

**ATTENUATING MASS CONCRETE EFFECTS IN DRILLED SHAFTS
BD-544-39**

FINAL REPORT

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Cover: Ringling causeway bridge pier founded on two 9ft diameter shafts (left); Demonstration shaft constructed during the study, 9ft diameter with 4ft diameter central void (right).

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				
in²	square inches	645.2	square millimeters	mm ²
ft²	square feet	0.093	square meters	m ²
yd²	square yard	0.836	square meters	m ²
ac	acres	0.405	hectares	ha
mi²	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
ft³	cubic feet	0.028	cubic meters	m ³
yd³	cubic yards	0.765	cubic meters	m ³
NOTE: volumes greater than 1000 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)				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C

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

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
m³	cubic meters	35.314	cubic feet	ft ³
m³	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/m²	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

*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 Drilled shafts are large diameter cast in place concrete foundation elements that until recently were not viewed with the same scrutiny as other massive concrete elements when considering mass concrete aspects. Upon closer review, even small diameter shafts (as small as 4 ft) develop mass concrete conditions which gave rise to this project. This study addressed three aspects of temperature generation in drilled shafts: (1) construction of drilled shaft with a full length centroidal void to minimize internal heat, (2) thermal integrity measurements of drilled shafts, and (3) field measurements of mass concrete elements taken to corroborate model findings with the intent of refining the definition of detrimental mass concrete conditions.			
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Executive Summary

Mass concrete elements are those concrete structures so massive in size that heat formation due to exothermic hydration reactions can induce cracking as a result of the excessive temperature differentials upon cooling. These conditions have been anticipated and dealt with on footings and in some cases pier columns and caps by supplying internal cooling systems (recirculation fluid lines in some cases outfitted with chillers). Until very recently, drilled shafts were not thought to exhibit mass concrete effects either due to their relative small diameters (4 ft diameter being the most common) and/or the perception that the surrounding environment was not conducive to producing mass concrete conditions. However, the larger diameter shafts (9 ft) used for the Ringling Causeway Bridge in Sarasota raised concerns that perhaps shafts had been slipping through the mass concrete specifications without review.

To address these concerns this study constructed a 9 ft diameter drilled shaft with a 4 ft diameter centrally located void to demonstrate that both the temperature could be controlled and that it was practically constructable. This demonstration showed model predictions were accurate, construction was reasonable, and that concrete savings could be realized without significant reduction in structural capacity. Further, the approach eliminated the need to mitigate high temperatures within the shaft requiring internal cooling lines / systems. However, voiding a shaft as demonstrated does not eliminate the possibility of mass concrete in more moderately sized shafts (e.g. 5 - 6 ft diameter). Therefore, use of less reactive concrete constituents or replacement with flyash was shown to be effective.

Modeling of all shaft sizes using varied shaft concrete mix designs was carried out and verified using thermal integrity testing. Several cases studies are documented by the study that solidify the calibration of the predictive models developed for mass concrete and shaft thermal integrity evaluation.

Table of Contents

List of Tables	x
List of Figures	xi
<i>Chapter One: Introduction</i>	1
1.1 Background Statement	1
1.2 Report Organization	3
<i>Chapter Two: Background</i>	5
2.1 Problem Statement	5
2.2 Mass Concrete	7
2.3 Case Studies	8
2.3.1 Hoover Dam	8
2.3.2 Ringling Causeway Bridge	9
2.3.3 I-35W Bridge Replacement Project	9
2.3.4 Clearwater Test Site	10
2.3.5 USF Nuclear Vault	10
<i>Chapter Three: Thermal Integrity Evaluation</i>	23
3.1 Site I: Voided Shaft	23
3.2 Site II: Bridge of Lions	24
3.3 Site III: I-4 Drilled Shafts	24
3.4 Site IV: Lake Okeechobee Dyke Remediation Project	25
3.5 Site V: Ocala Judicial Building Expansion	26
3.6 Site VI: UF 290 Site	30
<i>Chapter Four: Voided Shaft Construction</i>	73
4.1 Background of Concept	73
4.1.1 Construction Considerations	73
4.1.2 Strength Considerations	74
4.1.3 Curing Temperature Maintenance	75
4.1.4 Cost Effectiveness	75
4.2 Full Scale Demonstration Construction	75
4.2.1 Preparation	76
4.2.2 Excavation and Concreting	76
4.2.2.1 Excavation	77
4.2.2.2 Cage Placement	77
4.2.2.3 Central Casing (Full Length Void)	77
4.2.2.4 Concrete Placement	77
4.2.2.5 Surface Casing Removal	78
4.2.3 Long-Term Monitoring	78
4.3 Post Construction Testing	78
4.4 Results	79

<i>Chapter Five: Numerical Modeling</i>	93
5.1 Energy Production	93
5.2 Ground Temperature Distribution (Boundary Conditions).....	94
 <i>Chapter Six: Summary and Conclusions</i>	103
6.1 Recommendations for Thermal Evaluation	103
6.2 Sensitivity of Thermal Modeling for Thermal Integrity Evaluation.....	105
 <i>General References</i>	111
 <i>Appendix A: Concrete Mix Designs</i>	113
 <i>Appendix B: TSP for Thermal Evaluation</i>	127

List of Tables

Table 3-1 Shaft Testing Details.	51
Table 3-2 UF-290 Shaft Testing Details.	69
Table 6-1 Shaft Testing Details.	105

List of Figures

Figure 1-1. Mass concrete geometry criteria applied to various shaft diameters.	3
Figure 2-1. Hoover damn project (built circa 1935) used over 5 million yd ³ of concrete.	12
Figure 2-2. Ringling Causeway Bridge founded on 2- 9 ft diameter shafts at each pier. .	12
Figure 2-3. Temperature traces from the Ringling Causeway Bridge.	13
Figure 2-4. Modeled peak temperature traces for various shaft sizes, based on “Ringling” shaft mix design and conditions.	13
Figure 2-5. I-35W Bridge replacement southbound Pier 2.	14
Figure 2-6. I-35W Bridge Southbound Pier 2 Shaft 1 Thermocouple Data.	15
Figure 2-7. I-35W Bridge Southbound Pier 2 Shaft 2 Thermocouple Data.	16
Figure 2-8. I35W Pier 2 Southbound Footing Thermocouple Data.	17
Figure 2-9. I-35W Bridge Shaft 1 Thermal Data from Thermocouples and Thermistors.	18
Figure 2-10. I-35W Bridge Shaft 2 Thermal Data from Thermocouples and Thermistors...	18
Figure 2-11. Temperature traces for the 4ft (1.22m) Clearwater shaft (modeled and observed).	19
Figure 2-12 Peak and differential temperature model predictions for various-sized shafts using the “Clearwater” site conditions.	20
Figure 2-13. Temperature contours at mid-height of the nuclear vault walls.	20
Figure 2-14. Measured wall temperatures from mid-height of nuclear vault wall.	21
Figure 2-15. Although crack widths register as negligible, they run sporadically throughout wall.	21
Figure 3-1. Thermal testing of voided shaft.	32
Figure 3-2. Thermal field data analyzer front page.	32
Figure 3-3. Thermal field data analyzer individual test run worksheet.	33
Figure 3-4. Voided shaft thermal results for tube 1.	34
Figure 3-5. Voided shaft thermal results for tube 2.	35
Figure 3-6. Voided shaft thermal results for tube 3.	36
Figure 3-7. Voided shaft thermal results for tube 4.	37
Figure 3-8. Voided shaft thermal results for tube 5.	38
Figure 3-9. Voided shaft thermal results for tube 6.	39
Figure 3-10. Thermal Integrity Testing of Shafts 6-2 (left), 8-2 (center), and 25-3 (right).	40
Figure 3-11. Shaft 25-3 results: measured (left), excavation log (middle), preliminary model (right).	40
Figure 3-12. Concreting log for shaft 25-3 (red arrows shows concrete used to refill while casing was extracted)..	41
Figure 3-13. Shaft 25-3 results compared to 3-D modeled rendering showing where extra concrete finally came to reside.. . . .	42
Figure 3-14. I-4 & SR400 modeled temperature curve versus time.	43
Figure 3-15. I-4 & SR400 thermal testing results for Shaft 8-R.	43
Figure 3-16. I-4 & SR400 thermal testing results for Shaft 11-L.	44

Figure 3-17. Lake Okeechobee dyke modifications using TRD.	45
Figure 3-18. Lake Okeechobee HB150C Mix Design thermocouple data.	45
Figure 3-19. Lake Okeechobee HB1B Mix Design thermocouple data.	46
Figure 3-20. HB150C tube 1 at station 1413+25, 17 hours after mixing.	46
Figure 3-21. HB150C tube 2 at station 1413+33, 17 hours after mixing.	47
Figure 3-22. HB1B tube 3 at station 1414+36, 16 hours after mixing.	47
Figure 3-23. HB1B tube 4 at station 1414+44, 16 hours after mixing.	48
Figure 3-24 Normal access tube temperatures for shaft sizes tested.	49
Figure 3-25. Ocala Judicial Expansion Project Shaft 69 Thermal Test Results.	49
Figure 3-26 Signal match of Shaft DS-73 field results with modeled output, with and w/o neck.	50
Figure 3-27. DS-5 Measured temperature traces for Tubes 1 through 4.	52
Figure 3-28. DS-10 Measured temperature traces for Tubes 1 through 4 compared to the model norm.	53
Figure 3-29. DS-11 Measured temperature traces for Tubes 1 through 4 compared to the model norm.	54
Figure 3-30. DS-18 Measured temperature traces for Tubes 1 through 4 compared to the model norm.	55
Figure 3-31. DS-20 Measured temperature traces for Tubes 1 through 4 compared to the model norm.	56
Figure 3-32. DS-23 Measured temperature traces for Tubes 1 through 4 compared to the model norm.	57
Figure 3-33. DS-24 Measured temperature traces for Tubes 1 through 4 compared to the model norm.	58
Figure 3-34. DS-29 Measured temperature traces for Tubes 1 through 4 compared to the model norm.	59
Figure 3-35. DS-30 Measured temperature traces for Tubes 1 through 4 compared to the model norm.	60
Figure 3-36. DS-70 Measured temperature traces for Tubes 1 through 4 compared to the model norm.	61
Figure 3-37. DS-71 Measured temperature traces for Tubes 1 through 4 compared to the model norm.	62
Figure 3-38. DS-72 Measured temperature traces for Tubes 1 through 4 compared to the model norm.	63
Figure 3-39. DS-38 Measured temperature traces for Tubes 1 through 4 compared to the model norm.	64
Figure 3-40. Cage alignment in 54" excavation at a depth of 21 ft.	65
Figure 3-41. Cage alignment in 54" excavation at a depth of 48 ft.	65
Figure 3-42. Cage alignment in 54" excavation at a depth of 70 ft.	66
Figure 3-43. Inclinator results for shaft DS-38.	67
Figure 3-44. Inclinator results plotted in 3D relative to the excavation.	68
Figure 3-45. Normal access tube temperatures for the shafts tested.	69
Figure 3-46. Thermal results for Shaft 152.	70
Figure 3-47. Thermal results for Shaft 154.	70
Figure 3-48. Thermal results for Shaft 171.	71

Figure 4-1. Conceptual schematic of a voided drilled shaft, in profile view (left) and plan view (right).	80
Figure 4-2. Net hydrostatic pressure distribution during construction.	80
Figure 4-3. Rigid and flexible combination sealing flange is attached to the void casing, and engaged by the slurry and concrete load	81
Figure 4-4. Void (inner) casing and reinforcement cage piloting framework.	81
Figure 4-5. Interaction diagram of 9ft (2.75m) diameter voided and un-voided shafts. ...	82
Figure 4-6. Calculated temperatures for 9ft (2.75m) voided and un-voided drilled shafts in saturated sands, with summer installation (“Clearwater” case).	82
Figure 4-7. Cost savings per ft of void diameter from unused concrete, including the inner casing, which is assumed permanent.	83
Figure 4-8. Hottest summer conditions occur in September for both the air and soil.	84
Figure 4-9. Site location in Clearwater, Florida.	85
Figure 4-10. Reinforcement cage with monitoring tubes and thermocouples.	85
Figure 4-11. Installation of ground monitoring tubes by FDOT District I personnel.	86
Figure 4-12. Voided shaft center casing access tube supports.	86
Figure 4-13. Thermocouples and access tube Installed in center casing.	87
Figure 4-14. Drilling and clean-out of 9ft diameter excavation.	87
Figure 4-15. Lifting reinforcement cage.	88
Figure 4-16. Placement of reinforcement cage with 12in diameter wheel spacers.	88
Figure 4-17. Lifting central casing.	89
Figure 4-18. Placing and securing central casing.	89
Figure 4-19. Concrete placement using two tremies.	90
Figure 4-20. Temporary casing extraction.	90
Figure 4-21. Finish shaft with data acquisition system in place.	91
Figure 4-22. Realtime thermocouple data as posted on USF webpage.	91
Figure 4-23 On-line battery voltage monitoring to assure remote system remained viable throughout the monitoring period.	92
Figure 4-24 Annotated data from the voided shaft and surrounding vicinity.	92
Figure 5-1. Heat source calculator used to input new mix design parameters.	96
Figure 5-2. Expected access tube temperatures for various sizes of shaft and testing times.	97
Figure 5-3. Soil temperature profile at Clearwater test site.	98
Figure 5-4. Average daily temperature for the Tampa / St. Petersburg, FL area.	99
Figure 5-5. Average weekly temperature variations over one year period (Clearwater, FL)	99
Figure 5-6a. Modeled ground temperature profile (Feb to May).	100
Figure 5-6b. Modeled ground temperature profile (June to October).	100
Figure 5-6c. Modeled ground temperature profile (November to January).	101
Figure 6-1. Economical hand-held infrared thermometer.	106
Figure 6-2. Example shaft capacity usage as a function of position in shaft.	106
Figure 6-3. Normal integrity tube temperatures as a function of shaft diameter and hydration time.	107
Figure 6-4. Recommended testing timeframes for various shaft sizes.	107
Figure 6-5. Model response for cover loss for various size shafts.	108
Figure 6-6. Effects of dimensions on temperature for various shaft sizes.	109

Figure A-1. Ringling Causeway Mix Design.	114
Figure A-2 Concrete mix design for voided shaft.	115
Figure A-2b Concrete Truck Ticket for Voided Shaft Demonstration.	116
Figure A-3 Mix design for the I35W drilled shafts.	117
Figure A.4a Mix design for USF nuclear vault project.	118
Figure A.4b Mix design for USF nuclear vault project (continued).	119
Figure A.4c Concrete test results for USF nuclear vault project.	120
Figure A-5a. UF-290 Project Mix Design.	121
Figure A-5b. UF-290 Project Mix Design (continued).	122
Figure A-5c. UF-290 Project Mix Design (continued).	123
Figure A-6. Concrete mix design for Marion County Judicial Center.	124
Figure A-7. Lake Okeechobee Mix Design.	125

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Chapter One: Introduction

1.1 Background Statement

Florida's tremendous population growth has forced roadways and the associated highway structures to be constantly upgraded to maintain a reasonable level of service. As a result, structurally sound bridges have become functionally obsolete prior to their projected usable life span, which has required their replacement. This growth coupled with more stringent design criteria to accommodate extreme event design states has led to larger and larger sub-structural bridge elements that require scrutiny with regards to mass concrete issues.

Historically, mass concrete elements were those concrete structures so massive in size that heat formation due to exothermic hydration reactions would induce cracking as a result of the excessive temperature differentials upon cooling. These conditions were anticipated and dealt with on footings and, in some cases, pier columns and caps by supplying internal cooling systems (recirculation fluid lines in some cases outfitted with chillers). Until very recently, drilled shafts were not thought to exhibit mass concrete effects either due to their relative small diameters (4 ft diameter being the most common) and/or the perception that the surrounding environment was not conducive to producing mass concrete conditions. However, the larger diameter shafts (9 ft) used for the Ringling Causeway Bridge in Sarasota raised concerns that perhaps shafts had been slipping through the mass concrete specifications without review. In the absence of quantifiable values for mass concrete effects in shafts, the FDOT assigned a size of 6 ft diameter (based on past experiences with Ringling Bridge shaft sizes) to delineate when mass concrete specifications should be imposed until more information could be obtained from this study.

The traditional approach to ascertain mass concrete was to evaluate the volume to area ratio limit from the Structures Design Guideline (SDG) 3.9, which stated that any concrete element with volume in ft^3 greater than the dissipative surface area in ft^2 would likely be unable to stay within reasonable temperature limits. Further, if the minimum dimension of a concrete element was 3 ft or greater, the same lack of temperature control could be anticipated. When applying more performance specific restrictions to such elements, the differential temperature was limited to 35 deg F regardless of dimensions. Using this simple formula, the shaft cut off diameter should have been a bit more restrictive limiting it to 4 ft diameter shafts as shown in Figure 1-1.

A second issue plaguing State Materials Engineers deals with delayed intrinigte formation (DEF) in mass concrete of all kinds in which drilled shafts are now included. DEF arises when the peak concrete temperature exceeds values in the range of 60 deg C.

Shafts as small as 2 ft in diameter have been shown to exhibit mass concrete conditions (either differential or peak temperature limits) under certain circumstances (Johnson,

2007). This somewhat startling finding has led to an innovative construction process whereby mass concrete conditions can be averted by casting shafts with a full length centralized void (cast in place cylinder pile of sorts).

This project focused on new field temperature measurement equipment to measure the full length temperature profile of large diameter drilled shafts, modeling of concrete elements of all sizes to predict spatial temperature variations throughout, providing more sophisticated ways of predicting mass concrete conditions of civil engineering structures, and assessing the effectiveness of new construction methods to mitigate or at least drastically reduce mass concrete effects in large-diameter drilled shafts. Finally, the true determination of whether or not a particular concrete element should be considered “mass,” can now be based on the presence of cracking conditions (strength based) in lieu of simple temperature differentials (gradient based). The latter comes in the wake of extensive advances in the T3DModel developed by USF for FDOT in an earlier study (Mullins, et al, 2006).

1.2 Report Organization

The overall organization of this report is outlined below wherein four chapters identify the problem, the modeling approach, the results of the modeling, and recommendations for the useful application of the study findings.

Chapter 2 introduces the original problem as outlined in the USF proposal submitted to the FDOT. A background complete with case studies dealing with mass concrete is provided along with the fundamentals associated with adequate thermal model and field measurements of these systems.

Field temperature measurements and construction logs obtained from numerous thermal integrity test sites are detailed in Chapter 3. Advances to the T3DModel are discussed as it pertains to shaft integrity testing using thermal imaging.

Chapter 4 details the demonstration construction project of a 9ft diameter drilled shaft constructed with a full length 4ft diameter void. The emphasis therein was to verify that both temperature could be controlled and the construction process was feasible.

Advances in the modeling capabilities for mass concrete and shaft integrity evaluation are presented in Chapter 5. This includes mechanisms for predicting crack-induced distress and evaluation of thermal integrity data.

Finally, Chapter 6 presents a summary and conclusions that are aimed at providing usable specifications for identifying mass concrete and procedures for proper thermal integrity evaluation.

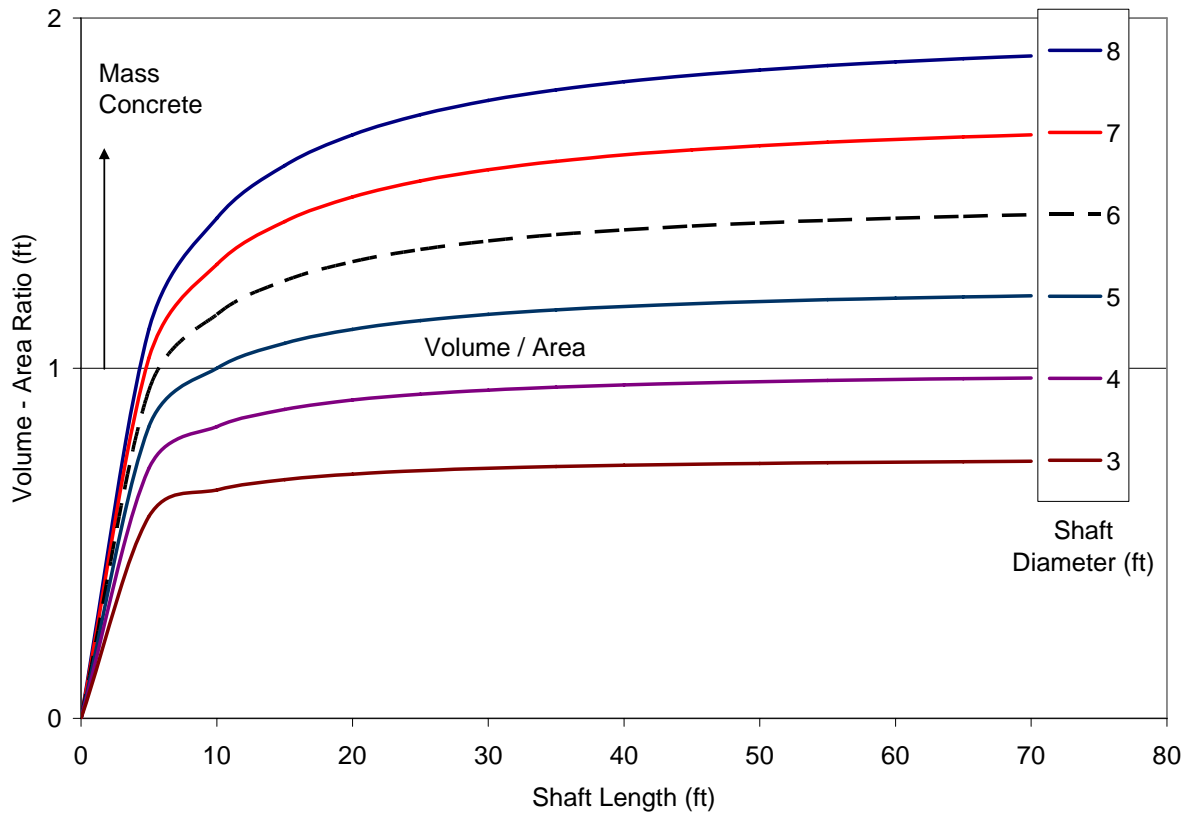


Figure 1-1. Mass concrete geometry criteria applied to various shaft diameters.

