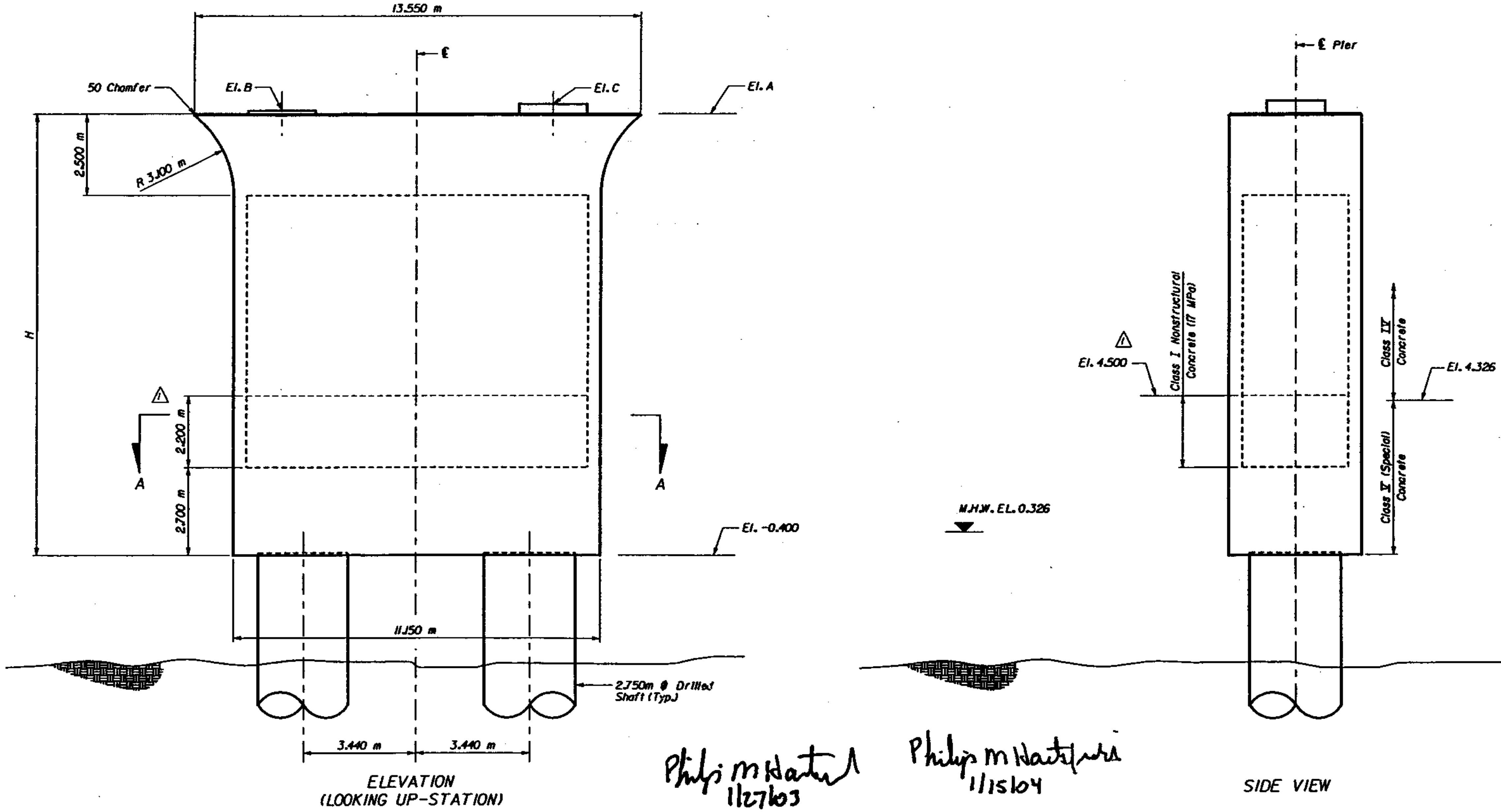




Philip M Hartsfield 1/27/03

Philip M Hartsfield 1/15/04

PIER DIMENSIONS

Pier	El. A	El. B	El. C	H
4	12.868	13.018	13.277	13.268
9	12.870	13.278	13.020	13.270
10	8.928	9.333	9.078	9.328

ESTIMATED QUANTITIES

Pier	Concrete	Quantity
4	Class I (21 MPa)	61.12 Cu. m.
	Class IV (38 MPa)	163.75 Cu. m.
	Class V (Special) (41 MPa)	121.43 Cu. m.
9	Class I (21 MPa)	61.12 Cu. m.
	Class IV (38 MPa)	163.75 Cu. m.
	Class V (Special) (41 MPa)	121.43 Cu. m.
10	Class I (21 MPa)	61.12 Cu. m.
	Class IV (38 MPa)	124.85 Cu. m.
	Class V (Special) (41 MPa)	121.43 Cu. m.

APPROVED FOR CONSTRUCTION
UMI ENGINEERS, INC.
BY
1/27/03

Notes:

1. Concrete Quantity is for pier only and does not include seal or drilled shafts.

BRIDGE NO. 170176

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
2/20/02	CAB	Revised as noted.			
7/18/02	CWH	Revised concrete class and compressive strength.			

	NAMES	DATES
DRAWN BY	STP	11/14/01
CHECKED BY	CWH	11/14/01
DESIGNED BY	DCG	11/14/01
CHECKED BY	PMH	11/14/01
APPROVED BY	Philip M. Hartsfield, P.E.	

ENGINEER OF RECORD: PARSONS

LOGO: PCL CONSTRUCTION LEADERS

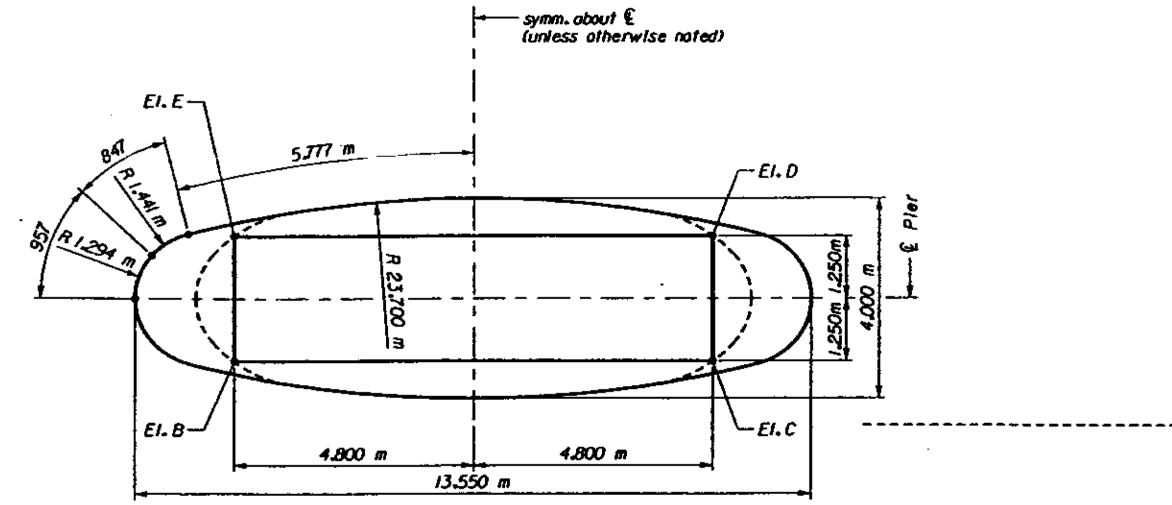
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

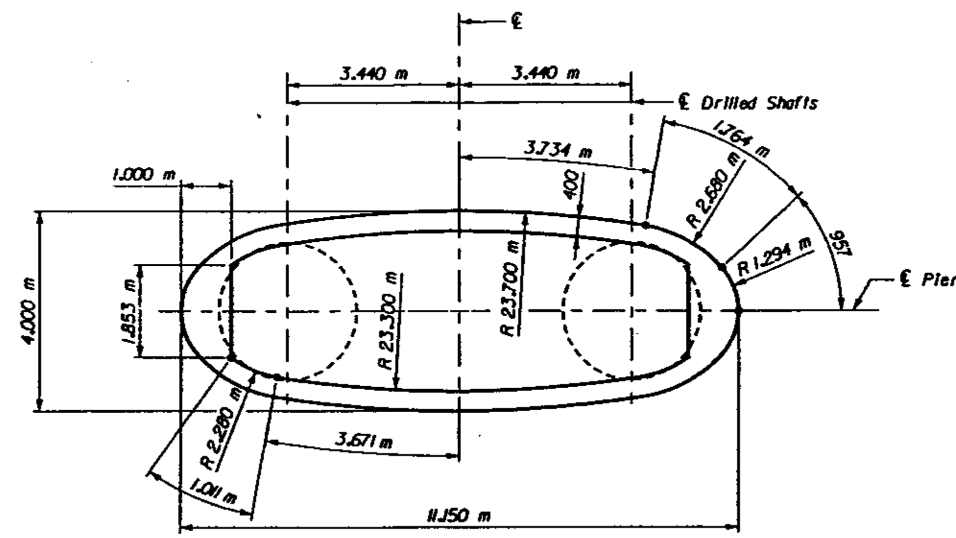
SHEET TITLE: PIERS 4, 9 & 10 DIMENSIONS

PROJECT NAME: STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY

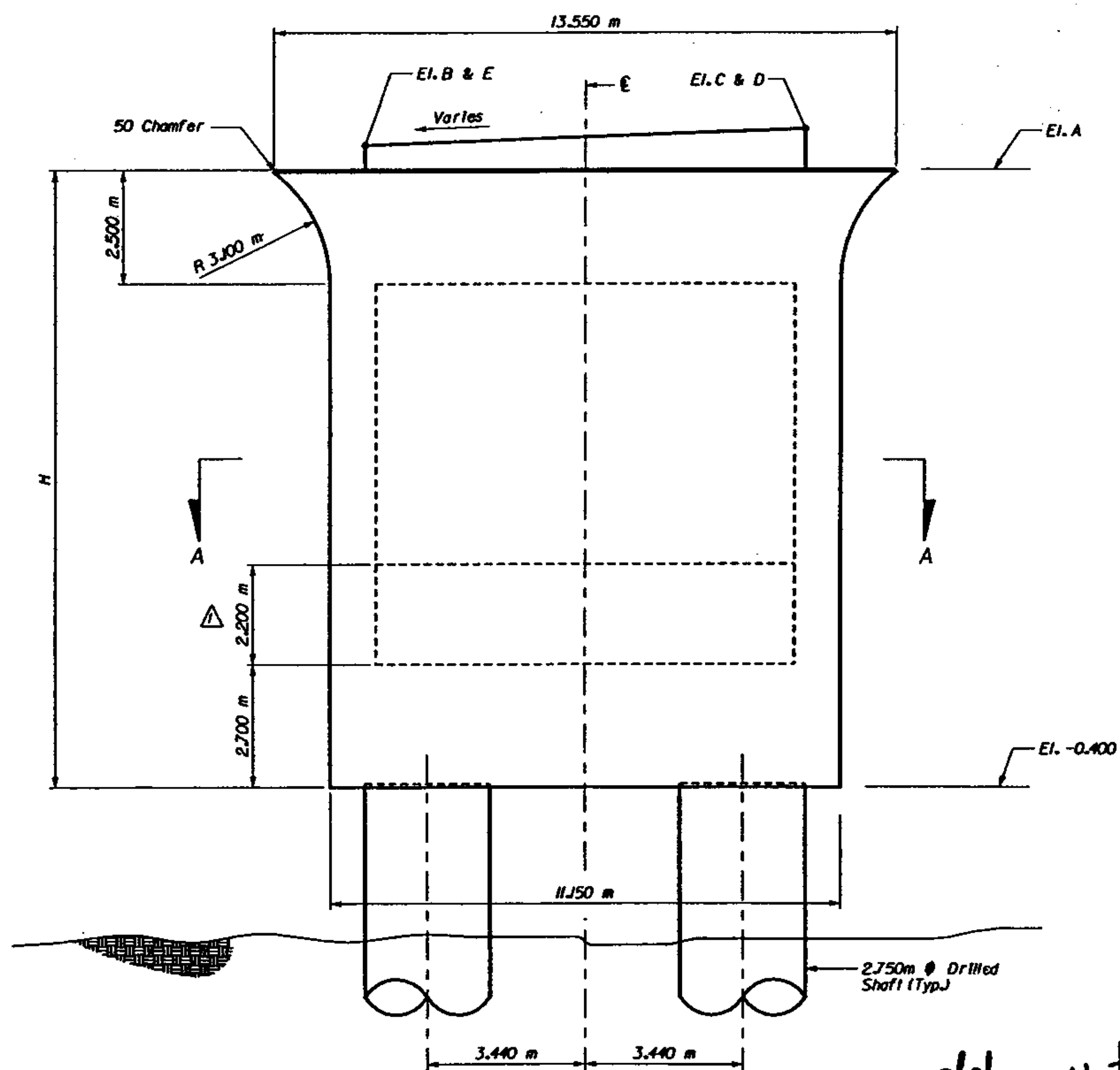
SHEET NO. B-31



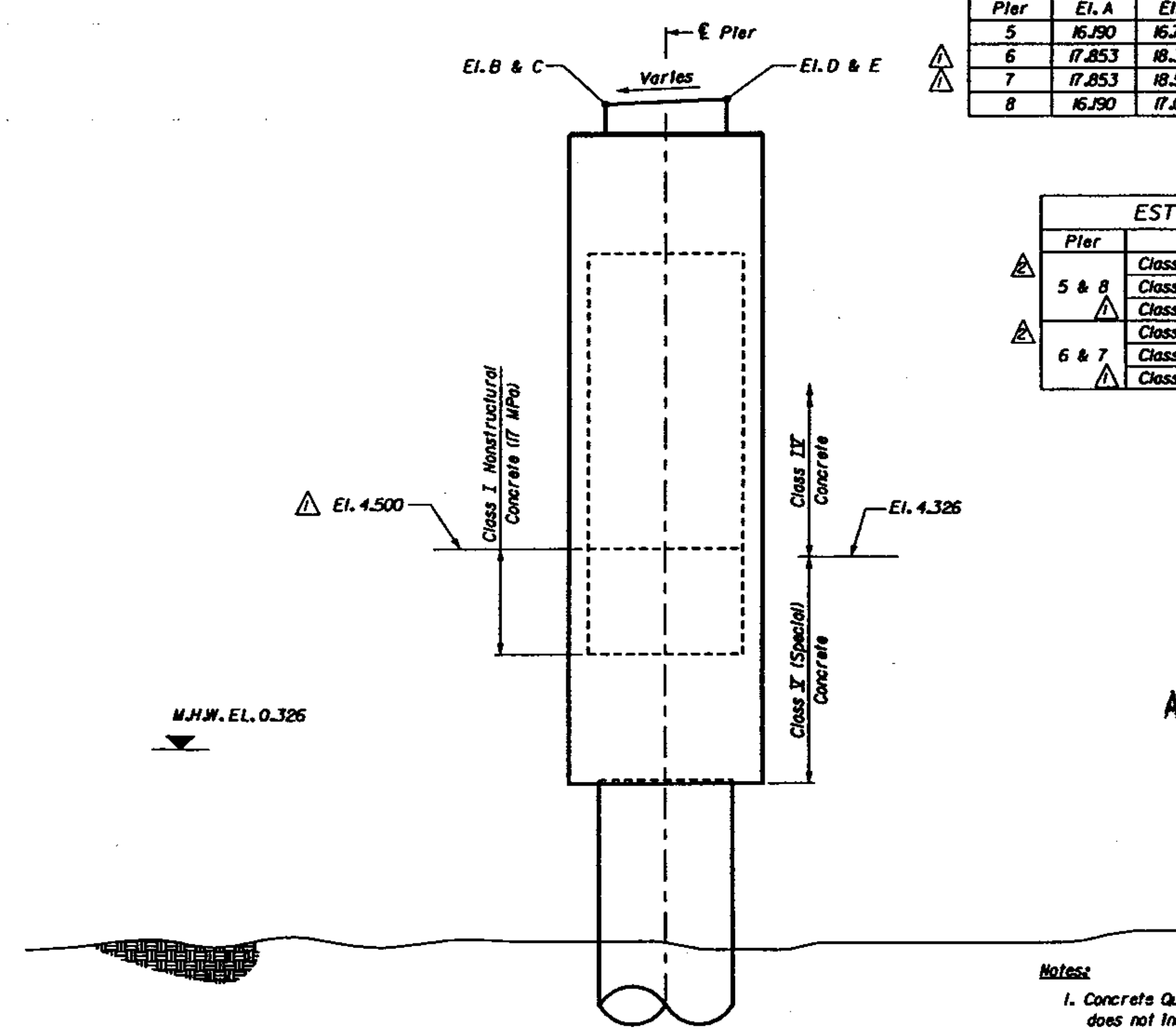
PLAN



SECTION A-A



ELEVATION



SIDE VIEW

PIER DIMENSIONS						
Pier	El. A	El. B	El. C	El. D	El. E	H
5	16.190	16.725	17.013	17.078	16.790	16.590
6	17.853	18.388	18.576	18.598	18.410	18.253
7	17.853	18.598	18.410	18.388	18.576	18.253
8	16.190	17.078	16.790	16.725	17.013	16.590

ESTIMATED QUANTITIES		
Pier	Concrete	Quantity
5 & 8	Class I (21 MPa)	57.76 Cu. m.
	Class IV (38 MPa)	226.67 Cu. m.
	Class V (Special) (41 MPa)	124.50 Cu. m.
6 & 7	Class I (21 MPa)	57.76 Cu. m.
	Class IV (38 MPa)	243.86 Cu. m.
	Class V (Special) (41 MPa)	124.50 Cu. m.

APPROVED FOR CONSTRUCTION
JMI ENGINEERS, INC.
BY
1/27/03

Notes:

1. Concrete Quantity is for pier only and does not include seal or drilled shafts.

Philip M Hartsfield 1/27/03

Philip M Hartsfield 1/15/04

BRIDGE NO. 170176

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
2/20/02	CAB	Revised as noted.			
7/16/02	CWN	Revised concrete class and compressive strength.			

	NAMES	DATES
DRAWN BY	STP	11/20/01
CHECKED BY	CWN	11/20/01
DESIGNED BY	DCG	11/20/01
CHECKED BY	PMH	11/20/01
APPROVED BY	Philip M. Hartsfield, P.E.	

ENGINEER OF RECORD: PARSONS

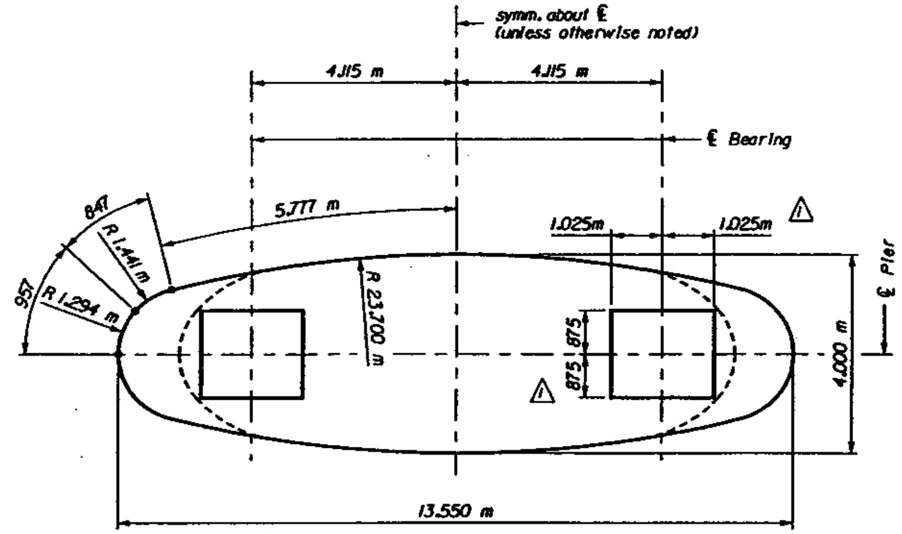
LOGO: PCL CONSTRUCTION LEADERS

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

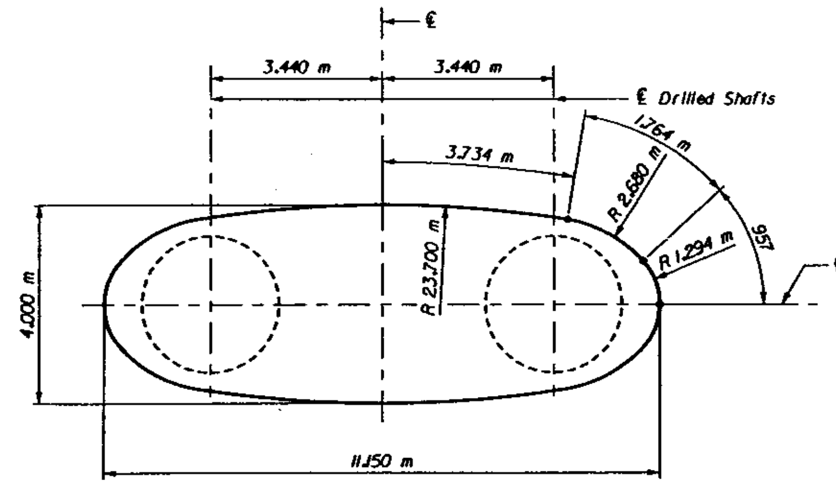
ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

SHEET TITLE: PIERS 5 THRU 8 DIMENSIONS

PROJECT NAME: STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY

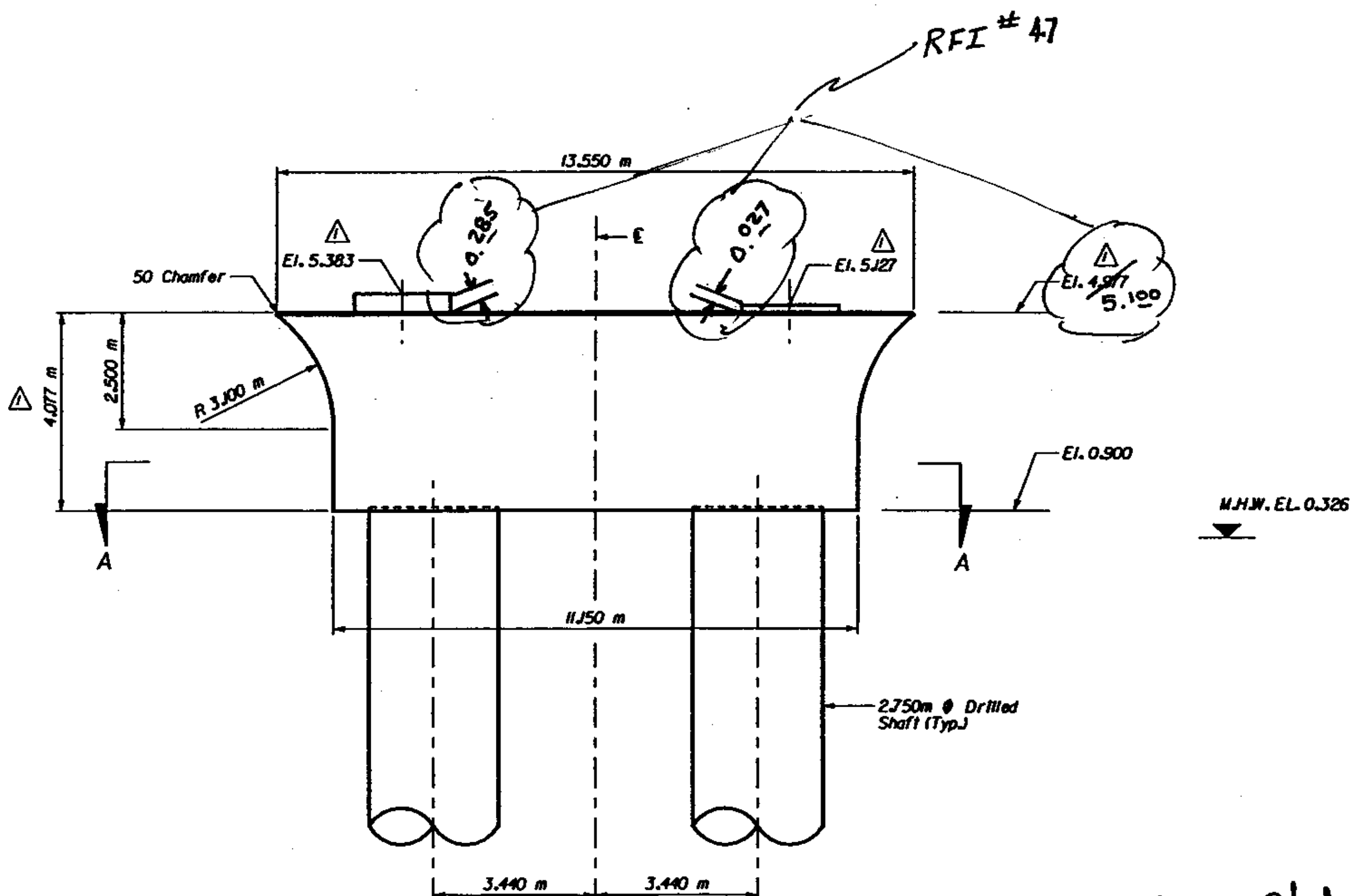


PLAN



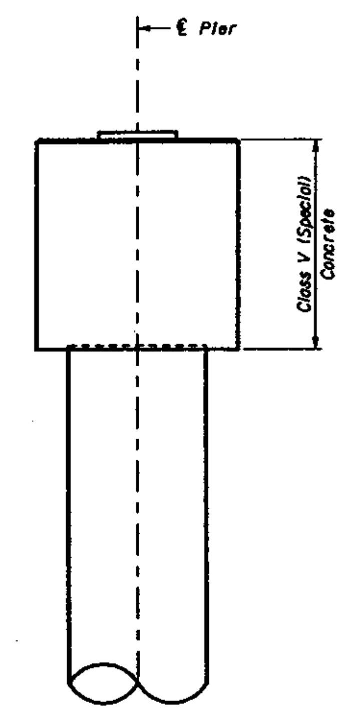
SECTION A-A

ESTIMATED QUANTITIES	
Concrete	Quantity
Class V (Special) (41 MPa)	163.32 Cu. m.



M.H.W. EL. 0.326

ELEVATION
(LOOKING UP-STATION)



SIDE VIEW

APPROVED FOR CONSTRUCTION
JMI ENGINEERS, INC.
BY
1/27/03

Notes:

1. Concrete Quantity is for pier only and does not include drilled shafts.

Philip M Hartsfield
1/27/03

Philip M Hartsfield
1/15/04

01/27/2003 04:44:04 PM p19576 - ringling dbloadsdgn\pier11.dgn

BRIDGE NO. 170176

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION
2/20/02	CAB	Revised as noted.			
01 04	JJ	AS BUILTS			

	NAMES	DATES
DRAWN BY	STP	11/14/01
CHECKED BY	CWN	11/14/01
DESIGNED BY	DCG	11/14/01
CHECKED BY	PMH	11/14/01
APPROVED BY	Philip M. Hartsfield, P.E.	

ENGINEER OF RECORD: PARSONS

LOGO: PCL CONSTRUCTION LEADERS

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-789	SARASOTA	17030-3509

SHEET TITLE: PIER 11 DIMENSIONS

PROJECT NAME: STATE ROAD 789 (RINGLING CSWY) OVER INTRACOASTAL WATERWAY

PARSONS

FISCAL YEAR	SHEET NO.
01	1

W.P.I. NO. 2110486

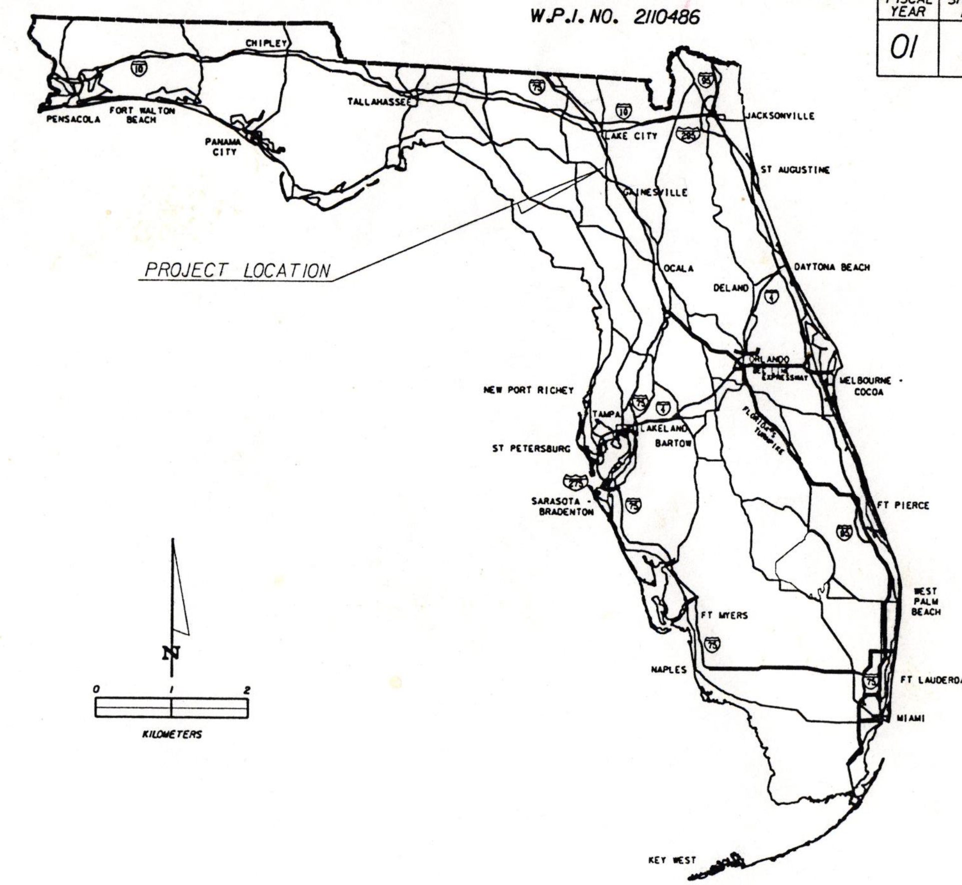
THIS CONTRACT PLAN SET INCLUDES

ROADWAY PLANS
SIGNING AND PAVEMENT MARKING PLANS
LANDSCAPE PLANS
STRUCTURE PLANS

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

CONTRACT PLANS

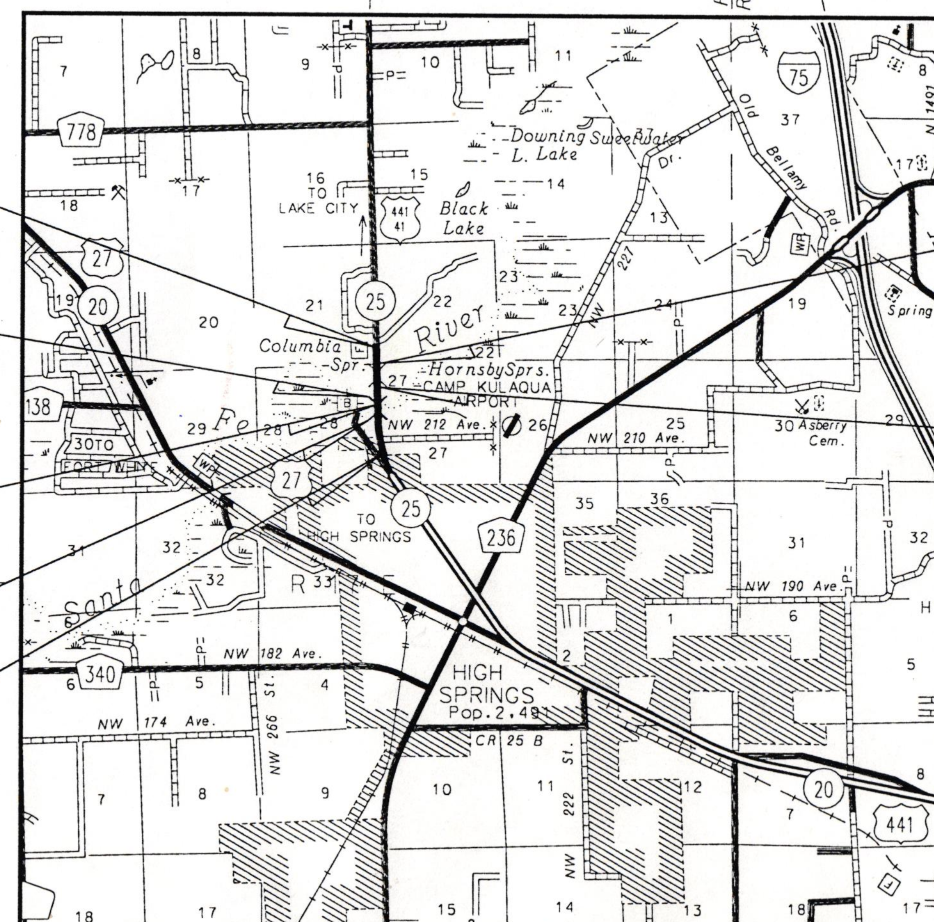
FINANCIAL PROJECT ID 207784-1-52-01
STATE PROJECT NO. 26020-3532
(FEDERAL FUNDS)
ALACHUA COUNTY
STATE ROAD NO. 25



A DETAILED INDEX APPEARS ON THE KEY SHEET OF EACH COMPONENT SET OF PLANS

INDEX OF ROADWAY PLANS

SHEET NO.	SHEET DESCRIPTION
1	KEY SHEET
2-3	SUMMARY OF PAY ITEMS
4	BOX CULVERT DATA SHEET
5	DRAINAGE MAP
6-8	TYPICAL SECTIONS
9	GENERAL AND PAY ITEM NOTES
10-11	SUMMARY OF QUANTITIES
12	SUMMARY OF DRAINAGE STRUCTURES
13	OPTIONAL MATERIALS
14-17	ROADWAY PLAN & PROFILES
18	SPECIAL DRIVEWAY DETAIL
19-20	DRAINAGE STRUCTURE SHEETS
21-22	SOIL DATA
23-32	CROSS SECTIONS
33-48	TRAFFIC CONTROL PLAN SHEETS
49-52	UTILITY ADJUSTMENTS
53-54	ENVIRONMENTAL CONTROL PLAN



Final Plans Documents
Computation Book 1-2
Field Book 8152-751 (Concrete thickness and cover verification)
Field Book 8152-752 (Final Measurements)

GOVERNING STANDARDS AND SPECIFICATIONS:
FLORIDA DEPARTMENT OF TRANSPORTATION,
ROADWAY AND TRAFFIC DESIGN STANDARDS
DATED JANUARY 2000, AND
STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE
CONSTRUCTION DATED 2000,
AS AMENDED BY CONTRACT DOCUMENTS

REVISIONS

FINANCIAL PROJECT ID 207784-1-52-01
Structure Sheet B-8 (Revised 11-09-00)
Roadway Sheets 1, 2 & 34 (Revised 1-10-01)
Landscape Sheets L-1, L-3 & L-4 (Revised 1-10-01)

CONSTRUCTION NOTES
Name of Contractor: **Anderson Columbia CO.,Inc.**
Name of Consultants
involved: **None**
District Secretary: **Aauge Schroder**
Resident Engineer: **Sharon L. Griffiths**
Project Engineer: **Michael Sandow**
Project Manager: **Freddie Wright**

LENGTH OF PROJECT	
	METERS
ROADWAY	758.338
BRIDGES	112.400
NET LENGTH OF PROJ.	870.738
EXCEPTIONS	0.000
GROSS LENGTH OF PROJ.	870.738

FDOT PROJECT MANAGER : FREDDIE WRIGHT

ROADWAY SHOP DRAWINGS TO BE SUBMITTED TO:

FRED WRIGHT
FLORIDA DEPARTMENT OF TRANSPORTATION
1636 LAKE JEFFERY ROAD
LAKE CITY, FLORIDA 32055
(904) 752-3300

PLANS PREPARED BY
LOCHRANE
ENGINEERING, INC.
408 WEST UNIVERSITY AVENUE - SUITE 306
GAINESVILLE, FLORIDA 32601 (352) 338-1846
VENDOR NO. 592036861.002

NOTE: THIS PROJECT WAS DESIGNED TO MEET FDOT R-R-R CRITERIA

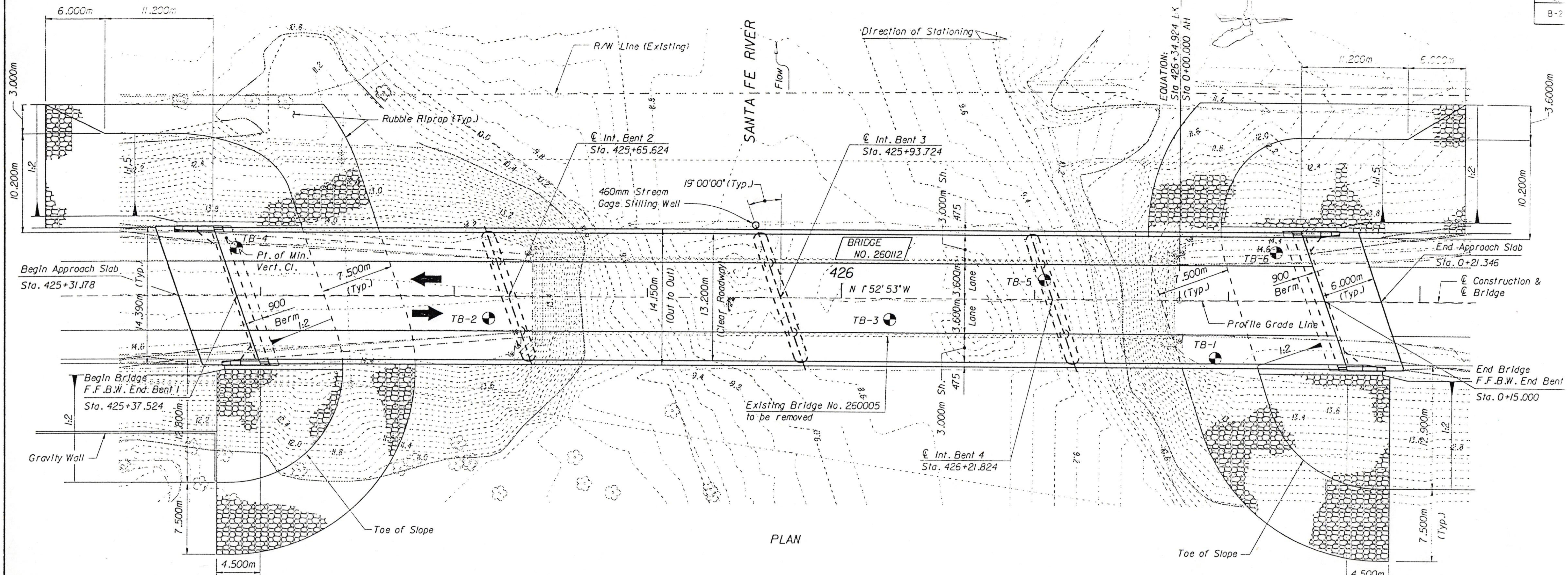
ATTENTION IS DIRECTED TO THE FACT THAT THESE PLANS MAY HAVE BEEN ALTERED IN SIZE BY REPRODUCTION. THIS MUST BE CONSIDERED WHEN OBTAINING SCALED DATA.

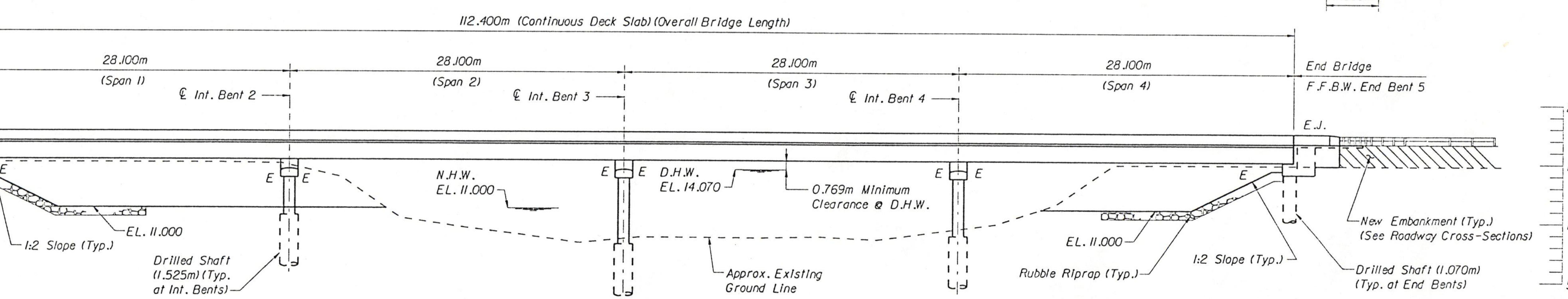
NOTE: THIS IS A METRIC UNIT PROJECT

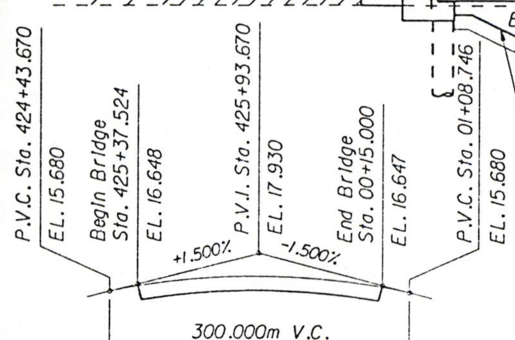
Bridge No. 260112

REVISIONS		
DATE	BY	DESCRIPTION

ROADWAY PLANS
ENGINEER OF RECORD: [signature] WILLIAM P. NAIRN P.E.
DATE: 12-19-01
P.E. NO.: 47078







VERTICAL CURVE DATA
(Along Profile Grade Line)

TRAFFIC DATA

1995 ADT = 4,700
2000 EST. ADT = 5,700
2020 EST. ADT = 9,800
K=11% D=54% T=4% (24 HR)
DESIGN SPEED = 105 KPH

NOTES:

1. Legend
   - E = Expansion Bearing
   - F = Fixed Bearing
   - E.J. = Location of Expansion Joint in Bridge Deck.
   - ⊕ = Boring Locations

W.P.I. No. 2110486

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	KGB	3/99
Checked by	MJM	4/99
Designed by	MJM	5/99
Checked by	KTK	6/99
Approved by	K.T. Kelley	

ENGINEER OF RECORD:
RALPH WHITEHEAD ASSOCIATES, INC.
3733 University Boulevard West
Suite 305
Jacksonville, FL 32217-2103
904.730.9777
Web Site: www.rwhitehead.com

LOGO:

SEAL: 8/18/00

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
25	ALACHUA	207784-1-52-01

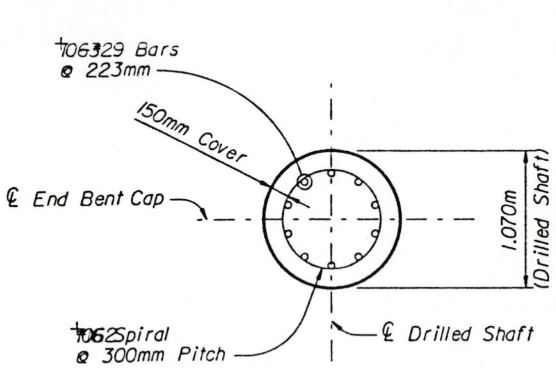
SHEET TITLE: PLAN AND ELEVATION

PROJECT NAME: SR 25 (US 441) OVER SANTA FE RIVER BRIDGE NO. 260112

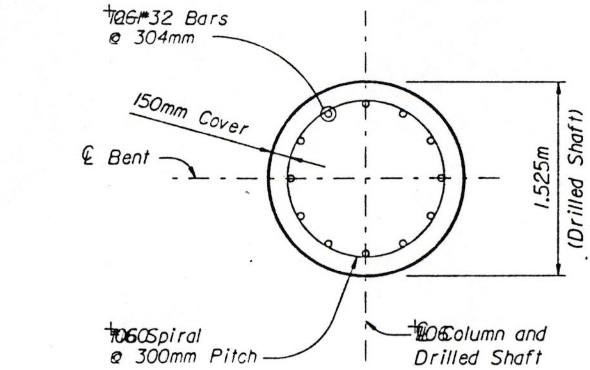
Drawing No.

B-2

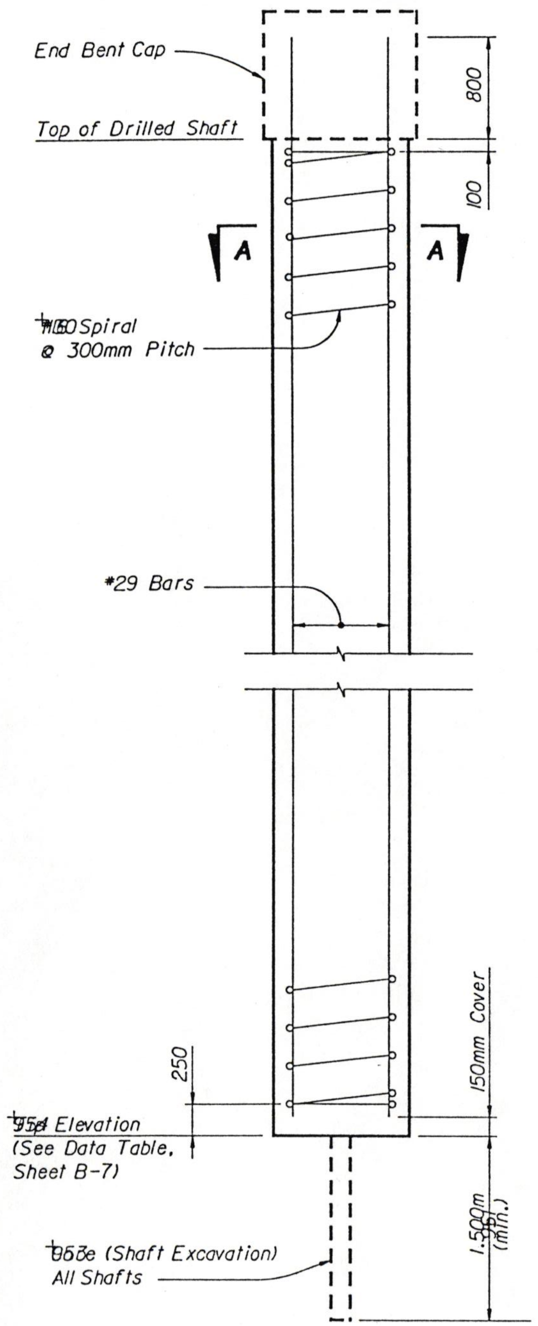
27 OCT 99



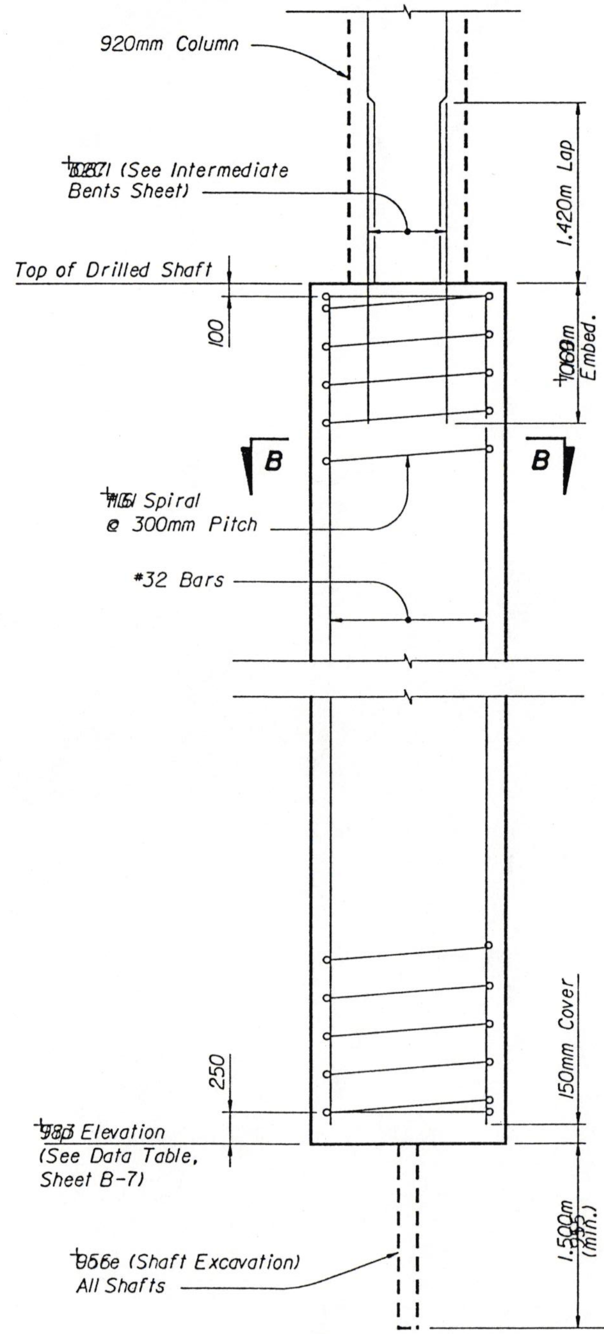
SECTION A-A



SECTION B-B



END BENT
DRILLED SHAFT



INTERMEDIATE BENT
DRILLED SHAFT

B-8

ESTIMATED QUANTITIES

ITEM	UNIT	QUANTITY					
		End Bent 1	Bent 2	Bent 3	Bent 4	End Bent 5	TOTAL
Drilled Shaft (1070mm Dia.) Item No. 2455-88-4	M1	33.9	-	-	-	38.7	72.6
Drilled Shaft (1525mm Dia.) Item No. 2455-88-6	M1	-	25.4	20.8	22.4	-	68.6
Core (Shaft Excavation) Item No. 2455-111	M1	4.5	3.0	3.0	3.0	4.5	18.0
Unclassified Shaft Excavation (1070mm Dia.) Item No. 2455-122-4	M1	33.9	-	-	-	38.7	72.6
Unclassified Shaft Excavation (1525mm Dia.) Item No. 2455-122-6	M1	-	25.4	20.8	22.4	-	68.6

* Extra Depth Excavation and Drilled Shaft Sidewall Overreaming shown as contingency items and are for bidding purposes only.

DRILLED SHAFT INSTALLATION NOTES:

1. The Contractor may elect to use mineral slurry and/or casing to maintain the open hole. If mineral slurry is used, adequate water tanks, slurry tanks and desanding equipment is required.
2. Artesian conditions were not noted by the driller during drilling. However, based on review of the St. Johns River Water Management District potentiometric map of the Upper Floridian Aquifer for the project area, the potential artesian head elevation is estimated to be +10m NGVD. The Contractor shall be prepared to use temporary casing or other methods as necessary to control artesian water levels up to elevation +10m NGVD. The cost of the artesian water level control shall be included in Pay Item 2455-122 - Unclassified Shaft Excavation.
3. The Contractor shall take special precaution to prevent discharge of the slurry or excavated material into the waterway during the excavation operations. Materials excavated from the drilled shafts shall be collected and disposed of satisfactorily in off site areas provided by the Contractor.
4. All shafts shall be inspected prior to placement of concrete using the Department's shaft inspection device (SID) in accordance with Section 455-15.11.3
5. Special precaution shall be taken by the Contractor to prevent discharge of slurry or concrete into the waterway during concreting operations. The Contractor shall provide a receptacle around the top of the casing to collect all slurry and/or concrete discharged from the shaft during concreting operations.
6. The Contractor shall employ turbidity control measures (i.e. containment boxes/collars, casings, culvert pipes, cofferdams, etc.) which satisfy the minimum requirements of the permitting agency(s). Standard floating silt curtains will not be sufficient. The Contractor shall submit turbidity control plans to the engineer for approval prior to starting any work related to drilled shaft excavation. The cost of drilled shaft excavation and turbidity control measures shall be included in Pay Item 2455-122 Unclassified Excavation Shaft.
7. Coring shall be performed in accordance with ASTM D-2113 Standard Practice for Diamond Core Drilling for site investigation, except that the single-tube core barrel will not be allowed for coring and retrieving the undisturbed sample produced during the core (shaft excavation). The core barrel shall meet or exceed the standards of ASTM D-2113 Section 4.3.5 and the Diamond Core Drill Manufacturer's Association (DCDMA) standards for core barrels. Cores are required at each shaft location to a minimum depth of 1.5m from the bottom of the drilled shaft excavation. Payment for coring will be made under Pay Item 2455-111 Core Shaft Excavation.
8. The Contractor shall take steps necessary to prevent damage to drilled shafts that have been constructed previously. The Contractor shall not vibrate any casing within 9.1 meters of the freshly drilled shaft up to 24 hours after the concrete placement.
9. The Contractor will construct a reinforced test shaft for each diameter drilled shaft in the location determined by the Engineer.

NOTES:

1. Concrete for Drilled Shafts shall be Class IV (Drilled Shaft).
2. Reinforcing Steel shall conform to ASTM A615M-96a Grade 420.
3. The Contractor shall perform a 100mm to 150mm diameter core from 1.500m to 6.000m below the shaft excavation as directed by the Engineer. A minimum of 1.500m length core is required for each shaft.
4. If splices are required for vertical reinforcement, alternate locations such that the maximum number of bars spliced at any given section does not exceed 50% of total number of bars.
5. Minimum lap splice for #16 spiral shall be 520mm.

DRILLED SHAFT QUANTITIES PER LINEAR METER

ITEM	UNIT	QUANTITY	
		1070mm	1525mm
Class IV Concrete (Drilled Shaft)	M3	0.90	1.83
Reinforcing Steel	KG	68.0	101.7

NOTE: Quantities shown for Reinforcing Steel do not include lap slices for Vertical Bars or Spiral Bars

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	KMR	6/99
Checked by	KTK	6/99
Designed by	DMW	6/99
Checked by	KTK	6/99
Approved by	K.T. Kelley	

ENGINEER OF RECORD:
RALPH WHITEHEAD ASSOCIATES, INC.
3733 University Boulevard West
Suite 305
Jacksonville, FL 32217-2103
904.730.9777
Web Site: www.rwhitehead.com

LOGO:

SEAL:
K. T. Kelley
8/18/00

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
25	ALACHUA	207784-1-52-01

SHEET TITLE:
DRILLED SHAFT DETAILS

PROJECT NAME:
SR 25 (US 441) OVER SANTA FE RIVER
BRIDGE NO. 260112

Drawing No.

Index No.

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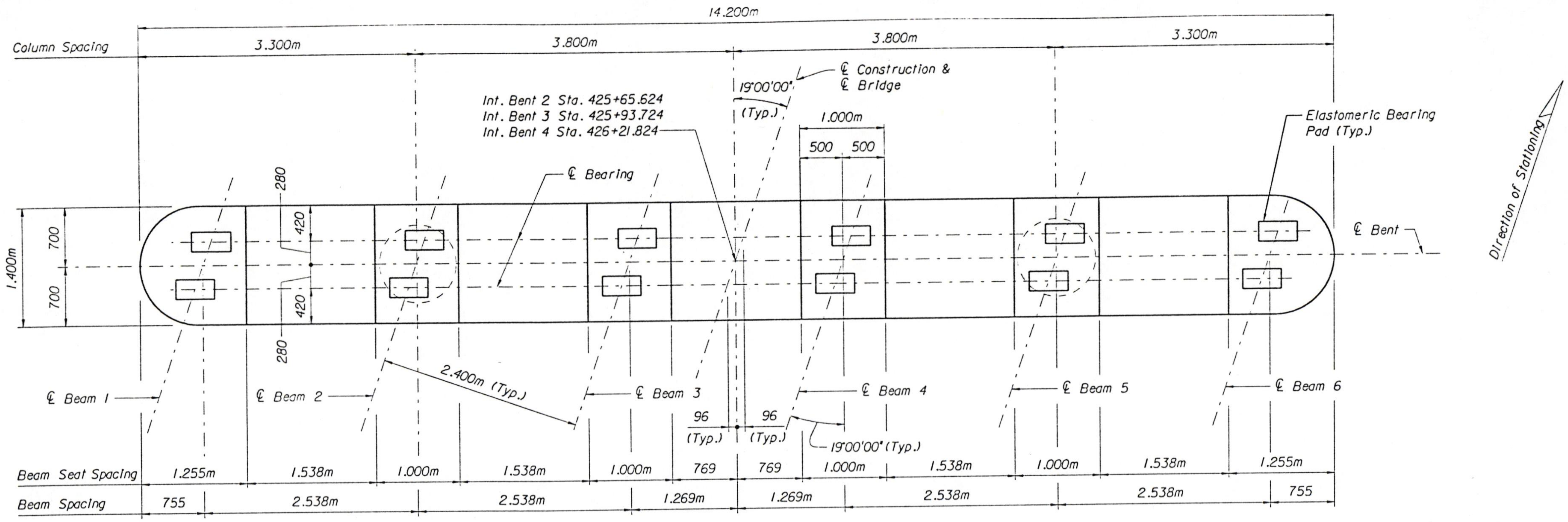
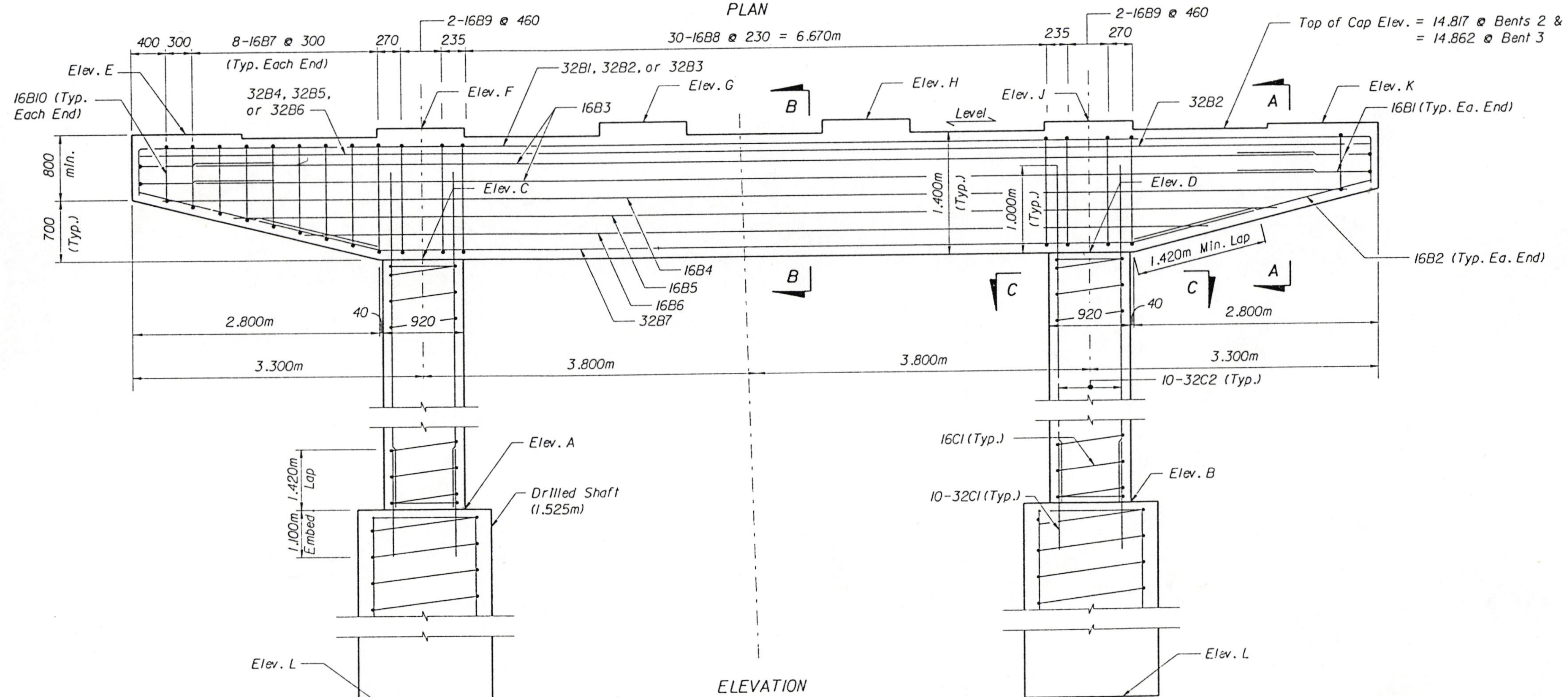
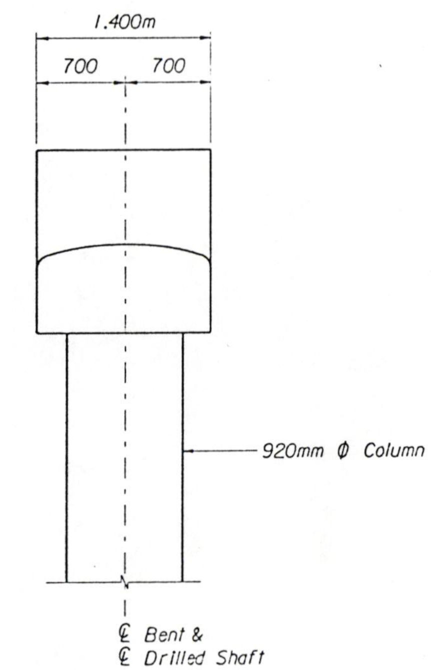


PLAN



ELEVATION
(Int. Bents 2 thru 4)



SECTION A-A

NOTES:

1. All Bearing Surfaces shall be finished level.
2. For Sections and other details, see Sheet B-14.
2. For Drilled Shaft Details, see Sheet B-8.
4. For Reinforcing Bar List, see Sheet B-22.
5. Lap 16B Bars a Minimum of 725mm

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10 NOV 99

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	KGB	5/99
Checked by	MJM	5/99
Designed by	MJM	5/99
Checked by	KGB	5/99
Approved by	K.T. Kelley	

ENGINEER OF RECORD:
RALPH WHITEHEAD ASSOCIATES, INC.
3733 University Boulevard West
Suite 305
Jacksonville, FL 32217-2103
904.730.9777
Web Site: www.rwhitehead.com

LOGO:

SEAL:
8/18/00

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
25	ALACHUA	207784-1-52-01

SHEET TITLE:
INTERMEDIATE BENTS

PROJECT NAME:
SR 25 (US 441) OVER SANTA FE RIVER
BRIDGE NO. 260112

Drawing No.
Index No.

[redacted]

THIS CONTRACT PLAN SET INCLUDES

ROADWAY PLANS
SIGNING AND PAVEMENT MARKING PLANS
STRUCTURE PLANS

A DETAILED INDEX APPEARS ON THE KEY SHEET OF EACH COMPONENT SET OF PLANS

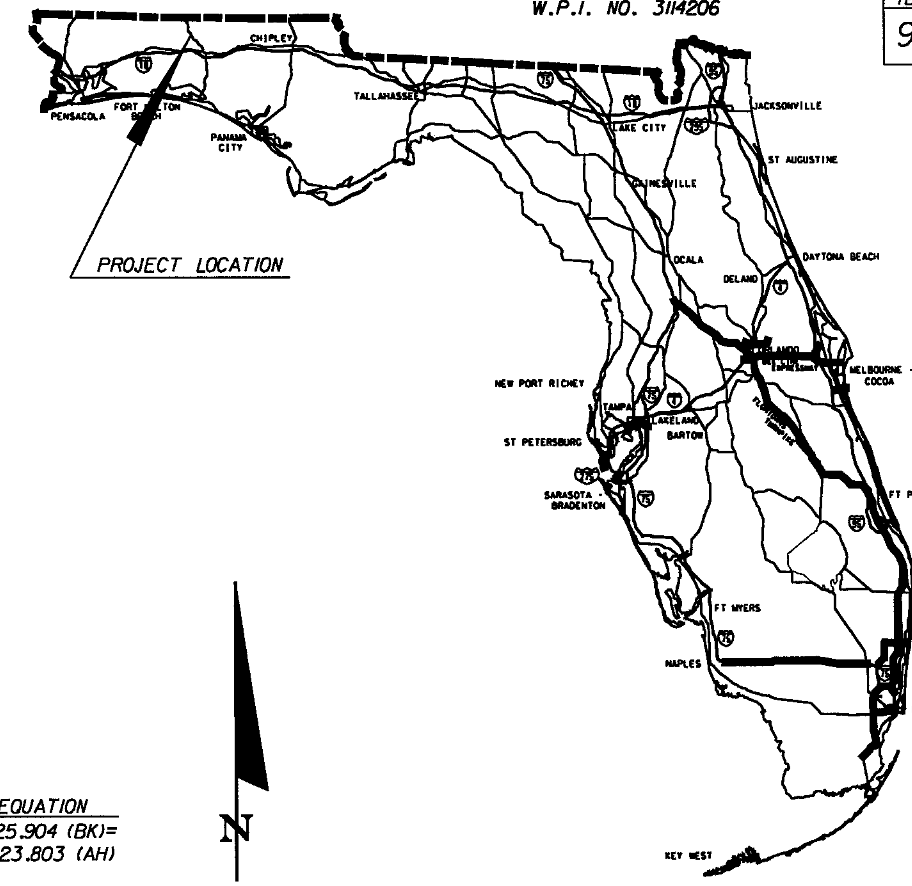
STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

FINAL PLANS
STATE HIGHWAY

FP-ID NO. 219154-1-52-01
STATE PROJECT NO. 52050-3524
HOLMES COUNTY
STATE ROAD NO. 2

W.P.I. NO. 3114206

FISCAL YEAR	SHEET NO.
99	1



INDEX OF ROADWAY PLANS

Roadway Computation Book
Bridge Computation Book
Field Book 01 - Bridge Final Measurements
Field Book 02 - Roadway Final Measurements

BEGIN BRIDGE
STA. 47+75.000

END BRIDGE
STA. 55+55.000

STATION EQUATION
STA. 142+24.648 (BK)=
STA. 42+24.333 (AH)

STATION EQUATION
STA. 65+25.904 (BK)=
STA. 165+23.803 (AH)

BEGIN PROJECT
STA. 141+20.000
kp 22.474 =
MP 13.965

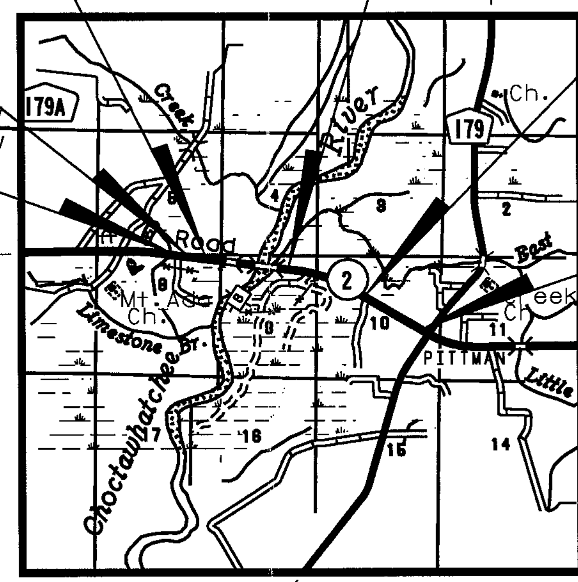
END PROJECT
STA. 175+60.000
kp 25.914 =
MP 16.103

TO GENEVA

TO SR 81

TO GRACEVILLE

TO CARYVILLE



N

0 1 2
KILOMETERS
1:100,000

ROADWAY SHOP DRAWINGS TO BE SUBMITTED TO
JOHN W. (WALLY) JORDAN, JR., P.E.
(850) 385-7450

PLANS PREPARED BY
JMI ENGINEERS
1424 PIEDMONT DRIVE EAST
SUITE 100
TALLAHASSEE, FL 32312
(850) 385-7450
VENDOR NO. VF-V30337209

THIS PROJECT WAS CONSTRUCTED IN SUBSTANTIAL COMPLIANCE WITH THESE PLANS AS PROVIDED BY THE ENGINEER OF RECORD. IF CHANGES WERE MADE, THOSE CHANGES ARE INDICATED BY BLACK INK REVISION AND BEAR THE SEAL AND SIGNATURE OF THE RESPONSIBLE ENGINEER. ______________

THESE PLANS HAVE BEEN PREPARED IN ACCORDANCE WITH AND ARE GOVERNED BY THE STATE OF FLORIDA, DEPARTMENT OF TRANSPORTATION, ROADWAY AND TRAFFIC DESIGN STANDARDS (BOOKLET DATED JANUARY, 1998).

Contractor	F & W Construction Company, Inc.
Consultants	Metric Engineering, Inc.
District Secretary	Edward Prescott
Resident Engineer	Steve Potter
Project Manager	Keith Hinson
Project Engineer	Herman Green

ATTENTION IS DIRECTED TO THE FACT THAT THESE PLANS MAY HAVE BEEN ALTERED IN SIZE BY REPRODUCTION. THIS MUST BE CONSIDERED WHEN OBTAINING SCALED DATA.

GOVERNING SPECIFICATIONS: STATE OF FLORIDA, DEPARTMENT OF TRANSPORTATION, STANDARD SPECIFICATIONS, DATED 1999, SUPPLEMENTS AND SPECIAL PROVISIONS THERETO IF NOTED IN THE CONTRACT SPECIFICATIONS FOR THIS PROJECT.

REVISIONS

Structure Sheet B-1 (Revised 4-29-99)
Roadway Sheets 2 & 5 (Revised 6-1-99)
Structure Sheets B-26 & B-29 (Revised 1-26-00)
Structure Sheet B-35 (Revised 4-12-00)

Julian B. McLean
11/14/01

LENGTH OF PROJECT IS BASED ON ℄ CONSTRUCTION

LENGTH OF PROJECT	
	METERS
ROADWAY	2 662.416
BRIDGES	780.00
NET LENGTH OF PROJ.	3 442.416
EXCEPTIONS	-
GROSS LENGTH OF PROJ.	3 442.416

NOTE: THIS IS A METRIC UNIT PROJECT

REVISIONS

DATE	BY	DESCRIPTION

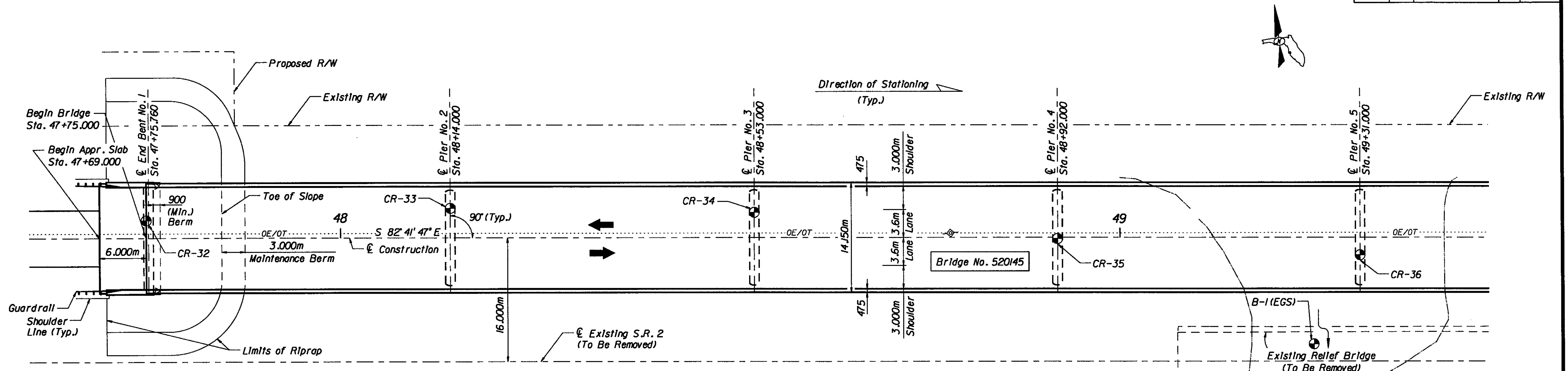
ROADWAY PLANS ENGINEER OF RECORD: JOHN W. (WALLY) JORDAN, JR., P.E.
DATE: 4/12/00
P.E. NO.: 50894

FDOT PROJECT MANAGER : BILL PAXTON, P.E.

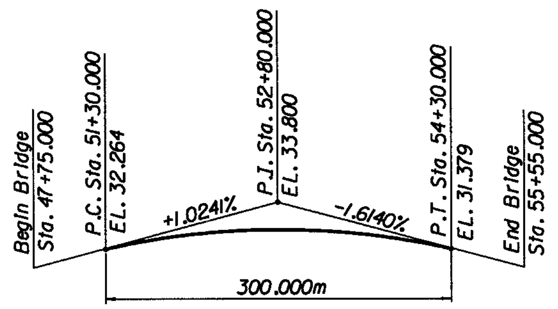
S.R. 2 CHOCTAWHATCHEE RIVER BRIDGE REPLACEMENT

JMI Engineers - Tallahassee - 12 APR 100 12:11:22 a:\952\drawing\roadway\working\keysrd01.dgn

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	52050-3524		B-8



PLAN-UNIT I



VERTICAL CURVE DATA
(Along ℄ Construction)

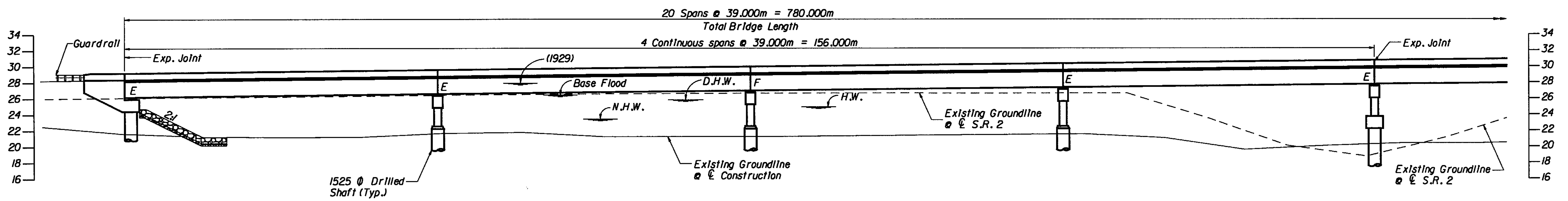
TRAFFIC DATA

1993 ADDT = 900
1997 EST. ADT = 1050
2007 EST. ADT = 1450
2017 EST. ADT = 1850
K30 = 14%
D30 = 60%
T = 17% (24 HR.)
T = 9% (DESIGN HR.) DT
T = 7% (DESIGN HR. HEAVY) DHT
T = 2% (DESIGN HR. MEDIUM) DMT

DESIGN SPEED 110 km/h

WATER ELEVATIONS

N.H.W. = 23.700m
H.W. = 25.210m
D.H.W. (50yr.) = 26.100m
Base Flood = 26.700m
(1929) = 28.190m



ELEVATION-UNIT I

Note:
For Adjustment and Relocation of Utilities, see Roadway Plans.

Joe W. Jordan
2/12/99

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	TGA	4-98
Checked by	EP	4-98
Designed by	EP	4-98
Checked by	JWJ	4-98
Approved by	J. JORDAN, P.E.	

ENGINEER OF RECORD:
JMI ENGINEERS, INC.
1424 Piedmont Drive East
Tallahassee, Florida 32312
Tel: 850-385-7450 Fax: 850-385-3545

LOGO:
JMI ENGINEERS

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 2	HOLMES	52050-3524

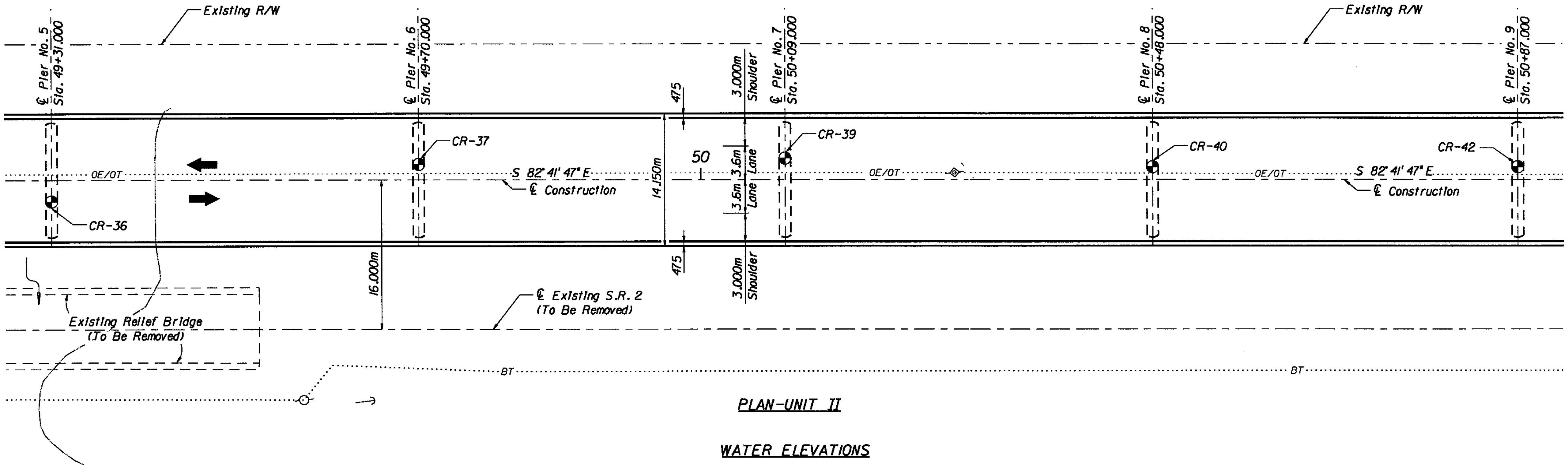
SHEET TITLE: PLAN AND ELEVATION I

PROJECT NAME: S.R. 2 BRIDGE OVER CHOCTAWHATCHEE RIVER

Drawing No.

Index No.

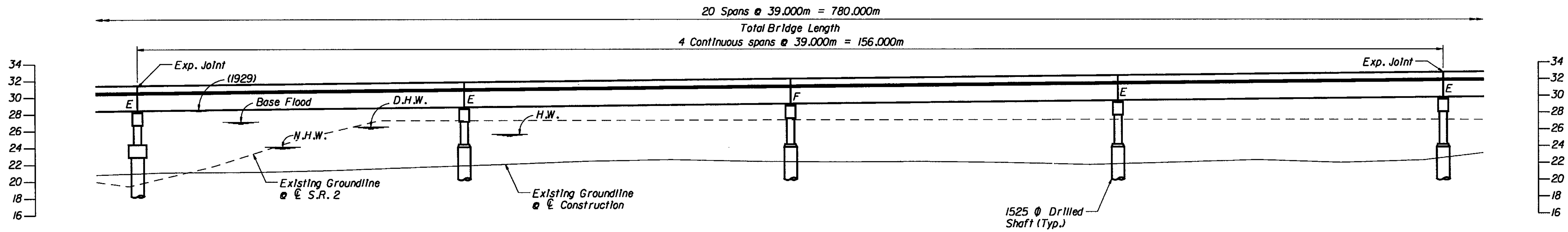
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	52050-3524		B-9



PLAN-UNIT II

WATER ELEVATIONS

N.H.W. = 23.700m
H.W. = 25.210m
D.H.W. (50yr.) = 26.100m
Base Flood = 26.700m
(1929) = 28.190m



ELEVATION-UNIT II

Note :
For Adjustment and Relocation of Utilities, see Roadway Plans.

2/12/99

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	TGA	4-98
Checked by	EP	4-98
Designed by	EP	4-98
Checked by	JWJ	4-98
Approved by	J. JORDAN, P.E.	

ENGINEER OF RECORD:
JMI ENGINEERS, INC.
1424 Piedmont Drive East
Tallahassee, Florida 32312
Tel: 850-385-7450 Fax: 850-385-3545

LOGO: JMI ENGINEERS

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 2	HOLMES	52050-3524

SHEET TITLE: PLAN AND ELEVATION II

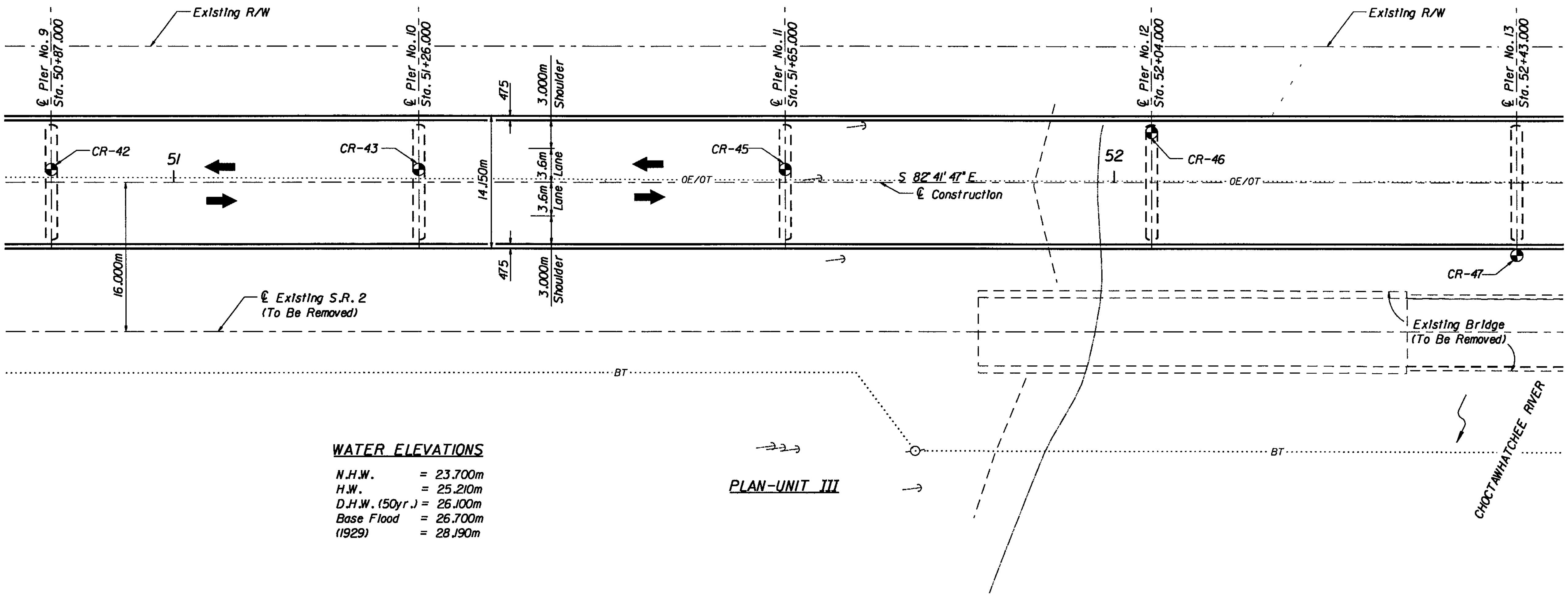
PROJECT NAME: S.R. 2 BRIDGE OVER CHOCTAWHATCHEE RIVER

Drawing No.

Index No.

JMI Engineers - Tallahassee - 10 FEB 99 09:01:48 o:\952\drawing\bridge\working\gnpe.dgn

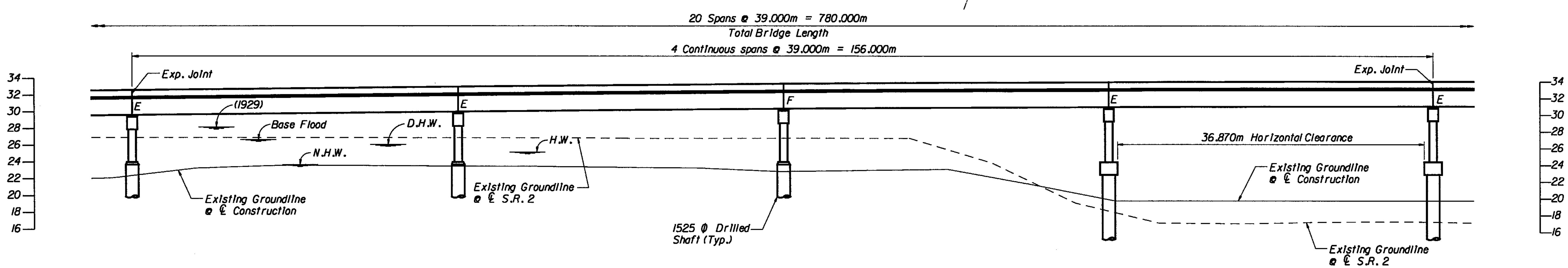
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	52050-3524		B-10



WATER ELEVATIONS

N.H.W. = 23.700m
H.W. = 25.210m
D.H.W. (50yr.) = 26.100m
Base Flood = 26.700m
(1929) = 28.190m

PLAN-UNIT III



J. W. Jordan
2/12/99

ELEVATION-UNIT III

Note :

For Adjustment and Relocation of Utilities, see Roadway Plans.

$FILE$ $TIME$ $DATE$ - JMI Engineers - Tallahassee

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	TGA	4-98
Checked by	EP	4-98
Designed by	EP	4-98
Checked by	JWJ	4-98
Approved by	J. JORDAN, P.E.	

ENGINEER OF RECORD:
JMI ENGINEERS, INC.
1424 Piedmont Drive East
Tallahassee, Florida 32312
Tel: 850-385-7450 Fax: 850-385-3545

LOGO: JMI ENGINEERS

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 2	HOLMES	52050-3524

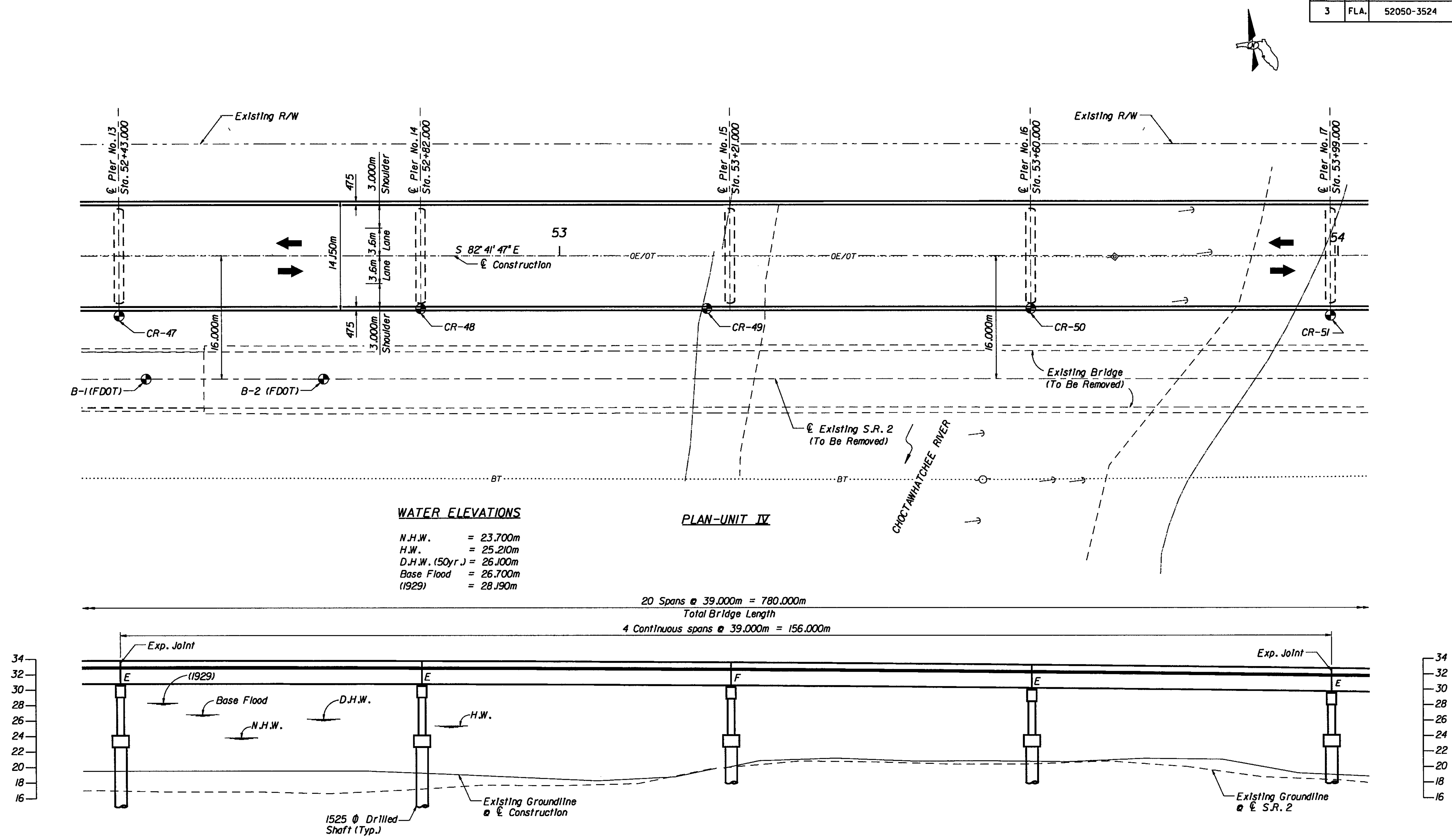
SHEET TITLE: PLAN AND ELEVATION III

PROJECT NAME: S.R. 2 BRIDGE OVER CHOCTAWHATCHEE RIVER

Drawing No.

Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	52050-3524		B-11



PLAN-UNIT IV

ELEVATION-UNIT IV

Note :

For Adjustment and Relocation of Utilities, see Roadway Plans.

2/12/99

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	TGA	4-98
Checked by	EP	4-98
Designed by	EP	4-98
Checked by	JWJ	4-98
Approved by	J. JORDAN, P.E.	

ENGINEER OF RECORD:
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LOGO: JMI ENGINEERS

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 2	HOLMES	52050-3524

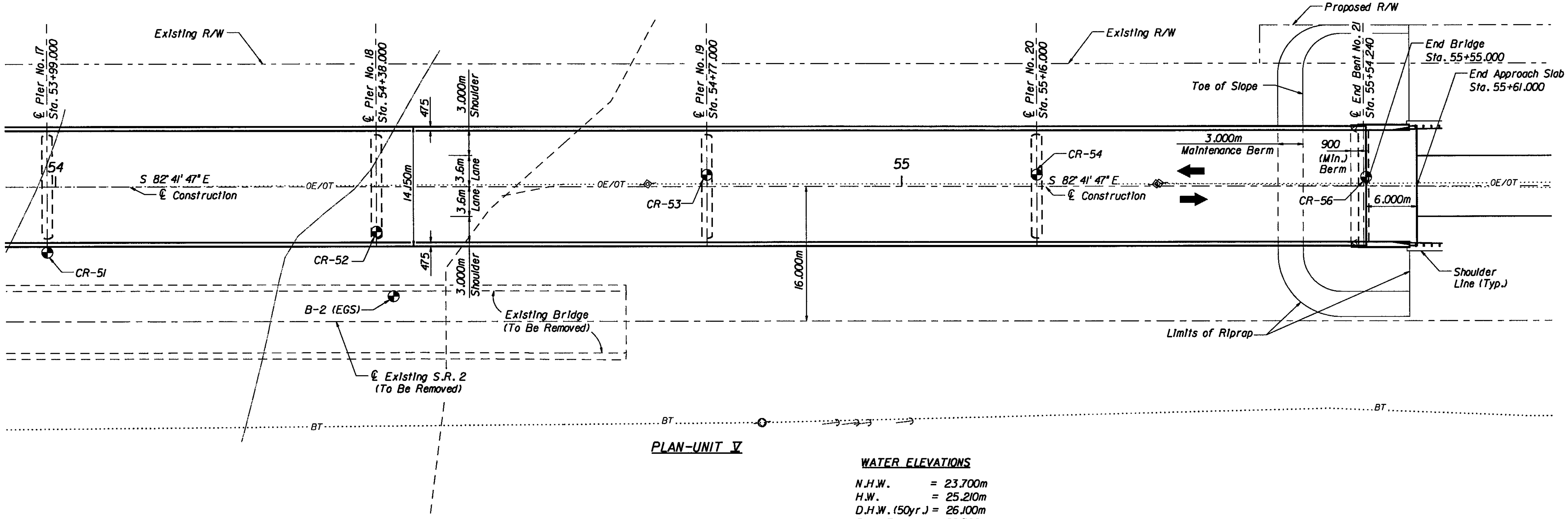
SHEET TITLE: PLAN AND ELEVATION IV

PROJECT NAME: S.R. 2 BRIDGE OVER CHOCTAWHATCHEE RIVER

Drawing No.

Index No.

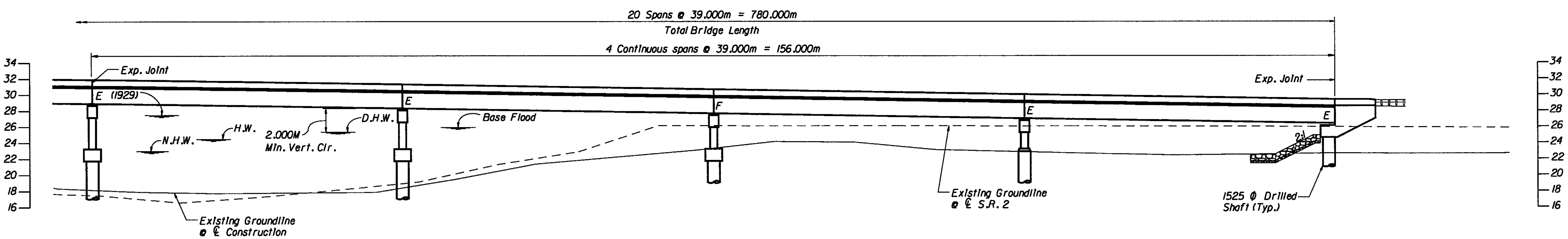
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	52050-3524		B-12



PLAN-UNIT V

WATER ELEVATIONS

N.H.W.	= 23.700m
H.W.	= 25.210m
D.H.W. (50yr.)	= 26.100m
Base Flood	= 26.700m
(1929)	= 28.190m



ELEVATION-UNIT V

Note :

For Adjustment and Relocation of Utilities, see Roadway Plans.

2/12/99

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	TGA	4-98
Checked by	EP	4-98
Designed by	EP	4-98
Checked by	JWJ	4-98
Approved by	J. JORDAN, P.E.	

ENGINEER OF RECORD:
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LOGO: JMI ENGINEERS

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STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 2	HOLMES	52050-3524

SHEET TITLE: PLAN AND ELEVATION V

PROJECT NAME: S.R. 2 BRIDGE OVER CHOCTAWHATCHEE RIVER

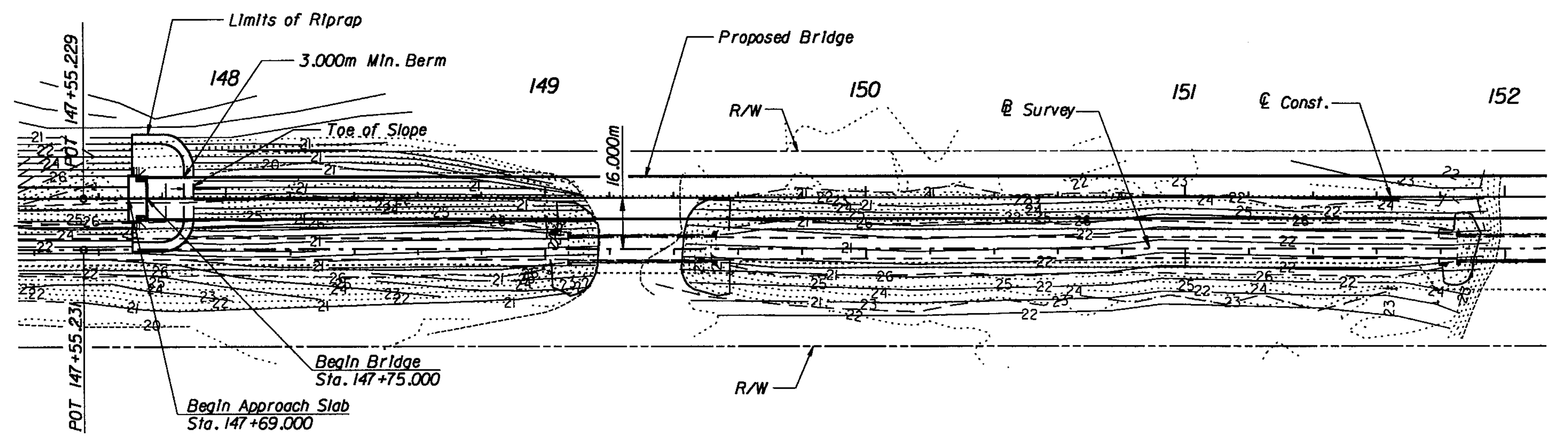
Drawing No.