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Chapter Two: Background

This chapter provides an overview of the concept of mass concrete and the somewhat diverse but related issues in better understanding, predicting, and obtaining field measurement of mass concrete conditions. Additionally, the project paid special attention to large diameter drilled shafts and how to minimize the adverse effects therein. For completeness, a description of the project problem statement has been provided.

2.1 Problem Statement

This project stemmed from a Request for Research Proposal (RFRP) defined by FDOT wherein the following proposal problem statement was identified:

The proposed research will undertake: (1) field temperature measurements of mass concrete structures with specific interest on drilled shafts (but not limited to) using the new thermal integrity probe, (2) numerical thermal modeling to verify the anticipated temperature response within a drilled shaft or mass concrete structure, (3) construction of a large diameter shaft with a central void to assess the constructability and temperature mitigation potential, and (4) development of a concrete maturation-dependent stress analysis algorithm to assess differential temperature-induced concrete cracking potential.

Task 1 Field Data Collection

After several generations of thermal integrity probes, a new probe has been developed that promises to be more robust both in downhole components and the data collection system. As on-going large diameter shaft construction is presently underway (e.g. St. Augustine Bridge of Lions) the new system will be employed to collect temperature profiles at various sites across the state. This information will be coupled with modeling results to develop a more diverse materials library in the State's new 3-D thermal software (yet unnamed).

Task 2 Thermal Software Review

The 3-D thermal software recently developed for FDOT by USF is capable of predicting the precise temperature traces for any location within a mass concrete element cast in a wide range of conditions. Therein, shafts, footings, columns, pier caps, etc. can be modeled for single pours and staged pours in various soils, water, or air (including diurnal) environments. As it is currently the alpha version, this task is aimed at working closely with the FDOT SMO to review the software and

edit to meet the needs of the State. Further, as a concurrent University of Florida project matures and begins to produce information for the software's cement mix input library, data received from the UF research team will be incorporated into the software. The ongoing large block specimen test results will be used to test the model's output. Based on the present UF timeline, it is envisioned that this portion of Task 2 will begin around the end of the first year of this project's two year timeline.

Task 3 Voided Shaft Construction

This task will involve the construction of a large diameter shaft with a centralized full-length void. The test shaft is envisioned to be on the order of 9 ft diameter with a 4 ft diameter central void and 25 ft in length. Numerous sub tasks are anticipated involving central casing preparation, cage preparation complete with instrumentation, construction equipment reviews, procedural protocol development, excavation and concreting, and finally extended monitoring of temperature and quality assurance. Initial dialog with State engineers and shaft contractors have outlined basic concerns that have already been addressed, but it is anticipated that numerous other issues are likely to arise that will be summarized in the procedural/construction protocol for casting void shafts.

Task 4 Stress Analysis Software

The undesirable tensile cracks that develop in mass concrete as a result of differential temperature are to-date controlled by a differential temperature limit specification (less than 35 deg F). This specification is only employed when the geometry of the element (vol/area ratio > 1ft; Fig. 1) dictates a Technical Special Provision. In reality, numerous factors play into whether or not concrete will crack under excessive temperature differentials. Specifically, the modulus or strength versus time relationship coupled with the heat generation versus time trace. The tensile strength in conventional structural softwares do not incorporate the early time / strength characteristics of maturing concrete which is crucial to predicting the crack potential in mass concrete elements.

As concrete strengthens with time its ability to withstand differential stresses increases. However, when differential stresses are developing at higher rates than the rising concrete strength, tensile cracks can initiate and the associated stress concentrations propagate those cracks almost unabated throughout the element. This gave rise to the present mass concrete specifications.

The presence of reinforcing steel within the concrete or a casing around a shaft can greatly reduce crack potential. This is evidenced by above

ground shaft casing removal and the infrequent occurrence of differential cracking (not that other undesirables are not found). Further, concrete mix designs continue to evolve whereby more moderate energy development rates can be attained.

Many of the parameters affecting cracking potential are being studied presently by UF. These outcomes coupled with the 3-D thermal modeling software output will be incorporated into a stress analysis package to be developed especially for mass concrete in this Task. Such a package will enable the State Materials Engineers to evaluate the types of mixes and geometries that will and will not crack in a more discerning manner.

2.2 Mass Concrete

Mass concrete is generally considered to be any concrete element that develops differential temperatures between the innermost core and the outer surface that in turn can develop tension cracks. Mass concrete is defined by the American Concrete Institute (ACI) as follows:

Any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change to minimize cracking.

Until recently, geometric guidelines involving the dimensions of a concrete element that defined the volume and surface area were used to predict the possibility of mass concrete related effects. These guidelines simply stated that the volume to surface area ratio (ft^3/ft^2) should not exceed a unity value nor should any minimum dimension be greater than 3ft. Presently, performance-oriented standards were adopted wherein the differential temperature between the highest and lowest temperatures within a concrete element at any given time could not exceed 35 deg F (20 deg C).

Some state DOTs have defined geometric guidelines that identify potential mass concrete conditions as well as limits on the differential temperature experienced. For instance, the Florida DOT designated any concrete element with minimum dimension exceeding 3 ft or a volume to surface area ratio greater than $1 \text{ ft}^3/\text{ft}^2$ will require precautionary measures to control temperature-induced cracking (FDOT, 2006). The same specifications set the maximum differential temperature to be 35 deg F (20 deg C) to control the potential for cracking. For drilled shafts, however, any element with diameter greater than 6ft is considered a mass concrete element despite the relatively high volume to area ratio mentioned in Chapter 1 (Figure 1-1).

The latter of the two integrity issues, *i.e.*, excess high temperature, is presently under investigation at a number of institutions. When concrete temperature exceeds safe limits on the order of 65 deg C (150 deg F), the concrete may not cure correctly and can ultimately degrade via latent expansive reactions termed delayed ettringite formation (DEF). This reaction may lay dormant for several years before occurring; or the expansion may not occur as it depends on numerous variables involving the concrete constituent properties and environment. Concrete mixes with low pozzolans have lower threshold temperatures whereas higher pozzolan content concretes may not exhibit adverse effects up to 85 deg C (185 deg F). At present, a definitive upper temperature limit is not available (Whitfield, 2006). What is known is that staying below 65 deg C appears to prevent temperature-related long-term detrimental effects.

2.3 Case Studies

Numerous case studies have documented the effects of hydrating cement in mass concrete structures. Although typically associated with enormous dams holding back multi-hectare reservoirs or footings coupling dozens of piles under a bridge pier, mass concrete is now understood to be a state and not necessarily a size. Therein, drilled shafts as small as 4ft diameter have been cited to have exhibited mass concrete conditions, much to the surprise of many designers (Mullins, et al, 2007). To this end, several case studies are presented in the ensuing sections dealing with all of the above types of concrete elements.

2.3.1 Hoover Dam

The most famous of the many cited mass concrete projects is perhaps the Hoover Dam project (Figure 2-1) constructed during the depression from 1932 to 1935 where over 5 million cubic yards of concrete were used. At that time it was understood that staged construction and a cooling system would be required to help control elevated temperatures. The primary concern was concrete cracking from differential temperature and the associated leakage. Without these considerations, temperature dissipation is estimated to have taken over 100 years and temperature-induced cracking would have severely compromised its structural integrity and its ability to prevent fluid ingress (DOI, 2004). As a result, over 500 miles of 1 inch diameter steel cooling tubes were plumbed throughout the structure through which chilled water was used transport the internal heat energy to the outside environment. From a more contemporary view point, preventing microcracking and fluid ingress are important to warding off DEF (Collepardi, 2003) and sulfate attack (Stark and Bollman, 1998).

2.3.2 Ringling Causeway Bridge

A relatively recent structure (built 2001-2003) which shed light on drilled shafts as mass concrete was the Ringling Causeway Bridge spanning across Sarasota Bay, Florida (Figure 2-2). This segmental, post-tensioned concrete box girder bridge is supported by

single column piers founded on two 2.75m (9ft) diameter drilled shafts. The concrete mix consisted of a Type I/II cement with a 20% by weight replacement of Class F fly ash. In spite of relatively cool weather (for Florida) and that it was bathed in cool bay water, the core temperature of the shafts reached 69 deg C (157 deg F). Figure 2-3 shows temperature traces over a 9 day period for the shaft center and edge as well as in the surrounding water and air. The bay water temperature averaged 17 deg C (63 deg F) while the air temperature dipped as low as 3 deg C (37 deg F). The cooler environment exacerbated differential temperature conditions wherein a maximum differential temperature of 37 deg C (67 deg F) was recorded shortly after the coldest spell.

Information gathered from this site was used to calibrate a 3-D numerical model from which predictions of how other sized shafts would have fared under these conditions. Figure 2-4 shows predictions wherein only the peak core temperatures are considered and that smaller sized shaft would not have exceeded safe peak temperature limits.

2.3.3 I-35W Bridge Replacement Project

In August of 2007 the I-35W bridge over the Mississippi River collapsed killing 13 and injuring hundreds. This catastrophe led to a truly amazing cooperative effort among local, state, and federal transportation authorities whereby an up-to-date replacement bridge could be put back in service in just over a year from the date of the collapse. Although only a small portion of a much larger quality assurance program, the project provided for mass concrete evaluation in the form of a low energy mix design and temperature measurements of selected concrete elements. This included two aspects for this study: (1) temperature monitoring of drilled shafts and (2) temperature monitoring of one of the footings. Figure 2-5 shows one of the footings (40ft x 90ft x 12ft) prior to casting during the formwork installation.

Figures 2-6 and 2-7 show the thermocouple data from shafts 1 and 2, respectively (Figure 2-5). Whereas shaft 2 (cast first) showed no mass concrete effects, shaft 1 measured a differential temperature slightly over 40 deg F (local state limit) near the top in spite of being identical in all other areas of the shaft. The reason was never concluded, but the USF researcher installing the central thermocouple immediately after the pour was convinced that the last truck was not the same SCC mix previously poured for shaft 2. This was based on the difficulty experienced in plunging the gage into the upper shaft concrete. Figure 2-8 shows the measured internal temperatures at various locations within the Pier 2 footing. The differential temperatures measured also exceeded the 40 deg F limit when comparing the bottom middle gage on the ground (F2-BM) with the Center gage positions directly between two cooling tubes (F3-CCT). Nevertheless, no distress was noted after removing the formwork.

Long term measurement were also made possible by way of the thermistors incorporated into the vibrating wire instrumentation throughout the shaft. Figures 2-9 and 2-10 show the thermocouple data prior to footing concrete placement and the thermistor data after footing concrete placement. The gap shown in the data accounts for the time when no

data could be collected due to interference with construction activities. Interestingly, the upper levels of the shafts show heating caused by the presence of the massive footing above.

2.3.4 Clearwater Test Site

The large diameter shafts used in the Ringling project were predicted to behave as mass concrete. However, more commonly used shaft sizes like 4ft (1.22m) shafts have been installed without concern for years. A recent study in Clearwater, Florida was conducted whereby a 25ft (7.6m) deep, 4ft (1.22m) diameter shaft was cast in saturated sandy to silty sandy soil complete with thermocouple and strain gage instrumentation. The full scope of the study involved anomaly detection, shaft integrity test method evaluations, and temperature development in a commonly used shaft size. Only the peak and differential temperature results are presented herein. Full details can be found elsewhere (Mullins et al., 2007).

Using the measured temperature response, the same numerical model was applied to this site's conditions which were significantly different from the Ringling site. Specifically, the ambient temperature was much higher, the surrounding heat diffusing materials were non-convective (no flowing water), and the mix design contained little to no pozzolans. Figure 2-11 shows the measured and modeled results superimposed with no recognizable differences aside from the diurnal temperature fluctuations on one thermocouple channel. A peak temperature of 84 deg C (183 deg F) was observed with a maximum differential temperature of 32 deg C (57 deg F). These exceeded both recommended limits for peak and differential temperatures.

Using the same Clearwater conditions, the numerical model was extended to simulated smaller shaft sizes (Figure 2-12). The results showed that even the smallest of constructible shafts (2ft) would exhibit mass concrete under these highly adverse conditions under at least one of the temperature criterion.

2.3.5 USF Nuclear Vault

As part of the new structure being built near the USF campus for medical applications involving nuclear radiation (testing systems such as X-Ray, Cat scans, etc.), an opportunity arose that permitted the study to instrument and monitor additional massive concrete elements.

The mix design slated for the structure called for no flyash or slag replacement and 775 lbs cement per cubic yard resulting in concrete energy production of 85.7 kJ/kg (kg of total concrete weight). This is higher than most mixes especially when compared to the somewhat lower strength requirements (designed for high early strength). This coupled with 10ft thick walls definitely was predicted to produce mass concrete conditions. The needs of the project, however, did not require crack-free concrete and additional

temperature steel was incorporated as well. The rationale was that X-rays follow lines of site pathways and the resultant cracks would not provide such a pathway.

Model predictions were prepared which are shown in Figure 2-13 as temperature contours from a horizontal slice through the wall at mid-height. The peak temperature of 182 deg F (83 deg C) and an edge to center differential of 62 deg F (34 deg C) were corroborated by field measurements taken at the mid-height of the wall from the inside face, outside face, and center of the wall (Figure 2-14). Upon form removal, a crack survey showed a sporadic distribution of small width cracks shown in Figure 2-15. Given the structure will not be exposed to marine conditions or possible soil sulfate attack, cosmetic sealing/waterproofing and painting were all that was required.



Figure 2-1. Hoover damn project (built circa 1935) used over 5 million yd³ of concrete.



Figure 2-2. Ringling Causeway Bridge founded on 2- 9 ft diameter shafts at each pier.

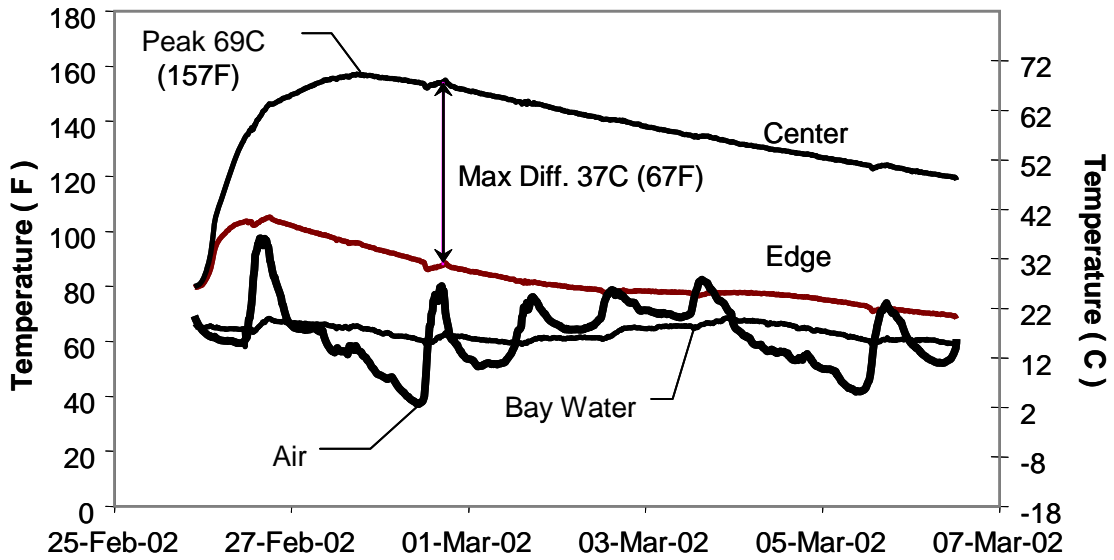


Figure 2-3. Temperature traces from the Ringling Causeway Bridge.

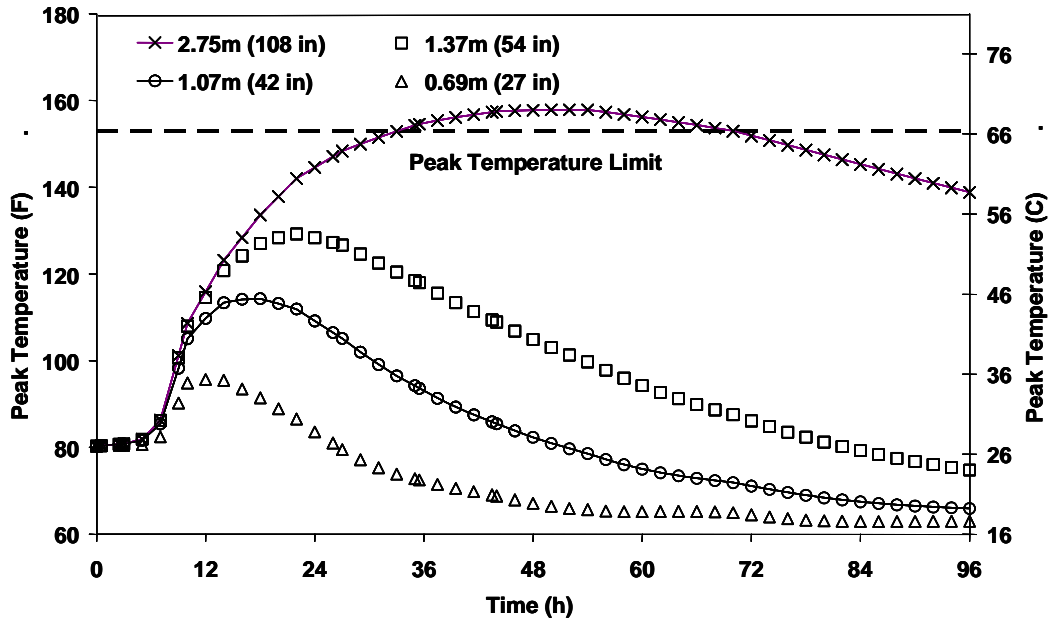


Figure 2-4. Modeled peak temperature traces for various shaft sizes, based on “Ringling” shaft mix design and conditions.



Figure 2-5. I-35W Bridge replacement southbound Pier 2.

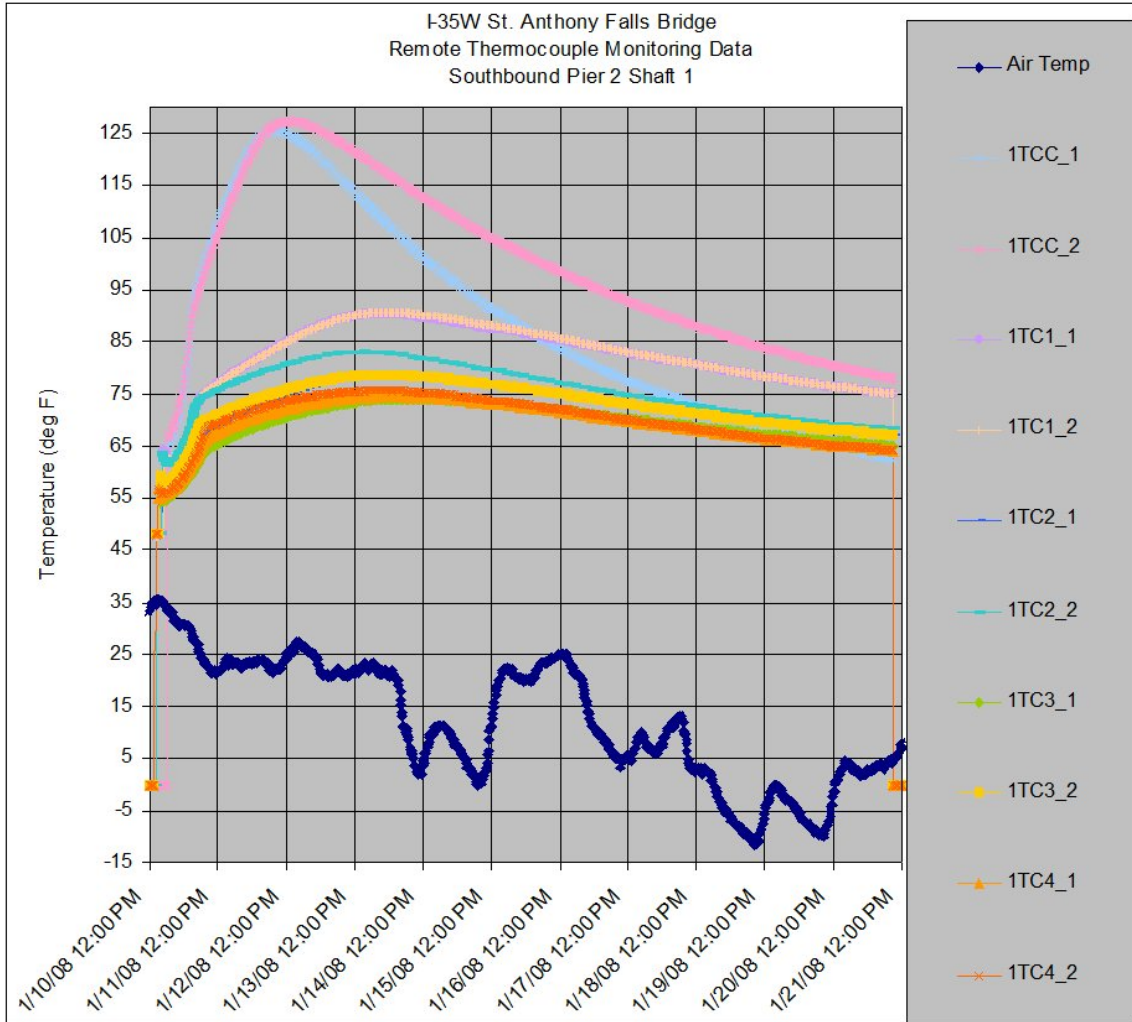


Figure 2-6. I-35W Bridge Southbound Pier 2 Shaft 1 Thermocouple Data.