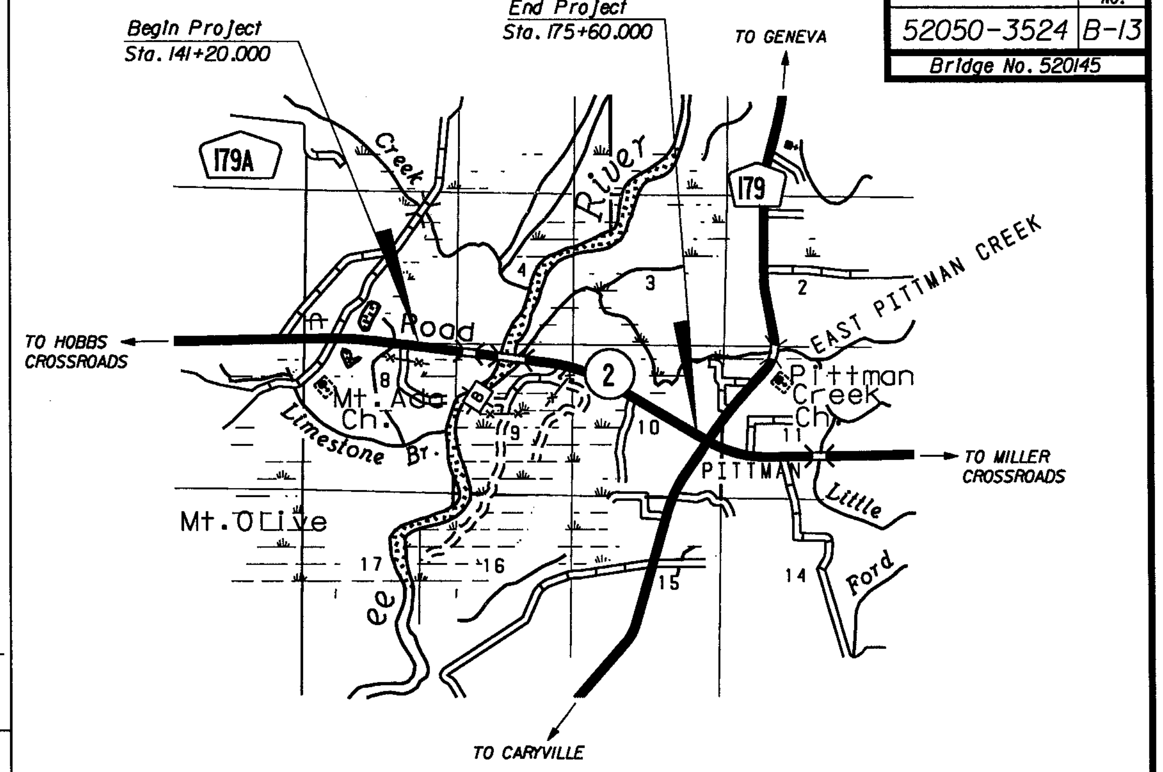
Index No.

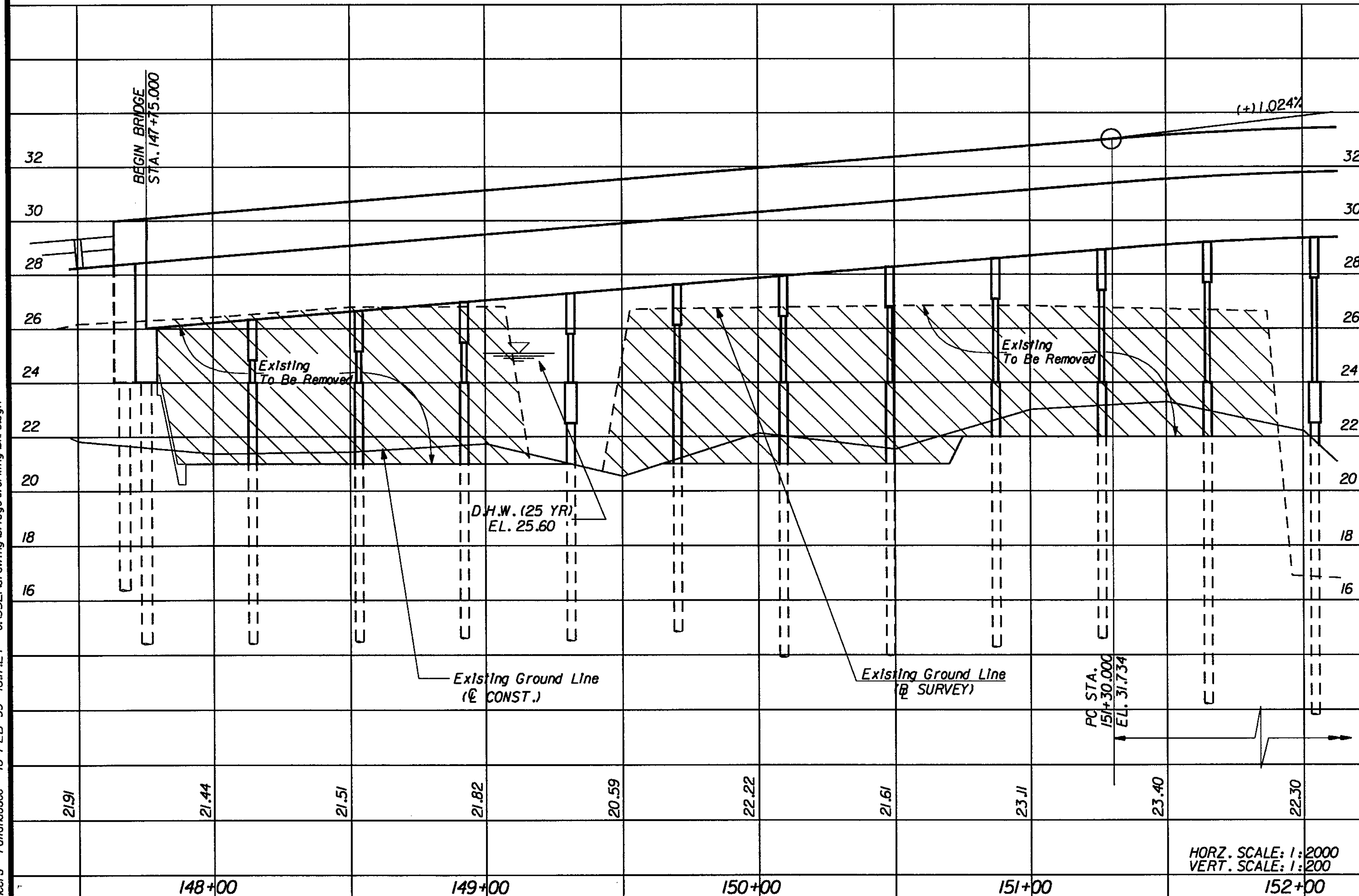
JMI Engineers - Tallahassee - 12 FEB 99 16:58:44 o:\952\drawing\bridge\working\gnpe.dgn

STATE PROJ. NO.	SHEET NO.
52050-3524	B-13

Bridge No. 520145







EXISTING STRUCTURES (REFERENCE)	(1) & (2)	(3) & (4)	(5)	(6)	ASSUMED CONFIGURATION
FOUNDATION	Pile Bent	Pile Bent			2-1520 Dia. Pile Bent
OVERALL LENGTH	45.72	281.94			780.00
SPAN LENGTH	15.24	15.24 & 22.86			39.00
TYPE CONSTRUCTION	I Beam	I Beam			Conc. Beams
AREA OF OPENING @ H.W.					
ROADWAY WIDTH	7.31	7.31			14.15
ELEV. LOW MEMBER	25.30	25.60			Varies

HYDRAULIC DESIGN DATA

NOTE: The hydraulic data is shown for informational purposes only to indicate the flood discharges and water surface elevations which may be anticipated in any given year. This data was generated using highly variable factors determined by a study of the watershed. Many judgements and assumptions are required to establish these factors. The resultant hydraulic data is sensitive to changes, particularly antecedent conditions, urbanization, channelization and land use. Users of this data are cautioned against the assumption of precision which cannot be obtained.

DEFINITIONS:
Design Flood: The flood utilized to assure a desired level of hydraulic performance.
Base Flood: The flood having a 1% chance of being exceeded in any year. (100 Year Frequency)
Overtopping Flood: The flood which causes flow over the highway, over a watershed divide or thru emergency relief structures.
Greatest Flood: The most severe flood which can be predicted where overtopping is not practicable.

WATER SURFACE ELEVATIONS: N.H.W. (Non-Tidal) 23.70 M.H.W. ______ M.L.W. ______

FLOOD DATA:	MAX. EVENT OF RECORD (1929)	DESIGN FLOOD (1)		BASE FLOOD	☒ OVERTOPPING FLOOD ☐ GREATEST FLOOD
(2) STAGE ELEV. NGVD (M)	28.19	25.60	26.10	26.70	25.70
DISCHARGE (CM/S)	5542	2580	3310	4230	2400
AVERAGE VELOCITY (M/S)	0.37	0.73	0.82	0.71	0.71
EXCEEDANCE PROB. (%)	0.0045	0.04	0.02	0.01	0.044
FREQUENCY (YR.)	220	25	50	100	22.5

HYDRAULIC RECOMMENDATIONS

1. BEGIN BRIDGE STATION 147+75 END BRIDGE STATION 155+55 SKEW ANGLE 0
2. CHANNEL SECTION: ℄ STATION - BOTTOM WIDTH - ELEV. Varies (3) SIDE SLOPE -
3. LIMITS OF CHANNEL EXCAVATION: RT. Remove Existing Causeway (3) LT. Remove Existing Causeway (3)
4. CLEARANCE: NAVIGATION: HORIZ. (1) N/A VERT. N/A ABOVE EL. N/A DRIFT: HORIZ. 22.8 VERT. 2.0 ABOVE EL. 26.1
5. SCOUR PREDICTION: Maximum Scour Depth occurs prior to road overtopping. Use Alt. 2x 50 Year Main Channel Scour Elevation of 7.4 for both Foundation Design and Check Elevations for all Bents.
6. SLOPE PROTECTION: STD FDOT Rubble Riprap 1V : 2H w 3m horizontal extension wmin = 27kg, w50 = 136kg, wmax = 318kg; sg = 2.3
7. DECK DRAINAGE: 0.1 m Scuppers at 3 m cc except eliminate Scuppers on Center Main Channel and both Abutment Spans
8. OTHER:

Patrick W. [illegible]
2-12-99

REMARKS: (1) There are no navigation requirements not satisfied by Drift. Minimum Drift Clearance In overbank area 0.6m above 25.6 (25 yr); In River Zone 0.6m over 26.1 (50 yr)
(2) Flood stage varies across floodplain. Stage shown is Main Channel Not, considering local drawdown except 'overtopping' where the stage is the flood stage at the overtopping location.
(3) Remove Existing Causeway to: El 21.0 147+80 - 150+70
EL 22.0 150+75 - 152+00, 154+75 - 155+50

REVISIONS											
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

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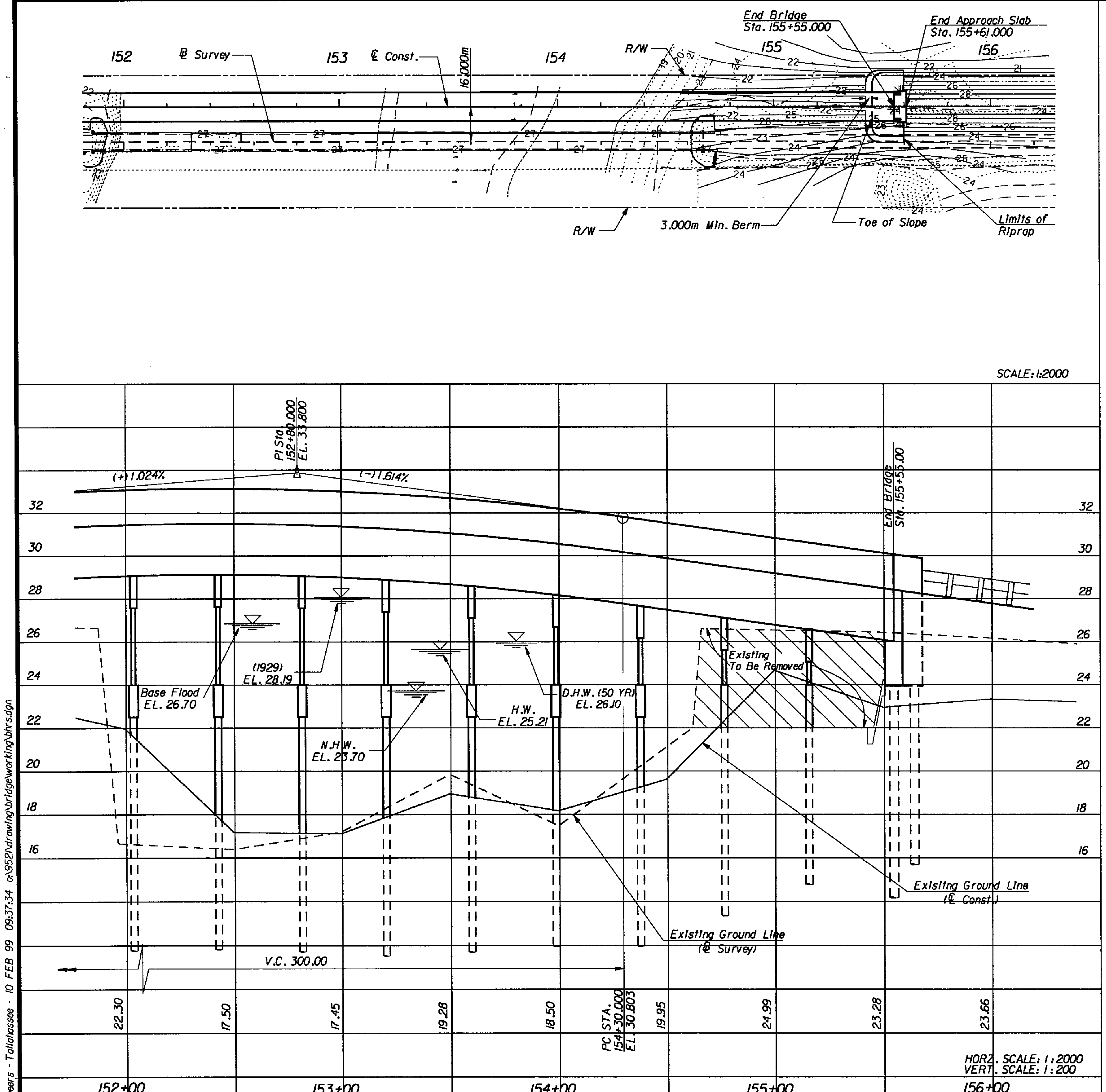
FLORIDA DEPARTMENT OF TRANSPORTATION

BRIDGE HYDRAULIC RECOMMENDATIONS I
S.R. 2 BRIDGE OVER CHOCTAWHATCHEE RIVER

JMI Engineers - Tallahassee - 10 FEB 99 18:17:24 o:\9521\drawing\bridge\working\bhrs.dgn

STATE PROJ. NO.	SHEET NO.
52050-3524	B-14

Bridge No. 520145



Patrick W. Price
2-12-99

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

JMI ENGINEERS, INC.
1424 Piedmont Drive East
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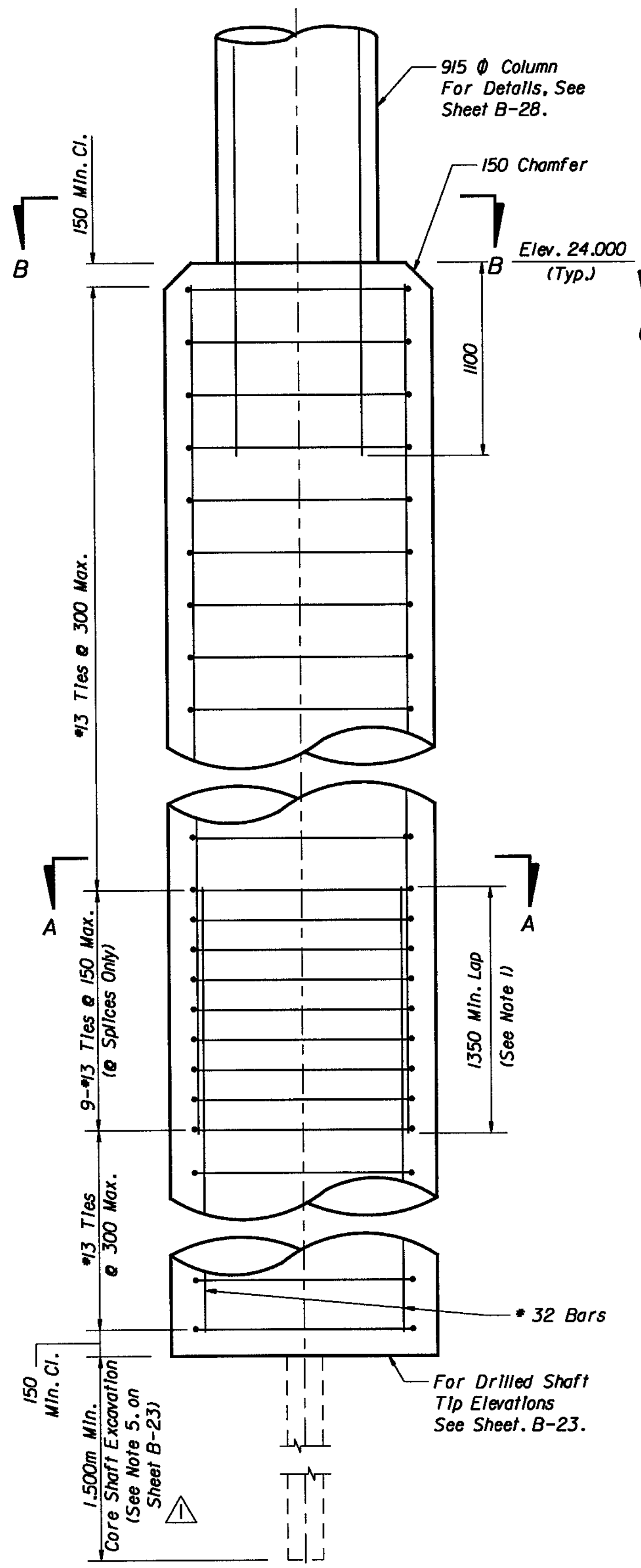
FLORIDA DEPARTMENT OF TRANSPORTATION

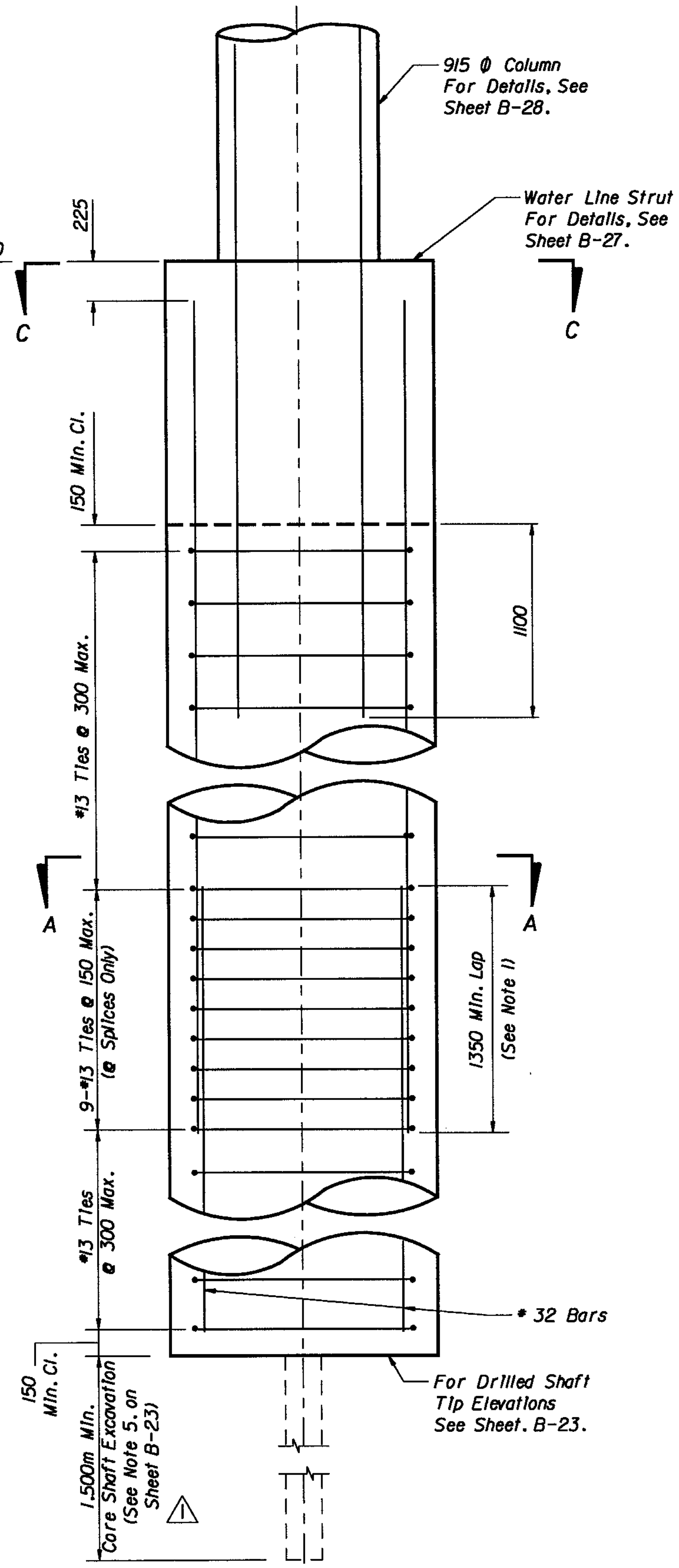
BRIDGE HYDRAULIC RECOMMENDATIONS II
S.R. 2 BRIDGE OVER CHOCTAWHATCHEE RIVER

JMI Engineers - Tallahassee - 10 FEB 99 09:37:34 o:\952\drawing\bridge\working\bhrs.dgn



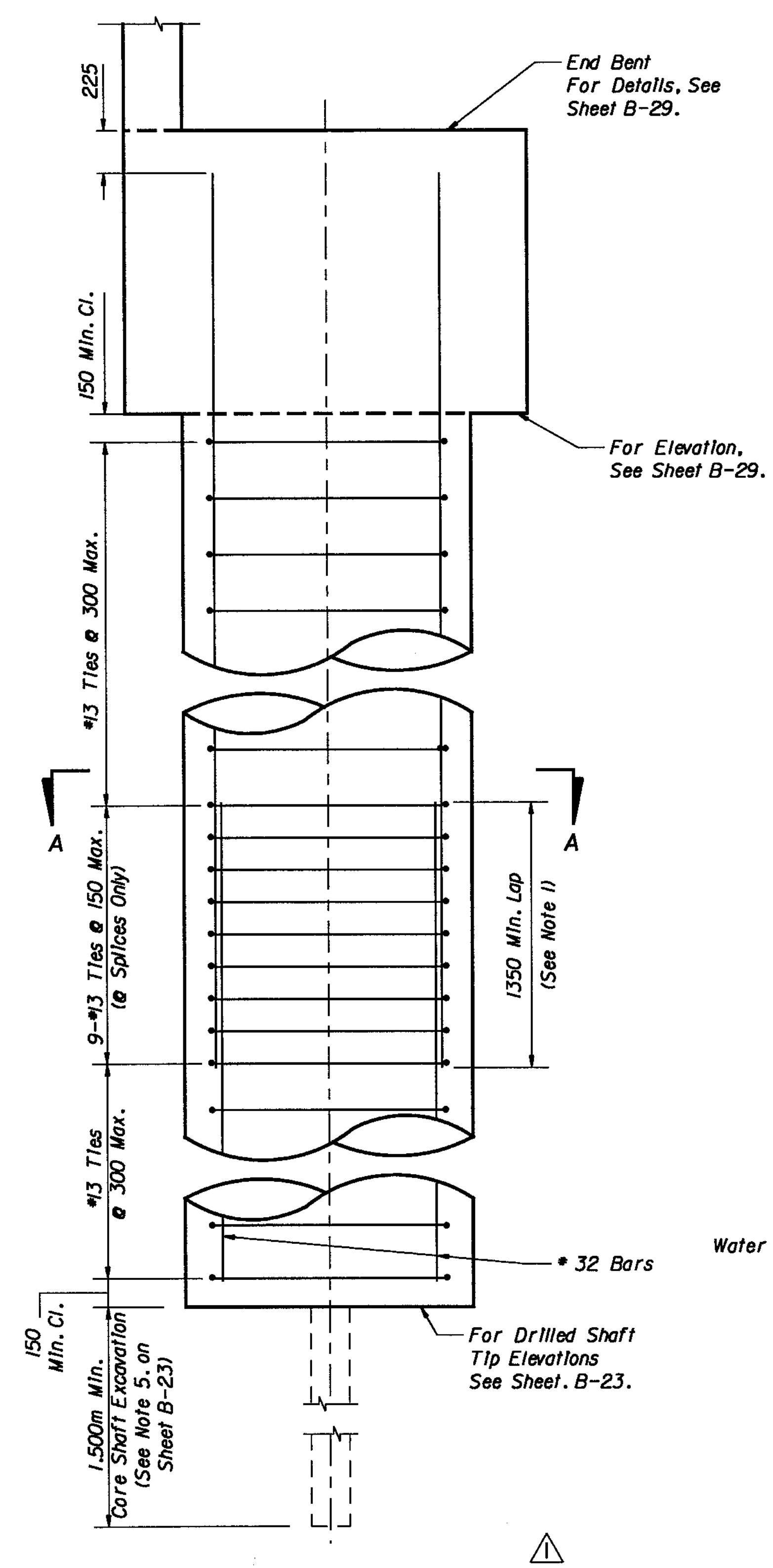
DRILLED SHAFT DETAIL WITH COLUMN

Piers 2-4, 6-11, & 20



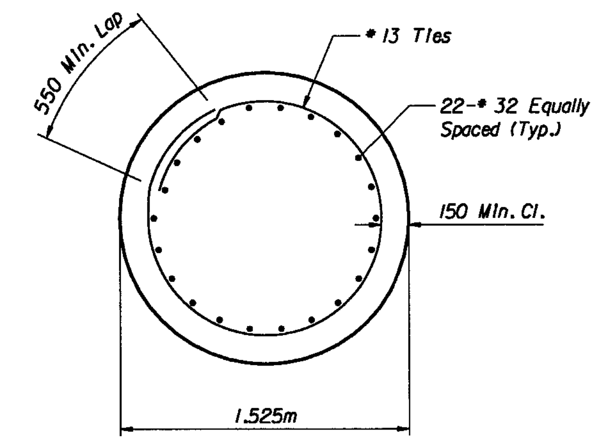
DRILLED SHAFT DETAIL WITH STRUT & COLUMN

Piers 5 & 12-19

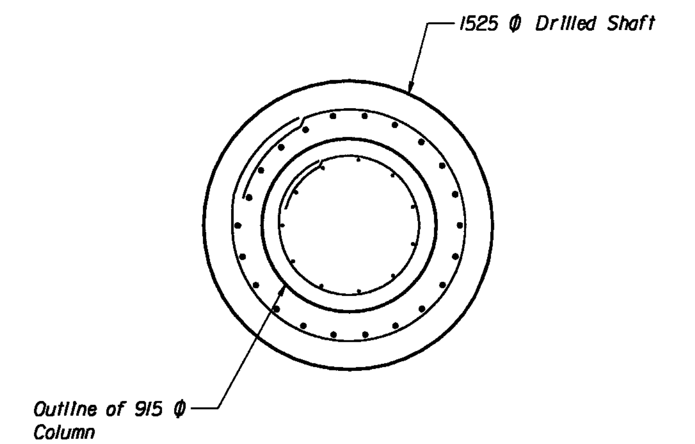


DRILLED SHAFT DETAIL @ END BENT

End Bents 1 & 21



SECTION A-A



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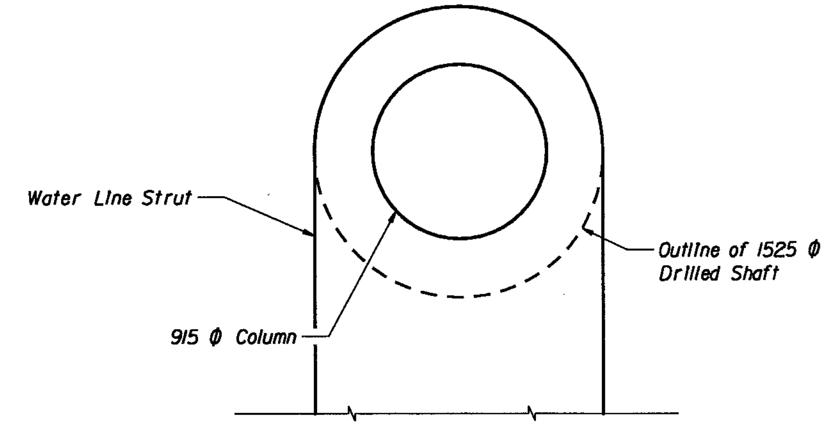

SECTION C-C

Note :

1. Alternate Lap Splice so that a maximum of 50% of the reinforcement is spliced at the same location.

REVISIONS

Date	By	Description	Date	By	Description
1/26/00	SGS	Added Detail @ End Bent			

	Names	Dates
Drawn by	TGA	7-98
Checked by	EP	7-98
Designed by	SL	7-98
Checked by	JWJ	7-98
Approved by	J. JORDAN, P.E.	

ENGINEER OF RECORD:
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Tel: 850-385-7450 Fax: 850-385-3545

LOGO:

JMI ENGINEERS

SEAL:

1/26/00

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 2	HOLMES	52050-3524

SHEET TITLE: DRILLED SHAFT DETAILS

PROJECT NAME: S.R. 2 BRIDGE OVER CHOCTAWHATCHEE RIVER

Drawing No.

Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	52050-3524		B-23

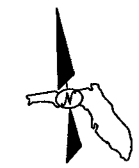
DRILLED SHAFT INSTALLATION TABLE

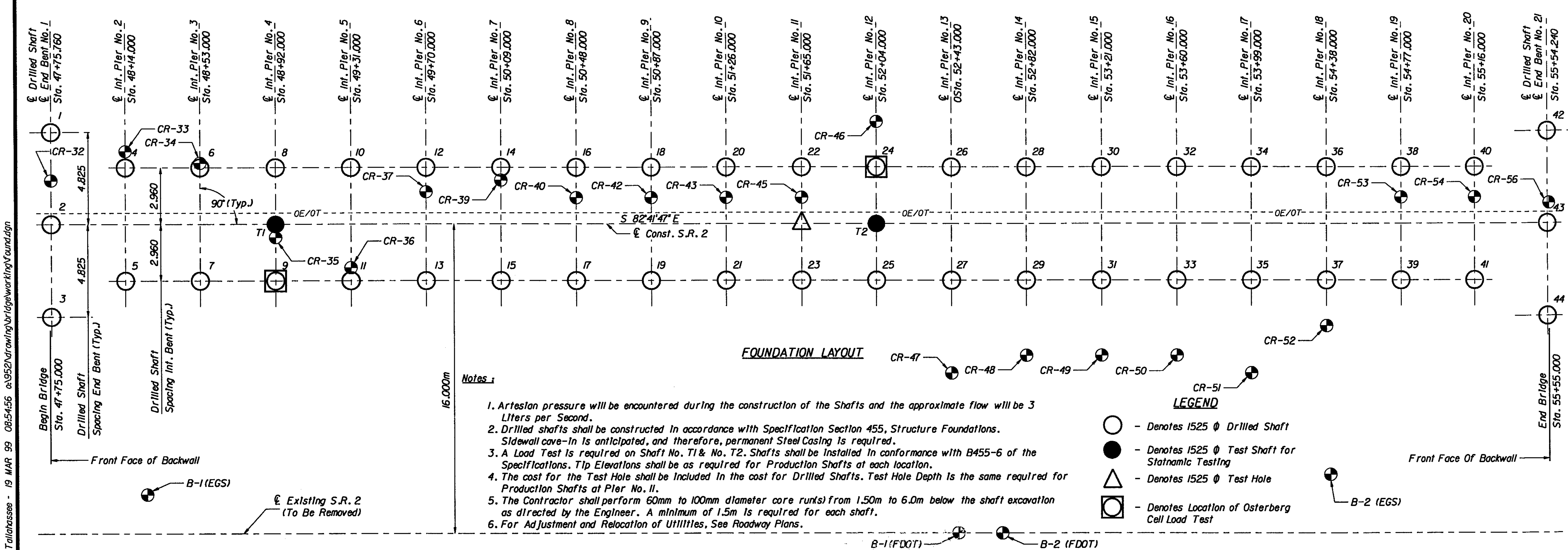
	INSTALLATION CRITERIA				DESIGN CRITERIA		
BENT or PIER NUMBER	RECOMMENDED SHAFT SIZE (METERS)	TIP ELEVATION * (METERS NGVD)	MINIMUM TIP ELEVATION ** (METERS NGVD)	MINIMUM SOCKET LENGTH (METERS)	FACTORED DESIGN LOAD (kN)	DOWN DRAG (kN)	100 YEAR SCOUR ELEVATION (METERS NGVD)
BENT NO. 1	1.525	4.300	4.300	12.400	3,600	N/A	16.700
PIER NO. 2	1.525	-0.300	-0.100	9.700	7,400	N/A	9.600
PIER NO. 3	1.525	-1.900	-1.900	11.500	7,400	N/A	9.600
PIER NO. 4	1.525	-3.400	-3.100	12.700	7,400	N/A	9.600
PIER NO. 5	1.525	0.700	1.000	8.600	7,400	N/A	9.600
PIER NO. 6	1.525	-1.300	-1.000	10.600	7,400	N/A	9.600
PIER NO. 7	1.525	-0.200	0.200	9.400	7,400	N/A	9.600
PIER NO. 8	1.525	-1.600	-1.500	11.100	7,400	N/A	9.600
PIER NO. 9	1.525	0.200	0.200	9.400	7,400	N/A	9.600
PIER NO.10	1.525	-0.900	-0.900	10.500	7,400	N/A	9.600
PIER NO.11	1.525	-1.900	-1.900	11.500	7,400	N/A	9.600
PIER NO.12	1.525	-2.200	-2.000	8.700	7,400	N/A	9.600
PIER NO.13	1.525	-1.300	-1.300	10.900	7,400	N/A	9.600
PIER NO.14	1.525	-0.600	-0.500	10.100	7,400	N/A	9.600
PIER NO.15	1.525	-0.800	-0.600	10.200	7,400	N/A	9.600
PIER NO.16	1.525	-1.300	-1.300	10.900	7,400	N/A	9.600
PIER NO.17	1.525	0.600	0.600	9.000	7,400	N/A	9.600
PIER NO.18	1.525	-0.900	-0.900	10.500	7,400	N/A	9.600
PIER NO.19	1.525	0.200	0.200	9.400	7,400	N/A	9.600
PIER NO.20	1.525	-1.500	-1.500	11.100	7,400	N/A	9.600
BENT NO.21	1.525	4.100	4.100	11.400	3,600	N/A	15.500

Notes for Drilled Shafts :

Tip Elevation * – the elevation to which the shaft shall be constructed unless test load data, rock cores or other geotechnical data obtained during construction allows the Engineer to authorize a different tip elevation. (Based on Axial Load Analysis)

Min. Tip Elevation ** – the highest elevation that the shaft tip may be constructed if adjustments are made to the tip elevation specified. (Based on Lateral Load Analysis)





FOUNDATION LAYOUT

Notes :

1. Artesian pressure will be encountered during the construction of the Shafts and the approximate flow will be 3 Liters per Second.
2. Drilled shafts shall be constructed in accordance with Specification Section 455, Structure Foundations. Sidewall cave-in is anticipated, and therefore, permanent Steel Casing is required.
3. A Load Test is required on Shaft No. T1 & No. T2. Shafts shall be installed in conformance with B455-6 of the Specifications. Tip Elevations shall be as required for Production Shafts at each location.
4. The cost for the Test Hole shall be included in the cost for Drilled Shafts. Test Hole Depth is the same required for Production Shafts at Pier No. 11.
5. The Contractor shall perform 60mm to 100mm diameter core run(s) from 1.50m to 6.0m below the shaft excavation as directed by the Engineer. A minimum of 1.5m is required for each shaft.
6. For Adjustment and Relocation of Utilities, See Roadway Plans.

LEGEND

- ○ – Denotes 1525 ϕ Drilled Shaft
- ● – Denotes 1525 ϕ Test Shaft for Statnamic Testing
- △ – Denotes 1525 ϕ Test Hole
- ▢ – Denotes Location of Osterberg Cell Load Test

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	TGA	7-98
Checked by	EP	7-98
Designed by	EP	7-98
Checked by	JWJ	7-98
Approved by	J. JORDAN, P.E.	

ENGINEER OF RECORD:
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1424 Piedmont Drive East
Tallahassee, Florida 32312
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LOGO: JMI ENGINEERS

SEAL: 3/21/99

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R.2	HOLMES	52050-3524

SHEET TITLE: FOUNDATION LAYOUT

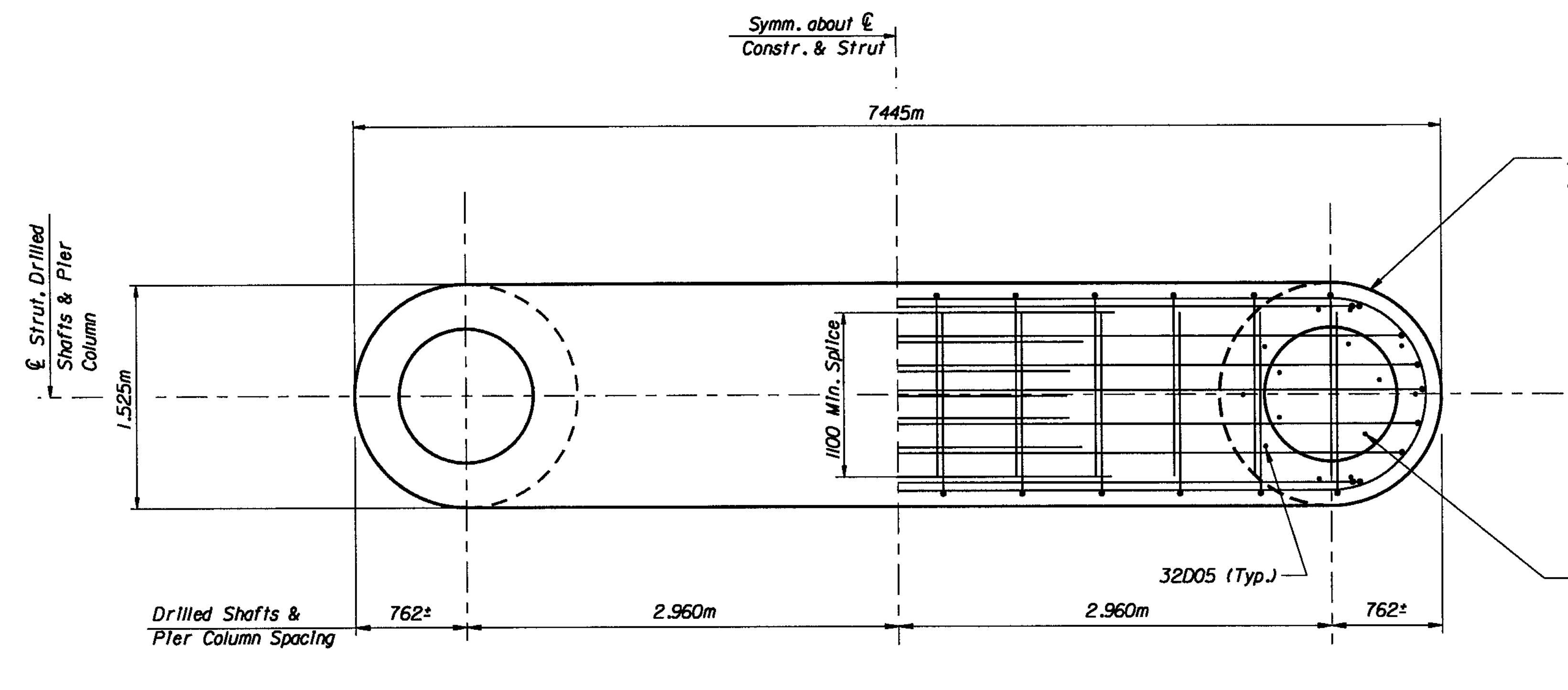
PROJECT NAME: S.R. 2 BRIDGE OVER CHOCTAWHATCHEE RIVER

Drawing No.

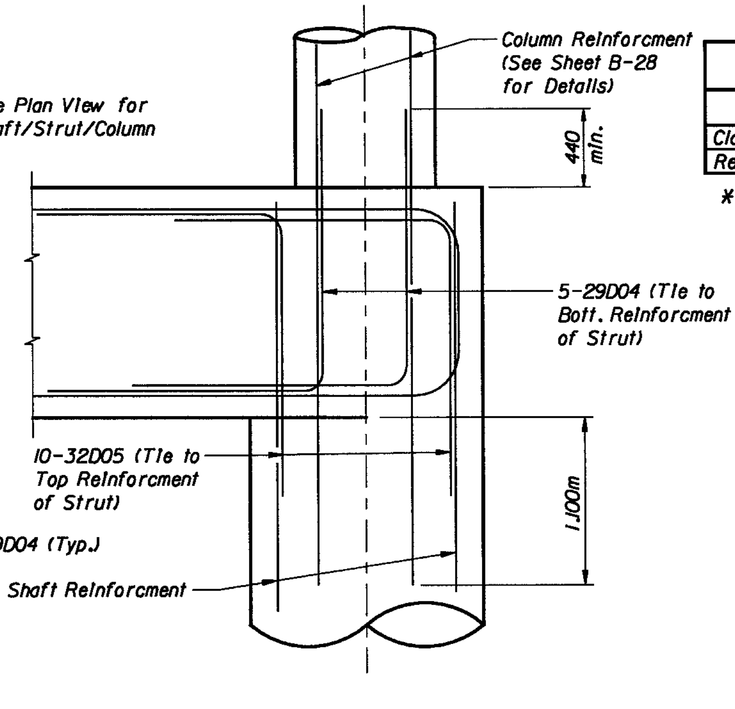
Index No.

JMI Engineers - Tallahassee - 19 MAR 99 08:54:56 o:\952\drawing\bridge\working\found.dgn

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	52050-3524		B-27



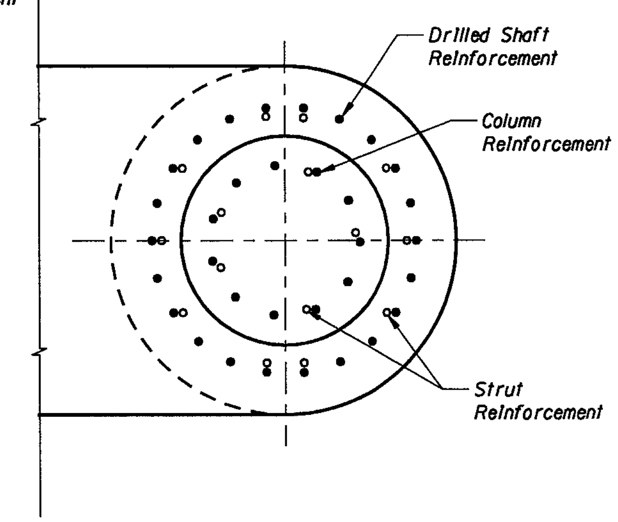
PLAN



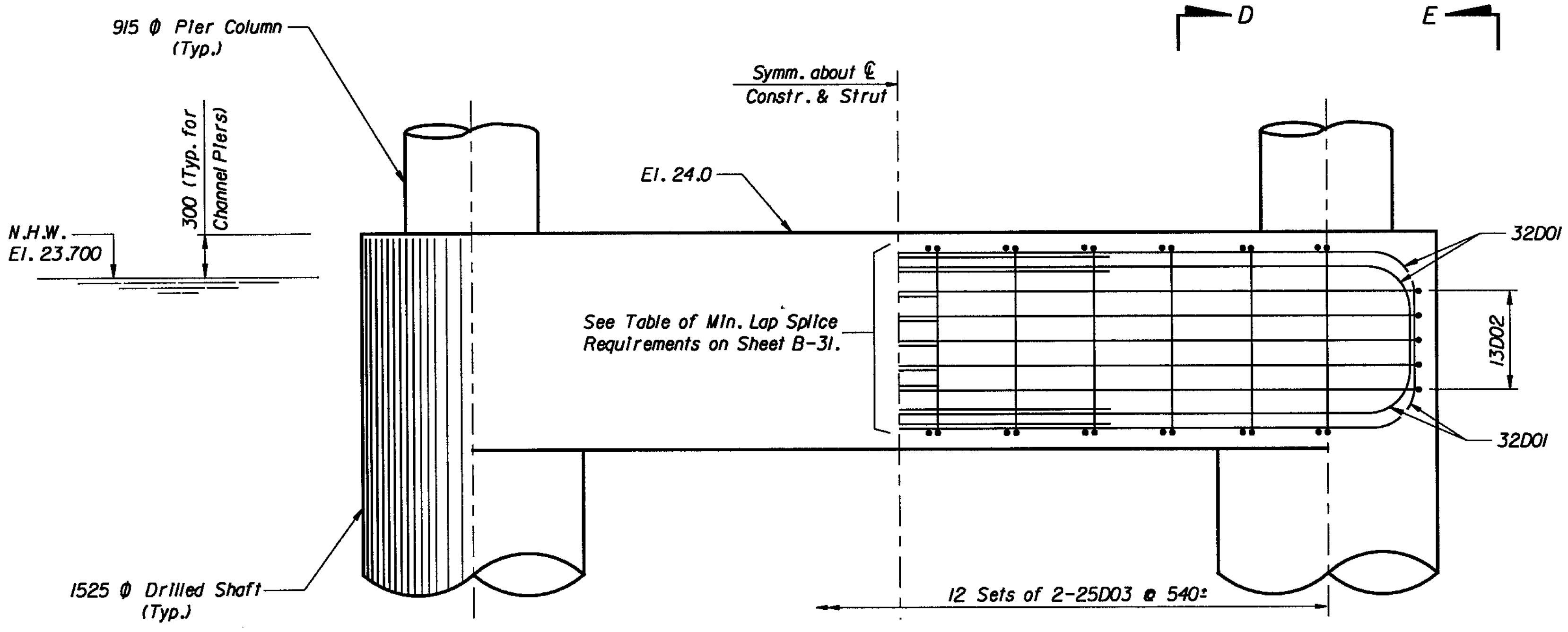
STRUT CONNECTION DETAIL

* ESTIMATED QUANTITIES		
ITEM	UNIT	QUANTITY
Class IV Concrete (Substructure-Mass)	m^3	16.3
Reinforcing Steel (Substructure)	kg	2362

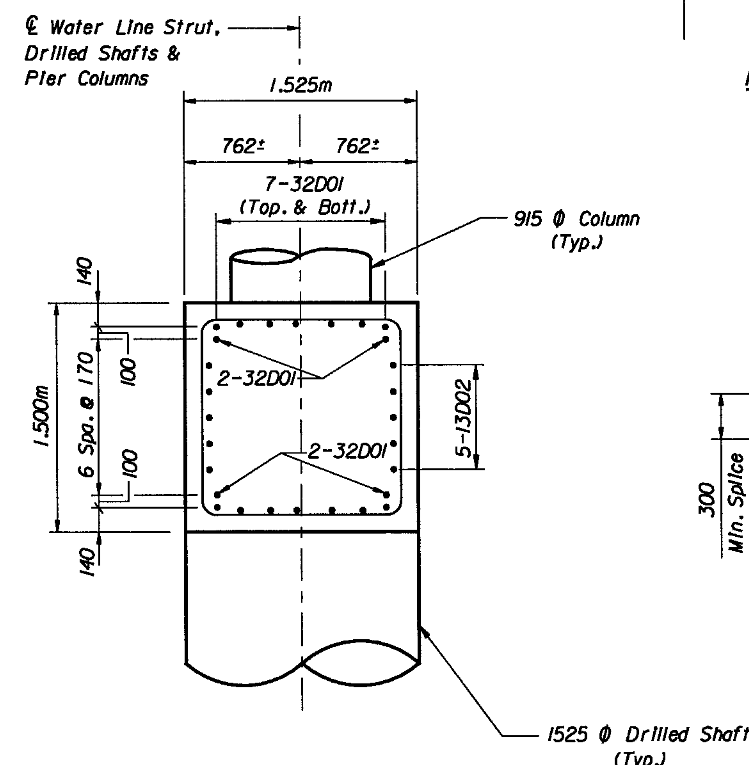
* Quantities are for one Strut only.



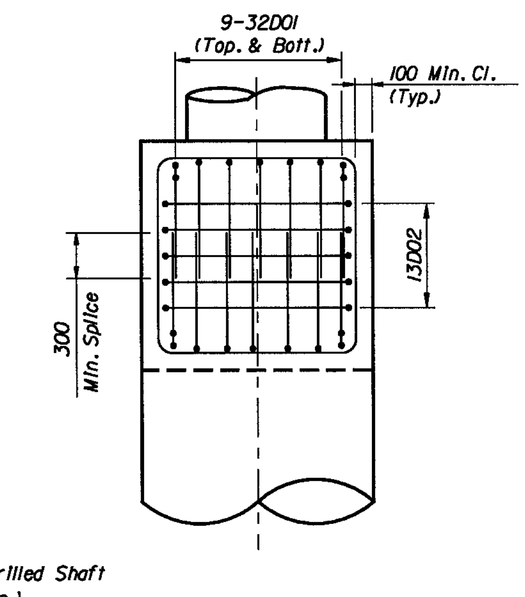
PLAN - SHAFT/STRUT/COLUMN REINFORCMENT DETAIL



ELEVATION



SECTION D-D



SECTION E-E

2/12/99

Notes:

1. Water Line Struts are located at Piers 5 and 12 through 19.
2. Strut shall be constructed as a Mass Concrete Member. Measures shall be taken to cope with the generation of heat and attendant volume change so as to minimize cracking. Mass Concrete Member excludes shafts and columns.

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	TGA	7-98
Checked by	EP	7-98
Designed by	SL	7-98
Checked by	JWJ	7-98
Approved by	J. JORDAN, P.E.	

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LOGO: JMI ENGINEERS

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R.2	HOLMES	52050-3524

SHEET TITLE: WATER LINE STRUT DETAILS FOR PIER NOS. 5 AND 12 - 19

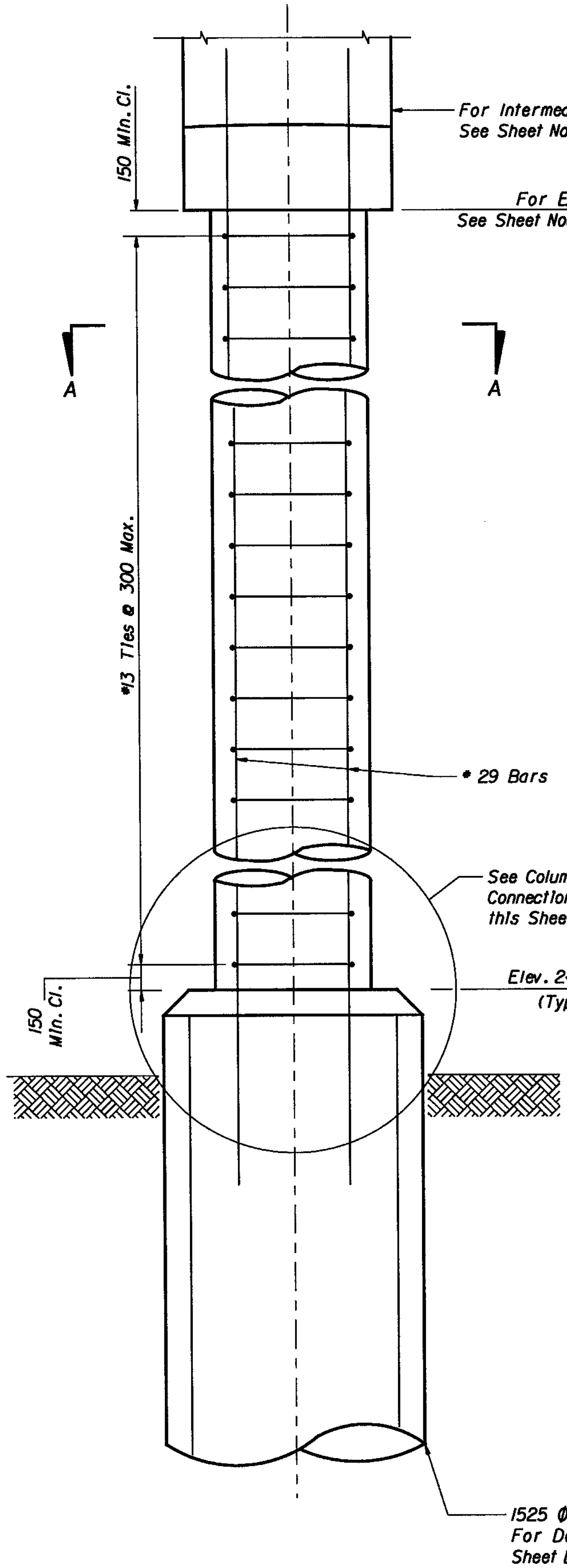
PROJECT NAME: S.R. 2 BRIDGE OVER CHOCTAWHATCHEE RIVER

Drawing No.

Index No.

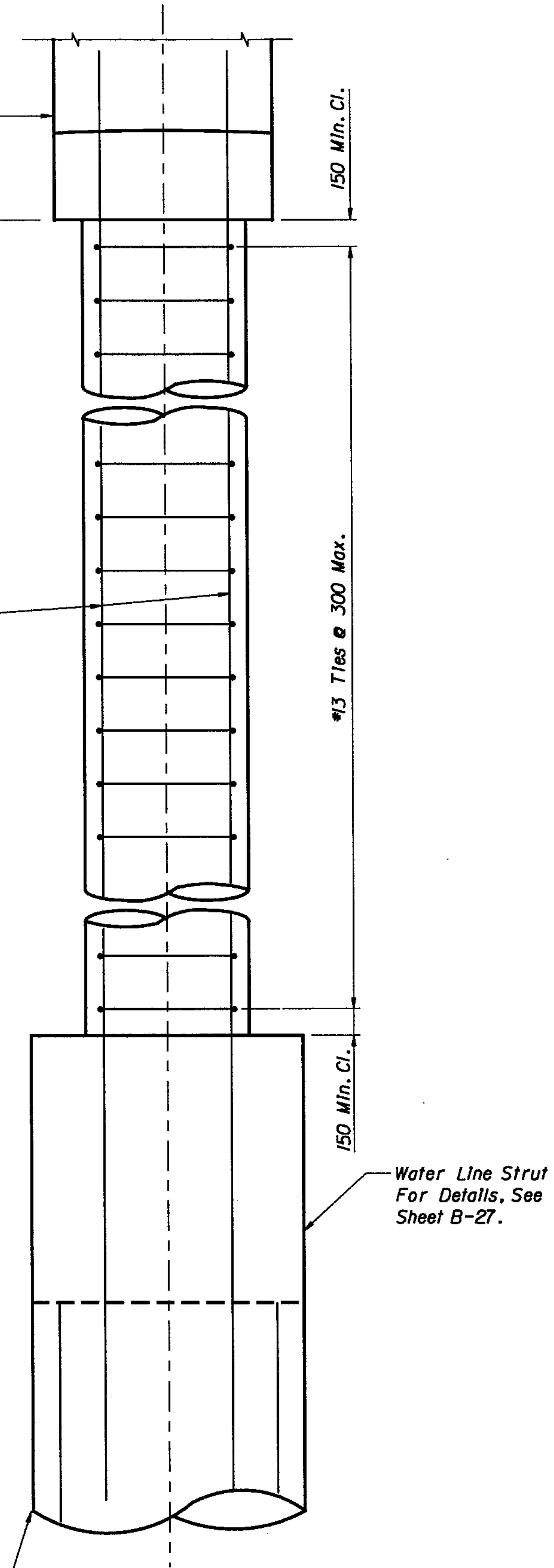
JMI Engineers - Tallahassee - 10 FEB 99 18:20:05 o:\952\drawing\bridge\working\strutdet.dgn

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	52050-3524		B-28



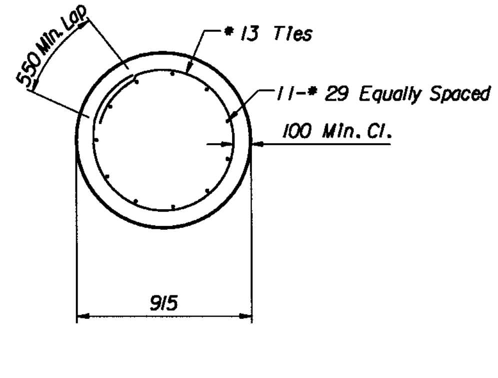
COLUMN DETAIL WITH DRILLED SHAFT

Piers 2-4, 6-11, & 20



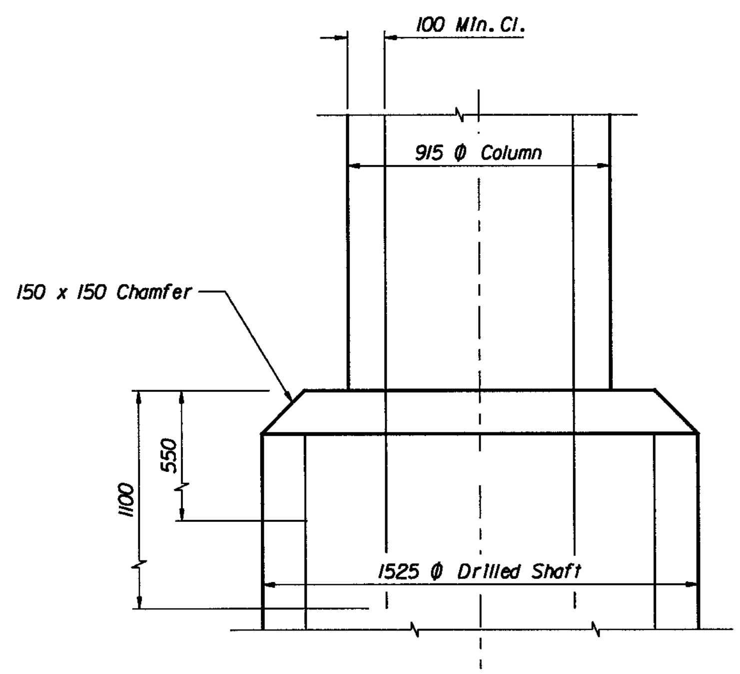
COLUMN DETAIL WITH STRUT & DRILLED SHAFT

Piers 5 & 12-19



SECTION A-A

COLUMN			REINFORCEMENT		
Location	Length (m)	Concrete Quantity (m^3)	Column Bars # 29 (m)	Ties # 13	Quantity (Kg)
Pier No.2	0.921	1.211	3.420	8	405
Pier No.3	1.320	1.736	3.820	10	456
Pier No.4	1.719	2.261	4.220	12	507
Pier No.5	2.119	2.786	6.120	16	730
Pier No.6	2.518	3.312	5.020	18	614
Pier No.7	2.918	3.837	5.420	20	665
Pier No.8	3.317	4.362	5.815	24	721
Pier No.9	3.716	4.887	6.215	26	772
Pier No.10	4.116	5.412	6.615	28	822
Pier No.11	4.479	5.890	6.980	30	869
Pier No.12	4.692	6.171	8.690	32	1066
Pier No.13	4.772	6.276	8.770	32	1075
Pier No.14	4.716	6.202	8.715	32	1068
Pier No.15	4.526	5.923	8.525	32	1047
Pier No.16	4.203	5.527	8.205	30	1005
Pier No.17	3.729	4.904	7.730	26	940
Pier No.18	3.141	4.130	7.140	22	862
Pier No.19	2.511	3.303	6.510	18	780
Pier No.20	1.882	2.475	5.880	14	698
Total		80.6			15103



COLUMN - SHAFT CONNECTION DETAIL

Piers 2-4, 6-11, & 20

2/12/99

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	TGA	7-98
Checked by	EP	7-98
Designed by	EP	7-98
Checked by	JWJ	7-98
Approved by	J. JORDAN, P.E.	

ENGINEER OF RECORD:
JMI ENGINEERS, INC.
1424 Piedmont Drive East
Tallahassee, Florida 32312
Tel: 850-385-7450 Fax: 850-385-3545

LOGO: JMI ENGINEERS

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R.2	HOLMES	52050-3524

SHEET TITLE: COLUMN DETAILS

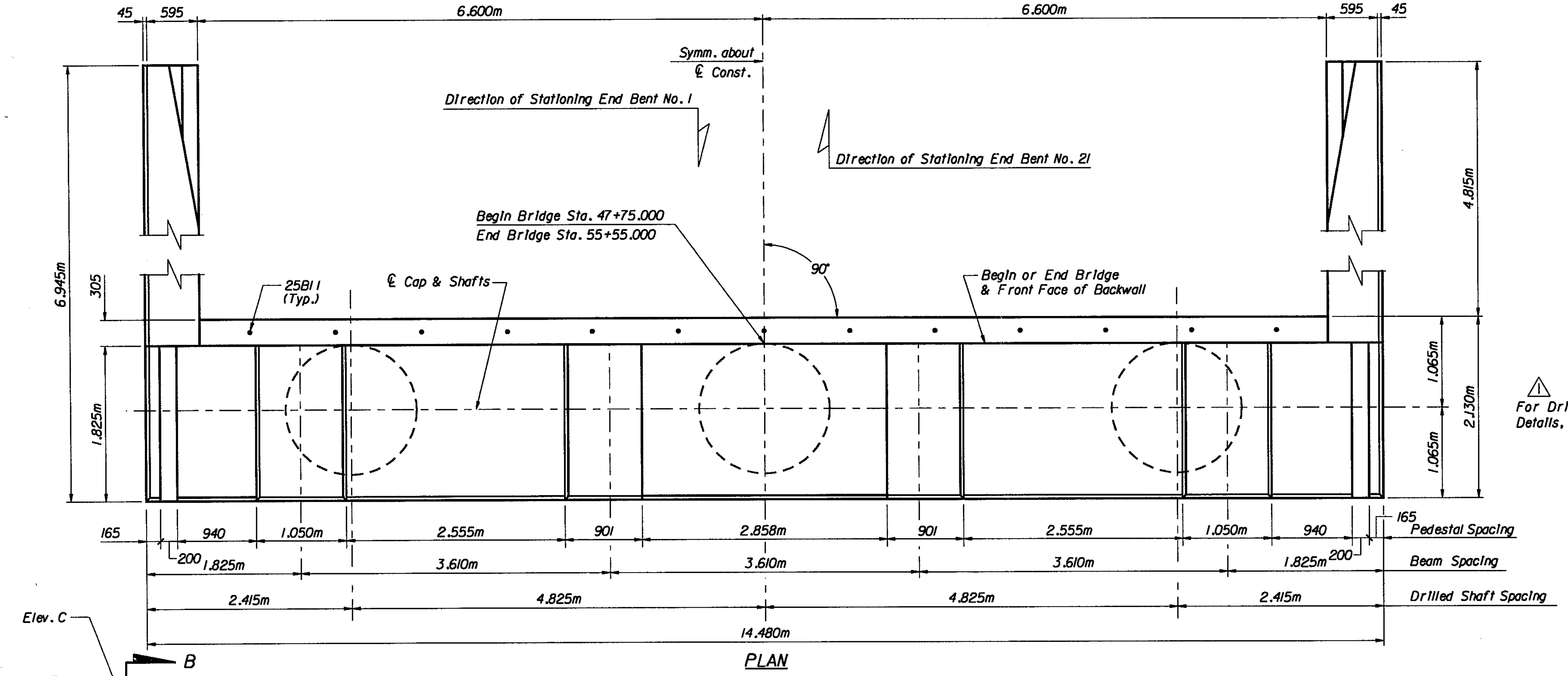
PROJECT NAME: S.R. 2 BRIDGE OVER CHOCTAWHATCHEE RIVER

Drawing No.

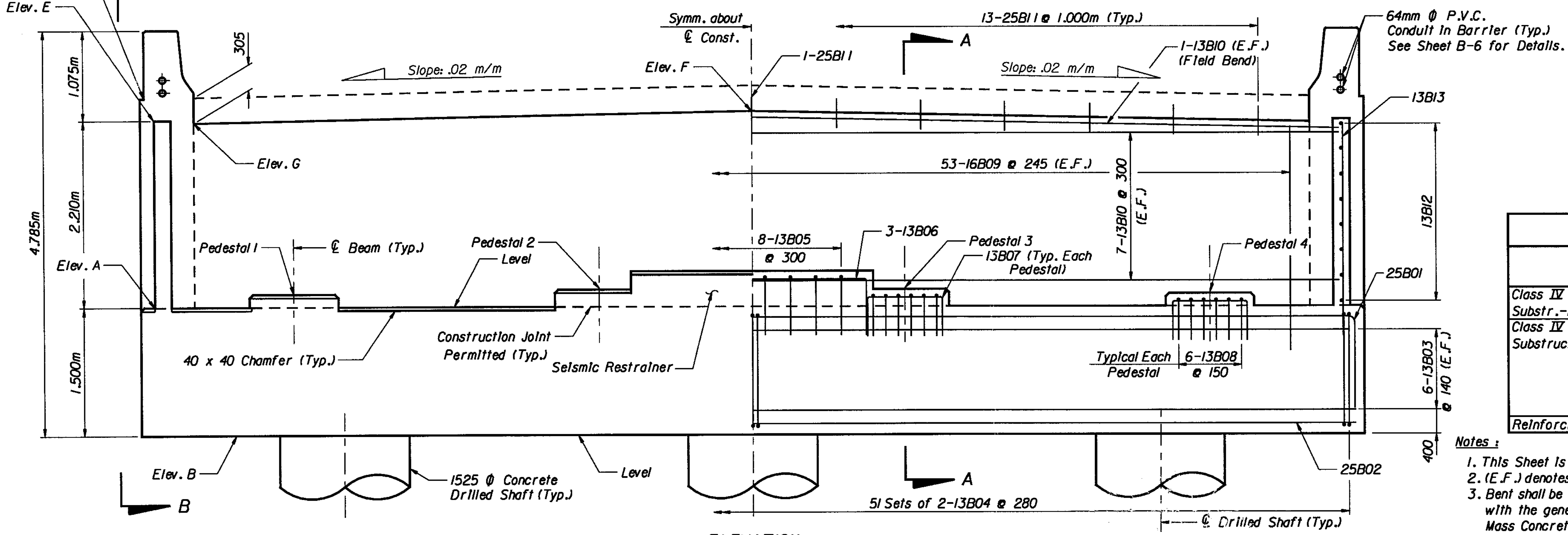
Index No.

JMI Engineers - Tallahassee - 10 FEB 99 18:21:52 o:\952\drawing\bridge\working\column.dgn

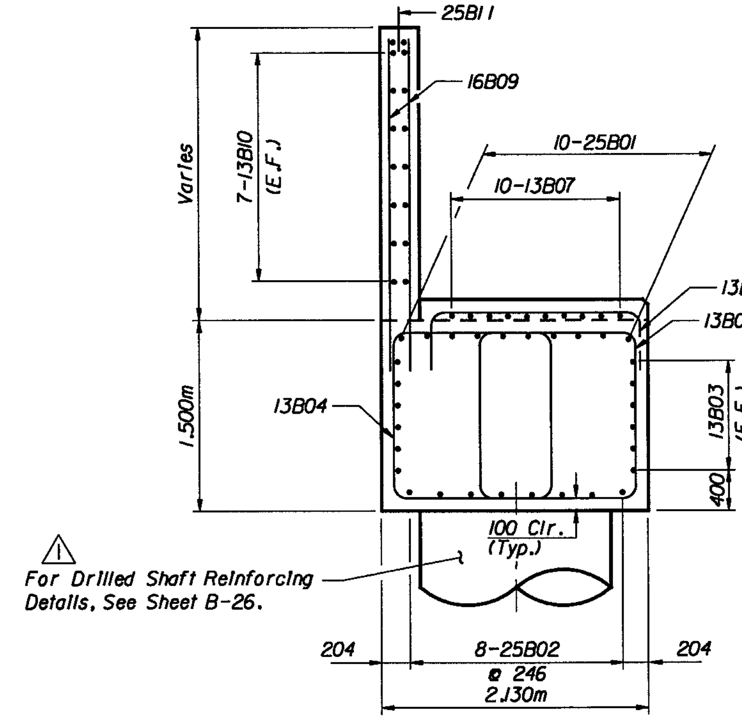
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	52050-3524		B-29



PLAN



ELEVATION



SECTION A-A

BENT PEDESTAL ELEVATIONS

Location	Bent No. 1 47+75.000	Bent No. 21 55+55.000
Pedestal 1	26.127	26.874
Pedestal 2	26.199	26.946
Seismic Restrainer	26.414	27.161
Pedestal 3	26.199	26.946
Pedestal 4	26.127	26.874

BENT CONSTRUCTION ELEVATIONS

Location	Bent No. 1 47+75.000	Bent No. 21 55+55.000
Elev. (A)	26.027	26.774
Elev. (B)	24.527	25.274
Elev. (C)	28.497	29.242
Elev. (D)	28.448	29.164
Elev. (E)	28.237	28.982
Elev. (F)	28.324	29.069
Elev. (G)	28.192	28.937

ESTIMATED QUANTITIES

ITEM		UNIT	QUANTITY END BENT 1	QUANTITY END BENT 21
Class IV Conc. Substr.-Mass	Cap	m³	46.264	46.264
Class IV Conc. Substructure	Cheek walls	m³	1.614	1.612
	Back wall	m³	8.982	8.974
	Wing walls	m³	16.272	16.272
	Barrier transition	m³	3.702	3.702
	Pedestals	m³	3.062	3.062
	Total	m³	33.6	33.6
Reinforcing Steel (Substructure)		kg	4885	4885

Notes :

1. This Sheet is to be used with Sheet B-30.
2. (E.F.) denotes Each Face
3. Bent shall be constructed as a Mass Concrete Member. Measures shall be taken to cope with the generation of heat and attendant volume change so as to minimize cracking. Mass Concrete Member excludes wingwalls, shafts, pedestals, backwall, cheekwall and barrier transitions.

REVISIONS

Date	By	Description	Date	By	Description
1/26/00	SGS	⚠ Added Reinforcing Note			

	Names	Dates
Drawn by	TGA	7-98
Checked by	EP	7-98
Designed by	SL	7-98
Checked by	JWJ	7-98
Approved by	J. JORDAN, P.E.	

ENGINEER OF RECORD:
JMI ENGINEERS, INC.
1424 Piedmont Drive East
Tallahassee, Florida 32312
Tel: 850-385-7450 Fax: 850-385-3545

LOGO:
JMI ENGINEERS

SEAL:
1/26/00

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 2	HOLMES	52050-3524

SHEET TITLE: END BENT NOS. 1 AND 21

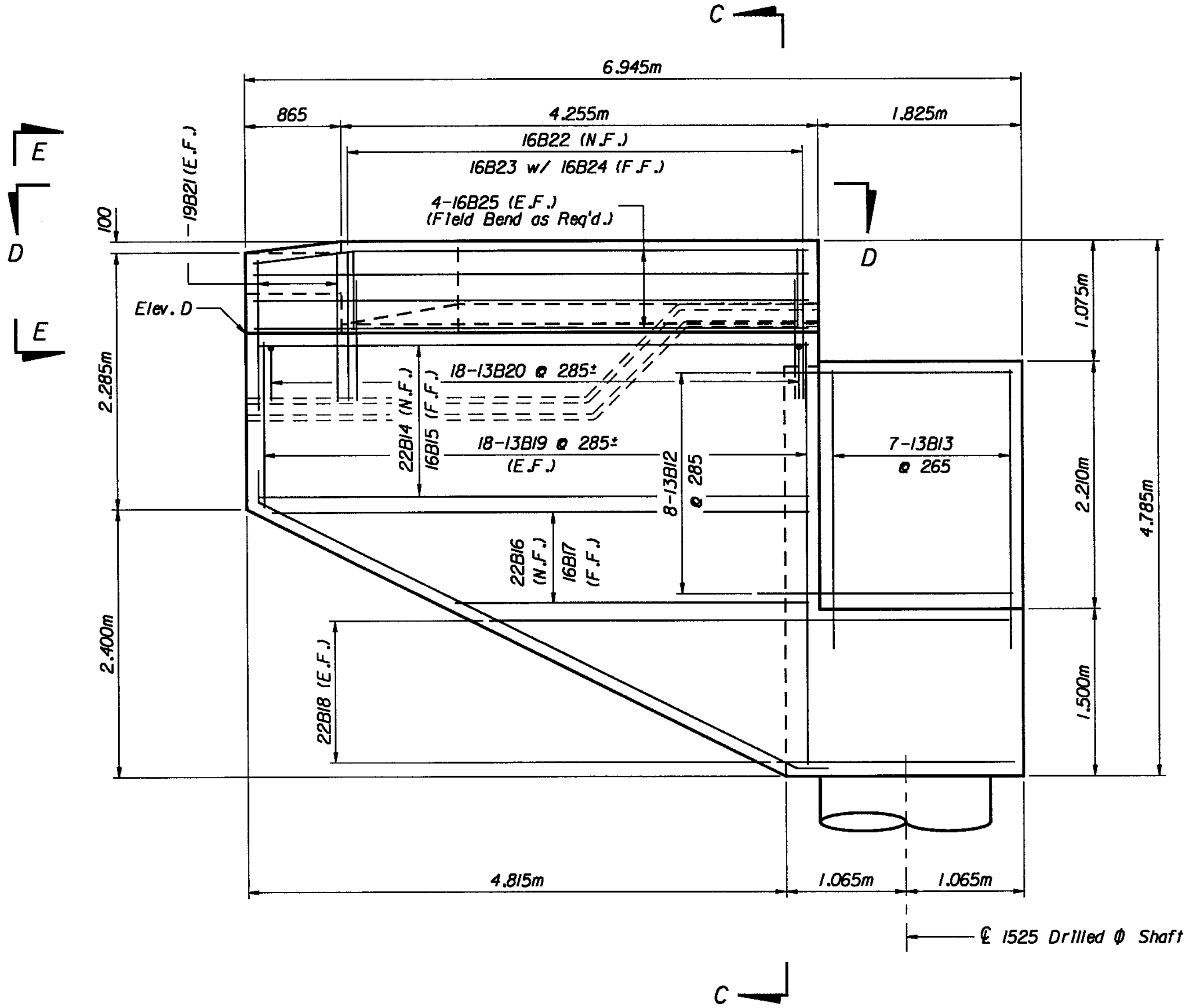
PROJECT NAME: S.R. 2 BRIDGE OVER CHOCTAWHATCHEE RIVER

Drawing No.

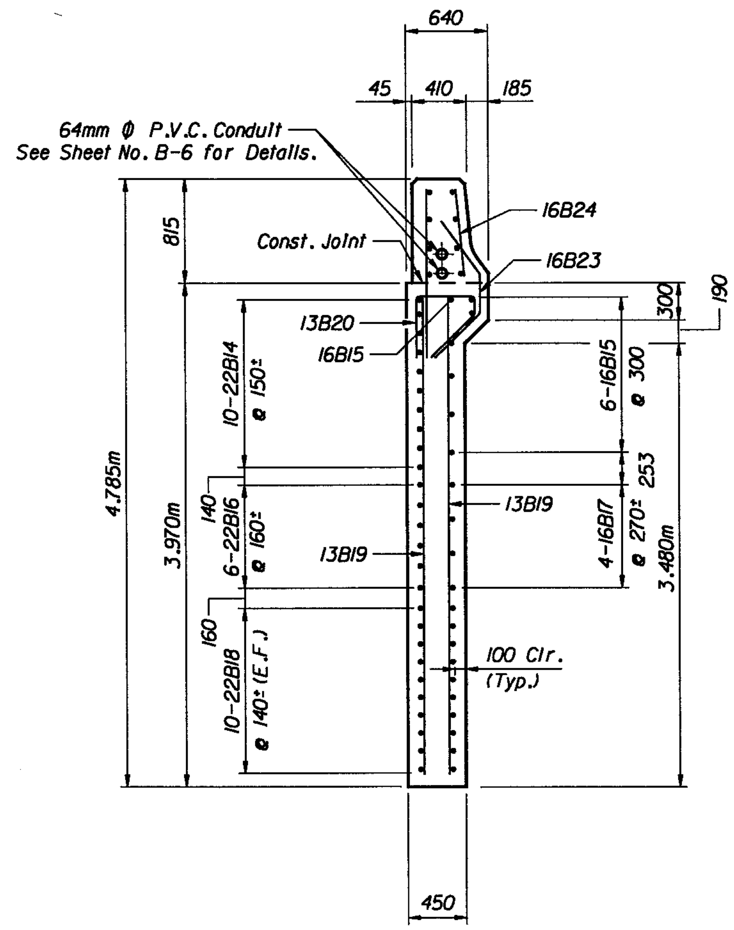
Index No.

JMI Engineers - Tallahassee - 26 JAN 100 14:49:04 a:\952\drawing\bridge\final\ebl&21.dgn

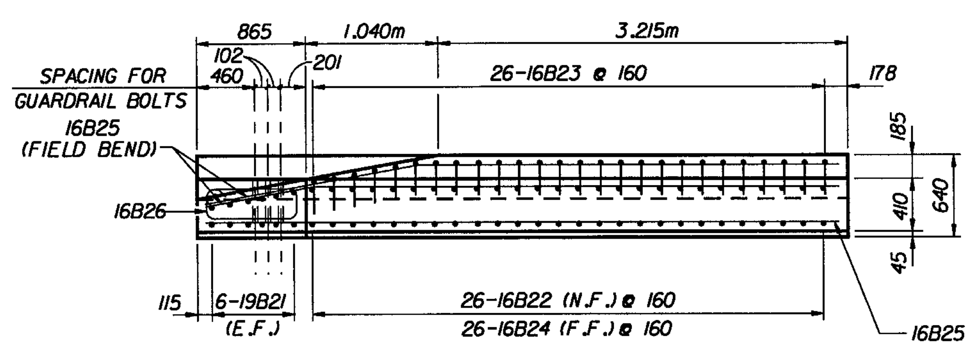
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	52050-3524		B-30



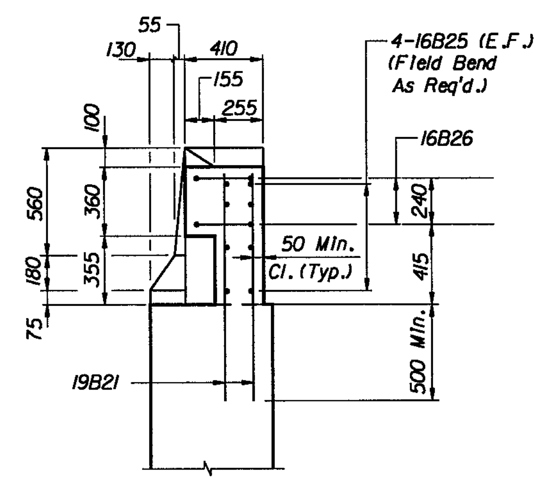
ELEVATION B-B



SECTION C-C



SECTION D-D



SECTION E-E

2/12/99

Notes :

1. This sheet is to be used in conjunction with Sheet B-29.
2. (E.F.) denotes Each Face.
3. (N.F.) denotes Near Face.
4. (F.F.) denotes Far Face.

o:\952\drawing\bridge\working\ebdetails.dgn 10 FEB 99 18:24:31 JMI Engineers - Tallahassee

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	TGA	7-98
Checked by	EP	7-98
Designed by	EP	7-98
Checked by	JWJ	7-98
Approved by	J. JORDAN, P.E.	

ENGINEER OF RECORD:
JMI ENGINEERS, INC.
1424 Piedmont Drive East
Tallahassee, Florida 32312
Tel: 850-385-7450 Fax: 850-385-3545

LOGO: JMI ENGINEERS

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R.2	HOLMES	52050-3524

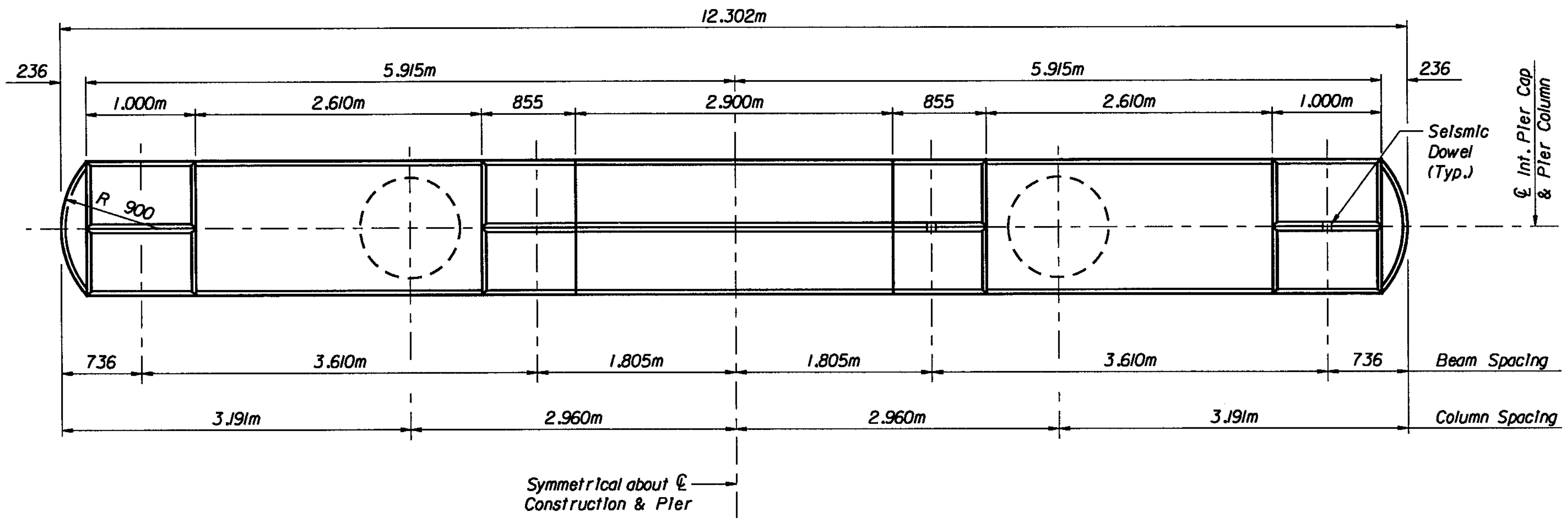
SHEET TITLE: END BENT DETAILS

PROJECT NAME: S.R. 2 BRIDGE OVER CHOCTAWHATCHEE RIVER

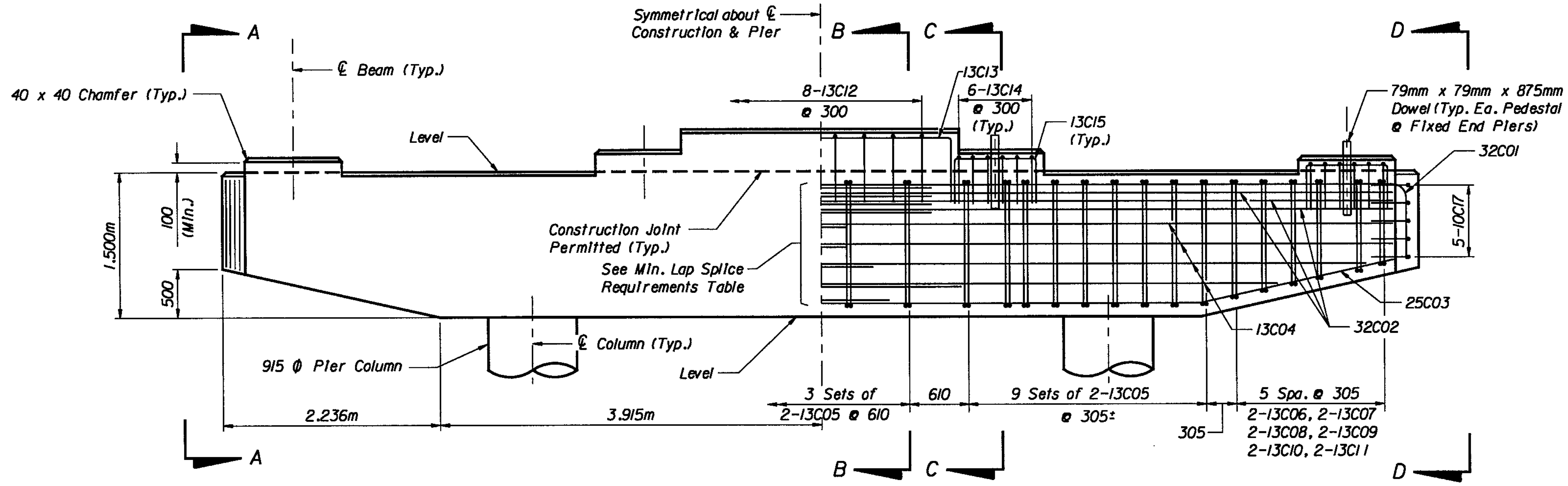
Drawing No.

Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	52050-3524		B-31



PLAN



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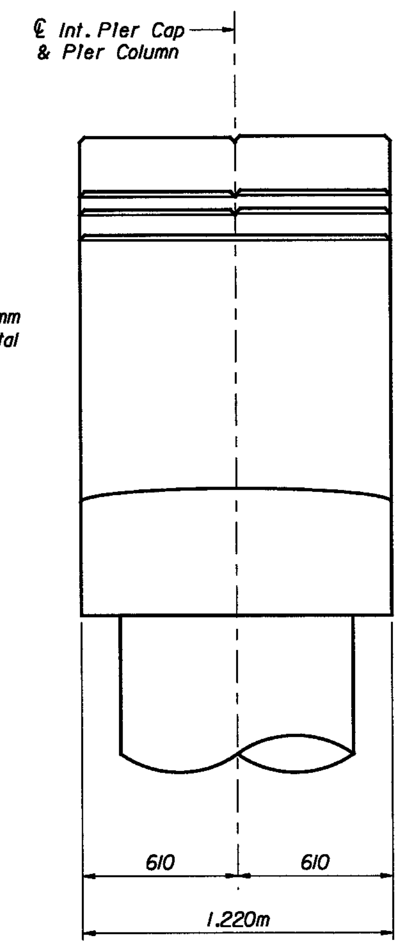
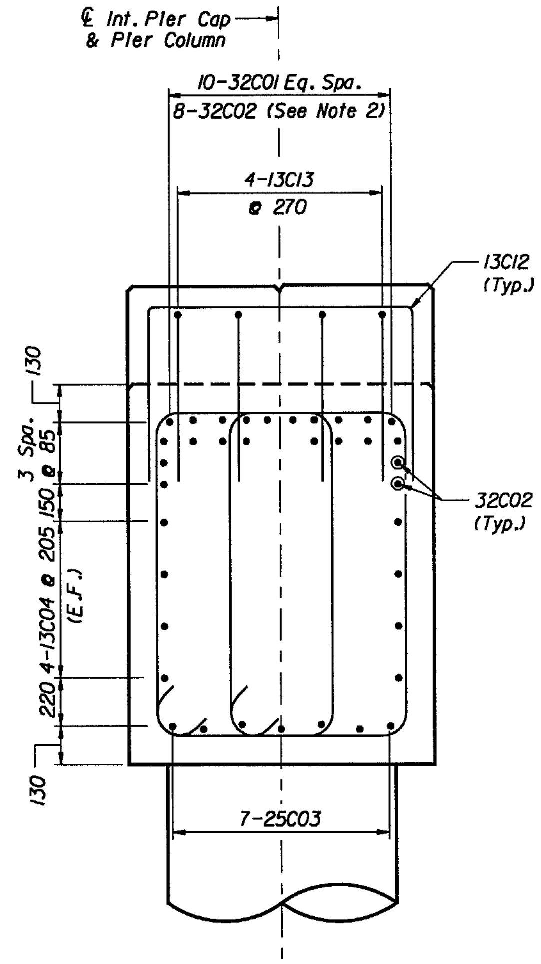


SECTION A-A



SECTION B-B

*ESTIMATED QUANTITIES			
ITEM		UNIT	QUANTITY INT. BENT
Class IV Conc. Substr.-Mass	Cap	m3	20.76
Class IV Conc. Substructure	Pedestals **	m3	0.68
	Column	m3	***
Reinforcing Steel (Substructure)		kg	3733

* Quantities are for one bent.
** Pedestal concrete volume is an average per bent.
*** See sheet B-28 for column quantities.

Min. Lap Splice Requirements	
Bar Type	Min. Splice Length
# 32	2.300m
# 25	1.450m
# 13	0.550m

2/12/99

Notes :

1. Refer to Sheet B-32 for SECTION C-C, D-D and for pedestal elevations.
2. Bars 32C02 shall be aligned with Bars 32C01 as shown.
3. Refer to Sheet B-32 and B-36 for seismic dowel details.
4. Bent shall be constructed as a Mass Concrete Member. Measures shall be taken to cope with the generation of heat and attendant volume change so as to minimize cracking. Mass Concrete Member excludes columns and pedestals.

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	TGA	7-98
Checked by	EP	7-98
Designed by	SL	7-98
Checked by	JWJ	7-98
Approved by	J. JORDAN, P.E.	

ENGINEER OF RECORD:
JMI ENGINEERS, INC.
1424 Piedmont Drive East
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Tel: 850-385-7450 Fax: 850-385-3545

LOGO: JMI ENGINEERS

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R.2	HOLMES	52050-3524

SHEET TITLE: INTERMEDIATE PIER NOS. 2 THRU 20

PROJECT NAME: S.R. 2 BRIDGE OVER CHOCTAWHATCHEE RIVER

Drawing No.

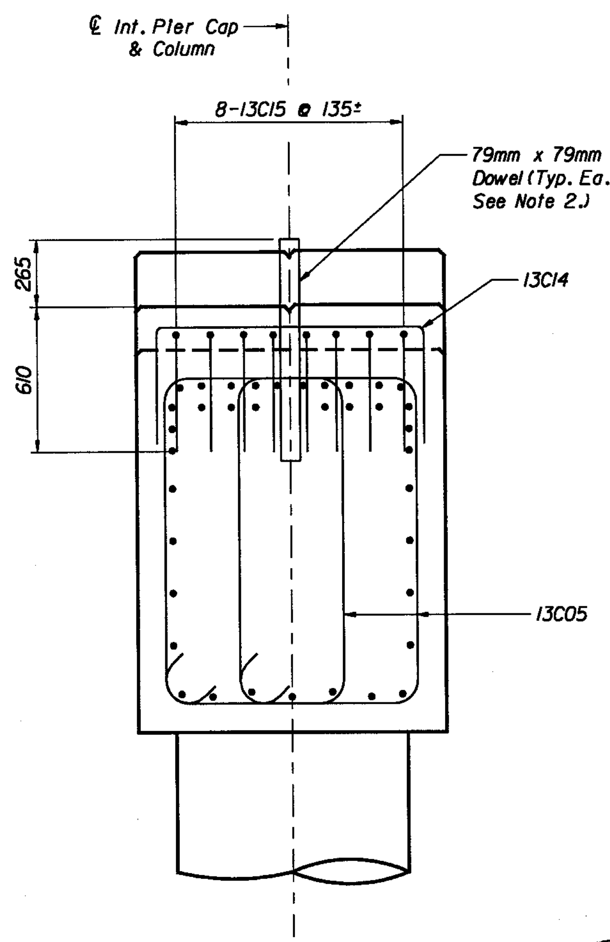
Index No.

JMI Engineers - Tallahassee - 10 FEB 99 18:25:13 o:\952\drawing\bridge\working\intb2-20.dgn

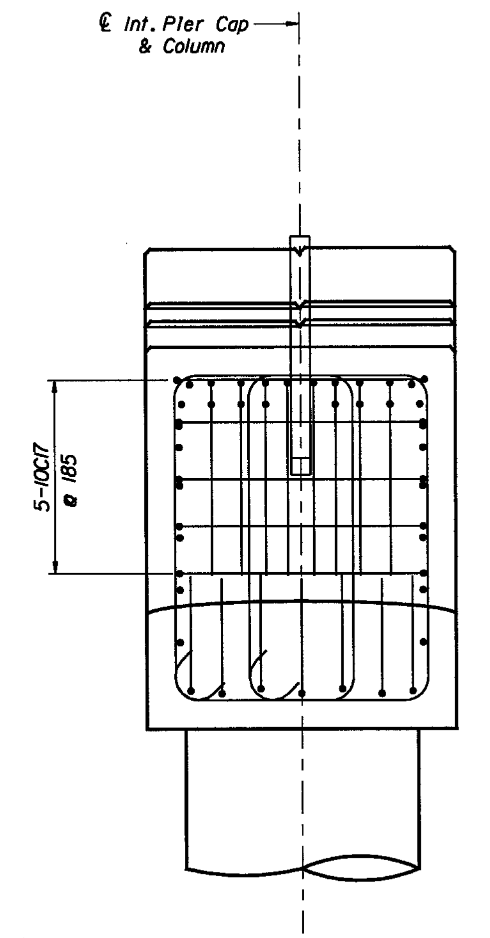
BENT CONSTRUCTION ELEVATIONS

Bent Location		Ped. (1)	Ped. (2)	Seismic Restrainer	Ped. (3)	Ped. (4)	Top of Cap	Bott. of Cap
Pier No. 2	DN Station	26.521	26.593	26.808	26.593	26.521	26.421	24.921
	UP Station	26.526	26.599	26.814	26.599	26.526		
Pier No. 3	DN Station	26.920	26.992	27.207	26.992	26.920	26.820	25.320
	UP Station	26.926	26.998	27.213	26.998	26.926		
Pier No. 4	DN Station	27.319	27.392	27.607	27.392	27.319	27.219	25.719
	UP Station	27.325	27.397	27.612	27.397	27.325		
Pier No. 5	DN Station	27.719	27.791	28.006	27.791	27.719	27.619	26.119
	UP Station	27.725	27.797	28.012	27.797	27.725		
Pier No. 6	DN Station	28.118	28.190	28.405	28.190	28.118	28.018	26.518
	UP Station	28.124	28.196	28.411	28.196	28.124		
Pier No. 7	DN Station	28.518	28.590	28.805	28.590	28.518	28.418	26.918
	UP Station	28.523	28.595	28.810	28.595	28.523		
Pier No. 8	DN Station	28.917	28.989	29.204	28.989	28.917	28.817	27.317
	UP Station	28.923	28.995	29.210	28.995	28.923		
Pier No. 9	DN Station	29.316	29.388	29.603	29.388	29.316	29.216	27.716
	UP Station	29.322	29.394	29.609	29.394	29.322		
Pier No. 10	DN Station	29.716	29.788	30.003	29.788	29.716	29.616	28.116
	UP Station	29.738	29.811	30.026	29.811	29.738		
Pier No. 11	DN Station	30.079	30.151	30.366	30.151	30.079	29.979	28.479
	UP Station	30.083	30.155	30.370	30.155	30.083		

Bent Location		Ped. (1)	Ped. (2)	Seismic Restrainer	Ped. (3)	Ped. (4)	Top of Cap	Bott. of Cap
Pier No. 12	DN Station	30.292	30.365	30.580	30.365	30.292	30.192	28.692
	UP Station	30.294	30.367	30.582	30.367	30.294		
Pier No. 13	DN Station	30.372	30.444	30.659	30.444	30.372	30.272	28.772
	UP Station	30.372	30.444	30.659	30.444	30.372		
Pier No. 14	DN Station	30.318	30.390	30.605	30.390	30.318	30.216	28.716
	UP Station	30.316	30.388	30.603	30.388	30.316		
Pier No. 15	DN Station	30.130	30.202	30.417	30.202	30.130	30.026	28.526
	UP Station	30.126	30.199	30.414	30.199	30.126		
Pier No. 16	DN Station	29.809	29.881	30.096	29.881	29.809	29.703	28.203
	UP Station	29.803	29.875	30.090	29.875	29.803		
Pier No. 17	DN Station	29.353	29.425	29.640	29.425	29.353	29.229	27.729
	UP Station	29.329	29.401	29.616	29.401	29.329		
Pier No. 18	DN Station	28.750	28.822	29.037	28.822	28.750	28.641	27.141
	UP Station	28.741	28.813	29.028	28.813	28.741		
Pier No. 19	DN Station	28.120	28.193	28.408	28.193	28.120	28.011	26.511
	UP Station	28.111	28.184	28.399	28.184	28.111		
Pier No. 20	DN Station	27.491	27.563	27.778	27.563	27.491	27.382	25.882
	UP Station	27.482	27.554	27.769	27.554	27.482		

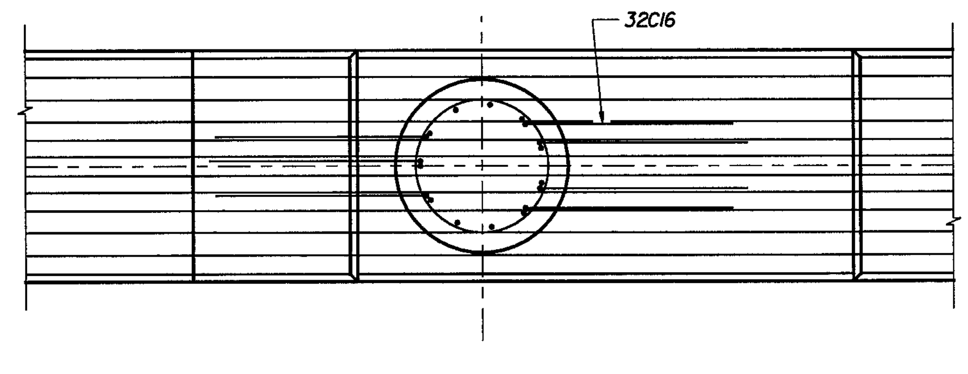


SECTION C-C

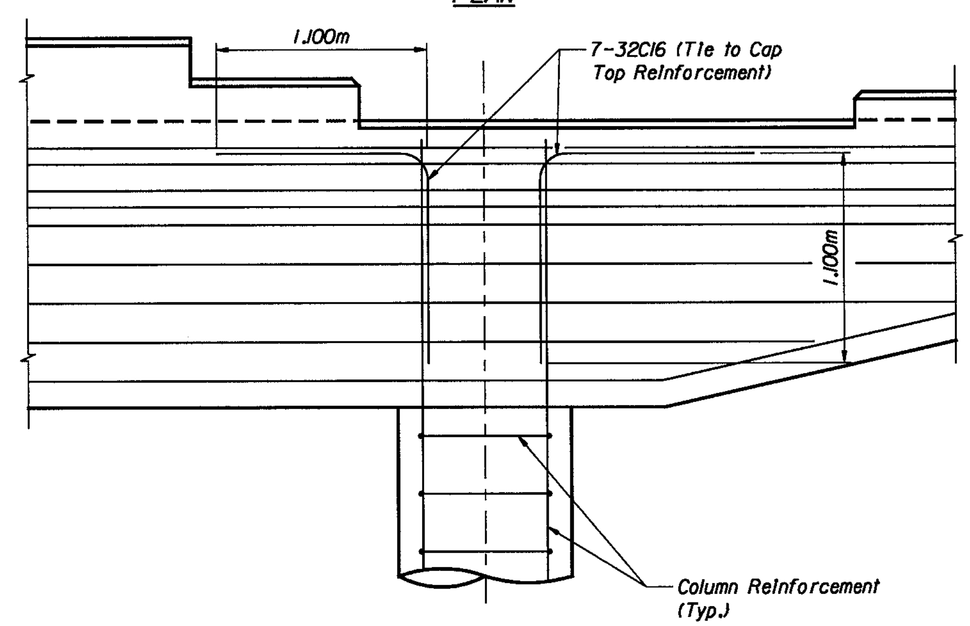


SECTION D-D

2/12/99



PLAN



ELEVATION
COLUMN-CAP CONNECTION DETAIL

Notes :
1. Use this Sheet in conjunction with Sheet B-31.
2. Seismic Dowels in Bents 3, 7, 11, 15, & 19, only.