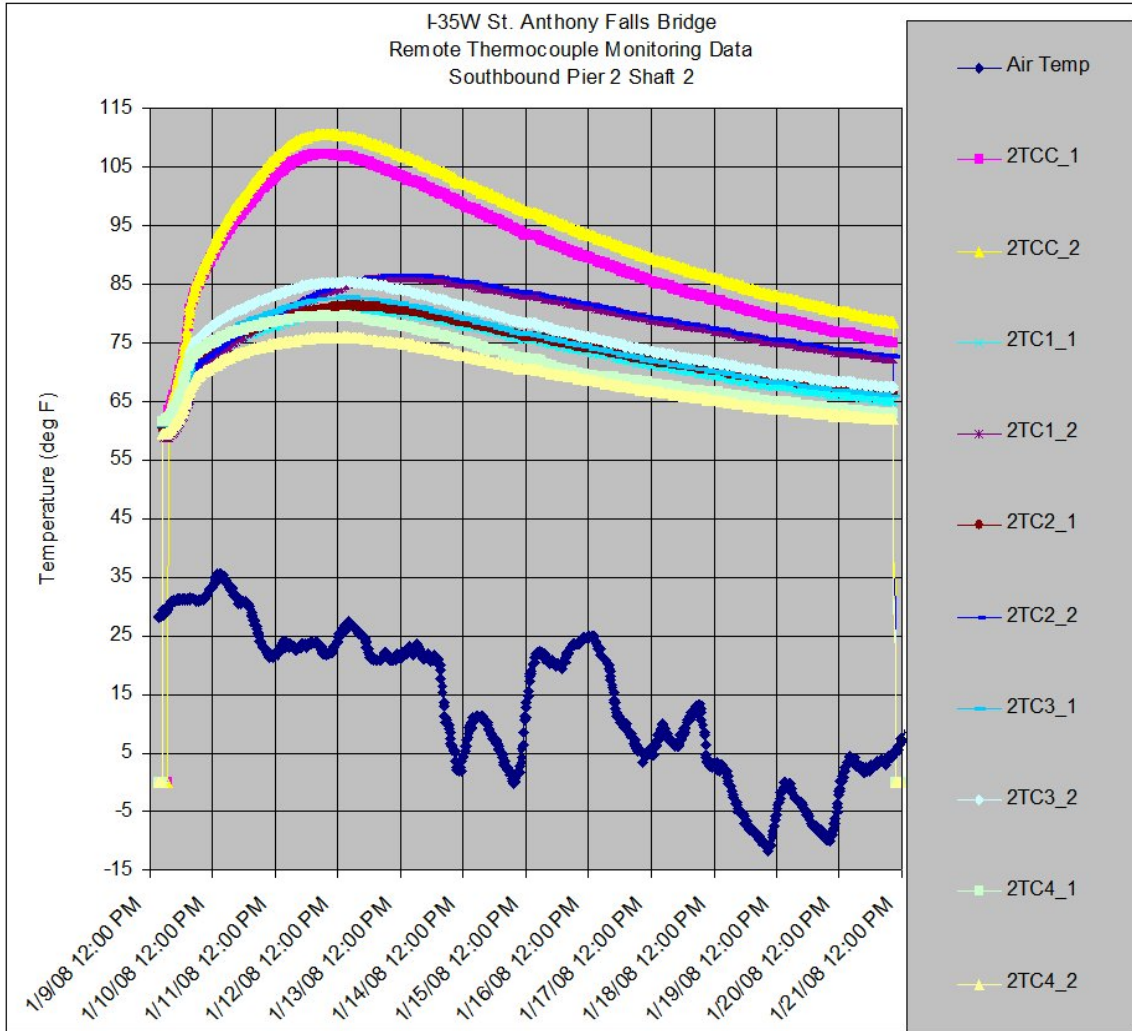


Figure 2-7. I-35W Bridge Southbound Pier 2 Shaft 2 Thermocouple Data.

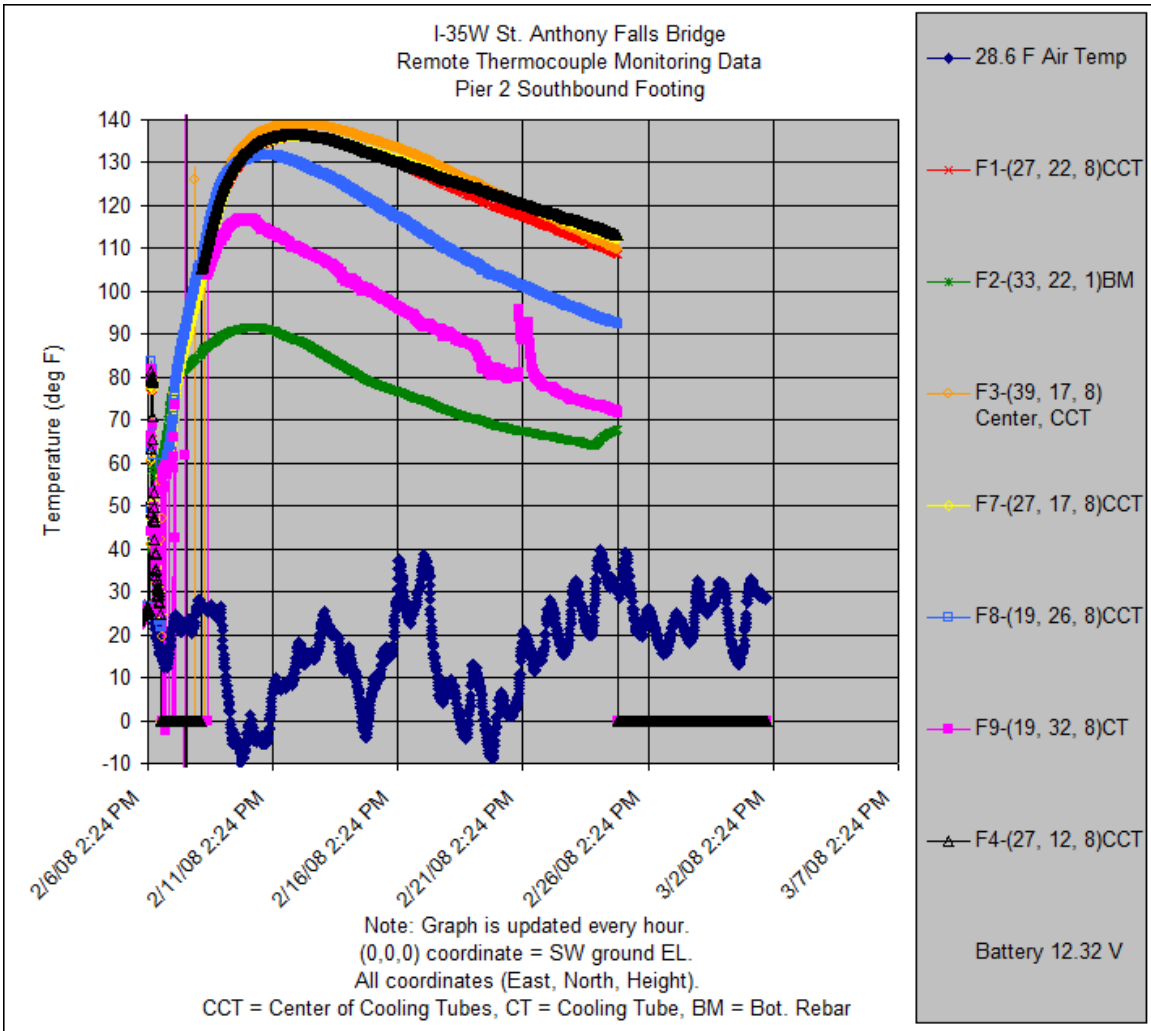


Figure 2-8. I35W Pier 2 Southbound Footing Thermocouple Data.

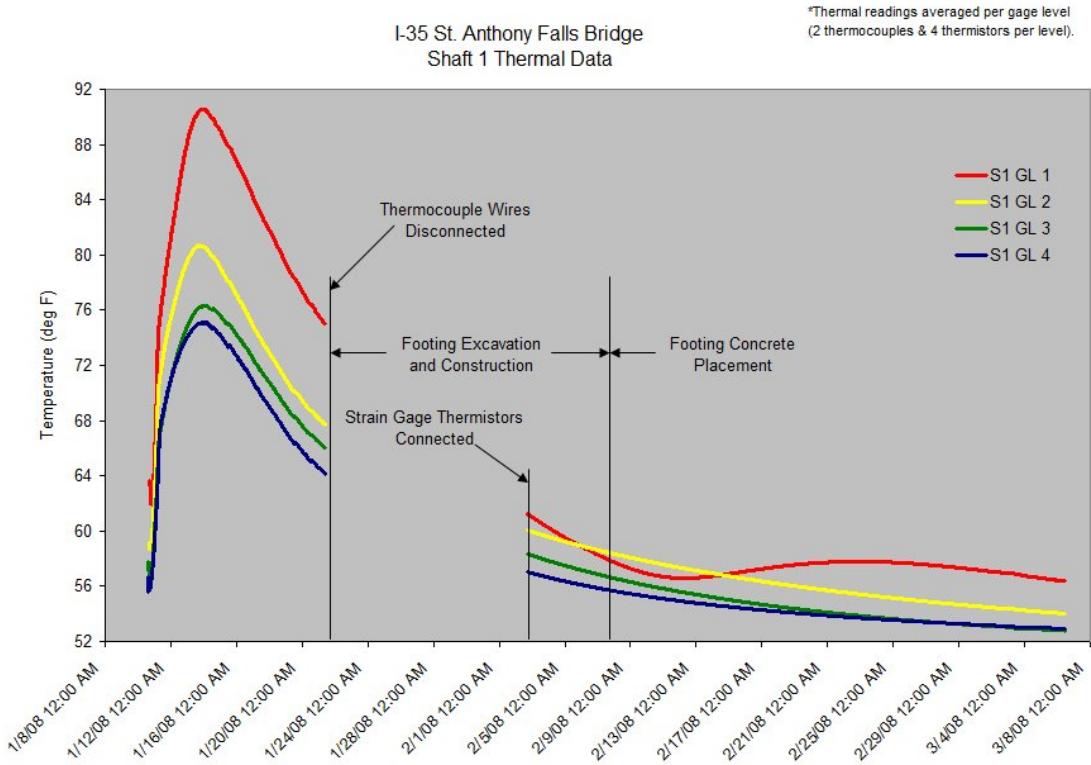


Figure 2-9. I-35W Bridge Shaft 1 Thermal Data from Thermocouples and Thermistors.

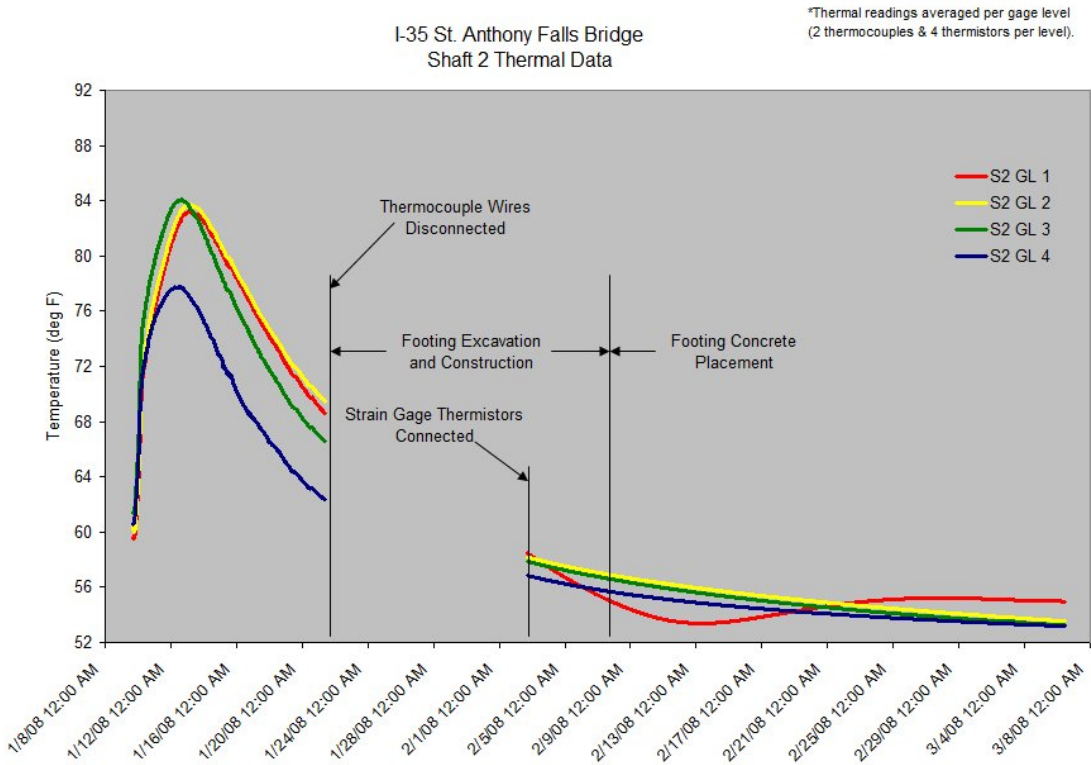


Figure 2-10. I-35W Bridge Shaft 2 Thermal Data from Thermocouples and Thermistors.

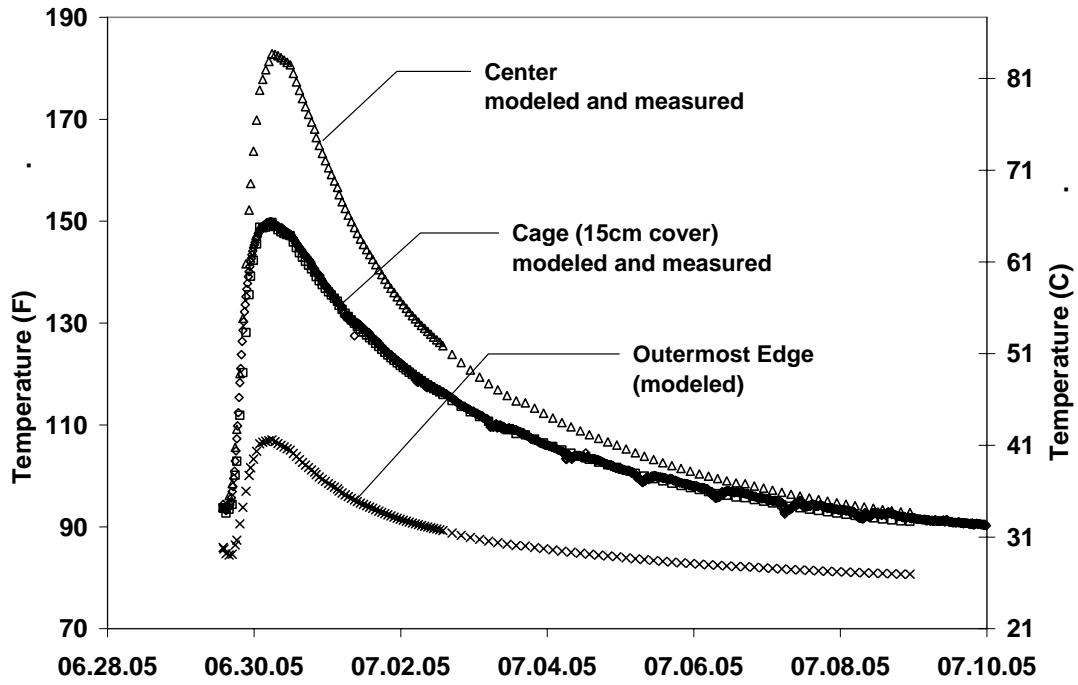


Figure 2-11. Temperature traces for the 4ft (1.22m) Clearwater shaft (modeled and observed).

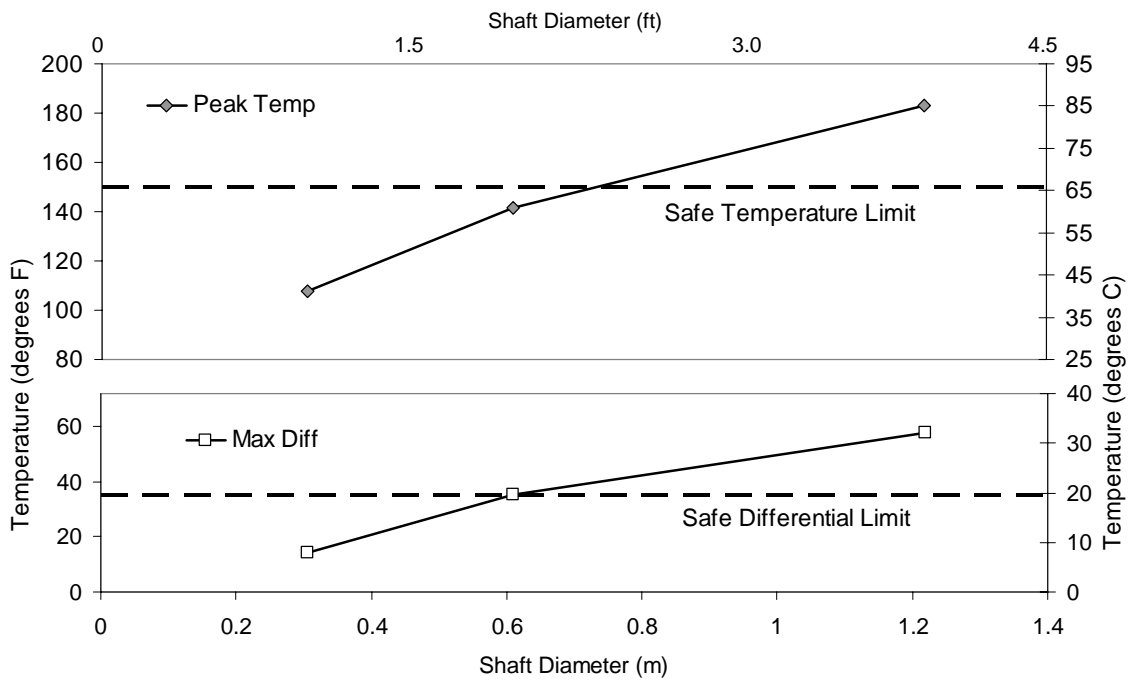


Figure 2-12 Peak and differential temperature model predictions for various-sized shafts using the “Clearwater” site conditions.

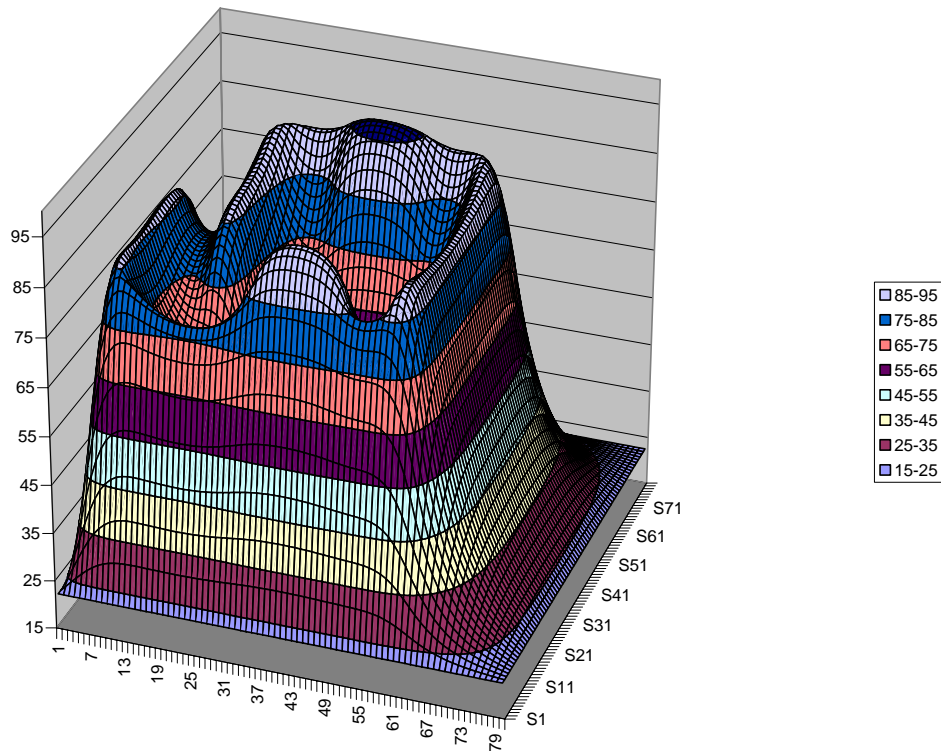


Figure 2-13. Temperature contours at mid-height of the nuclear vault walls.