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	TGA	7-98
Checked by	EP	7-98
Designed by	SL	7-98
Checked by	JWJ	7-98
Approved by	J. JORDAN, P.E.	

ENGINEER OF RECORD:
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Tel: 850-385-7450 Fax: 850-385-3545

LOGO: JMI ENGINEERS

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

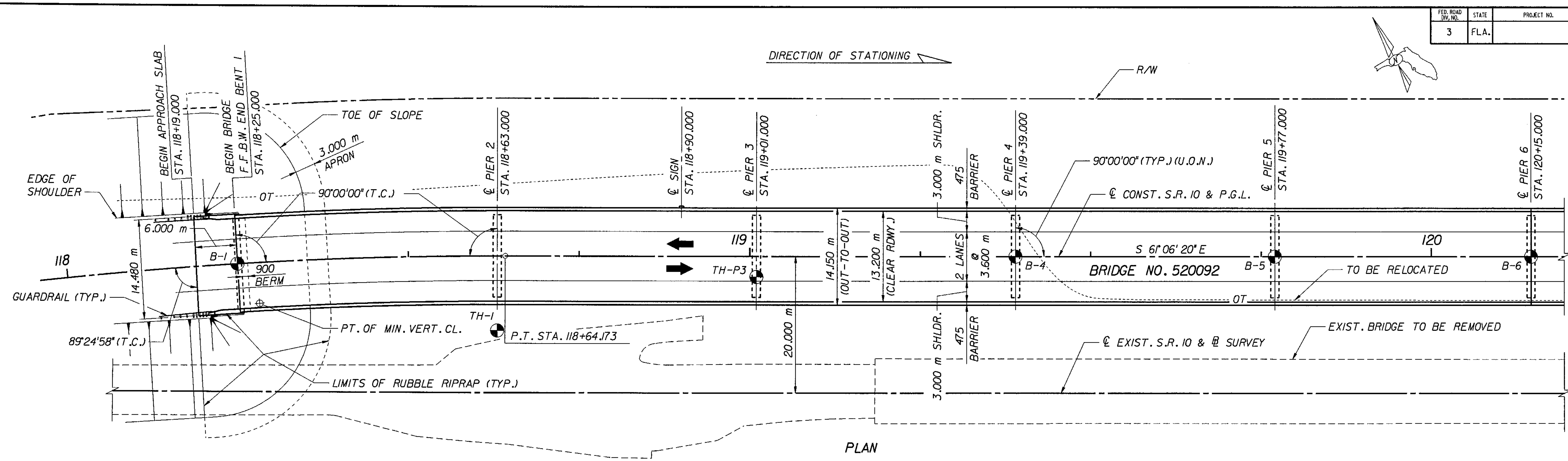
ROAD NO.	COUNTY	PROJECT NO.
S.R. 2	HOLMES	52050-3524

SHEET TITLE: INTERMEDIATE PIER DETAILS

PROJECT NAME: S.R. 2 BRIDGE OVER CHOCTAWHATCHEE RIVER

Drawing No.

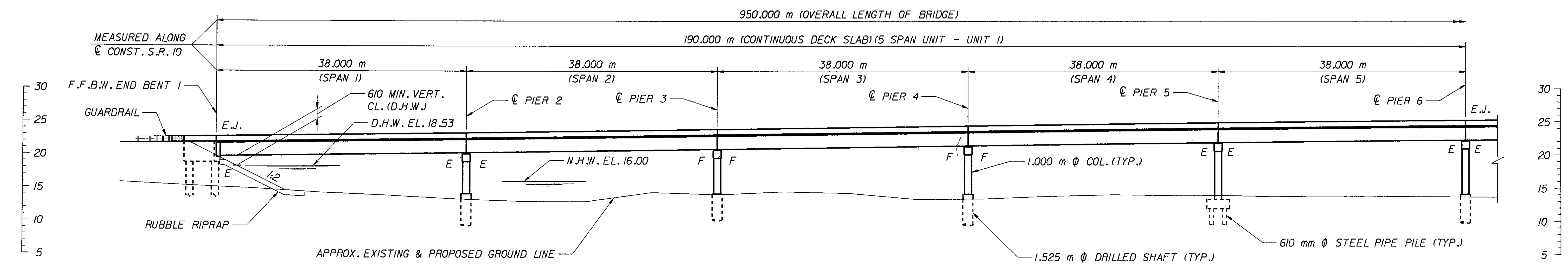
Index No.



PLAN

NOTES:

1. BEARING DESIGNATIONS:
   E = EXPANSION BEARING
   F = FIXED BEARING
2. E.J. INDICATES LOCATION OF EXPANSION JOINTS IN BRIDGE DECK.
3. (boring symbol) INDICATES BORING LOCATIONS, FOR BORING INFORMATION, SEE REPORT OF SPT BORINGS FOR STRUCTURES, SHEETS B-15 THRU B-41.
4. FOR GEOMETRIC DATA, SEE SHEET B-13.



ELEVATION

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	JSR	3/99
Checked by	DET	3/99
Designed by	DLC	3/99
Checked by	PRS	3/99
Approved by	JWB	

ENGINEER OF RECORD:
HDR ENGINEERING, INC.
5100 W. Kennedy Boulevard
Suite No. 300
Tampa, Florida 33609-1840

LOGO: HDR
HDR Engineering, Inc.
Tampa, Florida

SEAL: Wayman Bolly 3-26-99

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

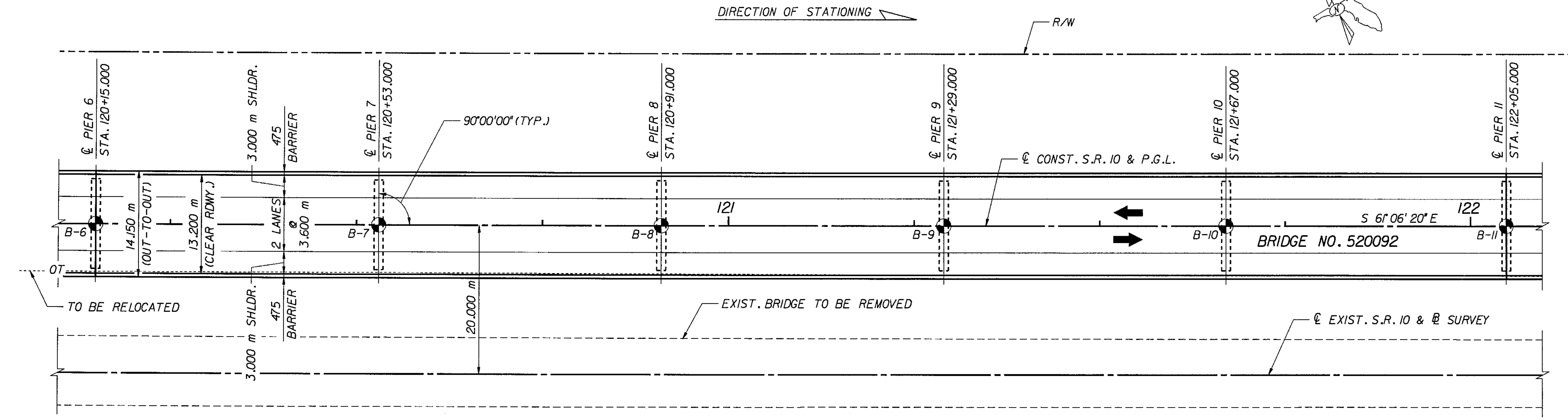
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
10	HOLMES	220784-2-52-01

SHEET TITLE: PLAN AND ELEVATION (SHEET 1 OF 5)

PROJECT NAME: S.R. 10 OVER CHOCTAWHATCHEE RIVER

Drawing No.

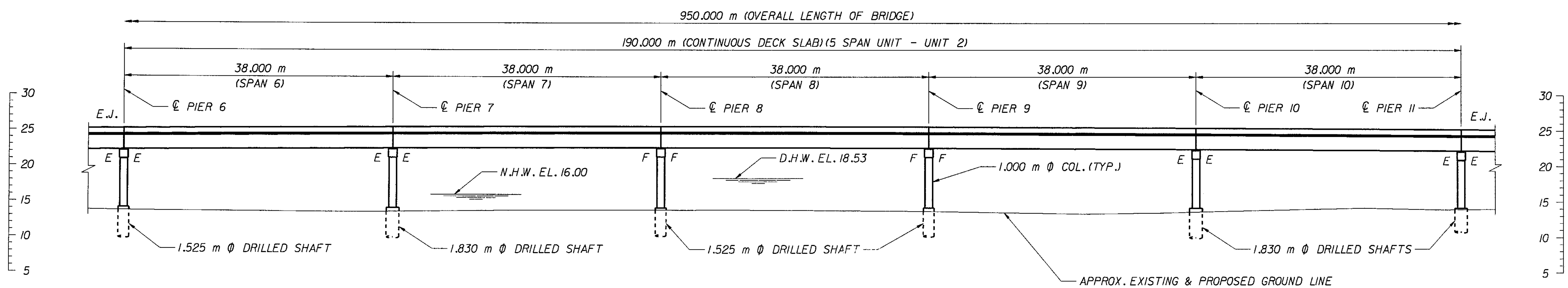
Index No.



PLAN

NOTES:

1. FOR NOTES, SEE SHEET B-8.
2. FOR GEOMETRIC DATA, SEE SHEET B-13.



ELEVATION

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	JSR	3/99
Checked by	DET	3/99
Designed by	DLC	3/99
Checked by	PRS	3/99
Approved by	JWB	

ENGINEER OF RECORD:
HDR ENGINEERING, INC.
5100 W. Kennedy Boulevard
Suite No. 300
Tampa, Florida 33609-1840

LOGO:
HDR Engineering, Inc.
Tampa, Florida

SEAL:
Wayne Bailey
3-26-99

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
10	HOLMES	220784-2-52-01

SHEET TITLE:
PLAN AND ELEVATION
(SHEET 2 OF 5)

PROJECT NAME:
S.R. 10 OVER CHOCTAWHATCHEE RIVER

Drawing No.

Index No.

Fri Mar 26 09:19:42 1999

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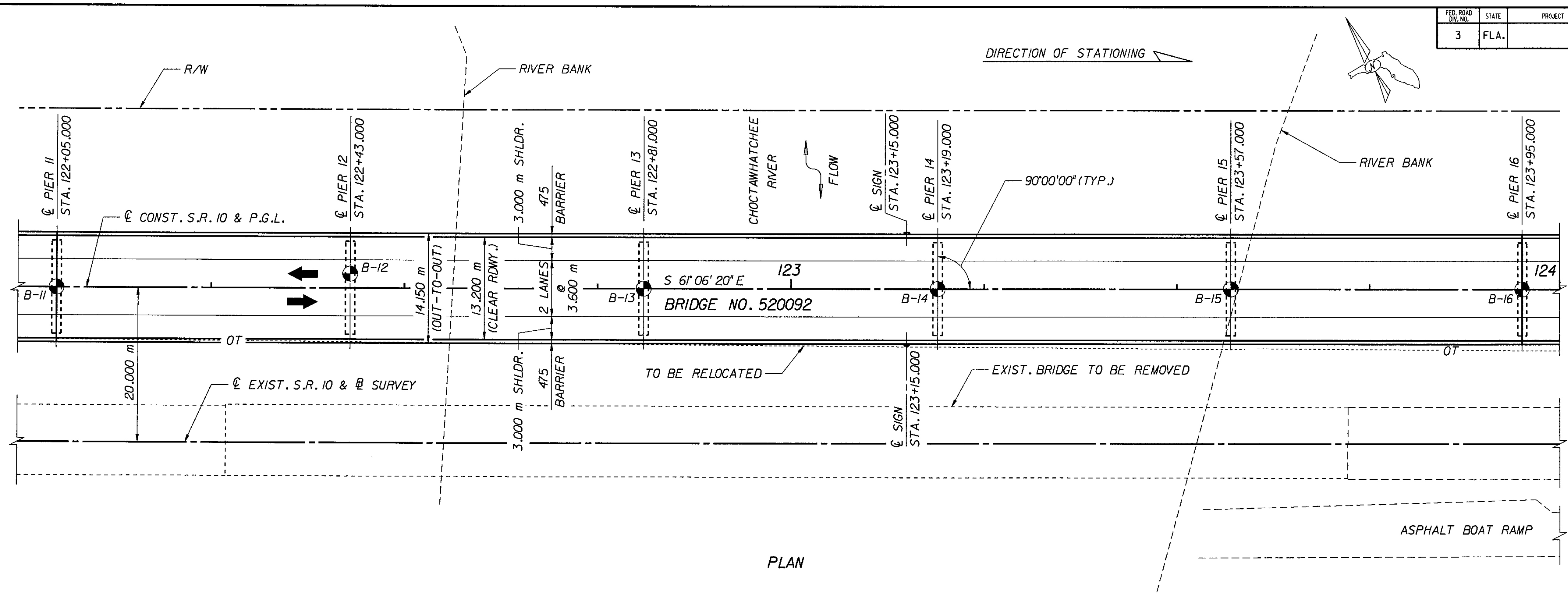
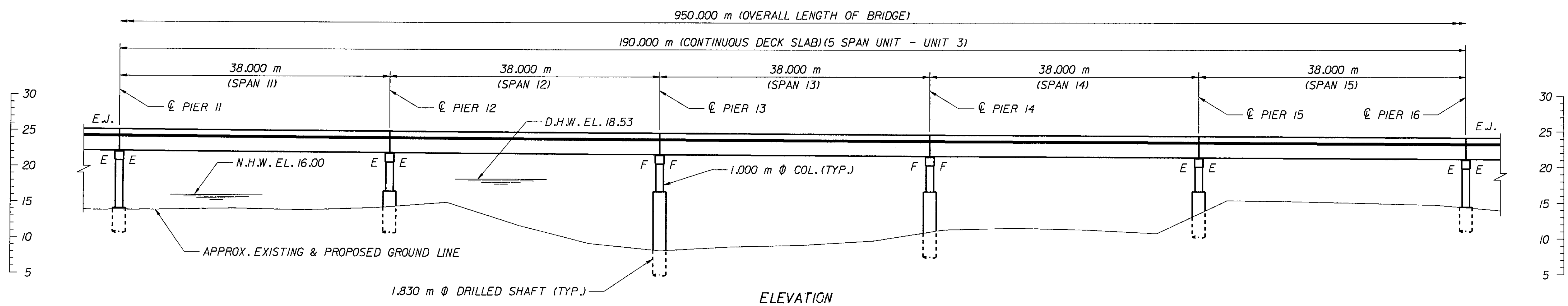


PLAN

NOTES:

1. FOR NOTES, SEE SHEET B-8.

2. FOR GEOMETRIC DATA, SEE SHEET B-13.



ELEVATION

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	JSR	3/99
Checked by	DET	3/99
Designed by	DLC	3/99
Checked by	PRS	3/99
Approved by	JWB	

ENGINEER OF RECORD:
HDR ENGINEERING, INC.
5100 W. Kennedy Boulevard
Suite No. 300
Tampa, Florida 33609-1840

LOGO:
HDR
HDR Engineering, Inc.
Tampa, Florida

SEAL:
Wayne Bolly
3-26-99

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

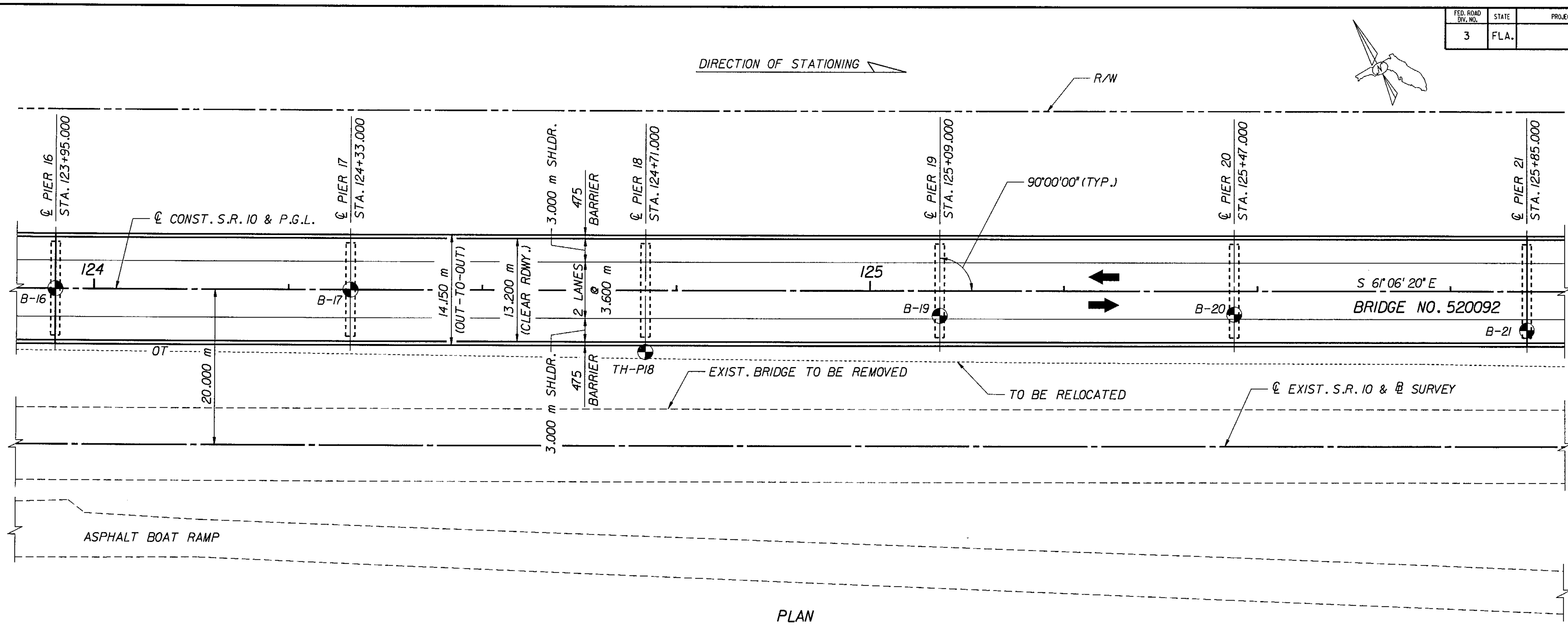
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
10	HOLMES	220784-2-52-01

SHEET TITLE:
PLAN AND ELEVATION
(SHEET 3 OF 5)

PROJECT NAME:
S.R. 10 OVER CHOCTAWHATCHEE RIVER

Drawing No.

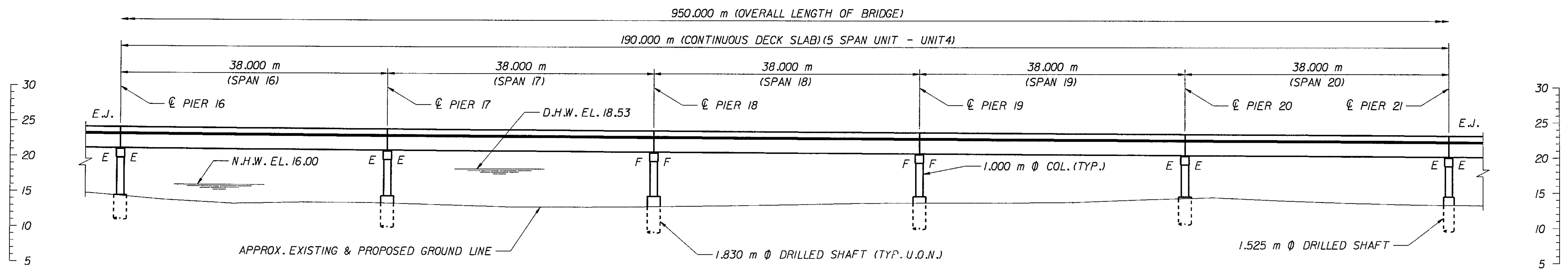
Index No.



PLAN

NOTES:

1. FOR NOTES, SEE SHEET B-8.
2. FOR GEOMETRIC DATA, SEE SHEET B-13.



ELEVATION

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	JSR	3/99
Checked by	DET	3/99
Designed by	DLC	3/99
Checked by	PRS	3/99
Approved by	JWB	

ENGINEER OF RECORD:
HDR ENGINEERING, INC.
5100 W. Kennedy Boulevard
Suite No. 300
Tampa, Florida 33609-1840

LOGO:
HDR
HDR Engineering, Inc.
Tampa, Florida

SEAL:
Wayne Bailey
3-26-99

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
10	HOLMES	220784-2-52-01

SHEET TITLE:
PLAN AND ELEVATION
(SHEET 4 OF 5)

PROJECT NAME:
S.R. 10 OVER CHOCTAWHATCHEE RIVER

Drawing No.

Index No.

Fri Mar 26 09:19:54 1999
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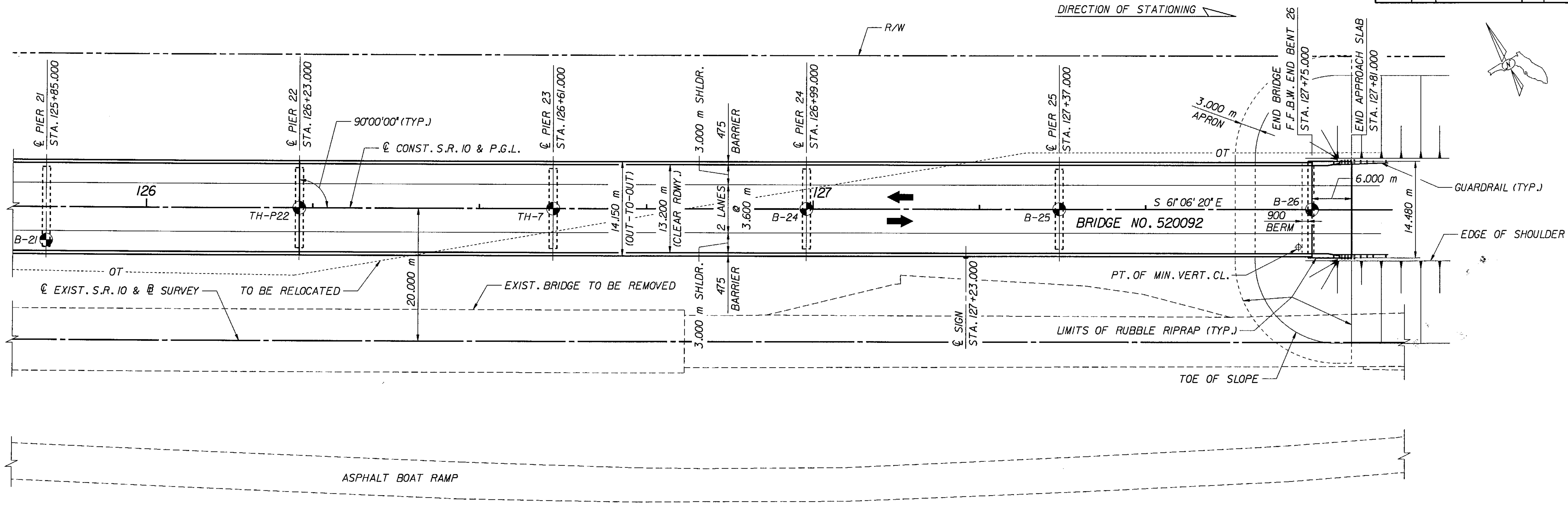
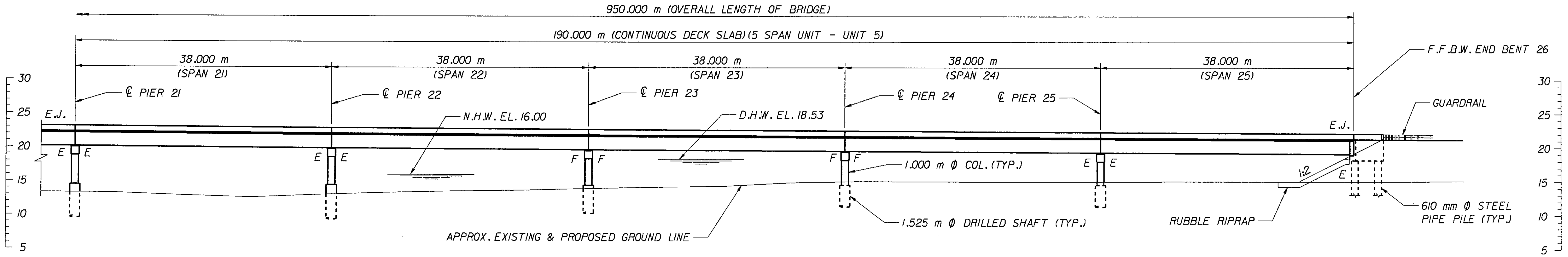
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-12



PLAN

NOTES:

1. FOR NOTES, SEE SHEET B-8.
2. FOR GEOMETRIC DATA, SEE SHEET B-13.



ELEVATION

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	JSR	3/99
Checked by	DET	3/99
Designed by	DLC	3/99
Checked by	PRS	3/99
Approved by	JWB	

ENGINEER OF RECORD:
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FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
10	HOLMES	220784-2-52-01

SHEET TITLE:
PLAN AND ELEVATION
(SHEET 5 OF 5)

PROJECT NAME:
S.R. 10 OVER CHOCTAWHATCHEE RIVER

Drawing No.

Index No.

Fri Mar 26 09:20:07 1999
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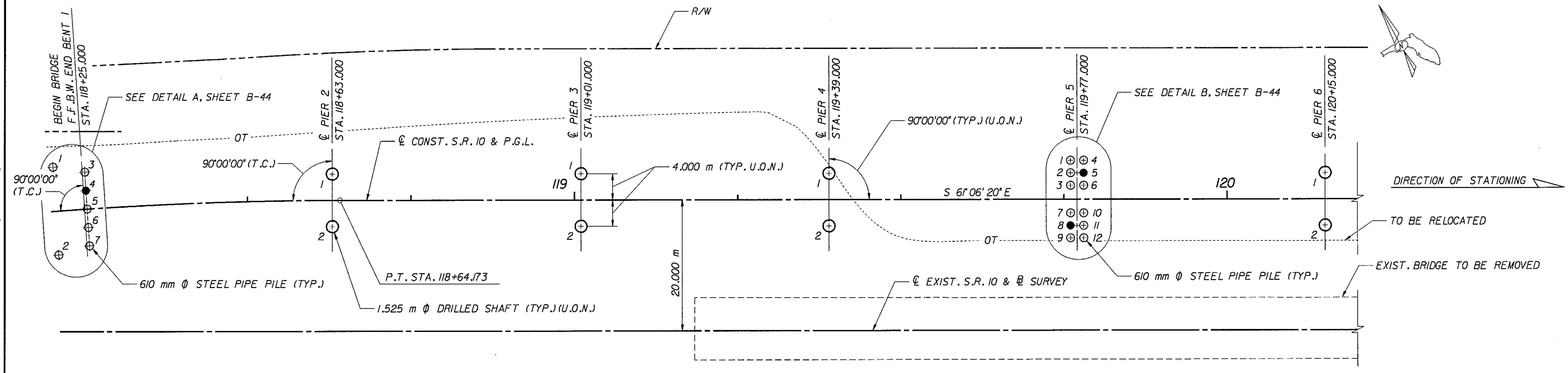
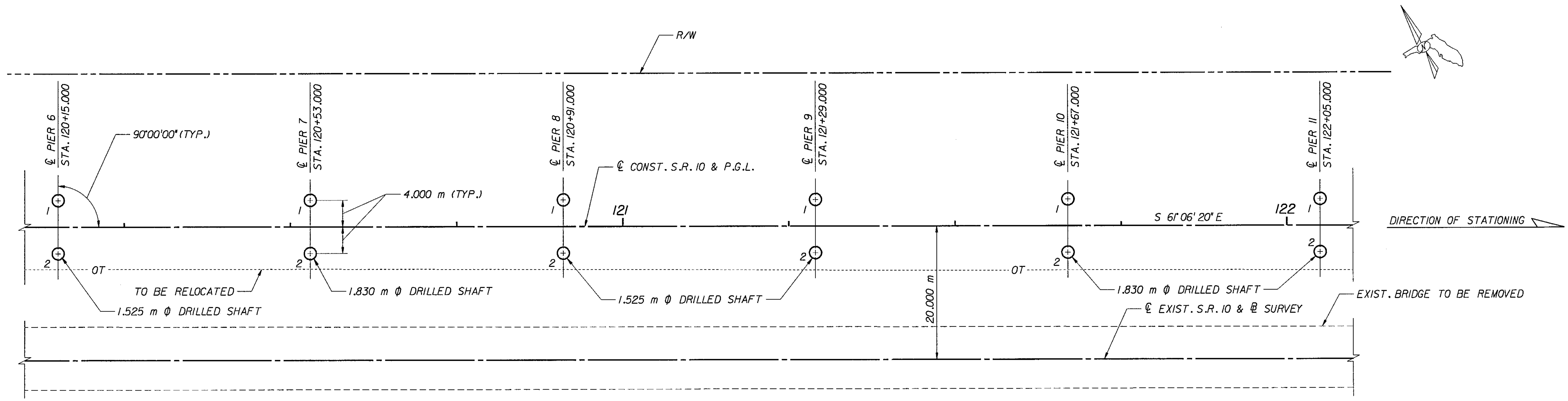
Fri Mar 26 09:21:06 1999

x:\sr10\bridge\sbfdla01.dgn



PLAN



PLAN

NOTES:

1. FOR NOTES, STEEL PIPE PILE & DRILLED SHAFT DATA, SEE SHEETS B-45 AND B-46.
2. FOR HORIZONTAL CURVE DATA, SEE SHEET B-13.
3. ● DENOTES STEEL PIPE TEST PILE.

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	JSR	3/99
Checked by	DET	3/99
Designed by	DLC	3/99
Checked by	GET	3/99
Approved by	JWB	

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FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
10	HOLMES	220784-2-52-01

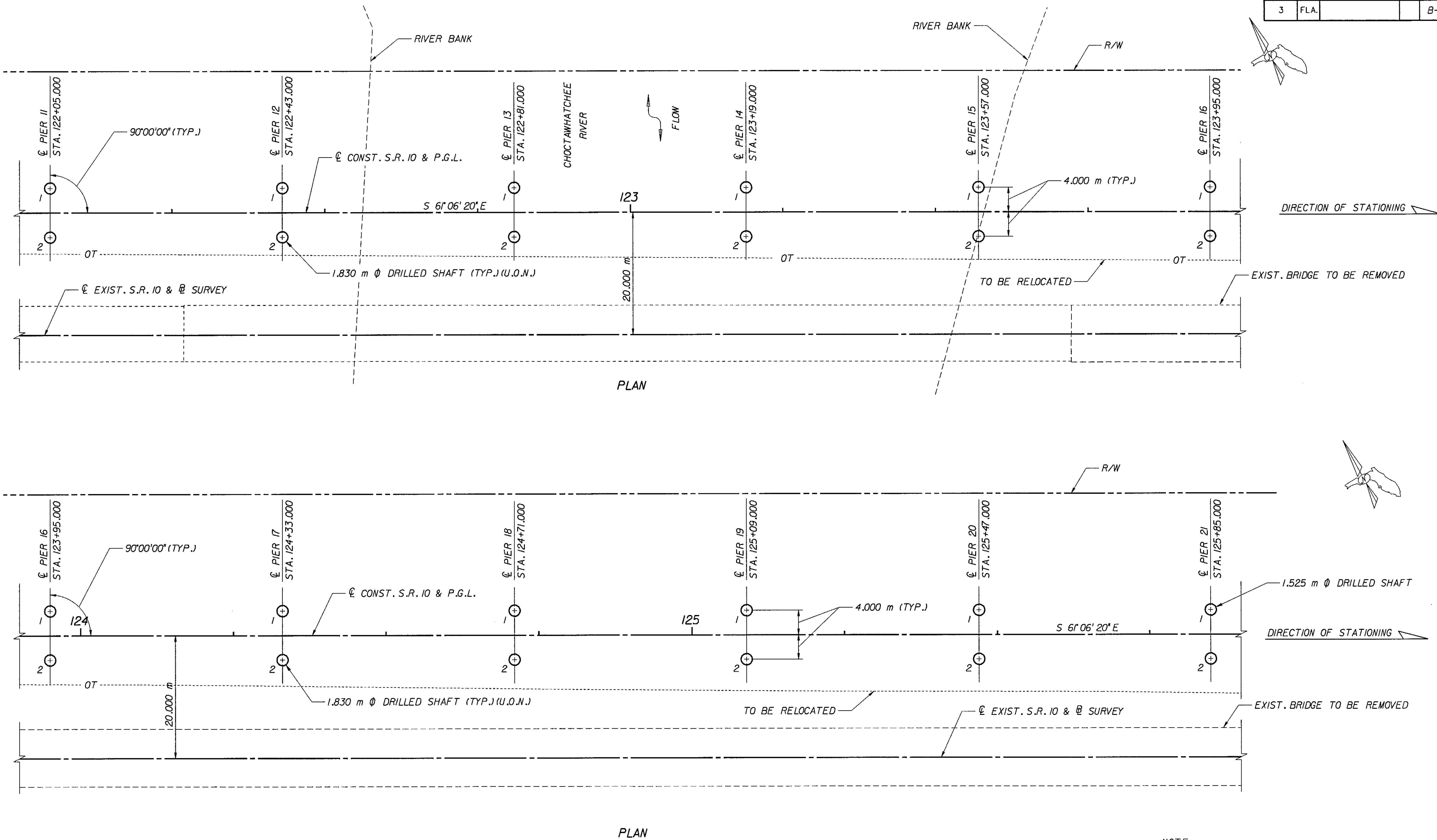
SHEET TITLE: FOUNDATION LAYOUT (SHEET 1 OF 3)

PROJECT NAME: S.R. 10 OVER CHOCTAWHATCHEE RIVER

Drawing No.

Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-43



NOTE:

FOR NOTES, STEEL PIPE PILE & DRILLED SHAFT DATA, SEE SHEETS B-45 AND B-46.

REVISIONS							Names	Dates
Date	By	Description	Date	By	Description	Drawn by	JSR	3/99
						Checked by	DET	3/99
						Designed by	DLC	3/99
						Checked by	GET	3/99
						Approved by	JWB	

ENGINEER OF RECORD:
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FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
10	HOLMES	220784-2-52-01

SHEET TITLE: FOUNDATION LAYOUT (SHEET 2 OF 3)

PROJECT NAME: S.R. 10 OVER CHOCTAWHATCHEE RIVER

Drawing No.

Index No.

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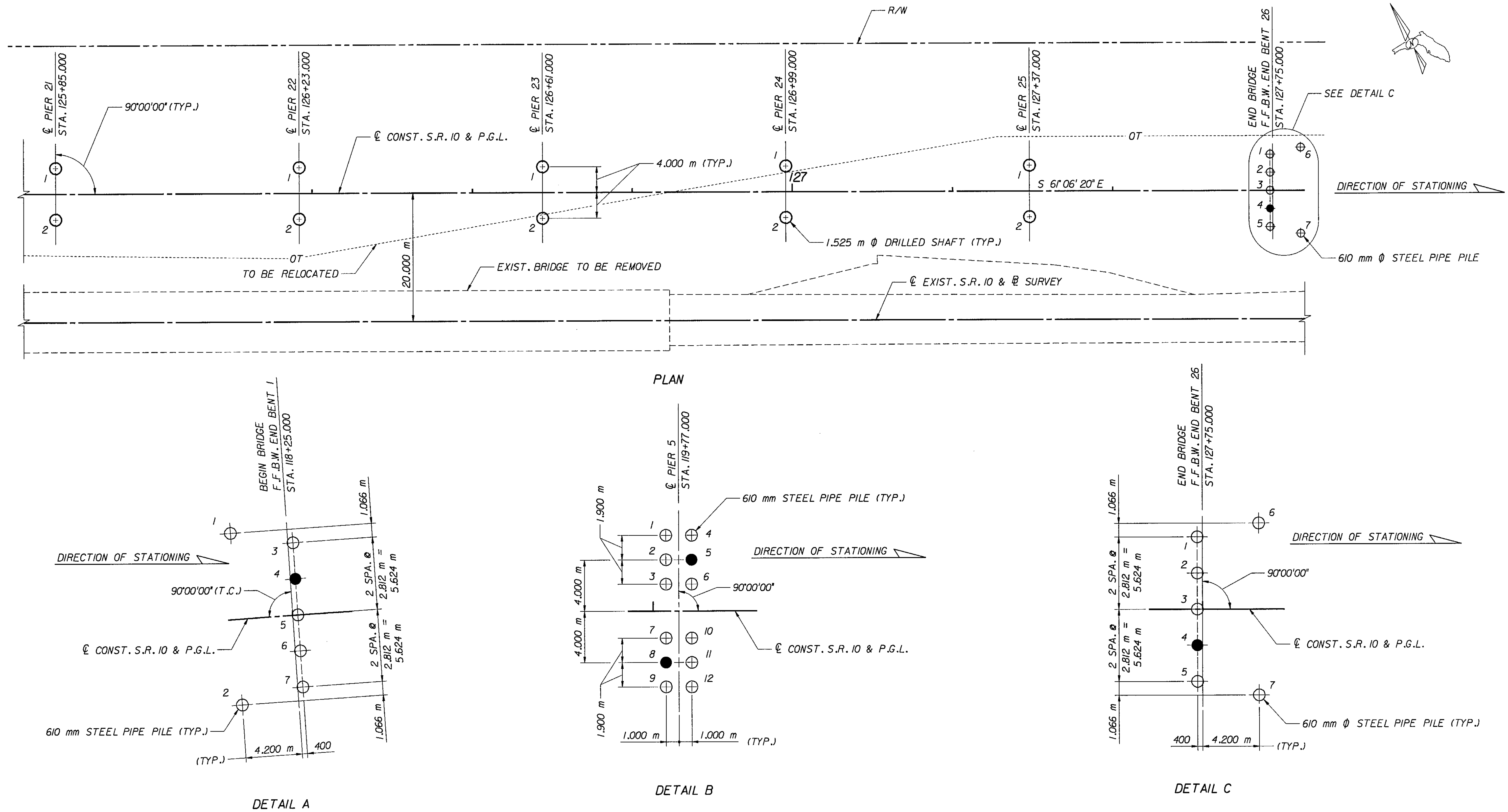
Fri Mar 26 09:21:18 1999

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-44



NOTES:

1. FOR NOTES, STEEL PIPE PILE & DRILLED SHAFT DATA, SEE SHEETS B-45 AND B-46.
2. ● DENOTES STEEL PIPE TEST PILE.

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	JSR	3/99
Checked by	DET	3/99
Designed by	DLC	3/99
Checked by	GET	3/99
Approved by	JWB	

ENGINEER OF RECORD:
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LOGO: HDR
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Tampa, Florida

SEAL: Wayman Bally 3-26-99

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
10	HOLMES	220784-2-52-01

SHEET TITLE: FOUNDATION LAYOUT (SHEET 3 OF 3)

PROJECT NAME: S.R. 10 OVER CHOCTAWHATCHEE RIVER

Drawing No.

Index No.

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Fri Mar 26 09:21:41 1999

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FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-45

DRILLED SHAFT DATA TABLE

INSTALLATION CRITERIA						DESIGN CRITERIA				
PIER	SHAFT SIZE (mm)	SHAFT TIP ELEVATION (m)	BEGIN SOCKET ELEVATION (m)	ROCK SOCKET LENGTH (m)	ANTICIPATED CASING TIP ELEVATION (m)	FACTORED DESIGN LOAD (kN)	DOWNDRAG (kN)	LONG TERM SCOUR ELEVATION (m)	100-YEAR SCOUR ELEVATION (m)	ϕ
2	1525	-9.50	+0.30	9.80	+3.30	7700	N/A	10.1	10.1	0.55
3	1525	-4.00	+1.52	5.52	+1.90	7700	N/A	10.1	10.1	0.55
4	1525	-6.00	-0.91	5.09	-0.48	7700	N/A	10.1	10.1	0.55
6	1525	-7.50	+4.88	12.38	+7.19	7700	N/A	10.1	10.1	0.55
7	1830	-7.00	-1.52	5.48	-1.52	7700	N/A	10.1	10.1	0.55
8	1525	-2.00	+6.10	8.10	+6.28	7700	N/A	10.1	10.1	0.55
9	1525	-6.00	+3.05	9.05	+3.08	7700	N/A	10.1	10.1	0.55
10	1830	-11.00	-2.74	8.26	+9.08	7700	N/A	1.6	1.6	0.55
11	1830	-10.00	-2.74	7.26	+3.93	7700	N/A	1.6	1.6	0.55
12	1830	-5.00	+1.52	6.52	+5.19	7700	N/A	1.6	1.6	0.55
13	1830	-7.00	-2.44	4.56	-0.64	7700	N/A	1.6	1.6	0.55
14	1830	-6.00	0.00	6.00	+1.86	7700	N/A	1.6	1.6	0.55
15	1830	-12.50	-3.35	9.15	-3.35	7700	N/A	1.6	1.6	0.55
16	1830	-11.50	-5.49	6.01	-3.59	7700	N/A	1.6	1.6	0.55
17	1830	-12.50	-7.32	5.18	-7.32	7700	N/A	1.6	1.6	0.55
18	1830	-6.50	+5.18	11.68	+5.35	7700	N/A	10.1	10.1	0.55
19	1830	-20.50	-0.30	20.20	-0.30	7700	N/A	10.1	10.1	0.55
20	1830	-9.00	-5.49	3.51	-5.00	7700	N/A	10.1	10.1	0.55
21	1525	-7.00	+6.10	13.10	+7.88	7700	N/A	10.1	10.1	0.55
22	1525	-9.00	+6.40	15.40	+6.50	7700	N/A	10.1	10.1	0.55
23	1525	-3.50	+5.49	8.99	+6.42	7700	N/A	10.1	10.1	0.55
24	1525	-7.50	+2.74	10.24	+2.87	7700	N/A	10.1	10.1	0.55
25	1525	-9.00	+1.22	10.22	+1.28	7700	N/A	10.1	10.1	0.55

TOP OF DRILLED SHAFT ELEVATIONS

PIER	ELEVATION
2	14.150
3	14.150
4	14.150
6	14.150
7	14.150
8	14.150
9	14.150
10	14.150
11	14.150
12	16.600
13	16.600
14	16.600
15	16.600
16	14.500
17	14.500
18	14.500
19	14.500
20	14.500
21	14.500
22	14.500
23	14.500
24	14.500
25	14.500

NOTES:

1. THE ROCK SOCKET LENGTH, NOTED ABOVE, IS THE DISTANCE FROM THE BEGIN ROCK SOCKET ELEVATION TO THE BOTTOM OF THE EXCAVATION. CASING SHALL NOT EXTEND BEYOND THE BEGIN SOCKET ELEVATION WITHOUT ADJUSTMENTS TO SHAFT TIP ELEVATION. FIELD ADJUSTMENTS TO THE SHAFT TIP ELEVATION OR ROCK SOCKET LENGTH SHALL BE APPROVED BY THE DISTRICT GEOTECHNICAL ENGINEER.

2. OUT OF POSITION, REINFORCED TEST SHAFTS SHALL BE CONSTRUCTED AT THE CENTERLINE OF CONSTRUCTION AT PIERS 2, 13 AND 19. TEST SHAFTS SHALL BE CONSTRUCTED IN THE SAME MANNER AND TO THE SAME REQUIREMENTS AS PRODUCTION SHAFTS. SEE ALSO SHEETS LT-1 THRU LT-5 FOR ADDITIONAL DETAILS.

3. THE DEPARTMENT'S SHAFT INSPECTION DEVICE (SID) SHALL BE USED TO INSPECT THE BOTTOMS OF THE DRILLED SHAFTS. ALL COSTS RELATED TO THE INSPECTION DEVICE SHALL BE INCLUDED IN THE CONTRACT UNIT PRICE FOR BID ITEM NUMBERS 2455-88-6 (1525 mm ϕ DRILLED SHAFT) AND 2455-88-7 (1830 mm ϕ DRILLED SHAFT).

4. A SLUMP LOSS TEST FOR THE DRILLED SHAFT CONCRETE MIX SHALL BE PERFORMED AT THE JOB SITE IN ACCORDANCE WITH SECTION 346 OF THE SPECIFICATIONS.

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	DET	3/99
Checked by	JWB	3/99
Designed by	DLC	3/99
Checked by	GET	3/99
Approved by	JWB	

ENGINEER OF RECORD:
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LOGO: HDR
HDR Engineering, Inc.
Tampa, Florida

SEAL: Wayne Bell
3-26-99

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
10	HOLMES	220784-2-52-01

SHEET TITLE: DRILLED SHAFT DATA SHEET

PROJECT NAME: S.R. 10 OVER CHOCTAWHATCHEE RIVER

Drawing No.

Index No.

Fri Mar 26 09:22:23 1999

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DRILLED SHAFT GENERAL NOTES

1. IT IS ANTICIPATED A MINERAL SLURRY WILL BE REQUIRED TO MAINTAIN STABILITY OF THE EXCAVATION. THEREFORE DESANDING EQUIPMENT WITH HOLDING TANKS SHALL BE PROVIDED TO CLEAN THE SLURRY FOR REUSE. THE DESANDING EQUIPMENT SUPPLIED BY THE CONTRACTOR SHALL BE EQUIPPED WITH SUITABLE SCREEN AND CYCLONE SEPARATORS AND SHALL HAVE ADEQUATE CAPACITY TO DESAND THE VOLUMES OF SLURRY REQUIRED FOR THE PROJECT.

2. SPECIAL PRECAUTIONS SHALL BE TAKEN BY THE CONTRACTOR TO PREVENT DISCHARGE OF SLURRY OR EXCAVATED MATERIALS INTO THE RIVER OR RIVER FLOOD PLAIN DURING THE EXCAVATION OPERATION. MATERIALS EXCAVATED FROM THE DRILLED SHAFTS SHALL BE COLLECTED AND DISPOSED OF SATISFACTORILY IN OFFSITE UPLAND AREAS PROVIDED BY THE CONTRACTOR AND APPROVED BY THE ENGINEER.

3. SPECIAL PRECAUTIONS SHALL BE TAKEN BY THE CONTRACTOR TO PREVENT DISCHARGE OF SLURRY OR CONCRETE INTO THE RIVER OR RIVER FLOOD PLAIN DURING THE CONCRETE PLACING OPERATION. THE CONTRACTOR SHALL PROVIDE A SUITABLE RECEPTACLE AROUND THE TOP OF THE CASING TO COLLECT SLURRY AND/OR CONCRETE DISCHARGED FROM THE SHAFT DURING THE PLACEMENT OF CONCRETE.

4. PRELIMINARY DRILLED SHAFT TIP ELEVATIONS ARE SHOWN IN THE DRILLED SHAFT DATA TABLE ON SHEET B-45. FINAL DRILLED SHAFT TIP ELEVATIONS MAY BE MODIFIED AFTER THE RESULTS OF THE LOAD TEST PROGRAM AND OTHER SOIL INVESTIGATIONS AND THEY MAY VARY SIGNIFICANTLY FROM THOSE SHOWN IN THE DRILLED SHAFT DATA TABLE.

5. THE SPACING OF THE VERTICAL COLUMN REINFORCING BARS AT THE TOP OF THE DRILLED SHAFT SHALL BE SET WITH A TEMPLATE.

6. DRILLED SHAFTS SHALL BE PAID FOR AT THE CONTRACT UNIT PRICE PER LINEAR METER FOR DRILLED SHAFTS UNDER PAY ITEM NUMBERS 2455-88-6 (1525 mm Ø DRILLED SHAFT) AND 2455-88-7 (1830 mm Ø DRILLED SHAFT). COMPENSATION FOR ADDITIONAL COST FOR MATERIALS AND CONSTRUCTION OF DRILLED SHAFTS SHALL BE PAID FOR UNDER THE APPROPRIATE ITEMS. SEE SUMMARY OF QUANTITIES AND SPECIFICATIONS.

7. CONCRETE FOR DRILLED SHAFTS SHALL BE CLASS IV (DRILLED SHAFTS). CONCRETE SHALL BE POURED CONTINUOUSLY FROM THE TIP ELEVATION TO THE TOP OF DRILLED SHAFT ELEVATION.

8. REINFORCING STEEL SHALL CONFORM TO ASTM A615M, GRADE 420.

9. THE COST OF FURNISHING AND INSTALLATION OF MECHANICAL COUPLERS SHALL BE INCLUDED IN THE CONTRACT UNIT PRICE FOR BID ITEM NUMBERS 2455-88-6 (1525 mm Ø DRILLED SHAFT) AND 2455-88-7 (1830 mm Ø DRILLED SHAFT).

10. ALL MECHANICAL REINFORCING CONNECTIONS SHALL DEVELOP AT LEAST 125 PERCENT OF THE SPECIFIED YIELD STRENGTH OF THE BAR. MECHANICAL SPLICES SHALL BE SELECTED FROM THE DEPARTMENT'S QUALIFIED PRODUCTS LIST. MECHANICAL COUPLERS FOR THE VERTICAL BARS SHALL BE STAGGERED 600 mm.

11. PERMANENT CASING SHALL BE PAID FOR PER LINEAR METER FOR CASING UNDER PAY ITEM NUMBERS 2455-107-6 (1525 mm Ø CASING) AND 2455-107-7 (1830 mm Ø CASING). PAYMENT LENGTH FOR PERMANENT CASING SHALL BE MEASURED FROM THE ANTICIPATED CASING TIP ELEVATION AS SHOWN IN THE DRILLED SHAFT DATA TABLE ON SHEET B-45 TO THE TOP OF SHAFT ELEVATION.

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	DET	3/99
Checked by	JWB	3/99
Designed by	JVD	3/99
Checked by	JWB	3/99
Approved by	JWB	

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FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
10	HOLMES	220784-2-52-01

SHEET TITLE: DRILLED SHAFT GENERAL NOTES

PROJECT NAME: S.R. 10 OVER CHOCTAWHATCHEE RIVER

Drawing No.

Index No.

Fri Mar 26 09:22:35 1999

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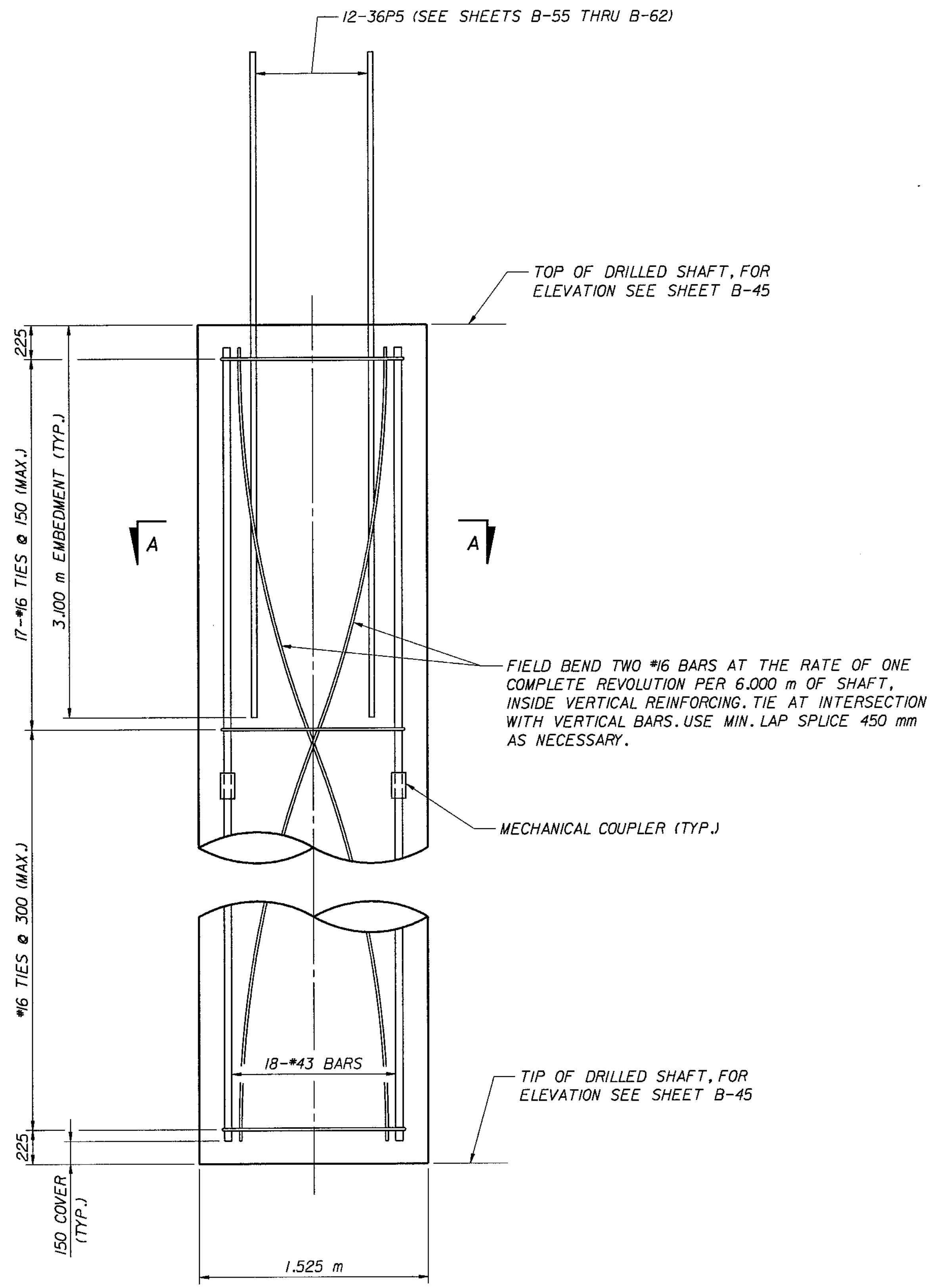
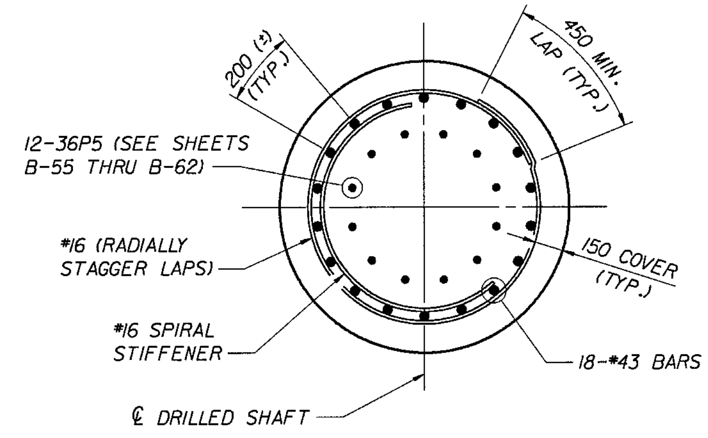


1.525 m DRILLED SHAFT ELEVATION



SECTION A-A

DRILLED SHAFT QUANTITIES (PER LIN. METER) *		
ITEM	UNIT	QUANTITY
CLASS IV CONCRETE (DRILLED SHAFT)	m^3	1.83
REINFORCING STEEL (SUBSTRUCTURE)	kg	227

* QUANTITIES SHOWN ARE FOR ESTIMATING PURPOSES ONLY.

ESTIMATED QUANTITIES		
ITEM	UNIT	QUANTITY
DRILLED SHAFT (1525 mm DIA.)	m	456.8
UNCLASSIFIED SHAFT EXCAVATION (1525 mm DIA.)	m	447.0
UNCLASSIFIED EXTRA DEPTH EXCAVATION (1525 mm DIA.)	m	64.7
DRILLED SHAFT SIDEWALL OVERREAMING	m	21.6
CORE (SHAFT EXCAVATION)	m	281.8
CASING (1525 mm DIA.)	m	222.4

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	DET	3/99
Checked by	JVD	3/99
Designed by	DLC	3/99
Checked by	GET	3/99
Approved by	JWB	

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Tampa, Florida

SEAL:
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3-26-99

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

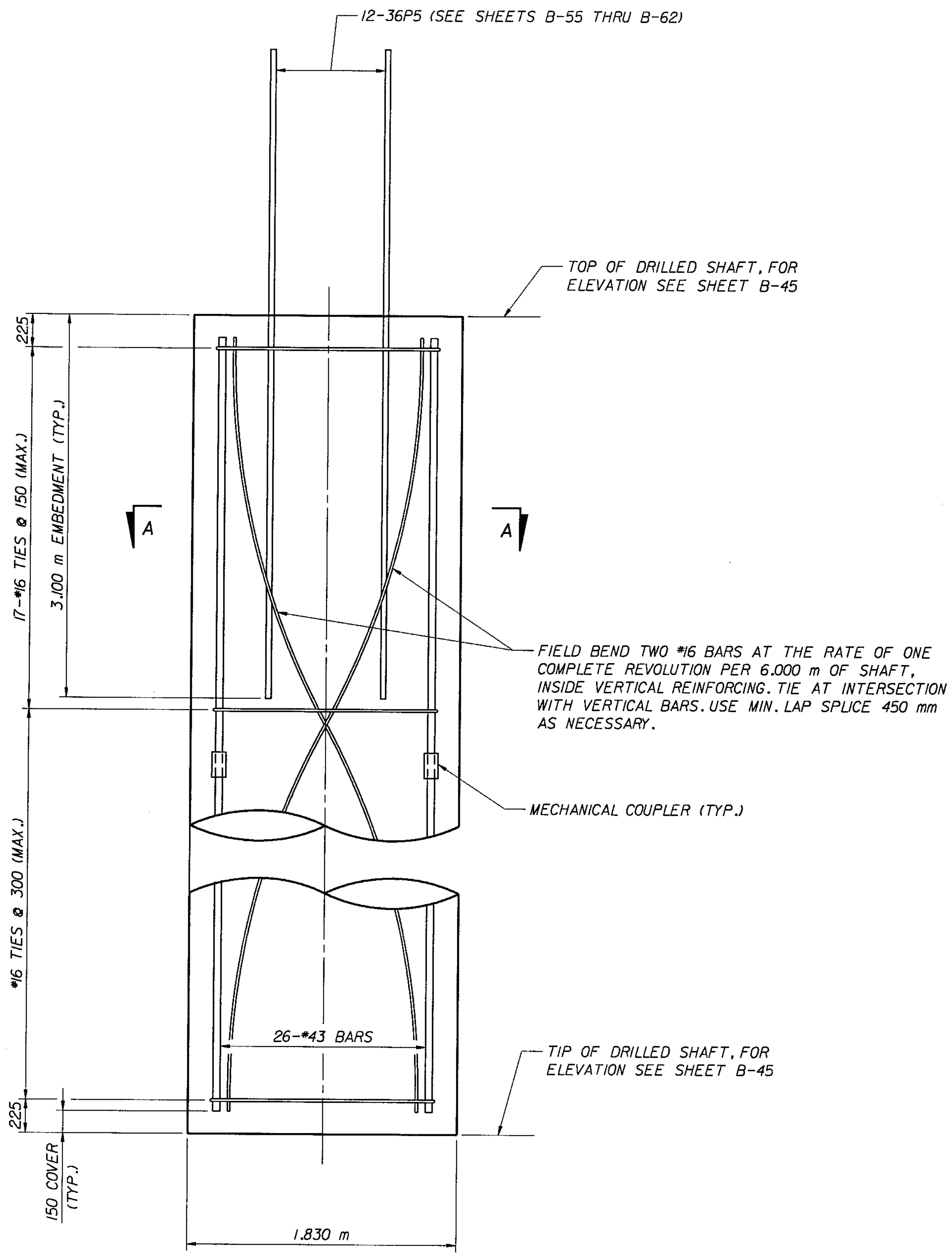
ROAD NO.	COUNTY	FINANCIAL PROJECT ID
10	HOLMES	220784-2-52-01

SHEET TITLE: 1.525 m Ø DRILLED SHAFT

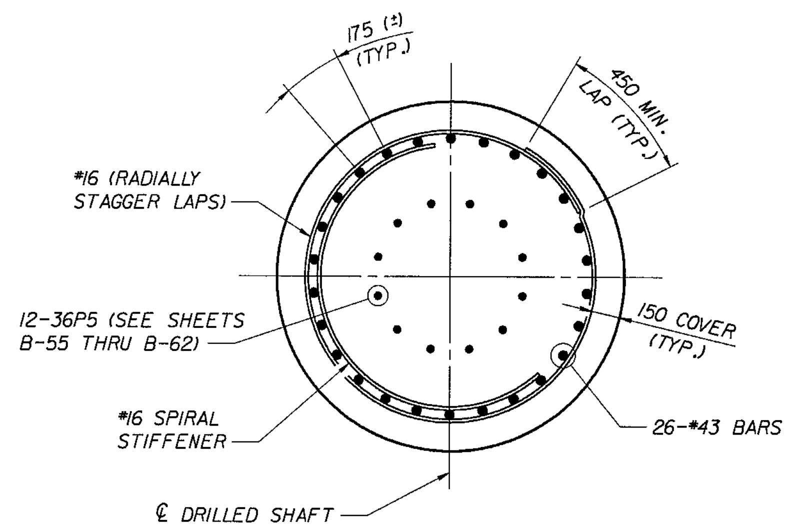
PROJECT NAME: S.R. 10 OVER CHOCTAWHATCHEE RIVER

Drawing No.

Index No.



1.830 m DRILLED SHAFT ELEVATION



SECTION A-A

DRILLED SHAFT QUANTITIES (PER LIN. METER) *		
ITEM	UNIT	QUANTITY
CLASS IV CONCRETE (DRILLED SHAFT)	m^3	2.63
REINFORCING STEEL (SUBSTRUCTURE)	kg	323

* QUANTITIES SHOWN ARE FOR ESTIMATING PURPOSES ONLY.

ESTIMATED QUANTITIES		
ITEM	UNIT	QUANTITY
DRILLED SHAFT (1830 mm DIA.)	m	599.7
UNCLASSIFIED SHAFT EXCAVATION (1830 mm DIA.)	m	552.6
UNCLASSIFIED EXTRA DEPTH EXCAVATION (1830 mm DIA.)	m	56.3
DRILLED SHAFT SIDEWALL OVERREAMING	m	18.7
CORE (SHAFT EXCAVATION)	m	259.6
CASING (1830 mm DIA.)	m	355.3

W.P.I. NO. 3111042
W.P.I. NO. 3116849

FISCAL YEAR	SHEET NO.
95	1

THIS CONTRACT PLAN SET INCLUDES:
ROADWAY PLANS
SUMMARY OF PAY ITEMS (7 SHEETS)
SIGNING AND PAVEMENT MARKING PLANS
BRIDGE STRUCTURE PLANS

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION
FINAL PLANS
PLANS OF PROPOSED
STATE HIGHWAY

STATE PROJ. NO. 47010-3519 (FEDERAL FUNDS)
STATE PROJ. NO. 56010-3520 (FEDERAL FUNDS)

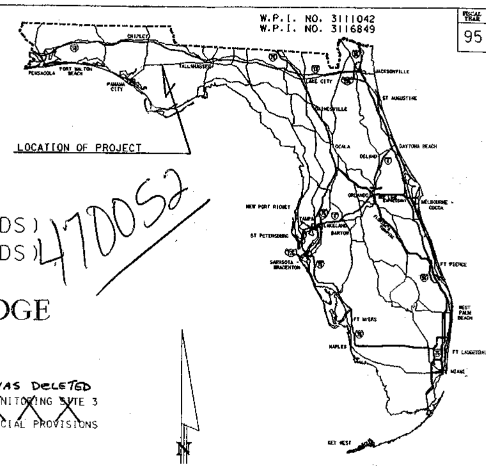
470052

CALHOUN COUNTY AND LIBERTY COUNTY

S.R. 20 APALACHICOLA RIVER BRIDGE

STRUCTURE NO. 470029 (EXISTING)
STRUCTURE NO. 470052 (NEW)

LOCATION OF PROJECT



INDEX OF ROADWAY PLANS

SHEET NO.	SHEET DESCRIPTION
1	KEY SHEET
2	OMITTED
3-8	DRAINAGE MAPS
9	TYPICAL SECTIONS
10-11	SUMMARY OF QUANTITIES
12-15	SUMMARY OF DRAINAGE STRUCTURES
16-23	ROADWAY PLANS - PROFILES
24-25	GEOMETRIC LAYOUT
26-27	DRAINAGE STRUCTURES
28-29	POND DETAIL SHEETS
30-35	DETAILS
36-53	ROADWAY SOIL SURVEY
54-87	ROADWAY CROSS SECTIONS
88	CROSS SECTION LAYOUT SHT.(RET. POND 1)
89-94	POND CROSS SECTIONS
95-100	TRAFFIC CONTROL PLANS
101-108	UTILITY ADJUSTMENT PLANS
109-110	APPROACH SLAB DETAIL
111-121	STORMWATER POLLUTION PREVENTION PLAN

END STATE PROJECT NO. 47010-3519
BEGIN STATE PROJECT NO. 56010-3520
STA. 658+80.75 ℄ CONST.
MP NO. 24.334

THIS ITEM WAS DELETED
~~INSTALL VOLUME MONITORING SITE 3~~
~~STA. 683+00~~
~~SEE TECHNICAL SPECIAL PROVISIONS~~

TO GRAND RIDGE

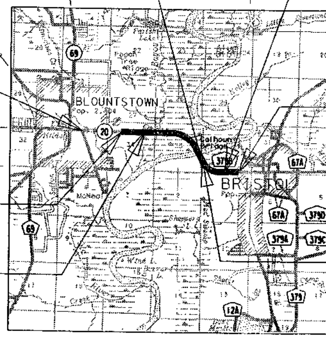
R8W | R7W

TO ALTHA

TO GREENSBORO

BEGIN CONSTRUCTION
STA. 6+05 ℄ SURVEY

TO CLARKSVILLE



END PROJECT
END STATE PROJECT NO. 56010-3520
STA. 698+82.95 ℄ CONST.
MP 25.091

T1N / T1S

TO HOSFORD

BEGIN PROJECT
BEGIN STATE PROJECT NO. 47010-3519
STA. 559+00 ℄ CONST.
MP NO. 22.444

BEGIN BRIDGE
STA. 575+18.75
℄ CONST.

END BRIDGE
STA. 658+80.75
℄ CONST.

R8W | R7W

TO WILMA

100 % PLANS
MARCH, 1995

THESE PLANS HAVE BEEN PREPARED IN ACCORDANCE WITH AND ARE GOVERNED BY THE STATE OF FLORIDA, DEPARTMENT OF TRANSPORTATION, ROADWAY AND TRAFFIC DESIGN STANDARDS (BOOKLET DATED JANUARY, 1994).

REVISIONS

BRIDGE PLAN SHEET NO. A-27, B-1, B-2, B-3, B-22, B-32, B-36, B-37, B-60, C-1, C-27, C-28, D-1, D-13, LT-15, LT-16, LT-18, LT-23, LT-24, LT-29, (REVISED 5/24/95)
CES (BRIDGE) (REVISED 5/24/95)
BRIDGE PLAN SHEET NO. B-121, B-122, B-123, B-124, B-125, B-126 (REVISED 6/8/95)
CES (BRIDGE - PAGE 1 OF 3) (REVISED 6/8/95)
ROADWAY PLANS SHEET NO. 10, 11, 28, 28A, 76, 77, 78, 87, 88, 88A, 88B, 88C, 88D, 89, 89A, 90, 90A, 91, 91A, 92, 92A, 93, 93A, 94 (REVISED 4/23/97)
ROADWAY PLANS SHEET NO. 10, 11, 14, 15, 21, 22, 27, 28, 28A, 32, 33, 76, 77, 78, 87-94 (88A, 88D - VOID) 106, 107, 119, 121 (REVISED 6/9/97)
ROADWAY PLANS SHEET NO. 9, 10, 11, 14, 15, 21, 27, 28, (34A ADDED), 35, 73-76, 87, 106, 119, 121 S-2, S-5, S-6 (REVISED 3/16/98)

CONTRACTOR: ODEBRECHT OF FLORIDA
DESIGN CONSULTANT: HNTB AND FDOT
CEI CONSULTANTANT: PBCS
DISTRICT SECRETARY: EDWARD PRESCOTT
RESIDENT ENGINEER: BRYAN ESTOCK
PROJECT ENGINEER: MARCUS KELLY
WORK ACCEPTED: 2/16/99
WORK WAS COMPLETED: 2/16/99
WORK CONDITIONALLY ACCEPTED: 2/16/99
WORK WAS ACCEPTED: 3/18/99

PLANS PREPARED BY :

HOWARD NEEDLES TAMMEN & BERGENDOFF
5850 T.G. LEE BOULEVARD
SUITE 600
ORLANDO, FLORIDA 32822
(407) 859-8380

ATTENTION IS DIRECTED TO THE FACT THAT THESE PLANS MAY HAVE BEEN REDUCED IN SIZE BY REPRODUCTION. THIS MUST BE CONSIDERED WHEN OBTAINING SCALED DATA.

GOVERNING SPECIFICATIONS: STATE OF FLORIDA, DEPARTMENT OF TRANSPORTATION, STANDARD SPECIFICATIONS, DATED 1991 AND SUPPLEMENTS THERETO IF NOTED IN THE SPECIAL PROVISIONS FOR THIS PROJECT.

LENGTH OF PROJECT

	47010-3519		56010-3520		TOTAL	
	LINEAR FT.	MILES	LINEAR FT.	MILES	LINEAR FT.	MILES
ROADWAY	1618.75	0.306	4002.20	0.758	5620.95	1.064
BRIDGES	8362.00	1.584	0.00	0.00	8362.00	1.584
NET LENGTH OF PROJ.	9980.75	1.890	4002.20	0.758	13982.95	2.648
EXCEPTIONS						
GROSS LENGTH OF PROJ.	9980.75	1.890	4002.20	0.758	13982.95	2.648

J.V. Moulton
3/1/99

DATE	BY	REVISIONS DESCRIPTION

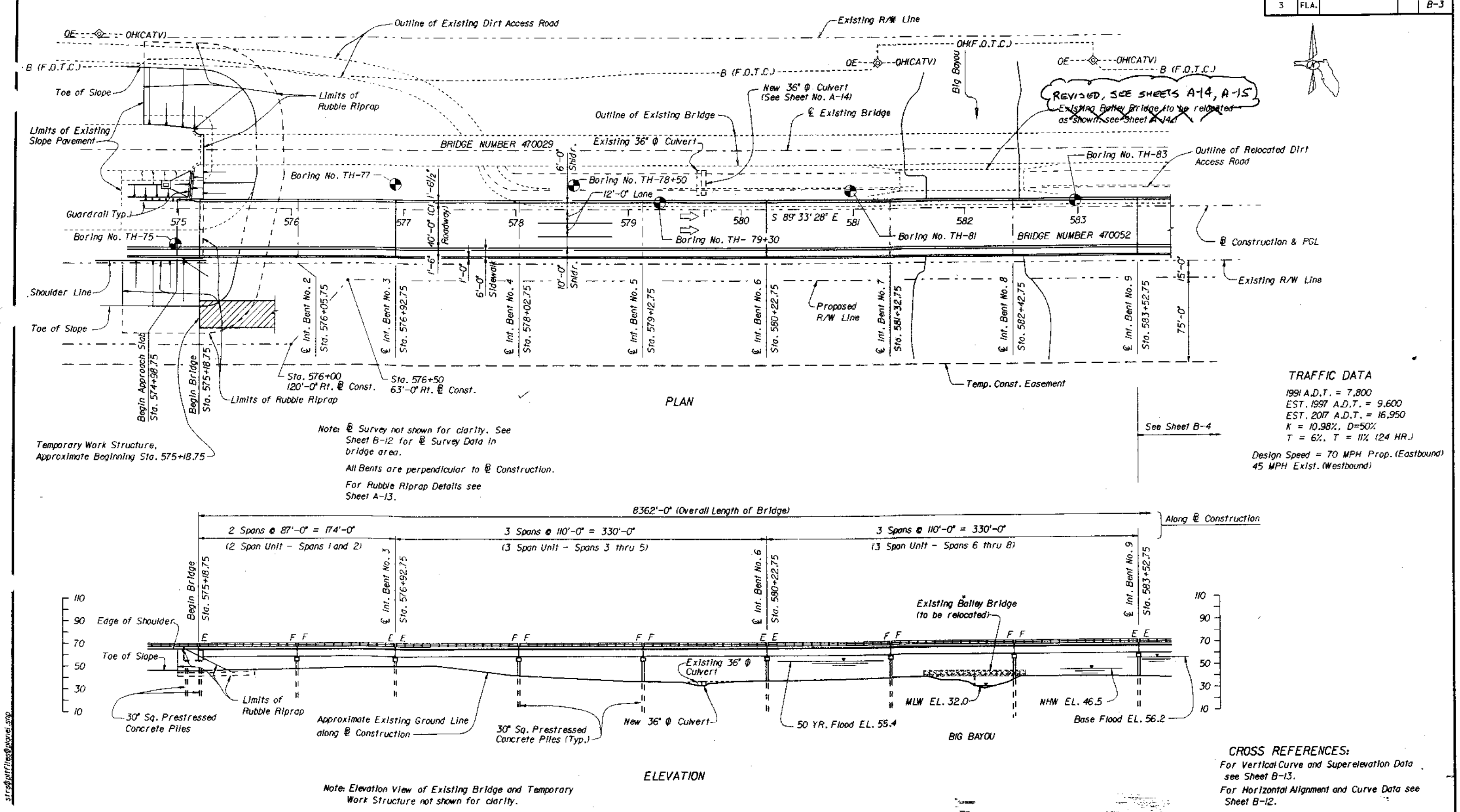
Bryan E Estock
3/2/99

5/7/98

ROADWAY PLANS APPROVED BY: YASSI MYERS, P.E.
DATE:
P.E. NO.:

FDOT PROJECT MANAGER : BRIAN A. BLANCHARD, P.E.

FISCAL ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-3



strs@ptfiles@porel.snp Wed May 24 14:37:06 1995

REVISIONS

Date	By	Description	Date	By	Description
5/24/95	JLF	Added Traffic Data			
2/19/99	MRW	FINAL AS BUILTS			

	Names	Dates
Drawn by	JLF	5-94
Checked by	CEB	2-95
Designed by	JLF	5-94
Checked by	CEB	2-95
Approved by	HTB	

ENGINEER OF RECORD:
STRUCTURES DESIGN OFFICE
CENTRAL OFFICE
605 Suwannee Street, MS 33
Tallahassee, Florida 32399-0450

LOGO:

SEAL:

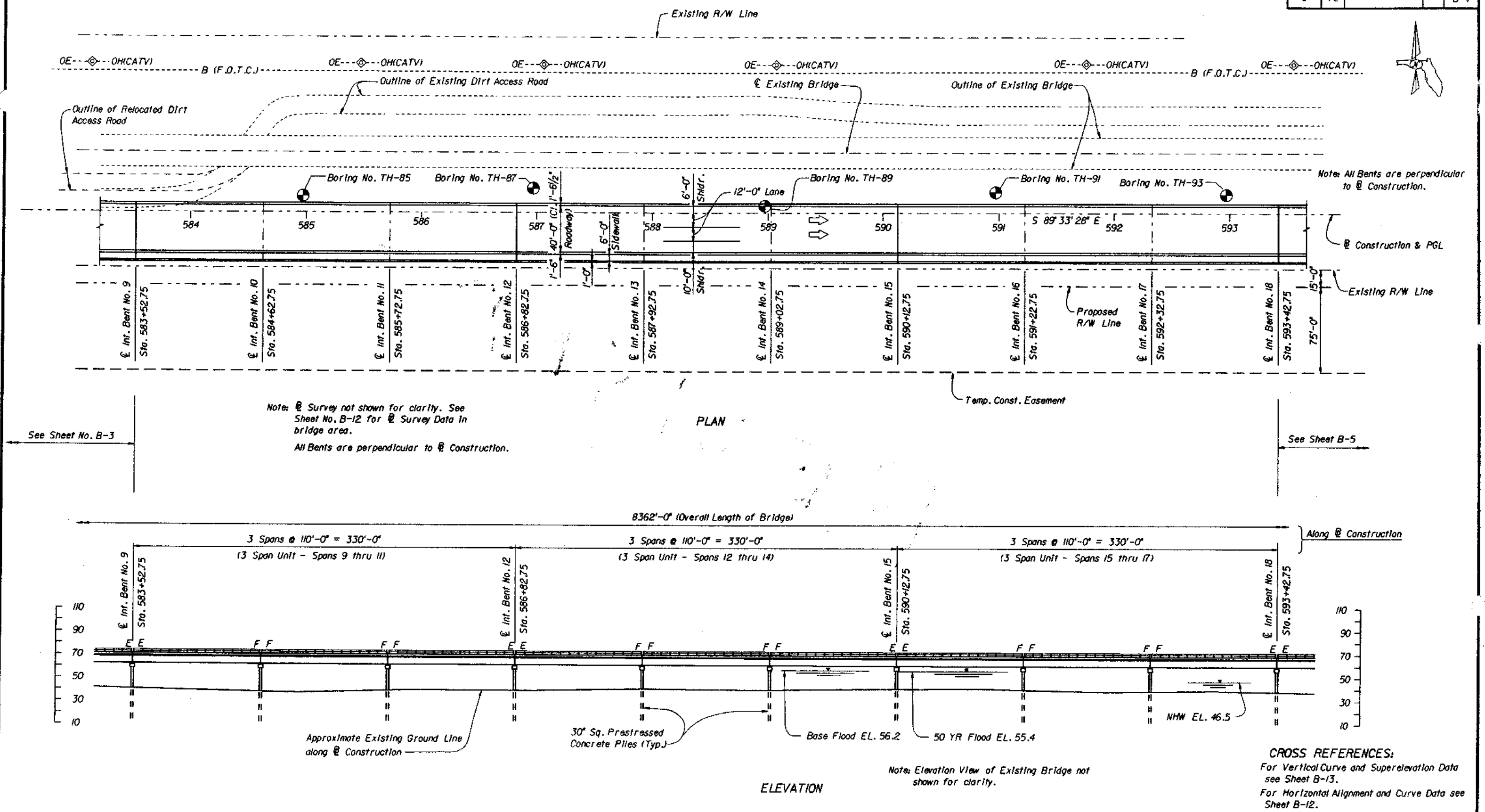
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
20	CALHOUN & LIBERTY	47010-3519 & 56010-3520

SHEET TITLE: PLAN AND ELEVATION (SHEET 1 OF 9)

PROJECT NAME: SR 20 OVER APALACHICOLA RIVER

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FL			B-4



CROSS REFERENCES:

For Vertical Curve and Superelevation Data see Sheet B-13.

For Horizontal Alignment and Curve Data see Sheet B-12.

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	JLF	5-94
Checked by	CEB	2-95
Designed by	JLF	5-94
Checked by	CEB	2-95
Approved by	HTB	

ENGINEER OF RECORD: STRUCTURES DESIGN OFFICE, CENTRAL OFFICE, 605 Suwannee Street, MS 33, Tallahassee, Florida 32399-0450

LOGO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

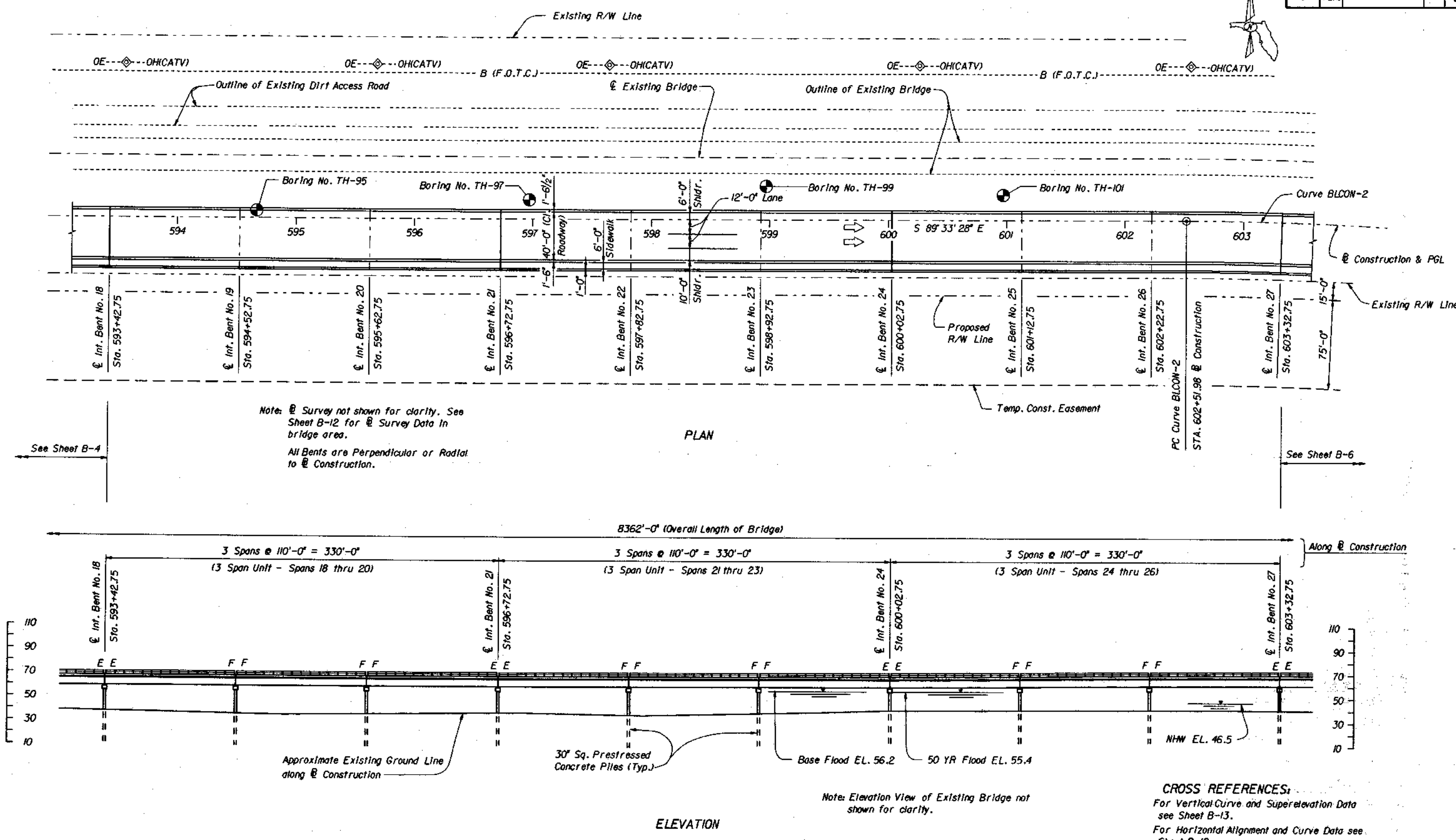
ROAD NO.	COUNTY	PROJECT NO.
20	CALHOUN & LIBERTY	47010-3519 & 56010-3520

SHEET TITLE: PLAN AND ELEVATION (SHEET 2 OF 9)

PROJECT NAME: SR 20 OVER APALACHICOLA RIVER

/usr5/proj/sr20/panel.dgn Mon Mar 13 17:32:13 1995

3	FLA.			B-5



CROSS REFERENCES:

For Vertical Curve and Superelevation Data see Sheet B-13.

For Horizontal Alignment and Curve Data see Sheet B-12.

	Names	Dates
Drawn by	JLF	5-94
Checked by	CEB	2-95
Designed by	JLF	5-94
Checked by	CEB	2-95
Approved by	HTB	

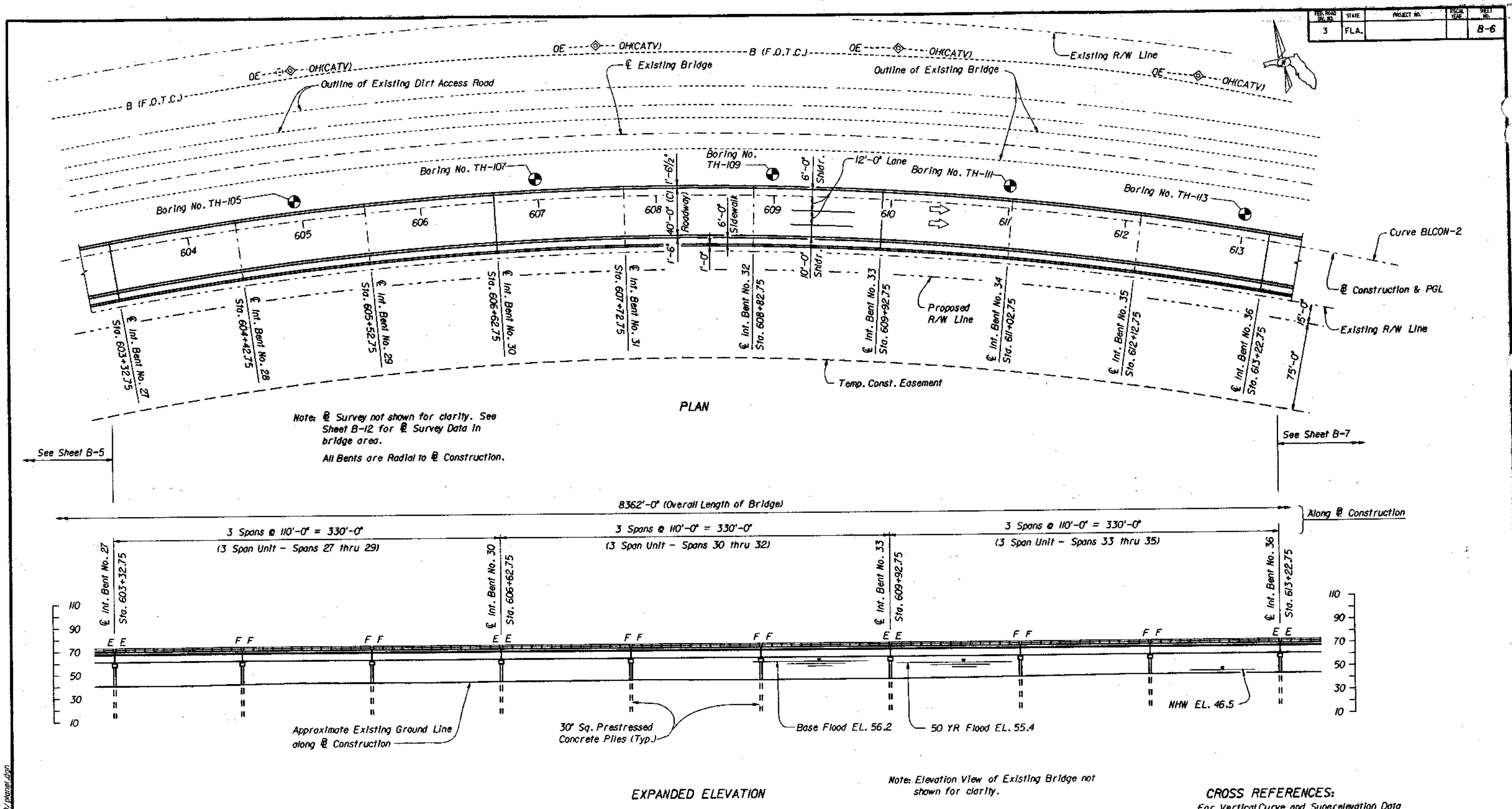
ENGINEER OF RECORD: STRUCTURES DESIGN OFFICE, CENTRAL OFFICE, 605 Suwannee Street, MS 33, Tallahassee, Florida 32399-0450

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
20	CALHOUN & LIBERTY	47010-3519 & 56010-3520

SHEET TITLE: PLAN AND ELEVATION (SHEET 3 OF 9)

PROJECT NAME: SR 20 OVER APALACHICOLA RIVER



Note: ℄ Survey not shown for clarity. See Sheet B-12 for ℄ Survey Data in bridge area.

All Bents are Radial to ℄ Construction.

Note: Elevation View of Existing Bridge not shown for clarity.

CROSS REFERENCES:

For Vertical Curve and Superelevation Data see Sheet B-13.

For Horizontal Alignment and Curve Data see Sheet B-12.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-6

	Names	Dates
Drawn by	JLF	5-94
Checked by	CEB	2-95
Designed by	JLF	5-94
Checked by	CEB	2-95
Approved by	HTB	

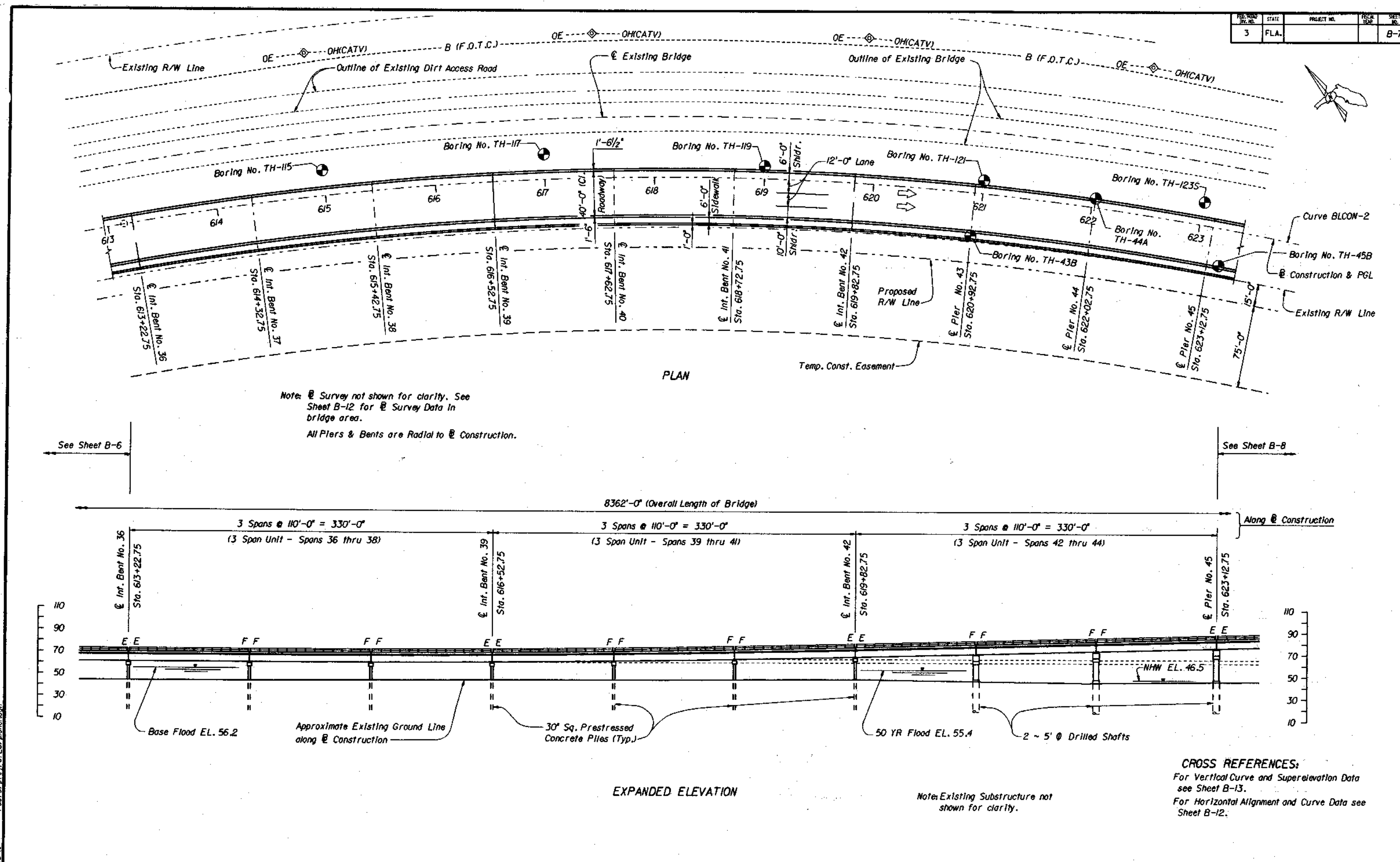
ENGINEER OF RECORD: STRUCTURES DESIGN OFFICE, CENTRAL OFFICE, 605 Suwannee Street, MS 33, Tallahassee, Florida 32399-0450

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
20	CALHOUN & LIBERTY	47010-3519 & 56010-3520

SHEET TITLE: PLAN AND ELEVATION (SHEET 4 OF 9)

PROJECT NAME: SR 20 OVER APALACHICOLA RIVER



CROSS REFERENCES:
For Vertical Curve and Superelevation Data see Sheet B-13.
For Horizontal Alignment and Curve Data see Sheet B-12.

Drawn by	JLF	1-94
Checked by	CEB	2-95
Designed by	JLF	5-94
Checked by	CEB	2-95
Approved by	HTB	

ENGINEER OF RECORD: STRUCTURES DESIGN OFFICE, CENTRAL OFFICE, 605 Suwannee Street, MS 33, Tallahassee, Florida 32399-0450

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

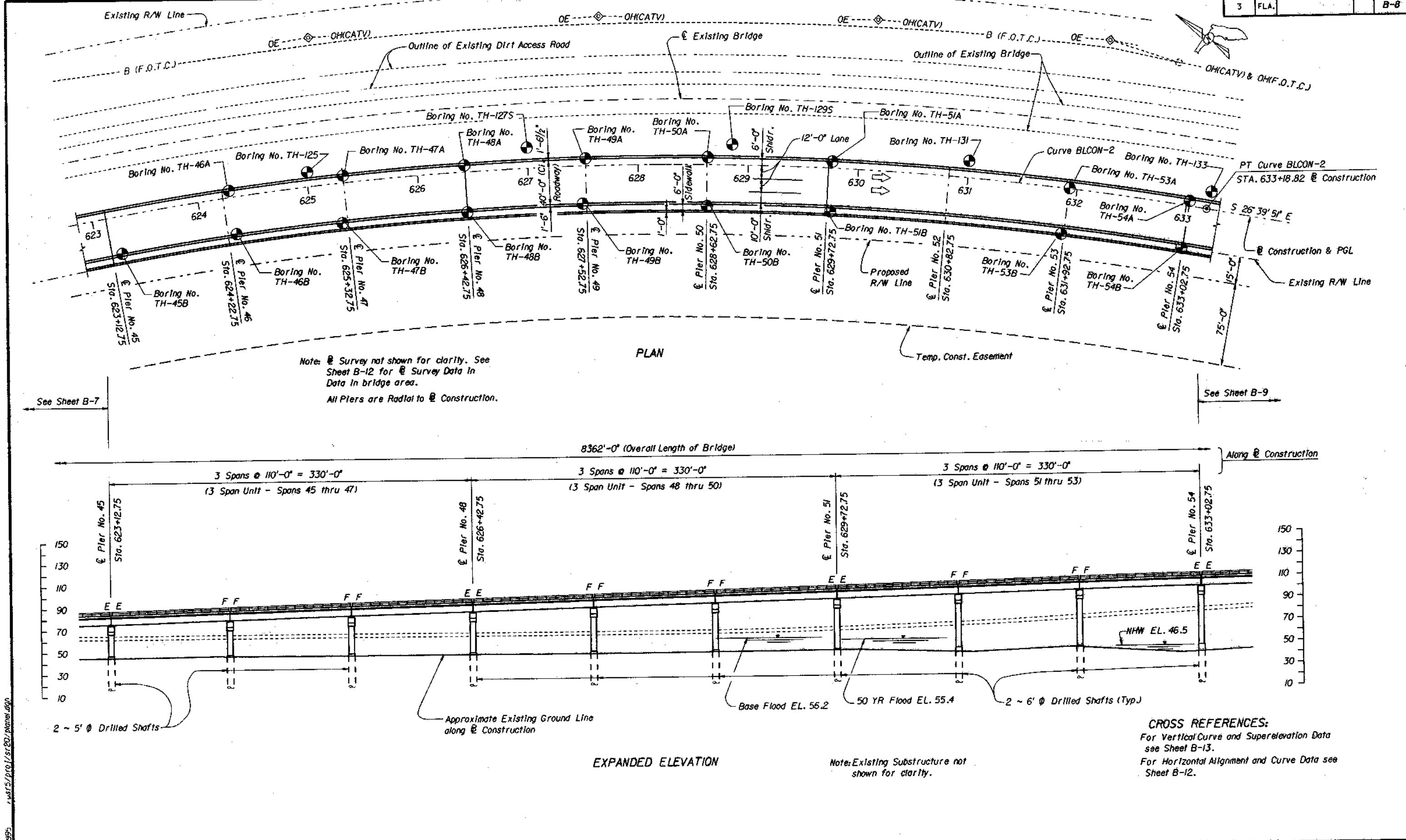
ROAD NO.	COUNTY	PROJECT NO.
20	CALHOUN & LIBERTY	47010-3519 & 56010-3520

PLAN AND ELEVATION (SHEET 5 OF 9)

SR 20 OVER APALACHICOLA RIVER

Sheet B-7

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-8



REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	JLF	5-94
Checked by	CEB	2-95
Designed by	JLF	5-94
Checked by	CEB	2-95
Approved by	HTB	

ENGINEER OF RECORD:
STRUCTURES DESIGN OFFICE
CENTRAL OFFICE
605 Suwannee Street, MS 33
Tallahassee, Florida 32399-0450

LOGO:

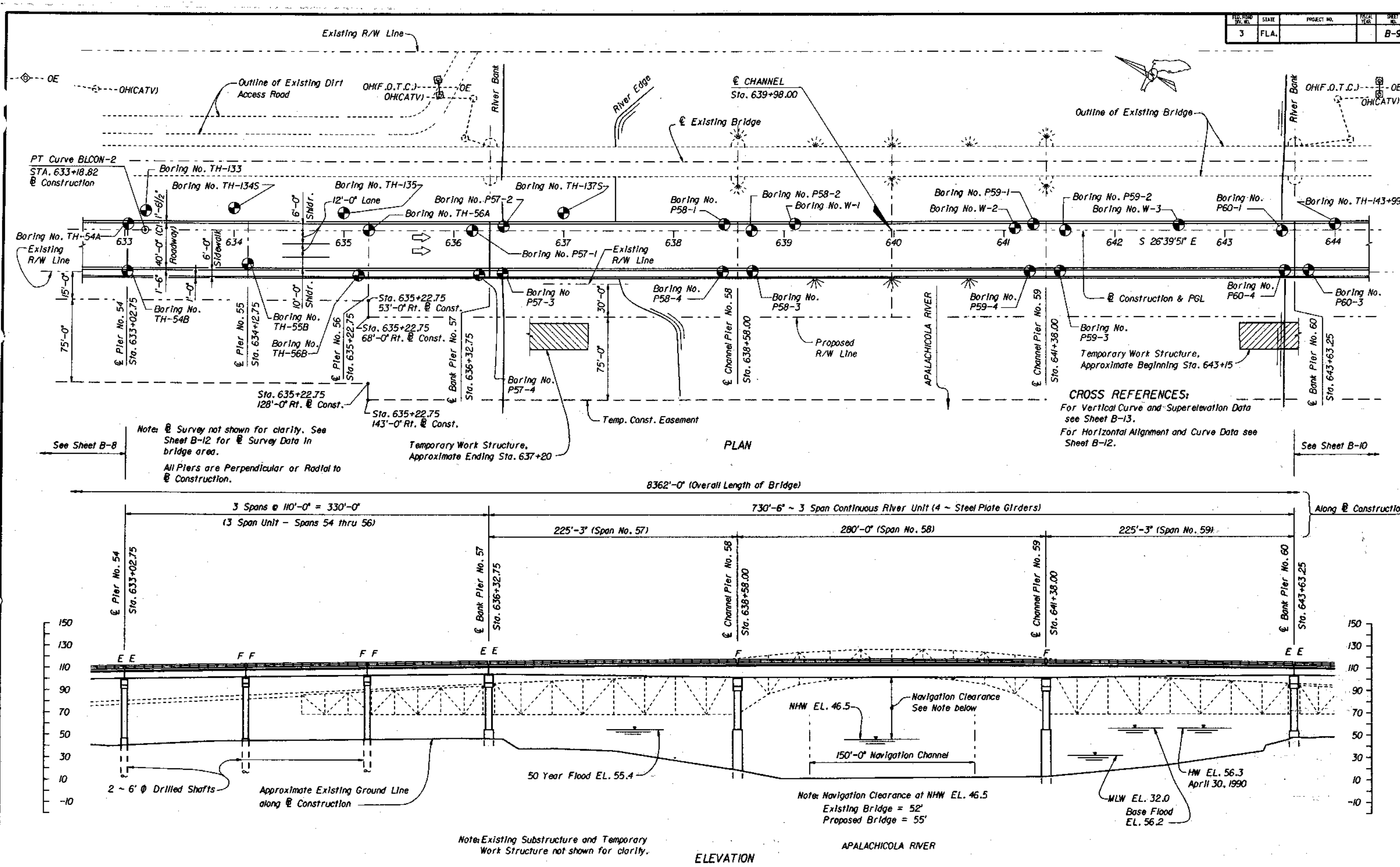
SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
20	CALHOUN & LIBERTY	47010-3519 & 56010-3520

SHEET TITLE: PLAN AND ELEVATION (SHEET 6 OF 9)

PROJECT NAME: SR 20 OVER APALACHICOLA RIVER



FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-9

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	JLF	5-94
Checked by	CEB	2-95
Designed by	JLF	5-94
Checked by	CEB	2-95
Approved by	HTB	

ENGINEER OF RECORD:
STRUCTURES DESIGN OFFICE
CENTRAL OFFICE
605 Suwannee Street, MS 33
Tallahassee, Florida 32399-0450

LOGO:

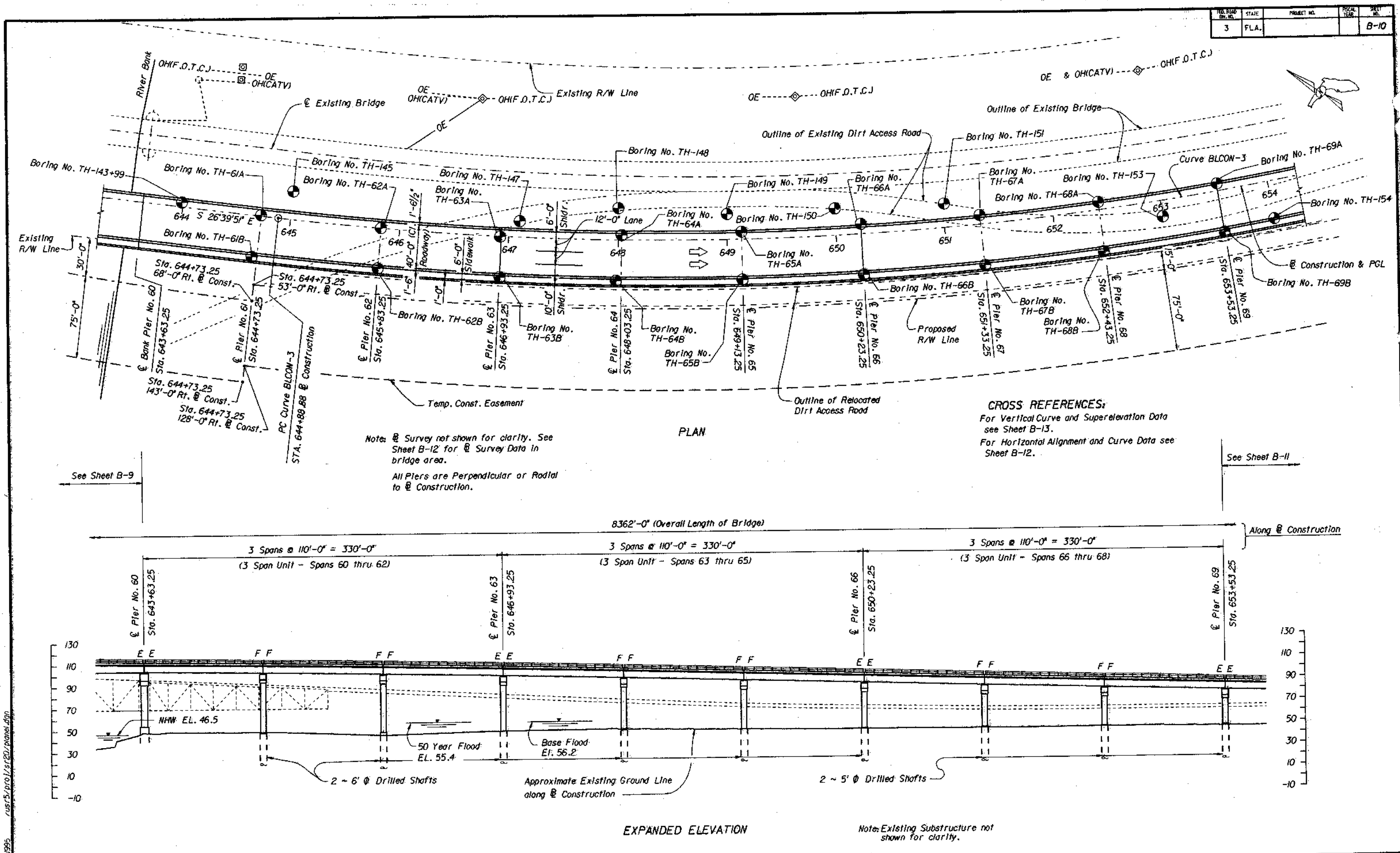
SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
20	CALHOUN & LIBERTY	47010-3519 & 56010-3520

SHEET TITLE: PLAN AND ELEVATION (SHEET 7 OF 9)

PROJECT NAME: SR 20 OVER APALACHICOLA RIVER



REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	JLF	5-94
Checked by	CEB	2-95
Designed by	JLF	5-94
Checked by	CEB	2-95
Approved by	HTB	

ENGINEER OF RECORD:
STRUCTURES DESIGN OFFICE
CENTRAL OFFICE
605 Suwannee Street, MS 33
Tallahassee, Florida 32399-0450

LOGO:

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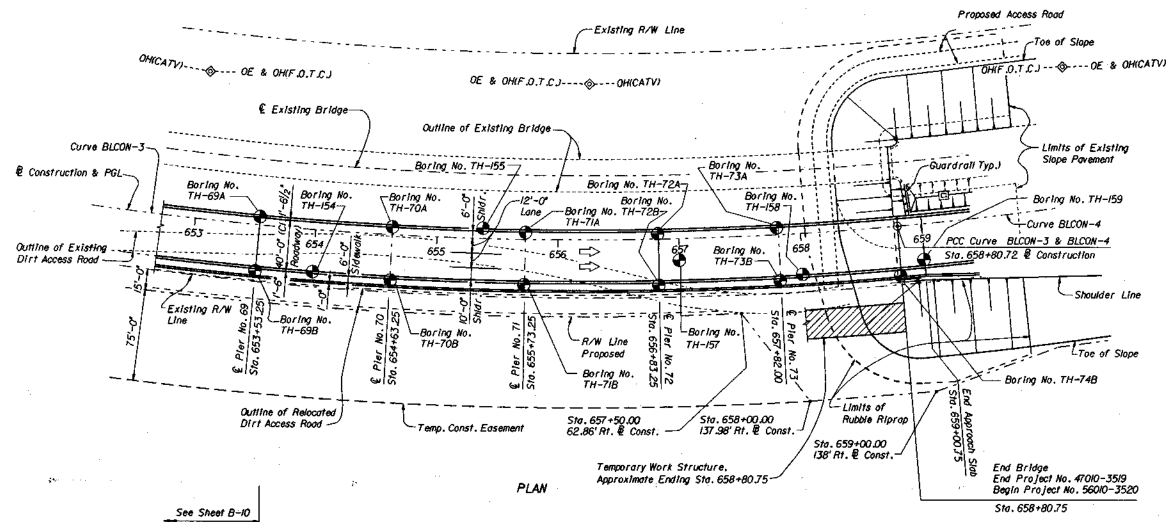
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
20	CALHOUN & LIBERTY	47010-3519 & 56010-3520

SHEET TITLE: PLAN AND ELEVATION (SHEET 8 OF 9)

PROJECT NAME: SR 20 OVER APALACHICOLA RIVER

FISCAL YEAR	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-11



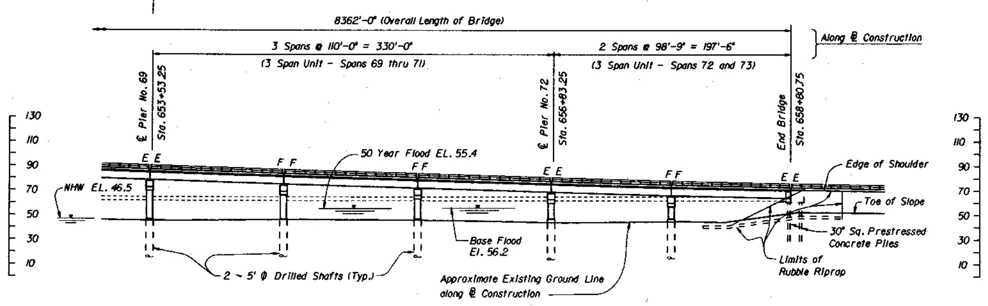
PLAN

See Sheet B-10

Note: ℄ Survey not shown for clarity. See Sheet B-12 for ℄ Survey Data in bridge area.

All Piers & FFBW End Bent No. 74 are Radial to ℄ Construction.

For Rubble Riprap Details see Sheet A-13.



EXPANDED ELEVATION

Note: Existing Substructure and Temporary Work Structure not shown for clarity.

CROSS REFERENCES:
For Vertical Curve and Superelevation Data see Sheet B-13.
For Horizontal Alignment and Curve Data see Sheet B-12.

/usr5/proj/sr20/planel.dgn

REVISIONS							Names	Dates
Date	By	Description	Date	By	Description	Drawn by	JLF	1-94
						Checked by	CEB	2-95
						Designed by	JLF	5-94
						Checked by	CEB	2-95
						Approved by	HTB	

ENGINEER OF RECORD:
STRUCTURES DESIGN OFFICE
CENTRAL OFFICE
605 Suwannee Street, MS 33
Tallahassee, Florida 32399-0450

LOGO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
20	CALHOUN & LIBERTY	47010-3519 & 56010-3520

SHEET TITLE: PLAN AND ELEVATION (SHEET 9 OF 9)

PROJECT NAME: SR 20 OVER APALACHICOLA RIVER

GENERAL NOTES

1. The shafts shall be constructed using the wet construction method with mineral slurry in conjuction with permanent casing as shown in the plans or temporary casing as necessary and desanding equipment with holding tanks. The desanding equipment supplied by the Contractor shall be equipped with suitable screen and cyclone separators and shall have adequate capacity to desand the volumes of slurry required for the project.

2. Special precautions shall be taken by the Contractor to prevent discharge of slurry or excavated materials into the river or river flood plain during the excavation operation. The Contractor shall provide a suitable receptacle around the top of the casing to collect slurry and/or concrete discharge from the shaft during the placement of concrete. Materials excavated from the drilled shafts shall be collected and disposed of satisfactorily in offsite upland areas provided by the Contractor and approved by the Engineer.

3. One or more test holes shall be performed for each shaft size. See Load Test Plans.

4. For the Drilled Shaft Test Holes and Production Shafts, it is the intention of the Department to utilize a Shaft Inspection Device (SID) in accordance with the provisions of Section B455 of the Supplemental Specifications and the Technical Special Provisions.

5. The spacing of the vertical bars at footing level shall be set with a template.

6. Permanent casing is required only for Alt. AB, Piers 57, 58, 59, 60 and for Alt. AA Piers 58 and 59. If the Contractor intends to use permanent casing at other Piers he must reestablish the minimum Tip elevations to be approved by the Engineer.

7. The cost of temporary casing shall be included in the bid price for Unclassified Shaft Excavation.

8. Minimum Tip Elevations are designated in the Plans. Adjustment of minimum Tip Elevations as a result of the Load Test Program lies solely with the Department. Adjustment of minimum Tip Elevations by the Department, either to higher or lower elevations, will be paid for at the Contract Unit Price for the applicable Drilled Shaft Items based on the quantity provided; no additional costs over and above the cost of the applicable Items will be incurred by the Department for adjustment of minimum Tip Elevations.

9. Construct Drilled Shafts in accordance with the provisions of the Supplemental Specification B455, Structures Foundations and Technical Special Provisions.

10. Reference Load Test Sheets for Load Test Requirements.

11. To the reinforcement shown in the Plan details, provide additional reinforcement in the form of ties, spacers and stiffener bars as necessary to facilitate handling and to insure proper alignment and location of reinforcement cage during cage and concrete placement operations. Additional reinforcement required to facilitate handling and to insure a proper alignment and location will be at no additional cost to the Department.

12. Note that Grade 75 ksi number 18 bars are specified for the 9 foot diameter Drilled Shafts Alternate AB. The delivery time for these bars may be longer than usual and the Contractor should schedule his work accordingly.

13. Welding per ANSI/AWS D1.4 of the Reinforcement, excluding the primary reinforcement, for the purpose of increasing rebar cage stiffness for handling is permitted.

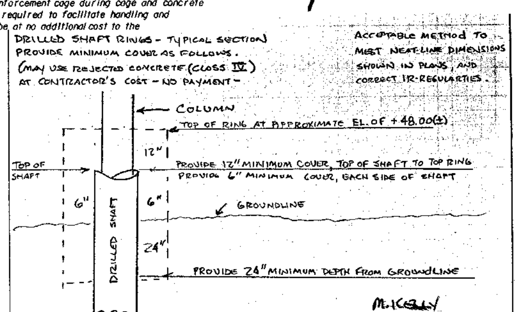
14. Lap splices shown may be substituted with Mechanical Splices. These Splices must develop 125% of bar yield strength.

15. The Permanent Casing Tip Elevations shown for the 7' Ø and 9' Ø Drilled Shafts may not be lowered without approval by the Engineer.

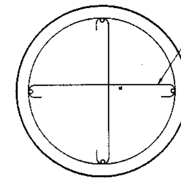
16. Note that all Drilled Shafts on the LT Sheets are Designated 'Test Holes' and no separate payment will be made for those as per Subarticle B455-12.7. Specific Pay Items exist for the Load Test Equipment and Performance.

17. The plans provide for permanent casing to be installed to estimated elevations at Pier Nos. 58 and 59 in Alternate AA and Pier Nos. 57, 58, 59 & 60 in Alternate AB. This casing is provided to reach the interface of the limerock and the overburden soils to provide support of the overburden soils. It is possible that sporadic caving of the materials below the permanent casing may occur.

* THE CONTRACTOR'S METHODS OF DRILLED SHAFT CONSTRUCTION RESULTED IN IRREGULAR SURFACES ON THE TOP OF SHAFT. THE ENGINEER AUTHORIZED A "RING" TO BE CONSTRUCTED, AS SHOWN IN DETAIL BELOW, AT NO ADDITIONAL COST TO THE DEPARTMENT, TO EVEN-OUT THE IRREGULAR SURFACES AND PROVIDE CONTINUITY.



M. KELLY



TEMPORARY CAGE STIFFENER DETAILS

J. V. Moulton
3/1/99

REVISIONS					
Date	By	Description	Date	By	Description
5/24/95	JLF	Revised Note 1			
2/2/99	MRK	FINAL AS-BUILT			

	Names	Dates
Drawn by	JP	10-94
Checked by	HTB	10-94
Designed by	HTB	8-94
Checked by	CEB	3-95
Approved by	HTB	

ENGINEER OF RECORD:
STRUCTURES DESIGN OFFICE
CENTRAL OFFICE
605 Suwannee Street, MS 33
Tallahassee, Florida 32399-0450

LOGO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

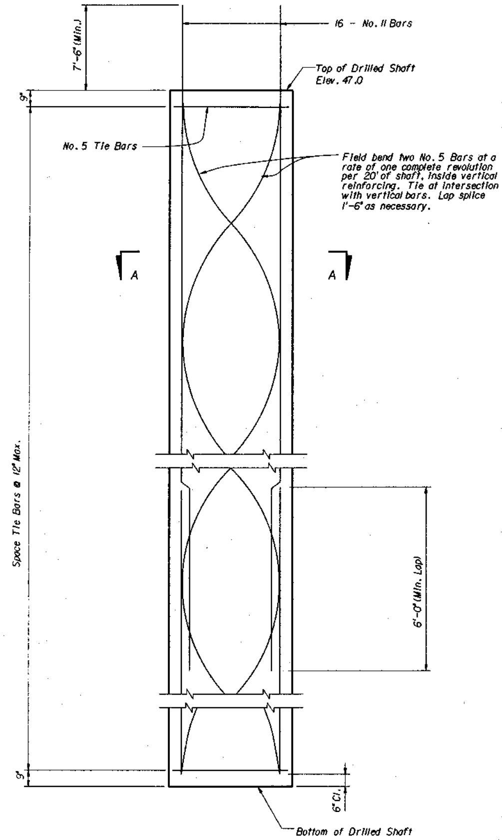
ROAD NO.	COUNTY	PROJECT NO.
20	CALHOUN & LIBERTY	47010-3519 & 56010-3520

SHEET TITLE: DRILLED SHAFT GENERAL NOTES

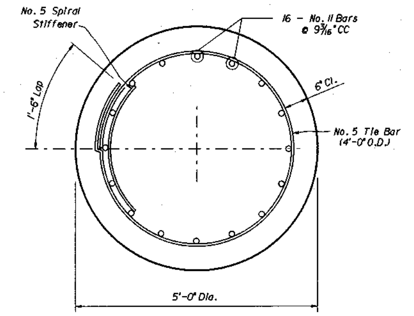
PROJECT NAME: SR 20 OVER APALACHICOLA RIVER

Thu May 25 10:31:44 1995 $strs0plfile@drllsnt.spg

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-33



ELEVATION



SECTION A-A

DRILLED SHAFT QUANTITIES PER LIN. FT.		
ITEM	UNIT	QUANTITY
Class IV Concrete (Drilled Shaft)	C.Y.	.727
Reinforcing Steel (Substructure)	LB.	98.12

Note: Quantities shown for Reinforcing Steel does not include lap splices for No. 11 Vertical Bars & No. 5 Spiral Bars.

NOTES:
1. Concrete for Drilled Shafts shall be Class IV (Drilled Shaft).
2. Reinforcing Steel shall conform to ASTM A-615 Grade 60.

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	JP	9-94
Checked by	HTB	9-94
Designed by	HTB	8-94
Checked by	CEB	3-95
Approved by	HTB	

ENGINEER OF RECORD:
STRUCTURES DESIGN OFFICE
CENTRAL OFFICE
605 Suwannee Street, MS 33
Tallahassee, Florida 32399-0450

LOGO:

SEAL:

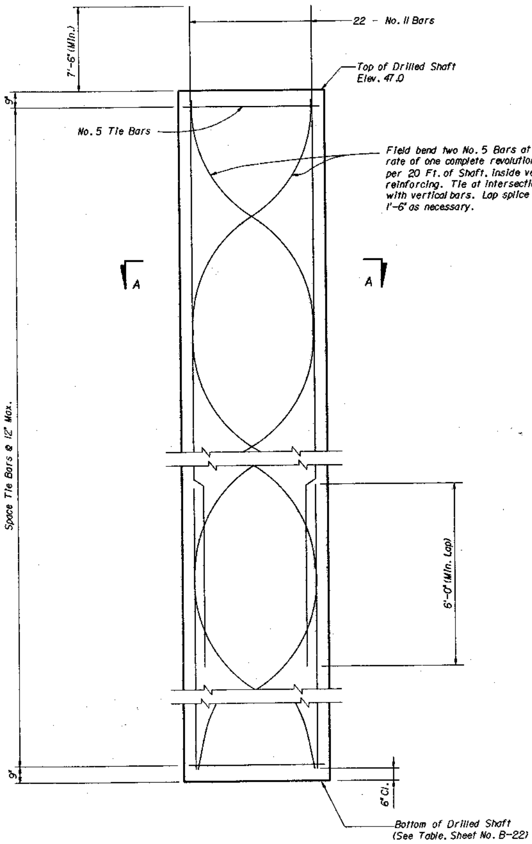
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
20	CALHOUN & LIBERTY	47010-3519 & 56010-3520

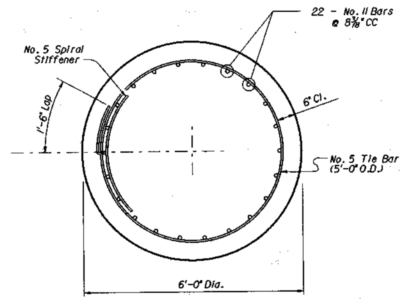
SHEET TITLE:
5 FT. Ø DRILLED SHAFTS
PIER NOS. 43 TO 47 & 67 TO 73

PROJECT NAME:
SR 20 OVER APALACHICOLA RIVER

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-34



DRILLED SHAFT ELEVATION



SECTION A-A

DRILLED SHAFT QUANTITIES PER LIN. FT.		
ITEM	UNIT	QUANTITY
Class IV Concrete (Drilled Shaft)	C.Y.	1.047
Reinforcing Steel (Substructure)	LB.	133.27

Note: Quantities shown for Reinforcing Steel does not include lap splices for No. 11 Vertical Bars & No. 5 Spiral Bars.

NOTES:

1. Concrete for Drilled Shafts shall be Class IV (Drilled Shaft).
2. Reinforcing Steel shall conform to ASTM A-615 Grade 60.

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	JP	9-94
Checked by	HTB	9-94
Designed by	HTB	8-94
Checked by	CEB	3-95
Approved by	HTB	

ENGINEER OF RECORD:
STRUCTURES DESIGN OFFICE
CENTRAL OFFICE
605 Suwannee Street, MS 33
Tallahassee, Florida 32399-0450

LOGO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

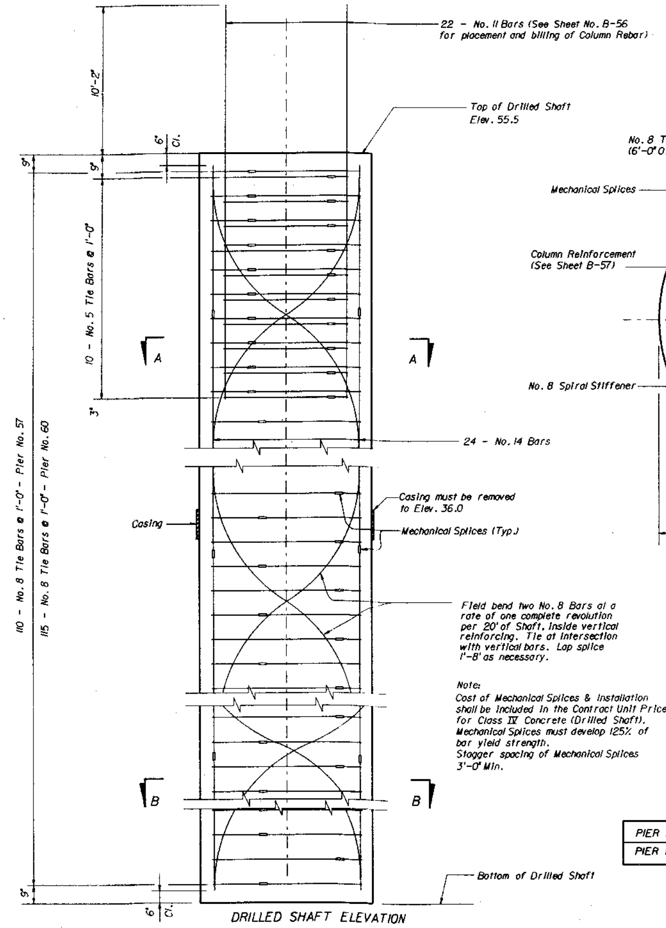
ROAD NO.	COUNTY	PROJECT NO.
20	CALHOUN & LIBERTY	47010-3519 & 56010-3520

SHEET TITLE:
6 FT. Ø DRILLED SHAFTS
PIER NOS. 48 TO 56 & 61 TO 66

PROJECT NAME:
SR 20 OVER APALACHICOLA RIVER

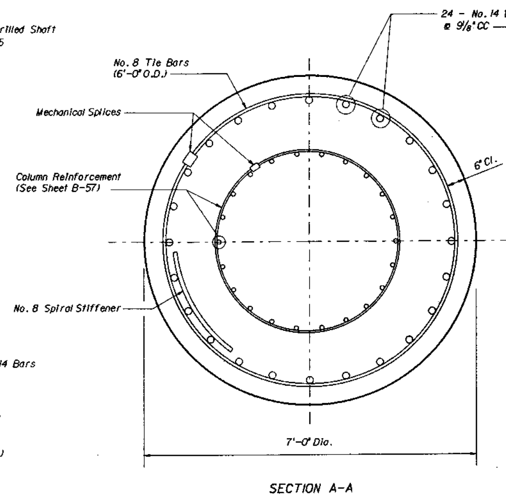
Tue Mar 14 09:18:52 1995 /usr5/proj/sr20/drilsht.dgn

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-37

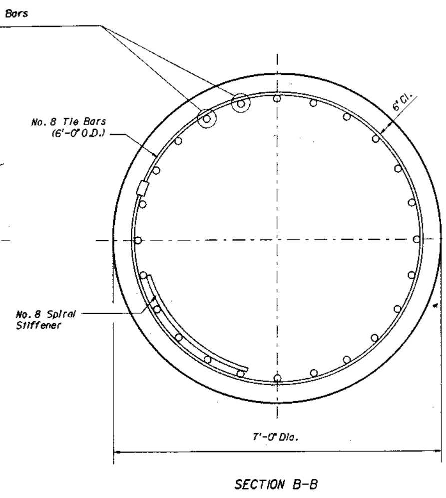


DRILLED SHAFT ELEVATION

Note:
Cost of Mechanical Splices & Installation shall be included in the Contract Unit Price for Class IV Concrete (Drilled Shaft).
Mechanical Splices must develop 125% of bar yield strength.
Stagger spacing of Mechanical Splices 3'-0" Min.



SECTION A-A



SECTION B-B

	MINIMUM BOTTOM SHAFT ELEVATION	ESTIMATED BOTTOM CASING ELEVATION
PIER NO. 57	-55.0	-20.0
PIER NO. 60	-65.0	-32.0

DRILLED SHAFT QUANTITIES PER LIN. FT.		
ITEM	UNIT	QUANTITY
Class IV Concrete (Drilled Shaft)	C.Y.	1.425
Reinforcing Steel (Substructure)	LB.	232.1

Note: Reinforcing Steel quantity does not include Spiral Stiffener or Mechanical Splices.

NOTES:

1. Concrete for Drilled Shafts shall be Class IV (Drilled Shaft).
2. Reinforcing Steel shall conform to ASTM A-615 Grade 60.
3. Mechanical Splices must develop 125% of Bar Yield Strength.
4. Permanent Casings required.
5. Two - 20 foot cores will be required at bottom of each 7 Ft.Ø Shaft.

ALTERNATE AB

REVISIONS					
Date	By	Description	Date	By	Description
5/24/95	JLF	Changed Casing Removal Elev. 30.0 to Elev. 36.0			

	Names	Dates
Drawn by	JP	2-95
Checked by	HTB	2-95
Designed by	HTB	2-95
Checked by	CEB	3-95
Approved by	HTB	

ENGINEER OF RECORD: STRUCTURES DESIGN OFFICE
CENTRAL OFFICE
605 Suwannee Street, MS 33
Tallahassee, Florida 32399-0450

LOGO:

SEAL:

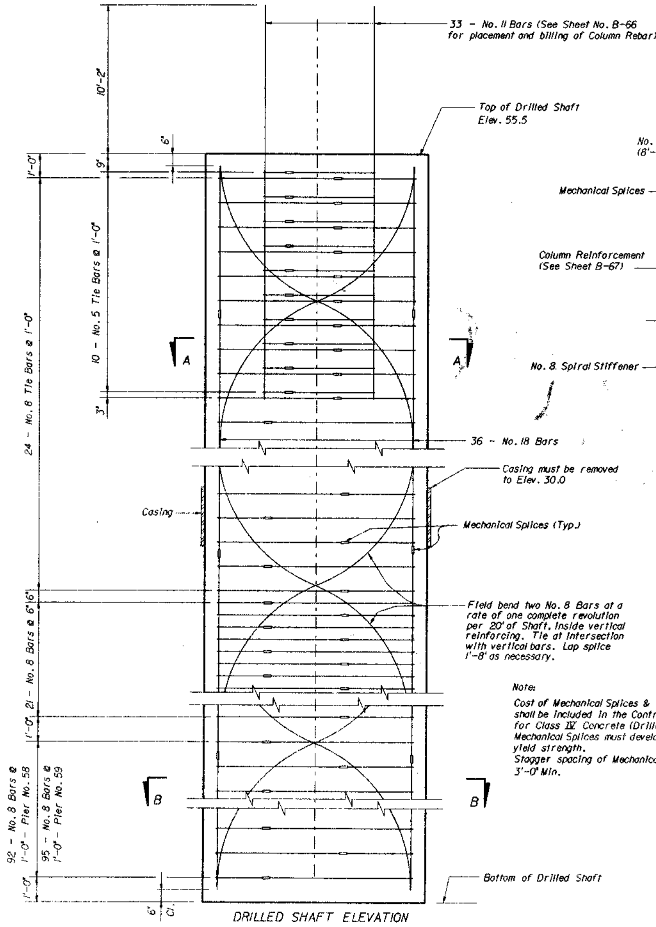
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
20	CALHOUN & LIBERTY	47010-3519 & 56010-3520

SHEET TITLE: 7 FT. Ø DRILLED SHAFTS PIER NOS. 57 AND 60

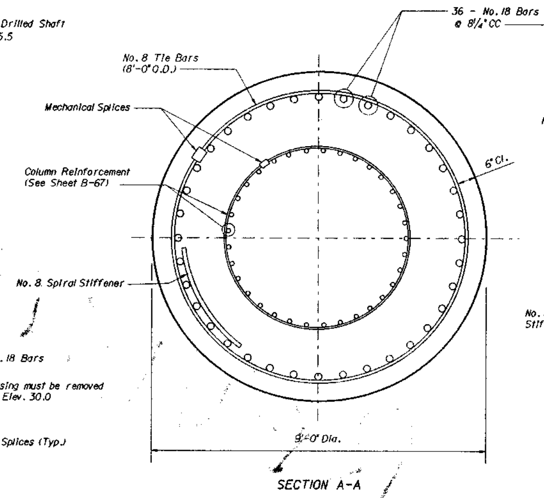
PROJECT NAME: SR 20 OVER APALACHICOLA RIVER

Mon May 22 16:27:39 19.. trs@pitfiles@drillsht.spg

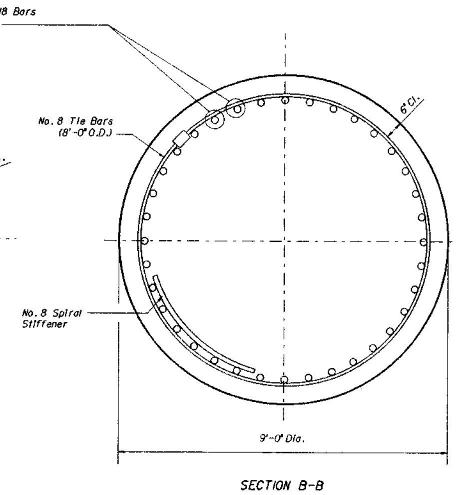


DRILLED SHAFT ELEVATION

Note:
Cost of Mechanical Splices & Installation shall be included in the Contract Unit Price for Class IV Concrete (Drilled Shaft). Mechanical Splices must develop 125% of bar yield strength. Stagger spacing of Mechanical Splices 3'-0" Min.



SECTION A-A



SECTION B-B

	MINIMUM BOTTOM SHAFT ELEVATION	ESTIMATED BOTTOM CASING ELEVATION
PIER NO. 58	-72.0	-26.0
PIER NO. 59	-75.0	-28.0

DRILLED SHAFT QUANTITIES PER LIN. FT.

ITEM	UNIT	QUANTITY
Class IV Concrete (Drilled Shaft)	C.Y.	2.356
Reinforcing Steel (Substructure)	LB.	561.7

Note: Reinforcing Steel quantity does not include Spiral Stiffener or Mechanical Splices.

NOTES:

1. Concrete for Drilled Shafts shall be Class IV (Drilled Shaft).
2. Reinforcing Steel shall conform to ASTM A-615 Grade 60 except No. 18 Bars shall be Grade 75.
3. Mechanical Splices must develop 125% of Bar Yield Strength.
4. Permanent Casing required.
5. Three ~ 20 foot deep cores will be required at bottom of each 9 Ft. Ø Shaft.

ALTERNATE AB

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-38

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	JP	2-95
Checked by	HTB	2-95
Designed by	HTB	2-95
Checked by	CEB	3-95
Approved by	HTB	

ENGINEER OF RECORD:
STRUCTURES DESIGN OFFICE
CENTRAL OFFICE
605 Suwannee Street, MS 33
Tallahassee, Florida 32399-0450

LOGO:

SEAL:

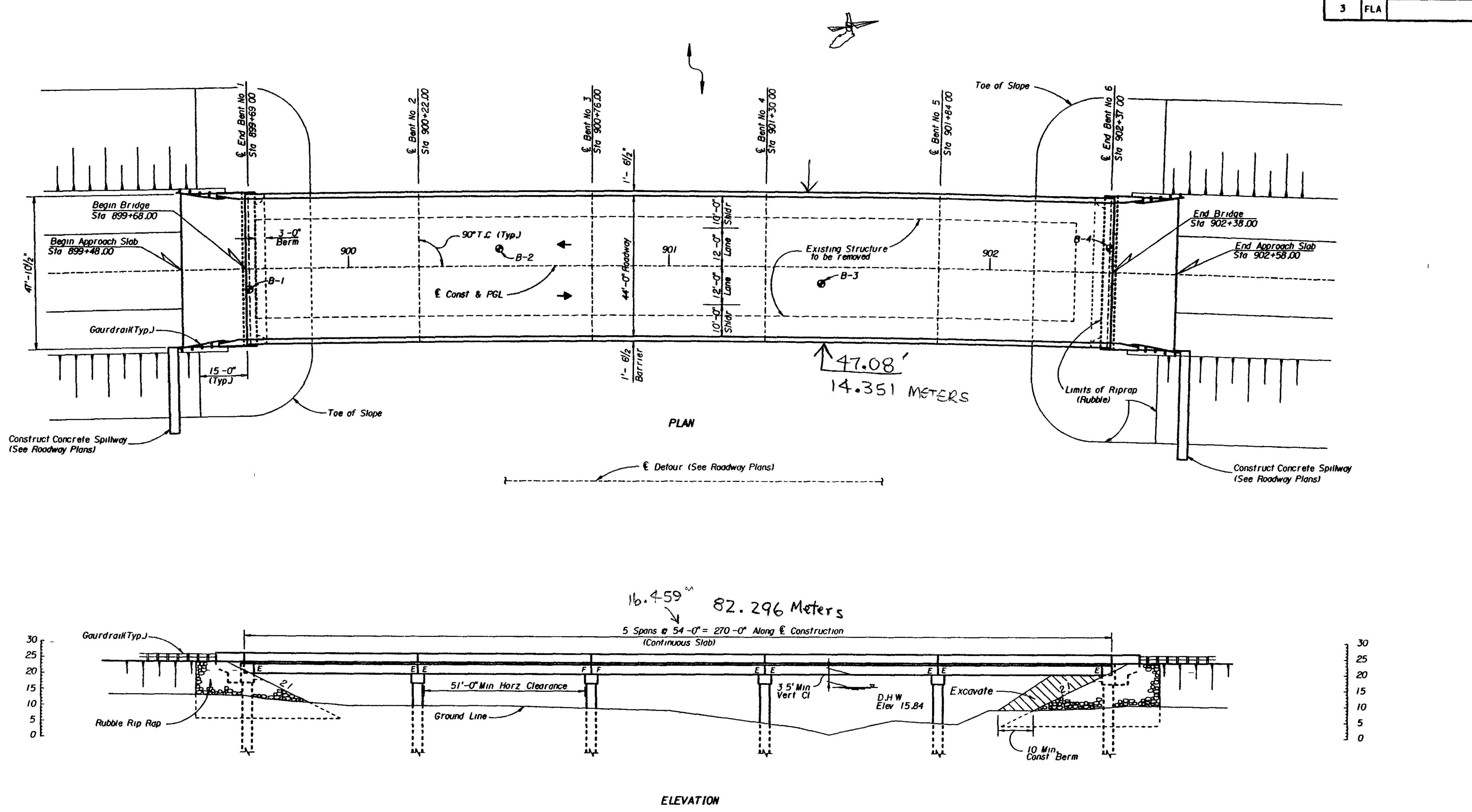
FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
20	CALHOUN & LIBERTY	47010-3519 & 56010-3520

SHEET TITLE: 9 FT. Ø DRILLED SHAFTS PIER NOS. 58 AND 59

PROJECT NAME: SR 20 OVER APALACHICOLA RIVER

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA			B-2

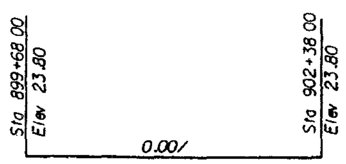


NOTES

1. For Riprap details see Sheet B-1
2. See Sheet B-4 and B-5 for location of borings

HORIZONTAL CURVE DATA

P.I Sta 907+70.79
Δ = 29°24'00"
D = 1°00'00"
T = 1503.18'
L = 2940.00
R = 5729.57
e = 0.033 /FT



PROFILE GRADE

TRAFFIC DATA

ADT 1990 - 5990
ADT 2005 - 9300
K=12.6%
D=58.0%
T=5.0%
DESIGN SPEED - 65 MPH

BRIDGE NO 590048

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	KMH	12/91
Checked by	PWG	12/91
Designed by	DKS	12/91
Checked by	PWG	12/91
Approved by	[signature]	

ENGINEER OF RECORD:
STRUCTURES DESIGN OFFICE
DISTRICT 3
U.S. 90 East
Chipley, Florida 32428-9990

LOGO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-61	WAKULLA	59010-3519

SHEET TITLE: GENERAL PLAN AND ELEVATION SHEET

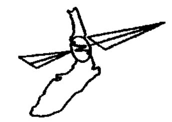
PROJECT NAME: SR-61 OVER LOST CREEK

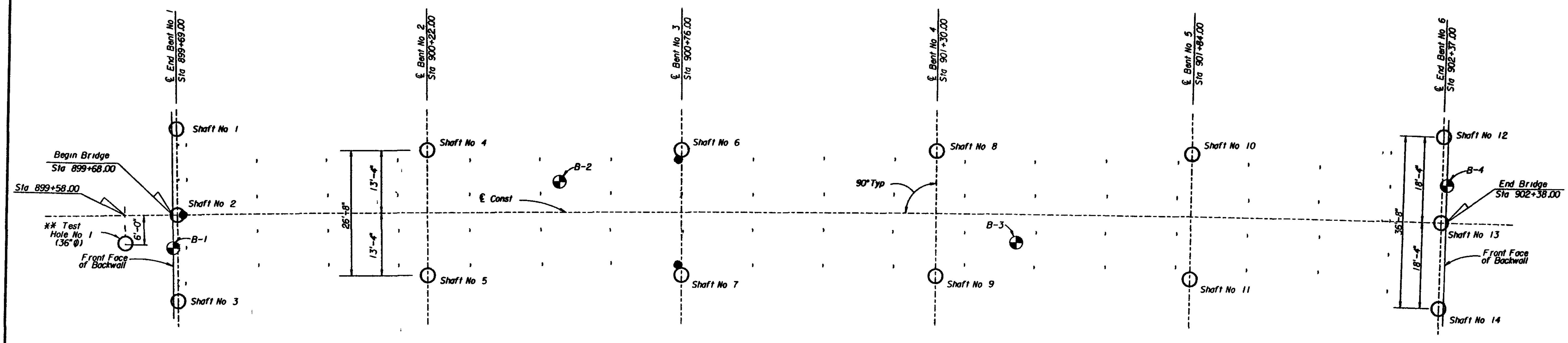
Drawing No.

Index No.

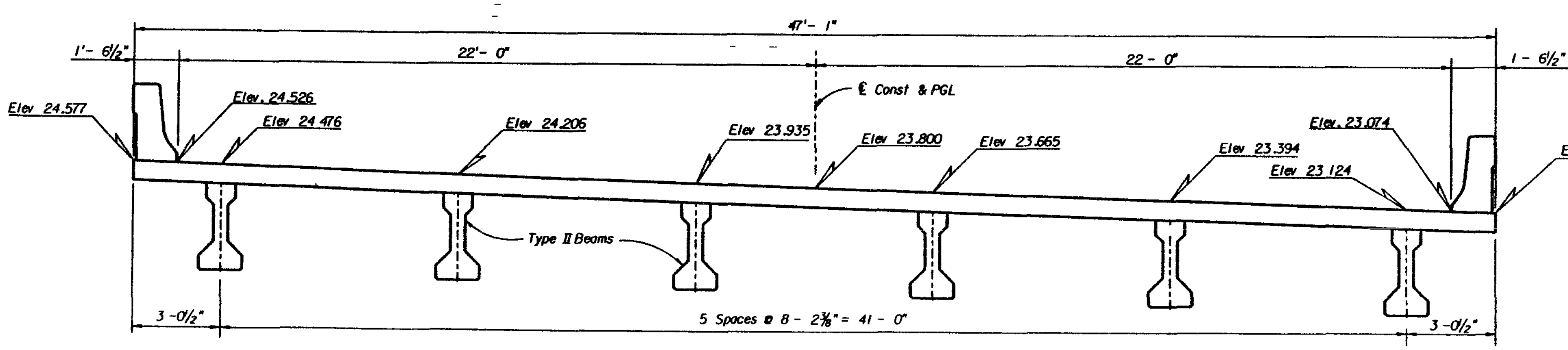
Tue Jul 27 05:02:03 1993 sr2/proj/59010/1519/plan.dgn

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA			B-6





** NOTE: Test Hole location is subject to revision by the Engineer prior to commencement of work



SECTION THRU SUPERSTRUCTURE
(Showing Finish Grade Elevations)

LEGEND

○ Denotes 36" Dia Drilled Shaft

● * Denotes Existing Pile to be extracted prior to Drilled Shaft Construction

◌ * Denotes Existing Pile to be removed 2' below natural ground

* To be paid for under Item 110-3 (Removal of Existing Structures)

NOTES

1 See Sheet B-2 For Horizontal Curve Data
2 See sheets B-4 and B-5 for location of borings
3 Scour has been considered in the design with scour elevations of El_{500} and El_{100} as in the table below. Under no circumstances shall the shafts be installed to tip elevations above the minimum tip elevation required
4 The Test Shaft shall be reinforced. Cost of reinforcing cage to be include under Item No 455-91-3
5 The slump loss test as described in Section 346-3.2 of the Concrete Specification shall be performed at the job site

DRILLED SHAFT INSTALLATION TABLE

BENT	DESIGN LOAD (TONS)	REQUIRED TIP ELEV (FT)	EL_{500} ① (FT.)	EL_{100} ② (FT.)
1	138	-23.00	*	*
2	226	-34.00	-2.2	-0.6
3	226	-34.00	-2.9	-1.3
4	226	-32.60	-7.1	-5.5
5	226	-32.40	-6.9	-5.3
6	138	-25.00	*	*

① Anticipated scour elevation for 500-Yr frequency
② Anticipated scour elevation for 100-Yr frequency
* No scour anticipated at End Bents

Tue Jul 27 06:03:05 1993 usr2/proj/59010/1519/layout.dgn

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	KMH	12/91
Checked by	PWG	12/91
Designed by	DKS	12/91
Checked by	PWG	12/91
Approved by	[signature]	

ENGINEER OF RECORD:
STRUCTURES DESIGN OFFICE
DISTRICT 3
U.S. 90 East
Chipley, Florida 32428-9990

LOGO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

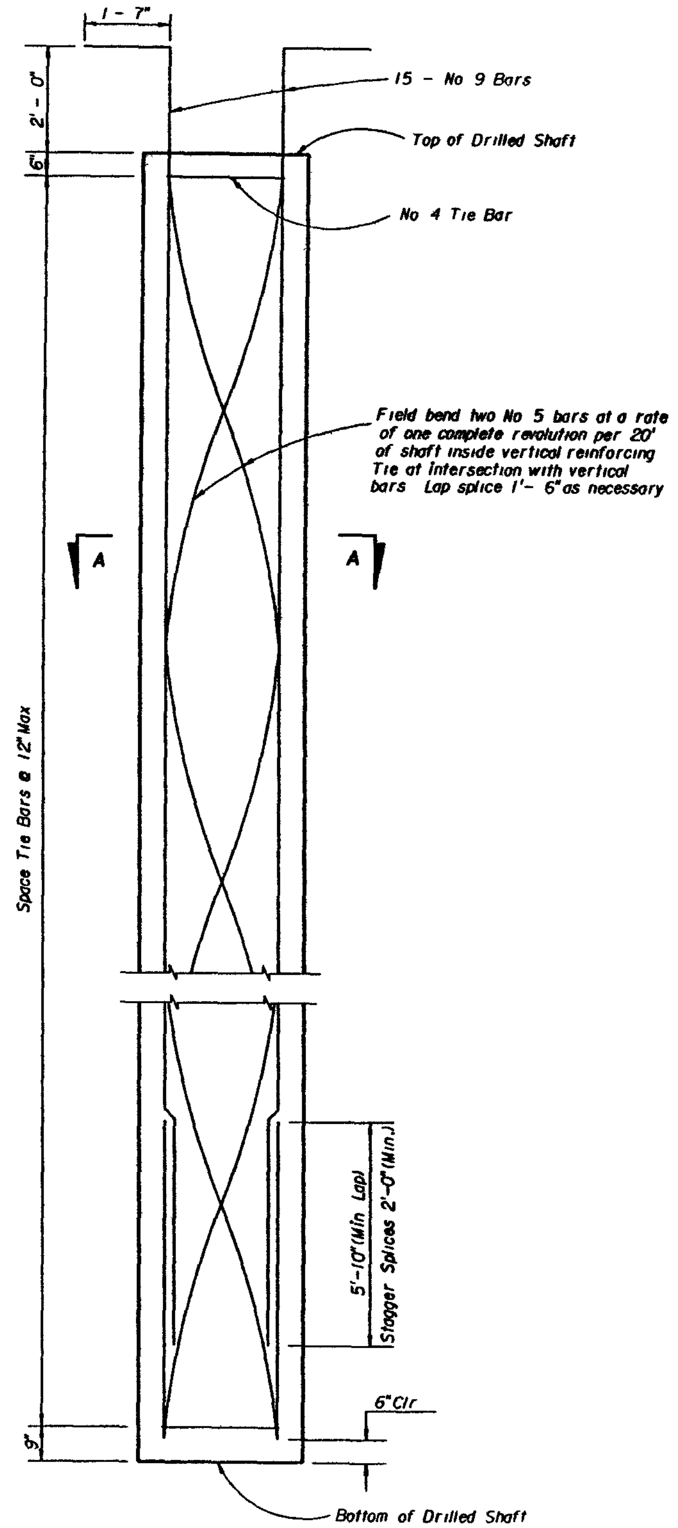
ROAD NO.	COUNTY	PROJECT NO.
SR-61	WAKULLA	59010-3519

SHEET TITLE: FOUNDATION LAYOUT AND FINISH GRADE ELEVATIONS

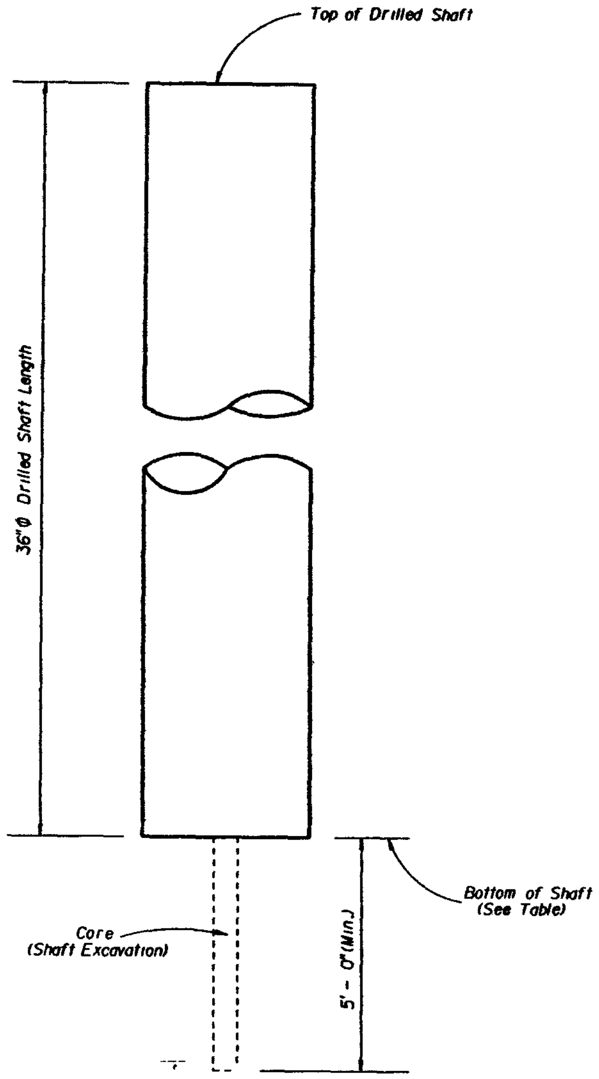
Drawing No.

PROJECT NAME: SR-61 OVER LOST CREEK

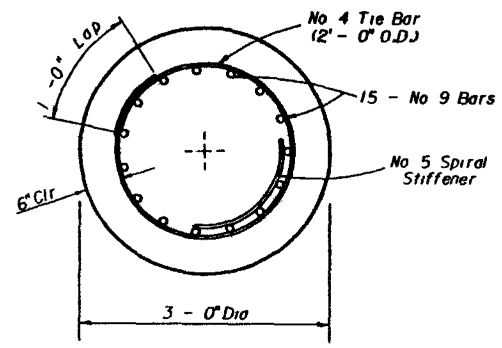
Index No.



DRILLED SHAFT ELEVATION



DRILLED SHAFT DETAIL



SECTION A-A

TABLE OF ELEVATION AND SHAFT LENGTHS			
Shaft No	Top of Shaft Elev.	Bottom of Shaft Elev.	Length Per Shaft (Ft.)
1	17.023	-23.000	40.023
2	16 418	-23.000	39.418
3	15.813	-23.000	38.813
4	16.887	-34.000	50.887
5	16.007	-34.000	50.007
6	16.887	-34.000	50.887
7	16.007	-34.000	50.007
8	16.887	-32.600	49 487
9	16.007	-32.600	48.607
10	16.887	-32.400	49.287
11	16.007	-32.400	48 407
12	17.023	-25.000	42.023
13	16 418	-25.000	41.418
14	15.813	-25.000	40.813
TEST HOLE NO 1	17.000	-34.000	51.000

DRILLED SHAFT QUANTITIES PER LIN.FT.		
ITEM	UNIT	QUANTITY
Class III Concrete (Drilled Shaft)	C.Y	0.26
*Reinforcing Steel (Substructure)	LB	58

*Quantity does not include lapsplice for No 9 vertical bars

NOTES:

1 Concrete for Drilled Shafts shall be Class III (Drilled Shaft)
2 Reinforcing Steel shall conform to ASTM A-615 Grade 60
3 The contractor shall perform a 4 to 6 inch diameter core run and/or a Standard Penetration Test boring continuous sampling for a minimum depth of 5 feet below the bottom of the drilled shaft excavation or as directed by the Engineer To be paid for under Item No 455-111 Any additional coring which exceeds the plan quantity will be paid at the contract price for coring
4 The contractor is required to construct an out of position test hole to demonstrate the adequacy of his construction technique and equipment The test hole location is indicated on the foundation layout sheet The test hole length is 51' with cutoff at elevation = 17.00' to be paid for under Item No 455-91-3

ESTIMATED QUANTITIES		
ITEM	UNIT	QUANTITY
Drilled Shaft (36"Φ) Item No 455-88-3	L F	640.08
Unclassified Shaft Excavation (36"Φ) Item No 455-122-3	L F	570.05
Extra Depth Excavation (36 Φ) Item No 455-125-3 *	L F	57.05
Core (Shaft Excavation) Item No 455-111	L F	70.00
Test Holes Item No 455-91-3	L F	51.00
Drilled Shaft Sidewall Overreaming Item No 455-124 *	L F	57.05
Class III Misc Item No 400-3-15 *	C.Y	16.78

* Extra Depth Excavation Drilled Shaft Sidewall Overreaming and Class III Drilled Shaft Concrete are contingency items and are shown for bidding purposes only

/usr2/proj/59010/1519/shaft.dgn

Tue Jul 27 06:04:05 1993

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	KMH	1/92
Checked by	PWG	1/92
Designed by	DKS	1/92
Checked by	PWG	1/92
Approved by	[signature]	

ENGINEER OF RECORD:
STRUCTURES DESIGN OFFICE
DISTRICT 3
U.S. 90 East
Chipley, Florida 32428-9990

LOGO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-61	WAKULLA	59010-3519

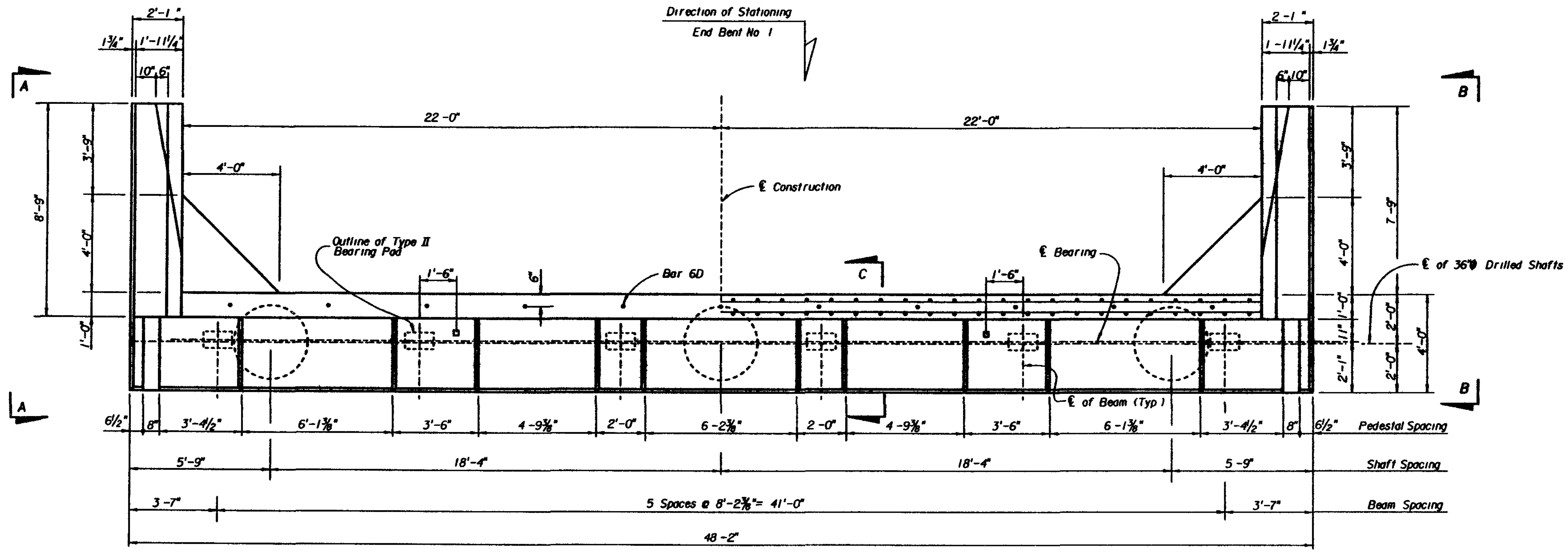
SHEET TITLE:
DRILLED SHAFT DETAILS

PROJECT NAME:
SR-61 OVER LOST CREEK

Drawing No.

Index No.

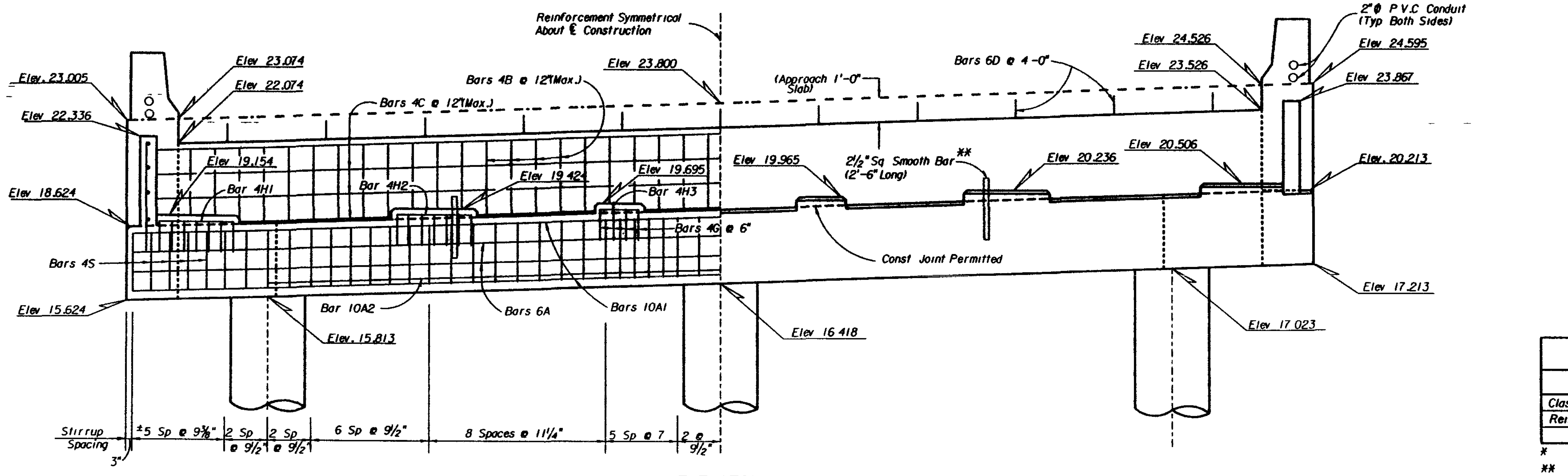
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA			B-8



PLAN

CONCRETE BREAKDOWN (C.Y.)

Cap	25.85
Walls	11.53
Wing Post	2.39



ELEVATION

* ESTIMATED QUANTITIES

ITEM	UNIT	QUANTITY
Class II Concrete (Substructure)	C.Y.	39.77
Reinforcing Steel (Substructure)	LB	5453

* Estimated Quantities are for one Bent only

** The cost of the 2½"Sq Bars to be included in the cost of Item No 460-2-15 (Structural Steel Misc.)

/usr2/proj/59010/1519/endbent.dgn

Tue Jul 27 06:07:30 1993

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	DKS	11/91
Checked by	PWG	11/91
Designed by	DKS	11/91
Checked by	PWG	11/91
Approved by	[signature]	

ENGINEER OF RECORD:
STRUCTURES DESIGN OFFICE
DISTRICT 3
U.S. 90 East
Chipley, Florida 32428-9990

LOGO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR 61	WAKULLA	59010-3519

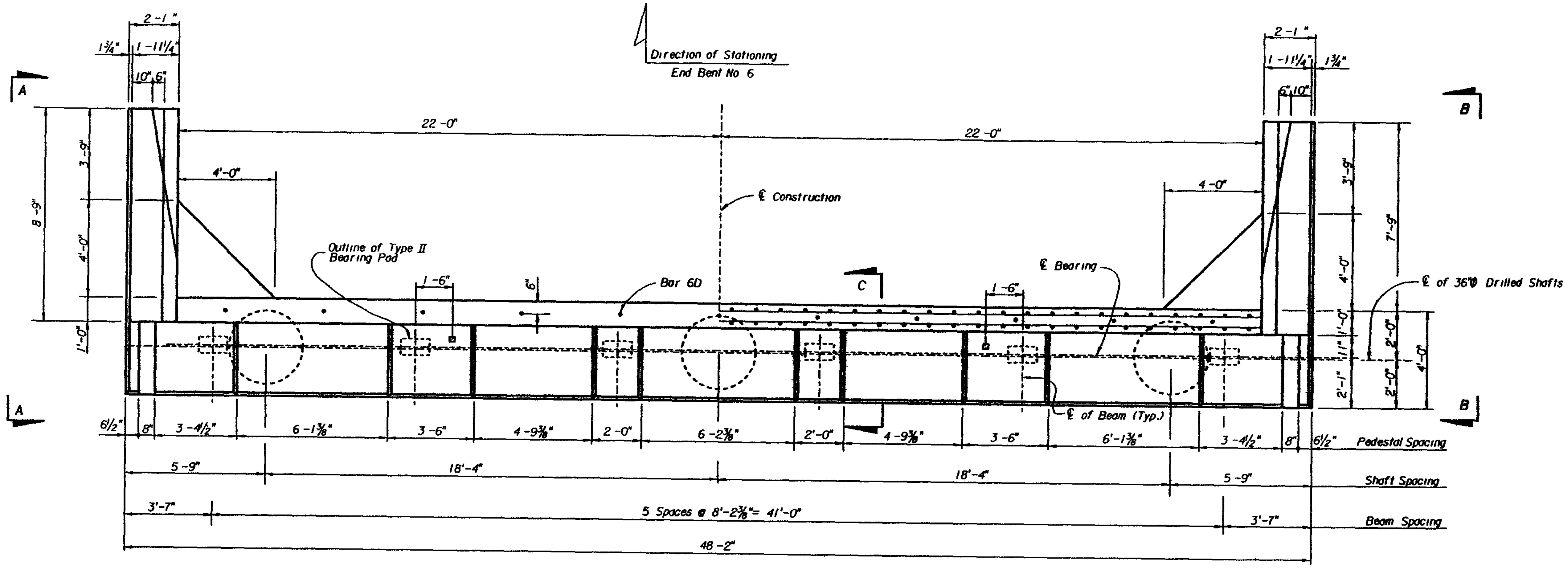
SHEET TITLE: END BENT NO. 1

Drawing No.

PROJECT NAME: SR-61 OVER LOST CREEK

Index No.

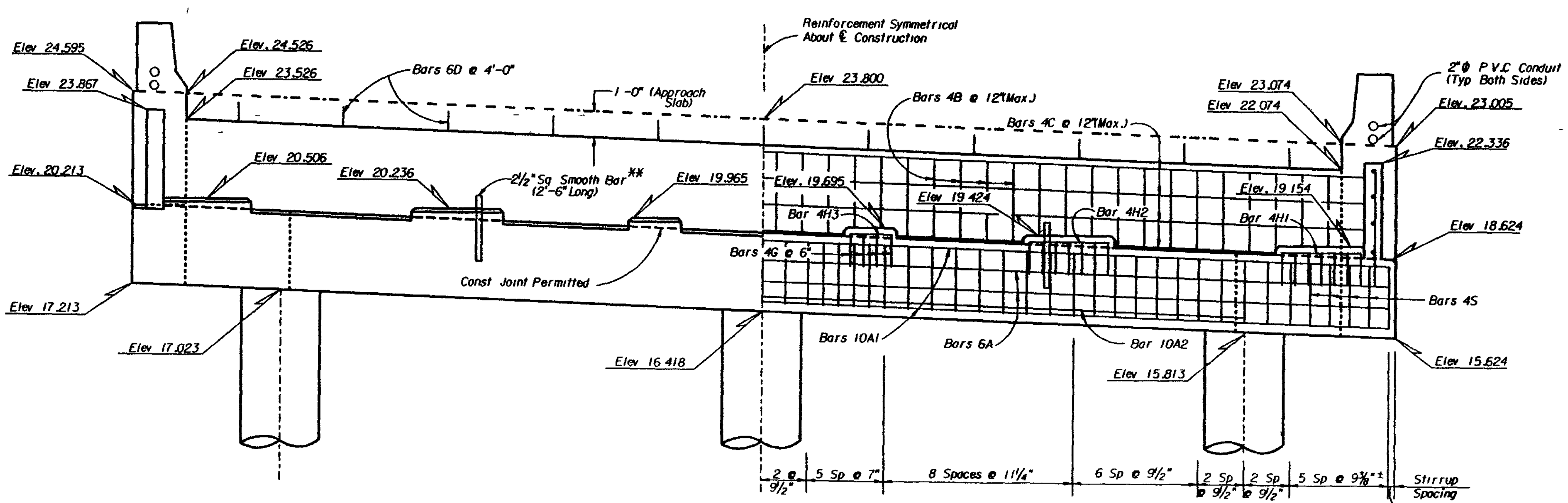
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA			B-9



PLAN

CONCRETE BREAKDOWN (C.Y.)

Cap	25.85
Walls	11.53
Wing Post	2.39



ELEVATION

*** ESTIMATED QUANTITIES**

ITEM	UNIT	QUANTITY
Class II Concrete (Substructure)	C.Y.	39.77
Reinforcing Steel (Substructure)	LB	5453

* Estimated Quantities are for one Bent only

** The cost of the 2 1/2 Sq Bars to be included in the cost of Item No 460-2-15 (Structural Steel Misc.)

/usr2/proj/59010/1519/endbent.dgn

Tue Jul 27 06:05:13 1993

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	DKS	11/91
Checked by	PWG	11/91
Designed by	DKS	11/91
Checked by	PWG	11/91
Approved by	[signature]	

ENGINEER OF RECORD:
STRUCTURES DESIGN OFFICE
DISTRICT 3
U.S. 90 East
Chipley, Florida 32428-9990

LOGO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO
SR 61	WAKULLA	59010-3519

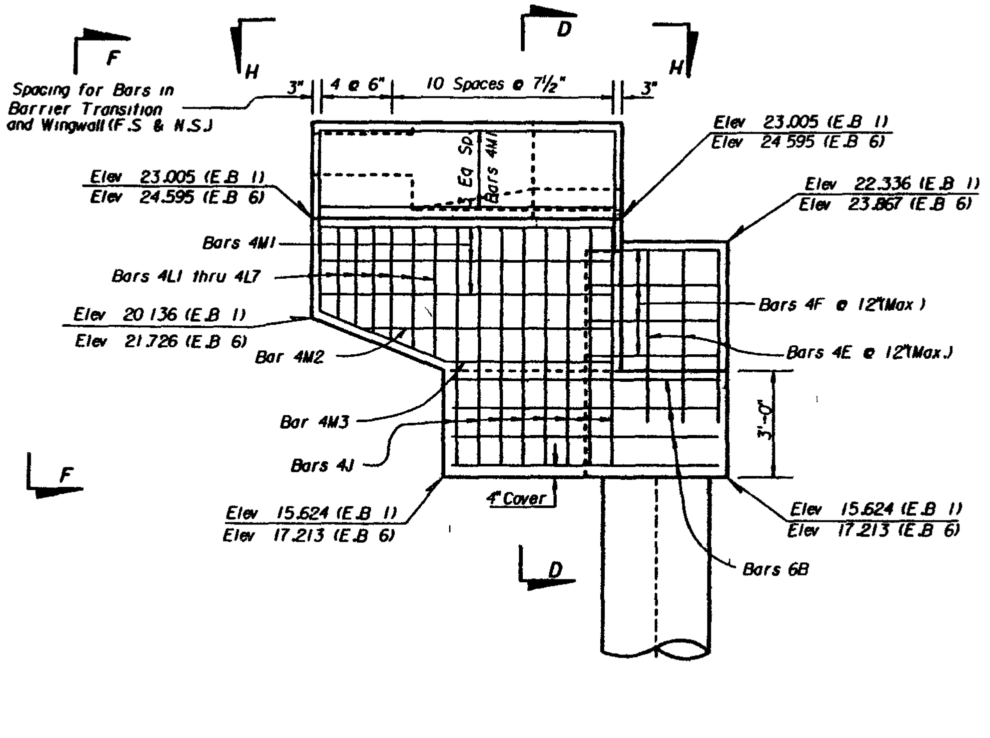
SHEET TITLE: END BENT NO. 6

PROJECT NAME: SR-61 OVER LOST CREEK

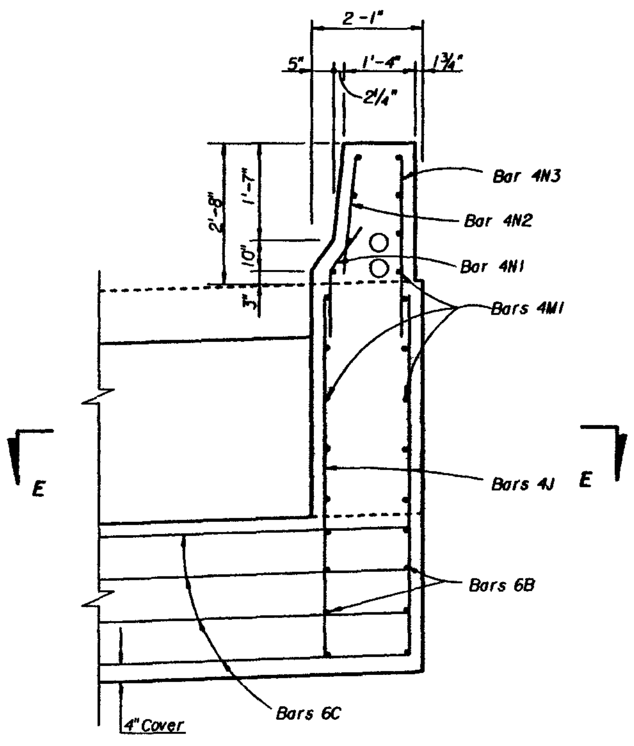
Drawing No.

Index No.

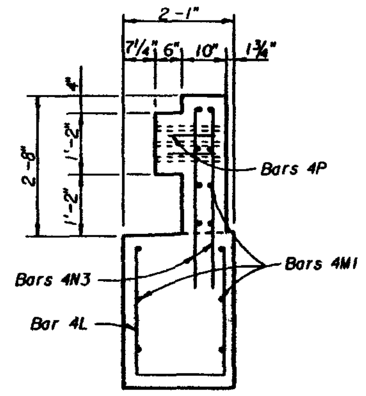
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA			B-10



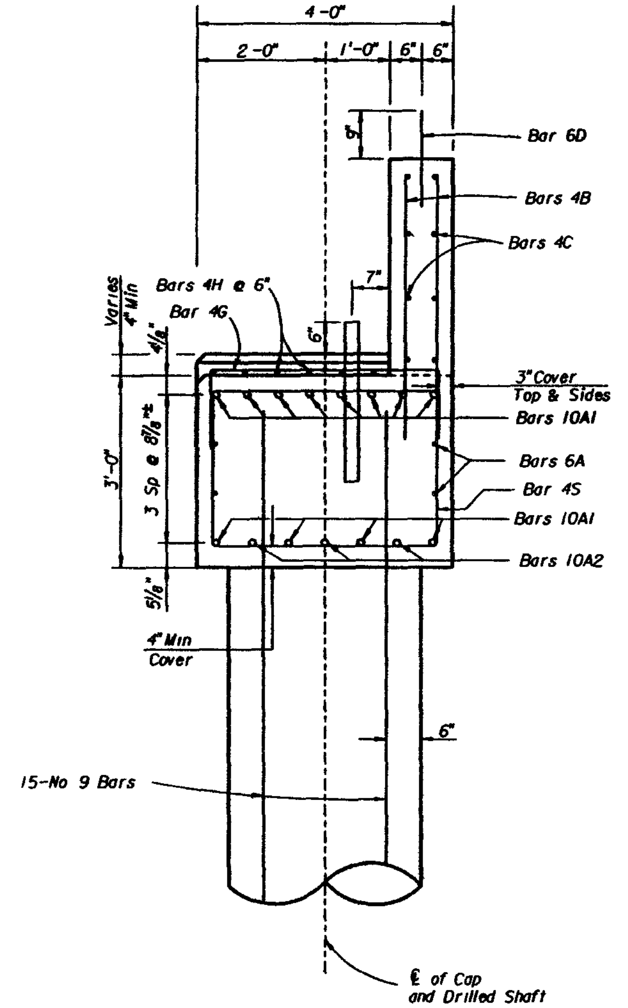
VIEW A-A



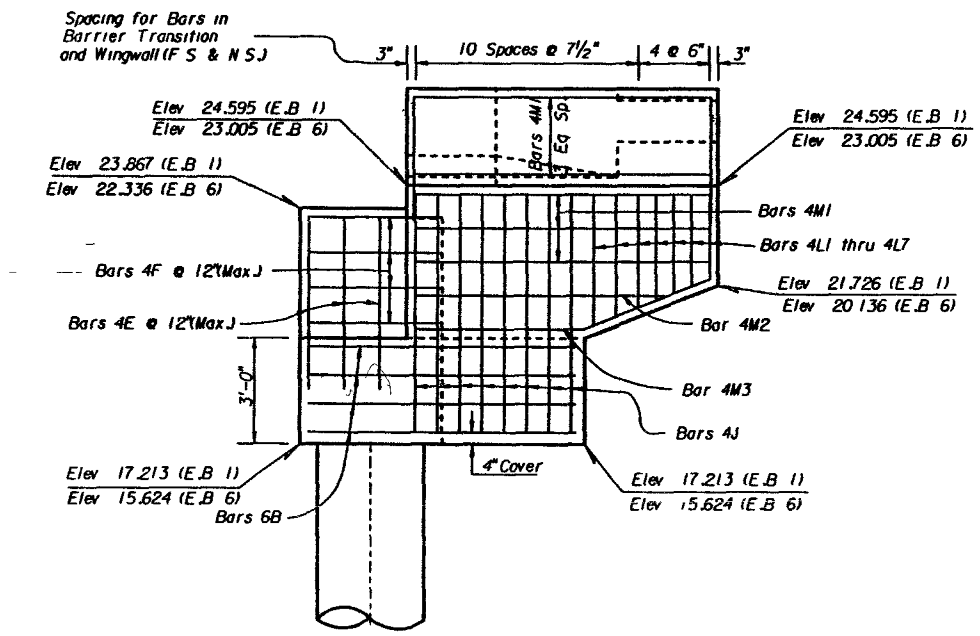
SECTION D-D



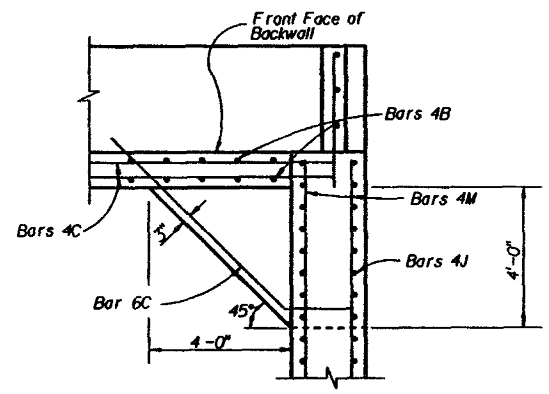
VIEW F-F



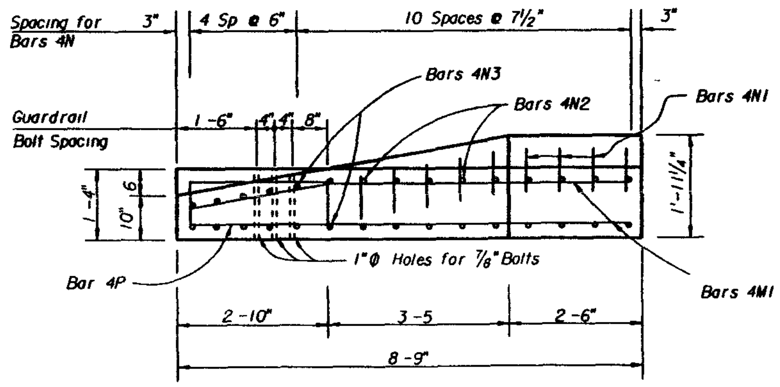
SECTION C-C



VIEW B-B



SECTION E-E



VIEW H-H

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	DKS	1/92
Checked by	PWG	1/92
Designed by	DKS	1/92
Checked by	PWG	1/92
Approved by	[signature]	

ENGINEER OF RECORD:
STRUCTURES DESIGN OFFICE
DISTRICT 3
U.S. 90 East
Chipley, Florida 32428-9990

LOGO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-61	WAKULLA	59010-3519

SHEET TITLE: END BENT DETAILS

PROJECT NAME: SR-61 OVER LOST CREEK

Drawing No.

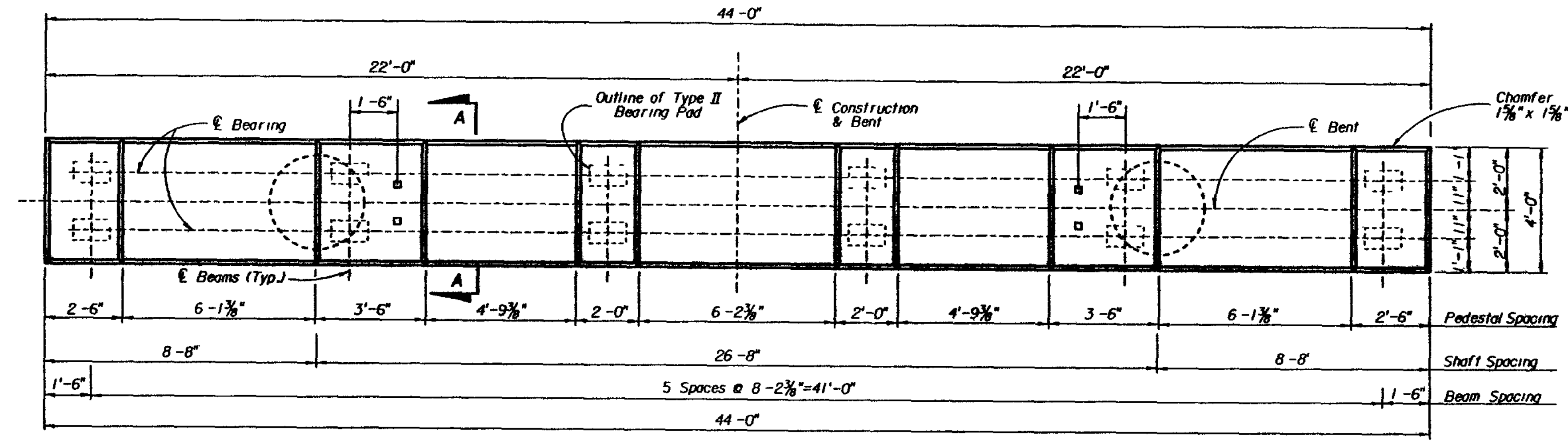
Index No.

/usr2/proj/59010/1519/endbent.dgn

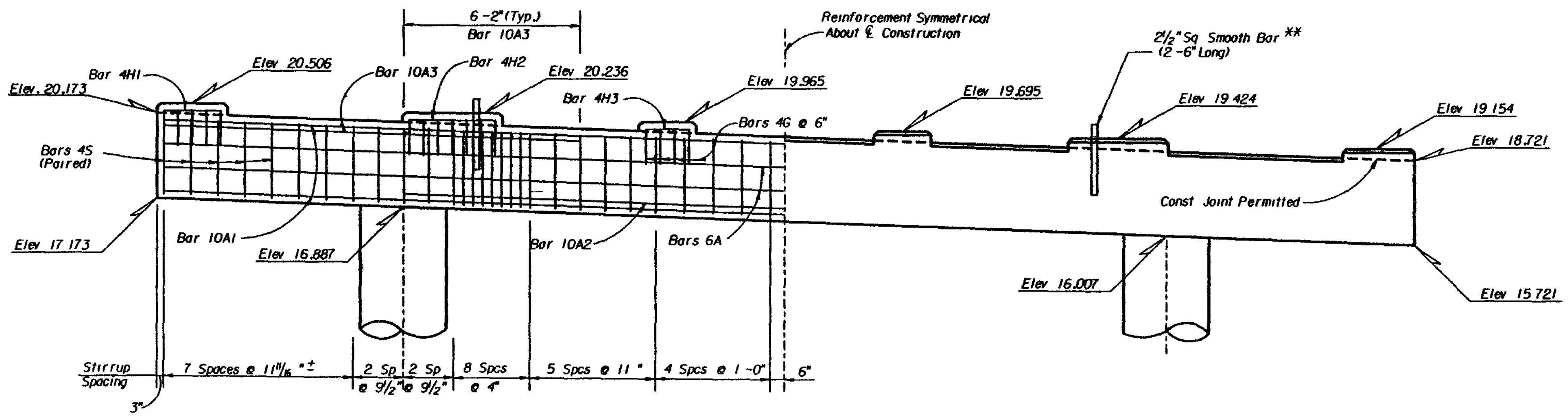
Tue Jul 27 06:06:22 1993

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA			B-11

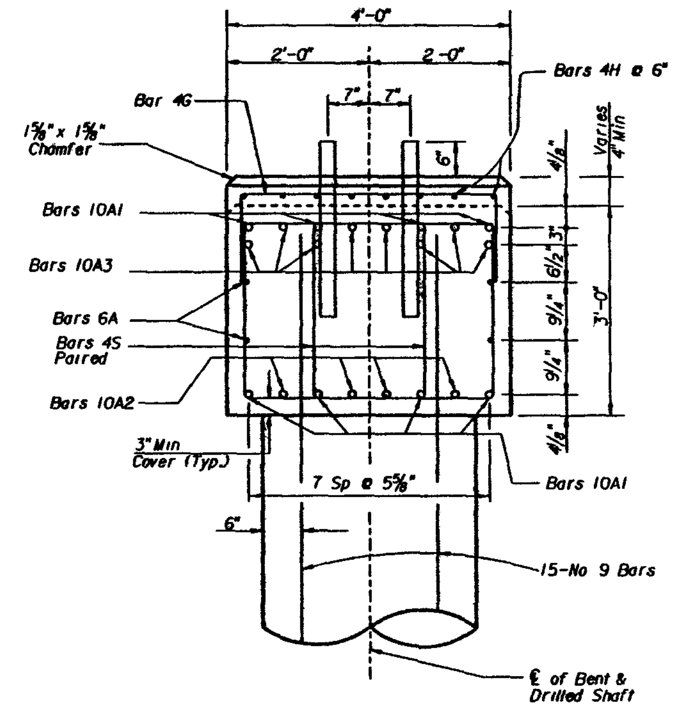
Direction of Stationing



PLAN



ELEVATION



SECTION A-A

* ESTIMATED QUANTITIES		
ITEM	UNIT	QUANTITY
Class II Concrete (Substructure)	C.Y.	20.46
Reinforcing Steel (Substructure)	LB	4346

* Estimated Quantities are for one Bent only

** The cost of the 2½" Sq Bars to be included in the cost of Item No 460-2-15 (Structual Steel Misc.)

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	KMH	12/91
Checked by	PWG	12/91
Designed by	DKS	12/91
Checked by	PWG	12/91
Approved by	[signature]	

ENGINEER OF RECORD:
STRUCTURES DESIGN OFFICE
DISTRICT 3
U.S. 90 East
Chipley, Florida 32428-9990

LOGO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR-61	WAKULLA	59010-3519

SHEET TITLE: INTERMEDIATE BENTS

PROJECT NAME: SR 61 OVER LOST CREEK

Drawing No.

Index No.

Tue Jul 27 06:08:31 1993 /usr2/proj/59010/1519/ibent.dgn

FISCAL YEAR	SHEET NO.
99	1

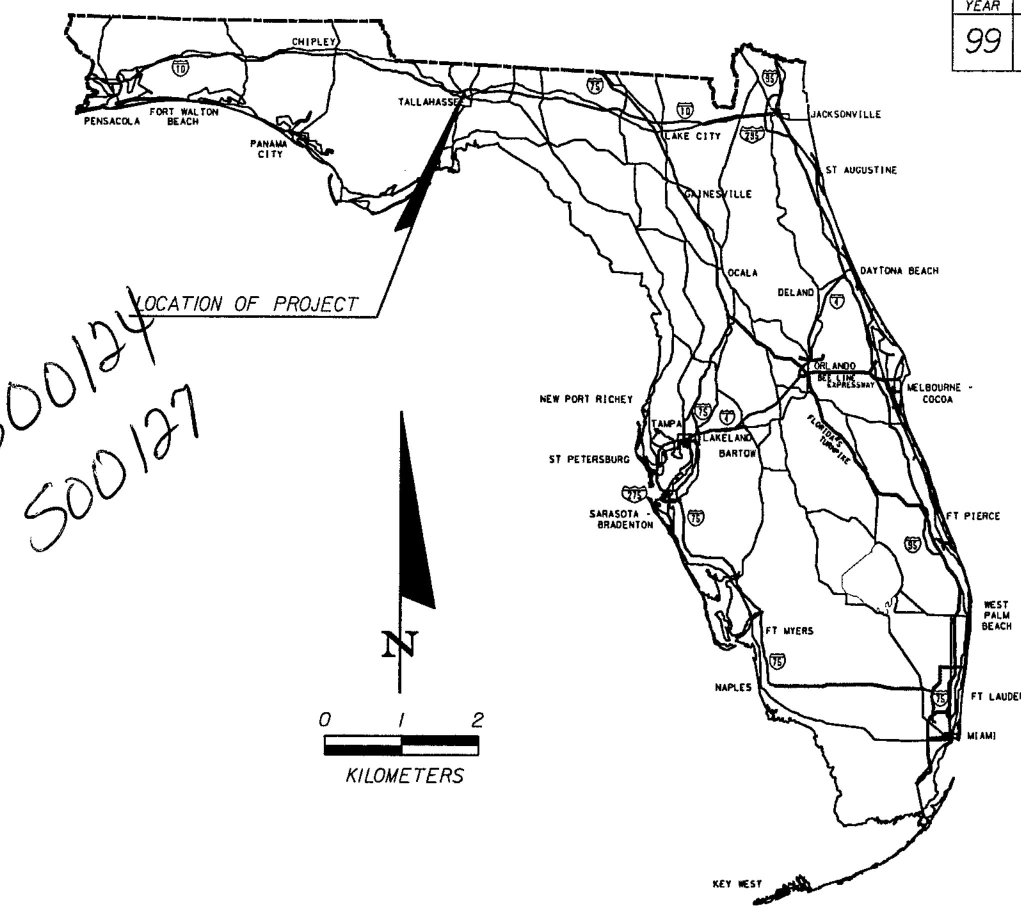
THIS CONTRACT PLAN SET INCLUDES

ROADWAY PLANS
SUMMARY OF PAY ITEMS (5 SHEETS)
SIGNING AND PAVEMENT MARKING PLANS
STRUCTURE PLANS

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION
FINAL
CONTRACT PLANS

FINANCIAL PROJECT ID. 218944-1-52-01
STATE PROJECT NO. 50020-3521
GADSDEN COUNTY
FINANCIAL PROJECT ID. 218944-1-52-02
STATE PROJECT NO. 55010-3521
LEON COUNTY
STATE ROAD NO. 63

500124
500127



LOCATION OF PROJECT

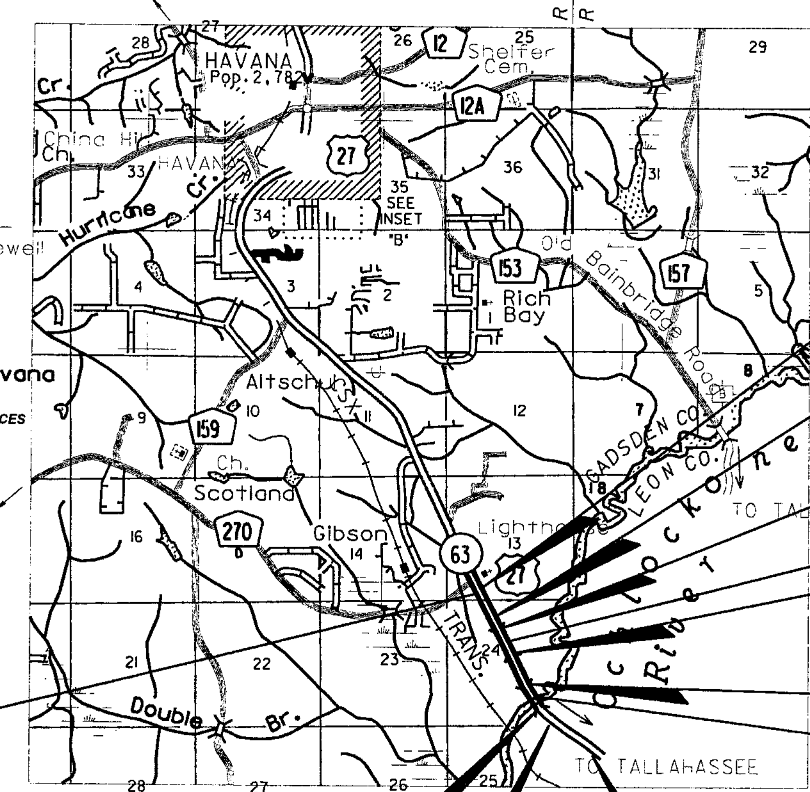
N

0 1 2
KILOMETERS

A DETAILED INDEX APPEARS ON THE KEY SHEET OF EACH COMPONENT SET OF PLANS

INDEX OF ROADWAY PLANS

SHEET NO.	SHEET DESCRIPTION
1	KEY SHEET
2	TYPICAL SECTIONS
3 - 5	SUMMARY OF QUANTITIES
6	DETAILS
7 - 7A	SUMMARY OF DRAINAGE STRUCTURES
8	PROJECT LAYOUT & GEOMETRY
9 - 14	PLAN AND PROFILE
15 - 17	DRAINAGE STRUCTURES
18A,18 - 29	ROADWAY SOIL SURVEY
30 - 42	CROSS SECTIONS
43 - 49	TRAFFIC CONTROL SHEETS
50 - 55	UTILITY ADJUSTMENTS
56	APPROACH SLABS
57 - 59	STORM WATER POLLUTION PROTECTION PLAN
60 - 61	GEOGRID DATA - NICOLON MIRAFI GROUP
62 - 71	GEOGRID DATA - TENSAR, SIERRA



T 3 N / T 2 N

END CONSTRUCTION
STA. 241+74.000

END PROJECT (S.P.N. 50020-3521)
STA. 236+89.492
K.P. 27.077 = M.P. 16.825

END BRIDGE
STA. 234+13.920

NB BRIDGE No. 500125
SB BRIDGE No. 500126

BEGIN BRIDGE
STA. 232+02.060

END BRIDGE
STA. 229+31.020

NB BRIDGE No. 500124
SB BRIDGE No. 500127

STA. EQN 238+77.325 BK = 238+96.320 AH

END S.P.N. 55010-3521
BEGIN S.P.N. 50020-3521
BEGIN BRIDGE
STA. 225+15.110 K.P. 25.903 = M.P. 16.095

BEGIN PROJECT (S.P.N. 55010-3521)
STA. 223+40.000 K.P. 25.728 = M.P. 15.987

BEGIN CONSTRUCTION
STA. 218+75.000

CONTRACTOR:	ANDERSON COLUMBIA
DESIGN CONSUTANT:	CONNELLY & WICKER
CEI:	PARSONS BRINCKERHOFF CONST. SERVICES
DISTRICT SECRETARY:	EDWARD PRESCOTT
RESIDENT ENGINEER:	BRYAN ESTOCK
PROJECT ENGINEER:	TY SMITH
WORK COMPLETED:	11/15/2001
WORK CONDITIONALLY ACCEPTED:	11/16/2001
WORK ACCEPTED:	11/15/2001

ROADWAY PLANS
ENGINEER OF RECORD
RICHARD C. WELCH, P.E.
1711 SOUTH 5TH STREET
JACKSONVILLE BEACH, FL. 32250

PLANS PREPARED BY
CONNELLY & WICKER INC.
1711 SOUTH 5TH STREET
JACKSONVILLE BEACH, FL. 32250
VENDOR # 592343737001

THIS SET OF PLANS TO BE LET WITH FINANCIAL PROJECT ID.'S:
218944-1-56-01
218944-1-56-02

ATTENTION IS DIRECTED TO THE FACT THAT THESE PLANS MAY HAVE BEEN ALTERED IN SIZE BY REPRODUCTION. THIS MUST BE CONSIDERED WHEN OBTAINING SCALED DATA.