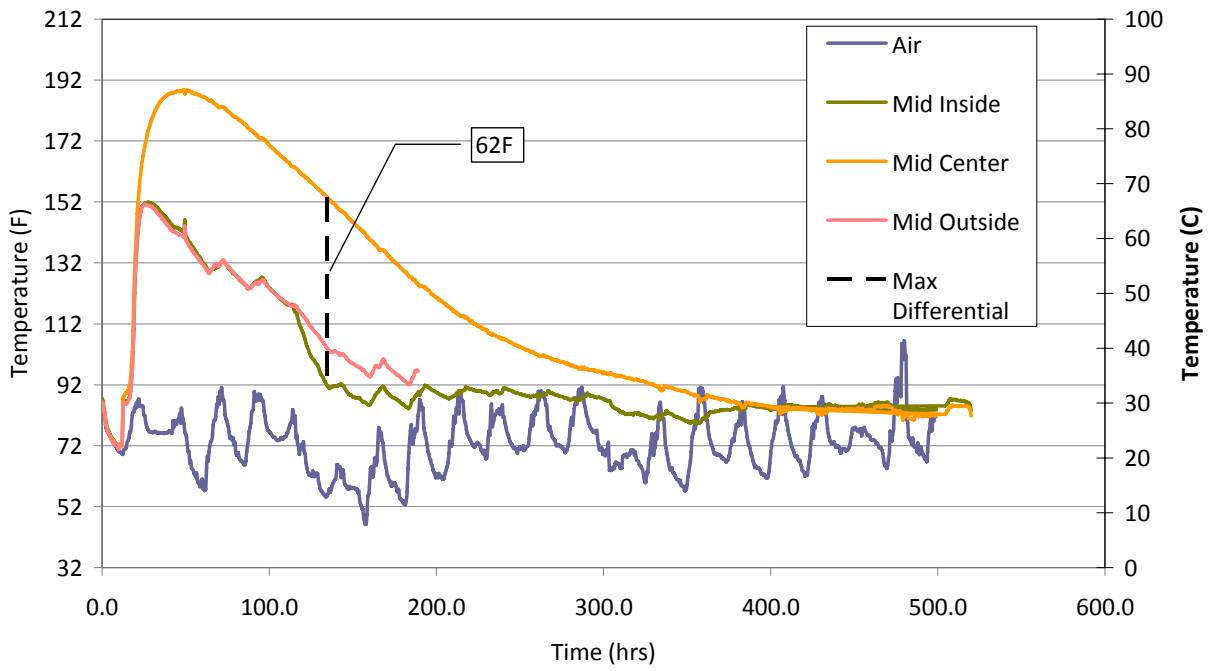


Figure 2-14. Measured wall temperatures from mid-height of nuclear vault wall.



Figure 2-15. Although crack widths register as negligible, they run sporadically throughout wall.

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Chapter Three: Thermal Integrity Evaluation

The secondary task for this research project was to conduct large-scale thermal integrity evaluations on drilled shafts and mass concrete and then evaluate the test data. A large effort was put forth into obtaining a sizable database of information for thermal testing and mass concrete effects. Thermal testing was performed at 6 different sites which included over 27 drilled shafts and 4 test soundings along a remediation wall. With the experience gained from the multiple test sites, a refined field testing system was developed. The following sections discuss the thermal program for each site.

Thermal Integrity Evaluation of drilled shafts relies on information from a Thermal Probe which contains four infrared temperature sensors that record the internal shaft temperature as it is lowered into standard 1.5” or 2” I.D. access tubes. A depth-encoded wheel mounted on a tripod at the shaft top records the position of the probe as it is lowered into the access tubes. Unlike CSL testing, the data is acquired as the probe descends rather than ascends; a data acquisition system records the field measurements for further processing.

3.1 Site I: Voided Shaft

Full discussion of the construction and thermocouple data is presented Chapter 4. Logging tubes placed within the reinforcing cage provided access to perform thermal integrity testing on the voided shaft. Thermal testing was performed on the test shaft 16 hours after final concrete placement (Figure 3-1). Tests were conducted on 6 hour intervals for the first three days and another at 144 hours. This provided a total of 180 data sets to be analyzed. With the numerous data sets, an Excel spreadsheet driven by VBA macros was written to easily import multiple tubes or timestamps.

The Excel software developed to import raw thermal data is in its alpha version / development stage. Figures 3-2 and 3-3 show the front sheet with all tubes imported and regressed and the individual tube data sheets, respectively. The general worksheet requires the user to define how many data sets to import and then proceeds to import each data set. Each data set is stored in individual sheets where the data is analyzed. Within these sheets, the infrared (IR) data is plotted versus depth and can be smoothed using a moving average defined by the user. The average IR data from each tube or timestamp is graphed in the general worksheet.

Figures 3-4 through 3-9 show the thermal testing results for tubes 1 through 6, respectively. Each figure plots 11 timestamps for each tube to show the heat generation throughout the curing of the shaft. Tube 1 shows an anomaly from 13 to 15 feet while the other tubes do not show any significant anomalies. Thermal modeling was also conducted for the test shaft and is discussed in Chapter 4.

3.2 Site II: Bridge of Lions

URS Corporation, who was providing inspection Q/A services for the State, contacted USF on June 20, 2008 to secure Thermal Integrity services for shafts that were in question (shafts 6-2, 16-4, and 25-3) for the Bridge of Lions project in St. Augustine, Florida. Therein, the nomenclature for these shafts is derived from the pier number followed by the shaft number (e.g. Pier 6 - Shaft 2 and so on). Thermal testing was performed on June 21, 2008 and June 22, 2008 wherein shafts 6-2, 8-2, and 25-3 were tested (Figure 3-10). Shaft 16-4 was not accessible. Shaft 8-2 was tested cold as a baseline comparison to shafts 6-2 & 16-4 which were also cold shafts having been poured month(s) before USF's arrival. Cold shaft testing on a known shaft (Mullins et al, 2007) did not show clear indications of the known anomalies that were easily detected soon after original concreting of that shaft. Therefore the data obtain from shafts 8-2 and 6-2 were unusable for thermal evaluation. Shaft 25-3, however, was within 24 hours of concreting upon arrival and was in the approximately within 48 hours of concreting by the time it could be tested.

Figure 3-11 shows the thermal data from shaft 25-3 which was tested 46 hours after concreting. The field conditions for this shaft were inputted into the T3DModel software and the results were compared against the field results. Variations in the measured results when compared to model predictions were used to add or subtract material from the model shaft to match the field measurements. In this case concrete was added around the shaft in regions that showed higher than anticipated temperature measurements. This was added by information obtained from the concreting logs (Figure 3-12).

The concreting log confirmed that additional concrete was needed (9 CY) to compensate for decreases in the concrete level during the casing extraction. Conveniently, as the casing was sectional, the amount of required concrete could be approximately placed at depths consistent with the length of casing removed and still in place. This full picture of the construction process is invaluable when confirming model input variables. As a result, the modeled results then varied after additional concrete was added so that a reasonable likeness/match to the measured results could be obtained. Figure 3-13 shows the original field measurements and the modeled predictions along with the resulting 3-dimensional shape.

3.3 Site III: I-4 Drilled Shafts

Field measurements were taken at the intersection of I-4 and SR-44 where problems with drilled shafts (on which sign and lighting poles are to be founded) continue to persist. Two 5 ft diameter shafts were tested 15 and 16 days after concreting. The shaft mix used contained no retarders or flyash and the shaft had fully cooled by the time of testing. The tested foundations were 60 inch diameter drilled shafts meant to support overhead signs. Construction logs were provided by the Florida Department of Transportation, District 5 Geotechnical group. These logs were used to establish approximate shaft lengths. Concrete volume logs for Shafts 8-R and 11-L indicated that the actual volume placed was less than the required theoretical volume. This is often a sign of potential shaft

defects. Shafts 8-R and 11-L have reported lengths of 22 and 33 feet, respectively.

Due to the age of both shafts and the concrete mix utilized, the recorded average temperature gradient from top to bottom was approximately 5 deg F for Shaft 8-R and 6 deg F for Shaft 11-L. Figure 3-14 plots the heat of hydration versus time for the predicted model response. This indicates that the shaft concrete was either at or closely approaching a steady state condition. Wherein, temperature variations are dependent on depth and soil type, with the predominant heat source being the ambient air at the top of shaft. This was assumed to be the case prior to testing these shafts based on the concrete mix and its lack of retarders or pozzolans. Therefore, Thermal Integrity Testing should be performed within the first 24 to 48 hours after shaft construction. However, it is possible to discern abnormal temperature readings from the predominant trend of the recorded temperatures.

Figure 3-15 plots the thermal data collected for Shaft 8-R. Shaft 8-R exhibited a temperature abnormality at an approximate depth of 18.4 feet to 19.5 feet deep and was recorded in Tubes 2 through 5. The temperature deviations recorded in this area were approximately 2 to 4 degrees. These results are not conclusive enough to provide an opinion of the shaft's integrity.

Figure 3-16 plots the thermal data collected for Shaft 11-L. Shaft 11-L exhibited a temperature abnormality at an approximate depth of 26.5 feet to 29.5 feet deep and was recorded in Tubes 1 through 5. The temperature deviations recorded in this area were approximately 2 to 4 degrees. These results are not conclusive enough to provide an opinion of the shaft's integrity.

3.4 Site IV: Lake Okeechobee Dyke Remediation Project

Another site where large amounts of cementitious materials were being used was the Army Corps of Engineers Lake Okeechobee Dyke remediation program. This project is investigating the use of a Trench Cutting and Remixing Deep Wall Modification system, TRD, (Figure 3-17) which is being proposed by Hayward Baker. The TRD cuts and remixes the soil with cement providing a higher strength wall. The mix designs for the section of the wall are included in the Appendix. The purpose of the project is to provide a cutoff wall with low permeability located at the existing dyke from elevations +36.0 to -20.5 (permeability $< 10^{-6}$). Wall strength must also fall within a range of 20 to 200 psi. Wall is approximately 57' high, 27.5" wide, & 500' long. This 500' test section is located at Port Mayaca, beginning at station 1410 + 00 and ending 1415 + 00. Although it is generally too thin to be considered for mass concrete effects, it provides data for verification and modeling calibration.

Verification testing including Cross-Hole Sonic Logging (CSL), thermal, strength, and permeability within the demonstration section. Therein, logging tubes (2" Sch.40 PVC) were placed in the mixed wall at different locations for both CSL and thermal testing. The set of logging tubes were instrumented with thermocouples to provide a thermal temperature trace over time for the mix designs. The thermocouples were placed at

depths of 30, 35, and 55 feet below the top of the wall for mix HB150C and 5, 10, and 20 feet below the top of wall for mix HB1B. Figure 3-18 and 3-19 show the thermocouple data for both mix designs along with the model temperature response.

Infrared Thermal Integrity Testing was conducted on the each set of tubes after mixing. Figures 3-20 through 3-23 show the thermal testing and model data results for each tube along the wall. Generally, each tube does not show any significant anomalies with the exception of tube 4. Tube 4 (Figure 3-23) shows a 5 degree temperature change from the top of wall to the bottom of wall. Further testing of the tube would be required to determine whether the lower section of the wall was not mixed properly or the access tube was misaligned within the wall.

3.5 Site V: Ocala Judicial Building Expansion

The Marion County Judicial Center expansion project is a five-story, 150,000 ft² structure connecting to the existing judicial center. The foundation of the new structure is supported by over 70 drilled shafts. A series of thermal scans were conducted between February 20, 2008 and April 3, 2008 at time frames ranging from 24 hrs after shaft concreting (ideal scenario) to several days after concreting. Shafts were equipped with four 1.5" I.D. steel access tubes in general accordance with standard practice in State specifications.

The concrete mix design for this project was provided by Universal Engineering Sciences on February 26, 2008 and is included in the Appendix. This information was used to create the input hydration energy parameters using the a, b, and t method outlined by Schindler (2005). The model parameters used in the T3DModel software were 0.831, 0.786, and 18.3, respectively, with an overall energy production of 70 kJ per kg of cementitious material; wherein, a type F flyash represented approximately 25% of the 785 lbs total cement /cu yd of concrete.

Prior to analysis of the field measurements, a model was created based on the heat generation properties of the above concrete mix, insulation properties of the soil around the shaft and the time of the test relative to shaft construction. The expected normal temperature varies with time as the shaft either heats or cools depending on its stage in the hydration process. Figure 3-24 shows the anticipated temperatures for 30, 36, 42, 48, and 54 inch diameter shafts under the ambient and soil conditions at that site. These help provide immediate feedback as to the condition of the shaft integrity.

Variations in the daily concrete placement temperature were used to tailor the predicted temperature graph above to exact field conditions. Deviations from the modeled norm were used to provide a quick assessment and indicate potential necking (decrease in shaft temperature) or bulging (increase in shaft temperature).

Field testing conducted on shaft DS-69 was performed approximately 24 hrs after partial concreting. Concern that led to this testing arose when the tremie pipe became lodged in the shaft and could not be removed without complication (small diameter cages are prone

to this condition). As a result, concreting was not completed leaving no concrete down to a depth of approximately 28 ft as determined by a weighted measuring tape referenced to the top of CSL tubes. Figure 3-25 shows five thermal traces corresponding to tubes 1 – 4 and the anticipated model results. At approximate depths between 28 and 32 is a more dramatic change in temperature than expected. As it is shown in all tubes, it is likely a partially cemented material which under normal concreting processes would have been expelled as debris. At approximate depths of 30 to 54 ft is a sizeable bulge in the shaft causing the higher than normal temperature in that zone which is of no concern to the integrity. Also apparent is that the cage is out of alignment near the rock socket interface pushing tube No. 1 closer to the wall (cooler) and the opposite tube (No. 3) farther away from the wall (warmer); this eccentricity in the cage decreases with depth. Finally, the bottom of the shaft shows a large reduction in temperature signal before reaching the bottom of the tube. Note the modeled response show a drastic decrease to the normal, but in this case it occurs prematurely, showing an anomaly at the toe of the shaft.

Shafts 69 and 73 were fully modeled and signal matched to assess the severity of measured low temperature conditions. Figure 3-26 shows the effect of a 2 inch inward neck, 2 ft tall on the modeled output and its comparison to the measured temperature trace for shaft DS-73.

In all, 16 shafts have been scanned for defects. Table 3-1 shows a summary of the test findings. A detailed discussion of each shaft follows:

DS-5 (Figure 3-27). Although elevated temperatures were still present, this shaft was tested long after the recommended 24 to 48 hours and is thermally inconclusive. An outward thermal gradient is required to clearly delineate inclusions. Full modeling would not be productive. CSL testing and report were produced separate to this document.

DS-10 (Figure 3-28). No structural or durability concerns. Shaft shows over-pour bulging in all areas above the rock socket which is in keeping with field logs indicating 147% concrete usage when compared to theoretical. Full modeling is not necessary.

DS-11 (Figure 3-29). No concerns. Shaft shows temperature signature of a normal shaft.

DS-18 (Figure 3-30). No concerns. Shaft shows temperature signature consistent with over-pour bulging at all depths which is in agreement to the 54" temporary casing used to a depth of 74 ft. Figure DS-18 shows this region (near 74') as a temperature transition zone to the rock socket.

DS-20 (Figure 3-31). No major concerns. Slight neck from 11 – 14 ft; no more than 2 inches of cover loss. Higher than normal temperatures down to the depth of temporary casing (76') is consistent with over-sized casing (54") used to that depth.

DS-23 (Figure 3-32). No concerns. Slightly higher than normal shaft temperature signature (typical of the site); testing performed at 72 hrs but produced usable data.

DS-24 (Figure 3-33). No concerns. Shaft shows extensive over-pour and bulging between 30 and 65 ft in depth. Concreting logs indicate 250% of the theoretical concrete volume.

DS-26. No structural or durability concerns. This shaft was selected to be tested as a result of the loss of 50ft and then 42 ft of concrete head during construction. In conjunction with the reported concrete loss, the rebar cage also fell about 2 feet. For this shaft, a 42 inch diameter temporary casing was set to a depth two feet above the original drilled shaft tip. Based on the hydration curves developed for this site the normal temperature of approximately 125 deg F was expected at the planned tube locations. The measured temperatures were within an acceptable tolerance or greater than that of the modeled temperatures. This is an indication the cage has acceptable coverage. The temperatures in excess of the model predicted norms indicate areas of over-pour bulging. It should also be noted that the usual temperature roll off near the shaft bottom was not observed. This is due to an increased amount of concrete located near the shaft tip. This is expected given the construction records which indicated concrete head loss prior to casing extraction. Due to the higher than expected temperature, full signal-matching/modeling was not necessary.

DS-27. No structural or durability concerns. A 48" shaft should have a baseline normal temperature of 116 deg F and a 54" should have a baseline normal temperature of 124 deg F. The 48" normal temperature can be seen at the bottom of the shaft near the reported bottom temporary casing (54") driven to 76 ft. Above that elevation, higher than expected temperatures for a 54" diameter are shown. This is indicative of a shaft with a larger diameter than planned. Shaft shows over-pour bulging in all areas above the rock socket which is in keeping with field logs indicating 144% concrete usage when compared to theoretical. Full modeling was not necessary.

DS-29 (Figure 3-34). No concerns. Typical over-pour bulging above the rock socket.

DS-30 (Figure 3-35). No major concerns. Mild cross-sectional reduction between 0 and 10 ft (no more than 2 inches). Over-pour bulging indicated at depths between 45 and 55 ft. Concrete volume is 280% of theoretical.

DS-69 (Figure 3-25). The construction of the shaft experienced difficulties removing the tremie and only partially poured the shaft from the bottom depth of 75' to 28'. At approximate depths between 28 and 32 is a more dramatic change in temperature than expected. As it is shown in all tubes, it is likely a partially cemented material which under normal concreting would have been expelled as debris. As with many of the other shafts, a sizeable bulge in the shaft between 30 and 54 ft is present that caused the higher than normal temperatures in that zone which is of no concern to the integrity. Also apparent is that the cage is out of alignment near the rock socket interface pushing tube No. 1 closer to the wall (cooler) and the opposite tube (No. 3) farther away from the wall (warmer); this 1.5 – 2" offset in the cage at depth 50' decreases with depth. Finally, the bottom of the shaft shows a large reduction in temperature signal before reaching the bottom of the tube. Note the modeled response show a drastic decrease to be normal, but

in this case it occurs prematurely.

DS-70 (Figure 3-36). No concerns. Although not as drastic as some, this shaft shows the common site characteristic of over-pour bulging in almost all regions above the rock socket. Inspector notes indicate 190% of theoretical concrete volume.

DS-71 (Figure 3-37). No concerns. Concrete logs were unavailable at the time of reporting; however, no extensive over-pour is indicated by the thermal scans with the exception of the toe region of the shaft where a bell-shape is prominent on all sides except T3.

DS-72 (Figure 3-38). No major concerns. Figure 3-38 shows slight necking between 8 and 11 ft in depth which is likely due to temporary casing extraction. Modeling of a similar condition in DS-73 revealed this is no greater than a 2" reduction in radius leaving 4" of concrete cover. A similar cross-section reduction is noted at the bottom 2 ft of the shaft. Finally, the cage appears to be slightly misaligned near the top on the order of 1 to 2 inches as indicated by opposite tubes T2 and T4 get cooler and warmer, respectively. The magnitude of the cage offset was determined by modeled normal response of the tube position relative to the excavation wall.

DS-73 (Figure 3-26). No major concerns. Figure 3-26 shows the same slight necking between 8 and 12 ft in depth just below the location of the temporary casing. The necking which was signal matched to be on the order of 2 inches is most prominent near tubes T2 and T3, reduces as it approaches T4, and is minimal at T1. The casing extraction process appears to have pushed against the excavation walls in the direction of T1 causing a bulge in that direction. This interpretation varies from DS-72 in that all other tubes return to a normal temperature (cover thickness) just above the neck whereas T1 experiences the higher than normal temperature.

DS-38. Figure 3-39 shows the results of thermal scans run on shaft DS-38 along with model predicted normal temperatures for 48" and 54" diameters measured at normal tube positions. Multiple scenarios can cause higher than normal temperature traces with the most common being over-pour bulging. However, when the higher than normal temperature is accompanied by lower than normal temperature directly across the cage, it is generally caused by poor cage alignment in the excavation. Alignment is important in assuring adequate structural capacity as well as sufficient durability which is provided in the form of minimum concrete cover levels (typically 3" or more). Alignment errors are generally no more than 6 inches due to the normal cage diameter relative to the excavation diameter. In the case of shaft DS-38, a 30 inch nominal cage diameter was placed in a 54 inch excavation which in turn was stabilized by full length temporary casing. Upon removal of the casing, a modest increase in cover can be realized from the volume of concrete that replaces the volume of the steel casing previously occupied. Figure 3-39 also shows three depths of interest where cage alignment was assessed: 21, 48, and 70 feet.