GOVERNING SPECIFICATIONS: STATE OF FLORIDA, DEPARTMENT OF TRANSPORTATION, STANDARD SPECIFICATIONS, DATED 1999 SUPPLEMENTS AND SPECIAL PROVISIONS THERETO IF NOTED IN THE CONTRACT SPECIFICATIONS FOR THIS PROJECT.

THESE PLANS HAVE BEEN PREPARED IN ACCORDANCE WITH AND ARE GOVERNED BY THE STATE OF FLORIDA, DEPARTMENT OF TRANSPORTATION, ROADWAY AND TRAFFIC DESIGN STANDARDS (BOOKLET DATED JANUARY, 1998).

REVISIONS

3/19/99 RW REVISED NOTES ON SHEETS 4, 8, 43, 56, 61, 63 & A-6
4/15/99 DM REVISED NOTES 10 AND 11 ON SHEET B-34
4/15/99 DM REVISED NOTE 10 ON SHEET C-25

Bryan Estock
12/3/01

NOTE: THIS IS A METRIC UNIT PROJECT

LENGTH OF PROJECT	
	METERS
ROADWAY	721.722
BRIDGES	627.770
NET LENGTH OF PROJ.	1349.492
EXCEPTIONS	0000.000
GROSS LENGTH OF PROJ.	1349.492

REVISIONS

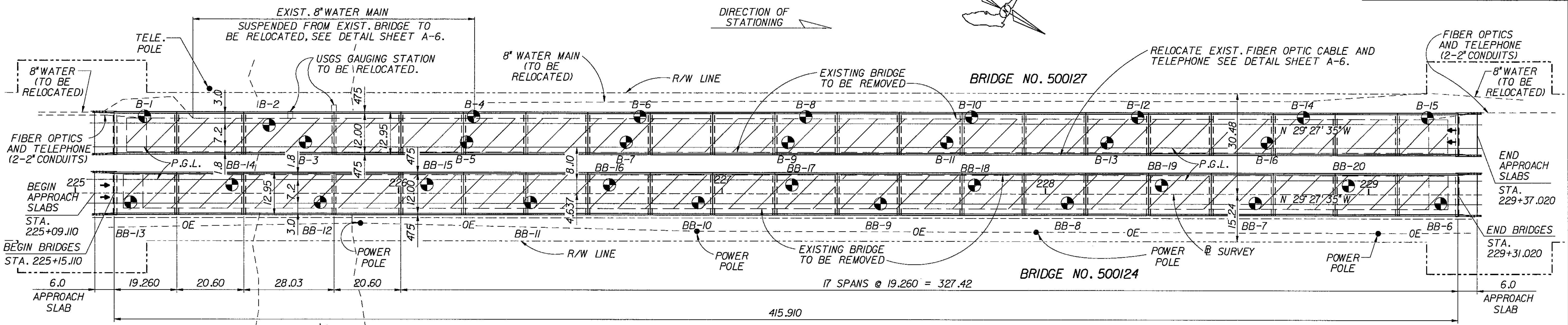
DATE	BY	DESCRIPTION

ROADWAY PLANS APPROVED BY:
DATE: 4/15/99
P.E. NO.: 40279

DESCRIPTION OF PROJECT:
S.R. 63 (U.S. 27), FROM OCHLOCKONEE BRIDGES TO BRIDGE No'S 500008, 500060, 500061, 500062.

FDOT PROJECT MANAGER : KIM WOLIVER, P.E. (PBS&J)

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	50020-3521		B-1

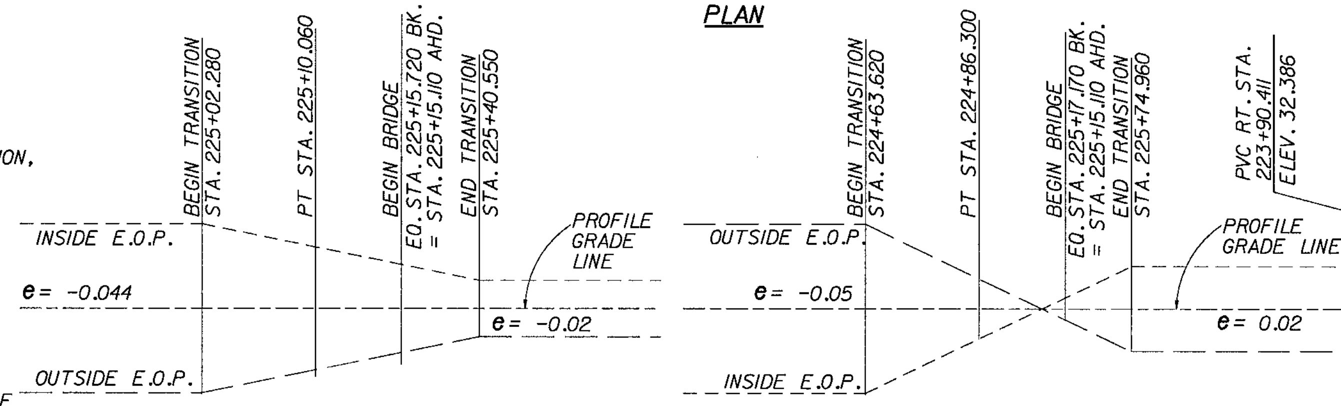


PLAN

S.R. 63 TRAFFIC DATA
DESIGN SPEED 90 km/h

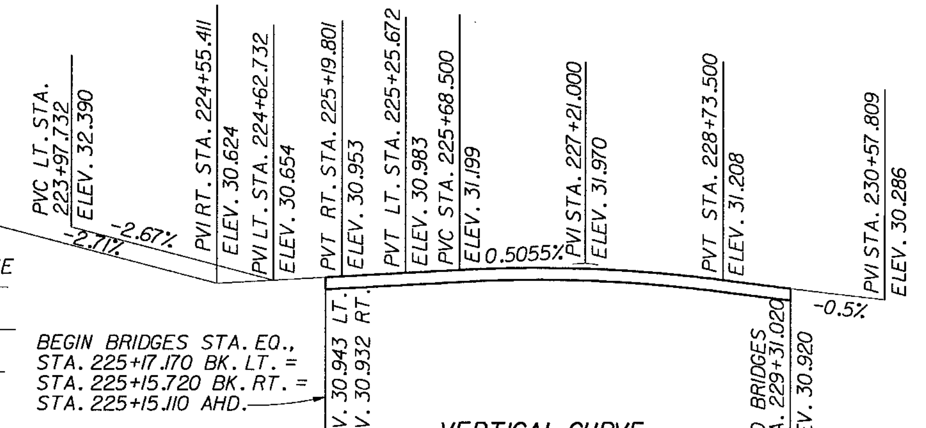
1990 FADT	=	12,400
1996 EST. ADT	=	14,800
2001 EST. ADT	=	17,150
2006 EST. ADT	=	19,600
2016 EST. ADT	=	24,450
K	=	11.70%
D	=	50.16%
T	=	11% (24 HR.)
T	=	5% DES. HR.
T	=	4% DES. HR. HEAVY
T	=	1% DES. HR. MEDIUM

1. FOR LIMITS OF RIPRAP SCOUR PROTECTION, SEE SHEET B-31.
2. APPROX. AREA OF EXIST. BRIDGES & APPROACH SLABS TO BE REMOVED = 8 468.8 m2.
3. SEE SHEETS B-4 THRU B-14 FOR BORING LOGS.
4. FOR APPROACH SLABS & GUARDRAIL SEE ROADWAY PLANS.
5. EXIST. POWER POLES WITH OVERHEAD ELECTRIC ARE TO BE REMOVED PRIOR TO CONSTRUCTION BY OTHERS.
6. ALL UTILITIES ARE EXIST. AND SHALL BE RELOCATED AS CALLED OUT ON PLAN.

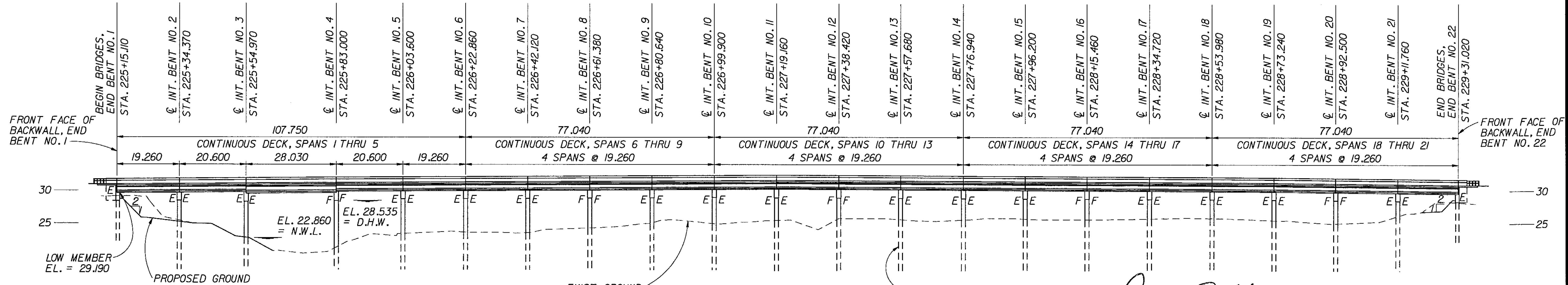


SUPERELEVATION TRANSITION DIAGRAM BRIDGE NO. 500124

SUPERELEVATION TRANSITION DIAGRAM BRIDGE NO. 500127



VERTICAL CURVE PROFILE GRADE LINE BRIDGE NO'S. 500124 & 500127



ELEVATION

E = EXPANSION (NOT FIXED AGAINST LONGITUDINAL MOVEMENT)
F = FIXED (FIXED AGAINST LONGITUDINAL MOVEMENT)

Duane R. Merrell
1/4/99

J:\929208-7\STRUCT\MAIN\PLELDGN 8DEC98 9:12:50 SCALE:600 USER:

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	E.D.L.	9/98
Checked by	L.W.M.	9/98
Designed by	D.R.M.	9/98
Checked by	W.H.M.	9/98
Approved by	D.R.MERRELL	

ENGINEER OF RECORD:
DUANE R. MERRELL
P.O. Box 51343
Jacksonville Beach, Florida
32240-1343

LOGO:
Connelly & Wicker Inc. Consulting Engineers
Jacksonville Beach, Florida

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	FINANCIAL PROJECT ID.
SR 63	GADSDEN	218944-1-52-01

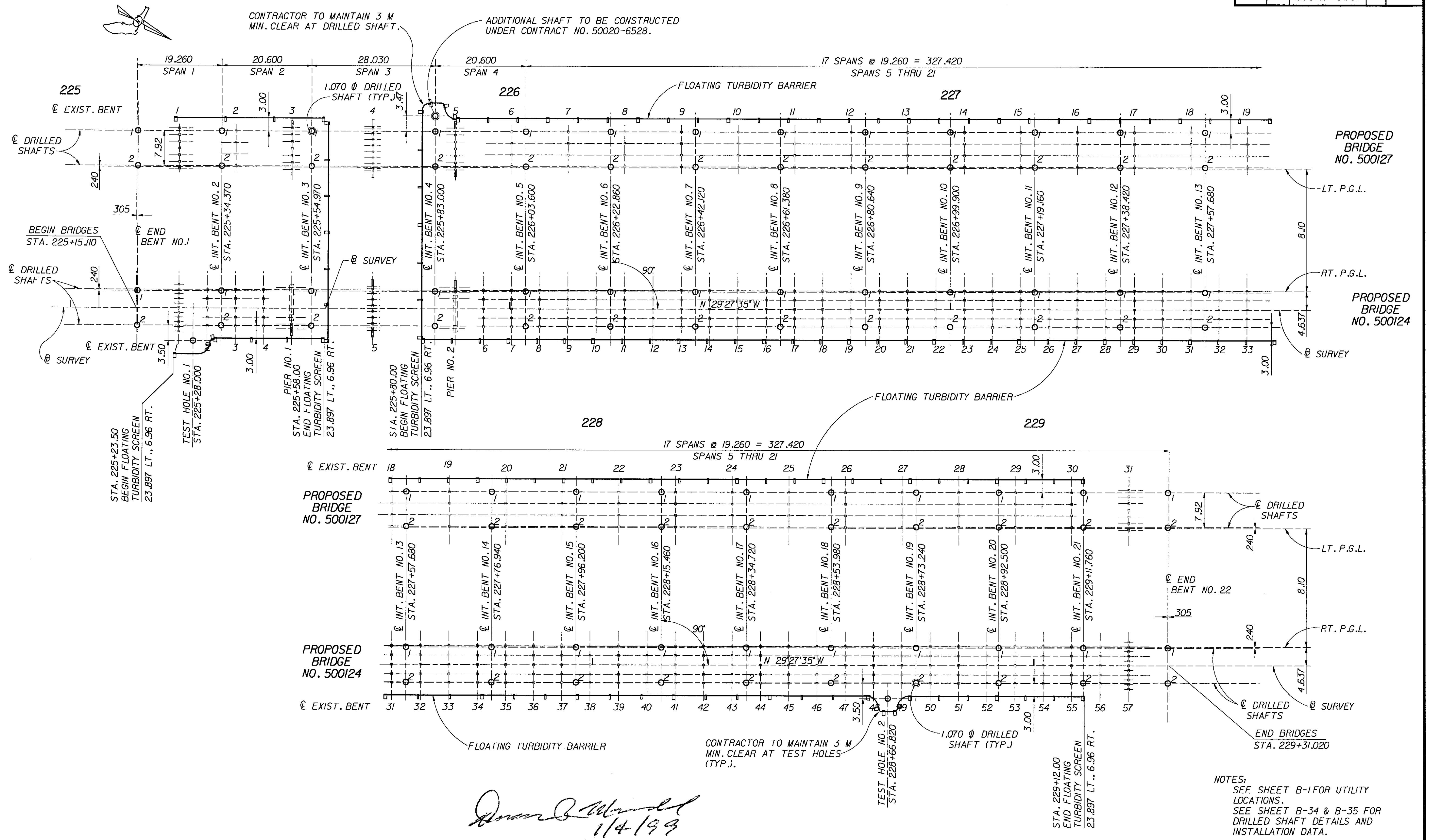
SHEET TITLE: PLAN AND ELEVATION

PROJECT NAME: SR 63 OVER OCHLOCKONEE RIVER BRIDGE NO'S. 500124 & 500127

Drawing No.

Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	50020-3521		B-15



NOTES:
SEE SHEET B-1 FOR UTILITY LOCATIONS.
SEE SHEET B-34 & B-35 FOR DRILLED SHAFT DETAILS AND INSTALLATION DATA.

1/4/99

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	E.D.L.	9/98
Checked by	L.W.M.	9/98
Designed by	D.R.M.	9/98
Checked by	W.H.M.	9/98
Approved by	D.R.MERRELL	

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STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	FINANCIAL PROJECT ID.
SR 63	GADSDEN	218944-1-52-01

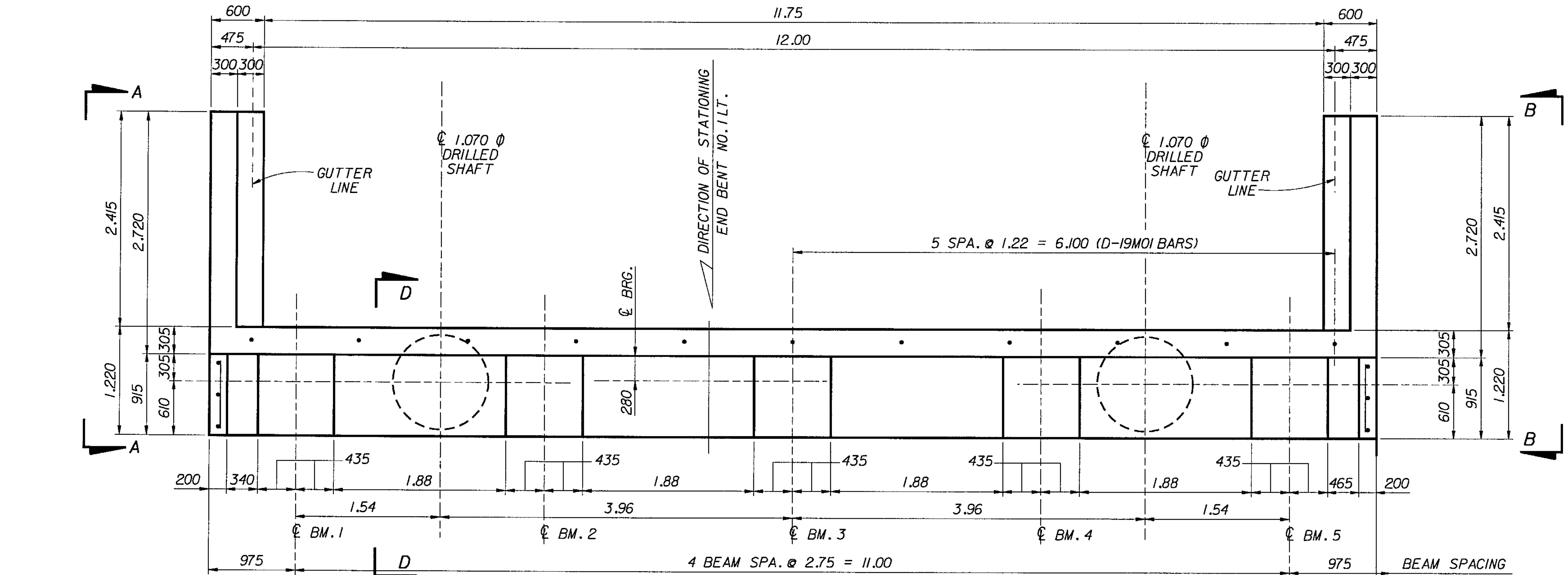
SHEET TITLE: FOUNDATION PLAN & TURBIDITY BARRIER LOCATIONS

PROJECT NAME: SR 63 OVER OCHLOCKONEE RIVER BRIDGE NOS. 500124 & 500127

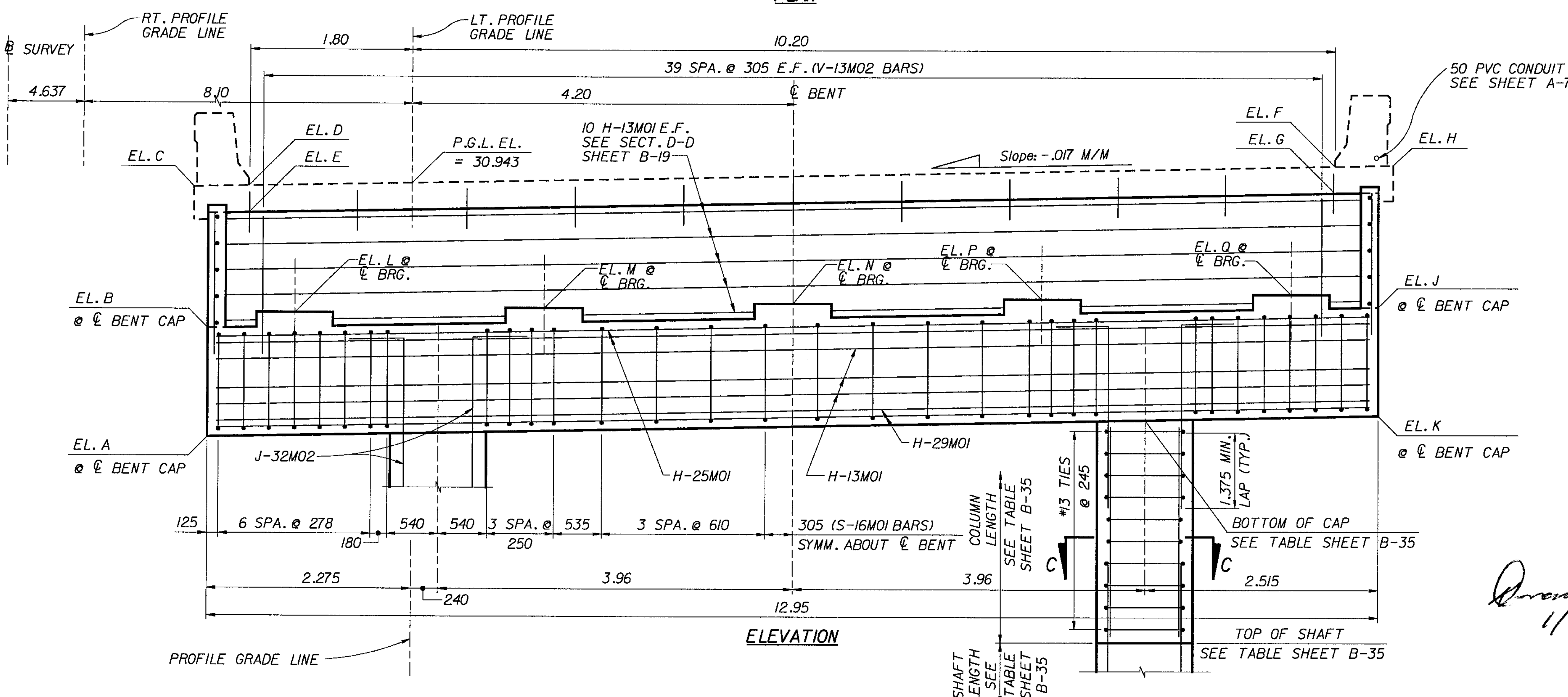
Drawing No.

Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	50020-3521		B-16



PLAN



ELEVATION

ELEVATIONS	
LOCATION	E. B. NO. 1 LT.
EL. A	28.093
EL. B	29.313
EL. C	30.904
EL. D	30.912
EL. E	30.607
EL. F	31.116
EL. G	30.811
EL. H	31.124
EL. J	29.530
EL. K	28.310
EL. L	29.494
EL. M	29.540
EL. N	29.586
EL. P	29.632
EL. Q	29.678

UNLESS OTHERWISE NOTED ALL ELEVATIONS ARE TAKEN AT THE FRONT FACE OF THE BACKWALL

ESTIMATED QUANTITIES		
ITEM	UNIT	QUANTITY
Class IV Concrete (Substructure)	m3	34.6
Reinforcing Steel (Substructure) *	kg	3 353

* Includes 313 kg for column steel, see estimated quantities (per lin. meter) sheet B-34.

BREAKDOWN OF CONCRETE QUANTITIES		
ITEM	UNIT	QUANTITY
Shaft Cap & Wing Footing	m3	23.5
Back Wall	m3	5.2
Wing Wall & Cheek Wall (both sides)	m3	2.4
Columns (3.898)	m3	3.5

Duane R. Merrell
1/4/99

NOTES:
FOR VIEWS A-A, B-B & SECTION D-D SEE SHEET NO. B-19.
FOR SECTION C-C SEE SHEET NO. B-34.
FOR PEDESTAL REINFORCING SEE SHEET NO. B-18.

J:\92\9208-7\STRUCT\MAIN\EBNT1LT.DGN 8DEC98 15:19:36 SCALE:30 USER:

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	E.D.L.	9/98
Checked by	L.W.M.	9/98
Designed by	D.R.M.	9/98
Checked by	W.H.M.	9/98
Approved by	D.R.MERRELL	

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FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	FINANCIAL PROJECT ID.
SR 63	GADSDEN	218944-1-52-01

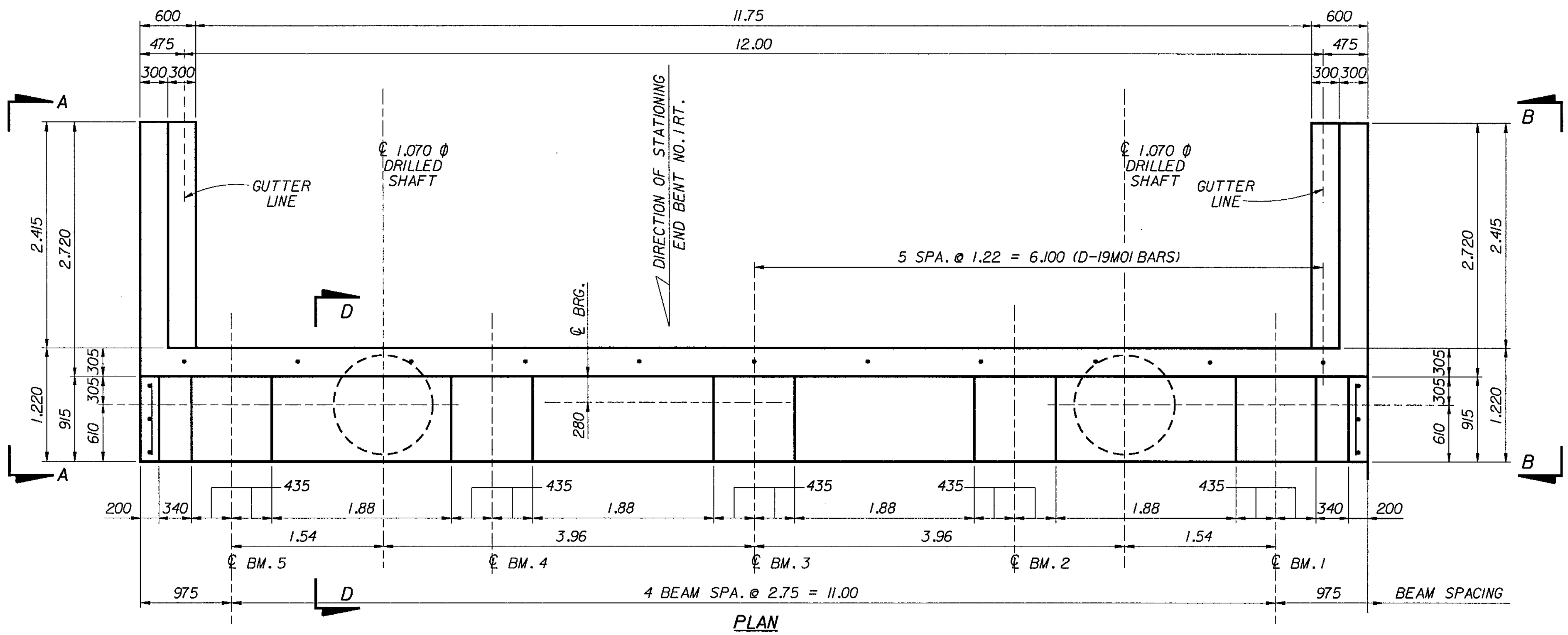
SHEET TITLE: END BENT NO. 1 LT.

Drawing No.

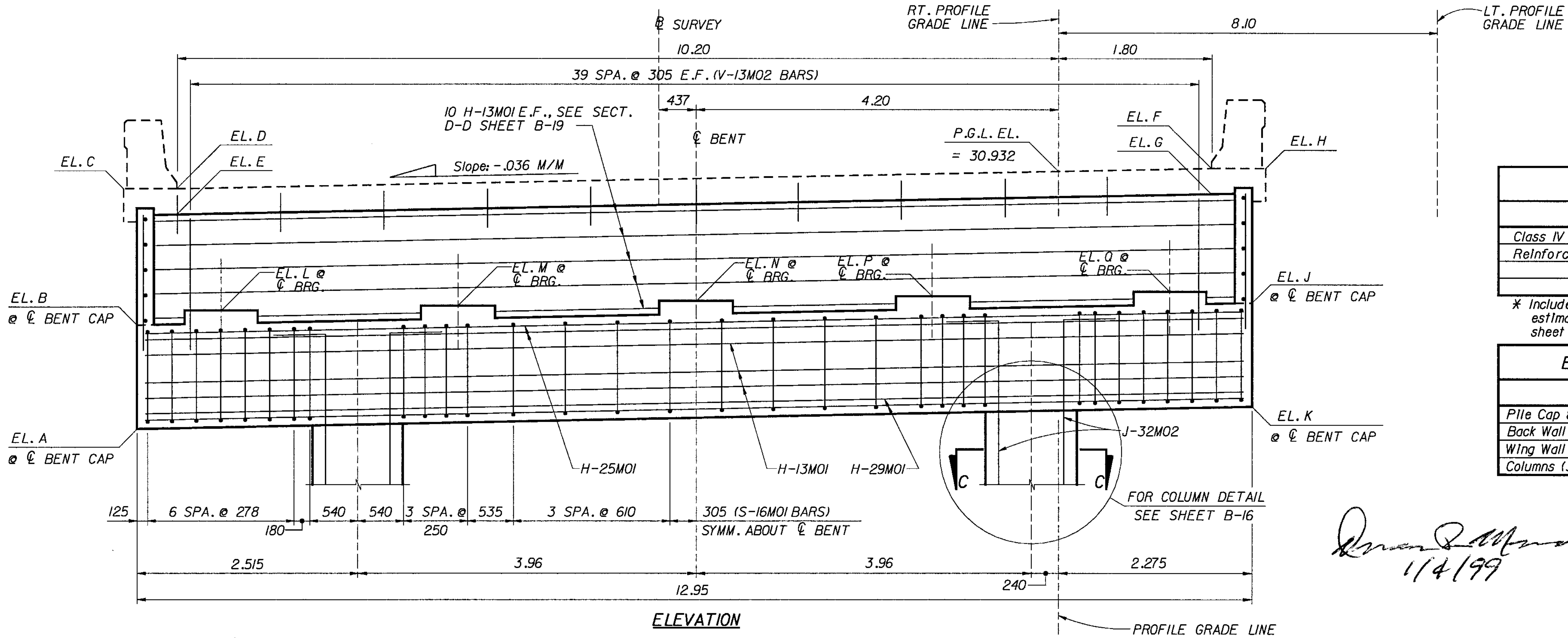
PROJECT NAME: SR 63 OVER OCHLOCKONEE RIVER BRIDGE NO. 500127

Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	50020-3521		B-17



PLAN



ELEVATION

ELEVATIONS	
LOCATION	E. B. NO. 1 RT.
EL. A	27.742
EL. B	28.962
EL. C	30.551
EL. D	30.567
EL. E	30.262
EL. F	30.996
EL. G	30.691
EL. H	31.013
EL. J	29.422
EL. K	28.202
EL. L	29.160
EL. M	29.258
EL. N	29.356
EL. P	29.453
EL. Q	29.551

UNLESS OTHERWISE NOTED ALL ELEVATIONS ARE TAKEN AT THE FRONT FACE OF THE BACKWALL

ESTIMATED QUANTITIES		
ITEM	UNIT	QUANTITY
Class IV Concrete (Substructure)	m3	34.2
Reinforcing Steel (Substructure) *	kg	3 317

* Includes 277 kg for column steel, see estimated quantities (per lin. meter) sheet B-34.

BREAKDOWN OF CONCRETE QUANTITIES		
ITEM	UNIT	QUANTITY
Pile Cap & Wing Footing	m3	23.5
Back Wall	m3	5.2
Wing Wall & Cheek Wall (both sides)	m3	2.4
Columns (3.443)	m3	3.1

Duane R. Merrell
1/4/99

NOTES:
FOR PEDESTAL REINFORCING SEE SHEET B-18.
FOR VIEWS A-A, B-B & SECTION D-D SEE SHEET NO. B-19.
FOR SECTION C-C SEE SHEET NO. B-34.

J:\929208-7\STRUCT\MAIN\EBNT1RT.DGN 8DEC98 15:24:08 SCALE:30 USER:

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	E.D.L.	9/98
Checked by	L.W.M.	9/98
Designed by	D.R.M.	9/98
Checked by	W.H.M.	9/98
Approved by	D.R.MERRELL	

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FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	FINANCIAL PROJECT ID.
SR 63	GADSDEN	218944-1-52-01

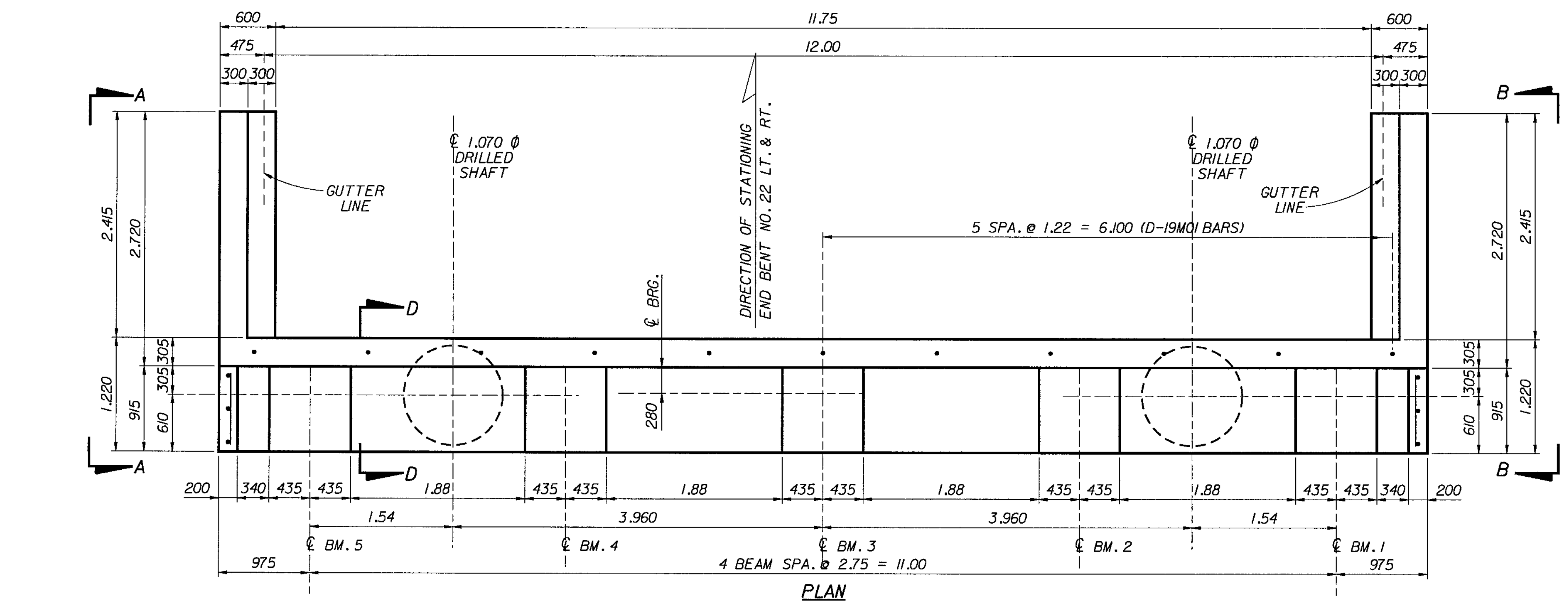
SHEET TITLE: END BENT NO. 1 RT.

PROJECT NAME: SR 63 OVER OCHLOCKONEE RIVER BRIDGE NO. 500124

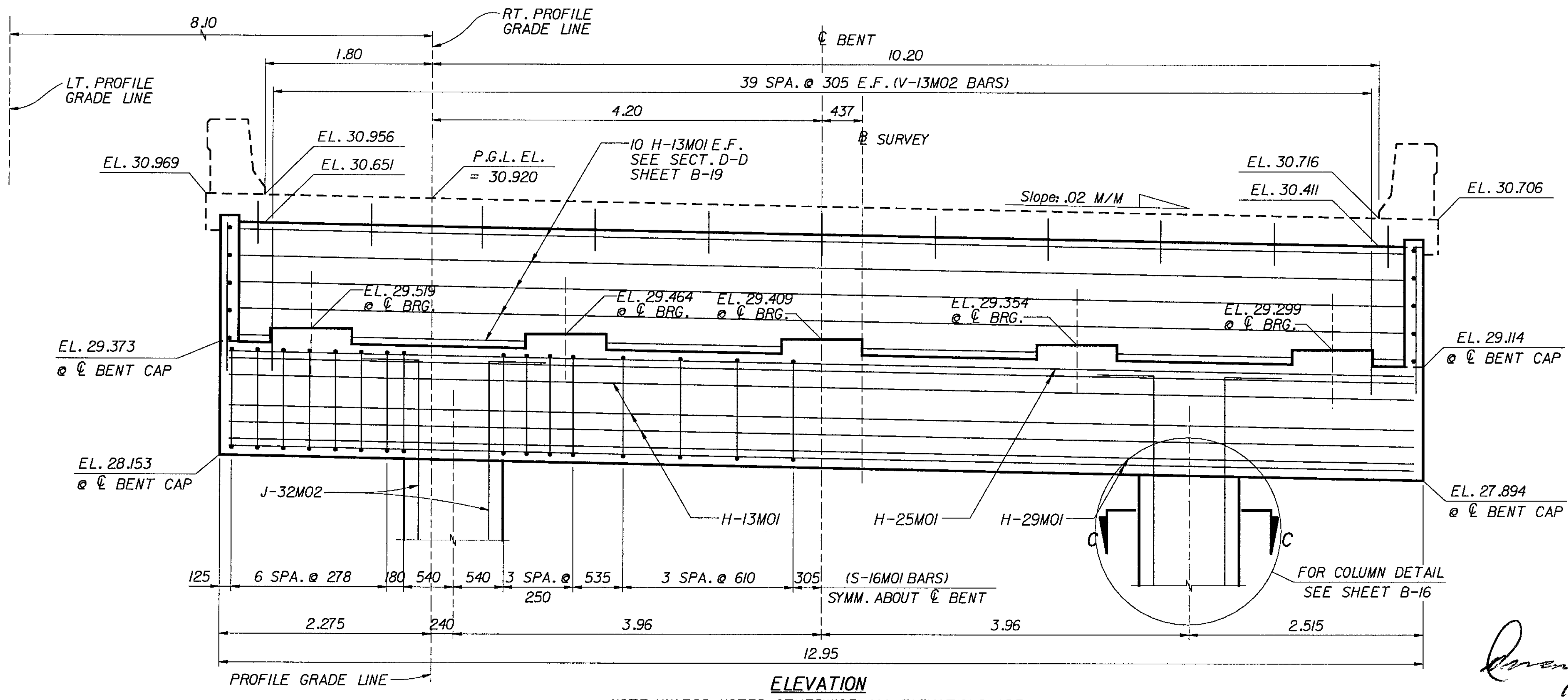
Drawing No.

Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	50020-3521		B-18

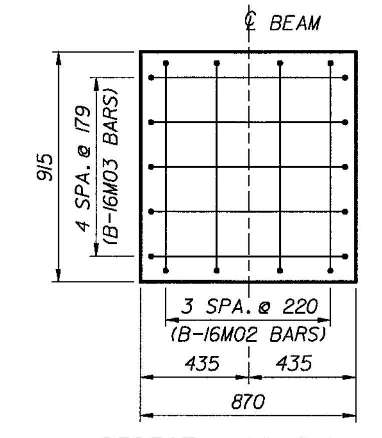


PLAN



ELEVATION

NOTE: UNLESS NOTED OTHERWISE ALL ELEVATIONS ARE TAKEN AT THE FRONT FACE OF BACKWALL.



PEDESTAL REINFORCING

ESTIMATED QUANTITIES ✱

ITEM	UNIT	QUANTITY
Class IV Concrete (Substructure)	m3	34.2
Reinforcing Steel (Substructure) ✱✱	kg	3 326

✱ Quantities shown are for one end bent only (2 req'd.)
✱✱ Includes 286 kg for column steel, see estimated quantities (per lin. meter) sheet B-34.

BREAKDOWN OF CONCRETE QUANTITIES

ITEM	UNIT	QUANTITY
Pile Cap & Wing Footing	m3	23.5
Back Wall	m3	5.1
Wing Wall & Cheek Wall (both sides)	m3	2.4
Columns (3.548)	m3	3.2

Duane R. Merrell
1/4/99

NOTES:
FOR VIEWS A-A, B-B & SECTION D-D SEE SHEET NO. B-19.
FOR SECTION C-C SEE SHEET NO. B-34.

J:\92\9208-7\STRUCT\MAIN\EBNT22.DGN 8DEC98 15:29:46 SCALE:30 USER:

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	E.D.L.	9/98
Checked by	L.W.M.	9/98
Designed by	D.R.M.	9/98
Checked by	W.H.M.	9/98
Approved by	D.R. MERRELL	

ENGINEER OF RECORD:
DUANE R. MERRELL
P.O. Box 51343
Jacksonville Beach, Florida
32240-1343

LOGO:
Connelly & Wicker Inc.
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Jacksonville Beach, Florida

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

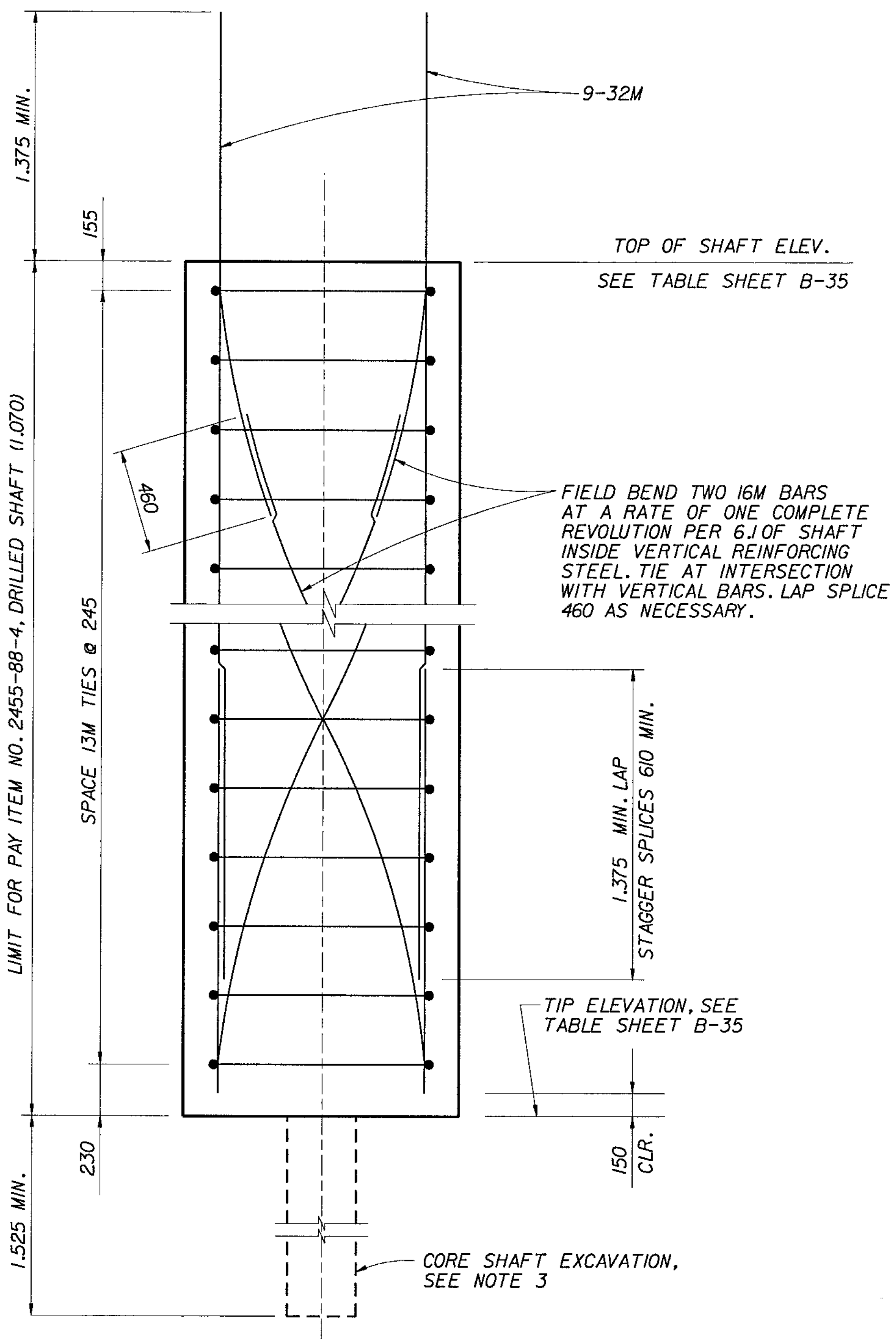
ROAD NO.	COUNTY	FINANCIAL PROJECT ID.
SR 63	GADSDEN	218944-1-52-01

SHEET TITLE:
END BENT NO. 22 RT.
END BENT NO. 22 LT. OPPOSITE HAND

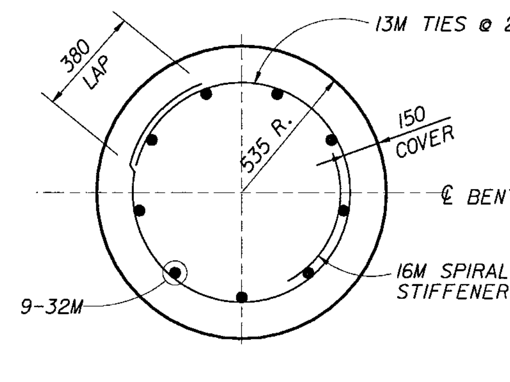
PROJECT NAME:
SR 63 OVER OCHLOCKONEE RIVER
BRIDGE NO'S. 500124 & 500127

Drawing No.

Index No.



ELEVATION
1.070 ∅ DRILLED SHAFT



SECTION C-C
1.070 ∅ DRILLED SHAFT

ESTIMATED QUANTITIES		
ITEM	UNIT	QUANTITY
Drilled Shaft (1.070) Item No. 2455-88-4	m	1 418.3
Core Shaft Excavation Item No. 2455-111	m	238.0
Unclassified Shaft Excavation (1.070) Item No. 2455-122-4	m	1 279.4
Drilled Shaft Sidewall Overreaming (1.070) Item No. 2455-124 *	m	141.9
Extra Depth Excavation (1.070) Item No. 2455-125-4 *	m	141.9

* Drilled Shaft Sidewall Overreaming and Extra Depth Excavation are contingency items and are for bidding purposes only.

NOTES:

1. CONCRETE FOR DRILLED SHAFTS SHALL BE CLASS IV (DRILLED SHAFT).
2. REINFORCING STEEL SHOWN SHALL CONFORM TO ASTM A-615M, GRADE 420 (Fy = 400MPa) AND SHALL BE PAID FOR UNDER PAY ITEM NO. 2455-88-4, DRILLED SHAFT.
3. THE CONTRACTOR SHALL PERFORM A 100 TO 150 DIAMETER SHAFT CORE AND/OR A STANDARD PENETRATION TEST BORING, CONTINUOUS SAMPLING, FOR A MINIMUM DEPTH OF 1.525 METERS BELOW THE BOTTOM OF THE DRILLED SHAFT EXCAVATION OR AS DIRECTED BY THE ENGINEER. THE CORE BARREL SHOULD BE DESIGNED SO THAT THE SAMPLE OF MATERIAL CORED CAN BE REMOVED FROM THE SHAFT EXCAVATION AND CORE BARREL IN AN UNDISTURBED STATE AND IN SUFFICIENT LENGTH TO PROVIDE CORE SAMPLES UP TO A DEPTH OF 6.1 METERS BELOW THE BOTTOM OF THE DRILLED SHAFT EXCAVATION.
4. THE CONTRACTOR IS REQUIRED TO CONSTRUCT TWO OUT OF POSITION TEST HOLES TO DEMONSTRATE THE ADEQUACY OF CONSTRUCTION METHODS AND EQUIPMENT. THE TEST HOLE LOCATIONS SHALL BE AS LOCATED ON THE FOUNDATION LAYOUT SHEET OR AS DIRECTED BY THE ENGINEER AND SHALL BE REINFORCED. THE TEST HOLE LENGTHS ARE SHOWN IN THE TABLE. IF THE TEST HOLES ARE RELOCATED FROM THOSE SHOWN, THE ELEVATIONS AND SHAFT LENGTHS MAY VARY FROM THOSE SHOWN IN THE TABLE.
5. DRILLED SHAFTS SHALL BE CONSTRUCTED IN ACCORDANCE WITH SPECIFICATION SECTION B455, STRUCTURES FOUNDATIONS - DRILLED SHAFTS.
6. THE MINIMUM TIP ELEVATIONS SHOWN IN THE TABLE ARE BASED ON THE BORING LOGS. THE ACTUAL GROUND LINE AND ROCK LINE ELEVATIONS MAY VARY. THE CONTRACTOR SHALL MAINTAIN THE MINIMUM ROCK SOCKET LENGTH AS NOTED ON THESE PLANS. MINIMUM ROCK SOCKET LENGTH SHOWN IS THAT REQUIRED TO RESIST LATERAL LOADS AND/OR TO DEVELOP AXIAL CAPACITY.
7. SLURRY TANKS AND DE-SANDING EQUIPMENT SHALL BE REQUIRED, AS DIRECTED BY THE ENGINEER, IN ACCORDANCE WITH SECTION B455.
8. SLUMP LOSS TEST SHALL BE PERFORMED AT THE SITE.
9. CORING SHALL BE PERFORMED IN ACCORDANCE WITH ASTM D 2113 STANDARD PRACTICE FOR DIAMOND CORE DRILLING FOR SITE INVESTIGATION, EXCEPT THAT A SINGLE-TUBE CORE BARREL WILL NOT BE ALLOWED FOR CORING AND RETRIEVING THE UNDISTURBED SAMPLES PRODUCED DURING THE CORE (SHAFT EXCAVATION).
10. A BORING USING STANDARD PENETRATION TESTING AND ROCK CORING, IN ACCORDANCE WITH THE FLORIDA DEPARTMENT OF TRANSPORTATION'S "SOILS AND FOUNDATION MANUAL" (TOPIC NO.: 675-020-012-d), SHALL BE PERFORMED AT EACH OF THE SHAFTS FOR BENTS 1 AND 10 OF STRUCTURE NO. 500127. THE CONTRACTOR SHALL SUPPLY THESE TESTS AT NO ADDITIONAL COST PER METER TO THE DEPARTMENT ABOVE THAT BID FOR CORE (SHAFT EXCAVATION). THE COST OF ROTARY WASHING TO THE SCOUR ELEVATION SHALL BE INCLUDED IN THE COST OF CORE (SHAFT EXCAVATION). THE ESTIMATED LENGTH OF THE BORINGS (EXCLUDING THE ROTARY WASHING LENGTH) AT EACH BENT IS 31.0 METERS.
11. AT THE DIRECTION OF THE ENGINEER, SHAFTS WILL BE INSPECTED PRIOR TO PLACEMENT OF CONCRETE USING THE DEPARTMENT'S SHAFT INSPECTION DEVICE (SID).

ESTIMATED QUANTITIES (PER LIN. METER)		
ITEM	UNIT	QUANTITY
Class IV Concrete (Drilled Shaft)	m3	0.899
Reinforcing Steel (Drilled Shaft)	kg	80.56

Quantity does not include lap splice for No. 32 Vertical Bars. These Quantities are provided for information only. The cost of Concrete and Reinforcing steel shall be included in the Contract Unit Price for Drilled Shafts.

J:\92\9208-7\STRUCT\MAIN\DRSHFT1.DGN 31DEC98 6:17:06 SCALE:20 USER:

REVISIONS					
Date	By	Description	Date	By	Description
4/99	DRM	REVISED NOTES 10 AND 11			

	Names	Dates
Drawn by	E.D.L.	4/99
Checked by	L.W.M.	4/99
Designed by	D.R.M.	4/99
Checked by	W.H.M.	4/99
Approved by	D.R.MERRELL	

ENGINEER OF RECORD:
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LOGO:
Connelly & Wicker Inc.
Consulting Engineers
Jacksonville Beach, Florida

SEAL:
Duane R. Merrell
4/15/99

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	FINANCIAL PROJECT ID.
SR 63	GADSDEN	218944-1-52-01

SHEET TITLE: DRILLED SHAFT DETAILS

PROJECT NAME: SR 63 OVER OCHLOCKONEE RIVER
BRIDGE NO'S. 500124 & 500127

Drawing No.

Index No.

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION
PLANS OF PROPOSED

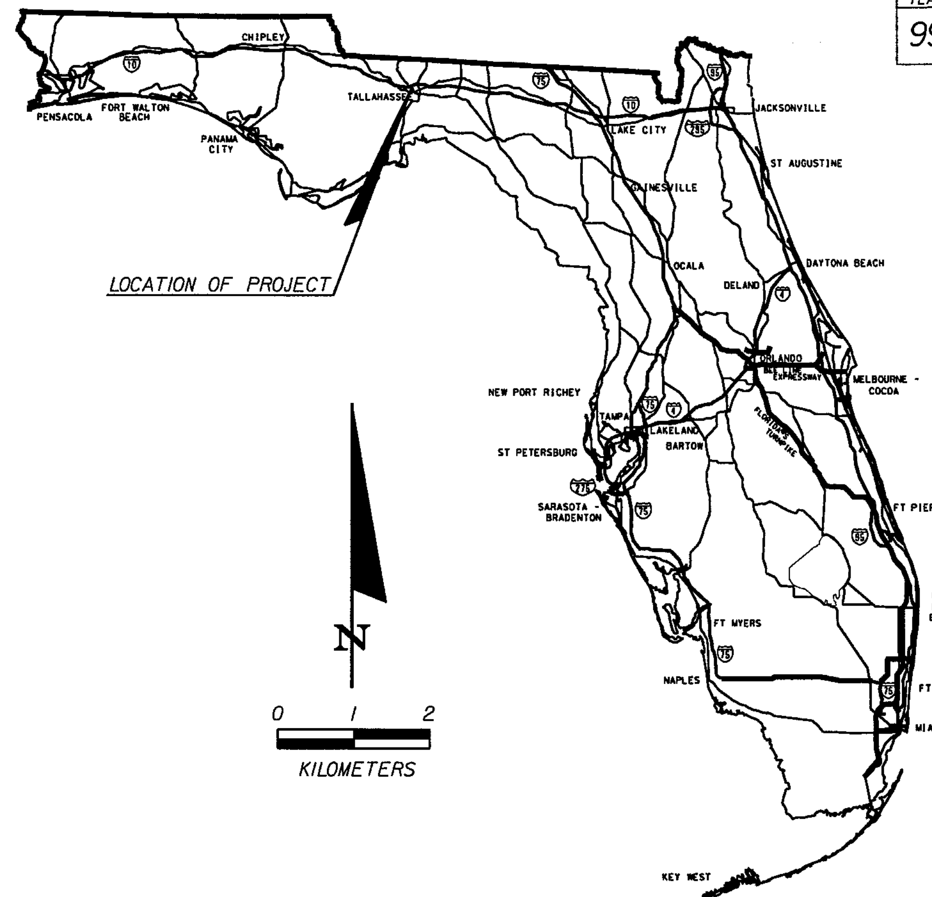
U.S.G.S. GAUGING STATION INSTALLATION PROJECT

FINANCIAL PROJECT ID. 218944-1-56-02

STATE PROJECT NUMBER 50020-6528

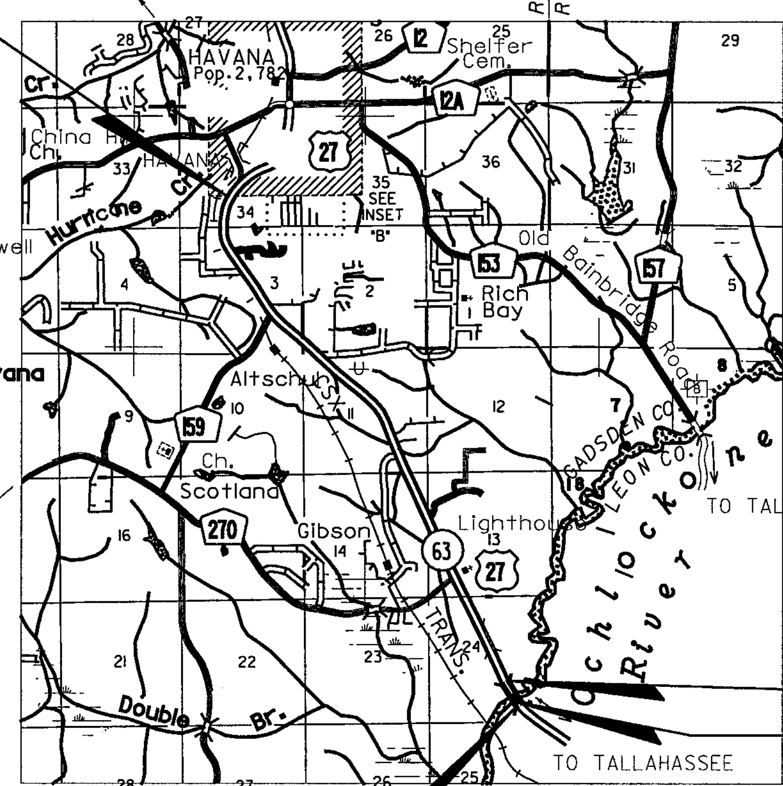
GADSDEN COUNTY

STATE ROAD NO. 63



INDEX OF UTILITY PLANS

SHEET NO.	SHEET DESCRIPTION
1	KEY SHEET
2	GAUGING STATION LOCATION, BENT NO. 4 LEFT
3	1.070 ∅ DRILLED SHAFT DETAILS
4	PRESTRESSED CONCRETE PILES (1 OF 3)
5	PRESTRESSED CONCRETE PILES (2 OF 3)
6	PRESTRESSED CONCRETE PILES (3 OF 3)



UTILITY PLANS ENGINEER OF RECORD

DUANE R. MURRELL, P.E.
1711 SOUTH 5TH STREET
JACKSONVILLE BEACH, FL. 32250

PLANS PREPARED BY

CONNELLY & WICKER INC.
1711 SOUTH 5TH STREET
JACKSONVILLE BEACH, FL. 32250
VENDOR # 592343737001

THIS UTILITY CONTRACT PLAN SET TO BE LET WITH FINANCIAL PROJECT ID.S:
218944-1-52-01
218944-1-52-02
218944-1-56-01

ATTENTION IS DIRECTED TO THE FACT THAT THESE PLANS MAY HAVE BEEN ALTERED IN SIZE BY REPRODUCTION. THIS MUST BE CONSIDERED WHEN OBTAINING SCALED DATA.

GOVERNING SPECIFICATIONS: STATE OF FLORIDA, DEPARTMENT OF TRANSPORTATION, STANDARD SPECIFICATIONS, DATED 1999 SUPPLEMENTS AND SPECIAL PROVISIONS THERETO IF NOTED IN THE CONTRACT SPECIFICATIONS FOR THIS PROJECT.

THESE PLANS HAVE BEEN PREPARED IN ACCORDANCE WITH AND ARE GOVERNED BY THE STATE OF FLORIDA, DEPARTMENT OF TRANSPORTATION, ROADWAY AND TRAFFIC DESIGN STANDARDS (BOOKLET DATED JANUARY, 1998).

REVISIONS

NOTE: THIS IS A METRIC UNIT PROJECT

LENGTH OF PROJECT	
	METERS
ROADWAY	0000.000
BRIDGES	67.890
NET LENGTH OF PROJ.	67.890
EXCEPTIONS	0000.000
GROSS LENGTH OF PROJ.	67.890

REVISIONS		
DATE	BY	DESCRIPTION

ROADWAY PLANS APPROVED BY: [signature]

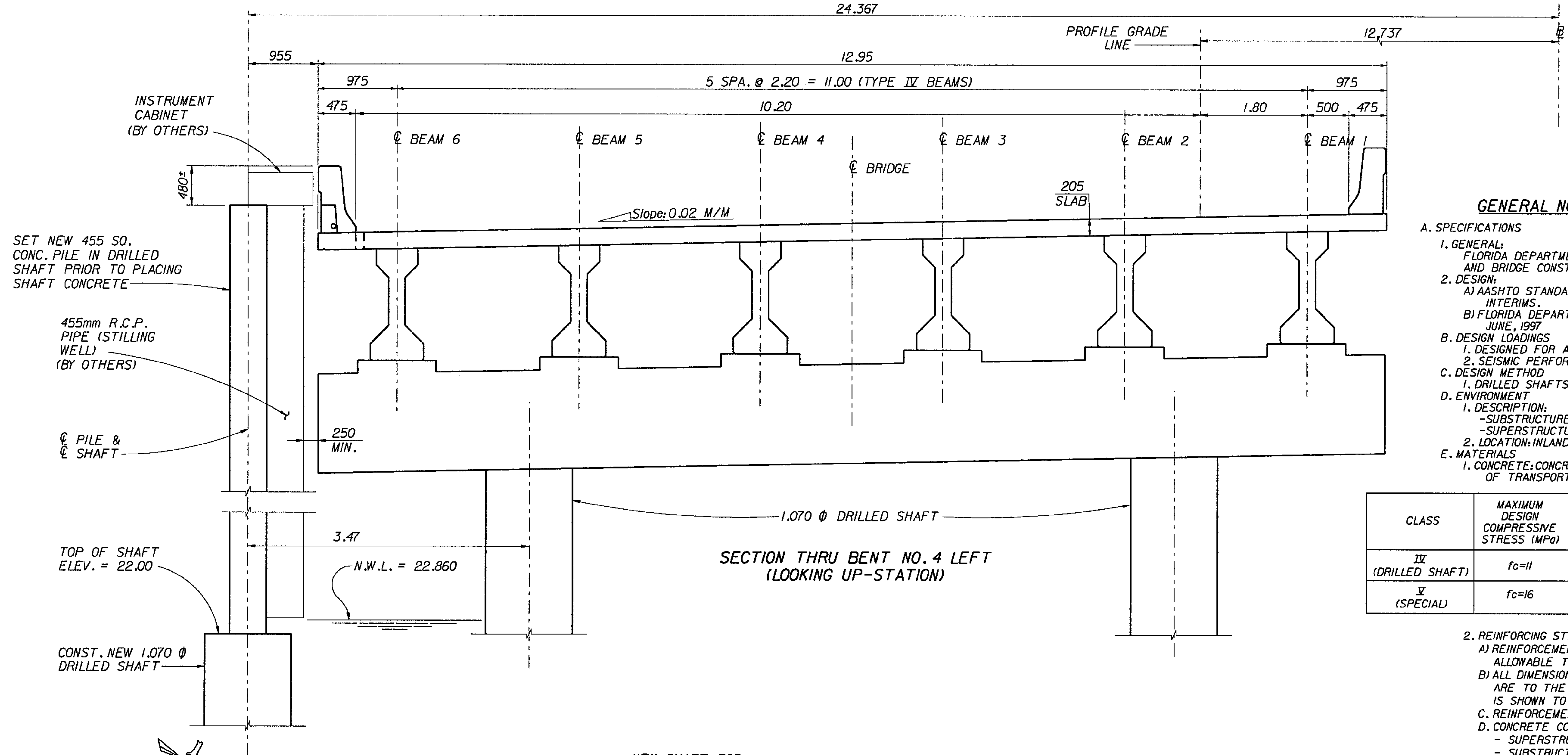
DATE: 1/4/99

P.E. NO.: 36843

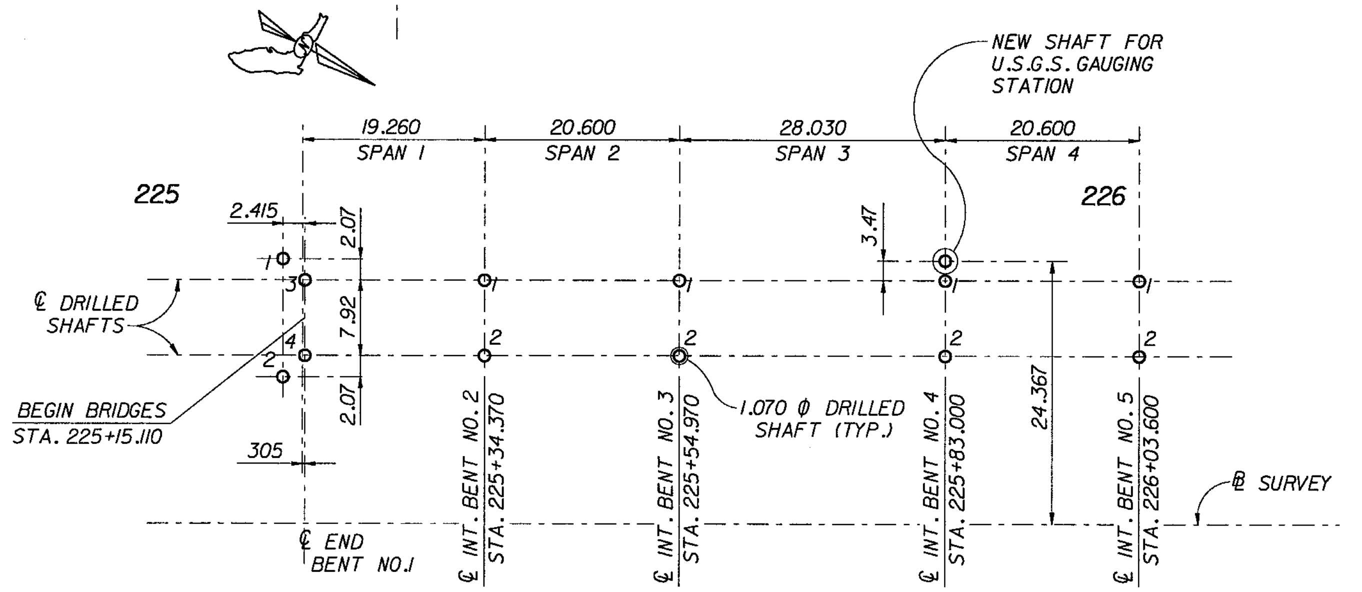
FDOT PROJECT MANAGER : KIM WOLIVER, P.E. (PBS&J)

DESCRIPTION OF PROJECT:
INSTALLATION OF U.S.G.S. GAUGING STATION ON BRIDGE NO. 500127 AT STA. 225+83.000 S.R. 63 (GADSDEN CO.)

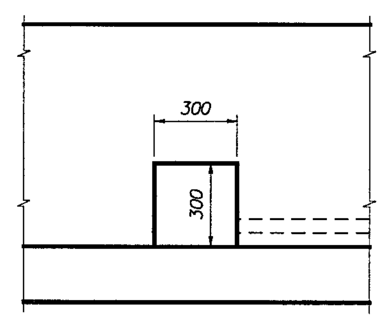
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	50020-6528		2



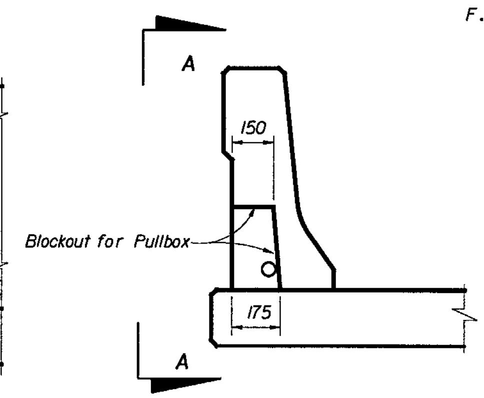
SECTION THRU BENT NO. 4 LEFT
(LOOKING UP-STATION)



PARTIAL DRILLED SHAFT LOCATION PLAN, BRIDGE NO. 500127



VIEW A-A



SECTION THRU BARRIER AT PULLBOX

NOTE:
Shift vertical reinforcement to avoid pullbox. Cut off horizontal reinforcement to provide 50 clearance.

GENERAL NOTES

A. SPECIFICATIONS
1. GENERAL:
FLORIDA DEPARTMENT OF TRANSPORTATION STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION, 1999 AND SUPPLEMENTS THERETO.
2. DESIGN:
A) AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 1992 AND APPLICABLE INTERIMS.
B) FLORIDA DEPARTMENT OF TRANSPORTATION STRUCTURES DESIGN GUIDELINES, JUNE, 1997

B. DESIGN LOADINGS
1. DESIGNED FOR AASHTO LOAD COMBINATION GROUP II.
2. SEISMIC PERFORMANCE CATEGORY A.

C. DESIGN METHOD
1. DRILLED SHAFTS DESIGNED BY THE SERVICE LOAD METHOD.

D. ENVIRONMENT
1. DESCRIPTION:
-SUBSTRUCTURE: EXTREMELY AGGRESSIVE (SOIL PH = 4.5)
-SUPERSTRUCTURE: SLIGHTLY AGGRESSIVE
2. LOCATION: INLAND

E. MATERIALS
1. CONCRETE: CONCRETE SHALL BE IN ACCORDANCE WITH FLORIDA DEPARTMENT OF TRANSPORTATION SUPPLEMENTAL SPECIFICATION SECTION 346.

CLASS	MAXIMUM DESIGN COMPRESSIVE STRESS (MPa)	MINIMUM 28 DAY COMPRESSIVE STRENGTH (MPa)	DESIGN MODULUS OF ELASTICITY (GPa)	LOCATION OF CONCRETE IN STRUCTURE
IV (DRILLED SHAFT)	fc=11	f'c=28	22.8	DRILLED SHAFTS
V (SPECIAL)	fc=16	f'c=41	27.6	PRESTRESSED PILING

2. REINFORCING STEEL:
A) REINFORCEMENT SHALL BE ASTM A615M, GRADE 420 (FY = 400 MPa) ALLOWABLE TENSILE STRESS (fs = 160 MPa)
B) ALL DIMENSIONS PERTAINING TO THE LOCATION OF REINFORCING STEEL ARE TO THE CENTERLINE OF BAR EXCEPT WHERE THE CLEAR DIMENSION IS SHOWN TO FACE OF CONCRETE.
C. REINFORCEMENT DETAIL DIMENSIONS ARE OUT TO OUT OF BARS.
D. CONCRETE COVERS SHALL BE:
- SUPERSTRUCTURE (CIP): 50mm EXCEPT AS NOTED ON PLANS
- SUBSTRUCTURE (CIP): 100mm EXCEPT AS NOTED BELOW
- SUBSTRUCTURE CAST AGAINST EARTH: 115mm
- TOP SURFACE OF BENT CAPS: 75mm

F. DATUM
ALL ELEVATIONS ARE REFERRED TO THE NATIONAL GEODETIC VERTICAL DATUM (NGVD) OF 1929 TRANSLATED DIRECTLY FROM ENGLISH TO SI UNITS.

PAY ITEM NOTES:

1. PAY ITEM NO. 2455-3-2 PILING FURNISHED (PRESTRESSED CONCRETE) (455 SQ.) SHALL INCLUDE ALL LABOR, EQUIPMENT AND MATERIALS NECESSARY TO SET NEW PILE IN DRILLED SHAFT PRIOR TO PLACING SHAFT CONCRETE AND HOLDING PILE IN PLACE UNTIL SHAFT CONCRETE HAS CURED FOR 72 HOURS.
2. PAY ITEM 2102-1 INCLUDES ANY MOT ACTIVITIES NOT COVERED BY FINANCIAL PROJECT ID. 918944-1-52-01 & 918944-1-52-02 MOT.

Duane R. Merrell
1/4/99

J:\96\9601-2\SPDET1.DGN 4JAN99 16:11:27 SCALE:15 USER:

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	E.D.L.	10/98
Checked by	D.R.M.	10/98
Designed by	D.R.M.	10/98
Checked by	D.R.M.	10/98
Approved by	D.R. MERRELL	

ENGINEER OF RECORD:
DUANE R. MERRELL
P.O. Box 51343
Jacksonville Beach, Florida
32240-1343

LOGO:
Connelly & Wicker Inc.
Consulting Engineers
Jacksonville Beach, Florida

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	FINANCIAL PROJECT ID.
SR 63	GADSDEN	218944-1-56-02

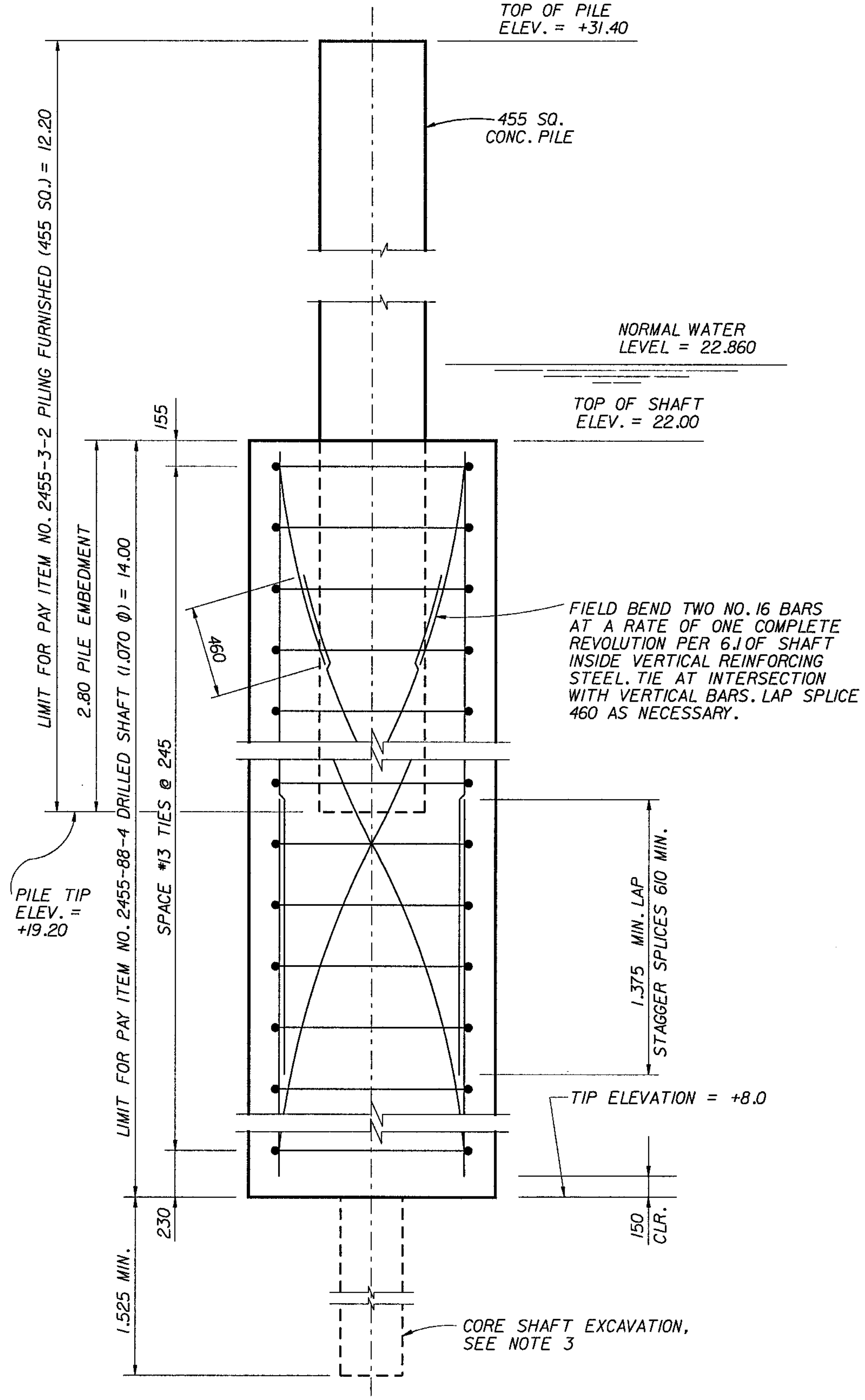
SHEET TITLE: GAUGING STATION LOCATION BENT NO. 4 LEFT

PROJECT NAME: SR 63 OVER OCHLOCKONEE RIVER BRIDGE NO. 500127

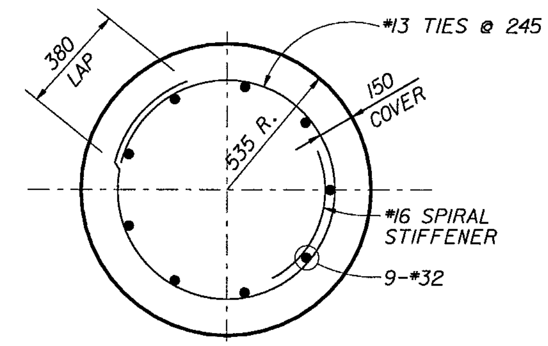
Drawing No.

Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	50020-6528		3



ELEVATION
1.070 Φ DRILLED SHAFT



SECTION C-C
1.070 Φ DRILLED SHAFT

NOTES:

1. CONCRETE FOR DRILLED SHAFTS SHALL BE CLASS IV (DRILLED SHAFT).
2. REINFORCING STEEL SHOWN SHALL CONFORM TO ASTM A-615M, GRADE 420 (Fy = 400MPa) AND SHALL BE PAID FOR UNDER PAY ITEM NO. 2455-88-4, DRILLED SHAFT.
3. THE CONTRACTOR SHALL PERFORM A 100 TO 150 DIAMETER SHAFT CORE AND/OR A STANDARD PENETRATION TEST BORING, CONTINUOUS SAMPLING, FOR A MINIMUM DEPTH OF 1.525 METERS BELOW THE BOTTOM OF THE DRILLED SHAFT EXCAVATION OR AS DIRECTED BY THE ENGINEER. THE CORE BARREL SHOULD BE DESIGNED SO THAT THE SAMPLE OF MATERIAL CORED CAN BE REMOVED FROM THE SHAFT EXCAVATION AND CORE BARREL IN AN UNDISTURBED STATE AND IN SUFFICIENT LENGTH TO PROVIDE CORE SAMPLES UP TO A DEPTH OF 6.1 METERS BELOW THE BOTTOM OF THE DRILLED SHAFT EXCAVATION.
4. DRILLED SHAFTS SHALL BE CONSTRUCTED IN ACCORDANCE WITH SPECIFICATION SECTION B455, STRUCTURES FOUNDATIONS - DRILLED SHAFTS.
5. THE MINIMUM TIP ELEVATION SHOWN IS BASED ON THE BORING LOGS. THE ACTUAL GROUND LINE AND ROCK LINE ELEVATION MAY VARY. THE CONTRACTOR SHALL MAINTAIN THE MINIMUM ROCK SOCKET LENGTH AS NOTED ON THESE PLANS. MINIMUM ROCK SOCKET LENGTH SHOWN IS THAT REQUIRED TO RESIST LATERAL LOADS AND/OR TO DEVELOP AXIAL CAPACITY.
6. SLURRY TANKS AND DE-SANDING EQUIPMENT SHALL BE REQUIRED, AS DIRECTED BY THE ENGINEER, IN ACCORDANCE WITH SECTION B455.
7. SLUMP LOSS TEST SHALL BE PERFORMED AT THE SITE.
8. CORING SHALL BE PERFORMED IN ACCORDANCE WITH ASTM D 2113 STANDARD PRACTICE FOR DIAMOND CORE DRILLING FOR SITE INVESTIGATION, EXCEPT THAT A SINGLE-TUBE CORE BARREL WILL NOT BE ALLOWED FOR CORING AND RETRIEVING THE UNDISTURBED SAMPLES PRODUCED DURING THE CORE (SHAFT EXCAVATION).
9. SHAFT WILL BE INSPECTED PRIOR TO PLACEMENT OF CONCRETE, USING THE DEPARTMENT'S SHAFT INSPECTION DEVICE (SID) IN ACCORDANCE WITH SECTION B455-3.11.3.

ESTIMATED QUANTITIES

ITEM	UNIT	QUANTITY
Piling Furnished (Prestressed Concrete) (455mm Sq.) Item No. 2455-3-2	m	12.20
Drilled Shaft (1.070) Item No. 2455-88-4	m	14.00
Core Shaft Excavation Item No. 2455-111	m	1.5
Unclassified Shaft Excavation (1.070) Item No. 2455-122-4	m	14.00
Drilled Shaft Sidewall Overreaming (1.070) Item No. 2455-124 *	m	1.4
Extra Depth Excavation (1.070) Item No. 2455-125-4 *	m	1.4

* Drilled Shaft Sidewall Overreaming and Extra Depth Excavation are contingency items and are for bidding purposes only.

DRILLED SHAFT INSTALLATION TABLE

TOP OF SHAFT ELEVATION	MIN. TIP ELEVATION	SHAFT LENGTH	SCOUR ELEVATION	MIN. ROCK SOCKET LENGTH
+22.00	+8.00	14.00	+15.849	1.0

For Soil Profile, Refer to Boring B-3 on Sheet B-11 of 50020-3521.

ESTIMATED QUANTITIES (PER LIN. METER)

ITEM	UNIT	QUANTITY
Class IV Concrete (Drilled Shaft)	m3	0.899
Reinforcing Steel (Drilled Shaft)	kg	80.56

Quantity does not include lap splice for No. 32 Vertical Bars. These Quantities are provided for information only. The cost of Concrete and Reinforcing steel shall be included in the Contract Unit Price for Drilled Shafts.

Duane R. Merrell
1/4/99

A:\96\9601-2\SBDRSHFT.DGN 4JAN99 16:18:04 SCALE:20 USER:

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	E.D.L.	10/98
Checked by	D.R.M.	10/98
Designed by	D.R.M.	10/98
Checked by	D.R.M.	10/98
Approved by	D.R.MERRELL	

ENGINEER OF RECORD:
DUANE R. MERRELL
P.O. Box 51343
Jacksonville Beach, Florida
32240-1343

LOGO:
Connelly & Wicker Inc.
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Jacksonville Beach, Florida

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	FINANCIAL PROJECT ID.
SR 63	GADSDEN	218944-1-56-02

SHEET TITLE: DRILLED SHAFT DETAILS

PROJECT NAME: SR 63 OVER OCHLOCKONEE RIVER BRIDGE NO. 500127

Drawing No.

Index No.

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	50020-3521		B-35

DRILLED SHAFT INSTALLATION TABLE (BRIDGE NO. 500124)

PIER OR BENT	SOIL PROFILE	DRILLED SHAFT SIZE (m)	DRILLED SHAFT NUMBER	FACTORED DESIGN LOAD (kN)	BOTTOM OF CAP ELEV. * (m)	TOP OF SHAFT ELEV. ** (m)	TIP ELEV. (m)	MIN. TIP ELEV. (m)	COLUMN LENGTH (m)	SHAFT LENGTH (m)	SCOUR ELEV. (m)	MIN. ROCK SOCKET LENGTH (m)
END BENT 1	BB-13	1.070	1	1633	+28.112	+26.250	+11.3	+11.3	1.862	14.95	+16.764	5.5
			2		+27.831	+26.250	+11.3	+11.3	1.581	14.95	+16.764	5.5
INT. BENT 2	BB-14	1.070	1	2812	+28.200	+26.250	+14.0	+14.0	1.95	12.25	+19.690	5.5
			2		+28.011	+26.250	+14.0	+14.0	1.761	12.25	+19.690	5.5
INT. BENT 3	BB-14	1.070	1	3687	+28.066	+26.250	+10.3	+10.3	1.816	15.95	+15.849	5.5
			2		+27.907	+26.250	+10.3	+10.3	1.657	15.95	+15.849	5.5
INT. BENT 4	BB-12	1.070	1	3687	+28.204	+26.250	+10.3	+10.3	1.954	15.95	+15.849	5.5
			2		+28.046	+26.250	+10.3	+10.3	1.796	15.95	+15.849	5.5
INT. BENT 5	BB-15	1.070	1	2812	+28.533	+26.250	+14.0	+14.0	2.283	12.25	+19.690	5.5
			2		+28.375	+26.250	+14.0	+14.0	2.125	12.25	+19.690	5.5
INT. BENT 6	BB-11	1.070	1	2547	+28.608	+26.250	+14.0	+14.0	2.358	12.25	+19.690	5.5
			2		+28.450	+26.250	+14.0	+14.0	2.20	12.25	+19.690	5.5
INT. BENT 7	BB-11	1.070	1	2958	+28.665	+26.250	+14.0	+14.0	2.415	12.25	+19.690	5.5
			2		+28.507	+26.250	+14.0	+14.0	2.257	12.25	+19.690	5.5
INT. BENT 8	BB-16	1.070	1	2679	+28.710	+26.250	+12.0	+12.0	2.460	14.25	+19.690	2.0
			2		+28.551	+26.250	+12.0	+12.0	2.301	14.25	+19.690	2.0
INT. BENT 9	BB-10	1.070	1	2958	+28.742	+26.250	+9.0	+12.0	2.492	17.25	+19.690	2.0
			2		+28.583	+26.250	+9.0	+12.0	2.333	17.25	+19.690	2.0
INT. BENT 10	BB-10	1.070	1	2547	+28.762	+26.250	+9.0	+12.0	2.512	17.25	+19.690	2.0
			2		+28.603	+26.250	+9.0	+12.0	2.353	17.25	+19.690	2.0
INT. BENT 11	BB-17	1.070	1	2958	+28.770	+26.250	+9.3	+12.0	2.520	16.95	+19.690	2.0
			2		+28.611	+26.250	+9.3	+12.0	2.361	16.95	+19.690	2.0
INT. BENT 12	BB-9	1.070	1	2679	+28.765	+26.250	+12.0	+12.0	2.515	14.25	+19.690	2.0
			2		+28.607	+26.250	+12.0	+12.0	2.357	14.25	+19.690	2.0
INT. BENT 13	BB-9	1.070	1	2958	+28.749	+26.250	+12.0	+12.0	2.499	14.25	+19.690	2.0
			2		+28.590	+26.250	+12.0	+12.0	2.340	14.25	+19.690	2.0
INT. BENT 14	BB-18	1.070	1	2547	+28.720	+26.250	+9.3	+12.0	2.470	16.95	+19.690	2.0
			2		+28.561	+26.250	+9.3	+12.0	2.311	16.95	+19.690	2.0
INT. BENT 15	BB-18	1.070	1	2958	+28.679	+26.250	+9.0	+12.0	2.429	17.25	+19.690	2.0
			2		+28.520	+26.250	+9.0	+12.0	2.270	17.25	+19.690	2.0
INT. BENT 16	BB-8	1.070	1	2679	+28.625	+26.250	+9.3	+12.0	2.375	16.95	+19.690	2.0
			2		+28.467	+26.250	+9.3	+12.0	2.217	16.95	+19.690	2.0
INT. BENT 17	BB-19	1.070	1	2958	+28.560	+26.250	+8.3	+12.0	2.310	17.95	+19.690	2.0
			2		+28.401	+26.250	+8.3	+12.0	2.151	17.95	+19.690	2.0
INT. BENT 18	BB-19	1.070	1	2547	+28.482	+26.250	+8.3	+12.0	2.232	17.95	+19.690	2.0
			2		+28.324	+26.250	+8.3	+12.0	2.074	17.95	+19.690	2.0
INT. BENT 19	BB-7	1.070	1	2958	+28.392	+26.250	+9.0	+12.0	2.142	17.25	+19.690	2.0
			2		+28.234	+26.250	+9.0	+12.0	1.984	17.25	+19.690	2.0
INT. BENT 20	BB-20	1.070	1	2679	+28.296	+26.250	+9.0	+12.0	2.046	17.25	+19.690	2.0
			2		+28.137	+26.250	+9.0	+12.0	1.887	17.25	+19.690	2.0
INT. BENT 21	BB-20	1.070	1	2958	+28.199	+26.250	+9.0	+12.0	1.949	17.25	+19.690	2.0
			2		+28.041	+26.250	+9.0	+12.0	1.791	17.25	+19.690	2.0
END BENT 22	BB-6	1.070	1	1633	+28.103	+26.250	+10.6	+10.6	1.853	15.65	+16.307	5.5
			2		+27.945	+26.250	+10.6	+10.6	1.695	15.65	+16.307	5.5
TEST HOLE 1	BB-14	1.070	—	—	—	+26.250	+8.3	+8.3	—	18.25	15.849	3.0
TEST HOLE 2	BB-7	1.070	—	—	—	+26.250	+9.0	+12.0	—	17.25	19.690	3.0

TIP ELEVATION - THE ELEVATION TO WHICH THE SHAFT SHALL BE CONSTRUCTED UNLESS TEST LOAD DATA, ROCK CORES OR OTHER GEOTECHNICAL DATA OBTAINED DURING CONSTRUCTION ALLOWS THE ENGINEER TO AUTHORIZE A DIFFERENT TIP ELEVATION.

MINIMUM TIP ELEVATION - THE HIGHEST ELEVATION THAT THE SHAFT TIP MAY BE CONSTRUCTED IF ADJUSTMENTS ARE MADE TO THE TIP ELEVATION SPECIFIED.

DRILLED SHAFT INSTALLATION TABLE (BRIDGE NO. 500127)

PIER OR BENT	SOIL PROFILE	DRILLED SHAFT SIZE (m)	DRILLED SHAFT NUMBER	FACTORED DESIGN LOAD (kN)	BOTTOM OF CAP ELEV. * (m)	TOP OF SHAFT ELEV. ** (m)	TIP ELEV. (m)	MIN. TIP ELEV. (m)	COLUMN LENGTH (m)	SHAFT LENGTH (m)	SCOUR ELEV. (m)	MIN. ROCK SOCKET LENGTH (m)
END BENT 1	B-2	1.070	1	1633	+28.268	+26.250	+9.0	+9.0	2.018	17.25	+16.764	1.0
			2		+28.136	+26.250	+9.0	+9.0	1.886	17.25	+16.764	1.0
INT. BENT 2	B-2	1.070	1	2812	+28.261	+26.250	+10.3	+12.0	2.011	15.95	+19.690	2.0
			2		+28.221	+26.250	+10.3	+12.0	1.971	15.95	+19.690	2.0
INT. BENT 3	B-2	1.070	1	3687	+28.018	+26.250	+6.6	+6.6	1.768	19.65	+15.849	2.0
			2		+28.079	+26.250	+6.6	+6.6	1.829	19.65	+15.849	2.0
INT. BENT 4	B-3	1.070	1	3687	+28.046	+26.250	+10.3	+10.3	1.796	15.95	+15.849	5.5
			2		+28.204	+26.250	+10.3	+10.3	1.954	15.95	+15.849	5.5
INT. BENT 5	B-4	1.070	1	2812	+28.375	+26.250	+14.0	+14.0	2.125	12.25	+19.690	5.5
			2		+28.533	+26.250	+14.0	+14.0	2.283	12.25	+19.690	5.5
INT. BENT 6	B-5	1.070	1	2547	+28.450	+26.250	+14.0	+14.0	2.200	12.25	+19.690	5.5
			2		+28.608	+26.250	+14.0	+14.0	2.358	12.25	+19.690	5.5
INT. BENT 7	B-5	1.070	1	2958	+28.507	+26.250	+14.0	+14.0	2.257	12.25	+19.690	5.5
			2		+28.665	+26.250	+14.0	+14.0	2.415	12.25	+19.690	5.5
INT. BENT 8	B-7	1.070	1	2679	+28.551	+26.250	+14.0	+14.0	2.301	12.25	+19.690	5.5
			2		+28.710	+26.250	+14.0	+14.0	2.460	12.25	+19.690	5.5
INT. BENT 9	B-6	1.070	1	2958	+28.583	+26.250	+12.0	+12.0	2.333	14.25	+19.690	2.0
			2		+28.742	+26.250	+12.0	+12.0	2.492	14.25	+19.690	2.0
INT. BENT 10	B-8	1.070	1	2547	+28.604	+26.250	+4.0	+12.0	2.354	22.25	+19.690	0.0 ***
			2		+28.762	+26.250	+4.0	+12.0	2.512	22.25	+19.690	0.0 ***
INT. BENT 11	B-9	1.070	1	2958	+28.611	+26.250	+14.0	+14.0	2.361	12.25	+19.690	5.5
			2		+28.770	+26.250	+14.0	+14.0	2.520	12.25	+19.690	5.5
INT. BENT 12	B-9	1.070	1	2679	+28.607	+26.250	+14.0	+14.0	2.357	12.25	+19.690	5.5
			2		+28.765	+26.250	+14.0	+14.0	2.515	12.25	+19.690	5.5
INT. BENT 13	B-11	1.070	1	2958	+28.590	+26.250	+12.0	+12.0	2.340	14.25	+19.690	2.0
			2		+28.749	+26.250	+12.0	+12.0	2.549	14.25	+19.690	2.0
INT. BENT 14	B-11	1.070	1	2547	+28.561	+26.250	+12.0	+12.0	2.311	14.25	+19.690	2.0
			2		+28.720	+26.250	+12.0	+12.0	2.470	14.25	+19.690	2.0
INT. BENT 15	B-10	1.070	1	2958	+28.520	+26.250	+11.6	+12.0	2.270	14.65	+19.690	2.0
			2		+28.679	+26.250	+11.6	+12.0	2.429	14.65	+19.690	2.0
INT. BENT 16	B-13	1.070	1	2679	+28.467	+26.250	+7.0	+12.0	2.217	19.25	+19.690	2.0
			2		+28.625	+26.250	+7.0	+12.0	2.375	19.25	+19.690	2.0
INT. BENT 17	B-12	1.070	1	2958	+28.401	+26.250	+6.6	+12.0	2.151	19.65	+19.690	2.0
			2		+28.560	+26.250	+6.6	+12.0	2.310	19.65	+19.690	2.0
INT. BENT 18	B-12	1.070	1	2547	+28.324	+26.250	+6.6	+12.0	2.074	19.65	+19.690	2.0
			2		+28.482	+26.250	+6.6	+12.0	2.232	19.65	+19.690	2.0
INT. BENT 19	B-16	1.070	1	2958	+28.234	+26.250	+8.0	+12.0	1.984	18.25	+19.690	2.0
			2		+28.392	+26.250	+8.0	+12.0	2.142	18.25	+19.690	2.0
INT. BENT 20	B-14	1.070	1	2679	+28.137	+26.250	+12.0	+12.0	1.887	14.25	+19.690	2.0
			2		+28.296	+26.250	+12.0	+12.0	2.046	14.25	+19.690	2.0
INT. BENT 21	B-15	1.070	1	2958	+28.041	+26.250	+10.0	+12.0	1.791	16.25	+19.690	2.0
			2		+28.199	+26.250	+10.0	+12.0	1.949	16.25	+19.690	2.0
END BENT 22	B-15	1.070	1	1633	+27.945	+26.250	+8.6	+8.6	1.695	17.65	+16.307	1.0
			2		+28.103	+26.250	+8.6	+8.6	1.853	17.65	+16.307	1.0

* BOTTOM OF CAP MEASURED AT ℄ COLUMN & ℄ BENT

** TOP OF SHAFT SET AT 26.250 TO MAINTAIN ELEVATION ABOVE NORMAL HIGH WATER

*** DRILLED SHAFTS THAT TERMINATE IN SOIL SHALL BE CONSTRUCTED TO PROVIDE A MINIMUM OF 15.7 METERS OF UNCASED SHAFT LENGTH.

ACCEPTANCE OF SHAFTS TERMINATING IN SOIL SHALL BE BASED ON AN AVERAGE SPT N-VALUE OF 28.5 FOR A CONTINUOUS SAMPLING SPT FROM GROUND LEVEL TO 1.5 METERS BELOW FINAL TIP ELEVATION. IN CALCULATING THE AVERAGE N-VALUE, NO INDIVIDUAL N-VALUE SHALL EXCEED 50.

1/4/99

G:\929208-7\STRUCT\MAIN\DRSHFT2.DGN 29DEC98 13:23:02 SCALE:20 USER:

REVISIONS

Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	E.D.L.	10/98
Checked by	L.W.M.	10/98
Designed by	D.R.M.	10/98
Checked by	W.H.M.	10/98
Approved by	D.R.MERRELL	

ENGINEER OF RECORD:
DUANE R. MERRELL
P.O. Box 51343
Jacksonville Beach, Florida
32240-1343

LOGO:
Connelly & Wicker Inc.
Consulting Engineers
Jacksonville Beach, Florida

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	FINANCIAL PROJECT ID.
SR 63	GADSDEN	218944-1-52-01

SHEET TITLE: DRILLED SHAFT INSTALLATION TABLES

Drawing No.

PROJECT NAME: SR 63 OVER OCHLOCKONEE RIVER BRIDGE NO'S. 500124 & 500127

Index No.