Figure 3-40 shows the likely cage alignment at depth 21 ft based on the normal temperature distribution across a 54" shaft at that time. As soil temperature is assumed, a range of temperature has been prepared to show the relative insensitivity to this parameter. Measured temperatures of 145 deg F and 125 deg F were recorded in tubes T3 and T1, respectively. These correspond well to the modeled temperature when the cage is misaligned as shown which appears to still have provided 5 to 6 inches of cover near T1. Tubes T2 and T4 are similar in temperature indicating somewhat centered alignment in that direction.

Figure 3-41 shows the likely cage alignment at depth 48 ft. Measured temperatures of 147 deg F and 119 deg F were recorded in tubes T4 and T2, respectively. Tubes T1 and T3 appear to be equidistant from the shaft side with a slightly lower than expected temperature which corresponds to a remaining cover of 6 to 8 inches. Based on measured temperatures the shown alignment provides an estimated 5 inches of cover near tube T2.

Finally, at depth 70 ft where the shaft transitions from 54 to 48 inches in diameter, a difference of 23 and 22 deg F is noted between tube pairs T4 - T2 and T3 - T1. This can be similarly shown to be caused by the same alignment variation causing a plus or minus 11 to 11.5 deg F change. Figure 3-42 shows a normally aligned cage with its normal temperature response as well as the minus 11.5 deg F value which corresponds to an estimated 6 inches of cover.

To confirm the thermal data, inclinometer testing was performed on shaft DS-38 on April 29, 2008. Figure 3-43 shows the inclinometer results for each tube within the shaft. As a reference, tube 1 is the most northerly tube and increasing in value in a clockwise fashion looking down on to the shaft top. By taking the inclinometer data and plotting the results for all four tubes in 3D, a visualization of the cage alignment relative to the excavation can be seen in Figure 3-44.

3.6 Site VI: UF 290 Site

A series of thermal scans were conducted October 22, 2008 at time frames ranging from 26.5 hrs to 32.3 hrs after shaft concreting for the UF-290 Southwest Parking Garage. The concrete mix design (included in the Appendix) for this project was supplied by Universal Engineering Sciences on October 23, 2008. This information was used to create model input hydration rate parameters using the a, b, and t method outlined by Schindler (2005). The model parameters assigned were 0.830, 0.651, and 14.9, respectively with an overall energy production of 68 kJ per kg of concrete (or 381 kJ/kg of total cement content). A Type F flyash represented approximately 20% of the 680 lbs total cement /cu yd of concrete. Concrete field test results were also supplied on October 24, 2008 along with some pertinent construction log information. Table 3-2 shows a summary of the physical parameters used to prepare the comparative models.

Figure 3-45 shows the anticipated temperatures for Shafts 152, 154, and 171 as a function of depth at the exact time of testing. Note slight variations in the predicted temperature

correspond to cooling after the peak which occurred at approximately 15 hrs (S171 hottest at 26.5 hrs and S154 coolest at 32.3 hrs). The variations in a given temperature trace correspond to either changes in the shaft diameter or soil stratification. The depth to top of rock was assumed from pilot borings provided; those depths were 32ft, 36ft, and 35.5ft for shafts 152, 154, and 171, respectively.

Figures 3-46 through 3-48 show the measured temperature traces from all four access tubes along with the model predicted temperature for shafts 152, 154, and 171, respectively. The shape of the temperature versus depth curve provides a basic rendering of the actual shaft shape. Those temperature values higher than predicted correspond to over-pour bulges in the shaft most closely adjacent the tube showing those values. Conversely, regions of lower than predicted temperature correspond to poorly cemented regions or absent concrete. Conservatively, by assuming low readings correspond to absent concrete, those regions can be signal matched to provide a simulated section loss.

In this case, all shafts showed close agreement with the predicted temperature response with one exception of concern showing a slight reduction at the base of shaft 152 corresponding to a 1" reduction in radius (or high w/c ratio; partial segregation). This zone tapers from no reduction at a depth of 40ft to the one inch reduction at the toe (depth 48ft). Above the rock socket in both of shafts 152 and 171 sizeable bulges can be seen which are not considered to be regions of concern. Shaft 154 shows no appreciable concerns; minimal segregation at the toe.



Figure 3-1. Thermal testing of voided shaft.

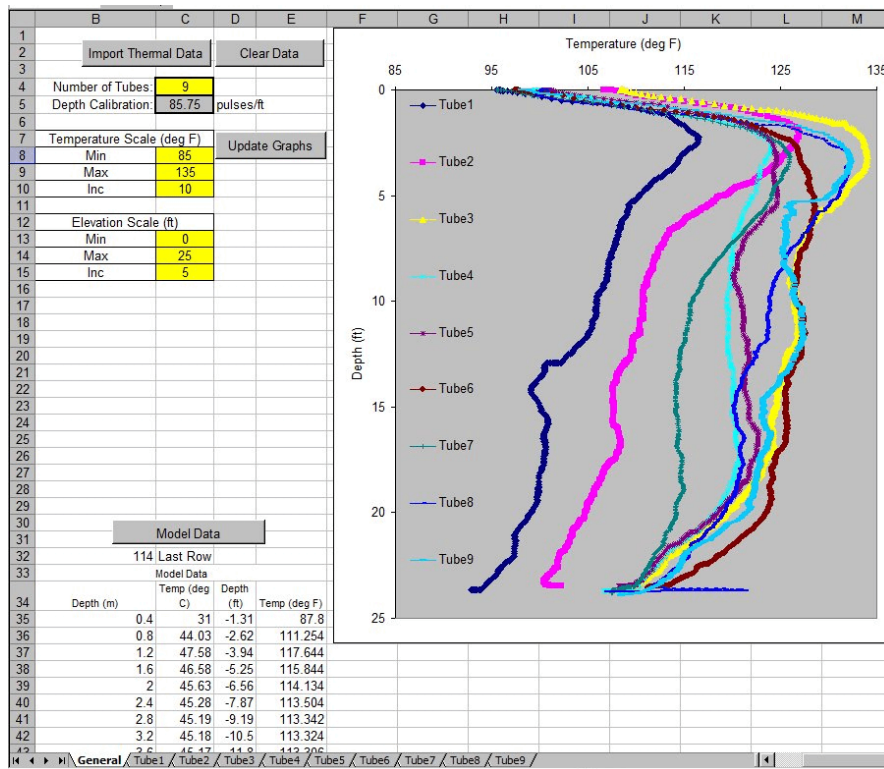


Figure 3-2. Thermal field data analyzer front page.

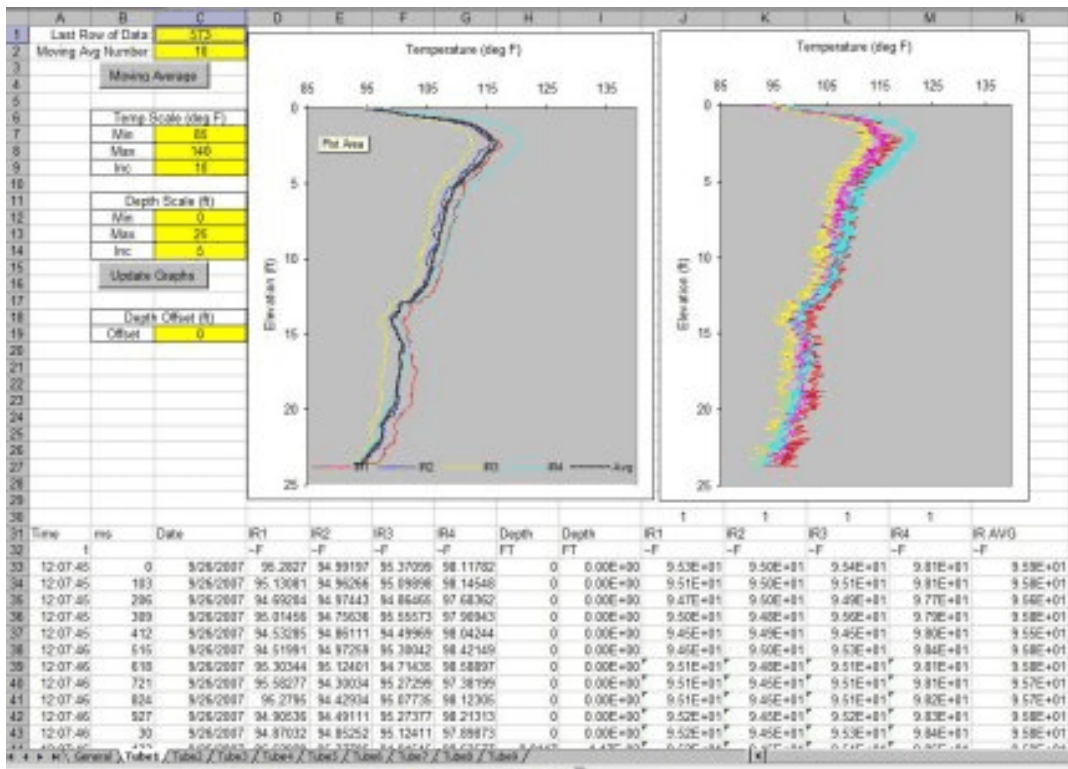


Figure 3-3. Thermal field data analyzer individual test run worksheet.

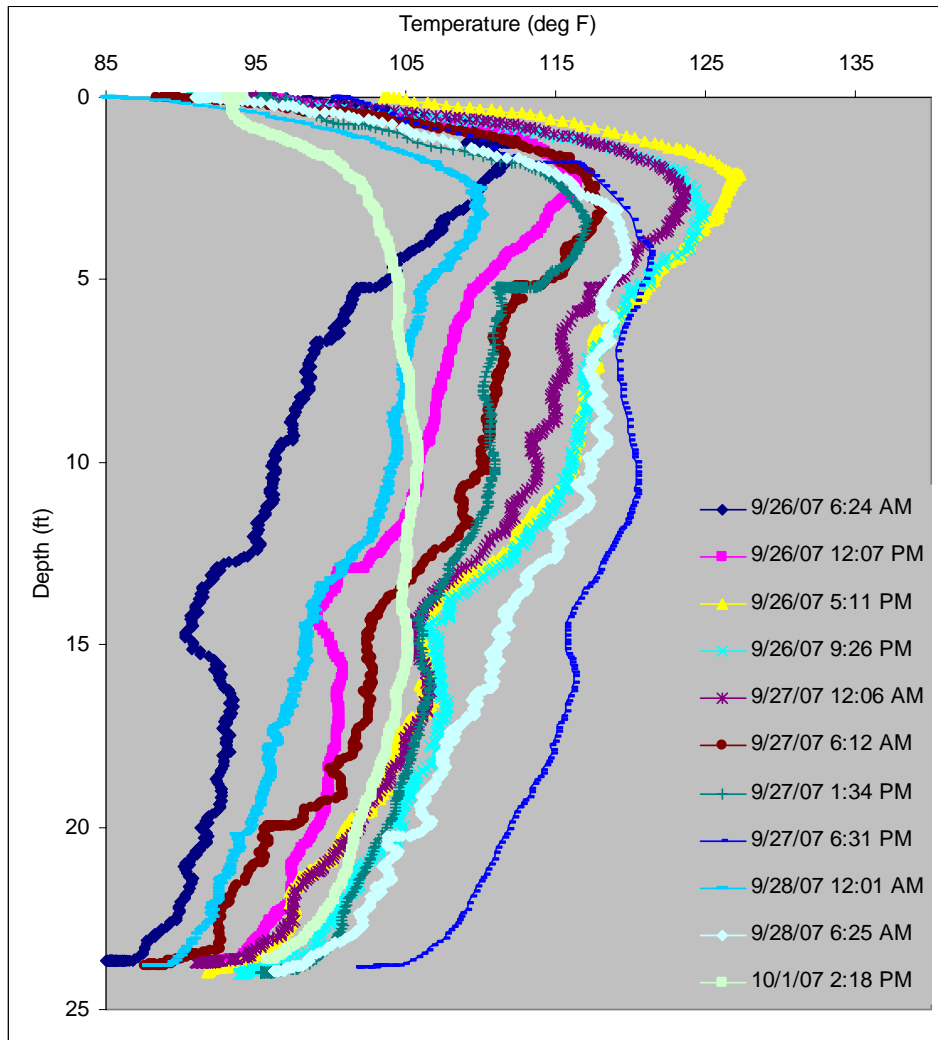


Figure 3-4. Voided shaft thermal results for tube 1.

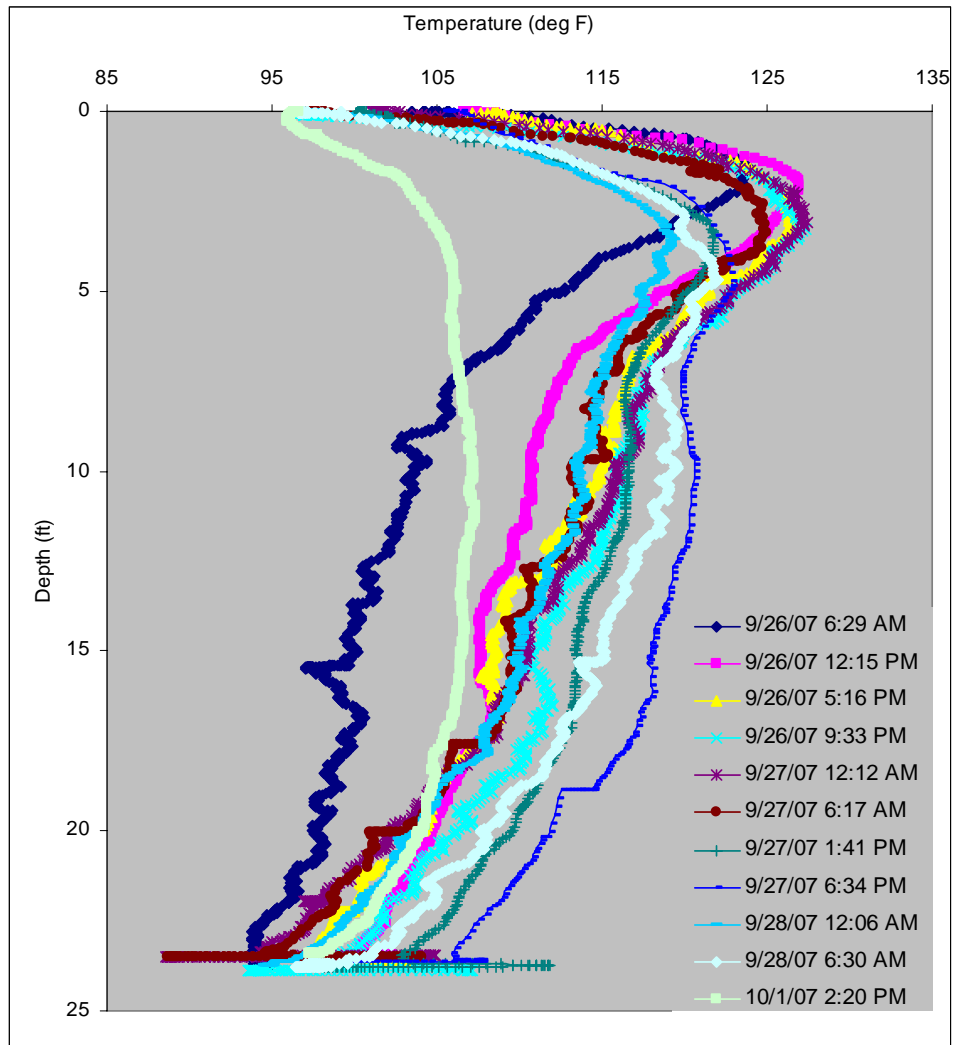


Figure 3-5. Voided shaft thermal results for tube 2.

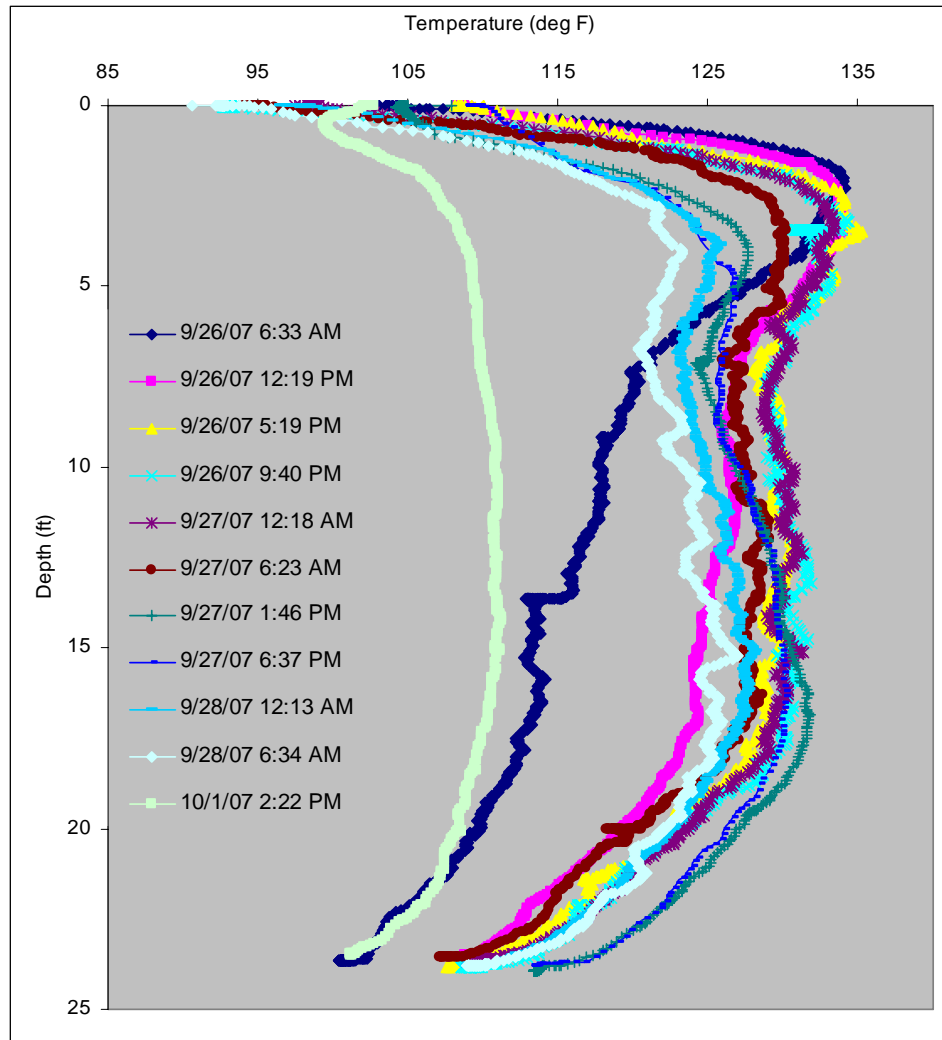


Figure 3-6. Voided shaft thermal results for tube 3.

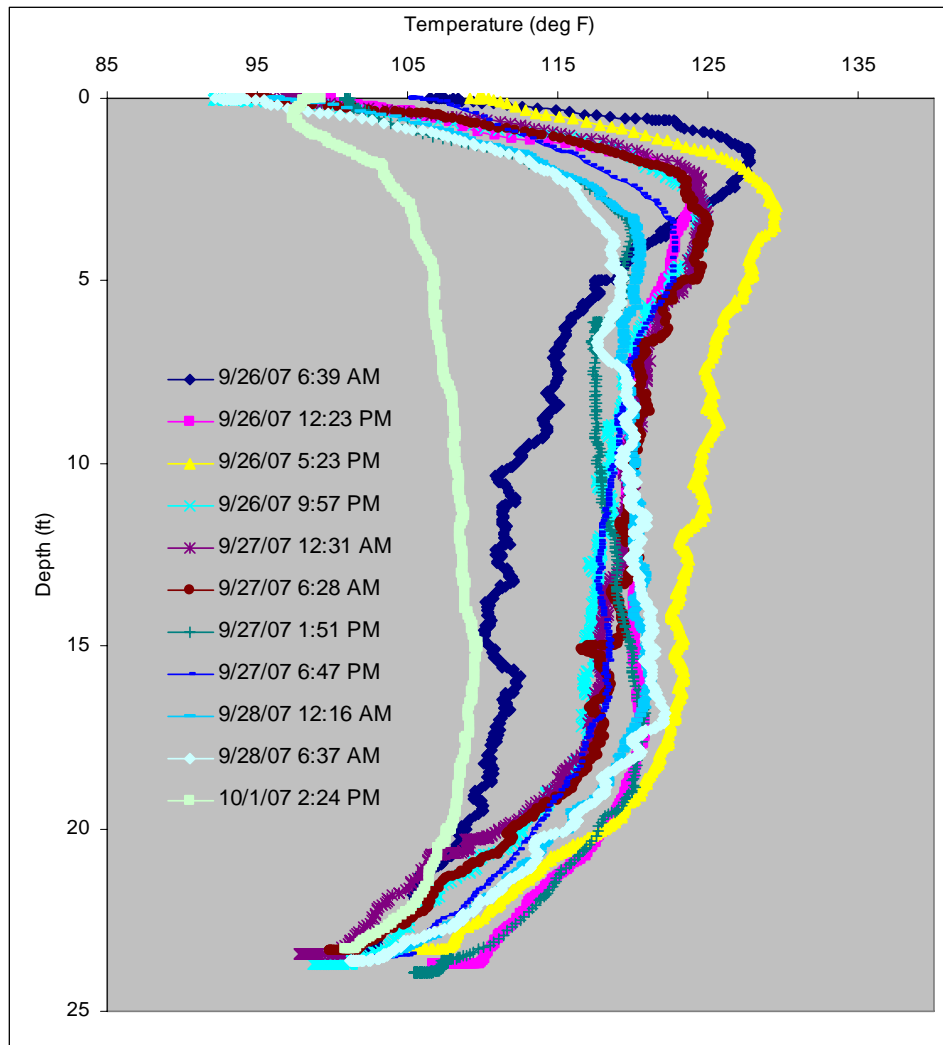


Figure 3-7. Voided shaft thermal results for tube 4.

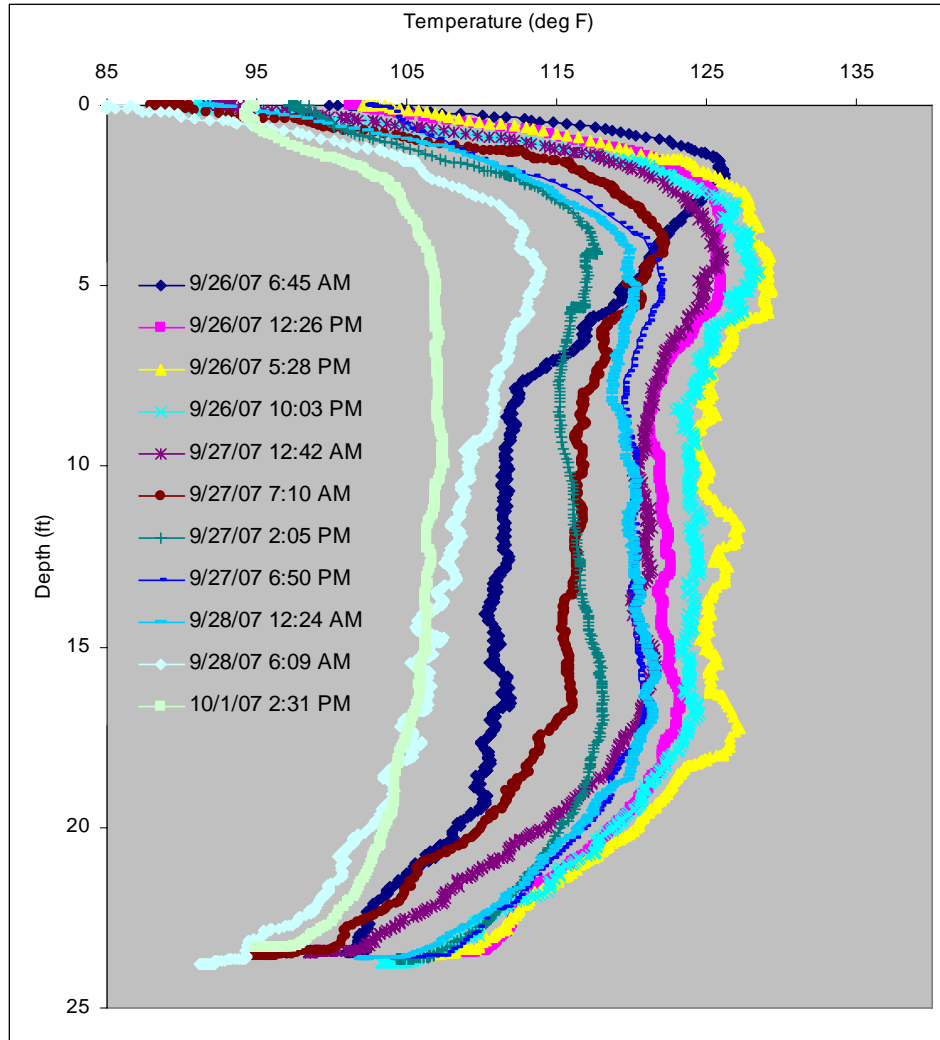


Figure 3-8. Voided shaft thermal results for tube 5.

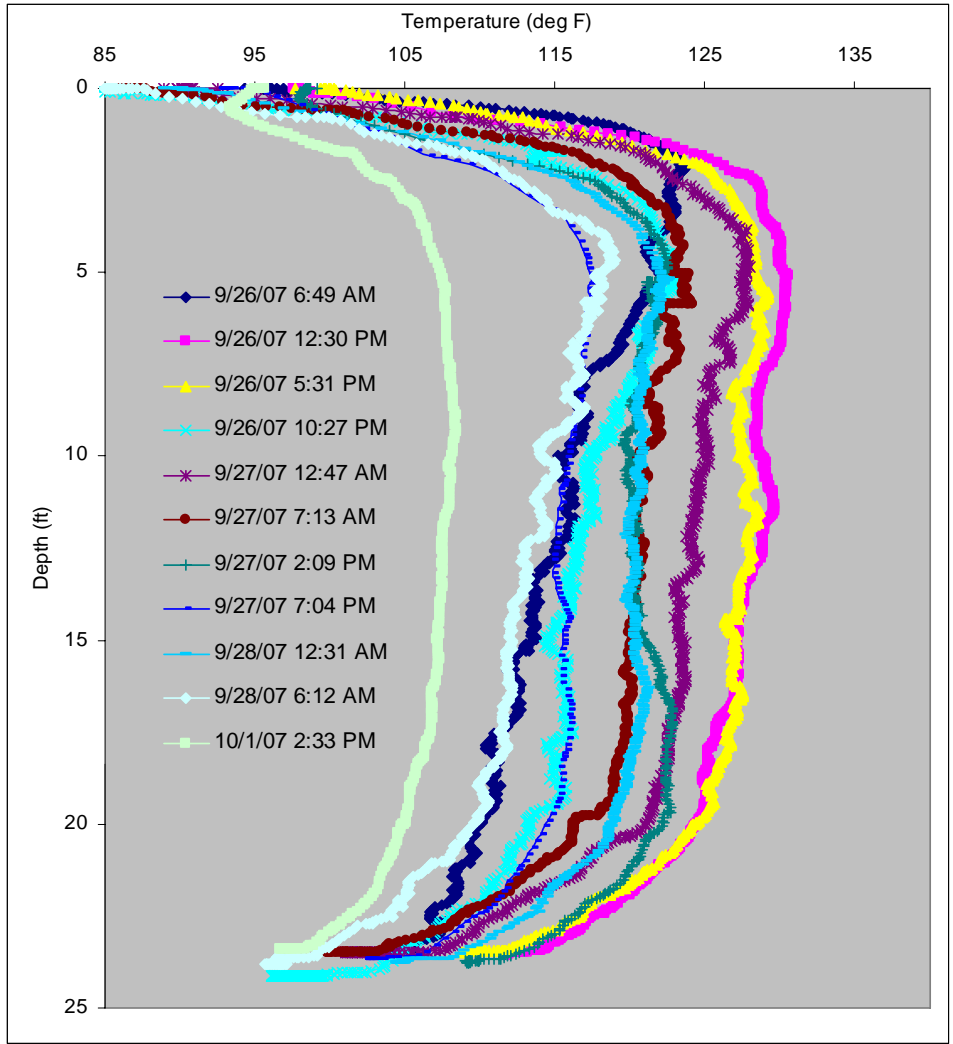


Figure 3-9. Voided shaft thermal results for tube 6.



Figure 3-10. Thermal Integrity Testing of Shafts 6-2 (left), 8-2 (center), and 25-3 (right).

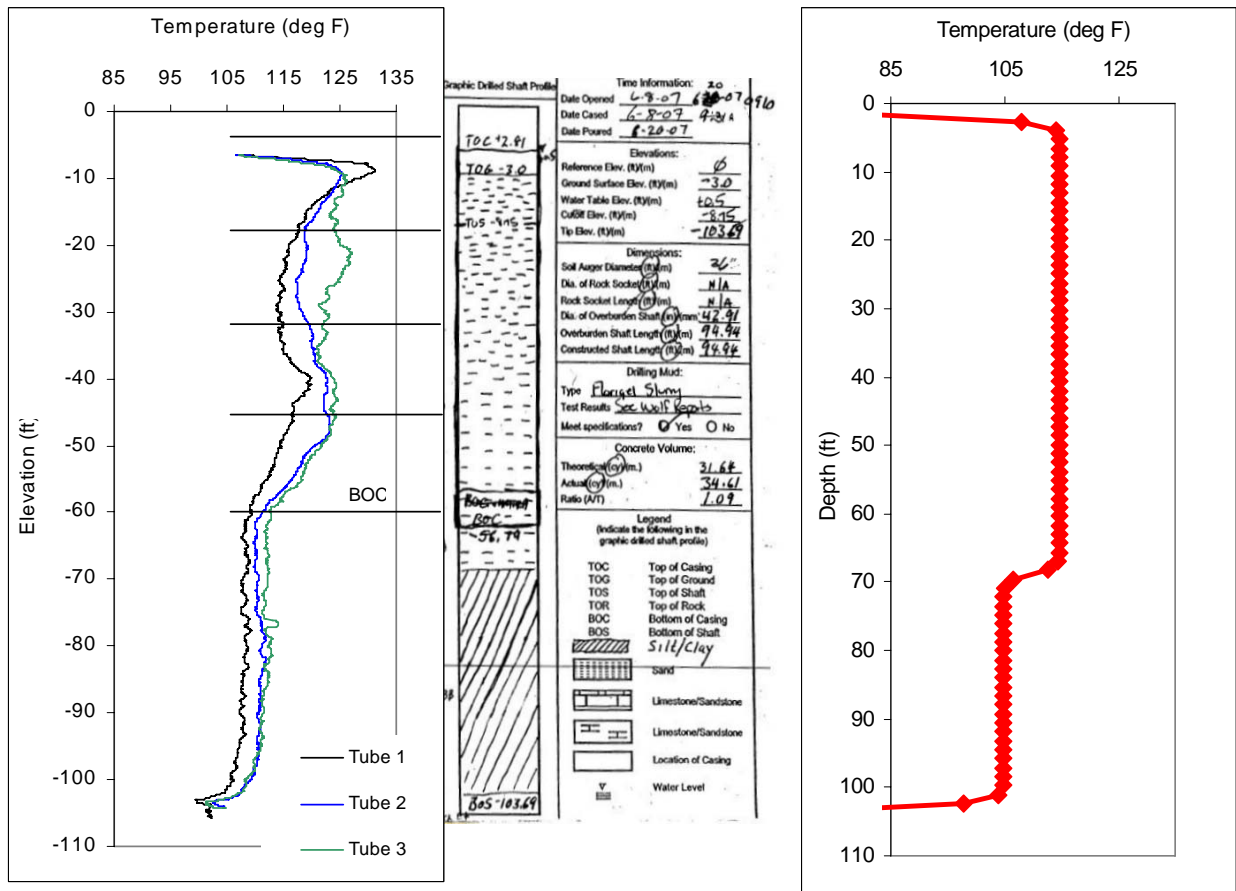


Figure 3-11. Shaft 25-3 results: measured (left), excavation log (middle), preliminary model (right).

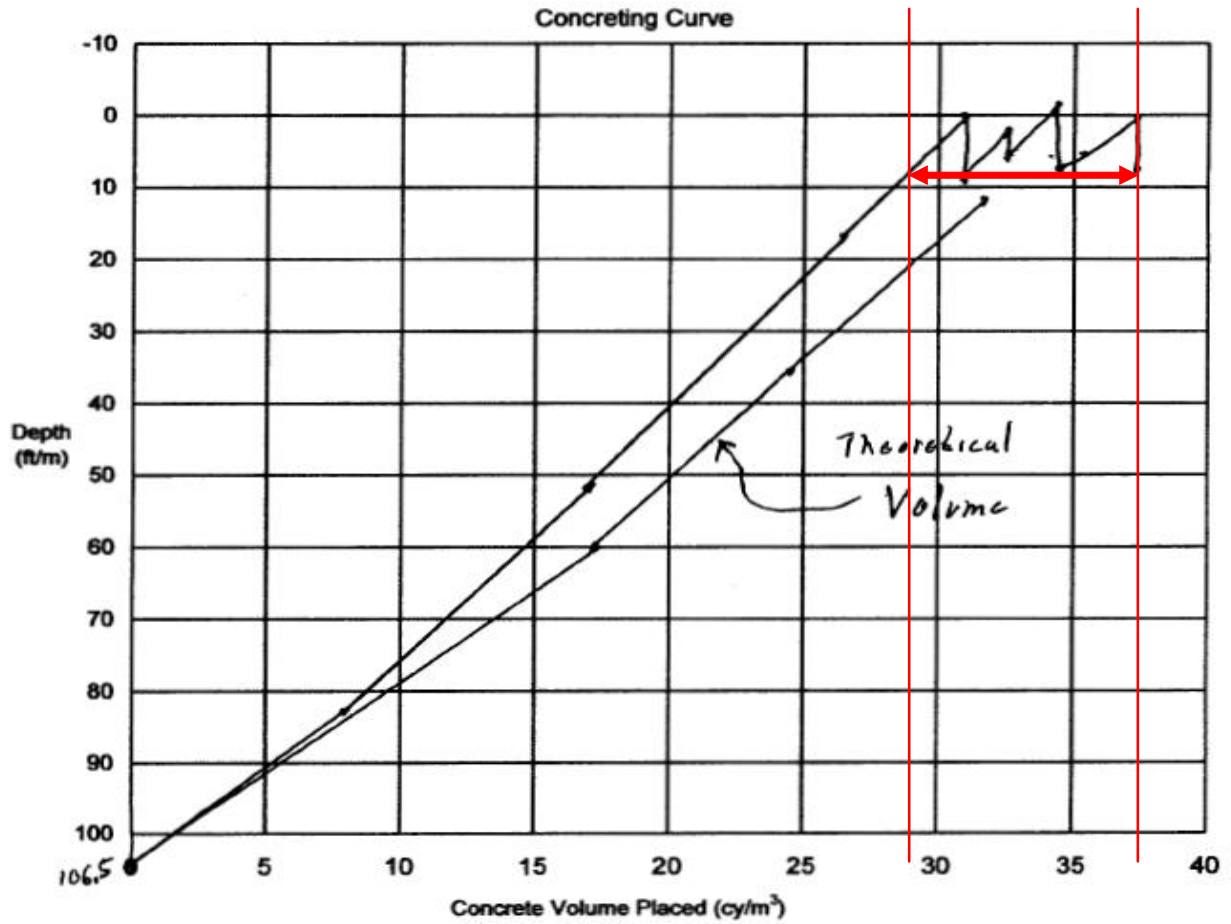


Figure 3-12. Concreting log for shaft 25-3 (red arrows shows concrete used to refill while casing was extracted).

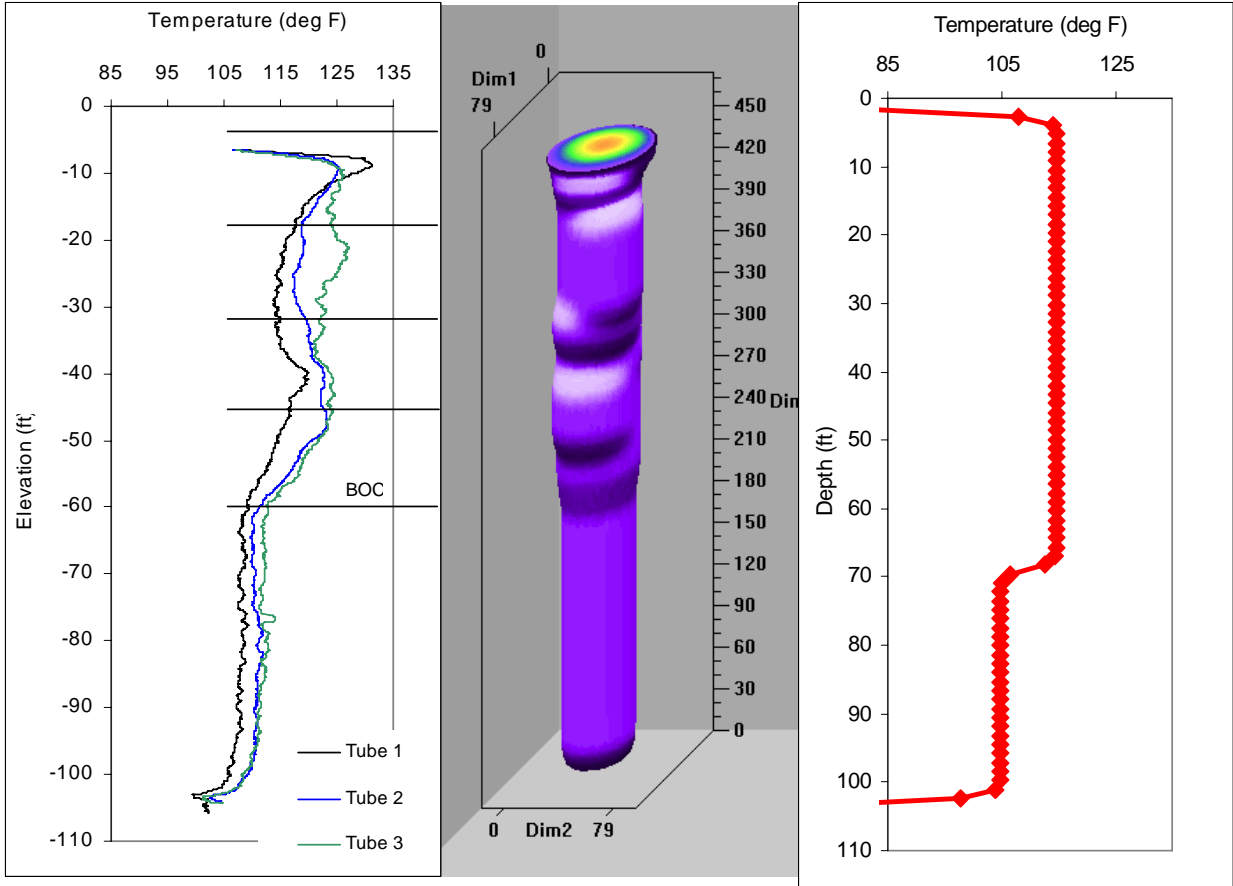


Figure 3-13. Shaft 25-3 results compared to 3-D modeled rendering showing where extra concrete finally came to reside.

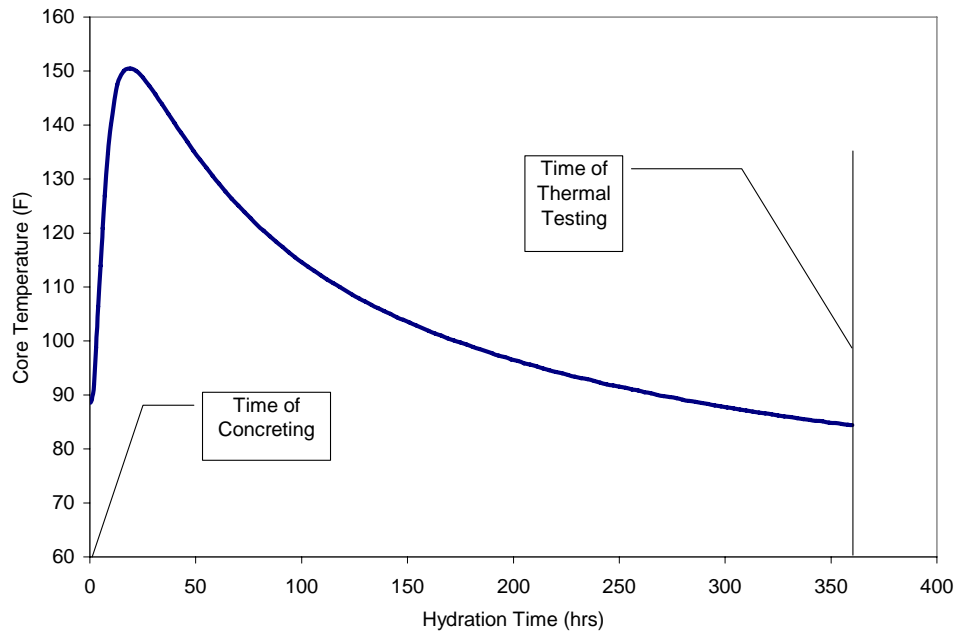


Figure 3-14. I-4 & SR400 modeled temperature curve versus time.

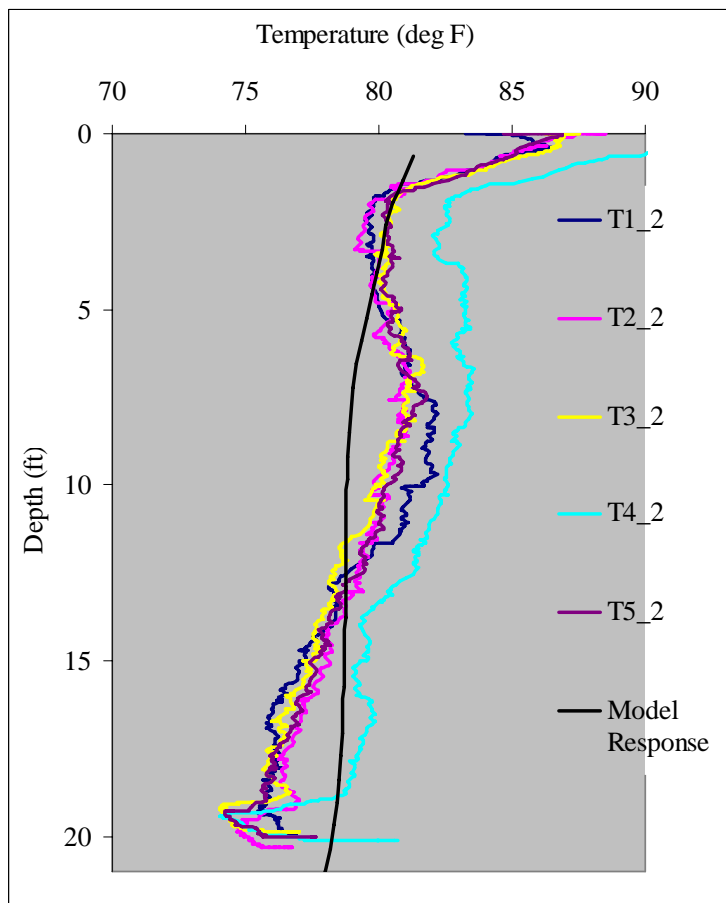


Figure 3-15. I-4 & SR400 thermal testing results for Shaft 8-R.