THIS CONTRACT PLAN SET INCLUDES:

ROADWAY PLANS
SUMMARY OF PAY ITEMS (4 SHEETS)
SIGNING AND PAVEMENT MARKING PLANS
STRUCTURE PLANS
LOAD TEST PLANS

INDEX OF ROADWAY PLANS

THESE PLANS HAVE BEEN PREPARED IN ACCORDANCE WITH AND ARE GOVERNED BY THE STATE OF FLORIDA, DEPARTMENT OF TRANSPORTATION, ROADWAY AND TRAFFIC DESIGN STANDARDS (BOOKLET DATED JANUARY, 1992).

NOTE:
REMOVAL OF THE EXISTING VICTORY BRIDGE AND CONSTRUCTION OF THE NEW BRIDGE OVER THE WESTERN FLOOD PLAIN MAY INVOLVE WORK IN WET GROUND CONDITIONS. THE CONTRACTOR WILL NEED TO BUILD A TEMPORARY WORK STRUCTURE AND SHALL CONFORM TO THE FLORIDA DEPARTMENT OF ENVIRONMENTAL PROTECTION PERMIT FOR CONSTRUCTING THE WORK BRIDGE. THE CONTRACTOR SHALL REMOVE ANY SUCH ITEMS AFTER THEIR USE AS DIRECTED BY THE ENGINEER. NO DIRECT PAYMENT WILL BE MADE FOR ITEMS USED TO FACILITATE CONSTRUCTION AND THE SUBSEQUENT REMOVAL OF THESE ITEMS. THIS SHALL BE INCIDENTAL TO THE CONTRACT.

REVISIONS

LOAD TEST PLAN SHEETS LT-1 THRU LT-9 (REVISED 3/16/94)
ROADWAY C.E.S. SHEET 01 (REVISED 3/16/94)
ROADWAY C.E.S. SHEETS 01 AND 02 (REVISED 3/30/94)
BRIDGE C.E.S. SHEET 01 (REVISED 4/11/94)
STRUCTURE PLAN SHEET B-1 (REVISED 4/11/94)
BRIDGE C.E.S. (REVISED 4-18-94)

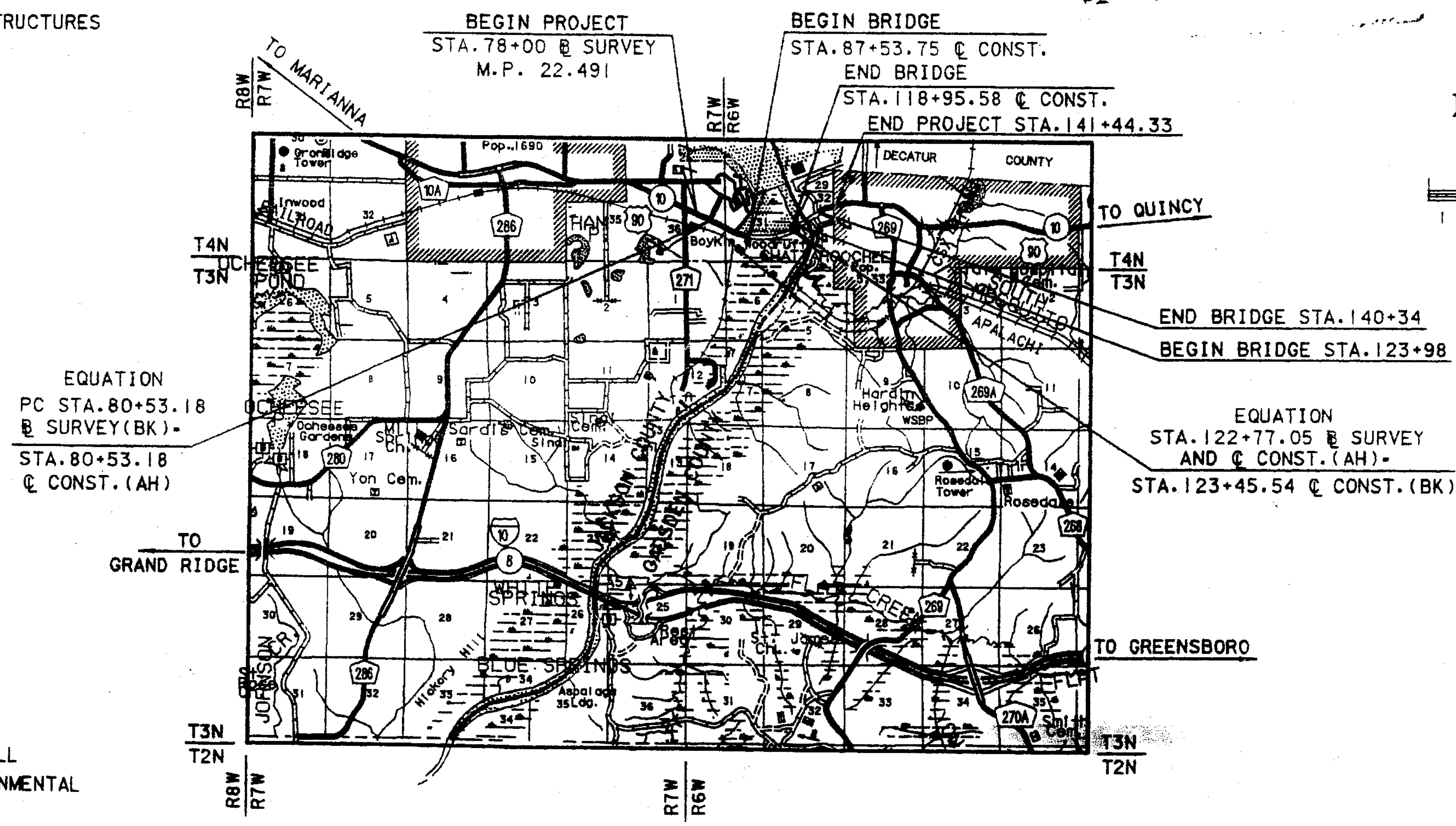
X
Design ✓
X

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

FM 219313

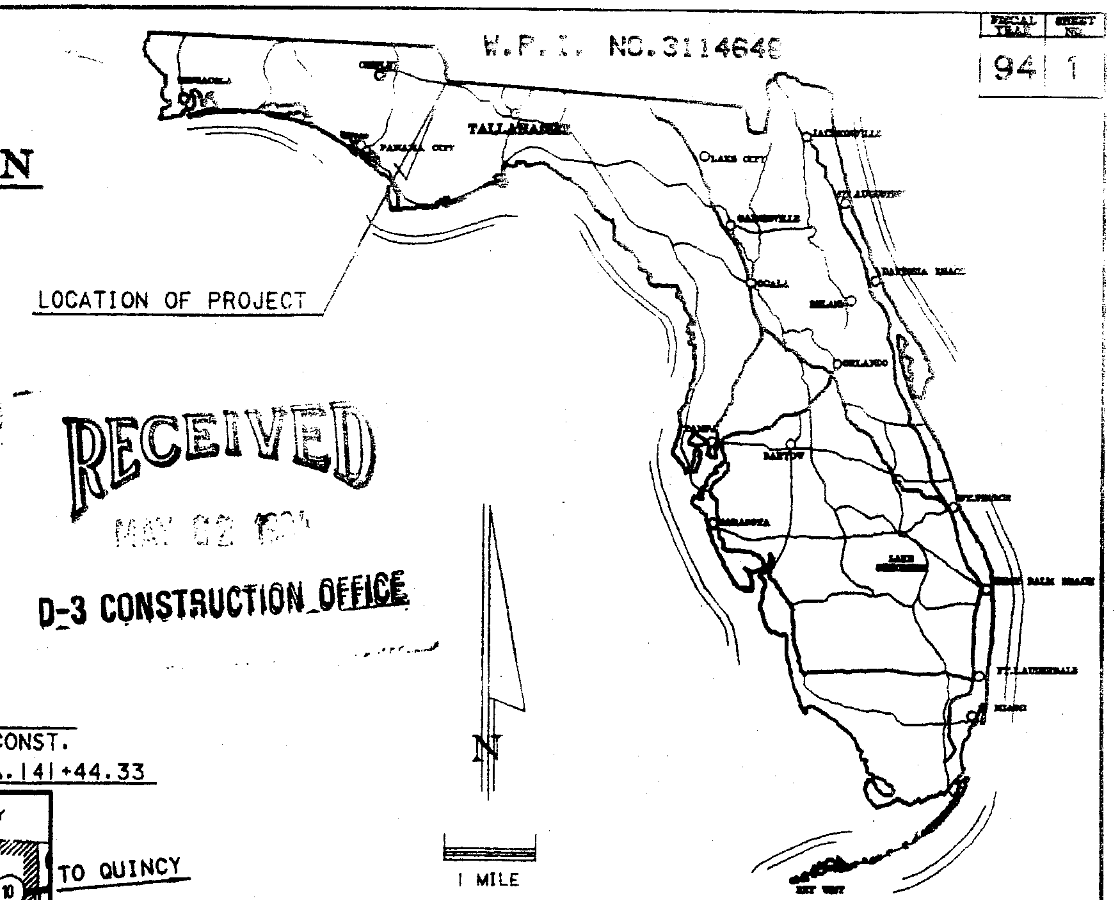
PLANS OF PROPOSED
STATE HIGHWAY

STATE PROJ. NO. 53020-3540
JACKSON AND GADSDEN COUNTIES
STATE ROAD NO. 10



W.P.I. NO. 3114646

FISCAL YEAR	SHEET NO.
94	1



RECEIVED
MAY 02 1994
D-3 CONSTRUCTION OFFICE

PLANS PREPARED BY:
H.W. LOCHNER, INC.
9721 EXEC. CTR. DR. N.
ST. PETERSBURG, FL. 33702

ROADWAY PLANS
ENGINEER OF RECORD:
HUGH T. WILLIAMS, P.E.
FL. P.E. NO. 23034

ATTENTION IS DIRECTED TO THE FACT THAT THESE PLANS MAY HAVE BEEN REDUCED IN SIZE BY REPRODUCTION. THIS MUST BE CONSIDERED WHEN OBTAINING SCALED DATA.

GOVERNING SPECIFICATIONS, STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION, STANDARD SPECIFICATIONS, DATED 1991 AND SUPPLEMENTS THERETO IF NOTED IN THE SPECIAL PROVISIONS FOR THIS PROJECT.

REVISIONS		
DATE	BY	DESCRIPTION
2/94	WK	ADDED LOAD TEST PLANS
4/94	HW	REVISED NOTE

RECYCLED PAPER

LENGTH OF PROJECT

	LINEAR FT.	MILES
ROADWAY	1634.99	0.309
BRIDGES	4777.83	0.905
NET LENGTH OF PROJ.	6412.82	1.214
EXCEPTIONS	-	-
GROSS LENGTH OF PROJ.	6412.82	1.214

PROJECT MANAGER: BRIAN BLANCHARD, P.E.

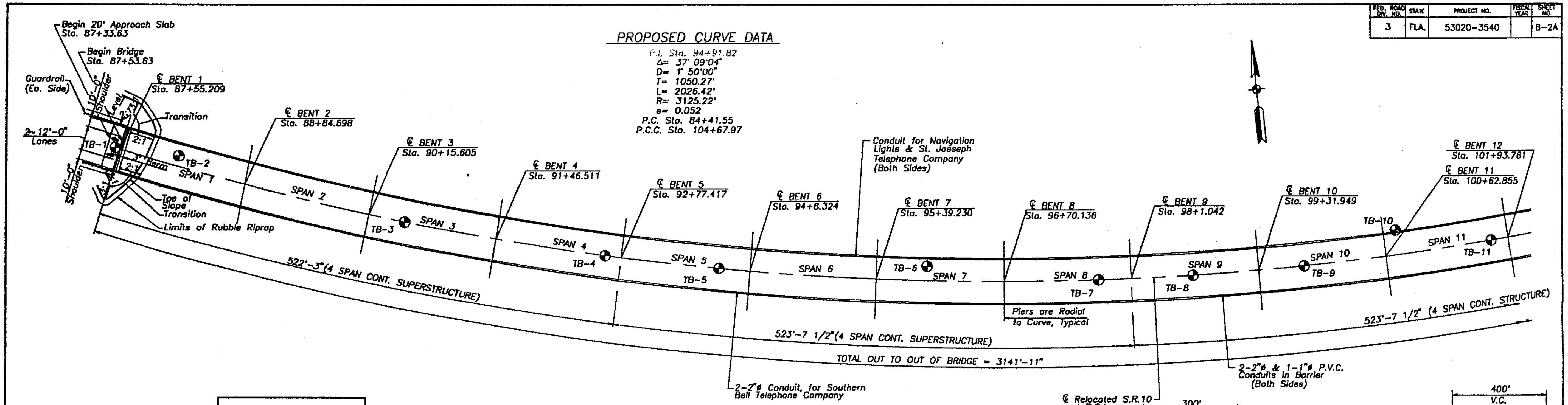
ROADWAY PLANS
APPROVED BY: HUGH T. WILLIAMS, P.E.
FL. P.E. NO. 23034 DATE: DEC. 29, 1993

S.R. 10 (U.S. 90) VICTORY BRIDGE REPLACEMENT AND APALACHICOLA RIVER BRIDGE WIDENING

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	53020-3540		B-2A

PROPOSED CURVE DATA

P.I. Sta. 94+91.82
Δ= 37° 09'04"
D= 1° 50'00"
T= 1050.27'
L= 2026.42'
R= 3125.22'
e= 0.052
P.C. Sta. 84+41.55
P.C.C. Sta. 104+67.97



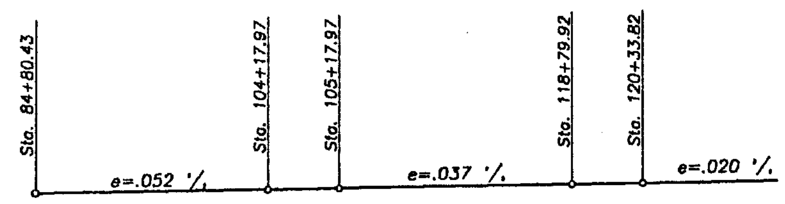
PLAN VIEW

(DIMENSIONS TAKEN ALONG L RELOC. S.R.10)

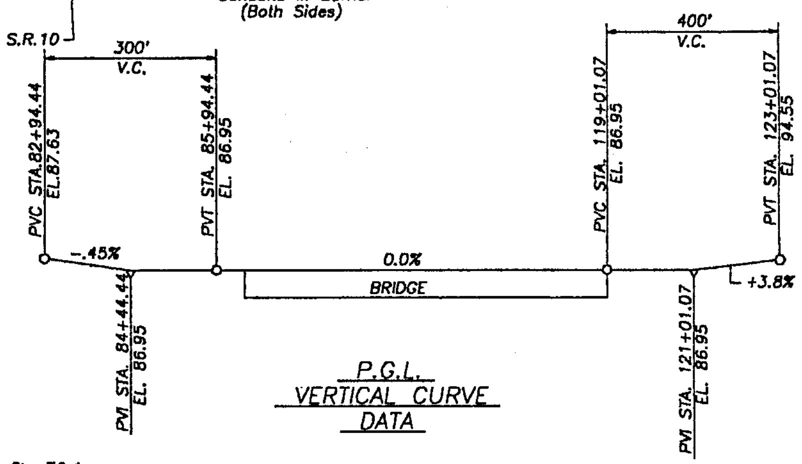
BORING HOLE LOCATIONS		
BORING NO.	STA.	OFFSET
TB-1	87+48	0
TB-2	88+75	10'LT
TB-3	90+38	0
TB-4	92+60	0
TB-5	93+75	0
TB-6	95+50	15'LT
TB-7	97+68	0
TB-8	98+64	0
TB-9	99+70	0
TB-10	100+75	25'LT
TB-11	101+75	0
TB-12	102+75	0

S.R.10 TRAFFIC DATA

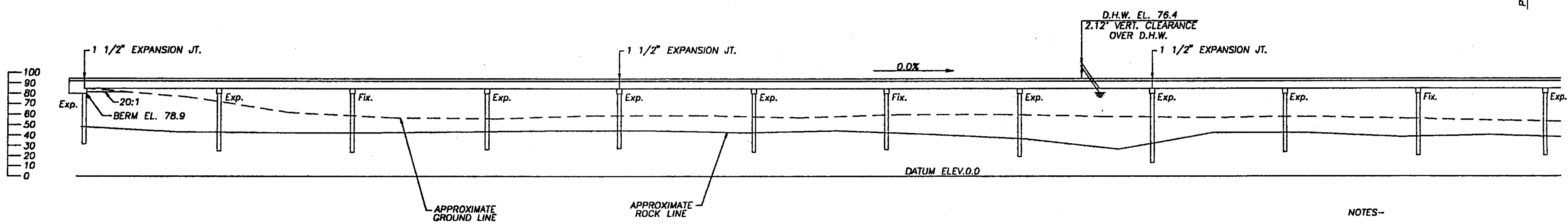
1990 ADT = 6350
1996 ADT = 7600
2016 ADT = 12,500
K= 12%
D= 58%
T= 9%
Design Speed = 60 mph



SUPERELEVATION DIAGRAM



P.G.L. VERTICAL CURVE DATA



ELEVATION

NOTES–

1. FOR GENERAL NOTES, SEE SHEET B-1A
2. FOR MINIMUM DRILLED SHAFT TIP ELEVATION, SEE SHEET B-6A

BRIDGE NO. 530111

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	TAL	11-9-94
Checked by	HHJ	11-9-94
Designed by		
Checked by		
Approved by	HHJ 11-9-94	

ENGINEER OF RECORD:
JOINT VENTURE
FINLEY McNARY ENG. & JANSSEN & SPAANS ENG.

LOGO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 10	JACKSON/GADSDEN	53020-3540

SHEET TITLE: GENERAL PLAN & ELEVATION

Drawing No.

PROJECT NAME: S.R.10 OVER THE WESTERN FLOOD PLAIN

Index No.

E:\094\094K02A

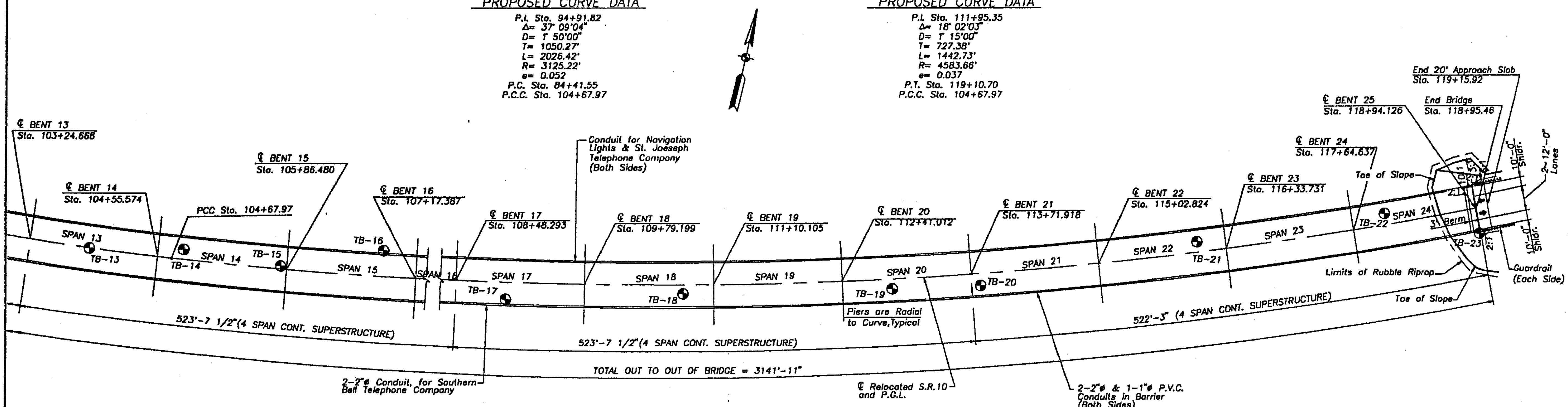
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	53020-3540		B-3A

PROPOSED CURVE DATA

P.I. Sta. 94+91.82
Δ= 37° 09'04"
D= 1° 50'00"
T= 1050.27'
L= 2026.42'
R= 3125.22'
e= 0.052
P.C. Sta. 84+41.55
P.C.C. Sta. 104+67.97

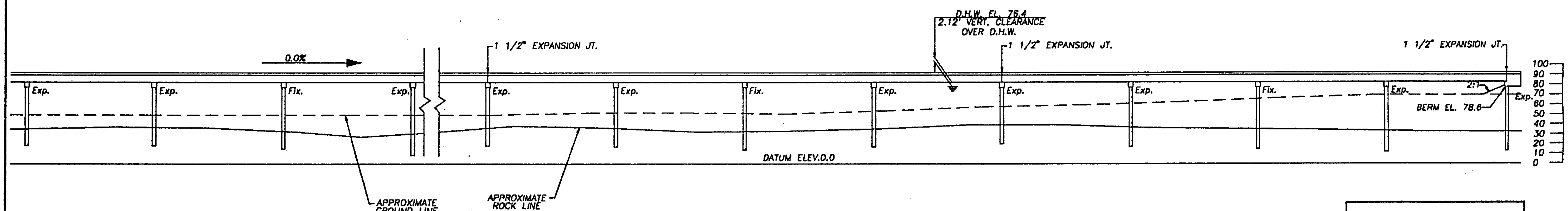
PROPOSED CURVE DATA

P.I. Sta. 111+95.35
Δ= 18° 02'03"
D= 1° 15'00"
T= 727.38'
L= 1442.73'
R= 4583.66'
e= 0.037
P.T. Sta. 119+10.70
P.C.C. Sta. 104+67.97



PLAN VIEW
(DIMENSIONS TAKEN ALONG ℄ RELOC. S.R.10)

BORING HOLE LOCATIONS		
BORING NO.	STA.	OFFSET
TB-13	103+85	0
TB-14	104+80	10'LT
TB-15	105+80	3'LT
TB-16	106+80	25'LT
TB-17	108+89	17'RT
TB-18	110+80	10'RT
TB-19	112+90	11'RT
TB-20	113+80	12'RT
TB-21	116+04.5	10'LT
TB-22	117+91	12'LT
TB-23	118+93	25'RT



ELEVATION

BRIDGE NO. 530111

NOTES-

1. FOR GENERAL NOTES, SEE SHEET B-1A
2. FOR MINIMUM DRILLED SHAFT TIP ELEVATION, SEE SHEET B-6A

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	TAL/REM	11-9-94
Checked by	HHJ	11-9-94
Designed by		
Checked by		
Approved by	HHJ 11-9-94	

ENGINEER OF RECORD:
JOINT VENTURE
FINLEY McNARY ENG. & JANSSEN & SPAANS ENG.

LOGO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 10	JACKSON/GADSDEN	53020-3540

SHEET TITLE: GENERAL PLAN & ELEVATION

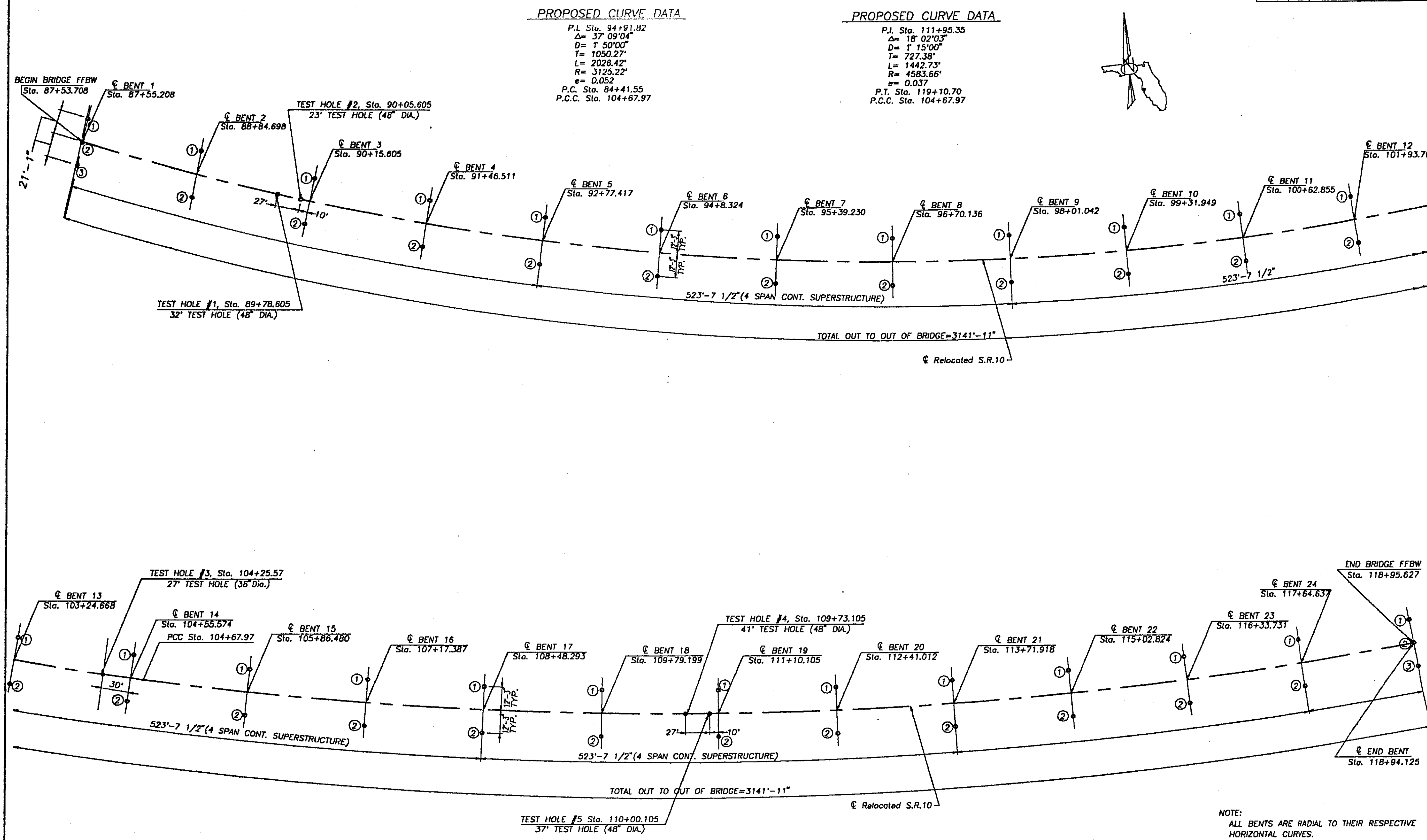
Drawing No.

PROJECT NAME: S.R.10 OVER THE WESTERN FLOOD PLAIN

Index No.

E:\094\094K03A

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	53020-3540		B-5A



NOTE:
ALL BENTS ARE RADIAL TO THEIR RESPECTIVE HORIZONTAL CURVES.

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	TAL	11-9-94
Checked by	HHJ	11-9-94
Designed by		
Checked by		
Approved by	HHJ 11-9-94	

ENGINEER OF RECORD:
JOINT VENTURE
FINLEY McNARY ENG. &
JANSSEN & SPAANS ENG.

LOGO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 10	JACKSON/GADSDEN	53020-3540

SHEET TITLE: FOUNDATION LAYOUT

Drawing No.

PROJECT NAME: S.R.10 OVER THE WESTERN FLOOD PLAIN

Index No.

E:\094\094KFTG

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.			B-6

DRILLED SHAFT INSTALLATION TABLE

BENT	SOIL PROFILE	SHAFT SIZE (INCHES)	NUMBER OF SHAFTS	DEAD LOAD (TONS)	LIVE LOAD (TONS)	TOTAL DESIGN LOAD (TONS)	MINIMUM TIP ELEVATION (FT.)	SCOUR ELEVATION (FT.)	* MINIMUM ROCK SOCKET LENGTH (FT.)
END BENT 1 CAP	A	36	2	231	75	306	31.438	74	10
END BENT 1 WINGWALL	A	36	2	42	0	42	31.665	74	10
BENT 2	A	36	2	388	75	463	26.735	59	15
BENT 3	A	36	2	388	75	463	25.735	51	15
BENT 4	A	36	2	388	75	463	25.735	49	15
BENT 5	A	36	2	388	75	463	26.735	49	15
BENT 6	A	36	2	388	75	463	27.735	49	15
BENT 7	A	36	2	388	75	463	27.735	48	15
BENT 8	A	36	2	388	75	463	25.735	47	15
BENT 9	A	36	2	388	75	463	27.735	48	15
BENT 10	A	36	2	388	75	463	24.735	48	15
BENT 11	A	36	2	388	75	463	19.735	47	10
BENT 12	A	36	2	388	75	463	9.735	46	10
BENT 13	A	36	2	388	75	463	25.735	47	15
BENT 14	A	36	2	388	75	463	25.735	45	12
BENT 15	B	36	3	359	58	417	21.735	40	15
BENT 16	B	36	3	359	58	417	23.735	41	12
BENT 17	B	36	3	359	58	417	20.735	40	12
BENT 18	B	36	3	359	58	417	20.735	40	12
BENT 19	B	36	3	359	58	417	22.735	40	12
BENT 20	B	36	3	359	58	417	22.009	40	12
BENT 21	B	36	3	359	58	417	18.035	39	12
BENT 22	B	36	3	359	58	417	11.035	40	12
BENT 23	B	36	3	359	58	417	17.035	41	12
BENT 24	C	36	2	388	75	463	24.035	42	10
BENT 25	C	36	2	388	75	463	22.035	44	10
BENT 26	C	36	2	388	75	463	18.035	46	10
BENT 27	C	36	2	388	75	463	19.035	45	10
BENT 28	C	36	2	388	75	463	21.035	43	10
BENT 29	C	36	2	388	75	463	24.035	45	10
BENT 30	C	36	2	388	75	463	24.035	48	10
BENT 31	C	36	2	388	75	463	22.035	51	10
BENT 32	C	36	2	388	75	463	21.035	54	10
BENT 33	C	36	2	388	75	463	20.035	57	10
BENT 34	C	36	2	388	75	463	18.035	60	10
END BENT 35 CAP	C	36	2	250	75	325	7.227	69	10
END BENT 35 WINGWALL	C	36	2	42	0	42	7.407	69	10

NOTES:

1.) Drilled Shafts will be constructed in accordance with Supplemental Specification Section 455, Structures Foundations.

2.) Sidewall cave should be anticipated during Drilling and, therefore, Temporary Steel Casings are required. Site Conditions indicated that the Wet Construction Method will be required for Shaft Construction.

3.) The Geotechnical Investigation advised that the casing could not be sealed into the Impermeable Stratum and, therefore, the shafts would need to be drilled in a wet condition. The Report noted that ground water would be controlled by setting the casing on top of rock so that the Artesian Head stabilizes within the casing. The Rock Socket is then drilled, the Reinforcing Steel Cage set, and Concrete poured by Tremie or Pump methods. The casing can be pulled after fresh concrete, free of rock cuttings, flows out of the top of the casings. This installation shall be in accordance with FDOT Supplemental Specification 455.

4.) The Florida Department of Transportation has prepared the load test program for the drilled shafts. For details, See Sheets LT-1 through Lt-9 and the special provisions.

5.) Maximum axial load for axial capacity = Dead Load + Live Load. Maximum axial load for Horizontal capacity = Dead Load Only.

6.) The Dead Load and Total Design Load shown is the maximum for a particular bent configuration. The Actual Dead Load for a specific bent may be slightly less depending upon its shaft length when compared with the maximum length. A 3'Ø shaft weighs about 0.5 tons per foot.

* The Minimum Pile Tip Elevation is based on the Boring Logs. The actual ground line and rock line elevations may vary. The Contractor shall maintain the minimum rock socket length as noted on these plans. Socket lengths are subject to change, dependent upon the Load Test Program results. Minimum Rock Socket length shown is that required to resist Lateral Loads.

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

	Names	Dates
Drawn by	W.R.A.	9/93
Checked by	J.H.M.	9/93
Designed by	J.H.M.	9/93
Checked by	R.C.S.	9/93
Approved by	C.F.C.	

ENGINEER OF RECORD:
LOCHNER
H.W. LOCHNER, INC.
CONSULTING ENGINEERS AND PLANNERS

LOGO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 10	JACKSON/GADSDEN	53020-3540

SHEET TITLE: DRILLED SHAFT INSTALLATION TABLE

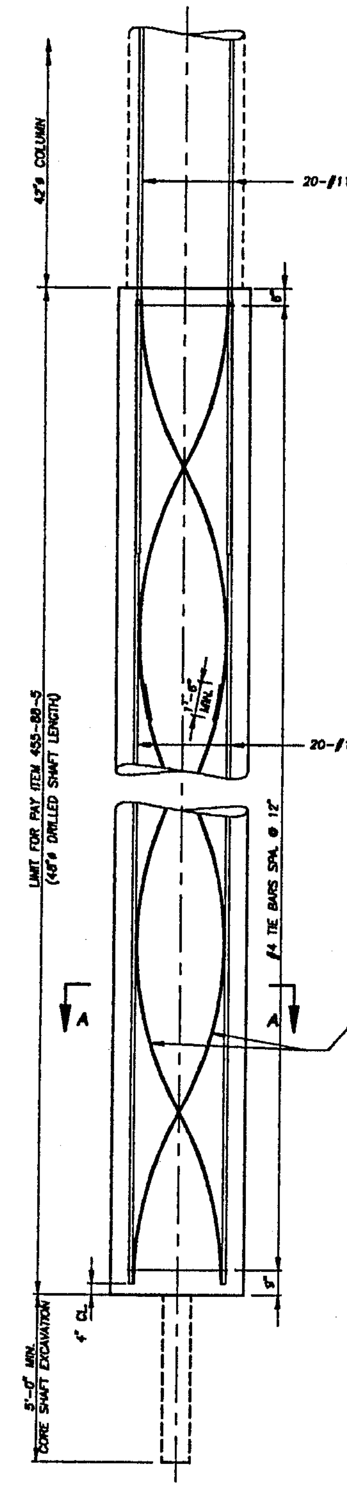
PROJECT NAME: S.R. 10 OVER THE WESTERN FLOODPLAIN

Drawing No. OF

Index No.

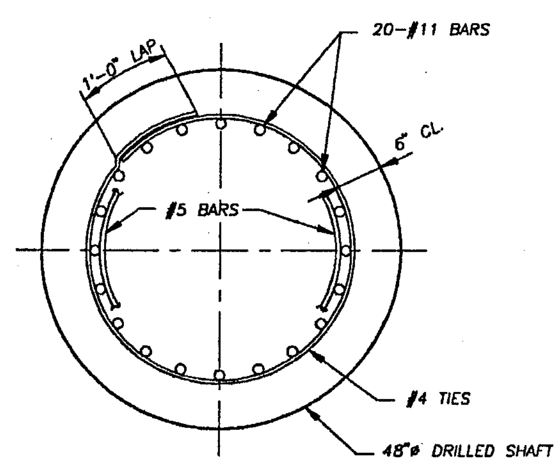
VICTPL2.DGN

FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	53020-3540		B-7A



ELEVATION

FIELD BEND TWO #5 BARS AT A RATE OF ONE COMPLETE REVOLUTION PER 20 FEET OF SHAFT INSIDE VERTICAL REINFORCING STEEL. TIE AT INTERSECTION WITH VERTICAL BARS. LAP SPLICE 1'-6" AS NECESSARY.



SECTION A-A

DRILLED SHAFT INSTALLATION TABLE

BENT	SOIL PROFILE	SIZE (INCHES)	SHAFT NO. OF SHAFTS	DEAD LOAD (TONS)	LIVE LOAD (TONS)	TOTAL DESIGN LOAD (TONS)	MIN. TIP ELEV. (FT.)	SCOUR ELEV. (FT.)	MIN. ROCK SOCKET LENGTH (FT.)
END BENT 1 CAP	A	48	3	241	60	301	26.438	74	15
BENT 2	A	48	2	452	133	585	26.343	59	15
BENT 3	A	48	2	452	133	585	25.735	51	15
BENT 4	A	48	2	452	133	585	26.962	49	15
BENT 5	A	48	2	452	133	585	27.735	49	15
BENT 6	A	48	2	452	133	585	25.860	49	15
BENT 7	A	48	2	452	133	585	26.294	48	15
BENT 8	A	48	2	452	133	585	15.754	47	15
BENT 9	A	48	2	452	133	585	11.369	48	15
BENT 10	A	48	2	452	133	585	23.534	48	15
BENT 11	B	48	2	452	133	585	21.584	47	15
BENT 12	B	48	2	452	133	585	19.028	46	15
BENT 13	B	48	2	452	133	585	17.735	47	15
BENT 14	B	48	2	452	133	585	19.441	45	15
BENT 15	B	48	2	452	133	585	15.741	40	15
BENT 16	B	48	2	452	133	585	9.476	41	15
BENT 17	C	48	2	452	133	585	18.641	40	13
BENT 18	C	48	2	452	133	585	18.734	40	13
BENT 19	C	48	2	452	133	585	15.528	40	13
BENT 20	C	48	2	452	133	585	17.857	40	13
BENT 21	C	48	2	452	133	585	21.035	39	13
BENT 22	C	48	2	452	133	585	19.542	40	13
BENT 23	C	48	2	452	133	585	17.871	41	13
BENT 24	C	48	2	452	133	585	15.871	42	13
END BENT 2 CAP	C	48	3	241	60	301	4.227	69	13

FOR TOP OF SHAFT ELEVATION AND ESTIMATED SHAFT LENGTH SEE DWG. B-15A.

DRILLED SHAFT QUANTITIES PER LIN. FT.

ITEM	UNIT	QUANTITY
CLASS IV CONCRETE (DRILLED SHAFT)	CYS.	0.465
REINFORCING STEEL (SUBSTRUCTURE)	LBS.	115.310

THESE QUANTITIES ARE PROVIDED FOR INFORMATION ONLY.

NOTES:

1. DRILLED SHAFTS WILL BE CONSTRUCTED IN ACCORDANCE WITH SUPPLEMENTAL SPECIFICATION SECTION 455, STRUCTURES FOUNDATIONS.
2. THE FLORIDA DEPARTMENT OF TRANSPORTATION HAS PREPARED THE LOAD TEST PROGRAM FOR THE DRILLED SHAFTS. FOR DETAILS, SEE SHEETS LT-1 THROUGH LT-9 AND THE SPECIAL PROVISIONS.
3. MAXIMUM AXIAL LOAD FOR AXIAL CAPACITY = DEAD LOAD + LIVE LOAD. MAXIMUM AXIAL LOAD FOR HORIZONTAL CAPACITY = DEAD LOAD ONLY.
4. THE DEAD LOAD AND TOTAL DESIGN LOAD SHOWN IS THE MAXIMUM FOR A PARTICULAR BENT CONFIGURATION. THE ACTUAL DEAD LOAD FOR A SPECIFIC BENT MAY BE SLIGHTLY LESS DEPENDING UPON ITS SHAFT LENGTH WHEN COMPARED WITH THE MAXIMUM LENGTH. A 4' DIAMETER SHAFT WEIGHS ABOUT 0.94 TONS PER FOOT.
5. CONCRETE FOR DRILLED SHAFTS SHALL BE CLASS IV (DRILLED SHAFT).
6. REINFORCING STEEL SHALL CONFORM TO ASTM A-615, GRADE 60.
7. THE CONTRACTOR SHALL PERFORM A 4 TO 6 INCH DIAMETER SHAFT CORE AND/OR A STANDARD PENETRATION TEST BORING, CONTINUOUS SAMPLING, FOR A MINIMUM DEPTH OF 5 FEET BELOW THE BOTTOM OF THE DRILLED SHAFT EXCAVATION OR AS DIRECTED BY THE ENGINEER.
8. THE CONTRACTOR IS REQUIRED TO CONSTRUCT FIVE OUT OF POSITION TEST HOLES TO DEMONSTRATE THE ADEQUACY OF CONSTRUCTION METHODS AND EQUIPMENT. THESE TEST HOLES WILL BE USED IN THE LOAD TEST PROGRAM. FOR DETAILS, SEE LOAD TEST PLANS. THE TEST HOLE LOCATIONS ARE LOCATED ON THE FOUNDATION LAYOUT SHEET. THE TEST HOLE LENGTH IS SHOWN ON THE FOUNDATION LAYOUT SHEET AND SHALL BE PAID FOR UNDER ITEM NO. 455-91-3. IF THE TEST HOLE IS RELOCATED FROM THAT SHOWN, THE ELEVATIONS AND SHAFT LENGTH MAY VARY FROM THAT SHOWN IN TABLE.
9. THE MINIMUM PILE TIP ELEVATION IS BASED ON THE BORING LOGS. THE ACTUAL GROUND LINE AND ROCK LINE MAY VARY. THE CONTRACTOR SHALL MAINTAIN THE MINIMUM ROCK SOCKET LENGTH AS NOTED ON THESE PLANS. SOCKET LENGTHS ARE SUBJECT TO CHANGE, DEPENDENT UPON THE LOAD TEST PROGRAM RESULTS. MINIMUM ROCK SOCKET LENGTH SHOWN IS THAT REQUIRED TO RESIST LATERAL LOADS.
10. THIS SHEET REPLACES DRAWINGS B-6 AND B-7 OF THE CONTRACT PLANS.

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	TAL	11-9-94
Checked by	HHJ	11-9-94
Designed by		
Checked by		
Approved by	HHJ 11-9-94	

ENGINEER OF RECORD:
JOINT VENTURE
FINLEY McNARY ENG. & JANSSEN & SPAANS ENG.

LOGO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 10	JACKSON/GADSDEN	53020-3540

SHEET TITLE: 48"ø DRILLED SHAFT DETAILS

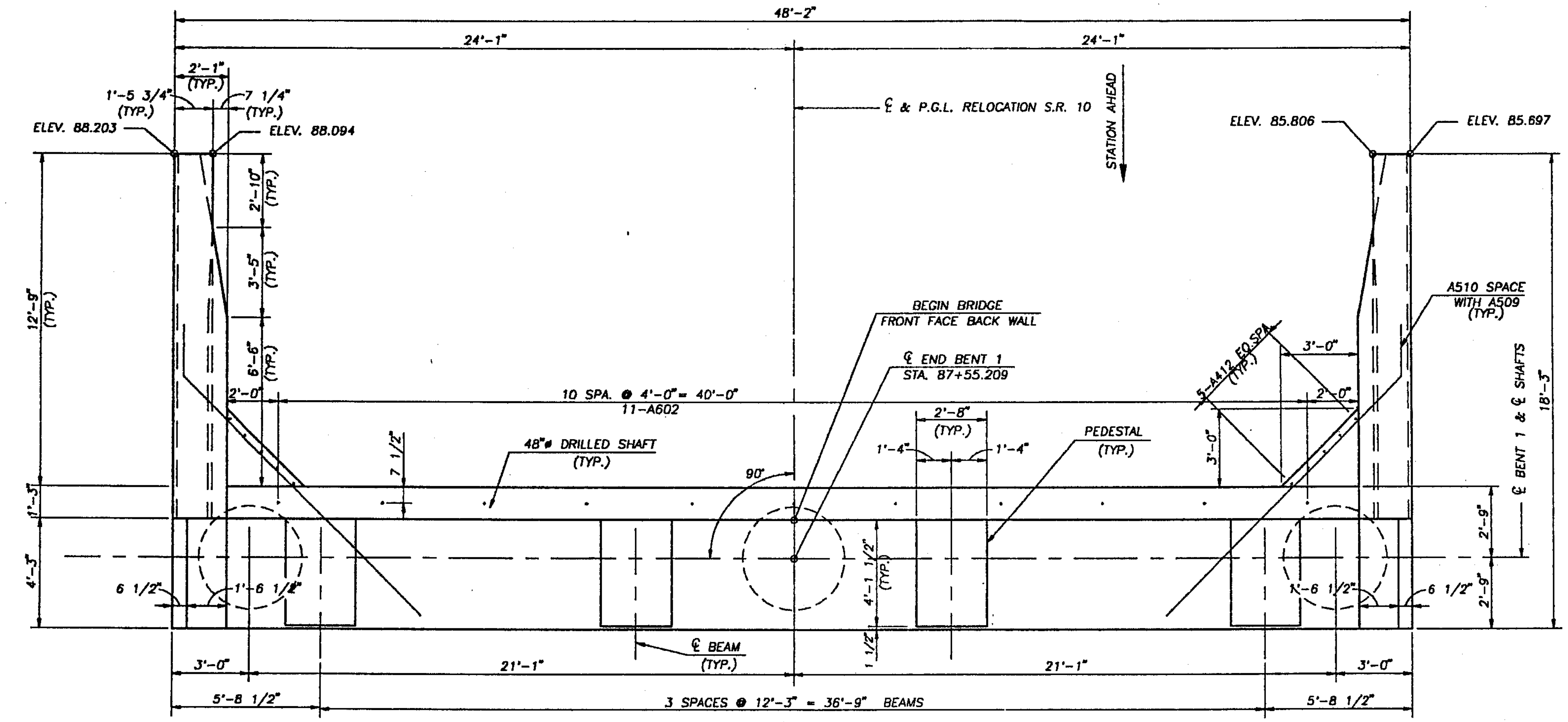
Drawing No.

PROJECT NAME: S.R.10 OVER THE WESTERN FLOOD PLAIN

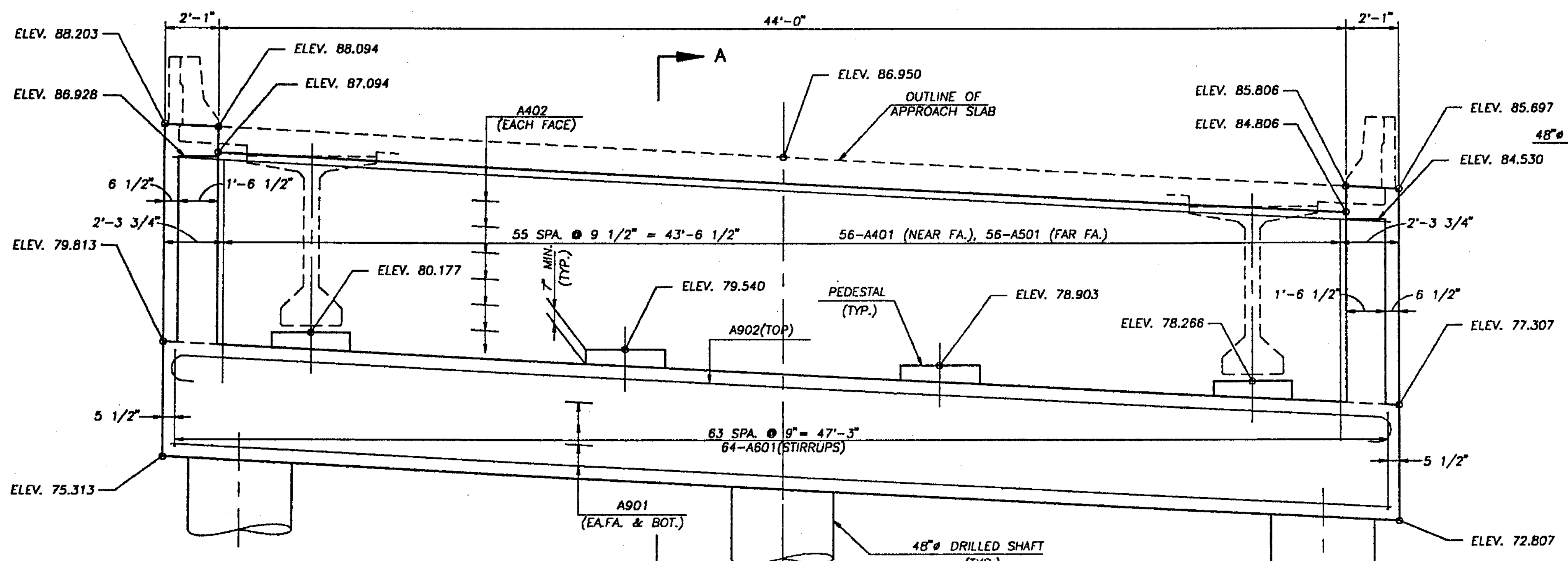
Index No.

E:\094\094GDRS

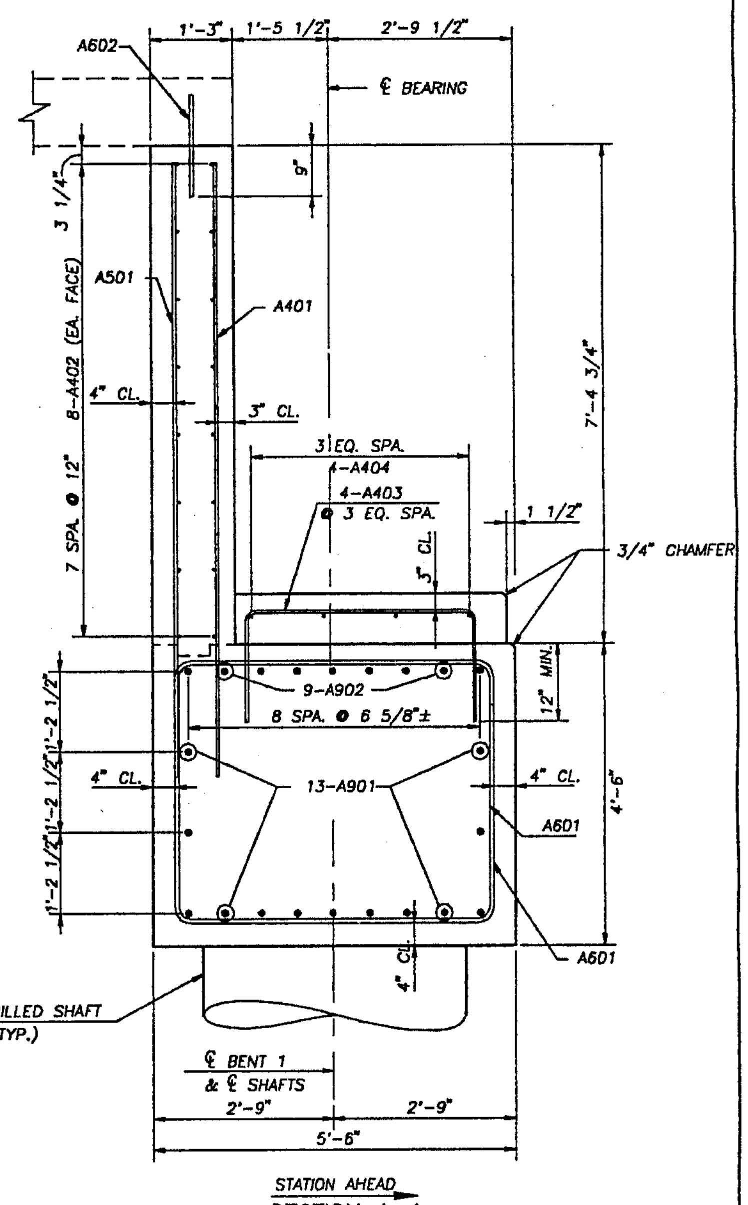
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	53020-3540		B-8A



PLAN



ELEVATION



SECTION A-A

NOTES:

1. FOR REINFORCING BAR LIST, SEE SHEET B-10A.
2. FOR WINGWALL DETAILS, SEE SHEET B-10A.
3. FOR QUANTITIES, SEE SHEET B-10A.

REVISIONS Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	TAL	11-9-94
Checked by	HHJ	11-9-94
Designed by		
Checked by		
Approved by	HHJ 11-9-94	

ENGINEER OF RECORD:
JOINT VENTURE
FINLEY McNARY ENG. & JANSSEN & SPAANS ENG.

LOGO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 10	JACKSON/GADSDEN	53020-3540

SHEET TITLE: END BENT 1 DETAILS

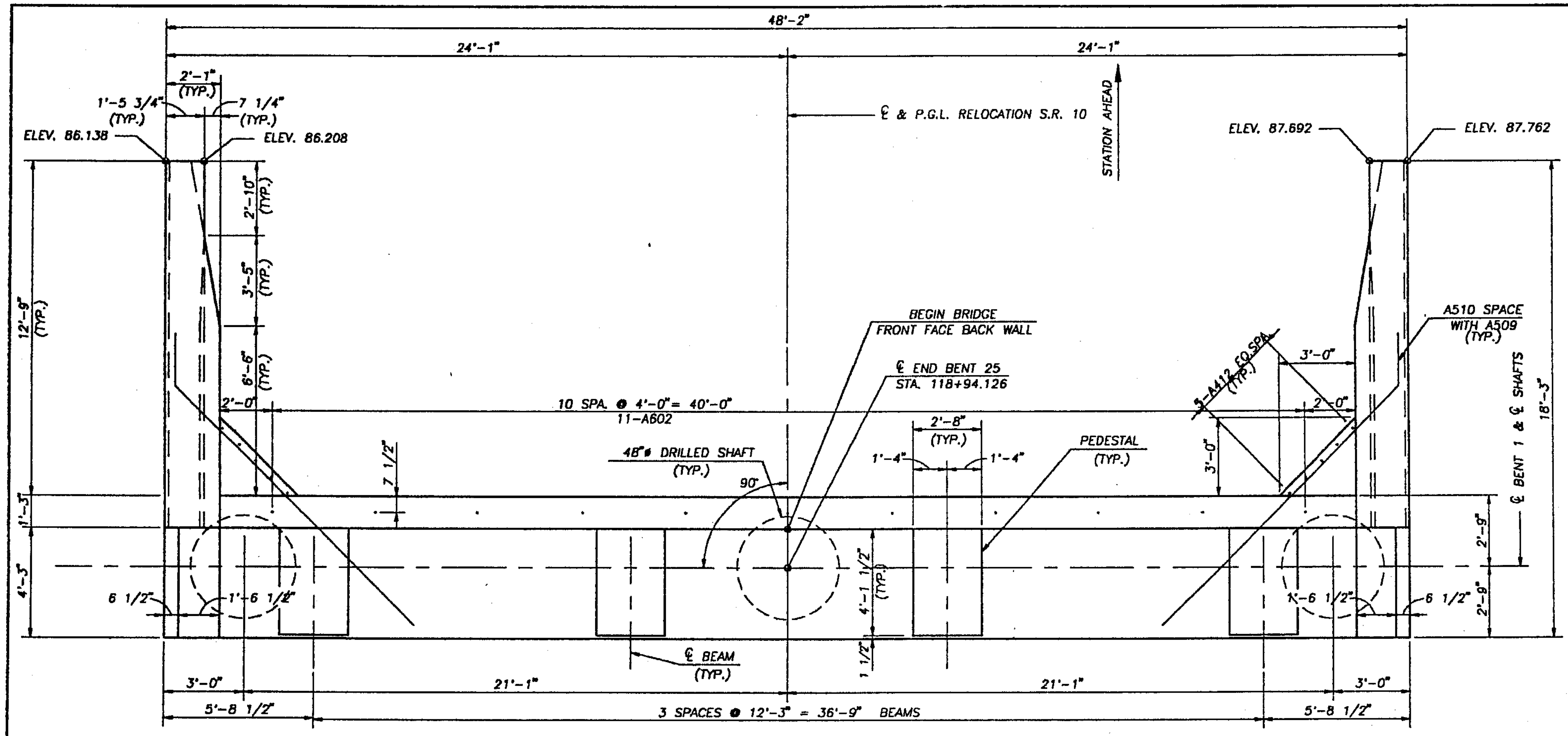
PROJECT NAME: S.R.10 OVER THE WESTERN FLOOD PLAIN

Drawing No.

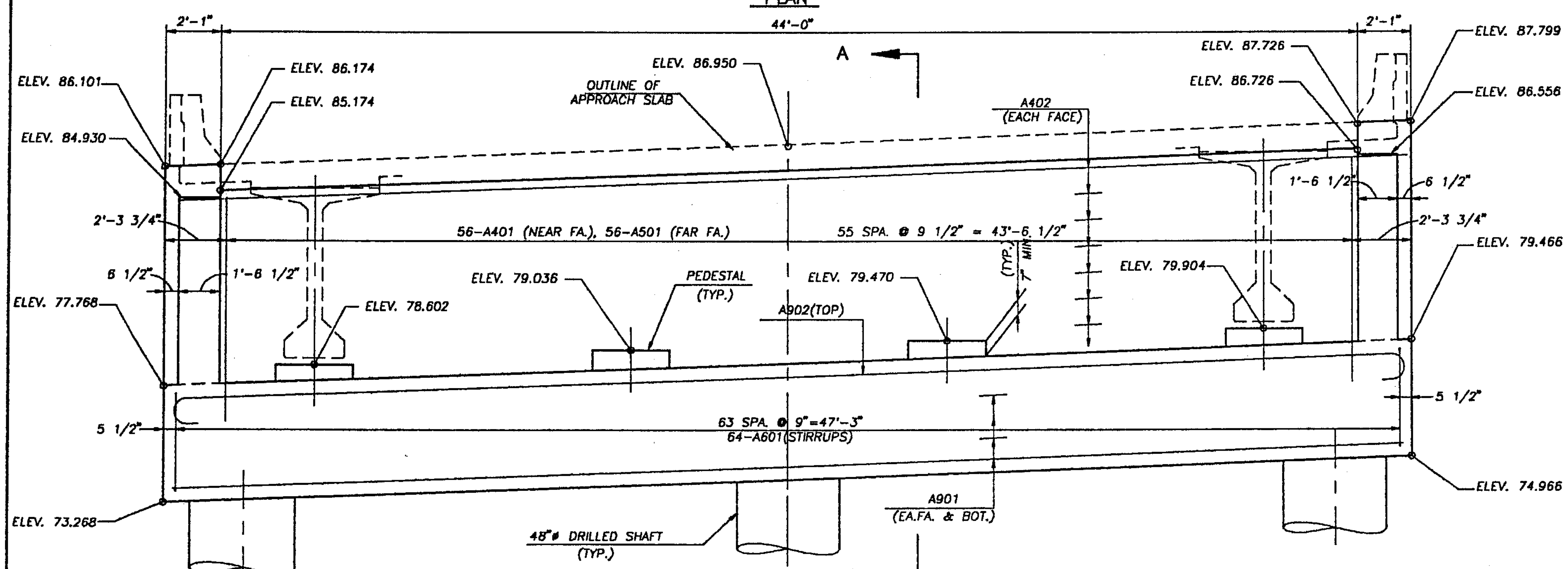
Index No.

E:\094\094FBT1

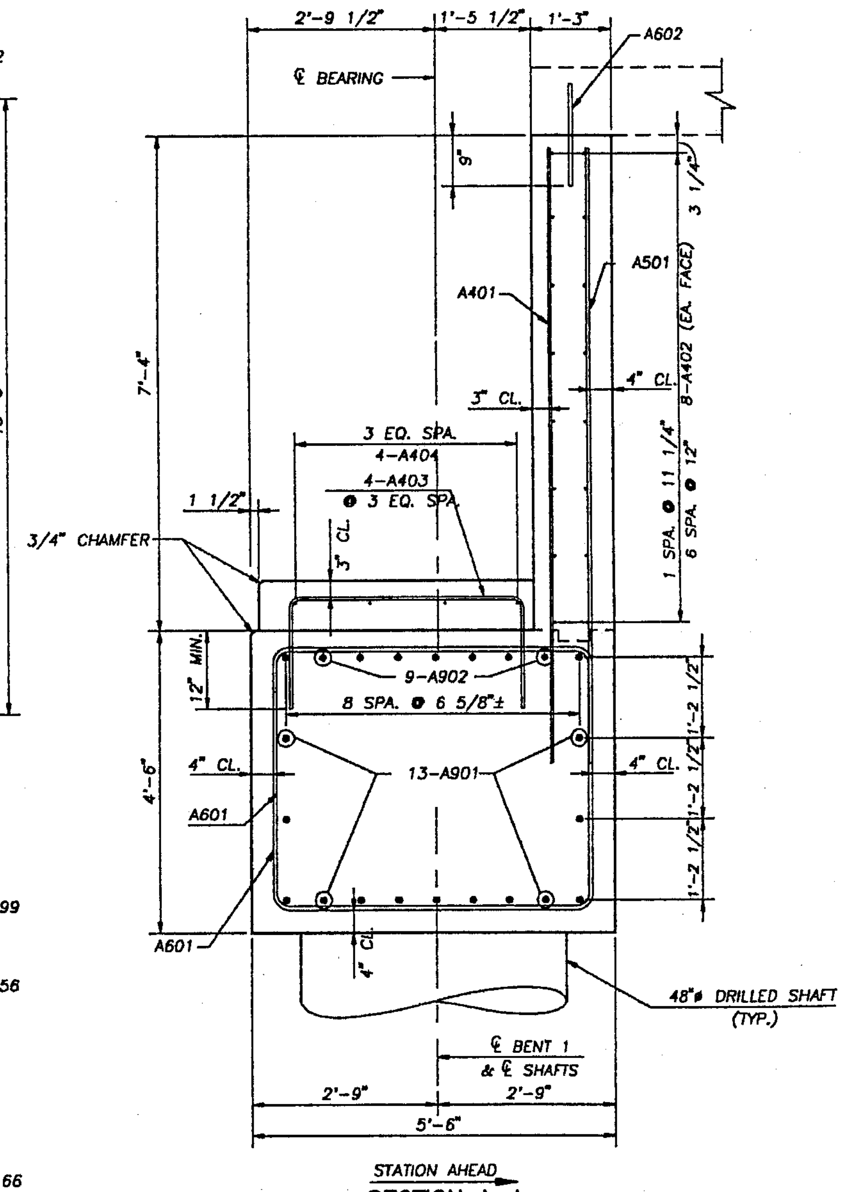
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
3	FLA.	53020-3540		B-9A



PLAN



ELEVATION



SECTION A-A

NOTES:
1. FOR REINFORCING BAR LIST, SEE SHEET B-10A.
2. FOR WINGWALL DETAILS, SEE SHEET B-10A.
3. FOR QUANTITIES, SEE SHEET B-10A.

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	TAL	11-9-94
Checked by	HHJ	11-9-94
Designed by		
Checked by		
Approved by	HHJ 11-9-94	

ENGINEER OF RECORD:
JOINT VENTURE
FINLEY McNARY ENG. &
JANSSEN & SPAANS ENG.

LOGO:

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
S.R. 10	JACKSON/GADSDEN	53020-3540

SHEET TITLE: END BENT 25 DETAILS

PROJECT NAME: S.R.10 OVER THE WESTERN FLOOD PLAIN

Drawing No.

Index No.

E:\094\094FBT2

APPENDIX L – FDOT INSPECTIONS COMPLETE

BRIDGE NO.	780074	FLORIDA DEPARTMENT OF TRANSPORTATION ***********************************	LOCATION	SR-A1A (BRIDGE OF LIONS) OVER MATANZAS RIVER
COUNTY SECTION NO.	78040000		INSPECTION DATE	02/22/16
STATE ROAD NO.	SR-A1A	UNDERWATER	LEAD INSPECTOR	Keith, Hoogland, CBI 00341
U.S. ROAD NO.	N/A	BRIDGE INSPECTION REPORT	MILE POST NO.	16.693

Routine Underwater Bridge Inspection Report
BOLTUNDERWATER SERVICES, INC.
for
BURGESS & NIPLE, INC.

Structure/Roadway Identification:

District (2):	02
County (3):	St. John's
Feature Intersected (6):	Matanzas River IWW
Facility Carried (7):	SR-A1A (Bridge of Lions)

Underwater Inspection Details:

Special Crew Hours:	24.0
Max. Depth:	40' at Bent 8
Type of Dive Insp.:	Level II (SCUBA)
Type of Boat Used:	23' Skiff
Water Type/Marine Growth:	Salt/Tidal – Oysters

Previous Inspection:

Lead Diver:	C.B.I. No.:	Inspection Date:
Durbin, James	00465	02/19/14

Inspection Personnel:

Field Personnel:	Title	C.B.I. No.:	Duty:	Signature:
Hoogland, Keith S.	C.B.I. Diver-Inspector	00341/Lead	Dive	KAH
Payne, Timothy N.	Diver-Inspector		Dive	
Jensen, Denise R.	Diver-Inspector		Tend	
Belangia, Korye A.	Diver		Tend	

PILES/COLUMNS
ELEMENT: 205 R/CONCRETE **102: ea.**

NOTE: The intermediate bents are multi-column piers. Abutments 1 and 25 each with two reinforced concrete piles, Bents 2 through 9 and 12 through 20 have two concrete columns above the footings. Bents 2 through 6 and Bents 15 through 18 have submerged footings supported by two drilled shaft reinforced concrete columns. Bents 7, 8, 9, 12, 13 and 14 have submerged footing supported by four drilled shafts reinforced concrete columns. Bascule Piers 10 and 11 with submerged footings supported by eleven reinforced concrete drilled shafts. Piers 19 and 20 have no visible drilled shafts. Exposed drill shafts have the steel casing in place from the footing to the groundline.

Condition State:	QTY:	Recommended Feasible Action:
CS-1	**102**	**Do Nothing**

Some columns above the footings have minor areas of peeling exterior coating, up to 8" H x 5" W x ¼" D – NO CHANGE.

NOTE: The opening in form on the SW quadrant of drilled shaft 9-3 was not found this inspection.

INCIDENTAL:
The SW and NW drilled shafts on Pier 10, NE and SE on Pier 11 have larger round steel casings extending up from the groundline to within 6' of the bottom of the footing.

Cleaning Log: Strips were cleaned on Piers 2-1, 2-2, 5-2, 6-2 and 17-2.

BRIDGE NO.	780074	FLORIDA DEPARTMENT OF TRANSPORTATION *************************************** BRIDGE INSPECTION REPORT	LOCATION	SR-A1A (BRIDGE OF LIONS) OVER MATANZAS RIVER IWW
COUNTY SECTION NO.	78040000		INSPECTION DATE	2/19/16
STATE ROAD NO.	SR-A1A		LEAD INSPECTOR	Roger Aker, CBI 00401
U.S. ROAD NO.	N/A		MILE POST NO.	16.693

SOUNDINGS PROFILE

Bridge No: 780074

County Section No: 78040000

State Road No: A1A

U.S. Road No:

Location: SR-A1A (Bdg. of Lions) over Matanzas River IWW

Inspection Date: 02/23/2016

Inspector: P. Sawyer

PROFILE COMPARISON REPORT

Date Groundline Recorded 08/23/2011

Bent No.	Action Left	Action Right	Critical Left	Critical Right	Lead Line Measurement Left	Lead Line Measurement Right
1					0.00	0.00
2					22.10	13.70
3					35.00	27.60
4					29.80	36.29
5					31.90	34.39
6					37.60	36.29
7					39.70	37.40
8					37.20	36.69
9					36.00	34.79
10					21.70	32.80
11					28.20	28.99
12					35.10	33.50
13					33.00	30.80
14					29.10	32.80
15					28.60	31.29
16					23.00	27.09
17					21.60	20.10
18					17.90	19.39
19					16.60	20.10
20					13.30	20.00

Comments:
Waterline reference at Column 2-1 outside top offset +7.5 elevation. All measurements are negative elevations calculated from 0.0 elevation.

Date Groundline Recorded 02/19/2014

Bent No.	Action Left	Action Right	Critical Left	Critical Right	Lead Line Measurement Left	Lead Line Measurement Right
1					0.00	0.00
2					14.70	12.90
3					18.40	27.00
4					32.00	32.20
5					33.00	32.90
6					33.00	35.10
7					36.70	34.10
8					35.50	34.30
9					35.00	32.20
10					25.20	29.40
11					21.30	24.00
12					30.90	27.50
13					28.50	27.60
14					27.70	29.10
15					27.40	26.80
16					21.60	24.00
17					19.20	20.70
18					17.00	19.00
19					10.70	15.60
20					13.90	15.70

Comments:
Waterline reference at Column 2-1 outside top offset +7.5 elevation. All measurements are negative elevations calculated from 0.0 elevation.

Date Groundline Recorded 02/23/2016

Bent No.	Action Left	Action Right	Critical Left	Critical Right	Lead Line Measurement Left	Lead Line Measurement Right
1					0.00	0.00
2					17.50	16.40
3					27.90	29.80
4					32.20	33.20
5					33.40	35.00
6					32.90	36.00
7					38.70	37.90
8					38.40	38.80
9					37.20	39.90
10					27.20	31.30
11					25.30	26.60
12					32.60	33.30
13					31.20	32.10
14					30.80	33.10
15					28.50	28.80
16					24.10	25.80
17					22.80	22.20
18					18.90	21.30
19					17.10	20.20
20					16.10	19.10

Comments:
An FDOT 4in. aluminum disk has been set in a 3ft. x 3ft. concrete pile located on the southeast fender system south of Pier 10, having an elevation of 3.92ft. (NAVD88). Hydrographic measurements were made with a Reson 7125 multibeam sounder configured with a 400 KHz transducer.

BRIDGE NO.	780074	FLORIDA DEPARTMENT OF TRANSPORTATION ************************************ BRIDGE INSPECTION REPORT	LOCATION	SR-A1A (BRIDGE OF LIONS) OVER MATANZAS RIVER IWW
COUNTY SECTION NO.	78040000		INSPECTION DATE	2/19/16
STATE ROAD NO.	SR-A1A		LEAD INSPECTOR	Roger Aker, CBI 00401
U.S. ROAD NO.	N/A		MILE POST NO.	16.693

SOUNDINGS PROFILE

Bridge No:	780074		
County Section No:	78040000	Location:	SR-A1A (Bdg. of Lions) over Matanzas River IWW
State Road No:	A1A	Inspection Date:	02/23/2016
U.S. Road No:		Inspector:	P. Sawyer

PROFILE COMPARISON REPORT

Date Groundline Recorded 08/23/2011							Date Groundline Recorded 02/19/2014							Date Groundline Recorded 02/23/2016						
Bent	Action		Critical		Lead Line Measurement		Bent	Action		Critical		Lead Line Measurement		Bent	Action		Critical		Lead Line Measurement	
No.	Left	Right	Left	Right	Left	Right	No.	Left	Right	Left	Right	Left	Right	No.	Left	Right	Left	Right	Left	Right
21					9.90	17.60	21					10.80	15.30	21					13.40	17.30
22					9.40	12.20	22					9.00	10.80	22					11.40	12.20
23					7.00	8.39	23					7.60	8.10	23					0.00	0.00
24					6.00	6.49	24					5.70	4.80	24					0.00	0.00
25					0.00	0.00	25					0.00	0.00	25					0.00	0.00

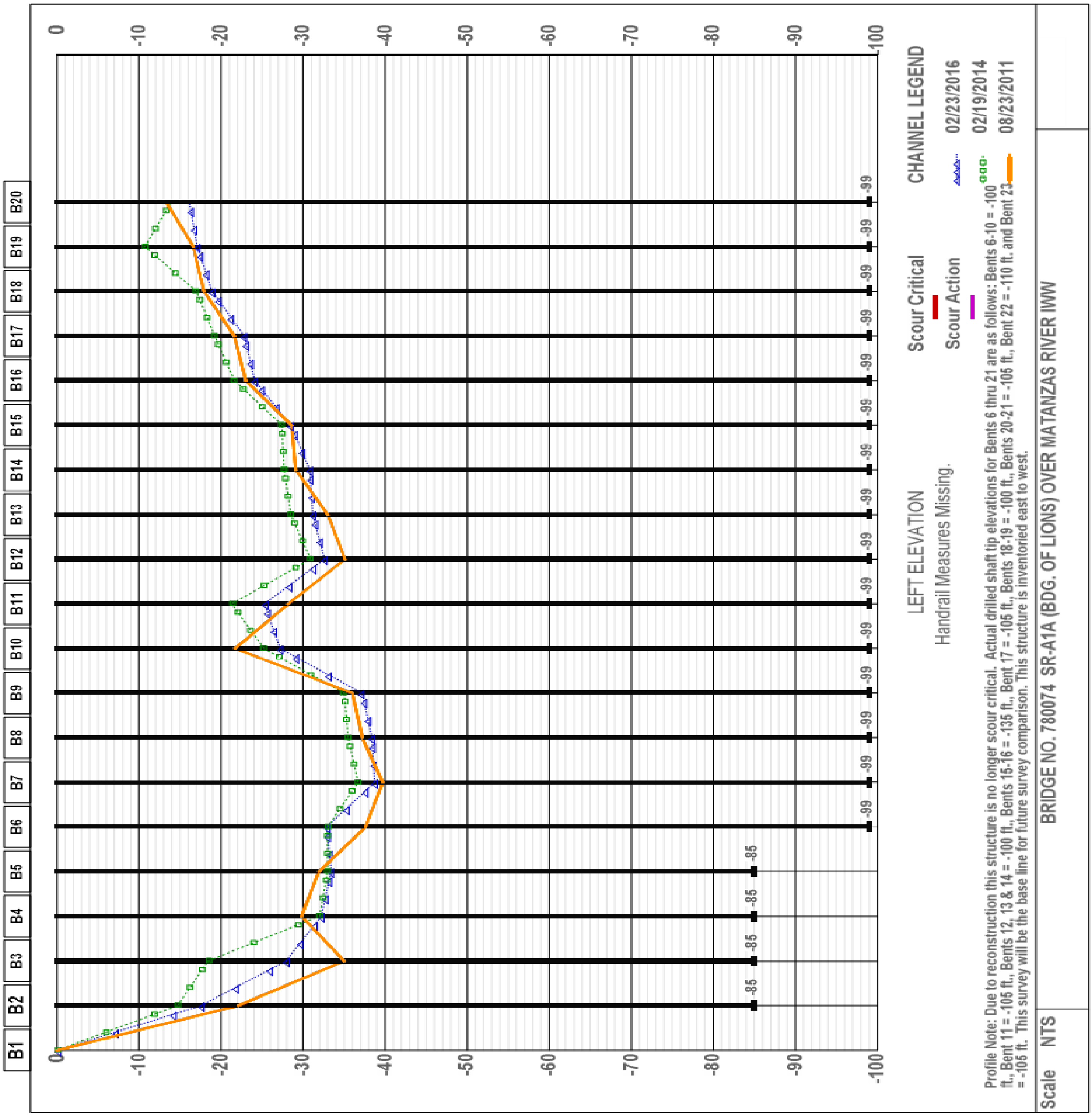
Comments:
Waterline reference at Column 2-1 outside top offset +7.5 elevation. All measurements are negative elevations calculated from 0.0 elevation.

Comments:
Waterline reference at Column 2-1 outside top offset +7.5 elevation. All measurements are negative elevations calculated from 0.0 elevation.

Comments:
An FDOT 4in. aluminum disk has been set in a 3ft. x 3ft. concrete pile located on the southeast fender system south of Pier 10, having an elevation of 3.92ft. (NAVD88). Hydrographic measurements were made with a Reson 7125 multibeam sounder configured with a 400 KHz transducer.

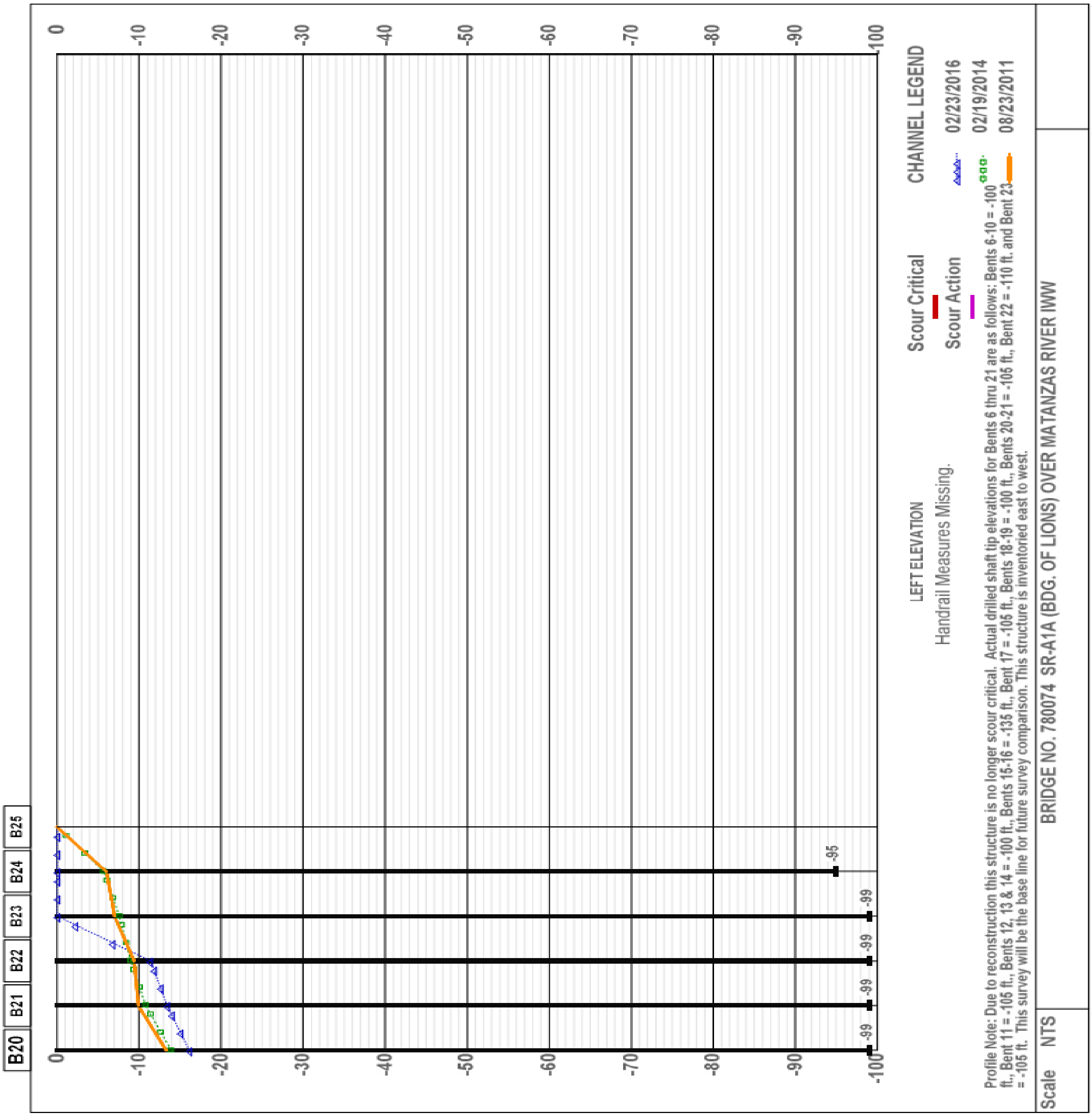
BRIDGE NO.	780074	FLORIDA DEPARTMENT OF TRANSPORTATION ************************************ BRIDGE INSPECTION REPORT	LOCATION	SR-A1A (BRIDGE OF LIONS) OVER MATANZAS RIVER IWW
COUNTY SECTION NO.	78040000		INSPECTION DATE	2/19/16
STATE ROAD NO.	SR-A1A		LEAD INSPECTOR	Roger Aker, CBI 00401
U.S. ROAD NO.	N/A		MILE POST NO.	16.693

SOUNDINGS PROFILE



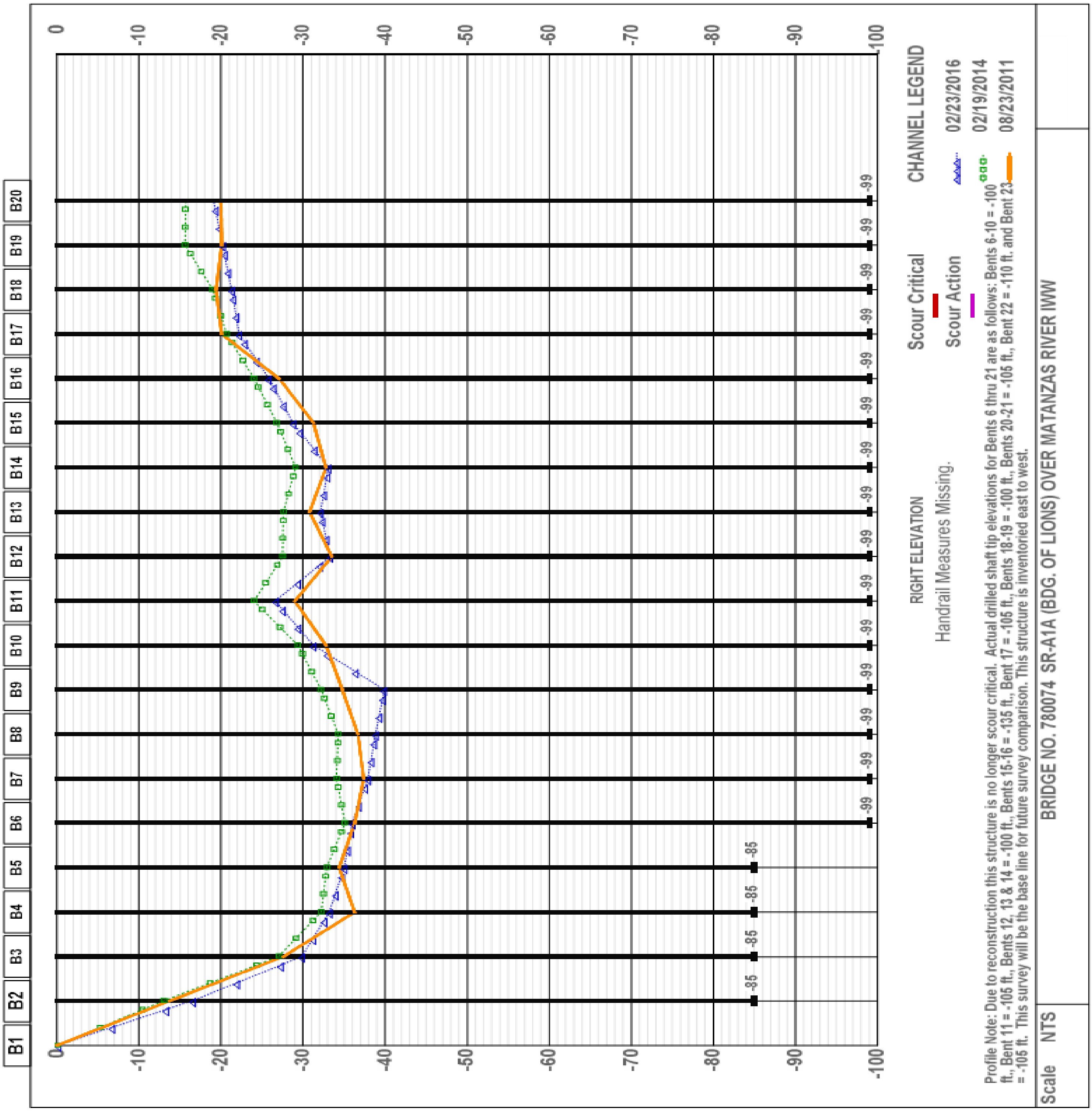
BRIDGE NO.	780074	FLORIDA DEPARTMENT OF TRANSPORTATION	LOCATION	SR-A1A (BRIDGE OF LIONS) OVER MATANZAS RIVER IWW
COUNTY SECTION NO.	78040000	************************************	INSPECTION DATE	2/19/16
STATE ROAD NO.	SR-A1A	BRIDGE INSPECTION REPORT	LEAD INSPECTOR	Roger Aker, CBI 00401
U.S. ROAD NO.	N/A		MILE POST NO.	16.693

SOUNDINGS PROFILE



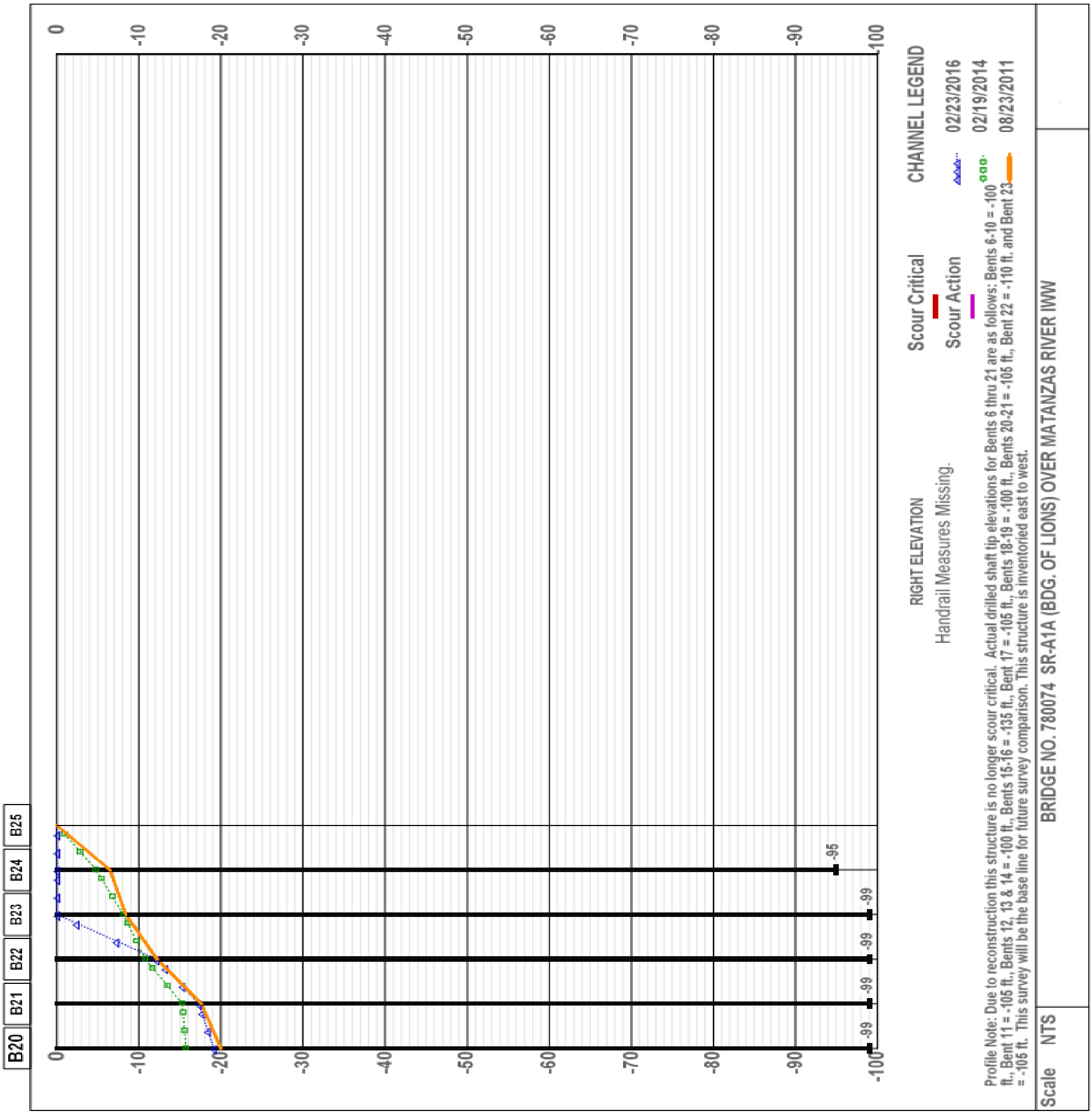
BRIDGE NO.	780074	FLORIDA DEPARTMENT OF TRANSPORTATION ************************************ BRIDGE INSPECTION REPORT	LOCATION	SR-A1A (BRIDGE OF LIONS) OVER MATANZAS RIVER IWW
COUNTY SECTION NO.	78040000		INSPECTION DATE	2/19/16
STATE ROAD NO.	SR-A1A		LEAD INSPECTOR	Roger Aker, CBI 00401
U.S. ROAD NO.	N/A		MILE POST NO.	16.693

SOUNDINGS PROFILE



BRIDGE NO.	780074	FLORIDA DEPARTMENT OF TRANSPORTATION ************************************* BRIDGE INSPECTION REPORT	LOCATION	SR-A1A (BRIDGE OF LIONS) OVER MATANZAS RIVER IWW
COUNTY SECTION NO.	78040000		INSPECTION DATE	2/19/16
STATE ROAD NO.	SR-A1A		LEAD INSPECTOR	Roger Aker, CBI 00401
U.S. ROAD NO.	N/A		MILE POST NO.	16.693

SOUNDINGS PROFILE



Routine Underwater Bridge Inspection Report
BOLT UNDERWATER SERVICES, INC.
for
KISINGER CAMPO & ASSOCIATES, CORP.

NBI Structure ID. (8): **150244** Underwater Date (93): 12/03/15

Structure/Roadway Identification:

District (2): 07
County (3): Pinellas
Feature Intersected (6): Clearwater Harbor
Facility Carried (7): SR-60

Underwater Inspection Details:

Special Crew Hours: 6.0
Max. Depth: 16ft. at Footing 6
Type of Dive Insp.: Level II (SCUBA)
Type of Boat Used: 23ft. Skiff
Water Type/Marine Growth: Salt/Tidal – Barnacles/Oysters

Previous Inspection:

Lead Diver:	C.B.I. No.:	Inspection Date:
Griswold, Mollie A.	00366	12/09/13

Inspection Personnel:

Field Personnel:	Title	C.B.I. No.:	Duty:	Signature:
Hoogland, Keith S.	C.B.I. Diver-Inspector	00341/Lead	Dive	KSH
Salazar, Pete Jr.	Diver-Inspector		Dive	
Jensen, Denise R.	Diver		Tend	

PILES/COLUMNS
ELEMENT: 205 R/CONCRETE **9**: ea.

NOTE: This quantity includes the nine drilled shafts with casings that are visible under footings 5, 6 and 7. Only the SW column is visible under Footing 7 (all four are exposed on Footings 5 and 6).

Condition State:	QTY:	Recommended Feasible Action:
CS-1	**9**	**Do Nothing**

The steel casings are still in-place from the footing to the groundline. The steel has light pitting and light corrosion.

Cleaning Log: No cleaning was done due to steel casings being in place.

SUBMERGED FOOTING
ELEMENT: 220 R/CONCRETE **3**: ea.

Condition State:	QTY:	Recommended Feasible Action:
CS-2	**3**	**Do Nothing**

The pre-cast footer slabs utilized at bottom forms are broken up with exposed rebar and are on the channel bottom or hanging between the footer and channel bottom.

There footings have vertical cracks up to full height x *1/32in.* on the sides of the footings – INCREASE.

Piers 5, 6 and 7 have impact spalls up to 5ft. H x 24in. W x ½in. D, primarily at the corners, extending up from the top of the marine growth.

Footing 5 east face, 29ft. from the north end, void, 4in. H x 2in. W, 7ft. below the top of the footing and a ruler can penetrate vertically up to 30in.

Cleaning Log: Strips on Footing 7 were cleaned.

CHANNEL
ELEMENT: 290 **1**: ea.

Condition State:	QTY:	**Recommended Feasible Action:**
CS-2	**1**	**Do Nothing**

There is pre-cast slab concrete debris and rebar under the footer which is broken and hanging down or accumulated on the channel bottom. There is a possible chance of previous slab debris moving.

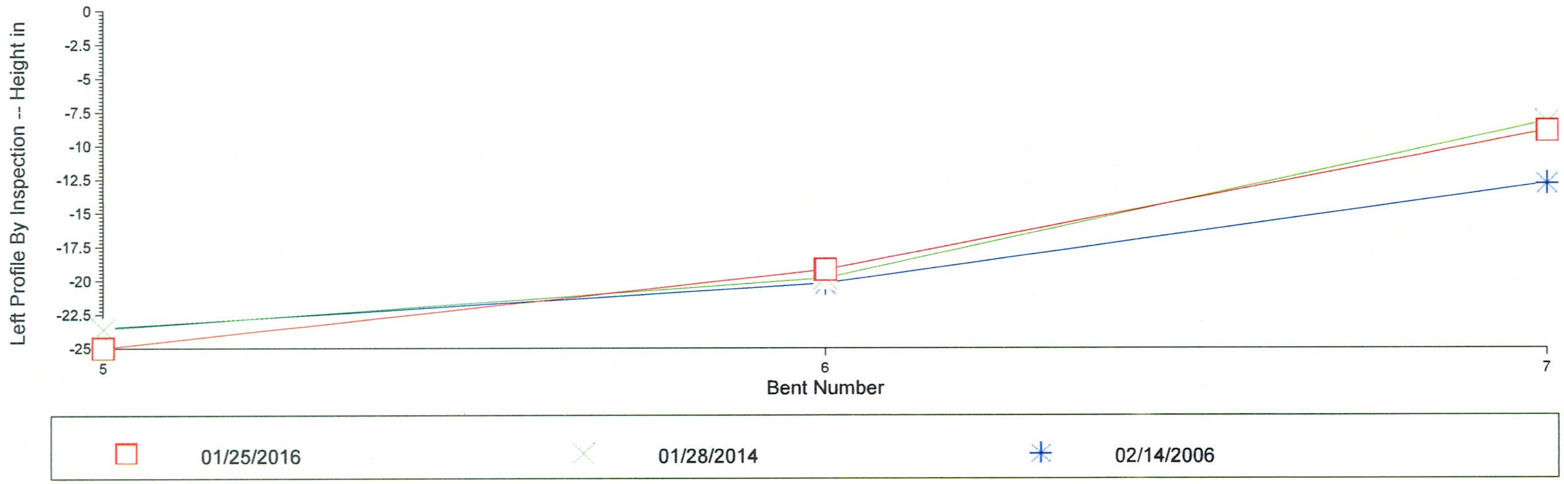
INSPECTION NOTES: Divers inspected nine concrete drilled shafts in Bents 5 through 7, Footings and Channel.
STRUCTURE NOTES: Structure inventoried west to east.

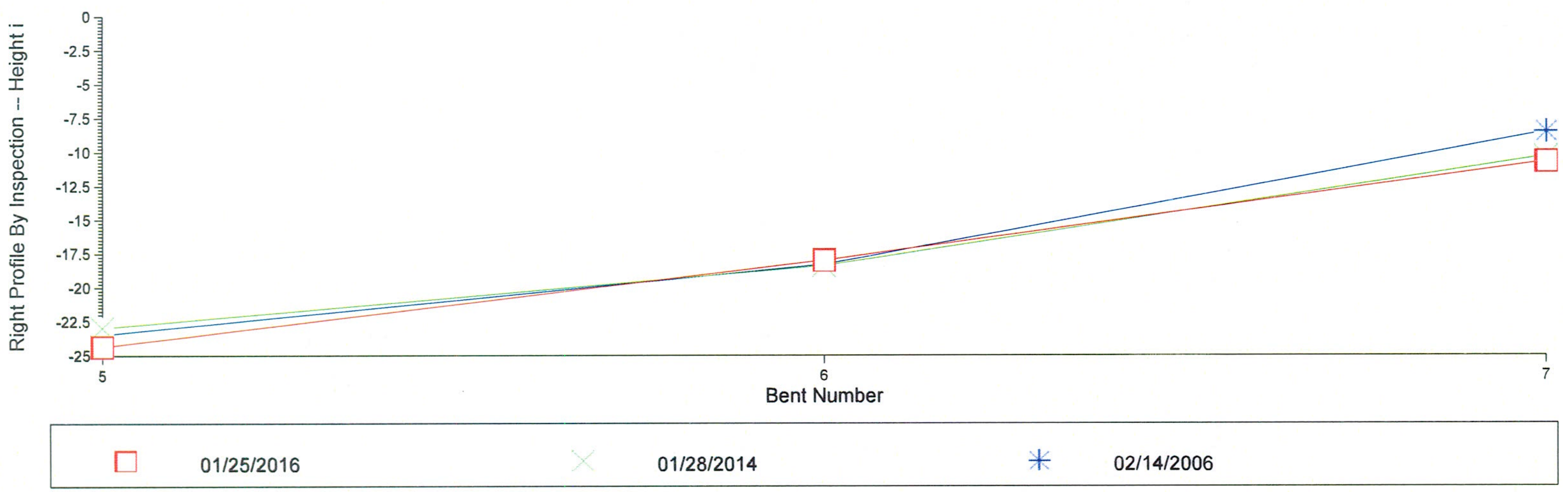
FLORIDA DEPARTMENT OF TRANSPORTATION BRIDGE MANAGEMENT SYSTEM
Bridge Profile Report

REPORT ID: INVT016

Structure #: 150244

DATE PRINTED: 03/02/2016





FLORIDA DEPARTMENT OF TRANSPORTATION BRIDGE MANAGEMENT SYSTEM
Bridge Profile Report

REPORT ID: INVT016
Structure #: 150244

DATE PRINTED: 03/02/2016

Profile Data - Numerical Summary

(All Heights Are In Feet)

Inspection Date and Key: 01/25/2016 LBMN

Bent #	Left Height	Right Height
5	25	24.4
6	19.2	18
7	8.9	10.7

Air Temp: 1
Profile Notes:
Profile Reference: Top of Pier 5 footer
Waterline: 6.1ft.
Pier 4 is dry on the left and right sides.

Inspection Date and Key: 01/28/2014 QKTE

Bent #	Left Height	Right Height
5	23.6	23
6	19.8	18.4
7	8.2	10.3

Air Temp: 1
Profile Notes:
Profile Reference: Top of Pier 5 footer
Waterline: 5ft.
Pier 4 is dry on the left and right side.