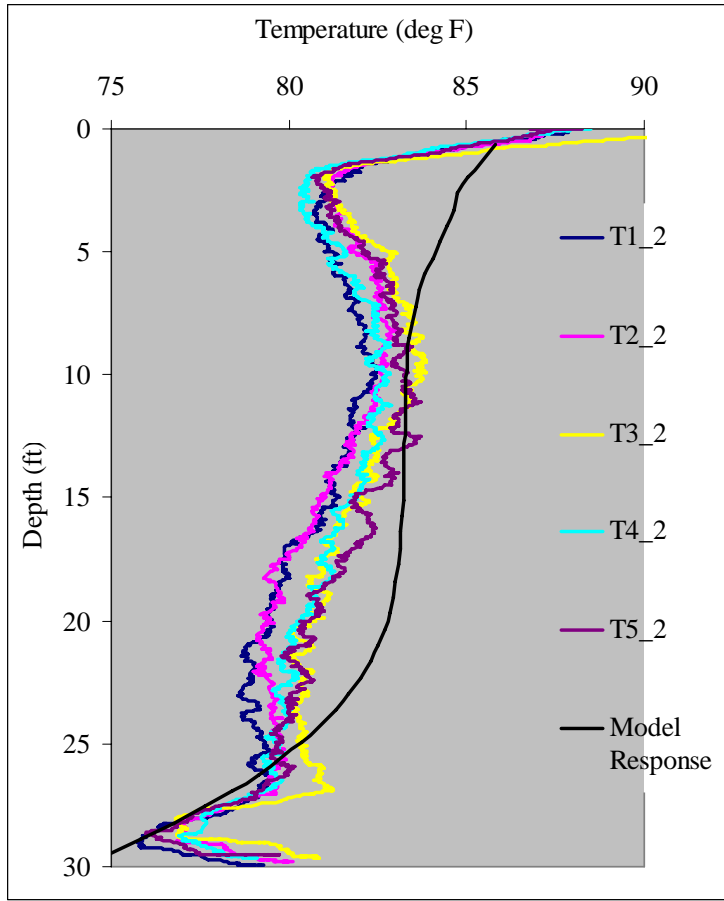


Figure 3-16. I-4 & SR400 thermal testing results for Shaft 11-L.



Figure 3-17. Lake Okeechobee dyke modifications using TRD.

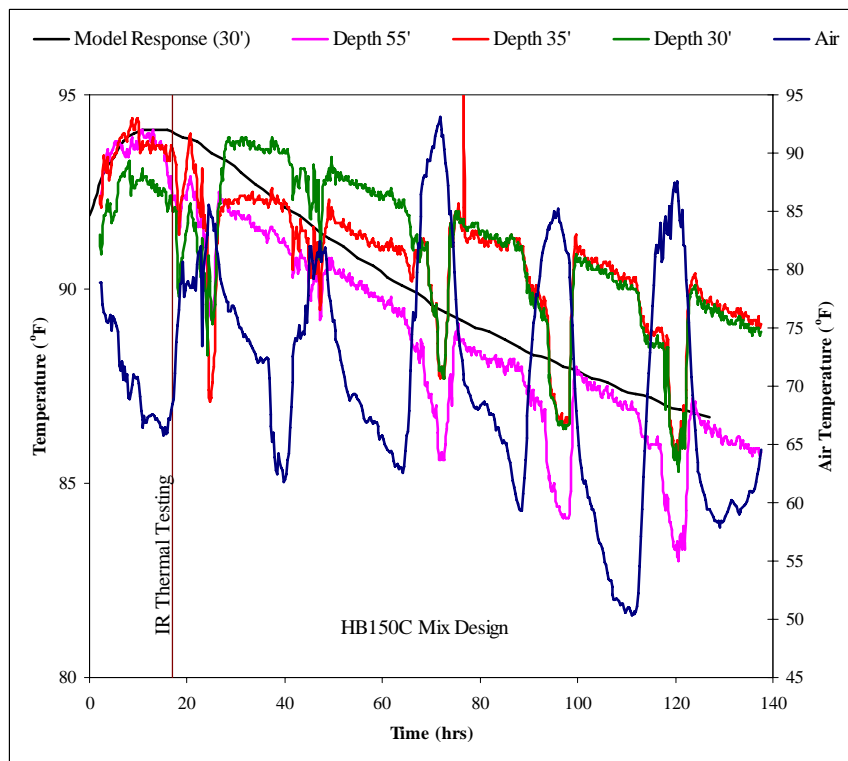


Figure 3-18. Lake Okeechobee HB150C Mix Design thermocouple data.

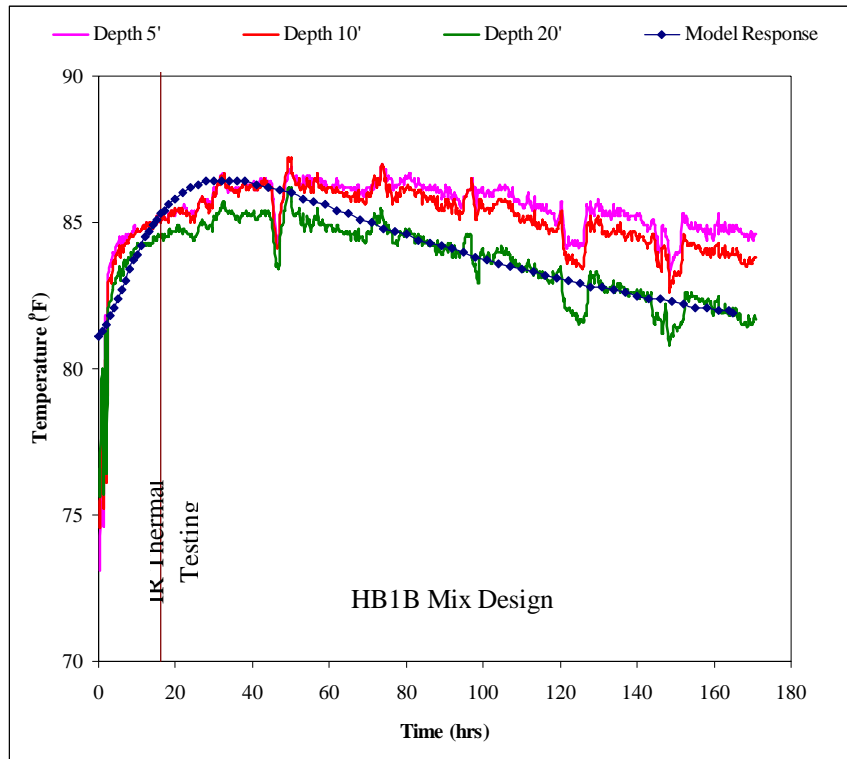


Figure 3-19. Lake Okeechobee HB1B Mix Design thermocouple data.

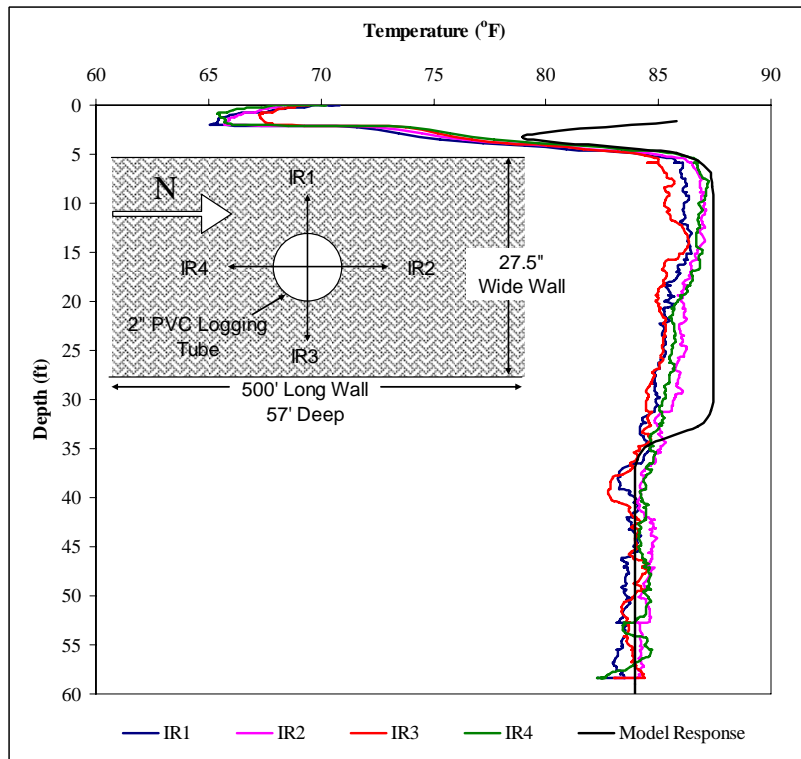


Figure 3-20. HB150C tube 1 at station 1413+25, 17 hours after mixing.

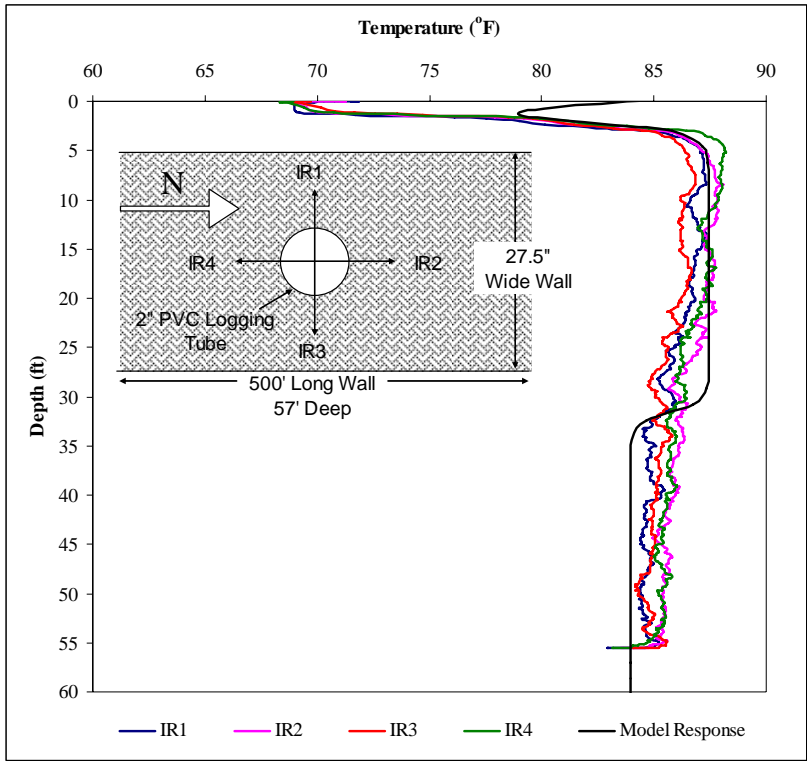


Figure 3-21. HB150C tube 2 at station 1413+33, 17 hours after mixing.

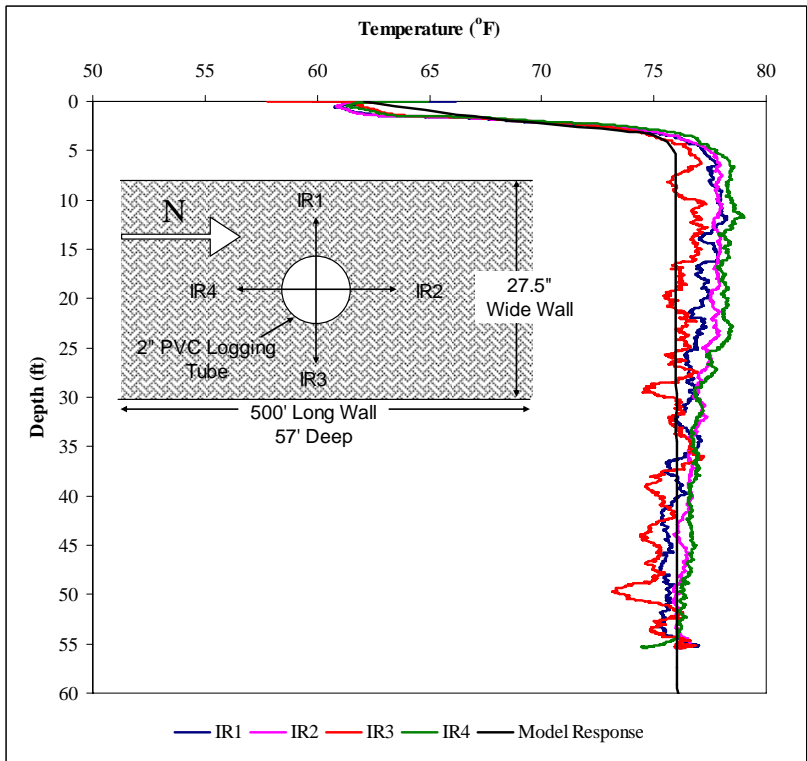


Figure 3-22. HB1B tube 3 at station 1414+36, 16 hours after mixing.

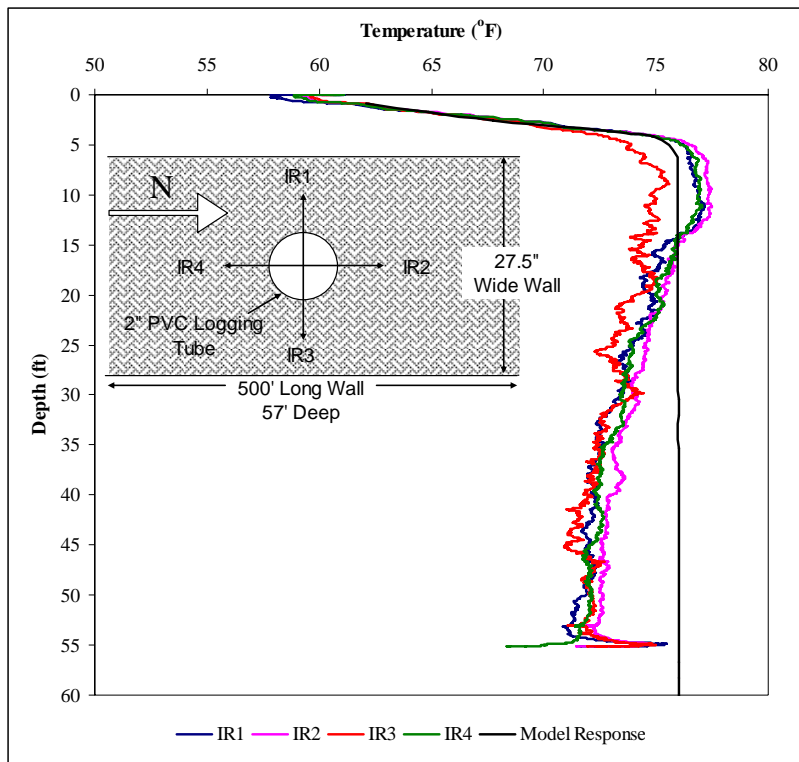


Figure 3-23. HB1B tube 4 at station 1414+44, 16 hours after mixing.

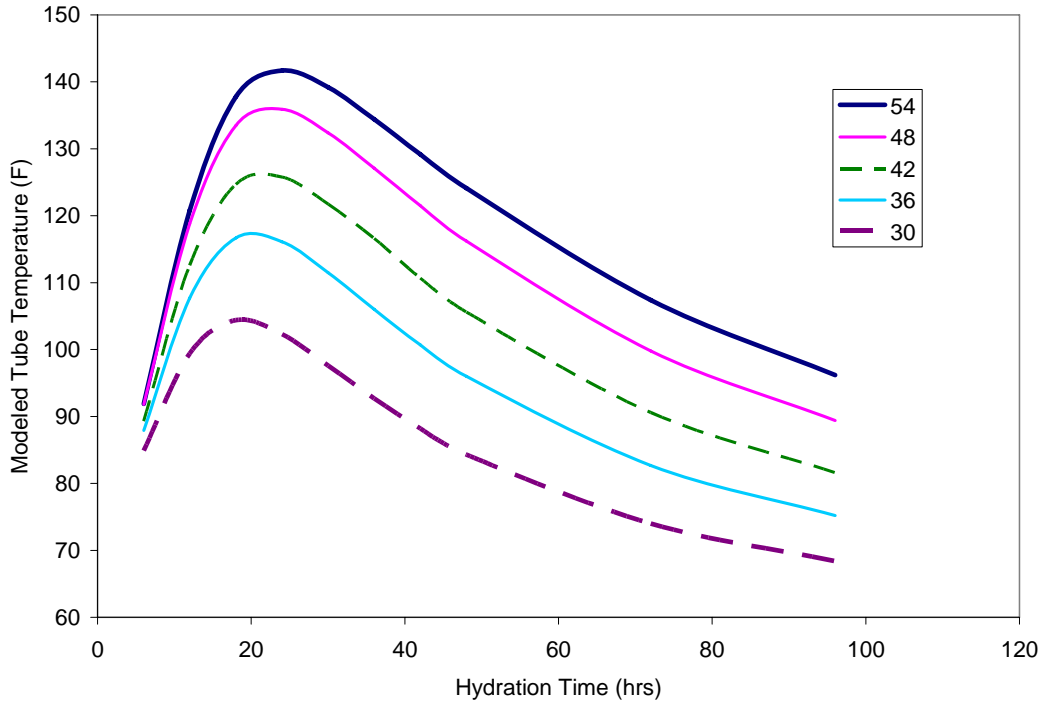


Figure 3-24 Normal access tube temperatures for shaft sizes tested.

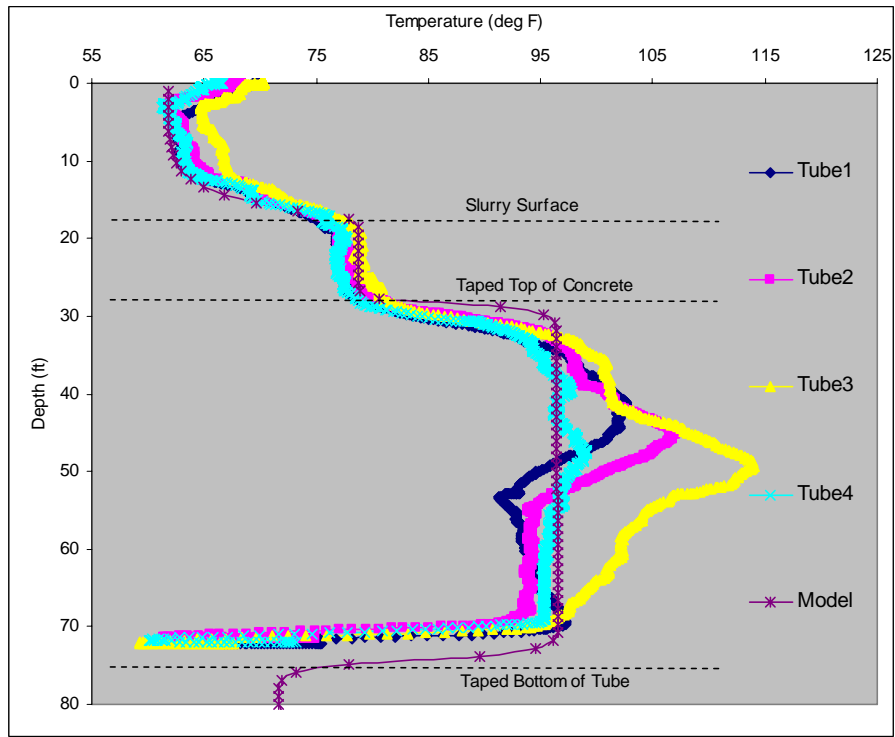


Figure 3-25. Ocala Judicial Expansion Project Shaft 69 Thermal Test Results.