Inspection Date and Key: 02/14/2006 JARY

Bent #	Left Height	Right Height
5	23.5	23.5
6	20.2	18.3
7	12.8	8.5

Air Temp:
Profile Notes:
Profile Reference: Top of Pier 5 Footing
Channel Depth: 17.5FT

Routine Underwater Bridge Inspection Report
BOLT UNDERWATER SERVICES, INC.
for
KISINGER CAMPO & ASSOCIATES, CORP.

NBI Structure ID. (8): **155522** Underwater Date (93): 12/30/16

Structure/Roadway Identification:		**Underwater Inspection Details:**	
District (2):	01	Special Crew Hours:	23.2
County (3):	Pinellas	Max. Depth:	35ft. at Bent 15
Feature Intersected (6):	Clearwater Pass Bridge	Type of Dive Insp.:	Level II (SCUBA)
Facility Carried (7):	CR 183 & 699 (Gulf Blvd.)	Type of Boat Used:	23ft. Skiff
		Water Type/Marine Growth:	Salt – Barnacles/Oysters

Previous Inspection:

Lead Diver:	**C.B.I. No.:**	**Inspection Date:**
Hoogland, Keith S.	00341	12/12/14

Inspection Personnel:

Field Personnel:	**Title**	**C.B.I. No.:**	**Duty:**	**Signature:**
Hitch, Victoria G.	C.B.I. Diver-Inspector	00414/Lead	Tend	
Hoogland, Keith S.	C.B.I. Diver-Inspector	00341	Dive	
Jensen, Denise R.	Diver-Inspector		Dive	
Brewer, James D.	Diver		Dive	
Goldman, Derek B.	Diver		Tend	

CHANNEL
ELEMENT: 8290 **1** EA.

NO NOTES.

RE CONC SUB PILE CAP/FTG
ELEMENT: 220 **432** FT.

CS-2 1130

The footings have vertical cracks up to 3ft. 6in. L x 1/32in. W, in the following locations:

Footing	Number of cracks North side	Number of cracks South side	Quantity
4	3	1	4FT.
5	0	0	0FT.
6	0	2	2FT.
7	0	4	4FT.
8	0	1	1FT.
9	0	3	3FT.
10	0	0	0FT.
11	1	3	4FT.
12	0	3	3FT.
13	0	4	4FT.
14	0	1	1FT.
15	0	1	1FT.
16	0	1	1FT.
17	1	3	4FT.
18	2	3	5FT.

CS-3 1080

Footing 17: North face, 3ft. from east end, 4ft. below top of footing, void, 6in. H x 6in. W x 4in. D.

INCIDENTAL:

There are minor washouts between the footing and seal, and portions of the seal are missing. Approximate footing thickness is 6ft.

Cleaning Log: Footing 5 and 6 were cleaned.

This report contains information relating to the physical security of a structure and depictions of the structure. This information is confidential and exempt from public inspection pursuant to sections 119.071(3)(a) and 119.071(3)(b), Florida Statutes.

RE CONC COLUMN
ELEMENT: 205 28 EA.

NOTE: There are no columns (shafts) visible under the footings at Pier 4 and Pier 19.
The columns on Piers 5 through Pier 18 have steel casings in place. On several of the columns, the bottom edge of the casings are visible up to 4ft. above the groundline. The concrete is irregular with no exposed steel. Refer to Element 8290 for the distance of the casings to the groundline.

CS-2 6000
Scour dishes/irregular bottom around the columns were up to 5ft. deep and extending up to 6ft. out from the columns.

Distance from bottom of casings to groundline:

Column	2009	2014	2016
8-1	N/A	12in	19in.
8-2	N/A	14in.	14in.
9-1	N/A	4ft.	4ft.
9-2	28in.	34in.	3ft. 6in.
10-1	12in.	18in.	28in.
10-2	6in.	20in.	28in.
13-1	11in.	16in.	14in.
13-2	6in.	12in.	17in.
14-1	22in.	3ft. 8in.	3ft. 6in.
14-2	8in.	22in.	20in.
15-1	–	–	7in.
15-2	8in.	0in.	3ft. 6in.

Changes greater than 24in. in the above measurements may be due to strong currents and loose channel bottom.

INCIDENTAL:
The steel casings have some areas of light corrosion and minor pitting up to 1/16in. deep.

Cleaning Log: No cleaning due to steel casings on the visible shafts.

PS FENDER/DOLPHIN
ELEMENT: 8387 502 FT.

NOTE: This element represents the epoxy jackets on the east plumb and batter piles, approximately 8ft. 3in. below the bottom wale.

CS-3 1080
North Fender:
West end 1st batter pile on NW edge at top of marine growth, spall, 30in. H x 6in. W x *3in. D* (below this spall there is a sound epoxy patch). INCREASE. 1FT.

The connecting hardware holes have spalls up to 8in. diameter x 1in. deep (approximately 10 per side). 20FT.

CS-4 1110
South Fender:
3rd plumb pile from the east at groundline on the south face is fractured, up to 24in. high and extending into the west and east faces. 1FT.

5th plumb pile from the east at groundline on all four faces is fractured, up to 18in. high. *One piece of pre-stress cable has up to 100% section loss.* INCREASE. 1FT.

3rd batter pile from east at groundline on the south face is fractured up to 18in. high. NEW. 1FT.

INCIDENTAL:
Connecting hardware has minor to moderate corrosion.

Several of the lower wales have areas that are deteriorated up to 10% due to marine borer activity, with areas up to 20% section loss typically at the connecting hardware.

INSPECTION NOTES: Divers inspected Channel, Footings, Piers 5 through 18 with twenty-eight drill shaft casings, Jackets and Fender System.
STRUCTURE NOTES: Structure inventoried south to north.

PHOTO LOG:
No. 1: Structure ID.
No. 2: East elevation
No. 3, 4: South fender 5th plum pile from east, fractured pile with exposed cable
No. 5: South fender 3rd batter pile from east, fractured pile
No. 6: Typical corrosion on lower connecting hardware
No. 7, 8: Fender lower wales, typical section loss
No. 9, 10: North fender west batter, NW corner, spall
No. 11, 12: Typical spall on fender pile at lower bolts
No. 13, 14: Footing 17 north face, void

REPORT ID : INSP005

Structure ID : 155522

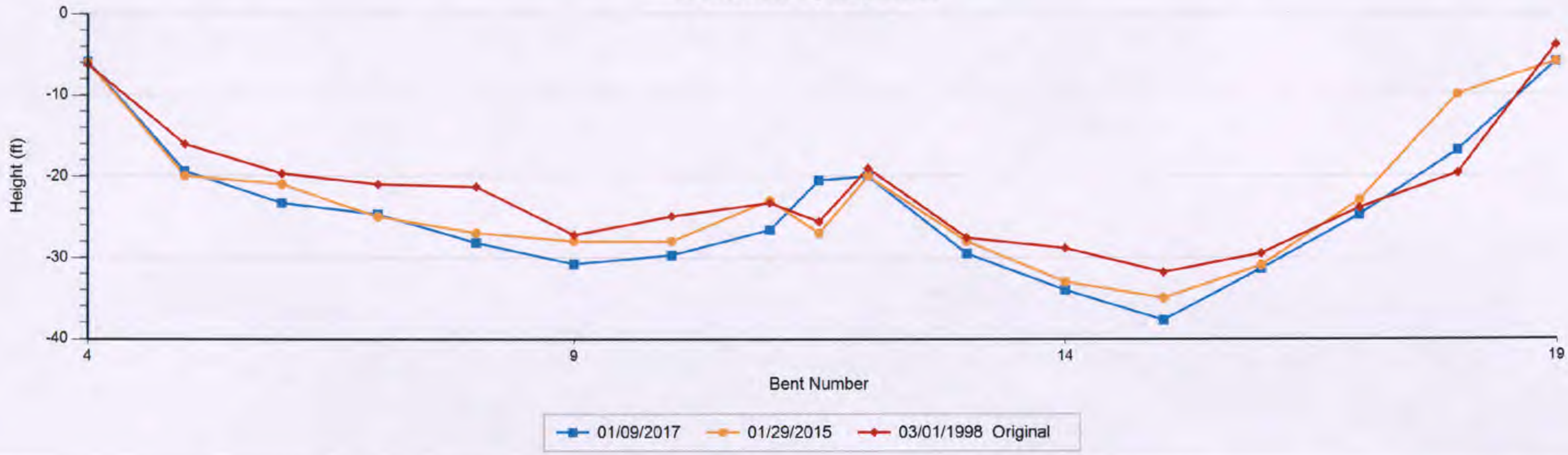
FLORIDA DEPARTMENT OF TRANSPORTATION BRIDGE MANAGEMENT SYSTEM

Bridge Profile Report

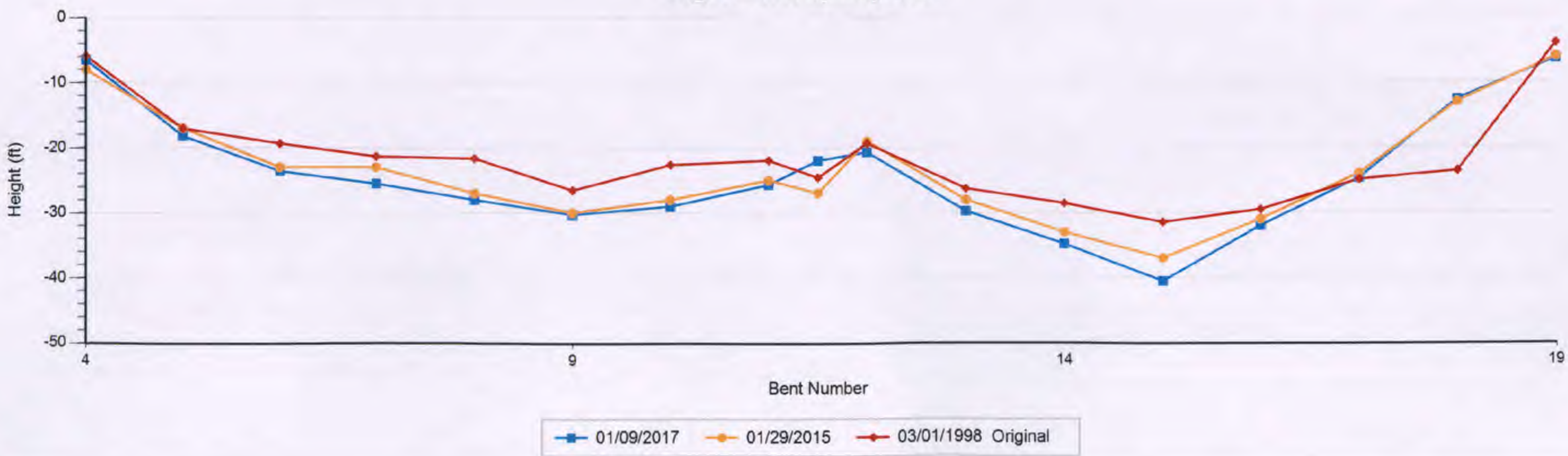
Bridge Profile

DATE PRINTED: 2/17/2017 8:27:19 AM

Left Profile by Inspection



Right Profile by Inspection



REPORT ID : INSP005
Structure ID : 155522

FLORIDA DEPARTMENT OF TRANSPORTATION BRIDGE MANAGEMENT SYSTEM
Bridge Profile Report
Bridge Profile

DATE PRINTED: 2/17/2017 8:27:19 AM

Profile Data - Numerical Summary

(All Heights are in Feet)

Inspection Date and Key: 1/9/2017 WULU

Bent #	Left Height	Right Height
4	5.90	6.50
5	19.40	18.20
6	23.30	23.60
7	24.70	25.50
8	28.20	28.00
9	30.80	30.30
10	29.70	29.00
11	26.60	25.70
11.5	20.50	22.00
12	20.00	20.70
13	29.50	29.70
14	34.00	34.80
15	37.70	40.50
16	31.40	32.00
17	24.80	24.70
18	16.90	12.60
19	6.00	6.30

Air Temp:
Profile Notes:

Profiles referenced to top of Footer 12.
Waterline = 3.2ft.
*Some measurements have 2ft. or greater differences from the previous inspection. This condition is possibly due to strong currents and a loose channel bottom.

Inspection Date and Key: 1/29/2015 NBCM

Bent #	Left Height	Right Height
4	6.00	8.00
5	20.00	17.00

REPORT ID : INSP005

Structure ID : 155522

FLORIDA DEPARTMENT OF TRANSPORTATION BRIDGE MANAGEMENT SYSTEM

Bridge Profile Report

Bridge Profile

DATE PRINTED: 2/17/2017 8:27:19 AM

Profile Data - Numerical Summary

(All Heights are in Feet)

Bent #	Left Height	Right Height
6	21.00	23.00
7	25.00	23.00
8	27.00	27.00
9	28.00	30.00
10	28.00	28.00
11	23.00	25.00
11.5	27.00	27.00
12	20.00	19.00
13	28.00	28.00
14	33.00	33.00
15	35.00	37.00
16	31.00	31.00
17	23.00	24.00
18	10.00	13.00
19	6.00	6.00

Air Temp: 1

Profile Notes:

Profiles referenced to top of Footer 12.
Waterline = 4ft.
Some measurements have 2ft. or greater differences from the previous inspection. This condition is possibly due to strong currents and a loose channel bottom.

Inspection Date and Key: 3/1/1998 STRT

(Original Inspection)

Bent #	Left Height	Right Height
4	6.23	5.91
5	16.08	17.06
6	19.69	19.36
7	21.00	21.33

REPORT ID : INSP005
Structure ID : 155522

FLORIDA DEPARTMENT OF TRANSPORTATION BRIDGE MANAGEMENT SYSTEM
Bridge Profile Report
Bridge Profile

DATE PRINTED: 2/17/2017 8:27:19 AM

Profile Data - Numerical Summary

(All Heights are in Feet)

Bent #	Left Height	Right Height
8	21.33	21.65
9	27.23	26.57
10	24.93	22.64
11	23.29	21.98
11.5	25.59	24.61
12	19.03	19.36
13	27.56	26.25
14	28.87	28.54
15	31.82	31.50
16	29.53	29.53
17	23.95	24.93
18	19.69	23.62
19	3.94	3.94

Air Temp:
Profile Notes:

BRIDGE NO.	720633	FLORIDA DEPARTMENT OF TRANSPORTATION **************************************	LOCATION	I-95 SB Off Ramp over St. Johns River
COUNTY SECTION NO.	72020065		INSPECTION DATE	08/10/15
STATE ROAD NO.	N/A	UNDERWATER	LEAD INSPECTOR	Qualls, Dion C. CBI 00470
U.S. ROAD NO.	I-95	BRIDGE INSPECTION REPORT	MILE POST NO.	0.016

BOLT UNDERWATER SERVICES, INC.
for
BURGESS & NIPLE, INC.

Structure/Roadway Identification:
District (2): 02 State
County (3): Duval
Feature Intersected (6): St. Johns River
Facility Carried (7): I-95 SB Off Ramp

Underwater Inspection Details:
Special Crew Hours: 6.0
Max. Depth: 18' at Bent 6
Type of Dive Insp.: Level II (SCUBA)
Type of Boat Used: 23' Skiff
Water Type/Marine Growth: Brackish/Tidal – Barnacles/Oysters

Prev. Inspection:

Lead Diver:	**C.B.I. No.:**	**Inspection Date:**
Hitch, Victoria G.	00414	08/26/13

Inspection Personnel:

Field Personnel:	**Title**	**C.B.I. No.:**	**Duty:**	**Signature:**
Qualls, Dion C.	C.B.I. Diver-Inspector	00470/Lead	Dive/Tend	DCQ
Hoogland, Keith S.	C.B.I. Diver-Inspector	00341	Tend	
Salazar, Pete Jr.	Diver-Inspector		Dive/Tend	
Payne, Timothy N.	Diver-Inspector		Tend	

PILES/COLUMNS
ELEMENT: 205 R/CONCRETE **13**: ea.
NBI: **8**

NOTE: Piers 4, 5 and 6 each have four 36" drilled shafts. The steel drilled shaft casings remain in-place. One pile is partially visible under Footing 3. Pier 7 is a shared pier also Pier 1 of Structure No. 720629 (this Pier was quantified and evaluated with Structure No. 720269).

Condition State:	**QTY:**	**Recommended Feasible Action:**
CS-1	**13**	**Do Nothing**

Cleaning Log: No cleaning due to steel casings in place.

SUBMERGED FOOTING
ELEMENT: 220 R/CONCRETE **4**: ea.
NBI: **8**

NOTE: The waterline footings are constructed by precast concrete forms 18" thick, then sealed with 6" of concrete seal. The footing thickness is 6' and the combined total thickness is 8' Piers 3, 4, 5 and 6 were evaluated under this element. Pier 7 is a shared pier and also Pier 1 of Structure No. 720629 (this Pier was quantified and evaluated with Structure No. 720629).

Condition State:	**QTY:**	**Recommended Feasible Action:**
CS-1	**4**	**Do Nothing**

Several footings have areas of precast slabs that are hanging and broken down at the edges and undersides with exposed rebar
– *INCREASE.*

Cleaning Log: Random strips on Footing 5 were cleaned.

CHANNEL
ELEMENT: 290 **1**: ea.
NBI: **7**

NOTE: This structure joins the main river crossing, Structure No. 720629 over the St. Johns River.

Condition State:	**QTY:**	**Recommended Feasible Action:**
CS-2	**1**	**Do Nothing**

There is construction debris on the channel bottom below the footings – *NEW.*

BRIDGE NO.	720633	FLORIDA DEPARTMENT OF TRANSPORTATION ************************************ **UNDERWATER** **BRIDGE INSPECTION REPORT**	LOCATION	I-95 SB Off Ramp over St. Johns River
COUNTY SECTION NO.	72020065		INSPECTION DATE	08/10/15
STATE ROAD NO.	N/A		LEAD INSPECTOR	Qualls, Dion C. CBI 00470
U.S. ROAD NO.	I-95		MILE POST NO.	0.016

SUB STRUCTURE ELEMENTS INSPECTED

Elements of the substructure inspected include the following:

- Bent 3 with one footing and one visible pile
- Bents 4 through 6 each with four drilled shafts and one footing
- Bent 7 – This Bent is common with Structure No. 720629; bent quantity is listed with that report
- Channel

INSPECTION CONDITIONS

Arrive on site:	12:00	Growth:	Barnacles/Oysters
Visibility:	12”	Water Type:	Brackish/Tidal
Water Temp.:	88°F	Wind:	Light
Air Temp.:	90°F	Weather:	Cloudy
Year Built:	2003	Current:	Strong
Bottom:	Mud/Sand		

Top of footing to waterline: 5’ at Footing 6-1
Top of footing to top of marine growth: 4’ 6” at Footing 6-1

Travel: 24.0 Field: 6.0 Report: 1.0

EQUIPMENT USED

SCUBA	Chipping Hammers	U/W Lights
Dive Flag	Rulers	Digital Depth Monitor
1 Vehicle	23’ Boat & Motor	

INSPECTION NOTES:
STRUCTURE NOTES: Bridge inventoried east to west on south to north route.

BRIDGE NO.	720628	FLORIDA DEPARTMENT OF TRANSPORTATION ***************************************	LOCATION	I-95 NB On Ramp over St. Johns River
COUNTY SECTION NO.	72020064		INSPECTION DATE	08/10/15
STATE ROAD NO.	N/A	**UNDERWATER**	LEAD INSPECTOR	Qualls, Dion C. CBI 00470
U.S. ROAD NO.	I-95	**BRIDGE INSPECTION REPORT**	MILE POST NO.	0.11

BOLT UNDERWATER SERVICES, INC.
for
BURGESS & NIPLE, INC.

Structure/Roadway Identification:

District (2): 02 State
County (3): Duval
Feature Intersected (6): St. Johns River
Facility Carried (7): I-95 NB On Ramp

Underwater Inspection Details:

Special Crew Hours: 8.0
Max. Depth: 18' at Bent 6
Type of Dive Insp.: Level II (SCUBA)
Type of Boat Used: 23' Skiff
Water Type/Marine Growth: Brackish/Tidal – Barnacles/Oysters

Prev. Inspection:

Lead Diver:	**C.B.I. No.:**	**Inspection Date:**
Hitch, Victoria G.	00414	08/26/13

Inspection Personnel:

Field Personnel:	**Title**	**C.B.I. No.:**	**Duty:**	**Signature:**
Qualls, Dion C.	C.B.I. Diver-Inspector	00470/Lead	Dive/Tend	DCQ
Hoogland, Keith S.	C.B.I. Diver-Inspector	00341	Tend	
Salazar, Pete Jr.	Diver-Inspector		Dive/Tend	
Payne, Timothy N.	Diver-Inspector		Tend	

PILES/COLUMNS
ELEMENT: 205 R/CONCRETE **8**: ea.
NBI: **8**

NOTE: Piers 5 and 6 each have 4 reinforced concrete drilled shafts below the waterline which are visible.

Condition State:	**QTY:**	**Recommended Feasible Action:**
CS-1	**8**	**Do Nothing**

There are steel casings on the drilled shafts below the footings – *NO CHANGE.*

Cleaning Log: Column 5-1 was cleaned.

SUBMERGED FOOTING
ELEMENT: 220 R/CONCRETE **2**: ea.
NBI: **8**

NOTE: The waterline footings are constructed by precast concrete forms 18" thick, then sealed with 6" of concrete seal. The footing thickness is 6' and the combined total thickness is 8'. Divers will be required to inspect the waterline footings of this structure at Piers 5 and 6. Pier 7 of this structure is a shared pier and is Pier 1 of Structure No. 720629 (this Pier is quantified and evaluated under Structure No. 720629).

Condition State:	**QTY:**	**Recommended Feasible Action:**
CS-1	**2**	**Do Nothing**

The footings have areas of precast slabs that are broken and hanging down at the edges and undersides with exposed rebar – *INCREASE.*

CHANNEL
ELEMENT: 290 **1**: ea.
NBI: **8**

NOTE: This structure joins the main river crossing, Bridge 720629 over St. Johns River. Only Spans 5 and 6 are over water.

Condition State:	QTY:	Recommended Feasible Action:
CS-2	**1**	**Do Nothing**

There is construction debris on the channel bottom below the footings – *NEW.*

BRIDGE NO.	720628	FLORIDA DEPARTMENT OF TRANSPORTATION ************************************ UNDERWATER BRIDGE INSPECTION REPORT	LOCATION	I-95 NB On Ramp over St. Johns River
COUNTY SECTION NO.	72020064		INSPECTION DATE	08/10/15
STATE ROAD NO.	N/A		LEAD INSPECTOR	Qualls, Dion C. CBI 00470
U.S. ROAD NO.	I-95		MILE POST NO.	0.11

SUB STRUCTURE ELEMENTS INSPECTED

Elements of the substructure inspected include the following:

- Bents 5 and 6 each with four drilled shafts and one footing
- Bent 7 – This bent is common with Structure No. 720629; bent quantity is listed with that report.
- Channel

INSPECTION CONDITIONS

Arrive on site:	12:30	Growth:	Barnacles
Visibility:	12in.	Water Type:	Brackish/Tidal
Water Temp.:	88°F	Wind:	Light
Air Temp.:	90°F	Weather:	Cloudy
Year Built:	2003	Current:	Strong
Bottom:	Mud/Sand		

Top of footing to waterline: 5’ at Footing 5-1
Top of footing to top of marine growth: 4’ 6” at Footing 5-1

Travel: 24.0 Field: 8.0 Report: 1.0

EQUIPMENT USED

SCUBA	Chipping Hammers	U/W Lights
Dive Flag	Rulers	Digital Depth Monitor
1 Vehicle	23’ Boat & Motor	

INSPECTION NOTES:
STRUCTURE NOTES: Bridge east to west on south to north route.

BOLT UNDERWATER SERVICES, INC.
for
BURGESS & NIPLE, INC.

Structure/Roadway Identification:
District (2): 02 State
County (3): Duval
Feature Intersected (6): St. Johns River
Facility Carried (7): I-95 SB Off Ramp

Underwater Inspection Details:
Special Crew Hours: 6.0
Max. Depth: 18' at Bent 6
Type of Dive Insp.: Level II (SCUBA)
Type of Boat Used: 23' Skiff
Water Type/Marine Growth: Brackish/Tidal – Barnacles/Oysters

Prev. Inspection:

Lead Diver:	**C.B.I. No.:**	**Inspection Date:**
Hitch, Victoria G.	00414	08/26/13

Inspection Personnel:

Field Personnel:	**Title**	**C.B.I. No.:**	**Duty:**	**Signature:**
Qualls, Dion C.	C.B.I. Diver-Inspector	00470/Lead	Dive/Tend	DCQ
Hoogland, Keith S.	C.B.I. Diver-Inspector	00341	Tend	
Salazar, Pete Jr.	Diver-Inspector		Dive/Tend	
Payne, Timothy N.	Diver-Inspector		Tend	

PILES/COLUMNS
ELEMENT: 205 R/CONCRETE **13**: ea.
NBI: **8**

NOTE: Piers 4, 5 and 6 each have four 36" drilled shafts. The steel drilled shaft casings remain in-place. One pile is partially visible under Footing 3. Pier 7 is a shared pier also Pier 1 of Structure No. 720629 (this Pier was quantified and evaluated with Structure No. 720269).

Condition State:	**QTY:**	**Recommended Feasible Action:**
CS-1	**13**	**Do Nothing**

Cleaning Log: No cleaning due to steel casings in place.

SUBMERGED FOOTING
ELEMENT: 220 R/CONCRETE **4**: ea.
NBI: **8**

NOTE: The waterline footings are constructed by precast concrete forms 18" thick, then sealed with 6" of concrete seal. The footing thickness is 6' and the combined total thickness is 8' Piers 3, 4, 5 and 6 were evaluated under this element. Pier 7 is a shared pier and also Pier 1 of Structure No. 720629 (this Pier was quantified and evaluated with Structure No. 720629).

Condition State:	**QTY:**	**Recommended Feasible Action:**
CS-1	**4**	**Do Nothing**

Several footings have areas of precast slabs that are hanging and broken down at the edges and undersides with exposed rebar
– *INCREASE.*

Cleaning Log: Random strips on Footing 5 were cleaned.

CHANNEL
ELEMENT: 290 **1**: ea.
NBI: **7**

NOTE: This structure joins the main river crossing, Structure No. 720629 over the St. Johns River.

Condition State:	**QTY:**	**Recommended Feasible Action:**
CS-2	**1**	**Do Nothing**

There is construction debris on the channel bottom below the footings – *NEW.*

SUB STRUCTURE ELEMENTS INSPECTED

Elements of the substructure inspected include the following:

- Bent 3 with one footing and one visible pile
- Bents 4 through 6 each with four drilled shafts and one footing
- Bent 7 – This Bent is common with Structure No. 720629; bent quantity is listed with that report
- Channel

INSPECTION CONDITIONS

Arrive on site:	12:00	Growth:	Barnacles/Oysters
Visibility:	12”	Water Type:	Brackish/Tidal
Water Temp.:	88°F	Wind:	Light
Air Temp.:	90°F	Weather:	Cloudy
Year Built:	2003	Current:	Strong
Bottom:	Mud/Sand		

Top of footing to waterline: 5’ at Footing 6-1
Top of footing to top of marine growth: 4’ 6” at Footing 6-1

Travel: 24.0 Field: 6.0 Report: 1.0

EQUIPMENT USED

SCUBA	Chipping Hammers	U/W
Lights		
Dive Flag	Rulers	Digital
Depth Monitor		
1 Vehicle	23’ Boat & Motor	

INSPECTION NOTES:
STRUCTURE NOTES: Bridge inventoried east to west on south to north route.

All Elements

UNIT: 0-MAIN SPAN SUBSTRUCTURE

ELEMENT/ENV: 205/4 R/Conc Column **468 ea.** **ELEM CATEGORY:** Substructure

CONDITION STATE (4)	DESCRIPTION	QUANTITY
1	The element shows little or no deterioration. There may be discoloration, efflorescence, and/or superficial cracking but without affect on strength and/or serviceability.	455 ea.
2	Minor cracks, spalls and scaling may be present but there is no exposed reinforcing or surface evidence of rebar corrosion.	13 ea.

ELEMENT INSPECTION NOTES:

NOTE: This element represents 96 R/Conc Columns and 372 drilled shaft columns in the west approach, east approach and main span.

CS1: The following was noted by the underwater inspectors:
There are no visible columns under Footing 2 due to rock rubble.

The majority of the columns have steel casings in-place. Some of the steel casings are covered with a grout coating up to 1in thick. The exposed portions of the steel casings have minor pitting and corrosion.

Several of the exposed portions of the concrete columns have areas of honeycombing up to 1ft x 1ft x 1in

There are several columns with no casings visible or only partial casings remain The exposed concrete is in good condition except as noted below. Many of the concrete columns have been gouged by the burning method used to remove casings. The following list is columns with no visible casings:
4-4, 6-2, 14-4, 26-1, 33-3, 33-4, 40-1, 40-2, 42-2, 42-4, 45-1, 45-2, 45-3, 45-4, 46-1, 46-2, 47-2, 47-4, 48-4, 50-1, 50-2, 50-4, 51-1, 52-1, 52-4, 53-2, 53-3, 53-4, 71-1, 71-4, 75-4, 76-2, 77-4, 78-2, 79-2, 85-2, 85-3, 87-1, 87-2, 90-1, 92-2, 93-3, 93-4, 95-2 and 95-4.

CS2: There are voids in the columns under the footings. Refer to Underwater Table 1 for size and locations.

Elements

Inspection Date: 1/14/2015LBGO

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0-MAIN §	580/4	Navigational Lights	0	.	1	100.	0	.	0	.	0	.	1 ea.

Notes NOTE: This element represents the two swing lights attached to the structure at the centerline of the channel (Span 74).

CS2: Span 74 south and north navigational lights have one of two bulbs missing. Refer to Photo 7. P3WO

CORRECTIVE ACTION TAKEN:
The locking arms for the navigational lights have been repaired.
Install one bulb in the south and north navigational lights - repair not evident. Repeat recommendation.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0-MAIN §	205/4	R/Conc Column	455	97.22	13	2.78	0	.	0	.	0	.	468 ea.

Notes NOTE: This element represents 96 R/Conc Columns and 372 drilled shaft columns in the west approach, east approach and main span.

CS1: The following was noted by the underwater inspectors:
There are no visible columns under Footing 2 due to rock rubble.

The majority of the columns have steel casings in-place. Some of the steel casings are covered with a grout coating up to 1in thick. The exposed portions of the steel casings have minor pitting and corrosion.

Several of the exposed portions of the concrete columns have areas of honeycombing up to 1ft x 1ft x 1in

There are several columns with no casings visible or only partial casings remain The exposed concrete is in good condition except as noted below. Many of the concrete columns have been gouged by the burning method used to remove casings. The following list is columns with no visible casings:
4-4, 6-2, 14-4, 26-1, 33-3, 33-4, 40-1, 40-2, 42-2, 42-4, 45-1, 45-2, 45-3, 45-4, 46-1, 46-2, 47-2, 47-4, 48-4, 50-1, 50-2, 50-4, 51-1, 52-1, 52-4, 53-2, 53-3, 53-4, 71-1, 71-4, 75-4, 76-2, 77-4, 78-2, 79-2, 85-2, 85-3, 87-1, 87-2, 90-1, 92-2, 93-3, 93-4, 95-2 and 95-4.

CS2: There are voids in the columns under the footings. Refer to Underwater Table 1 for size and locations.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0-MAIN §	215/4	R/Conc Abutment	118	100.	0	.	0	.	0	.	0	.	118 lf.

Notes CS1: The top of the southwest cheek wall and the south end of Abutment 1 backwall has a 1ft x 8in delaminating repair - DECREASE. Refer to Photo 8. P3WO

CORRECTIVE ACTION TAKEN:
Repair SPL-DEL in top of the south cheek wall & south face of Abutment 1 backwall - repaired.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0-MAIN §	220/4	R/C Sub Pile Cap/Ftg	69	71.88	14	14.58	13	13.54	0	.	0	.	96 ea.

Notes CS1: It appears that several forms around the footings "bulged" during construction.

The following was noted by the underwater inspectors:
There are minor washouts between the footings and seals. In some areas, portions of the seal have fallen off and the bottom edges of the footings are rounded. At Piers 54 through 89 (larger footers) below the poured seal, there are precast collars on the columns and a precast slab with beams running between the shafts.

Several of the footings have vertical cracks from the top down, up to 2ft long x 1/32in wide.

CS2 & CS3: There are spalls with exposed steel and delaminations in the footing. Refer to Table 2 for sizes and locations, and Photos 9 and 10. P3WO

CORRECTIVE ACTION TAKEN:
Repair 15 deficiencies listed in Table 2 of the 1-31-13 report - repair not evident. Repeat recommendation.

The following deficiencies were not found during the underwater inspection (12/08/14):
Footing 3, spall, northwest corner 30in below top of footing.
Footing 6, crack, southeast corner 20in below top of footing is a form line, previously reported as a crack.
Footing 87, spall/failed patch at the top northeast corner.

TABLE 1
Underwater Inspection
ELEMENT: 205 R/Conc Column

BENT/ SHAFT	LOCATION	DEFICIENCY	SIZE	STATUS	REPAIR
45-3	North quadrant just below footing/seal	Void	30in H x 18in W x 3in D		No
46-1	SW at bottom of footing	Void	3ft H x 16in W x 1in D		No
49-1	SW quadrant 3ft 3in below footing/seal	Void	33in H x 14in W x 3in D (partially buried)		No
49-2	SW quadrant 3ft below footing/seal	Void	3in H x 6in W x 3½in D (appears to be a cut hole in casing)		No
71-1	South quadrant 12in below jacket	Void	5in diameter x 3in deep	NEW	No
71-4	West quadrant 3ft below the collar	Void	14in H x 14in W x 2in D	INCREASE	No
72-1	SW quadrant 5ft 6in below footing	Voids/Honey-combing	Up to 18in diameter x 2in deep in an area 5ft H x 4ft W	INCREASE	No
74-2	South quadrant 7ft below footing	Void	12in H x 4in W x 2½in D	NEW	No
75-3	South and SW quadrant 2ft below footing	Void/Honey-combing	Intermittent up to 7ft H x 4in W x 2in D	INCREASE	No
75-4	North quadrant 32in below footing	Void/ Honey-combing	Intermittent up to 9in H x 4in W x 3in D -	NEW	No
84-1	NE quadrant 4ft below collar	Void	2in H x 9in W x 2in D		No
84-2	NE quadrant 5ft below collar	Void	2in H x 9in W x 2in D		No
84-3	East quadrant 10ft below footing	Void	8in H x 3in W x 3in D	NEW	No
84-4	NE quadrant 10ft below collar	2 Voids	Up to 14in H x 5in W x 4in D		No
86-2	NE quadrant 5ft below collar	Void	15in H x 4in W x 3in D		No
	NE quadrant 7ft below collar	Void	9in H x 7in W x 3in D extends into groundline		No
95-1	NE quadrant 12in below footing/seal	Void	32in H x 12in W x 2in D		No
95-2	NW quadrant just below footing/seal	Void	12in H x 20in W x 2in D		No
72-2	SE quadrant 4ft below footing	Void	2ft H x 18in W x 3in D	NEW	No

Routine Underwater Bridge Inspection Report
BOLT UNDERWATER SERVICES, INC.
for
VOLKERT, INC.

NBI Structure No. (8): **100585** Underwater Date (93): 12/08/14

Structure/Roadway Identification:		**Underwater Inspection Details:**	
District (2):	7	Special Crew Hours:	113.4
County (3):	Hillsborough	Max. Depth:	24ft.
Feature Intersected (6):	Tampa Bay	Type of Dive Insp.:	SCUBA
Facility Carried (7):	US-92 (SR-600)	Type of Boat Used:	21ft. & 23ft. Boats
		Water Type/Marine Growth:	Salt/Tidal – Barnacles/Oysters

Previous Inspection:

Lead Diver:	**C.B.I. No.:**	**Inspection Date:**
Hays, Stephen F.	00438	12/10/12

Inspection Personnel:

Field Personnel:	**Title**	**C.B.I. No.:**	**Duty:**	**Signature:**
Elborne, Paul R.	C.B.I. Diver-Inspector	00469/Lead	Dive/Tend	PRE
Young, Ryan C.	Diver-Inspector		Dive/Tend	
Payne, Timothy N.	Diver-Inspector		Dive/Tend	
Ivey, Jonathan J.	Diver-Inspector		Dive/Tend	
Davis, Chris S.	Diver		Dive/Tend	

PILES/COLUMNS
ELEMENT: 205 R/CONCRETE **372**: ea.

NOTE: There are no columns visible under Footing 2 due to rock rubble.

Condition State:	**QTY:**	**Recommended Feasible Action:**
CS-1	**354**	**Do Nothing**

The majority of the columns have steel casings in place. Some of the steel casings are covered with a grout coating up to 1in. thick. The exposed steel casings have minor to moderate corrosion and pitting.

Several of the exposed portions of the concrete columns have areas of voids/honeycombing with no exposed steel up to 12in. H x 12in. W x 1in. D.

There are several columns with no casings visible or only partial casings remaining. The exposed concrete is in good condition except as noted below. Many of the concrete columns have been gouged by the burning method used to remove the casings. The following is a list of columns with no visible casings: 4-4, 6-2, 14-1 *NEW*, 14-2 *NEW*, 14-3 *NEW*, 14-4, 26-1, 33-3, 33-4, 40-1, 40-2, 42-2, 42-4, 45-1, 45-2, 45-3, 45-4, 46-1, 46-2, 47-2, 47-4, 48-4, 50-1, 50-2, 50-4, 51-1, 52-1, 52-4, 53-2, 53-3, 53-4, 71-1, 71-4, 75-4, 76-2, 77-4, 78-2, 79-2, 85-2, 85-3, 87-1, 87-2, 90-1, 92-2, 93-3, 93-4, 95-2, and 95-4.

CS-2	**18**	**Do Nothing**

Bent/ Shaft	Location	Deficiency	Size
45-3	North quadrant just below footing/seal	Void	30in. H x 18in. W x 3in. D
46-1	SW at bottom of footing	Void	3ft. H x 16in. W x 1in. D
49-1	SW quadrant 3ft. 3in. below footing/seal	Void	33in. H x 14in. W x 3in. D (partially buried)
49-2	SW quadrant 3ft. below footing/seal	Void	3in. H x 6in. W x 3½in. D (appears to be a cut hole in casing)
71-1	South quadrant 12in. below jacket	Void	5in. diameter x 3in. deep - *NEW*
71-4	West quadrant 3ft. below the collar	Void	14in. H x 14in. W x 2in. D - *INCREASE*
72-1	SW quadrant 5ft. 6in. below footer	Voids/Honeycombing	Up to 18in. diameter x 2in. deep in an area 5ft. H x 4ft. W - *INCREASE*

PILES/COLUMNS CONTINUED

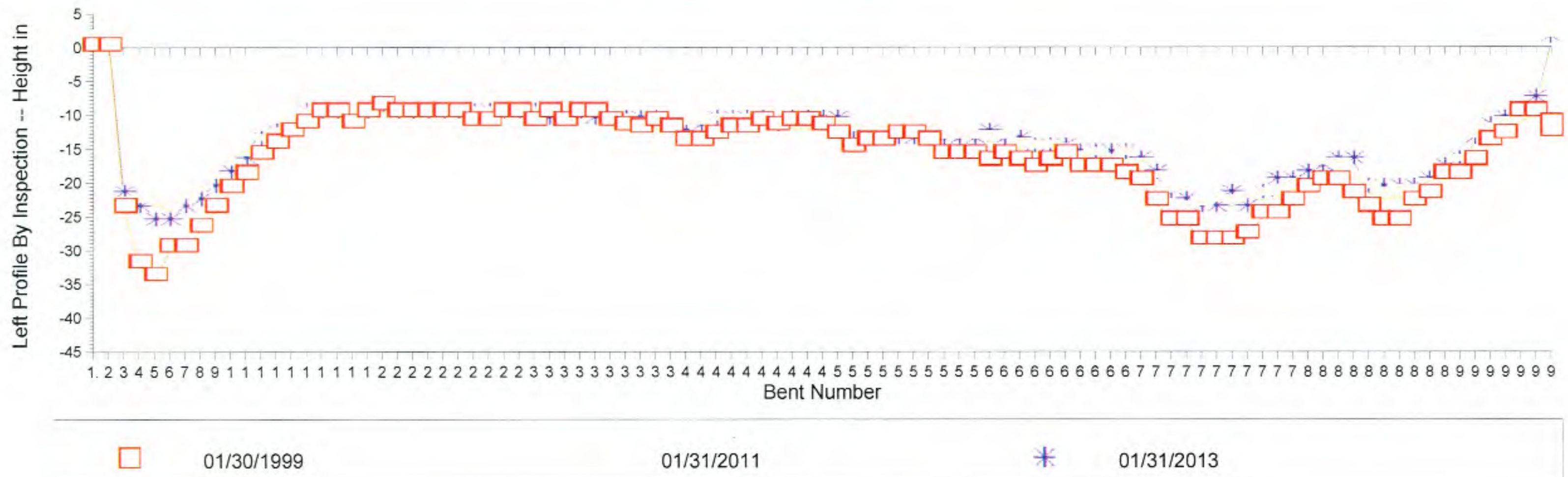
Bent/ Shaft	Location	Deficiency	Size
72-2	SE quadrant 4ft. below footer	Void	2ft. H x 18in. W x 3in. D - *NEW*
74-2	South quadrant 7ft. below footer	Void	12in. H x 4in. W x 2½in. D - *NEW*
75-3	South and SW quadrant 2ft. below footer	Void/Honeycombing	Intermittent up to 7ft. H x 4in. W x 2in. D - *INCREASE*
75-4	North quadrant 32in. below footer	Void/ Honeycombing	Intermittent up to 9in. H x 4in. W x 3in. D - *NEW*
84-1	NE quadrant 4ft. below collar	Void	2in. H x 9in. W x 2in. D
84-2	NE quadrant 5ft. below collar	Void	2in. H x 9in. W x 2in. D
84-3	East quadrant 10ft. below footer	Void	8in. H x 3in. W x 3in. D - *NEW*
84-4	NE quadrant 10ft. below collar	2 Voids	Up to 14in. H x 5in. W x 4in. D
86-2	NE quadrant 5ft. below collar	Void	15in. H x 4in. W x 3in. D
	NE quadrant 7ft. below collar	Void	9in. H x 7in. W x 3in. D extends into groundline
95-1	NE quadrant 12in. below footing/seal	Void	32in. H x 12in. W x 2in. D
95-2	NW quadrant just below footing/seal	Void	12in. H x 20in. W x 2in. D

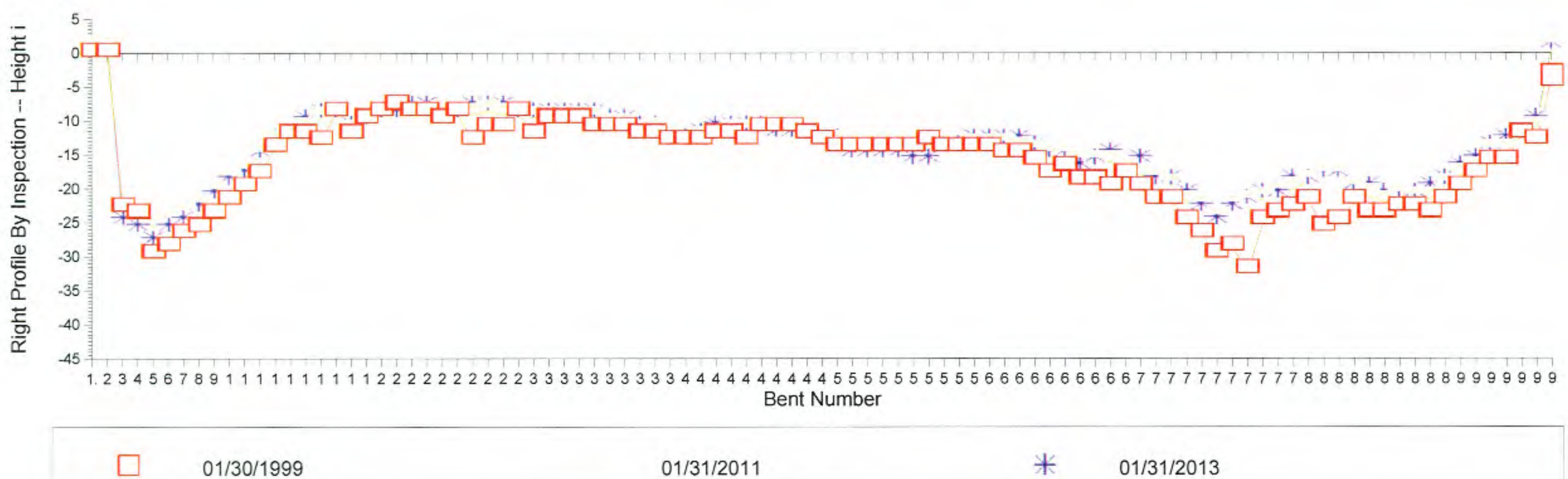
NOTE: Columns 14-1, 14-2, 14-3, 14-4, 85-3 and deficiencies were cleaned.

JACKETS
ELEMENT: 298 PILE JACKET BARE **7**: ea.

NOTE: The following shafts have larger casings (approximately 8ft. diameter) extending down from the footing/seal 5ft. to 8ft. These larger casings are possibly jackets installed during construction, they are as follows: 71-1, 71-2, 71-3, 72-4, 76-4, 85-3 and 87-3.

Condition State:	QTY:	Recommended Feasible Action:
CS-1	7	Do Nothing


Left Profile By Inspection -- Height in
5
0
-5
-10
-15
-20
-25
-30
-35
-40
-45
Bent Number
01/30/1999
01/31/2011
01/31/2013


Right Profile By Inspection -- Height i
5
0
-5
-10
-15
-20
-25
-30
-35
-40
-45
Bent Number
01/30/1999
01/31/2011
01/31/2013

Profile Data - Numerical Summary

Inspection Date and Key: 01/31/2013 GLWK

(All Heights Are In Feet)

Bent #	Left Height	Right Height
1.5	-1	-1
2	-1	-1
3	22	25
4	24	26
5	26	28
6	26	26
7	24	25
8	23	23
9	21	21
10	19	19
11	17	18
12	14	15
13	13	13
14	12	12
15	10	10
16	10	9
17	10	9
18	11	11
19	10	10
20	9	9
21	10	9
22	10	8
23	10	8
24	10	9
25	10	9
26	10	8
27	10	8
28	10	8
29	10	9
30	10	9
31	11	9
32	10	9
33	10	9
34	11	9

Profile Data - Numerical Summary

Inspection Date and Key: 01/31/2013 GLWK

(All Heights Are In Feet)

Bent #	Left Height	Right Height
35	10	10
36	11	10
37	11	11
38	11	11
39	12	12
40	13	13
41	12	12
42	11	11
43	11	11
44	11	11
45	11	11
46	12	12
47	11	12
48	11	12
49	11	13
50	11	13
51	13	15
52	13	15
53	13	15
54	14	15
55	14	16
56	13	16
57	14	14
58	14	14
59	14	13
60	13	13
61	14	13
62	14	13
63	15	14
64	15	15
65	15	15
66	16	17
67	16	16
68	16	15

Profile Data - Numerical Summary

Inspection Date and Key: 01/31/2013 GLWK

(All Heights Are In Feet)

Bent #	Left Height	Right Height
69	16	18
70	17	16
71	19	19
72	22	19
73	23	21
74	24	23
74.5	24	25
75	22	23
76	24	22
77	22	21
78	20	21
79	20	19
80	19	19
81	18	18
82	17	18
83	17	20
84	21	20
85	21	21
86	21	21
87	21	21
88	20	20
89	18	19
90	17	17
91	14	16
92	12	14
93	11	13
94	10	12
95	8	10

Profile Data - Numerical Summary

Inspection Date and Key: 01/31/2013 GLWK

(All Heights Are In Feet)

Bent #	Left Height	Right Height
95.5	-1	-1

Air Temp: 1
Profile Notes:
Measurements referenced from the top of Footer 3.
Waterline = 4.0ft.
Measurements showing a -1 indicates that location was dry during this inspection.

Inspection Date and Key: 01/31/2011 ZLOL

Bent #	Left Height	Right Height
1.5	-1	-1
2	-1	-1
3	23.7	23.6
4	23.5	24.1
5	23	25.6
6	24	24.6
7	23	23.6
8	21.5	22.6
9	20	20.6
10	18	18.6
11	16.5	17.6
12	14.5	15.1
13	13.5	13.1
14	12	12.6
15	10.5	11.1
16	10	9.6
17	10	9.1
18	10.5	11.6
19	10	9.6
20	9.5	8.6
21	10	8.6
22	10	9.1
23	10	9.1
24	10	9.1
25	10	9.1

Profile Data - Numerical Summary

Inspection Date and Key: 01/31/2011 ZLOL

(All Heights Are In Feet)

Bent #	Left Height	Right Height
26	10.5	9.1
27	10.5	8.6
28	10	9.1
29	10	9.1
30	10.5	9.6
31	10.5	9.6
32	10	9.6
33	10	9.6
34	10	9.6
35	10	10.6
36	11.5	10.6
37	12	11.6
38	12	11.1
39	12	12.1
40	12	13.1
41	12	12.6
42	11.5	12.1
43	11.5	11.6
44	11.5	11.6
45	11.5	11.1
46	11.5	10.1
47	12	10.1
48	12	10.6
49	12	12.6
50	12	13.1
51	13	13.6
52	13	13.6
53	13	13.1
54	13	13.6
55	14	13.6
56	13	13.6
57	14	13.6
58	14	14.1
59	14	13.6

Profile Data - Numerical Summary

Inspection Date and Key: 01/31/2011 ZLOL

Bent #	Left Height	Right Height	(All Heights Are In Feet)
60	14	13.6	
61	14	13.6	
62	15	14.1	
63	15.5	14.6	
64	15.5	15.1	
65	16	15.1	
66	17	16.1	
67	16.5	16.1	
68	17.5	16.1	
69	16.5	17.1	
70	18	18.1	
71	20	18.6	
72	22	19.6	
73	24.5	22.1	
74	24	22.6	
74.5	23.5	24.6	
75	23.5	22.6	
76	23	22.1	
77	22.5	21.6	
78	22	20.6	
79	21	20.1	
80	21	19.1	
81	18	18.1	
82	18	18.1	
83	20.5	20.1	
84	21.5	21.1	
85	22.5	22.1	
86	22	21.1	
87	22	21.6	
88	21	21.6	
89	19.5	19.6	
90	17.5	18.1	
91	14	17.1	
92	13	14.6	

Profile Data - Numerical Summary

Inspection Date and Key: 01/31/2011 ZLOL

Bent #	Left Height	Right Height
93	12	14.6
94	11	12.1
95	10	11.1
95.5	-1	-1

(All Heights Are In Feet)

Air Temp:

Profile Notes:

Measurements referenced from the top of Footer 3.
Waterline = 4.0ft.
Measurements showing a -1 indicates that location was dry during this inspection.

Inspection Date and Key: 01/30/1999 YMYR

Bent #	Left Height	Right Height
1.5	0.03	0.03
2	0.03	0.03
3	23.95	22.97
4	32.15	23.95
5	34.12	29.86
6	29.86	28.87
7	29.86	26.9
8	26.9	25.92
9	23.95	23.95
10	21	21.98
11	19.03	20.01
12	16.08	18.04
13	14.44	14.11
14	12.8	12.14
15	11.48	12.14
16	9.84	13.12
17	9.84	8.86
18	11.48	12.14
19	9.84	9.84
20	8.86	8.86
21	9.84	7.87
22	9.84	8.86

Profile Data - Numerical Summary

Inspection Date and Key: 01/30/1999 YMYR

(All Heights Are In Feet)

Bent #	Left Height	Right Height
23	9.84	8.86
24	9.84	9.84
25	9.84	8.86
26	11.15	13.12
27	11.15	11.15
28	9.84	11.15
29	9.84	8.86
30	11.15	12.14
31	9.84	9.84
32	11.15	9.84
33	9.84	9.84
34	9.84	11.15
35	11.15	11.15
36	11.81	11.15
37	12.14	12.14
38	11.15	12.14
39	12.14	13.12
40	14.11	13.12
41	14.11	13.12
42	13.12	12.14
43	12.14	12.14
44	12.14	13.12
45	11.15	11.15
46	11.81	11.15
47	11.15	11.15
48	11.15	12.14
49	11.81	13.12
50	13.12	14.11
51	15.09	14.11
52	14.11	14.11
53	14.11	14.11
54	13.12	14.11
55	13.12	14.11
56	14.11	13.12

Profile Data - Numerical Summary

Inspection Date and Key: 01/30/1999 YMYR

(All Heights Are In Feet)

Bent #	Left Height	Right Height
57	16.08	14.11
58	16.08	14.11
59	16.08	14.11
60	17.06	14.11
61	16.08	15.09
62	17.06	15.09
63	18.04	16.08
64	17.06	18.04
65	16.08	17.06
66	18.04	19.03
67	18.04	19.03
68	18.04	20.01
69	19.03	18.04
70	20.01	20.01
71	22.97	21.98
72	25.92	21.98
73	25.92	24.93
74	28.87	26.9
74.5	28.87	29.86
75	28.87	28.87
76	27.89	32.15
77	24.93	24.93
78	24.93	23.95
79	22.97	22.97
80	21	21.98
81	20.01	25.92
82	20.01	24.93
83	21.98	21.98
84	23.95	23.95
85	25.92	23.95
86	25.92	22.97
87	22.97	22.97
88	21.98	23.95
89	19.03	21.98

Profile Data - Numerical Summary

Inspection Date and Key: 01/30/1999 YMYR

(All Heights Are In Feet)

Bent #	Left Height	Right Height
90	19.03	20.01
91	17.06	18.04
92	14.11	16.08
93	13.12	16.08
94	9.84	12.14
95	9.84	13.12
95.5	11.48	3.28

Air Temp:

Profile Notes:

Soundings referenced from top of FOOTINGS.
The soundings at spans 1.5 and 95.5 refer to the toe of bulkheads.
The original soundings are from 11/1996

Routine Underwater Bridge Inspection Report
BOLT UNDERWATER SERVICES, INC.
for
KISINGER CAMPO & ASSOCIATES, CORP.

NBI Structure ID. (8): **170176** Underwater Date (93): 12/01/15

Structure/Roadway Identification:		**Underwater Inspection Details:**	
District (2):	01	Special Crew Hours:	33.1
County (3):	Sarasota	Max. Depth:	13ft. at Pier 6
Feature Intersected (6):	Intracoastal Waterway	Type of Dive Insp.:	Level II (SCUBA)
Facility Carried (7):	SR-789 / John Ringling Cswy.	Type of Boat Used:	23ft. Skiff
		Water Type/Marine Growth:	Salt/Tidal – Barnacles/Oysters

Previous Inspection:

Lead Diver:	**C.B.I. No.:**	**Inspection Date:**
Hitch, Victoria G.	00414	12/13/13

Inspection Personnel:

Field Personnel:	**Title**	**C.B.I. No.:**	**Duty:**	**Signature:**
Hoogland, Keith S.	C.B.I. Diver-Inspector	00341/Lead	Dive	KSH
Salazar, Pete Jr.	Diver-Inspector		Dive	
Jensen, Denise R.	Diver-Inspector		Tend	
Ulrich, Kenneth	FDOT/ QA			

PILES/COLUMNS
ELEMENT: 205 R/CONCRETE **14**: ea.

NOTE: This element represents the visible columns under Piers 4 through 10.

Condition State:	**QTY:**	**Recommended Feasible Action:**
CS-1	**14**	**Do Nothing**

The columns have steel casings that extend from the pier wall into the groundline and have several areas of light corrosion with no pitting.

Cleaning Log: No cleaning due to steel casings in-place.

PIER WALLS
ELEMENT: 210 R/CONCRETE **312** lf.

NOTE: The pier walls extend approximately 4ft. into the marine growth and appear to have a precast slab on the bottom.

Condition State:	**QTY:**	**Recommended Feasible Action:**
CS-1	**312**	**Do Nothing**

There are vertical cracks in several locations, up to 5ft. L x 1/32in. W, extending up to 12in. into the marine growth.

Cleaning Log: Strips on Pier 7 were cleaned.

This report contains information relating to the physical security of a structure and depictions of the structure. This information is confidential and exempt from public inspection pursuant to sections 119.071(3)(a) and 119.071(3)(b), Florida Statutes.

CHANNEL
ELEMENT: 290 1: ea.

NOTE: Divers inspected the west and east seawalls (this includes both steel and concrete sheet piles) under the structure to the first bend or 30ft. out from the structure.

Condition State:	QTY:	Recommended Feasible Action:
CS-1	**1**	**Do Nothing**

There is scattered rock rubble along both seawalls.

The coating on the steel sheet piles have *12in.* diameter areas of light corrosion and *minor blisters* in the splash zone – INCREASE.

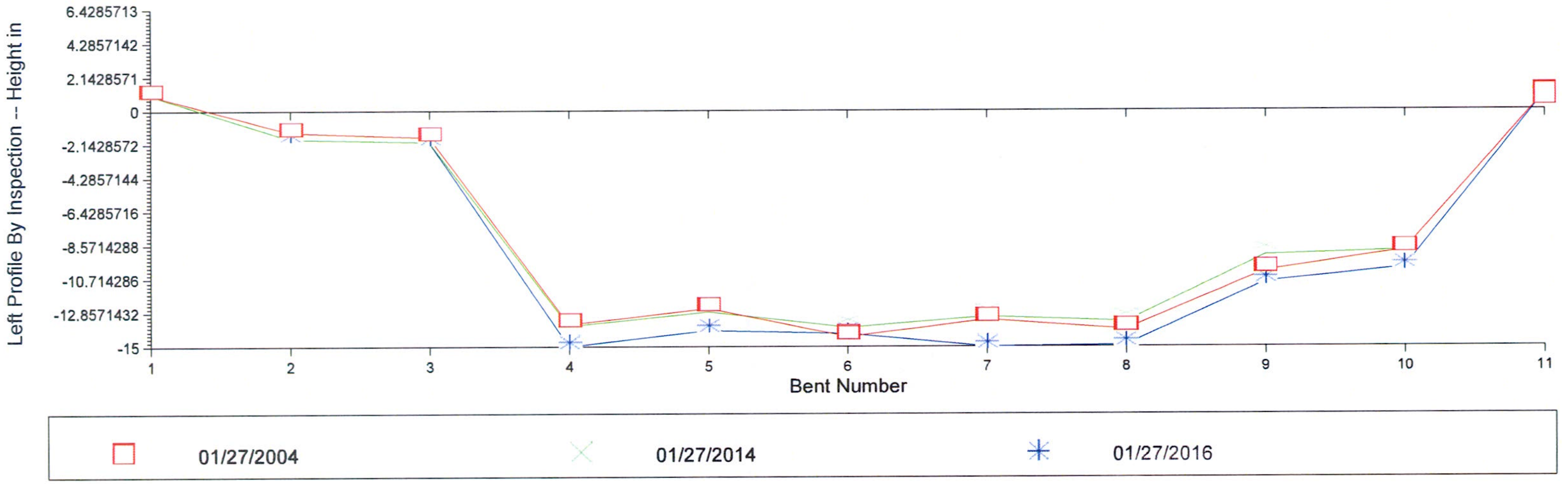
East seawall, 6ft. north of the second cap joint to the south of the structure, 5in. below cap (in concrete sheet pile), spall, 18in. H x 8in. W x 5in. D.

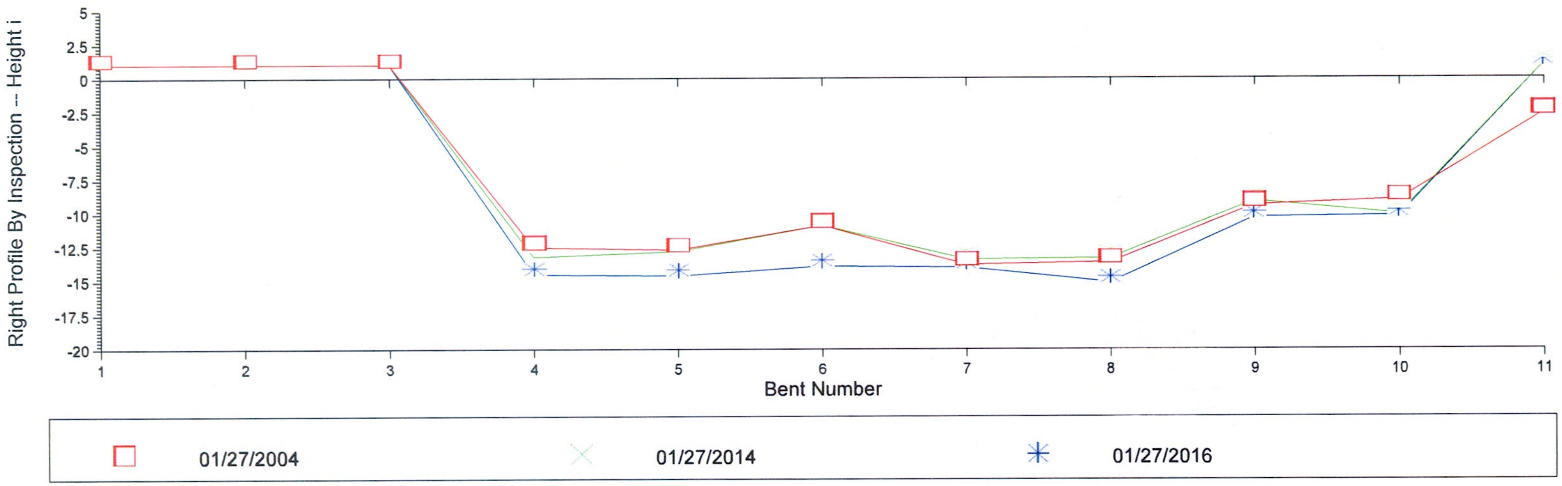
INSPECTION NOTES: Divers inspected Bents 4 through 10 each with two drilled shafts, Pier Walls 4 through 10 and Channel.
STRUCTURE NOTES: Structure inventoried west to east.

REPORT ID: INVT016

Structure #: 170176

DATE PRINTED: 03/02/2016





REPORT ID: INVT016

Structure #: 170176

DATE PRINTED: 03/02/2016

Profile Data - Numerical Summary

Inspection Date and Key: 01/27/2016 OHKU

(All Heights Are In Feet)

Bent #	Left Height	Right Height
1	-1	-1
2	1.8	-1
3	2	-1
4	15	14.5
5	14	14.6
6	14.2	13.9
7	15	14
8	14.9	15.1
9	10.9	10.3
10	10	10.2
11	-1	-1

Air Temp: 1

Profile Notes:

Measurementa are referenced from the top of the marine growth on the columns.
Waterline to top of marine growth: 1.2ft.

Note: -1 indicates area out of water.

Inspection Date and Key: 01/27/2014 RGWV

Bent #	Left Height	Right Height
1	-1	-1
2	1.8	-1
3	2	-1
4	13.7	13.2
5	12.8	12.8
6	13.8	10.9
7	13.1	13.4
8	13.4	13.3
9	9.2	9.1
10	8.9	10.1

Profile Data - Numerical Summary

(All Heights Are In Feet)

Inspection Date and Key: 01/27/2014 RGWV

Bent #	Left Height	Right Height
11	-1	-1

Air Temp: 1
Profile Notes:
Measurementa are referenced from the top of the marine growth on the columns.
Waterline to top of marine growth: 1.2ft.

NOTE: -1 indicates area out of water.

Inspection Date and Key: 01/27/2004 QPJC

Bent #	Left Height	Right Height
1	-1	-1
2	1.4	-1
3	1.7	-1
4	13.6	12.5
5	12.6	12.7
6	14.4	10.9
7	13.3	13.8
8	13.9	13.6
9	10.2	9.4
10	8.9	9
11	-1	2.6

Air Temp:
Profile Notes:
Measurements recorded with fathometer and referenced from top of marine growth.

Elements

Inspection Date: 4/25/2016FCRX

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	12/3	Bare Concrete Deck	0	.	17120	100.	0	.	0	.	0	.	17120 sf.

Notes There is minor amounts of dirt and debris reappearing along both barriers, but does not pose a problem at this time.

There are insignificant size map pattern type cracks in the deck top throughout most spans. The deck top surface exhibits insignificant size transverse cracks located near/over Piers 2, 3 and 4, adjacent to the control joints. Some diagonal and longitudinal cracks propagate from the transverse cracks over the piers.

The deck underside between the girders is not visible for inspection due to steel stay-in-place forms. There are insignificant size transverse cracks (some with efflorescence) in the deck underside of the overhangs, over the intermediate piers.

Previously requested repair (remove dirt and debris from shoulder areas along both bridge barriers) was effectively completed.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	301/3	Pourable Joint Seal	81	88.04	11	11.96	0	.	0	.	0	.	92 lf.

Notes There are minor surface spalls at the lip of the joint, random locations over both abutments. The joints exhibit dirt and debris, mostly in the outside emergency lanes.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	331/3	Conc Bridge Railing	738	100.	0	.	0	.	0	.	0	.	738 lf.

Notes There are random insignificant size vertical cracks in the barrier walls exhibiting efflorescence, typically located midway between the control joints.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	109/3	P/S Conc Open Girder	2213	100.	0	.	0	.	0	.	0	.	2213 lf.

Notes The grouted joint between the poured beam ends of the fascia beams exhibit minor spalling due to independent movement of the beams. Also, several of the poured beam ends have minor cracking in the lower portion adjacent to the bottom flange.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	310/3	Elastomeric Bearing	48	100.	0	.	0	.	0	.	0	.	48 ea.

Notes

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	205/3	R/Conc Column	6	100.	0	.	0	.	0	.	0	.	6 ea.

Notes Diver's were required to inspect columns of Piers 3 and 4 during this routine inspection.

There are minor honeycomb areas in Columns 3-1 and 4-1, up to 12 in. x 4 in. x 3/4 in. deep. These areas are at or near the groundline. Poses no problem.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	215/3	R/Conc Abutment	97	98.98	1	1.02	0	.	0	.	0	.	98 lf.

Notes

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	234/3	R/Conc Cap	146	99.32	1	.68	0	.	0	.	0	.	147 lf.

Notes The caps of Piers 2, 3 and 4 have insignificant size vertical cracks which extend down from the top of the cap. These cracks are typically located along both sides of the beam seats of Girders 2 and 5 over the drilled shafts.

There is a water level gauge attached to the outside barrier face and Pier Cap 3 on left side of structure. The attaching hardware type could not be determined at this time.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	396/3	Other Abut Slope Pro	23229	99.91	21	.	0	.	0	.	0	.	23250 sf.

Notes The slopes at both abutments are protected by loose rubble rip-rap. There are several random areas adjacent to the abutment caps that have missing rubble exposing minor undermined areas along the caps, with the worst area along Abutment 5. Poses no problem at this time.

Elements

Inspection Date: 4/25/2016FCRX

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	290/3	Channel	1	100.	0	.	0	.	0	.	0	.	1 ea.

Notes There is up to a 4 ft. wide x 18 in. deep eroded area around the base of Column 2-2. Posing no problem at this time.

See Bridge Profile Comparison Report attached.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	321/3	R/Conc Approach Slab	2	100.	0	.	0	.	0	.	0	.	2 ea.

Notes The approach roadway asphalt is up to 3/4 in. lower than the approach slabs. This condition is causing a slight live load impact to the structure. See Photo 1 attached for typical view. Poses no problem at this time.

There are numerous insignificant size map pattern type cracks in the South approach slab. The North slab exhibits similar cracks, but to a lessor degree.

Previously requested repairs (remove dirt and vegetation from joints along base of Southwest, Southeast, Northwest and Northeast wingwalls and seal) was effectively completed.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	475/3	R/Conc Walls	53	100.	0	.	0	.	0	.	0	.	53 lf.

Notes

Total Number of Elements: 12

Inspection Information

Inspection Date: 04.25.2016 **Type:** Regular NBI

Inspector: MT238JD-P - James Durbin

Inspection Notes: Sufficiency Rating Calculation Accepted by mt238cw-P at 2016-04-27 11:54:02
Divers were required to inspect Piers 3 and 4 at the time of this inspection.

Structure Notes

1) On 9/22/03, load rating updated to reflect posting value change to 99 tons. (Rick Vallier)
2) A new Photo Inventory will be required April, 2018, unless upgrades/reconstruction alter structure or approach roadway prior to 2018.
3) Divers are sometimes required, due to high water levels, Element 205, Piers 3 and 4.
4) Utility attached to this structure, see Element 234 for details.

Bridge No: 260112

County Section No: 26020000

State Road No: 25

U.S. Road No: 441/41

Location: SR25 - US441 / Sante Fe River

Inspection Date: 04/25/2016

Inspector: J. Durbin

PROFILE COMPARISON REPORT

Date Groundline Recorded 12/19/2001						
Bent	Action		Critical		Lead Line Measurement	
No.	Left	Right	Left	Right	Left	Right
1					10.81	5.89
2					20.19	20.85
3					32.68	29.19
4					24.69	25.63
5					9.06	9.07

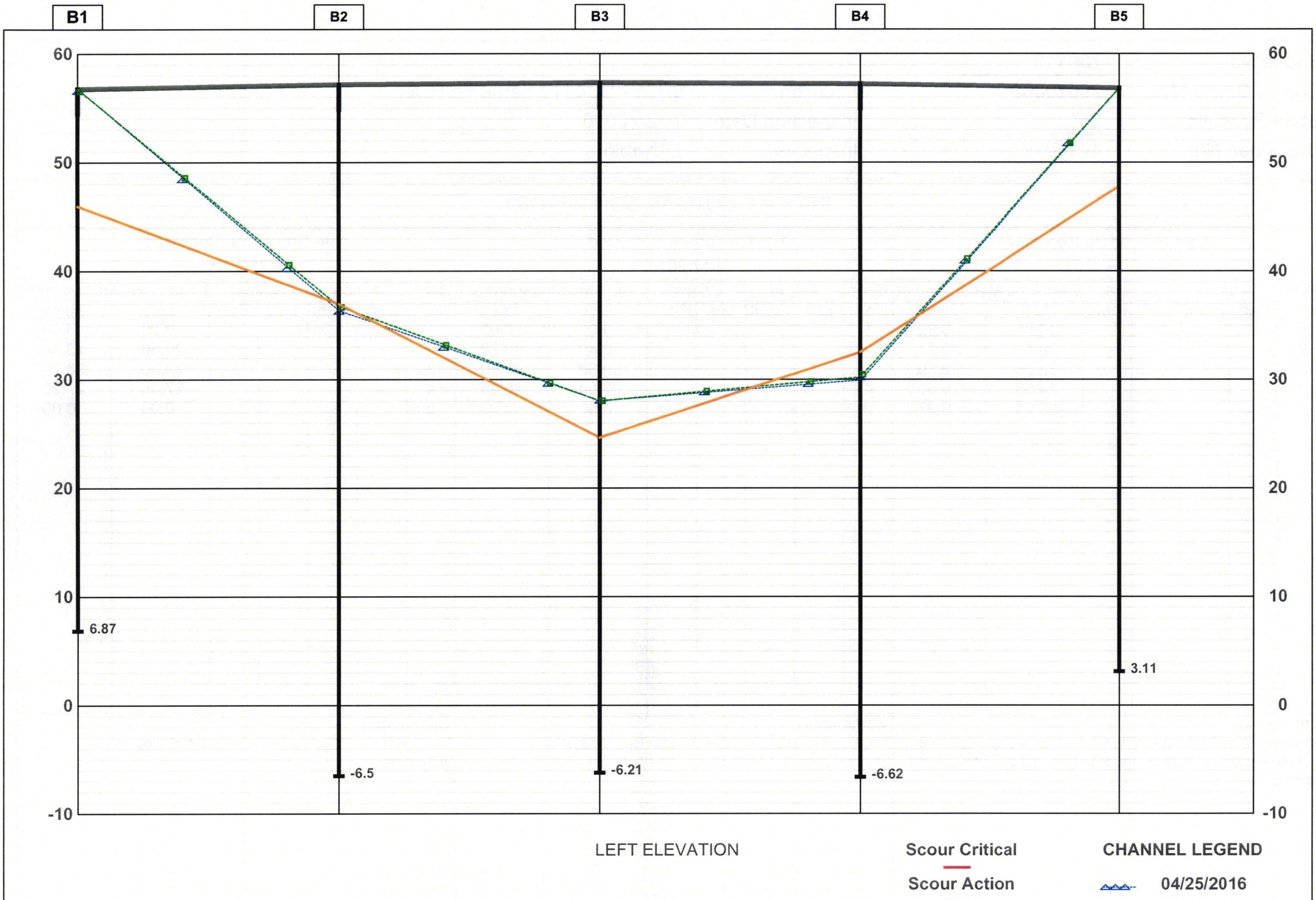
Date Groundline Recorded 04/17/2014						
Bent	Action		Critical		Lead Line Measurement	
No.	Left	Right	Left	Right	Left	Right
1					0.00	0.00
2					20.40	20.30
3					29.30	29.10
4					27.00	28.40
5					0.00	0.00

Date Groundline Recorded 04/25/2016						
Bent	Action		Critical		Lead Line Measurement	
No.	Left	Right	Left	Right	Left	Right
1					0.00	0.00
2					20.80	20.40
3					29.30	28.70
4					27.30	27.20
5					0.00	0.00

Comments:
All original calculations taken from the 2001 construction plans. All measurements taken from the top of bridge rail to groundline. KMB 05/2002

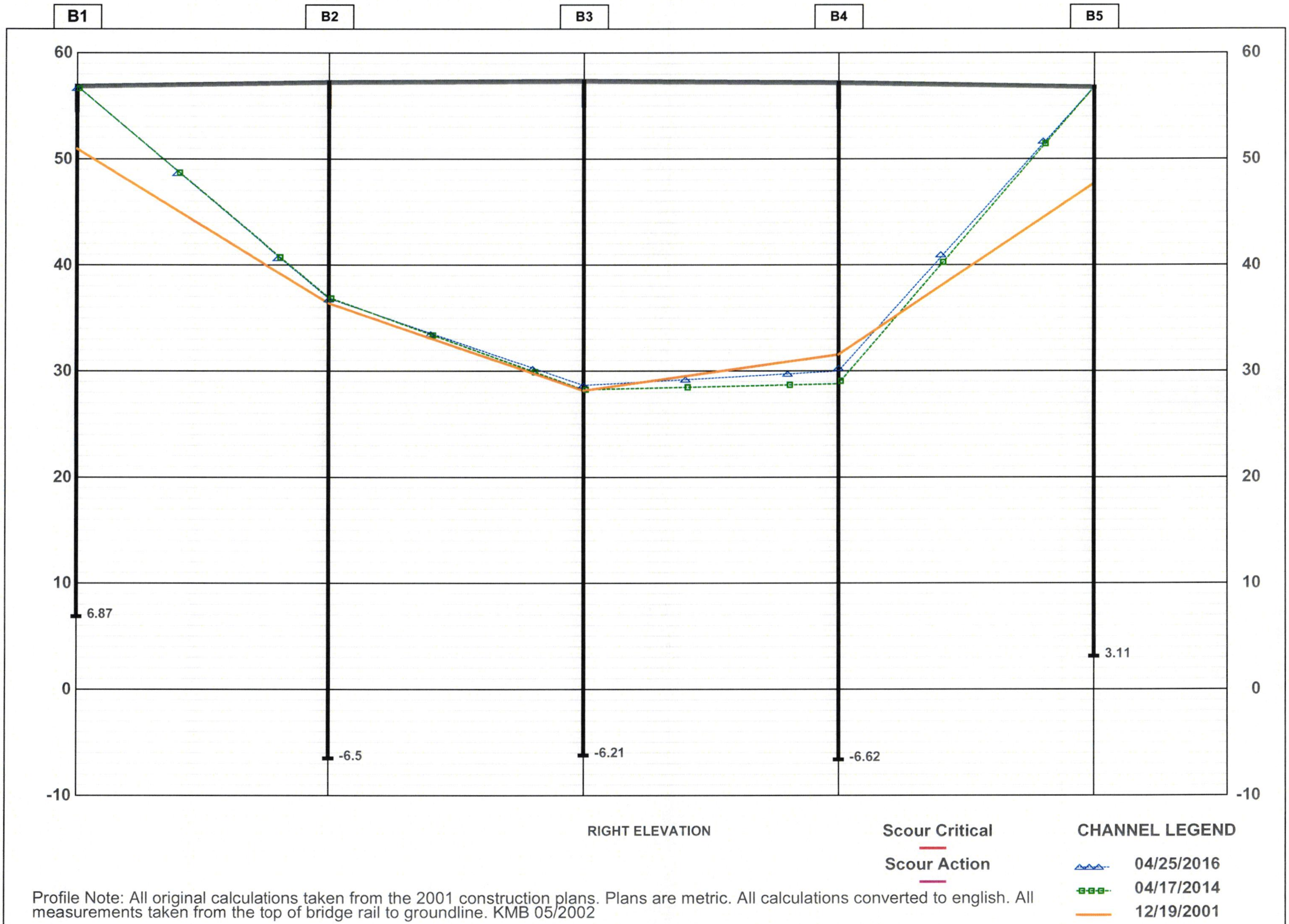
Comments:
Leadline measurements taken from the top of bridge rail to groundline.

Comments:
Leadline measurements taken from the top of bridge rail to groundline.



Profile Note: All original calculations taken from the 2001 construction plans. Plans are metric. All calculations converted to english. All measurements taken from the top of bridge rail to groundline. KMB 05/2002

Scale NTS	BRIDGE NO. 260112 SR25 - US441 / SANTE FE RIVER	



Profile Note: All original calculations taken from the 2001 construction plans. Plans are metric. All calculations converted to english. All measurements taken from the top of bridge rail to groundline. KMB 05/2002

Scale NTS	BRIDGE NO. 260112 SR25 - US441 / SANTE FE RIVER	Page 3 of 3

FLORIDA DEPARTMENT OF TRANSPORTATION
BRIDGE MANAGEMENT SYSTEM
Inspection/CIDR/Bridge Profile Report
Inspection

Structure ID: 520145

DISTRICT: D3 - Chipley **INSPECTION DATE: 1/19/2017 XPPU**

SUBSTRUCTURE : Substructure

Str Unit	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	T Qty
UNIT 0	205 / 2	Re Conc Column	37	64.91	20	35.09	0	.	0	.	57 each
UNIT 0	1130 / 2	Cracking (RC and Other)	0	.	12	100	0	.	0	.	12 each
UNIT 0	6000 / 2	Scour	0	.	8	100	0	.	0	.	8 each

Element Inspection Notes:

205 / 2 Condition State 2
NEW:
1) The left and right columns at bent 15 exhibits minor scaling up to the high water mark.
2) The underwater report noted up to 3.0 ft. of local scour at bents 12, 13, 14 & 15.

NO CHANGE:
1) The bottom of several pier columns exhibit vertical cracks up to 0.025 inch wide X 1.5 ft. long with efflorescence, in the drilled shaft casing.
2) The left and right columns at bents 12, 13 and 14 exhibit minor scaling up to the high water mark.

1130 / 2 -

6000 / 2 _

SUBSTRUCTURE : Substructure

Str Unit	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	T Qty
UNIT 0	215 / 2	Re Conc Abutment	91.86	100	0	.	0	.	0	.	91.86 ft

Element Inspection Notes:

215 / 2 <none>

SUBSTRUCTURE : Substructure

Str Unit	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	T Qty
UNIT 0	234 / 2	Re Conc Pier Cap	523.72	68.22	244	31.78	0	.	0	.	767.72 ft
UNIT 0	1080 / 2	Delamination/Spall/Patched Area	0	.	4	100	0	.	0	.	4 ft
UNIT 0	1130 / 2	Cracking (RC and Other)	0	.	240	100	0	.	0	.	240 ft

Element Inspection Notes:

234 / 2 Condition State 2
NO CHANGE:
1) Several intermediate bent caps exhibit random cracks up to 0.025 inch wide in the near and far faces.
2) There is evidence of bat infestation over bent caps 5, 9, 13, and 17.
3) The near face of intermediate cap 5 exhibits a diagonal crack up to 0.040 inch wide X 3.0 ft. long.
4) The far face of intermediate cap 15 between girders 2 and 3 exhibits a spall, 4.0 ft. long X 1.2 ft. high X 0.6 ft. deep, with no exposed steel.
5) There are diagonal cracks up to 0.040 inch wide X 5.0 ft. long, in the near right and far right faces of bent cap 19.

1080 / 2 -

1130 / 2 -

SUBSTRUCTURE : Substructure

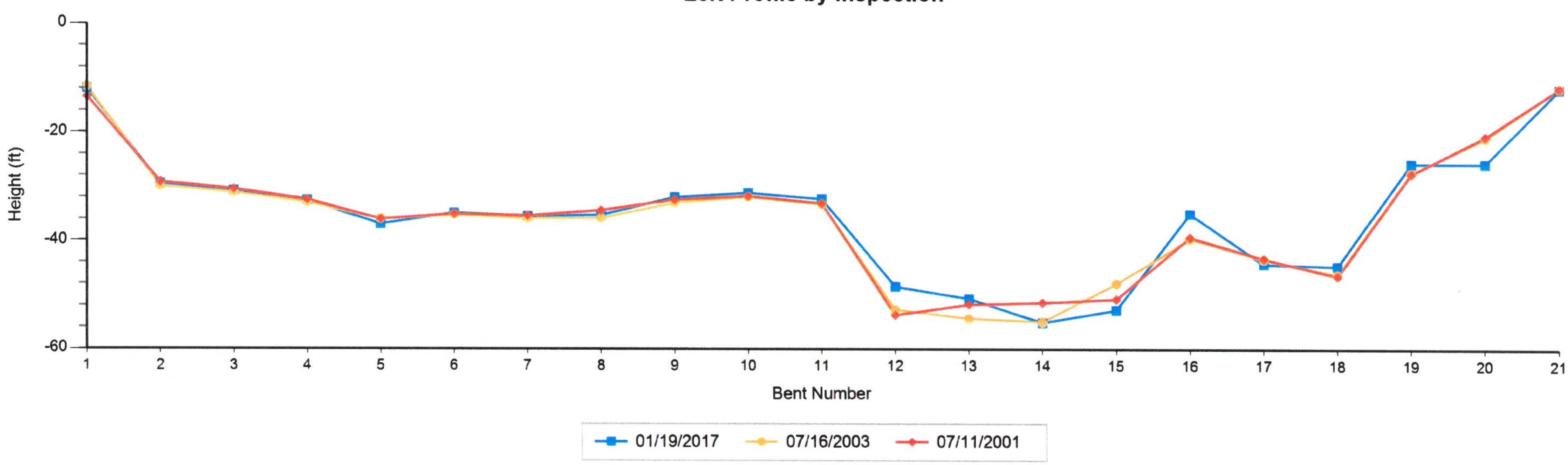
FLORIDA DEPARTMENT OF TRANSPORTATION BRIDGE MANAGEMENT SYSTEM

Inspection/CIDR/Bridge Profile Report

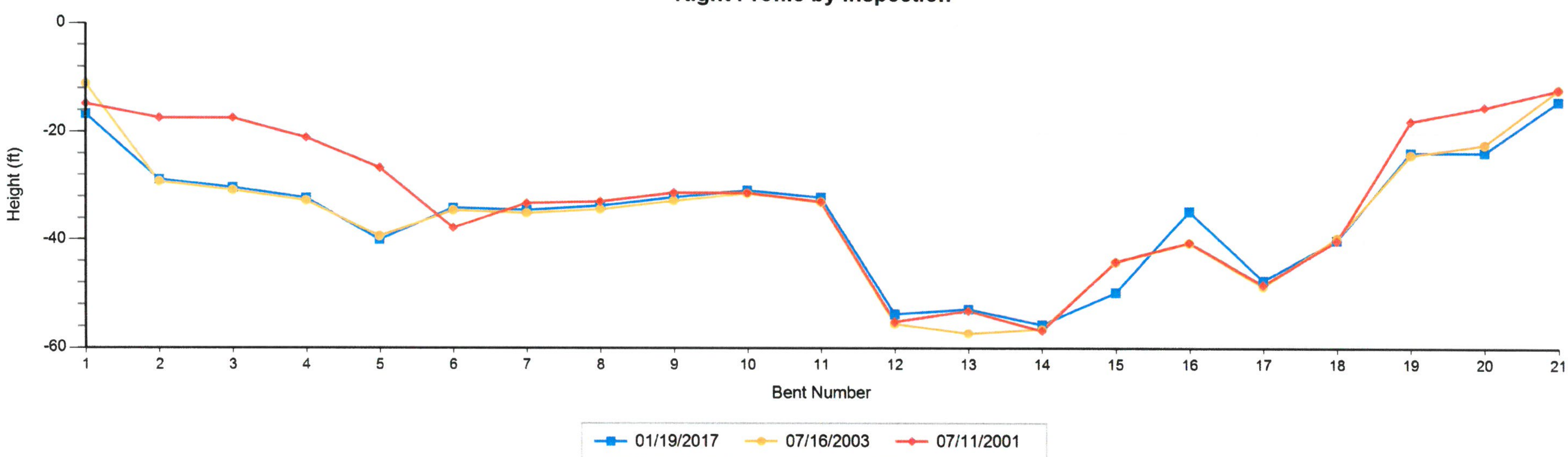
Bridge Profile

DATE PRINTED: 2/1/2017 10:22:33 AM

Left Profile by Inspection



Right Profile by Inspection



Profile Data - Numerical Summary

Inspection Date and Key: 1/19/2017 XPPU

(All Heights are in Feet)

Bent #	Left Height	Right Height
1	12.00	16.70
2	29.50	28.80
3	30.80	30.30
4	32.60	32.20
5	37.00	40.00
6	34.90	34.00
7	35.50	34.40
8	35.30	33.60
9	32.00	32.00
10	31.20	30.70
11	32.30	32.00
12	48.50	53.70
13	50.70	52.80
14	55.10	55.60
15	52.80	49.70
16	35.00	34.50
17	44.30	47.40
18	44.70	39.90
19	25.70	23.50
20	25.70	23.50
21	12.00	14.00

Air Temp: 70

Profile Notes:

GROUND LINE MEASUREMENTS FROM TOP OF BARRIER WALL.
WATER LINE MEASUREMENTS FROM TOP OF BARRIER WALL AT BENT 13.
LEFT SIDE - 46.1 FT. RIGHT SIDE - 46.0 FT.

FLORIDA DEPARTMENT OF TRANSPORTATION BRIDGE MANAGEMENT SYSTEM
Inspection/CIDR/Bridge Profile Report
Bridge Profile

Profile Data - Numerical Summary

(All Heights are in Feet)

Inspection Date and Key: 7/16/2003 UWIL

Bent #	Left Height	Right Height
1	11.60	11.10
2	30.00	29.20
3	31.20	30.80
4	33.00	32.70
5	36.00	39.30
6	35.30	34.50
7	36.00	35.00
8	35.90	34.30
9	33.10	32.70
10	32.00	31.30
11	33.30	32.90
12	52.80	55.50
13	54.40	57.30
14	55.00	56.50
15	48.00	44.10
16	39.70	40.50
17	43.40	48.60
18	46.30	39.50
19	27.40	24.00
20	21.00	22.00
21	11.80	11.80

FLORIDA DEPARTMENT OF TRANSPORTATION BRIDGE MANAGEMENT SYSTEM

Inspection/CIDR/Bridge Profile Report

Bridge Profile

DATE PRINTED: 2/1/2017 10:22:33 AM

Profile Data - Numerical Summary

(All Heights are in Feet)

Air Temp: 88

Profile Notes:

Water line measurements from top of barrier wall at bent no. 13.
44.0 Left 44.0 Right
Ground line measurements from top of barrier wall.

Inspection Date and Key: 7/11/2001 GEUL

Bent #	Left Height	Right Height
1	13.45	14.76
2	29.20	17.39
3	30.51	17.39
4	32.48	21.00
5	36.09	26.57
6	35.10	37.73
7	35.43	33.14
8	34.45	32.81
9	32.48	31.17
10	31.82	31.17
11	33.14	32.81
12	53.81	55.12
13	51.84	53.15
14	51.51	56.76
15	50.85	43.96
16	39.37	40.35
17	43.31	48.23
18	46.59	40.03
19	27.56	17.72
20	20.67	15.09

FLORIDA DEPARTMENT OF TRANSPORTATION BRIDGE MANAGEMENT SYSTEM
Inspection/CIDR/Bridge Profile Report
Bridge Profile

DATE PRINTED: 2/1/2017 10:22:33 AM

Profile Data - Numerical Summary

Bent #	Left Height	Right Height	(All Heights are in Feet)
21	11.81	11.81	

Air Temp:

Profile Notes:

WATER LINE MEASUREMENTS FROM TOP OF BARRIER WALL AT BENT NO. 13
15.4 LEFT 15.5 RIGHT.
GROUND LINE MEASUREMENTS FROM TOP OF BARRIER WALL.

FLORIDA DEPARTMENT OF TRANSPORTATION
BRIDGE MANAGEMENT SYSTEM
Inspection/CID/Bridge Profile Report

Inspector Recommendations

UNIT: UNIT 0 DECKS

ELEMENT/ENV: 304/3 Open Expansion Joint ELEM CATEGORY: Joints

CONDITION STATE (3)		Priority
2	**34 lf.**	3

WORK ORDER RECOMMENDATION:

Epoxy inject the loose sounding armor over bents 16 and 21. Area marked in wht. paint.

Structure Notes

This structure replaced bridge number 610003.

INSPECTION NOTES: KMZQ 4/11/2016

Sufficiency Rating Calculation Accepted by mt338gb-P at 2016-05-02 20:02:06

Previous comments > Sufficiency Rating Calculation Accepted by mt338gm-P at 2014-05-14 08:36:17
Sufficiency Rating Calculation Accepted by mt338dr-P at 2012-05-24 15:13:17
Previous comments > Sufficiency Rating Calculation Accepted by mt338wd-P at 2010-06-29 13:55:13
MT338WD inspection comments - The Divers reported there were no significant underwater deficiencies noted on this inspection.
Sufficiency Rating Calculation Accepted by mt338jb-P at 2008-07-28 13:34:52
Sufficiency Rating Calculation Accepted by mt338pg-P at 2006-08-23 15:28:19
MT338DR-P element inspection comments -
Structure 520149 - DATE 08/22/2006 -
Previous comments > Sufficiency Rating Calculation Accepted by mt338wd-P at 2005-01-11 07:33:32
Sufficiency Rating Calculation Accepted by mt338dr-P at 2003-01-14 06:53:52
MT338EP-P inspection comments -
Structure 520149 - DATE 2003-01-08 -
Previous comments > MT338LH inspection comments -
Structure 520149 - DATE 2/13/01 -
Previous comments > Sufficiency Rating Calculation Accepted by mt338jm at 2/14/01 12:15:49

Elements

Inspection Date: 4/11/2016 KMZQ

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
UNIT 0	234/3	R/Conc Cap	0	.	1009	100.	0	.	0	.	0	.	**1009 lf.**

Notes CORRECTIVE ACTION: The overgrown vegetation at bent 16 has been removed.

Condition State 2
NO CHANGE:
1) Several intermediate bent caps exhibit random cracks up to 0.050 inch wide cracks with efflorescence.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
UNIT 0	396/3	Other Abut Slope Pro	15242	100.	0	.	0	.	0	.	0	.	**15242 sf.**

Notes < none >

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
UNIT 0	290/3	Channel	1	100.	0	.	0	.	0	.	0	.	**1 ea.**

Notes < none >

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
UNIT 0	321/3	R/Conc Approach Slab	0	.	2	100.	0	.	0	.	0	.	**2 ea.**

Notes Condition State 2
NO CHANGE:
1) The are sealed and unsealed full length random cracks up to 0.050 inch wide in both approach slabs.

NOTE: The roadway and approach slab transitions are rough, due to minor settlement in the roadway asphalt.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
UNIT 0	475/3	R/Conc Walls	60	100.	0	.	0	.	0	.	0	.	**60 lf.**

Notes CORRECTIVE ACTION: The poured seal adjacent to the wing walls has been repaired.

Total Number of Elements: 12

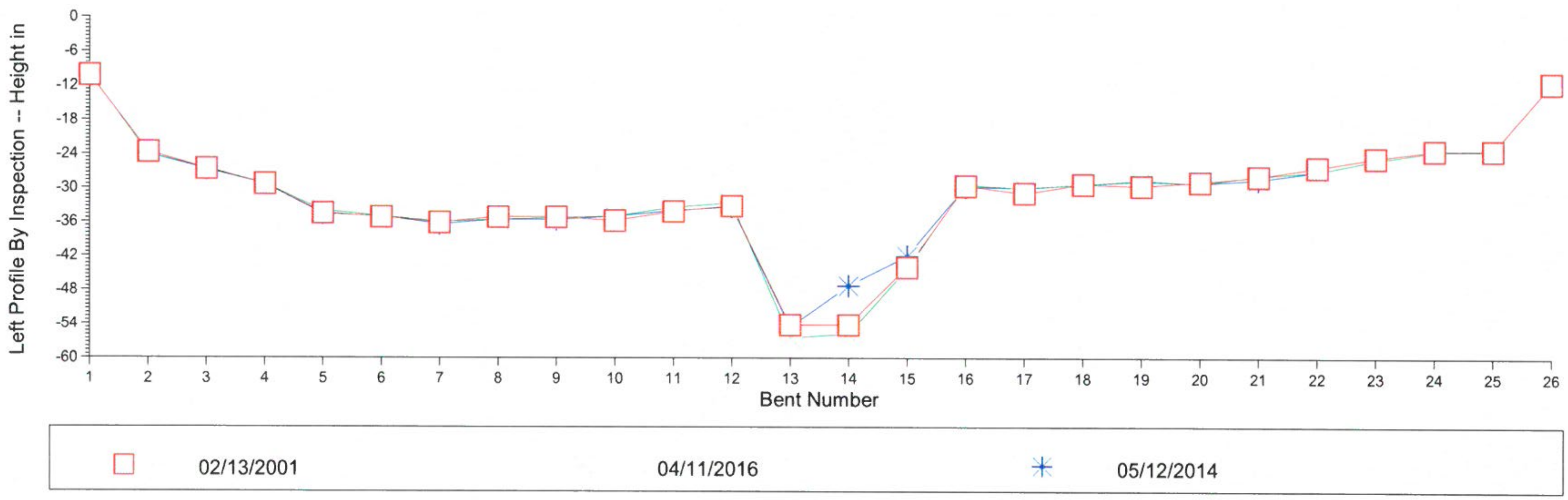
Inspection Information

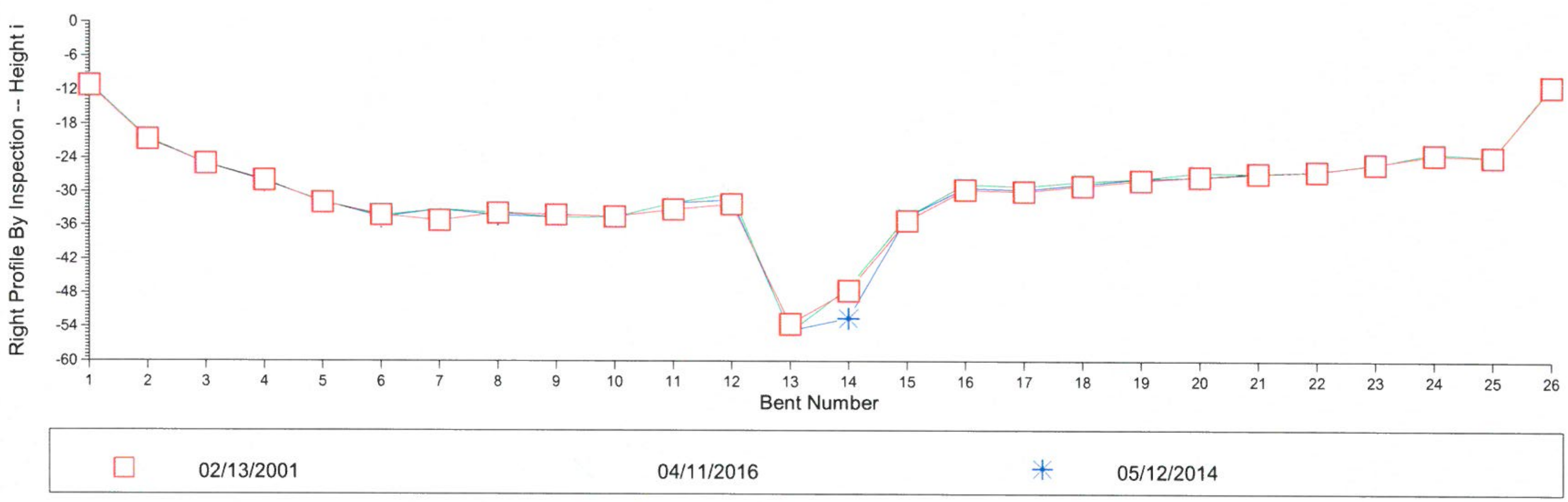
Inspection Date: 04.11.2016 **Type:** Regular NBI

Inspector: MT338GB-P - Glen Borges

Inspection Notes: Sufficiency Rating Calculation Accepted by mt338gb-P at 2016-05-02 20:02:06

Previous comments > Sufficiency Rating Calculation Accepted by mt338gm-P at 2014-05-14 08:36:17
Sufficiency Rating Calculation Accepted by mt338dr-P at 2012-05-24 15:13:17
Previous comments > Sufficiency Rating Calculation Accepted by mt338wd-P at 2010-06-29 13:55:13
MT338WD Inspection comments - The Divers reported there were no significant underwater deficiencies noted on this inspection.
Sufficiency Rating Calculation Accepted by mt338jb-P at 2008-07-28 13:34:52
Sufficiency Rating Calculation Accepted by mt338pg-P at 2006-08-23 15:28:19
MT338DR-P element inspection comments -
Structure 520149 - DATE 08/22/2006 -
Previous comments > Sufficiency Rating Calculation Accepted by mt338wd-P at 2005-01-11 07:33:32
Sufficiency Rating Calculation Accepted by mt338dr-P at 2003-01-14 06:53:52
MT338EP-P inspection comments -
Structure 520149 - DATE 2003-01-08 -
Previous comments > MT338LH inspection comments -
Structure 520149 - DATE 2/13/01 -
Previous comments > Sufficiency Rating Calculation Accepted by mt338jm at 2/14/01 12:15:49


Left Profile By Inspection -- Height in
0
-6
-12
-18
-24
-30
-36
-42
-48
-54
-60
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26
Bent Number
02/13/2001
04/11/2016
05/12/2014


Right Profile By Inspection -- Height i
0
-6
-12
-18
-24
-30
-36
-42
-48
-54
-60
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26
Bent Number
02/13/2001
04/11/2016
05/12/2014

REPORT ID: INVT016

Structure #: 520149

DATE PRINTED: 05/03/2016

Profile Data - Numerical Summary

Inspection Date and Key: 04/11/2016 KMZQ

(All Heights Are In Feet)

Bent #	Left Height	Right Height
1	10	11
2	24.1	20.5
3	26.5	25
4	29.3	28
5	34	31.7
6	35	34.3
7	36	33.2
8	35.6	33.8
9	35.3	34.6
10	35	34.6
11	33.4	32
12	32.6	30.3
13	56.5	55
14	55.8	47
15	44.3	34.4
16	29.4	28.6
17	30	29
18	29.3	28.1
19	28.6	27.6
20	29.1	26.5
21	27.8	26.7
22	27	26.2
23	25	25
24	23.5	23
25	23.3	23.6
26	11.5	11

Air Temp: 75

Profile Notes:

GROUNDLINE MEASUREMENTS FROM TOP OF BARRIERWALL.
WATERLINE MEASUREMENTS FROM TOP OF BARRIERWALL AT BENT 13.
Left = 31.0 ft. Right = 31.0 ft.

Profile Data - Numerical Summary

Inspection Date and Key: 05/12/2014 ODZR

(All Heights Are In Feet)

Bent #	Left Height	Right Height
1	10	11
2	24.1	20.7
3	26.7	25
4	29.3	28.1
5	34.6	31.7
6	35	34.6
7	36.4	33.2
8	35.6	34.2
9	35.6	34.6
10	35	34.6
11	34	32
12	33.3	31.4
13	54.6	54.7
14	47.3	52.5
15	42	34.5
16	29.7	29.2
17	29.9	29.6
18	29.3	28.6
19	28.7	27.6
20	29.1	27.3
21	28.4	26.7
22	27	26.3
23	25	25
24	23.5	23
25	23.3	23.6
26	11.5	11

Air Temp: 75

Profile Notes:

GROUNDLINE MEASUREMENTS FROM TOP OF BARRIERWALL.
WATERLINE MEASUREMENTS FROM TOP OF BARRIERWALL AT BENT 13.
Left = 32.5 ft. Right = 32.5 ft.

REPORT ID: INVT016

Structure #: 520149

DATE PRINTED: 05/03/2016

Profile Data - Numerical Summary

Inspection Date and Key: 02/13/2001 BVTL

(All Heights Are In Feet)

Bent #	Left Height	Right Height
1	10.17	11.15
2	23.62	20.67
3	26.57	24.93
4	29.2	27.89
5	34.45	31.82
6	35.1	34.12
7	36.09	35.1
8	35.1	33.79
9	35.1	34.12
10	35.76	34.45
11	34.12	33.14
12	33.14	32.15
13	54.13	53.48
14	54.13	47.57
15	43.96	35.1
16	29.53	29.53
17	30.84	29.86
18	29.2	28.87
19	29.53	27.89
20	28.87	27.23
21	27.89	26.57
22	26.25	26.25
23	24.61	24.93
24	23.29	23.29
25	23.29	23.62
26	11.48	11.15

Air Temp:

Profile Notes:

GROUND LINE MEASUREMENTS FROM TOP OF BARRIER WALL.
WATER LINE MEASUREMENTS FROM TOP OF BARRIER WALL AT BENT NO. 13
11.1 LEFT 11.1 RIGHT.

All Elements

UNIT: 0 SUBSTRUCTURE

ELEMENT/ENV: 388/3 R/Conc Fender/Dolphi 10 lf. ELEM CATEGORY: Substructure

CONDITION STATE (4)	DESCRIPTION	QUANTITY
1	The element shows little or no deterioration. There may be discoloration, efflorescence, and/or superficial cracking but without affect on strength and/or serviceability. Random open joints may exist.	10 lf.

ELEMENT INSPECTION NOTES:

NOTE: There is an abandoned test drilled shaft, at the right side of pier 59, that is serving as a dolphin.

ELEMENT/ENV: 396/3 Other Abut Slope Pro 24240 sf. ELEM CATEGORY: Substructure

CONDITION STATE (4)	DESCRIPTION	QUANTITY
1	There is little or no deterioration. Surface defects only are in evidence. Random open joints may exist.	24240 sf.

ELEMENT INSPECTION NOTES:

< none >

Elements

Inspection Date: 10/1/2015CAGR

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	207/3	P/S Conc Holl Pile	184	89.76	16	7.8	5	2.44	0	.	0	.	**205 ea.**

Notes Condition State 2
NEW:
1) The near face of pile 20-5 exhibits a full length longitudinal crack up to 0.025 inch wide.
2) The left and far face of pile 24-2, adjacent to the ground line, exhibits a minor area of honey combing with no exposed steel.
3) Pile 30-1 exhibits a wrap around crack up to 0.020 inch wide adjacent to the splice area.

NO CHANGE:
1) The following piles exhibit minor spalls with no exposed steel:
A) 5-5, near face
B) 21-4 right face
C) 23-3, far right face
D) 23-4, near right face
E) 36-3, near right face

2) The following piles exhibit random cracks, up to 0.025 inch wide, near the splice area:
A) 18-1, left face
B) 19-3, left face
C) 20-2, left face (full length)
D) 21-1, left face with discoloration
E) 21-2, all faces
F) 21-4, right face
G) 27-3, near and far face
H) 29-5, near and far face

Condition State 3
NEW:
1) The piles in bent 22 exhibit loss of ground line up to 6.7 ft. since the 2013 inspection. (See Photo 4)

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	210/3	R/Conc Pier Wall	0	.	156	100.	0	.	0	.	0	.	**156 lf.**

Notes Condition State 2
NEW:
1) The pier walls exhibit minor to moderate scaling up to the high water mark.

NO CHANGE:
1) There is a section of seal concrete 0.8 ft. thick under the pier wall at pier 59. This section has separated from the wall concrete and dropped down exposing the reinforcing steel underwater. (Photo could not taken at time of inspection due to high water and visibility)

NOTE: This should be monitored during routine inspections.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	215/3	R/Conc Abutment	100	100.	0	.	0	.	0	.	0	.	**100 lf.**

Notes < none >

Elements

Inspection Date: 10/1/2015CAGR

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	234/3	R/Conc Cap	3293	98.51	50	1.49	5	.	0	.	0	.	3348 lf.

Notes CORRECTIVE ACTION: The overgrown vegetation has been removed from the near end of the structure.

Condition State 2
NO CHANGE:
1) The right face of intermediate cap 65 exhibits a transverse crack up to 0.025 inch wide.
2) Several intermediate bent caps exhibits a minor build-up of bat guano.

Condition State 3
NO CHANGE:
1) The right end of cap 47 exhibits a delamination 5.0 ft. long X 0.4 ft. high X 2.0 ft. deep. (See Photo 5)

NOTE: The delamination on cap 47 has been smeared with epoxy.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	298/3	Pile Jacket Bare	1	100.	0	.	0	.	0	.	0	.	1 ea.

Notes NOTE: There is a concrete jacket on pile 22-4.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	388/3	R/Conc Fender/Dolphi	10	100.	0	.	0	.	0	.	0	.	10 lf.

Notes NOTE: There is an abandoned test drilled shaft, at the right side of pier 59, that is serving as a dolphin.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	396/3	Other Abut Slope Pro	24240	100.	0	.	0	.	0	.	0	.	24240 sf.

Notes < none >

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	290/3	Channel	1	100.	0	.	0	.	0	.	0	.	1 ea.

Notes < none >

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	321/3	R/Conc Approach Slab	0	.	2	100.	0	.	0	.	0	.	2 ea.

Notes Condition State 2
NEW:
1) Both approach slabs exhibit random cracks up to 0.030 inch wide.

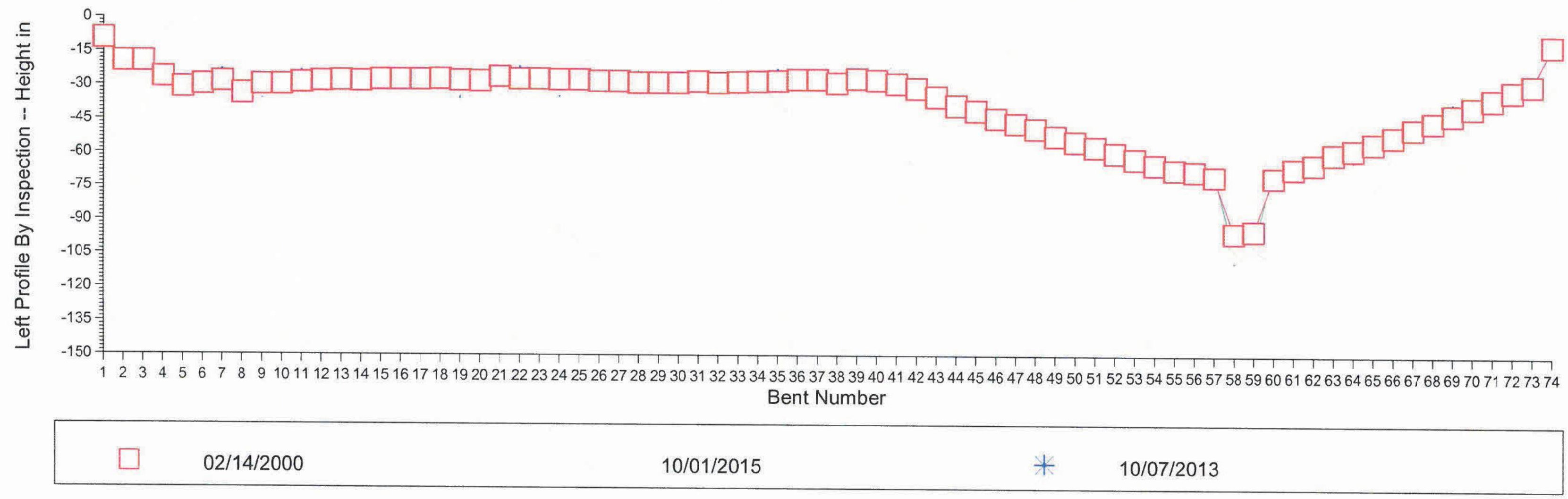
NO CHANGE:
1) There is a full length longitudinal crack up to 0.025 inch wide in the safety lane of the near approach slab and the left travel lane of the far approach slab.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	475/3	R/Conc Walls	57	100.	0	.	0	.	0	.	0	.	57 lf.

Notes CORRECTIVE ACTION: The open joint adjacent to the near left wing wall has been repaired.

Total Number of Elements: 20

FLORIDA DEPARTMENT OF TRANSPORTATION BRIDGE MANAGEMENT SYSTEM
Inspection/CID/Bridge Profile Report



Profile Data - Numerical Summary

Inspection Date and Key: 10/01/2015 CAGR

(All Heights Are In Feet)

Bent #	Left Height	Right Height
1	9	9.5
2	19.4	18
3	20.8	22.3
4	25.6	23.7
5	30.9	32.1
6	30.2	27.6
7	29.8	28.5
8	32.3	36
9	30.7	31.7
10	28	29.7
11	32.2	31.5
12	30.5	29.6
13	29.3	27.5
14	27.2	31
15	28.5	28.3
16	27	29.2
17	28.9	28.5
18	31	32.5
19	29.8	30
20	27	34
21	29.4	28.5
22	32.7	32.5
23	30.4	28.5
24	29.3	30.8
25	29.7	27.5
26	27.5	29.9
27	29.7	28.7
28	29	28.5
29	29.3	26.5
30	29	28.1
31	28.6	25.3
32	28.5	27.8
33	28	27.5

Profile Data - Numerical Summary

Inspection Date and Key: 10/01/2015 CAGR

(All Heights Are In Feet)

Bent #	Left Height	Right Height
34	27.5	27
35	36.7	23
36	26	25.6
37	26.4	22.5
38	26.5	25
39	25.8	21.5
40	26.6	24.9
41	28.3	23.5
42	30.3	28.9
43	34.2	30
44	38	37.2
45	44.4	39
46	44.5	43.2
47	46	42
48	48	47
49	51.4	47
50	54	52.9
51	56	54.5
52	60.5	57
53	61.5	60.5
54	64.4	64.6
55	66.7	65
56	68	68.2
57	68	69
58	103	100
59	101	98.7
60	66.2	69.7
61	67	66.5
62	62.7	66.1
63	57.3	64.1
64	57	62.5
65	54.5	56
66	51	55.8

Profile Data - Numerical Summary

(All Heights Are In Feet)

Inspection Date and Key: 10/01/2015 CAGR

Bent #	Left Height	Right Height
67	47.2	53
68	44.6	49.3
69	41.9	44.3
70	38	43.1
71	34.8	36.8
72	31.6	35.9
73	29	30.5
74	11	11

Air Temp: 72

Profile Notes:

GROUND LINE MEASUREMENTS FROM TOP OF BARRIERWALL.
WATER LINE MEASUREMENTS FROM TOP OF BARRIERWALL AT BENT 59.
LEFT = 82.0 FT. RIGHT = 83.0 FT.

Inspection Date and Key: 10/07/2013 FWWP

Bent #	Left Height	Right Height
1	9	9.5
2	19.4	18
3	20.8	22.3
4	25.6	23.6
5	30.9	32.1
6	30.2	29
7	27.5	28.5
8	32.3	38.3
9	31	31.7
10	28	31
11	28	31.5
12	30.5	32.5
13	28.4	27.5
14	27.2	31
15	28.6	28.3
16	27	30.2
17	29	28.5
18	31	32.5

Profile Data - Numerical Summary

Inspection Date and Key: 10/07/2013 FWWP

(All Heights Are In Feet)

Bent #	Left Height	Right Height
19	31	30
20	27	34.1
21	29.5	28.5
22	26	33
23	30.4	28.5
24	30	30.9
25	29.4	27.5
26	27.5	29.6
27	29.4	25.6
28	28.2	28.5
29	29.4	26.5
30	29	28.2
31	28.6	25.3
32	28.5	27.9
33	28.3	27.5
34	27.5	26.9
35	26.9	23
36	26	25.8
37	26.4	22.5
38	26.5	25
39	25.8	21.5
40	27.2	24.8
41	28.3	23.5
42	30.3	28.9
43	34.3	30
44	38	37
45	40.7	39
46	44.5	44.5
47	46	42
48	48	47
49	51.4	47
50	54	52.9
51	56	55.2

Profile Data - Numerical Summary

Inspection Date and Key: 10/07/2013 FWWP

(All Heights Are In Feet)

Bent #	Left Height	Right Height
52	60.5	57
53	61.5	60.5
54	64.7	64.6
55	66.7	65
56	68	68.2
57	68	69
58	103.5	100.7
59	101	98.7
60	66.2	69.7
61	67	66.5
62	62.7	66.1
63	57.3	63
64	58	62.5
65	54.5	56
66	51	55.8
67	47.2	50.5
68	44.6	49.3
69	41.6	44.3
70	38	43.1
71	34.8	36.8
72	31.6	35.9
73	29	30.5
74	11	11

Air Temp: 88

Profile Notes:

GROUND LINE MEASUREMENTS FROM TOP OF BARRIERWALL.
WATER LINE MEASUREMENTS FROM TOP OF BARRIERWALL AT BENT 59.
LEFT = 83.8 RIGHT = 84.6

Inspection Date and Key: 02/14/2000 PQZM

Bent #	Left Height	Right Height
1	9.19	9.51
2	19.36	18.04
3	19.36	19.36

Profile Data - Numerical Summary

Inspection Date and Key: 02/14/2000 PQZM

(All Heights Are In Feet)

Bent #	Left Height	Right Height
4	26.25	23.29
5	30.84	29.53
6	29.53	27.89
7	28.22	28.87
8	33.46	35.43
9	29.53	28.87
10	29.53	29.53
11	28.54	31.17
12	27.89	29.53
13	27.56	27.89
14	27.89	28.22
15	27.23	27.89
16	27.23	27.56
17	27.23	28.87
18	26.9	29.86
19	27.56	30.51
20	27.89	30.84
21	25.92	29.53
22	26.9	30.51
23	26.9	29.2
24	27.23	28.54
25	27.23	27.56
26	27.89	27.23
27	27.89	26.57
28	28.54	26.57
29	28.54	26.25
30	28.54	25.92
31	27.89	25.26
32	28.54	25.59
33	28.22	25.59
34	27.89	24.61
35	27.56	23.29
36	26.9	22.97

Profile Data - Numerical Summary

Inspection Date and Key: 02/14/2000 PQZM

(All Heights Are In Feet)

Bent #	Left Height	Right Height
37	26.9	22.31
38	28.54	22.64
39	26.57	21.98
40	27.23	22.64
41	28.87	23.62
42	30.84	26.9
43	34.45	29.86
44	38.39	33.79
45	40.68	36.75
46	43.96	44.29
47	46.26	42.32
48	48.56	44.62
49	51.84	46.59
50	54.46	50.52
51	56.76	54.79
52	59.38	57.41
53	62.01	58.07
54	64.63	62.34
55	66.93	65.29
56	67.59	65.94
57	69.88	69.88
58	95.14	94.82
59	94.16	94.49
60	70.21	76.77
61	66.27	65.94
62	64.3	63.98
63	59.71	60.04
64	57.74	59.71
65	54.79	56.43
66	51.84	53.15
67	48.23	50.52
68	45.28	46.59
69	41.99	43.64

Profile Data - Numerical Summary

Inspection Date and Key: 02/14/2000 PQZM

(All Heights Are In Feet)

Bent #	Left Height	Right Height
70	38.71	40.35
71	35.1	37.07
72	31.5	33.79
73	28.87	30.18
74	11.15	10.83

Air Temp: 68

Profile Notes:

GROUNDLINE MEASUREMENT FROM TOP OF CONCRETE RAIL.
WATERLINE MEASUREMENT FROM TOP OF CONCRETE RAIL AT BENT 59.
Left = 24 Right = 24

FLORIDA DEPARTMENT OF TRANSPORTATION
BRIDGE MANAGEMENT SYSTEM
Inspection/CID/Bridge Profile Report

Inspector Recommendations

UNIT: 0 SUBSTRUCTURE

ELEMENT/ENV:396/4 Other Abut Slope Pro ELEM CATEGORY: Substructure

CONDITION STATE (4)		Priority
2	**1410 sf.**	3

WORK ORDER RECOMMENDATION:

Remove overgrown vegetation from the near and far slope protections.

Structure Notes

PRIOR BRIDGE# = 590007
Revised 11/03/06

INSPECTION NOTES: GZTZ 6/21/2016

Sufficiency Rating Calculation Accepted by mt338gb-P at 2016-06-28 16:36:45
An underwater inspection was not needed due to low water depth.

Previous comments > Sufficiency Rating Calculation Accepted by knvolss-P at 2014-07-30 14:50:38
UW NOT REQUIRED PER LEAD INSPECTOR
Sufficiency Rating Calculation Accepted by knvolwc-P at 2012-07-30 16:22:20
UW TANK = 8/1/12
Sufficiency Rating Calculation Accepted by KN338CD-P at 2010-08-03 11:50:27
LOW WATER - DIVE NOT NEEDED
Sufficiency Rating Calculation Accepted by KN338CD-P at 2008-08-19 16:46:54
LOW WATER - DIVE NOT NEEDED
Sufficiency Rating Calculation Accepted by kn338cd-P at 2006-09-19 15:39:54
LOW WATER - DIVE NOT NEEDED
Sufficiency Rating Calculation Accepted by kn338cd-P at 2004-10-08 16:21:56
LOW WATER - DIVE NOT NEEDED
Sufficiency Rating Calculation Accepted by kn338mv-P at 2002-10-18 13:54:31
KN338MV-P inspection comments - LOW WATER - DIVE NOT NEEDED
Structure 590048 - Date 2002-10-08
Sufficiency Rating Calculation Accepted by mt338ep at 11/1/00 13:01:42
MT338EP inspection comments - Structure 590048 - Date 10/25/00
MT338CT inspection comments - Structure 590048 - Date 11/16/1998

Elements

Inspection Date: 6/21/2016GZTZ

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	321/4	R/Conc Approach Slab	0	.	2	100.	0	.	0	.	0	.	2 ea.

Notes Condition State 2
NO CHANGE:
1) The near and far approach slabs exhibit sealed and unsealed longitudinal cracks up to 0.020 inch wide.

Span Id	Elem/Env	Description	Qty1	%1	Qty2	%2	Qty3	%3	Qty4	%4	Qty5	%5	T Qty
0	475/4	R/Conc Walls	35	97.22	1	2.78	0	.	0	.	0	.	36 lf.

Notes Condition State 2
NEW:
1) The near right wingwall exhibits a minor spall with no exposed steel adjacent to the strip seal joint.

Total Number of Elements: 12

Inspection Information

Inspection Date: 06.21.2016 **Type:** Regular NBI

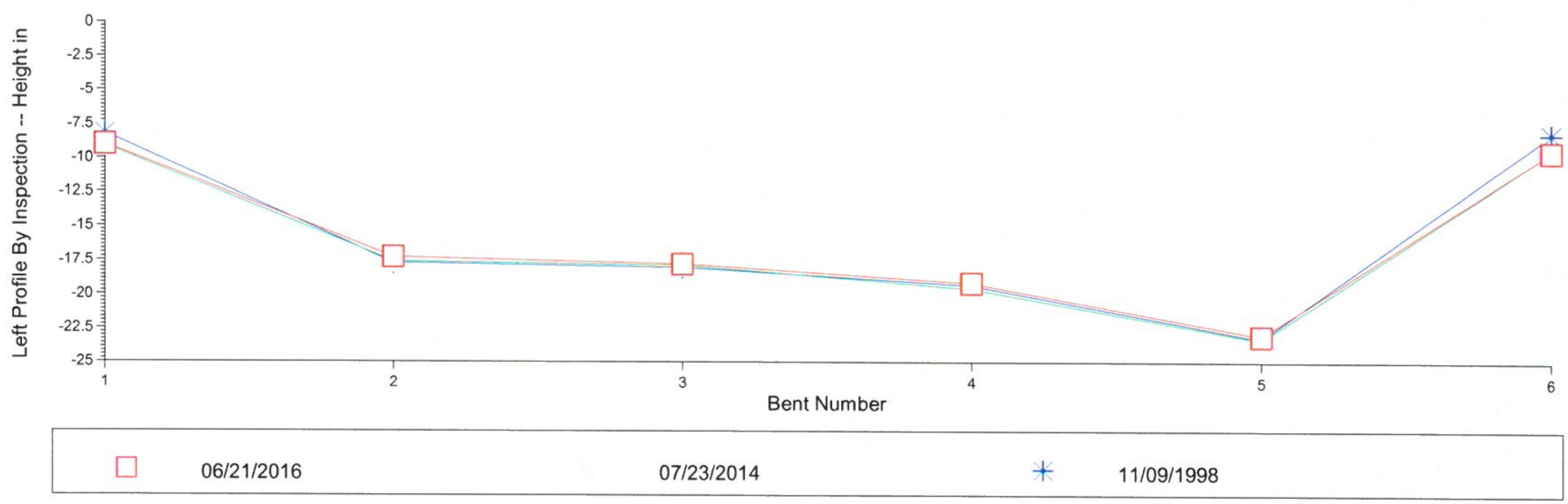
Inspector: MT338GB-P - Glen Borges

Inspection Notes: Sufficiency Rating Calculation Accepted by mt338gb-P at 2016-06-28 16:36:45
An underwater inspection was not needed due to low water depth.

Previous comments > Sufficiency Rating Calculation Accepted by knvolss-P at 2014-07-30 14:50:38
UW NOT REQUIRED PER LEAD INSPECTOR
Sufficiency Rating Calculation Accepted by knvolwc-P at 2012-07-30 16:22:20
UW TANK = 8/1/12
Sufficiency Rating Calculation Accepted by KN338CD-P at 2010-08-03 11:50:27
LOW WATER - DIVE NOT NEEDED
Sufficiency Rating Calculation Accepted by KN338CD-P at 2008-08-19 16:46:54
LOW WATER - DIVE NOT NEEDED
Sufficiency Rating Calculation Accepted by kn338cd-P at 2006-09-19 15:39:54
LOW WATER - DIVE NOT NEEDED
Sufficiency Rating Calculation Accepted by kn338cd-P at 2004-10-08 16:21:56
LOW WATER - DIVE NOT NEEDED
Sufficiency Rating Calculation Accepted by kn338mv-P at 2002-10-18 13:54:31
KN338MV-P inspection comments - LOW WATER - DIVE NOT NEEDED
Structure 590048 - Date 2002-10-08
Sufficiency Rating Calculation Accepted by mt338ep at 11/1/00 13:01:42
MT338EP inspection comments - Structure 590048 - Date 10/25/00
MT338CT inspection comments - Structure 590048 - Date 11/16/1998

Structure Notes

PRIOR BRIDGE# = 590007
Revised 11/03/06


Left Profile By Inspection -- Height in
0
-2.5
-5
-7.5
-10
-12.5
-15
-17.5
-20
-22.5
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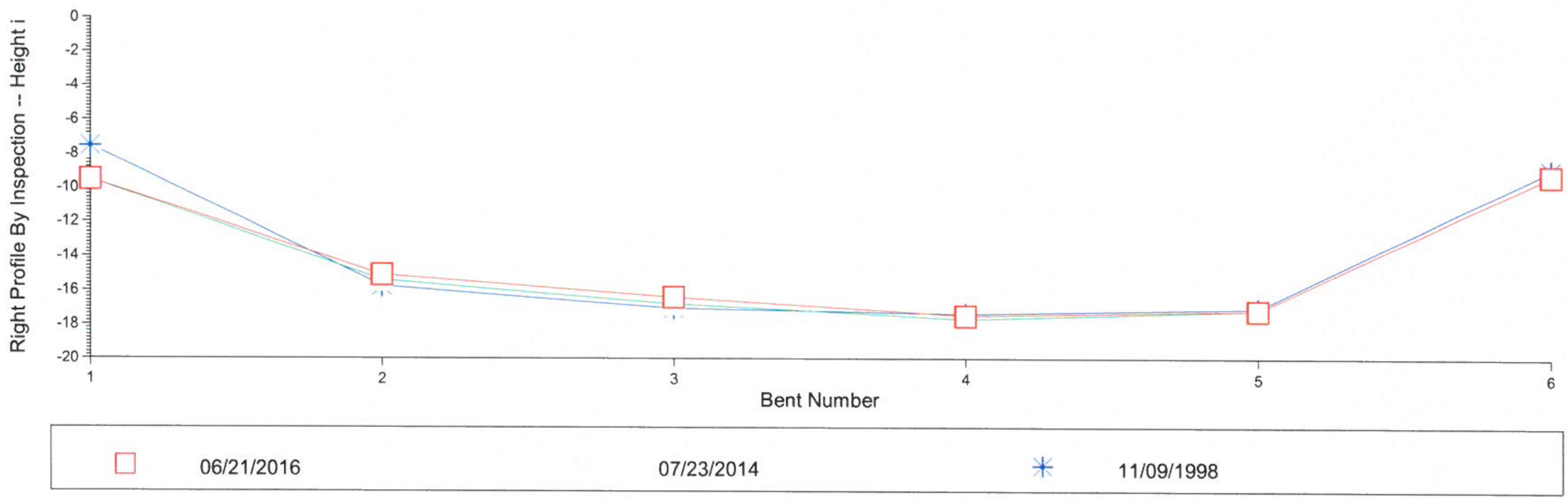
1
2
3
4
5
6
Bent Number
06/21/2016
07/23/2014
11/09/1998


Right Profile By Inspection -- Height i
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
1
2
3
4
5
6
Bent Number
06/21/2016
07/23/2014
11/09/1998

Profile Data - Numerical Summary

Inspection Date and Key: 06/21/2016 GZTZ

(All Heights Are In Feet)

Bent #	Left Height	Right Height
1	9	9.5
2	17.3	15.1
3	17.8	16.4
4	19.2	17.5
5	23.1	17.2
6	9.5	9.2

Air Temp: 90

Profile Notes:

GROUND LINE MEASUREMENTS FROM TOP OF BARRIERWALL.
WATER LINE MEASUREMENTS FROM TOP OF BARRIERWALL AT BENT 4.
LEFT = 17.0 RIGHT = 15.8

Inspection Date and Key: 07/23/2014 MMRM

Bent #	Left Height	Right Height
1	9.1	9.5
2	17.6	15.4
3	17.9	16.8
4	19.6	17.7
5	23.4	17.2
6	9.5	9.2

Air Temp:

Profile Notes:

Waterway Measurements: Top of rail to water line at Bent 5; 18.0 ft left and 16.6 ft right.
Groundline Measurements from top of rail.

Inspection Date and Key: 11/09/1998 YHYO

Bent #	Left Height	Right Height
1	8.2	7.55
2	17.72	15.75
3	18.04	17.06
4	19.36	17.39
5	23.29	17.06

Profile Data - Numerical Summary

Inspection Date and Key: 11/09/1998 YHYO

Bent #	Left Height	Right Height
6	8.2	8.86

(All Heights Are In Feet)

Air Temp: 0

Profile Notes:

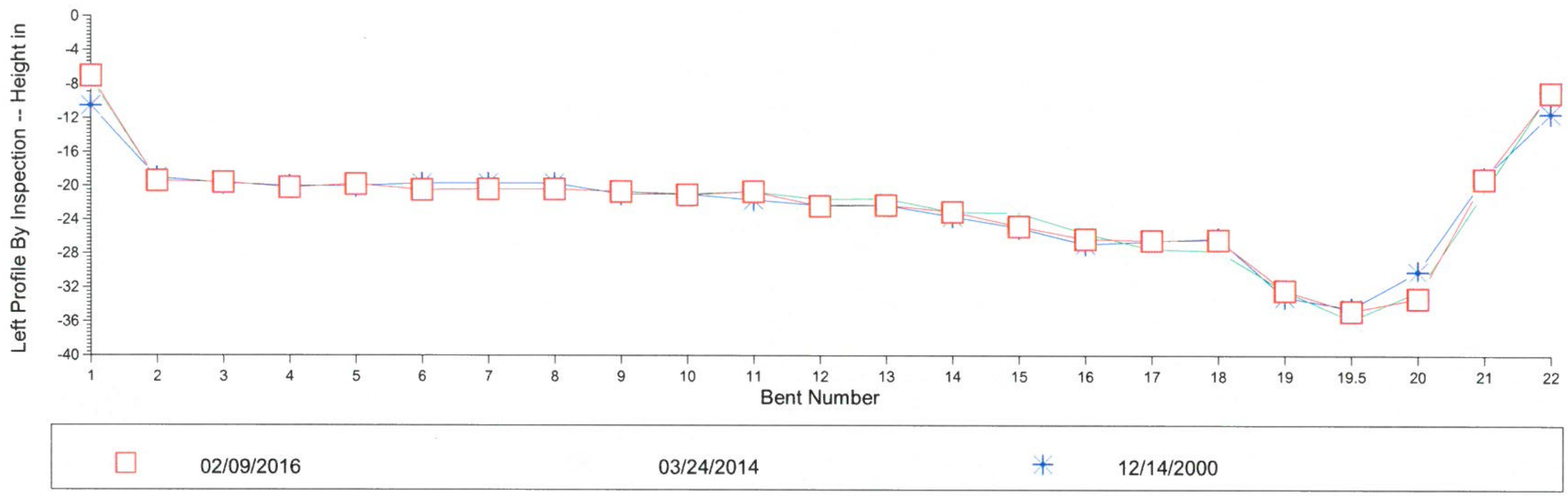
GROUND LINE MEASUREMENTS FROM TOP OF BARRIER RAIL.
WATER LINE MEASUREMENTS FROM TOP OF BARRIER RAIL AT BENT NO. 4.
LEFT 5.4 RIGHT 4.9

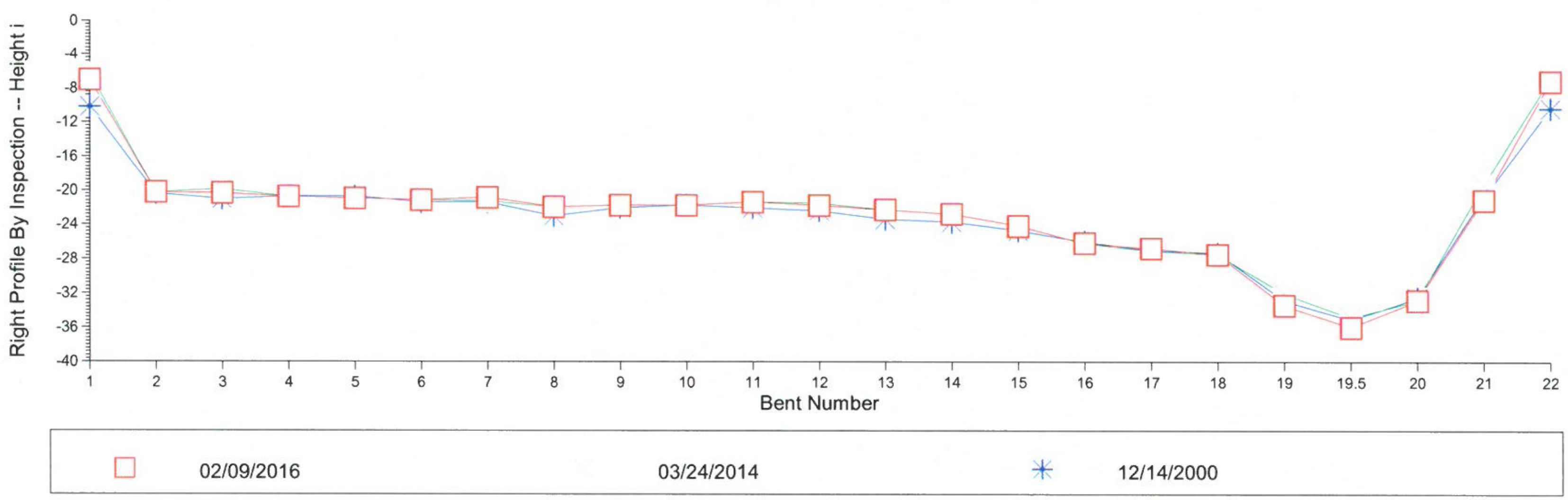
FLORIDA DEPARTMENT OF TRANSPORTATION
BRIDGE MANAGEMENT SYSTEM
Inspection/CID/Bridge Profile Report

INSPECTION NOTES: UWCP 2/9/2016

Sufficiency Rating Calculation Accepted by mt338gb-P at 2016-02-25 14:27:18

Previous comments > Sufficiency Rating Calculation Accepted by mt338gm-P at 2014-03-31 17:15:41
MT338GM - Inspection Comments - The underwater inspection could not be preformed due to unsafe / high water conditions.
Previous comments > Sufficiency Rating Calculation Accepted by mt338jb-P at 2012-04-05 13:46:43
Sufficiency Rating Calculation Accepted by mt338gm-P at 2010-05-18 07:57:16
Previous comments >Sufficiency Rating Calculation Accepted by mt338jb-P at 2008-06-04 13:52:15
Sufficiency Rating Calculation Accepted by mt338jb-P at 2006-07-31 15:46:55
Previous comments > On the day of inspection Sam Weed was notified about the boiling spring at drill shaft 2-20 and he is going to have it sealed off in the near future.
Sufficiency Rating Calculation Accepted by mt338dr-P at 2004-08-24 06:22:19
MT338DR-P-inspection comments-
Structure 500124- DATE 8/23/04-
Previous comments > Sufficiency Rating Calculation Accepted by mt338dr-P at 2002-11-06 07:46:32
MT338EP-P inspection comments -
Structure 500124 - DATE 2002-11-05 -
Previous comments > Sufficiency Rating Calculation Accepted by mt338wd at 12/26/00 14:12:12
MT338TK inspection comments - This structure was inspected north to south. At time of inspection the structure was carrying north and south bound traffic. Upon completion of Bridge Number 500127, the structure will carry north bound traffic only. The Inspector 50 was not used due to temporary barricades on center line of structure.
Structure 500124 - DATE 12/14/2000 -


Left Profile By Inspection -- Height in
0
-4
-8
-12
-16
-20
-24
-28
-32
-36
-40
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 19.5 20 21 22
Bent Number
02/09/2016
03/24/2014
12/14/2000


Right Profile By Inspection -- Height i
0
-4
-8
-12
-16
-20
-24
-28
-32
-36
-40
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 19.5 20 21 22
Bent Number
02/09/2016
03/24/2014
12/14/2000

Profile Data - Numerical Summary

Inspection Date and Key: 02/09/2016 UWCP

Bent #	Left Height	Right Height
1	7	7
2	19.4	20.2
3	19.6	20.3
4	20.2	20.7
5	19.8	20.9
6	20.5	21.1
7	20.4	20.8
8	20.4	21.9
9	20.7	21.7
10	21.1	21.7
11	20.7	21.3
12	22.4	21.7
13	22.3	22.2
14	23.1	22.7
15	24.8	24.1
16	26.3	26.1
17	26.5	26.7
18	26.4	27.4
19	32.4	33.4
19.5	34.8	36
20	33.3	32.8
21	19.2	21
22	9	7

(All Heights Are In Feet)

Air Temp: 55

Profile Notes:

GROUNDLINE MEASUREMENTS FROM TOP OF BARRIER WALL.
WATERLINE MEASUREMENTS FROM TOP OF BARRIER WALL AT BENT 19.5.
LEFT = 20.7 FT RIGHT = 21.4 FT

Inspection Date and Key: 03/24/2014 SILC

Bent #	Left Height	Right Height
1	7.3	6.3
2	19.4	20.2
3	19.6	19.8

Profile Data - Numerical Summary

(All Heights Are In Feet)

Inspection Date and Key: 03/24/2014 SILC

Bent #	Left Height	Right Height
4	20.2	20.7
5	19.8	20.9
6	20.5	21.1
7	20.4	21.2
8	20.4	21.9
9	20.7	21.7
10	21	21.7
11	20.7	21.3
12	21.6	21.4
13	21.5	22.2
14	23.1	22.7
15	23.2	24.1
16	25.6	26.1
17	27.5	27
18	27.7	27.4
19	32.4	32
19.5	35.8	34.8
20	32.5	32.8
21	20.7	19.2
22	8.8	6.7

Air Temp: 73

Profile Notes:

GROUNDLINE MEASUREMENT FROM TOP OF BARRIER WALL.
WATERLINE MEASUREMENT FROM TOP OF BARRIER WALL AT BENT 19.5.
LEFT = 22.1 FT RIGHT = 21.3 FT

Inspection Date and Key: 12/14/2000 AVRY

Bent #	Left Height	Right Height
1	10.5	10.17
2	19.03	20.34
3	19.69	21
4	20.01	20.67
5	20.01	20.67
6	19.69	21.33

Profile Data - Numerical Summary

Inspection Date and Key: 12/14/2000 AVRY

(All Heights Are In Feet)

Bent #	Left Height	Right Height
7	19.69	21.33
8	19.69	22.97
9	21	21.98
10	21	21.65
11	21.65	21.98
12	22.31	22.31
13	22.31	23.29
14	23.62	23.62
15	24.93	24.61
16	26.9	25.92
17	26.57	26.9
18	26.25	27.23
19	33.14	32.81
19.5	34.45	35.1
20	30.18	32.48
21	19.03	20.67
22	11.48	10.17

Air Temp: 55

Profile Notes:

GROUND LINE MEASUREMENTS FROM TOP OF BARRIER WALL.
WATER LINE MEASUREMENT FROM TOP OF BARRIER WALL AT MID-SPAN 19.
LEFT SIDE - 10.5 RIGHT SIDE - 10.7

INSPECTION NOTES: DQZH 2/9/2016

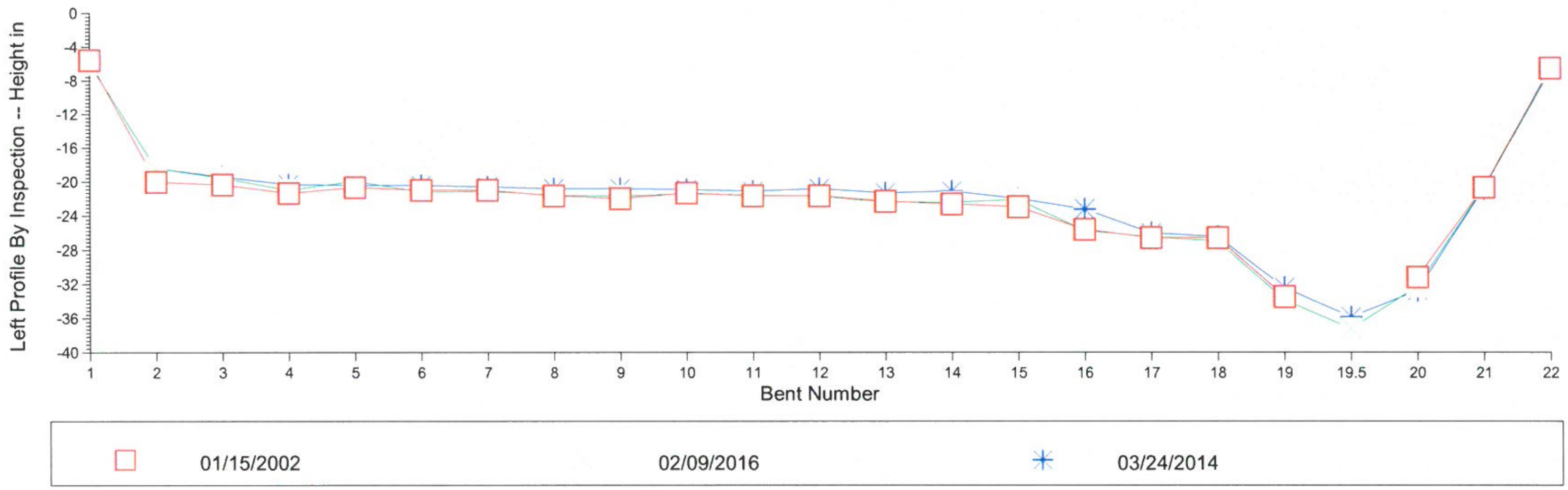
Sufficiency Rating Calculation Accepted by mt338gb-P at 2016-02-25 15:13:31

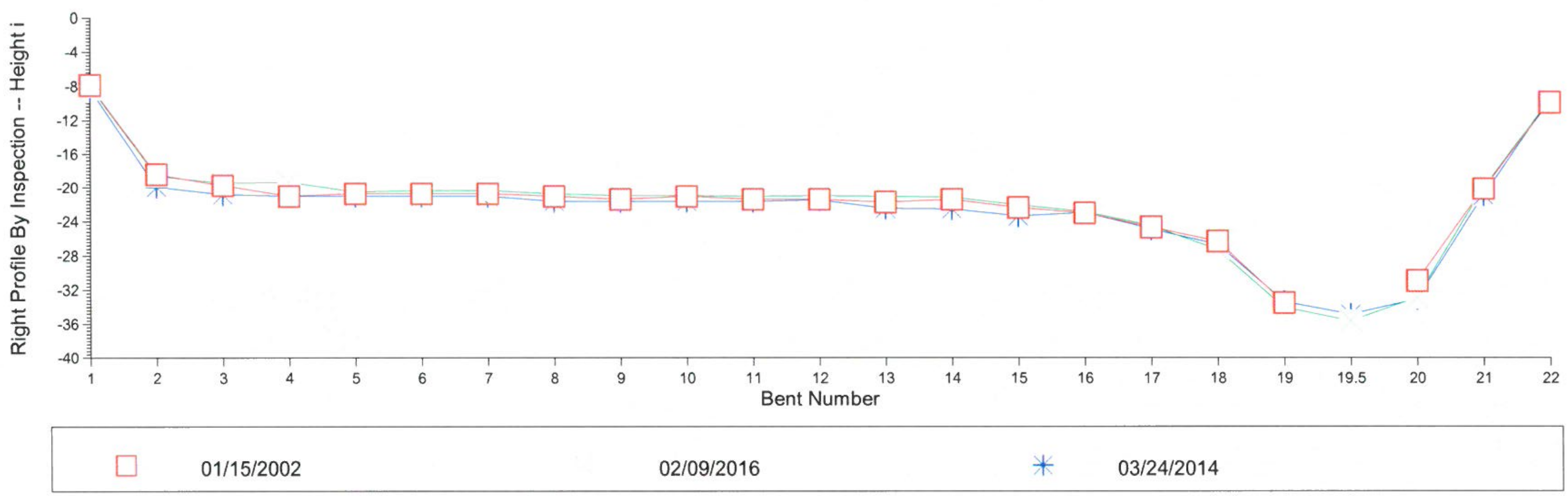
Previous comments > Sufficiency Rating Calculation Accepted by mt338gm-P at 2014-04-02 09:16:24
MT338GM - Inspection Comments - The underwater inspection could not be preformed due to unsafe / high water conditions.
Previous comments > Sufficiency Rating Calculation Accepted by mt338jb-P at 2012-04-17 14:23:03
Sufficiency Rating Calculation Accepted by mt338gm-P at 2010-05-18 08:22:50
Previous comments >Sufficiency Rating Calculation Accepted by mt338pg-P at 2008-06-04 15:49:44
Sufficiency Rating Calculation Accepted by mt338jb-P at 2006-07-24 14:38:27
Sufficiency Rating Calculation Accepted by mt338wb-P at 2004-08-26 12:50:59
MT338WD inspection comments - DATE 2004-08-25
Previous comments > Sufficiency Rating Calculation Accepted by mt338wb-P at 2002-12-16 14:16:51
MT338AS-P inspection comments -
Structure 500127 - DATE 2002-12-16 -
Previous comments > Sufficiency Rating Calculation Accepted by mt338wd at 1/23/02 08:40:47
MT338TK inspection comments -
Structure 500127 - DATE 1/15/02 -
Previous comments >

REPORT ID: INVT016

Structure #: 500127

DATE PRINTED: 02/25/2016





Profile Data - Numerical Summary

Inspection Date and Key: 02/09/2016 DQZH

(All Heights Are In Feet)

Bent #	Left Height	Right Height
1	6	8
2	18.4	18.6
3	19.5	19.4
4	21	19.3
5	19.9	20.5
6	21.2	20.3
7	21.1	20.3
8	21.6	20.7
9	21.7	20.9
10	21.4	20.9
11	21.6	21
12	21.7	20.9
13	22.4	21
14	22.4	21.1
15	22.1	22
16	25.7	22.8
17	26.5	24.4
18	26.9	27.2
19	33.8	34
19.5	37.2	35.6
20	32.2	32.8
21	20.7	19.9
22	7	9.5

Air Temp: 55

Profile Notes:

GROUND LINE MEASUREMENTS FROM TOP OF BARRIER WALL.
WATER LINE MEASUREMENTS FROM TOP OF BARRIER WALL AT BENT 19.5
LEFT = 21.3 FT RIGHT = 20.8 FT

Inspection Date and Key: 03/24/2014 QDGG

Bent #	Left Height	Right Height
1	6	8.3
2	18.4	19.9
3	19.4	20.8

Profile Data - Numerical Summary

Inspection Date and Key: 03/24/2014 QDGG

(All Heights Are In Feet)

Bent #	Left Height	Right Height
4	20.3	21
5	20.4	21
6	20.4	21
7	20.6	21
8	20.8	21.6
9	20.8	21.6
10	20.9	21.6
11	21.1	21.6
12	20.8	21.4
13	21.3	22.4
14	21.1	22.5
15	22	23.3
16	23.2	22.9
17	26	24.8
18	26.4	26.6
19	32.4	33.3
19.5	35.8	34.8
20	32.7	33
21	20.8	20.8
22	6.8	9.5

Air Temp: 73

Profile Notes:

GROUND LINE MEASUREMENTS FROM TOP OF BARRIER WALL.
WATER LINE MEASUREMENTS FROM TOP OF BARRIER WALL AT BENT 19.5
LEFT = 21.9 FT RIGHT = 21.3 FT

Inspection Date and Key: 01/15/2002 VNOH

Bent #	Left Height	Right Height
1	5.58	7.87
2	20.01	18.37
3	20.34	19.69
4	21.33	21
5	20.67	20.67
6	21	20.67

Profile Data - Numerical Summary

Inspection Date and Key: 01/15/2002 VNOH

(All Heights Are In Feet)

Bent #	Left Height	Right Height
7	21	20.67
8	21.65	21
9	21.98	21.33
10	21.33	21
11	21.65	21.33
12	21.65	21.33
13	22.31	21.65
14	22.64	21.33
15	22.97	22.31
16	25.59	22.97
17	26.57	24.61
18	26.57	26.25
19	33.46	33.46
20	31.17	30.84
21	20.67	20.01
22	6.56	9.84

Air Temp: 65

Profile Notes:

GROUND LINE MEASUREMENTS FROM TOP OF BARRIER WALL.
WATER LINE MEASUREMENT FROM TOP OF BARRIER WALL AT BENT 19.
LEFT SIDE - 10.2 RIGHT SIDE - 10.2

APPENDIX M: UNDERWATER IMAGES
Photoshopped Images with Originals

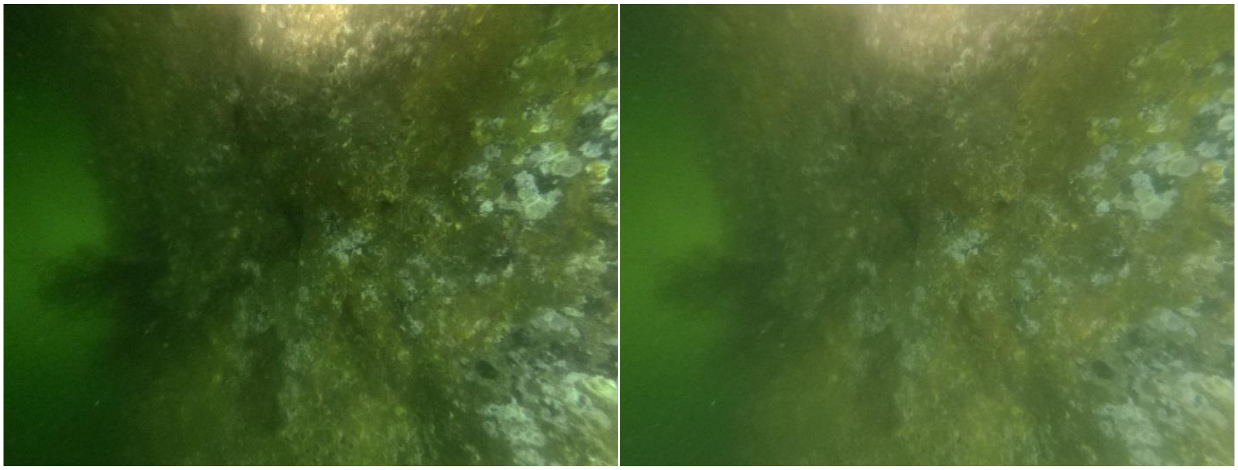


Figure M.1. Figure 10.20, photoshopped (left) and original (right)

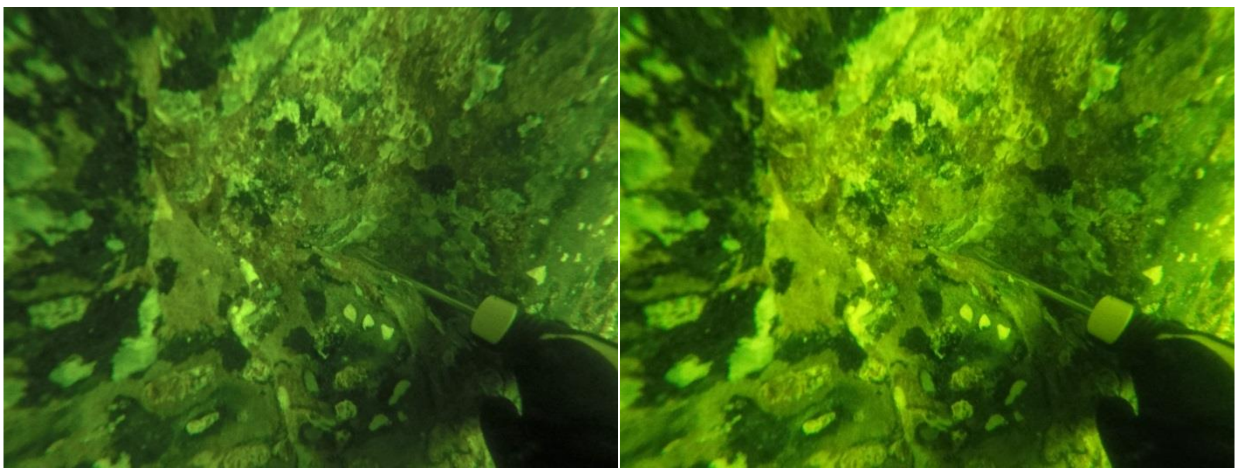


Figure M.2. Figure 10.22, photoshopped (left) and original (right)

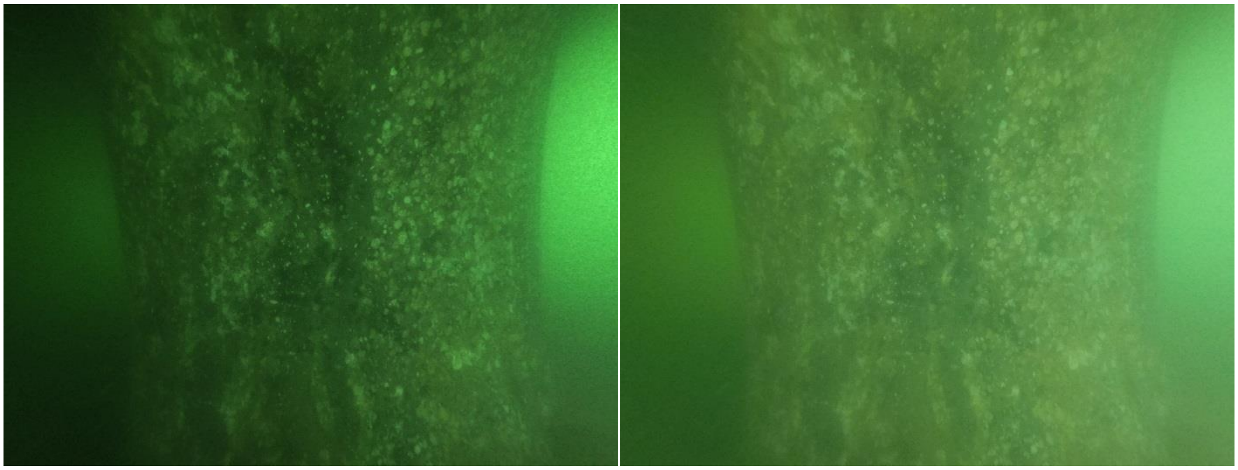


Figure M.3. Figure 10.23(a), photoshopped (left) and original (right)

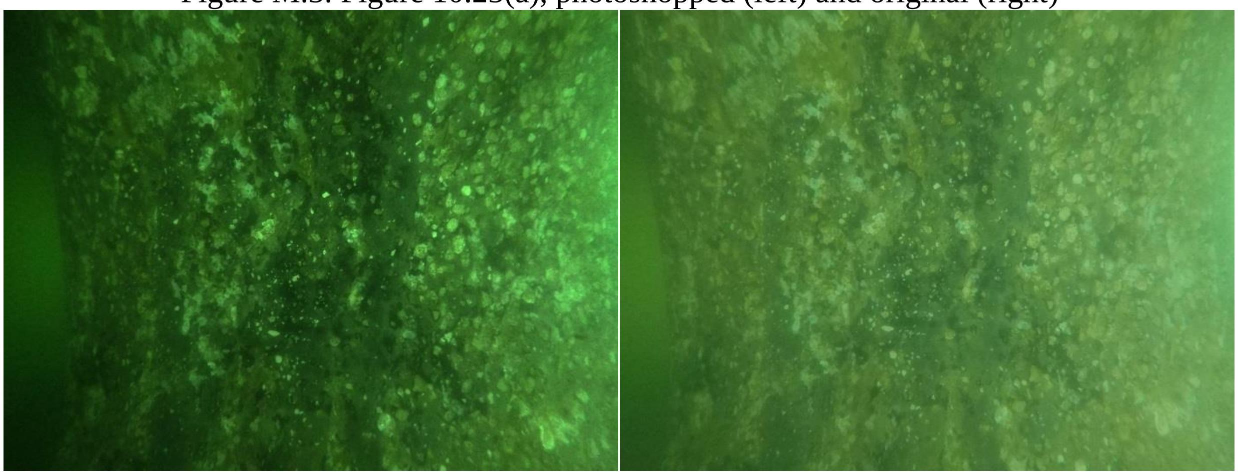


Figure M.4. Figure 10.23(b), photoshopped (left) and original (right)

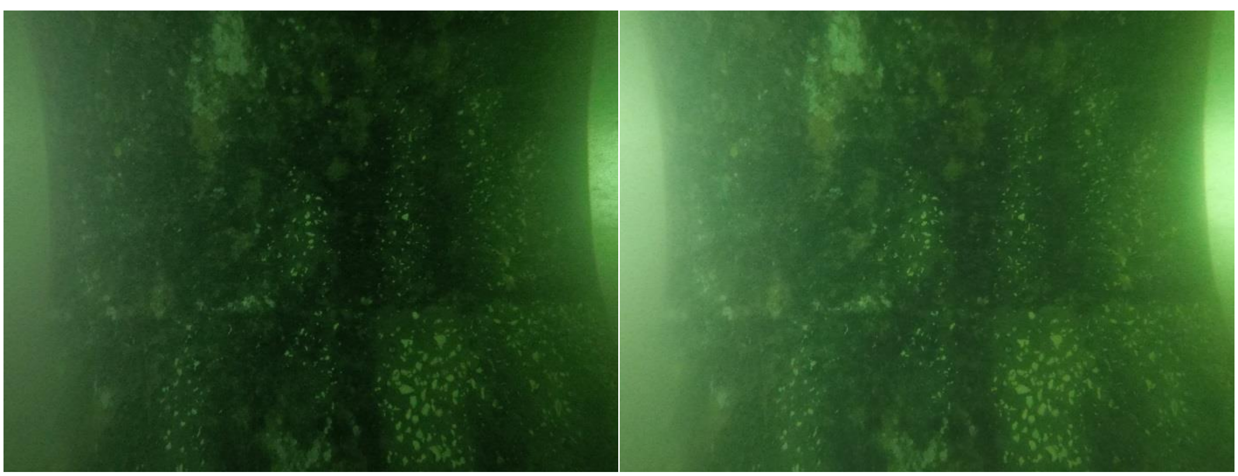


Figure M.5. Figure 10.24(a), photoshopped (left) and original (right)

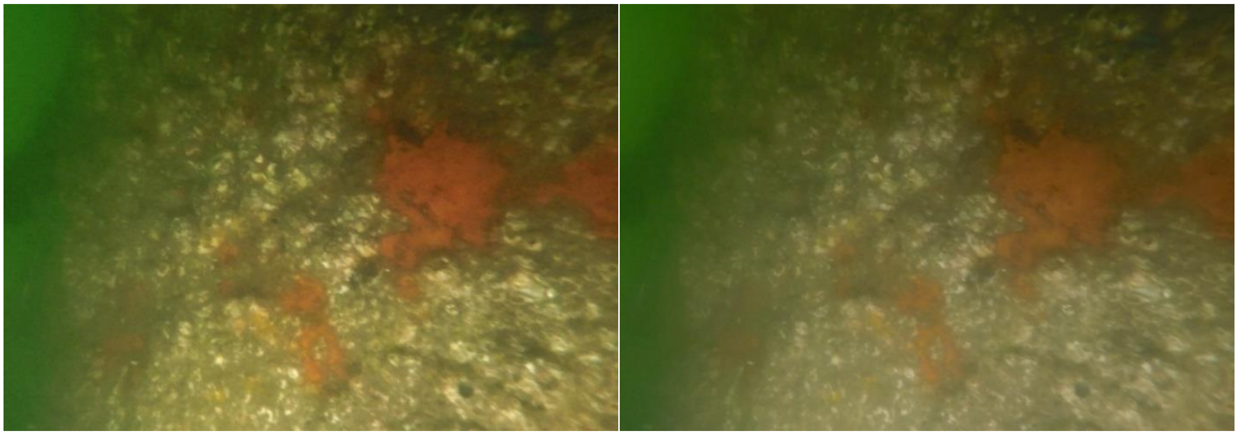


Figure M.6. Figure 10.27 (left image), photoshopped (left) and original (right)

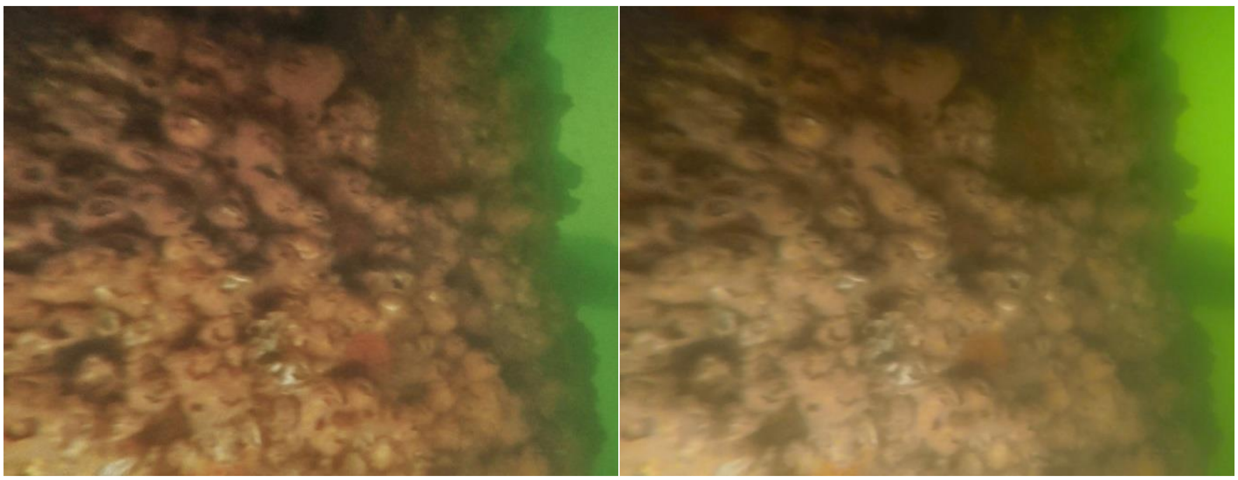


Figure M.7. Figure 10.27 (right image), photoshopped (left) and original (right)

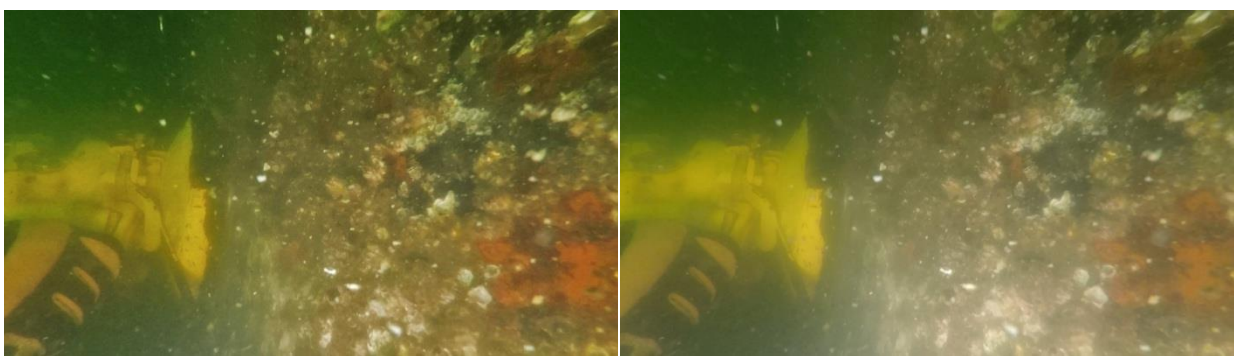


Figure M.8. Figure 10.28, photoshopped (left), original (right)

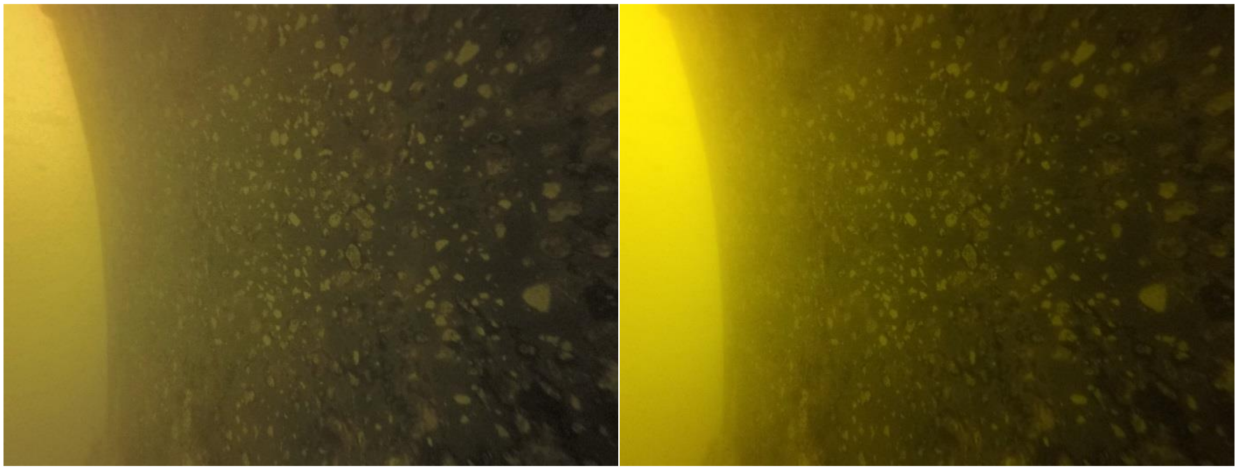


Figure M.9. Figure 10.30 (left image), photoshopped (left) and original (right)

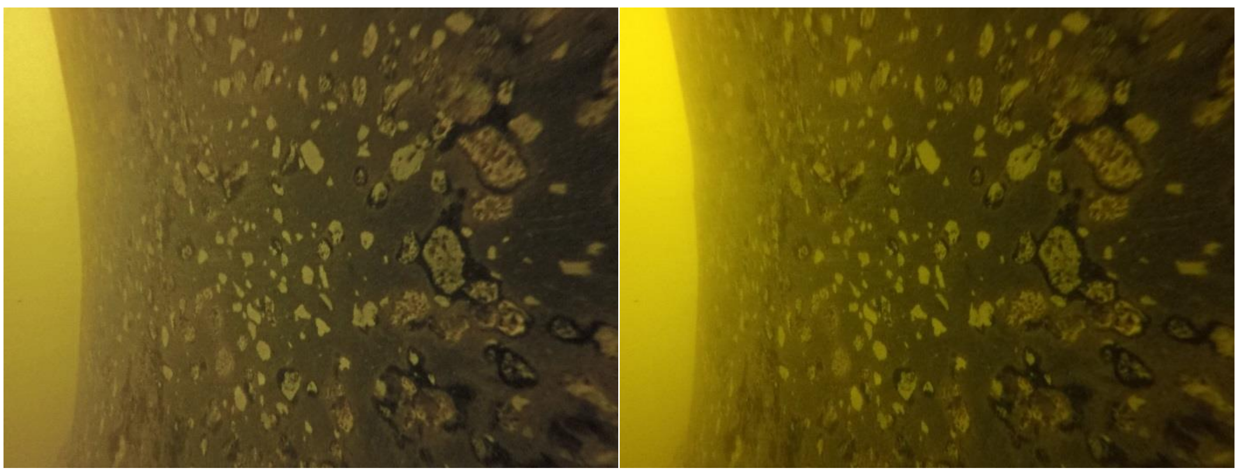


Figure M.10. Figure 10.30 (right image), photoshopped (left) and original (right)

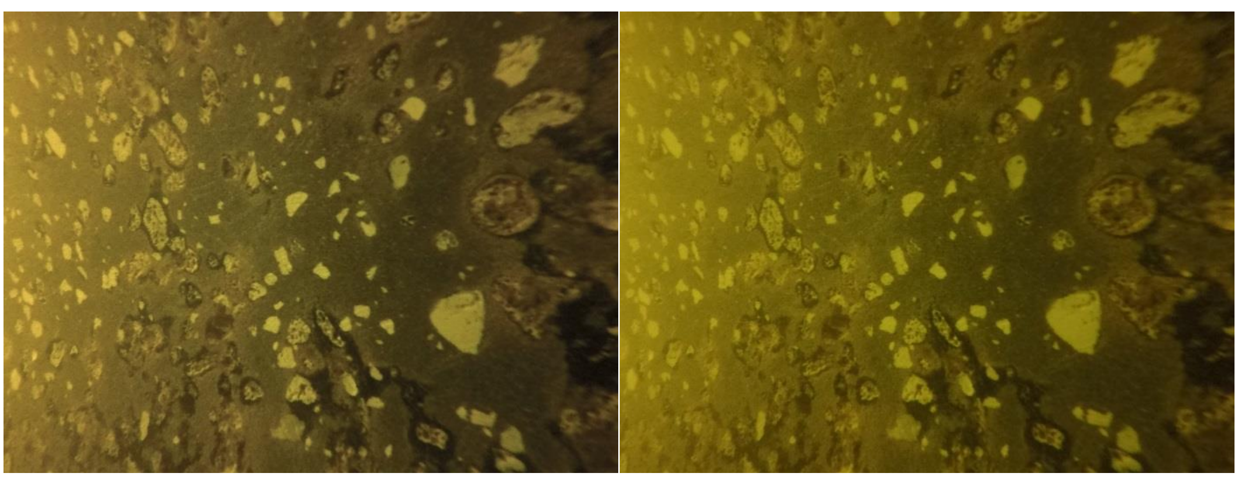


Figure M.11. Figure10.31 (left image), photoshopped (left) and original (right)

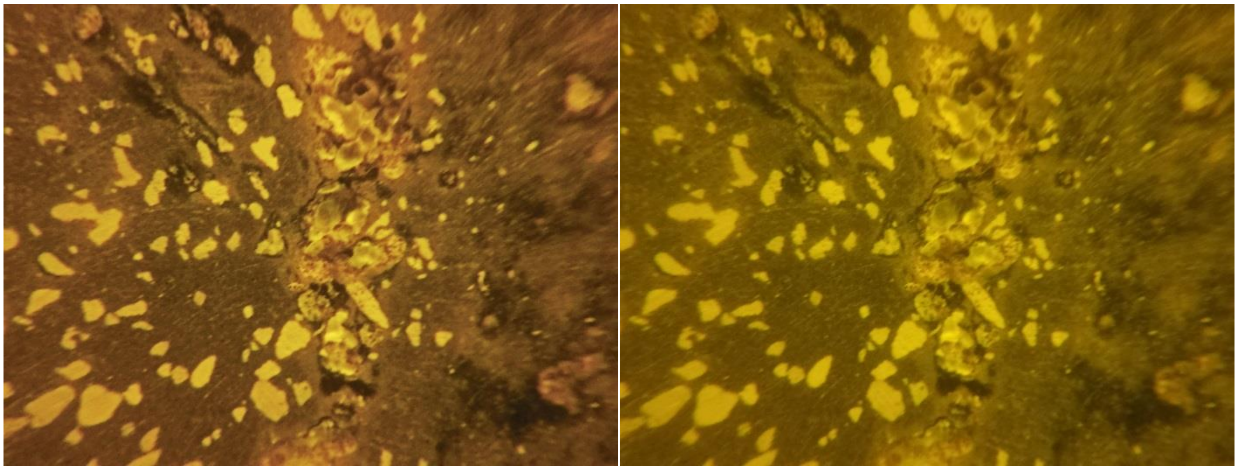


Figure M.12. Figure 10.31 (right image), photoshopped (left) and original (right)

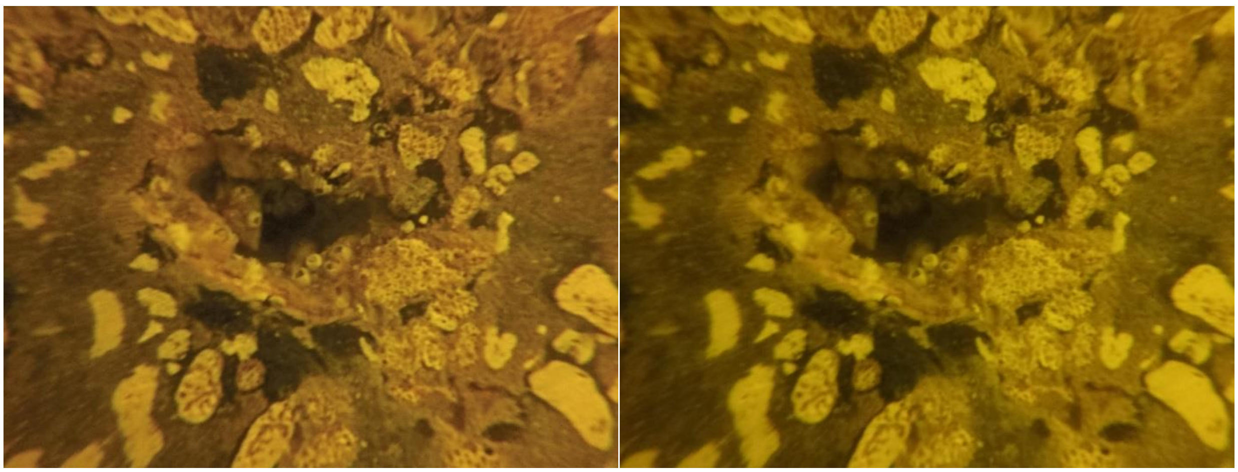


Figure M.13. Figure 10.32 (left image), photoshopped (left) and original (right)

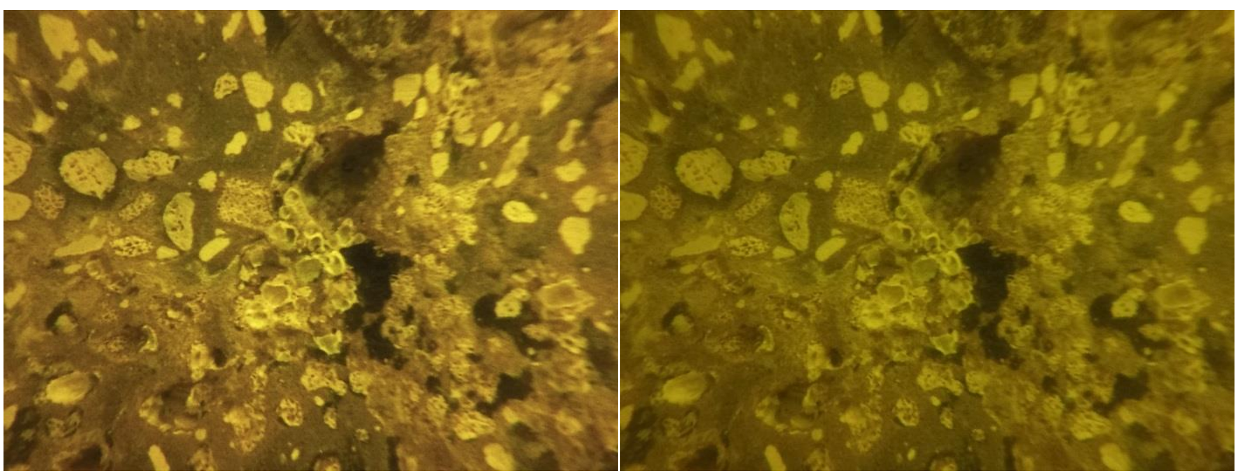


Figure M.14. Figure 10.32 (right image), photoshopped (left) and original (right)

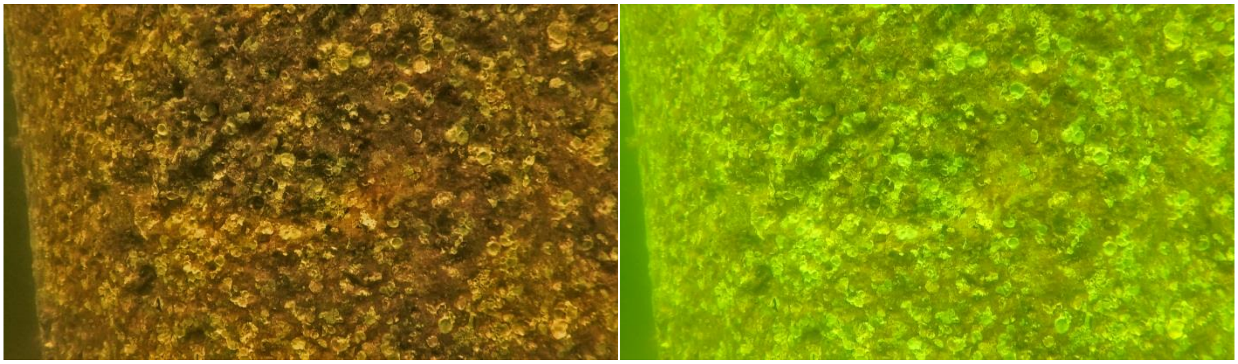


Figure M.15. Figure 10.33, photoshopped (left) and original (right)

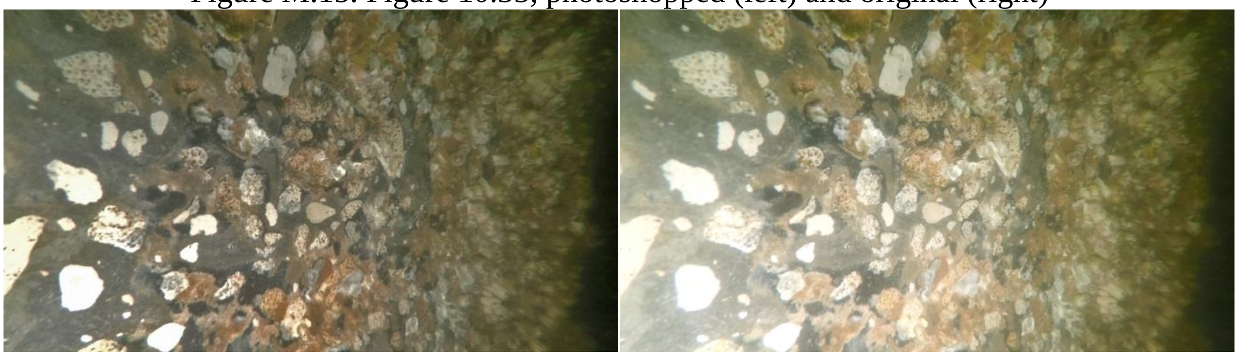


Figure M.16. Figure 10.34, photoshopped (left) and original (right)

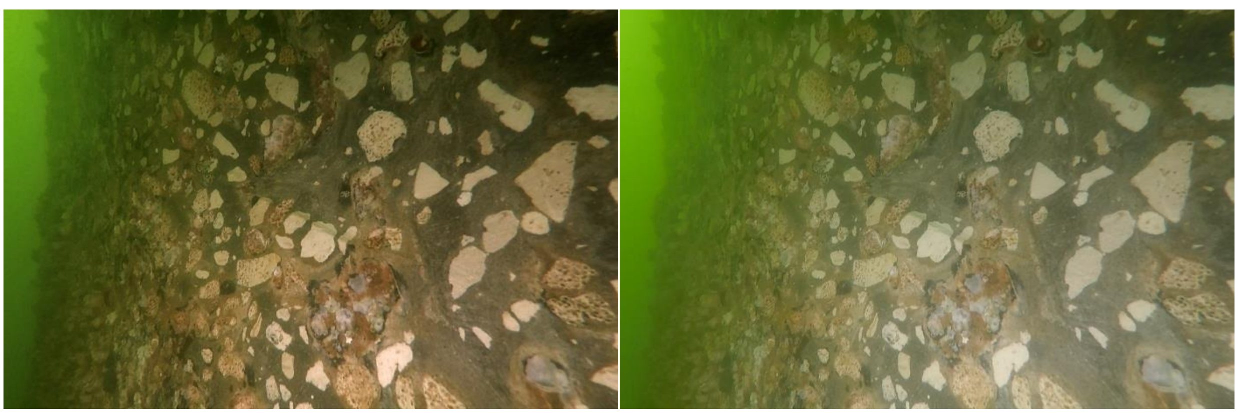


Figure M.17. Figure 10.35, photoshopped (left) and original (right)

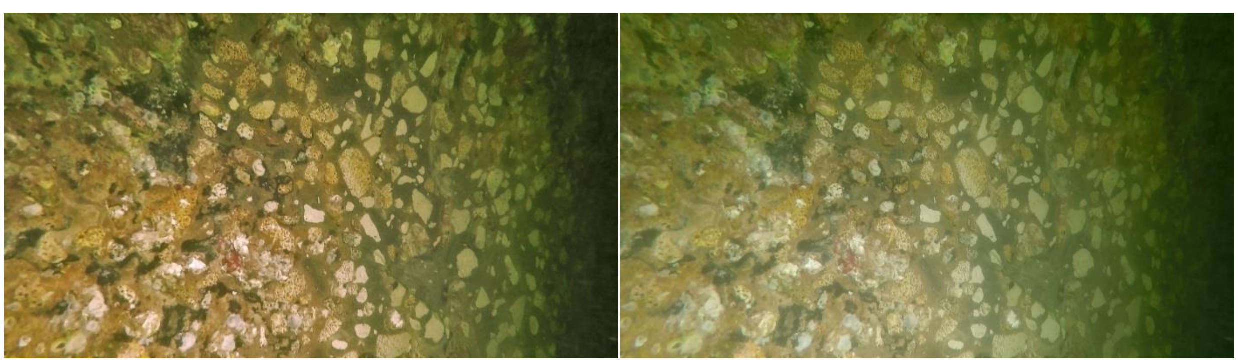


Figure M.18. Figure 10.36, photoshopped (left) and original (right)

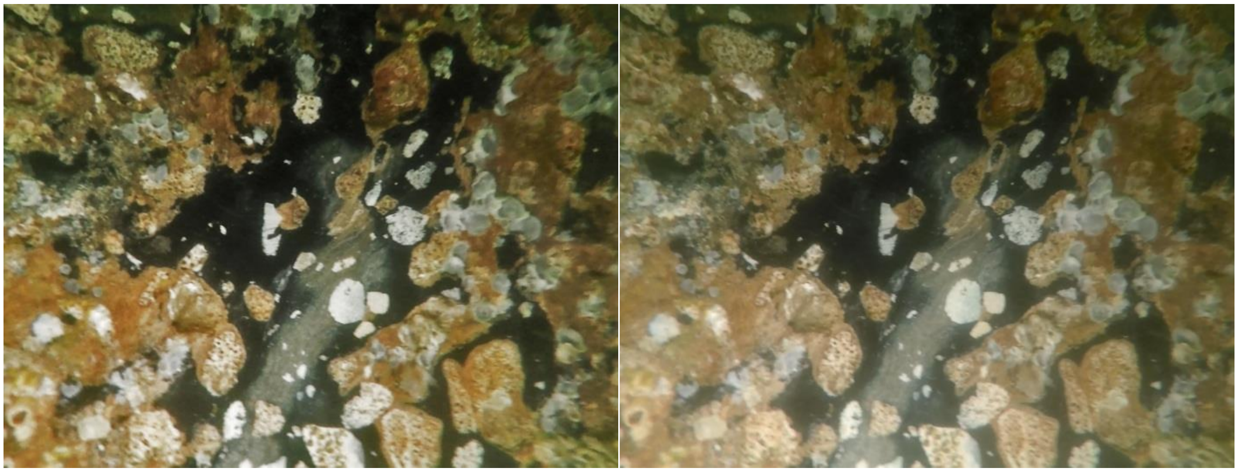


Figure M.19. Figure 10.37, photoshopped (left) and original (right)

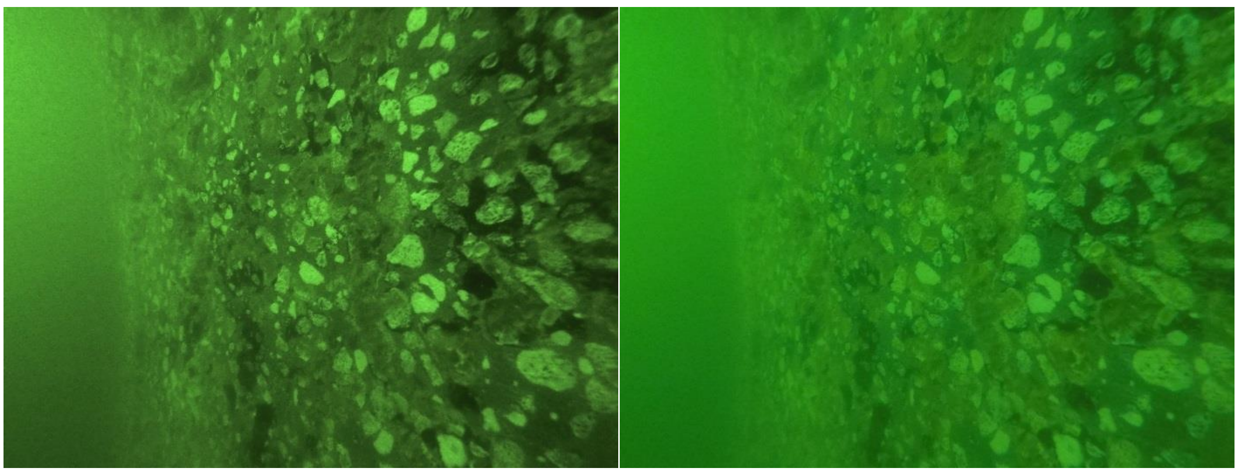


Figure M.20. Figure 10.38 (left image), photoshopped (left) and original (right)

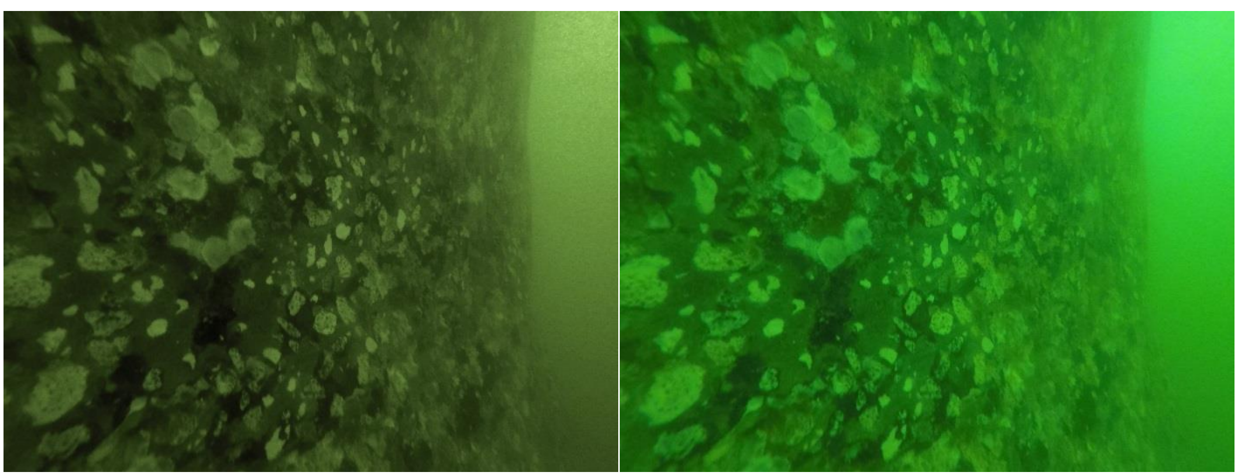


Figure M.21. Figure 10.38 (right image) , photoshopped (left) and original (right)

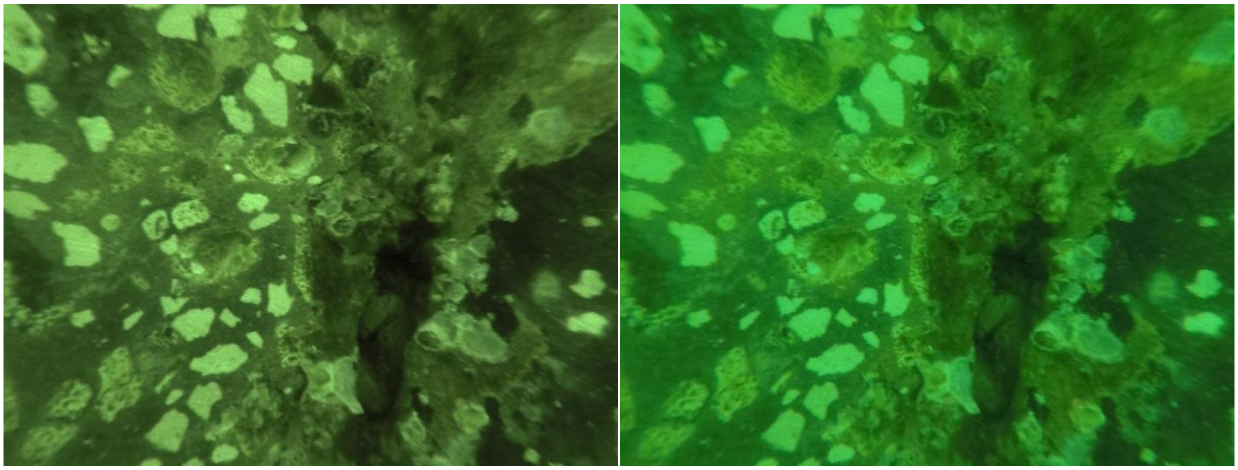


Figure M.22. Figure 10.39, photoshopped (left) and original (right)

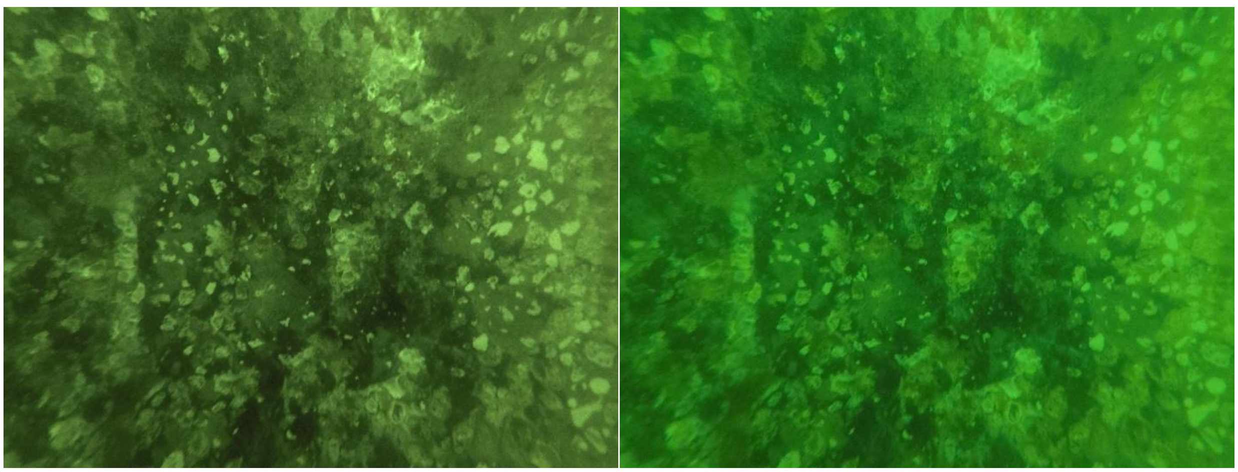


Figure M.23. Figure 10.40 (left image), photoshopped (left) and original (right)

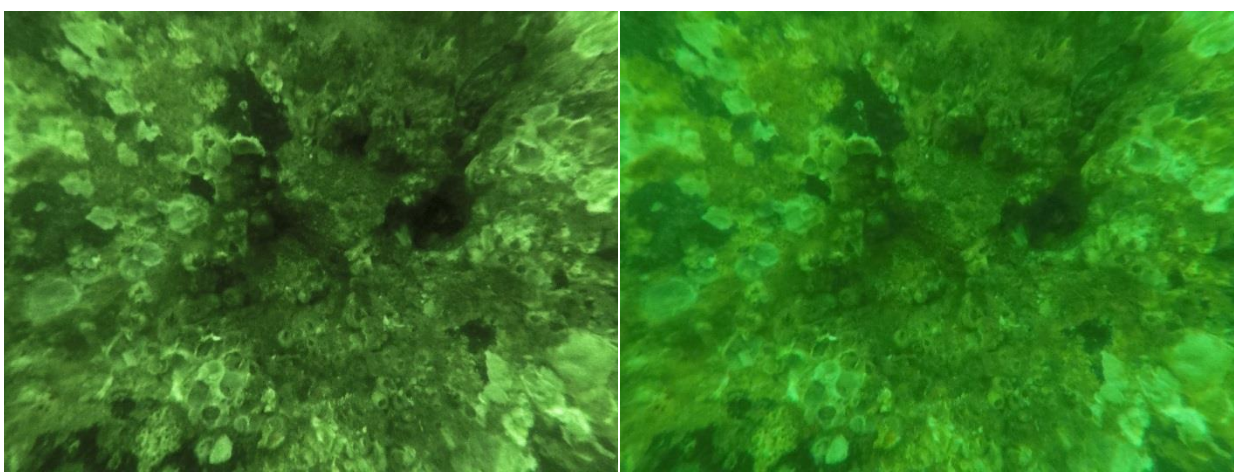


Figure M.24. Figure 10.40 (right image), photoshopped (left) and original (right)

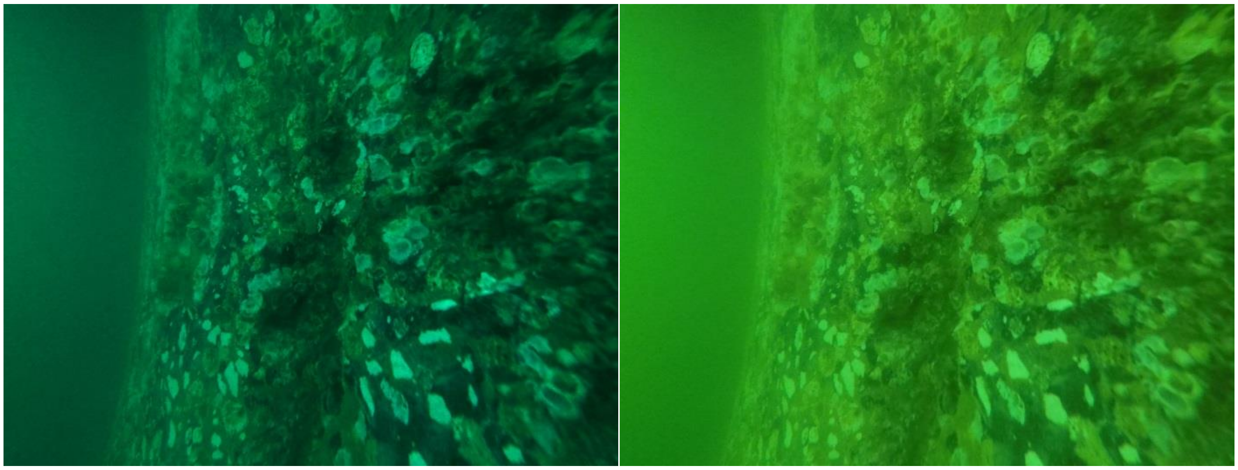


Figure M.25. Figure 10.41, photoshopped (left) and original (right)