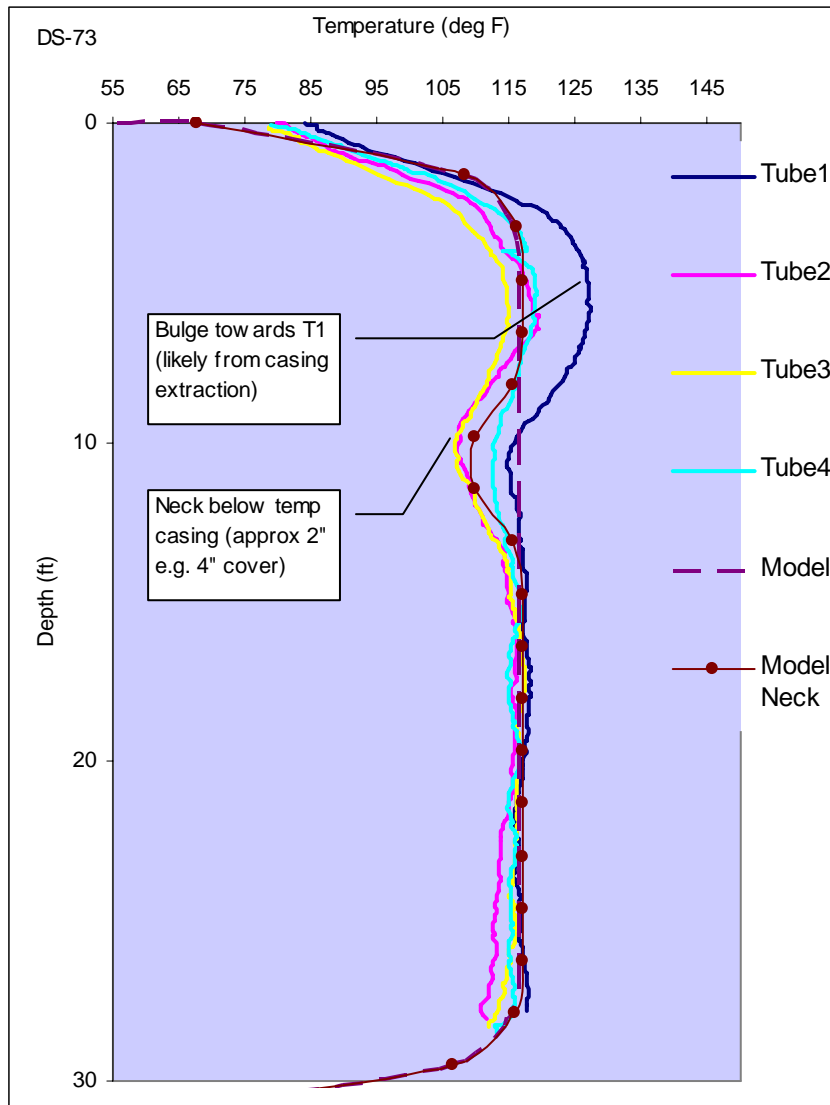


Figure 3-26 Signal match of Shaft DS-73 field results with modeled output, with and w/o neck.

Table 3-1 Shaft Testing Details

Shaft I.D.	Date/Time Test Performed	Date/Time Cast	Casting Air Temp		Casting Conc Temp		Hydration Time (hrs)	Diam (in)	GWT (ft)	Length (ft)	Rock Socket Length (ft)	Vol. Theory (cy)	Vol. Field (cy)	Vol. (%)	General Comments
			C	F	C	F									
DS-5	2/22/08 13:22	2/1/08 16:00	21	70	22	72	501.4	36	N/A	90	25	24	N/A	N/A	Over 72 hr hydration / temp too low for proper evaluation
DS-10	2/22/08 16:10	2/21/08 16:30	19	66	20	68	23.7	36	N/A	83.25	N/A	22	32	147	Slight cage misalignment 45-55' bulge
DS-11	2/29/08 16:45	2/28/08 15:05	8	46	17	62	25.7	36	N/A	32.33	25	8	11	130	No concerns
DS-18	3/21/08 10:16	3/18/08 10:30	21	69	22	72	71.8	54	49	90	40	53	N/A	N/A	54" casing to 74'; 48" rock socket
DS-20	3/27/08 13:00	3/26/08 2:57	23	73	26	78	34.0	54	N/A	90	N/A	53	67	126	54" casing to 76'; 48" rock socket
DS-23	3/7/08 13:05	3/4/08 10:00	23	73	26	78	75.1	36	N/A	71	23	19	N/A	N/A	No concerns
DS-24	2/22/08 14:48	2/21/08 11:00	17	63	19	66	27.8	36	43	92	42	24	61	253	Bulge @ depth 30-65' all tubes
DS-26	4/3/08 13:09	4/2/08 11:02	27	81	27	80	26.1	42	N/A	80	32	29	76	267	Casing set to 78' concrete fell 50' and 42' in two pours
DS-27	4/3/08 12:00	4/1/08 10:52	24	75	26	79	49.1	54	N/A	85	41	50	72	144	No concerns; drilled 33' w/54; set casing to 76ft; 48" to bottom
DS-29	3/7/08 10:33	3/5/08 2:45	22	72	26	79	55.8	36	N/A	77	N/A	20	N/A	N/A	No Concerns
DS-30	2/29/08 14:45	2/28/08 10:05	8	46	17	62	28.7	30	N/A	65	20	12	33	279	Bulge @ depth 37 - 45' Slight bullet tip shape bottom 2'
DS-69	2/20/08 16:10	2/19/08 15:00	19	67	21	70	25.2	36	N/A	75	15	20	N/A	N/A	Partial pour / bullet end / low temp surface concrete
DS-70	2/26/08 13:11	2/25/08 13:38	22	72	22	72	23.5	36	N/A	65	41	17	33	194	Bulge 15-30' and 35-45'
DS-71	2/27/08 12:57	2/25/08 16:00	23	73	22	72	45.0	42	N/A	45	17	16	N/A	N/A	Bulge at toe near T1, T2, and T4
	2/28/08 12:31	2/25/08 16:00	23	73	22	72	68.5	42	N/A	45	17	16	N/A	N/A	
DS-72	2/28/08 11:19	2/27/08 10:03	14	58	18	64	25.3	42	N/A	30	19	11	25	234	Bulge @ depth 2-7' Slight neck @ depth 7-11'
DS-73	2/27/08 14:45	2/26/08 11:00	26	79	21	70	27.8	42	N/A	30	17	11	15	140	Slight neck near T1, T2, and T3 @ bottom of temp casing / depth 7-12' (approx 2 - 2.5" radius reduction)
	2/28/08 9:18	2/26/08 11:00	26	79	21	70	46.3	42	N/A	30	17	11	15	140	

Note: Some information was unavailable at the time of reporting marked as N/A.

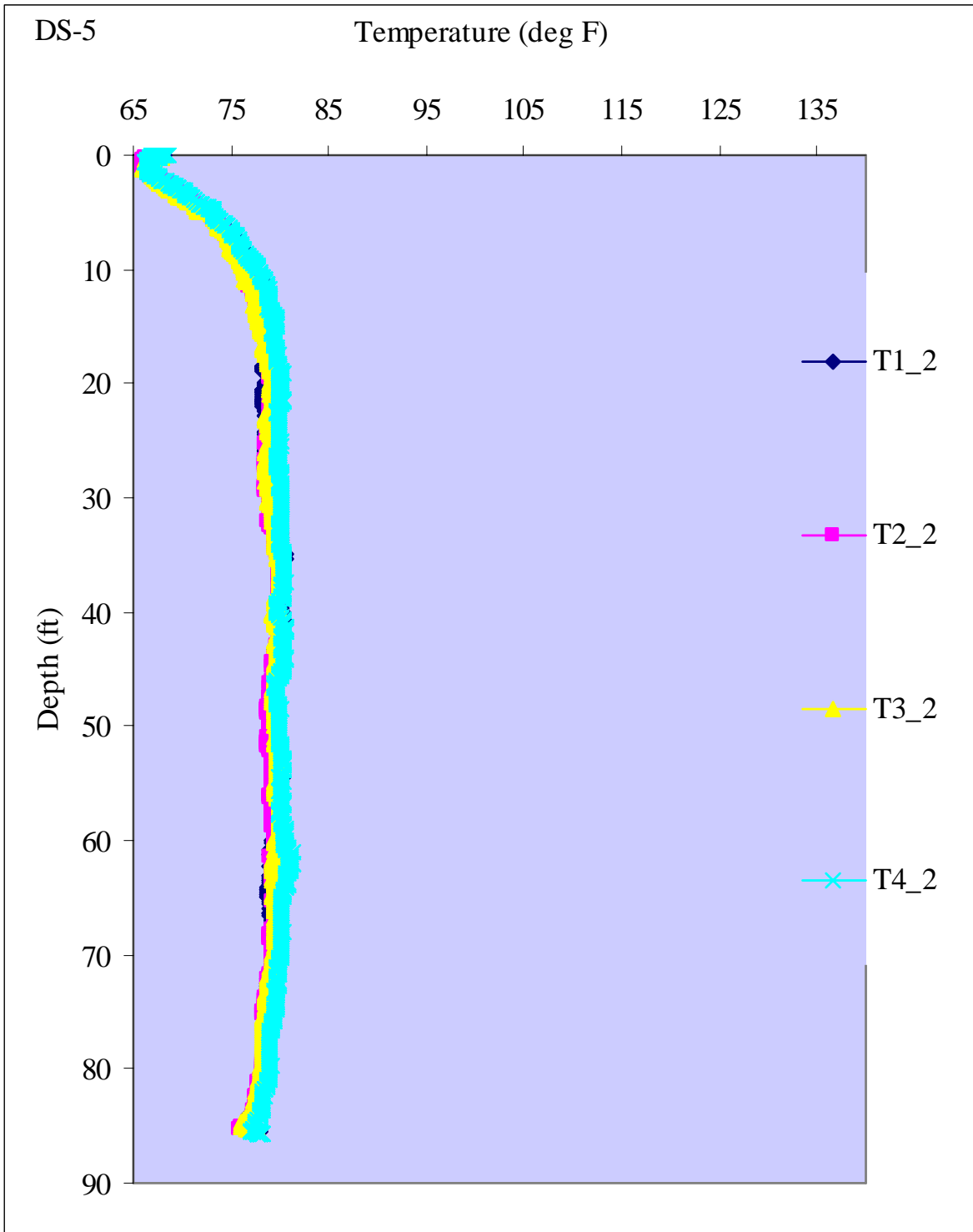


Figure 3-27. DS-5 Measured temperature traces for Tubes 1 through 4.

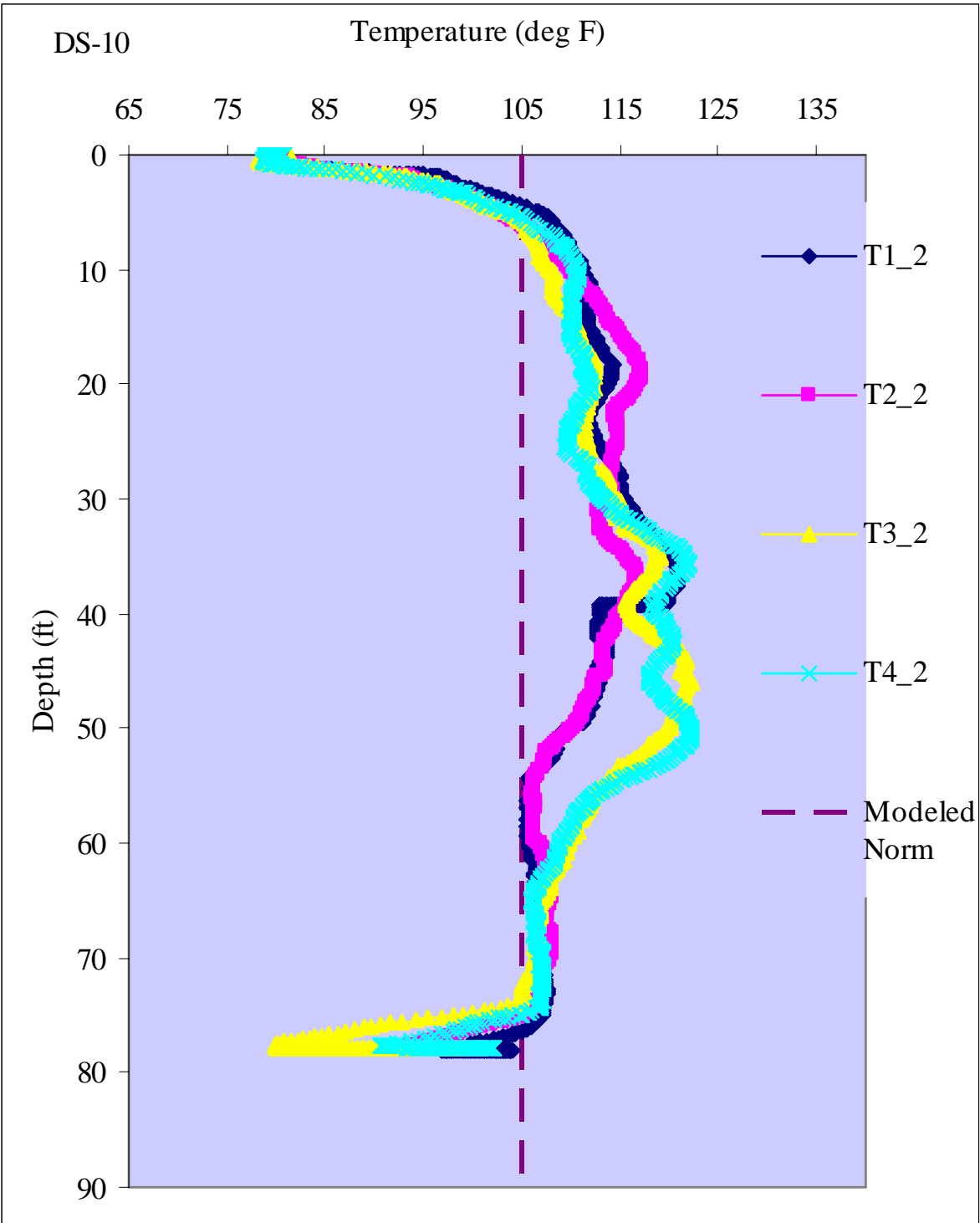


Figure 3-28. DS-10 Measured temperature traces for Tubes 1 through 4 compared to the model norm.

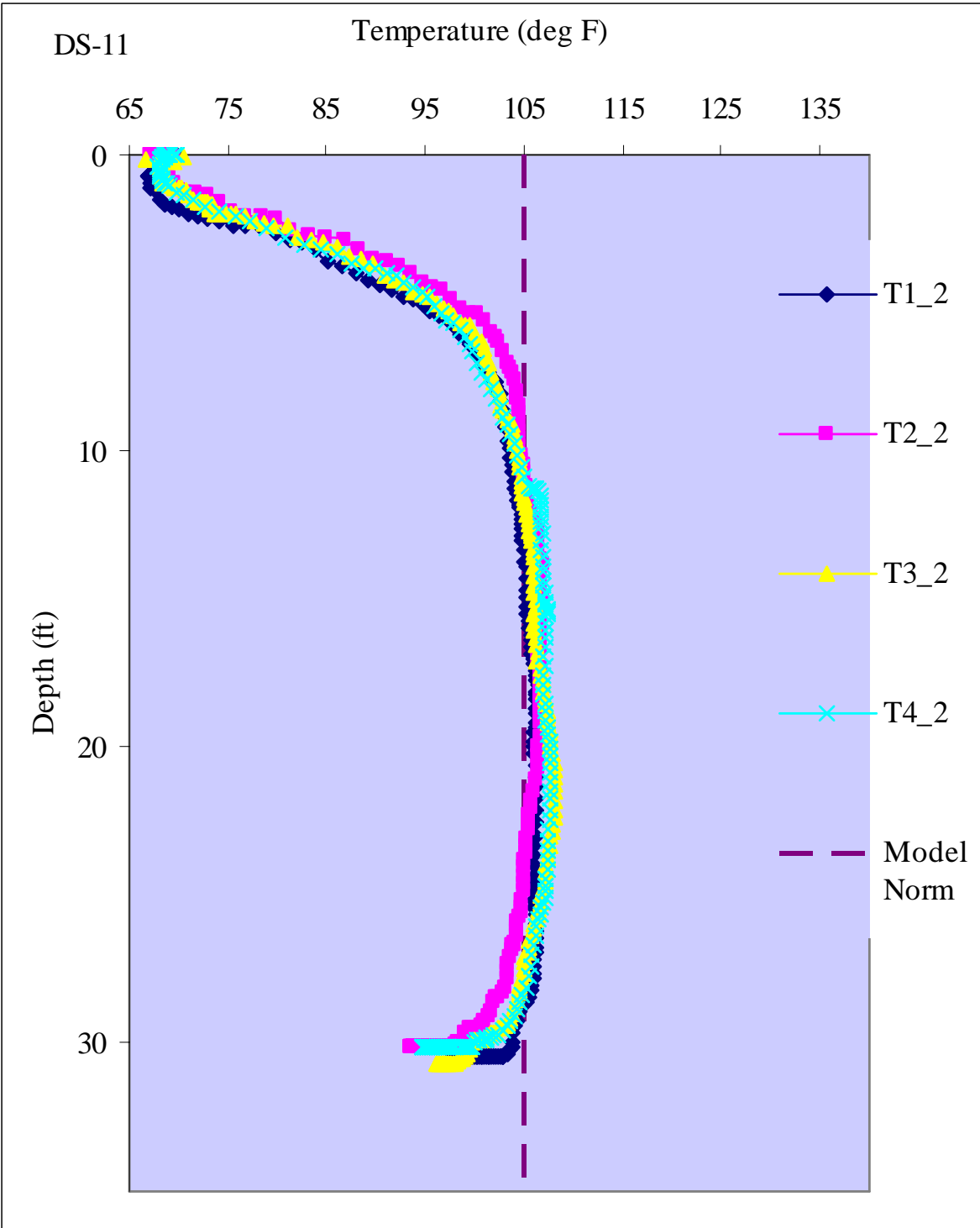


Figure 3-29. DS-11 Measured temperature traces for Tubes 1 through 4 compared to the model norm.

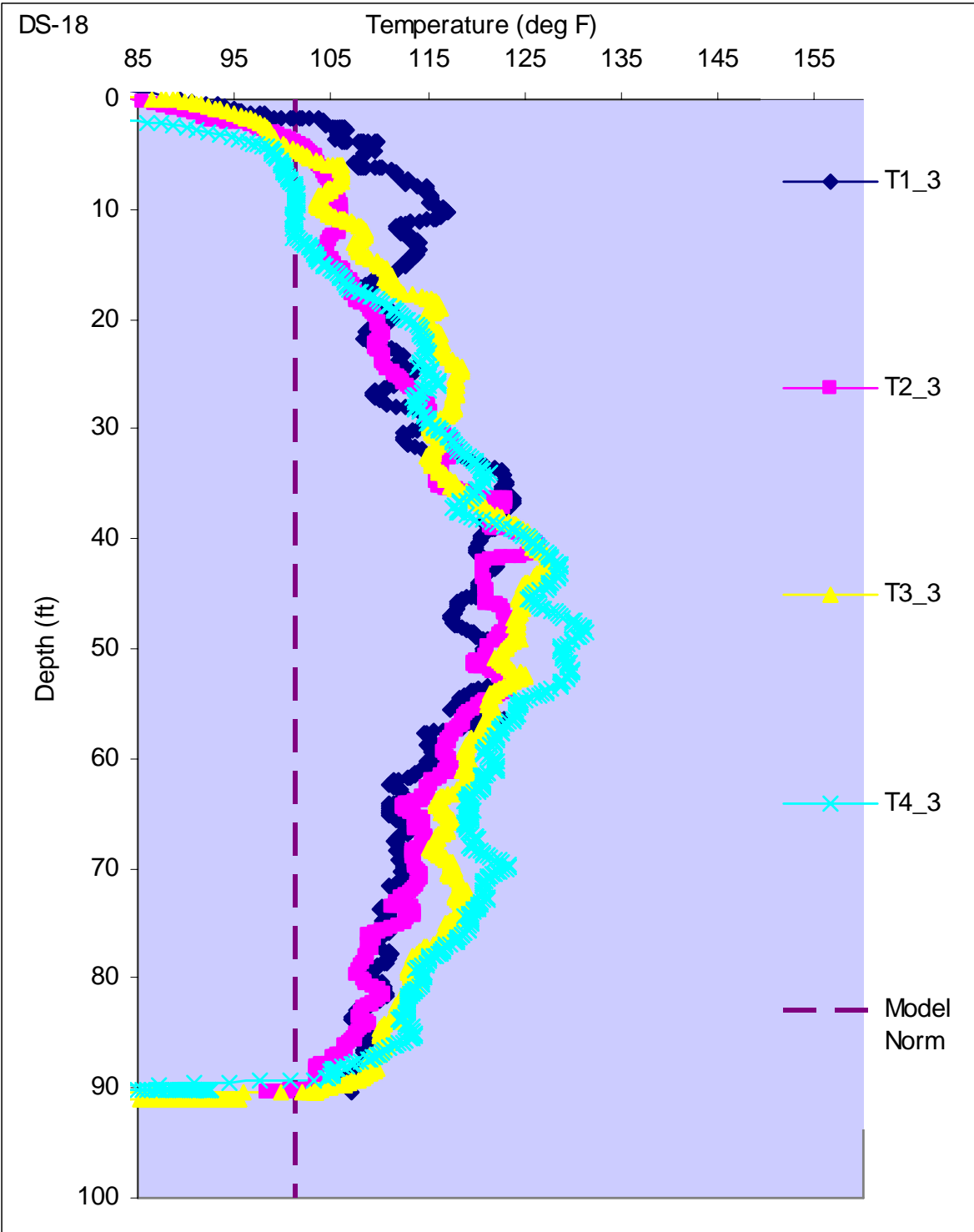


Figure 3-30. DS-18 Measured temperature traces for Tubes 1 through 4 compared to the model norm.

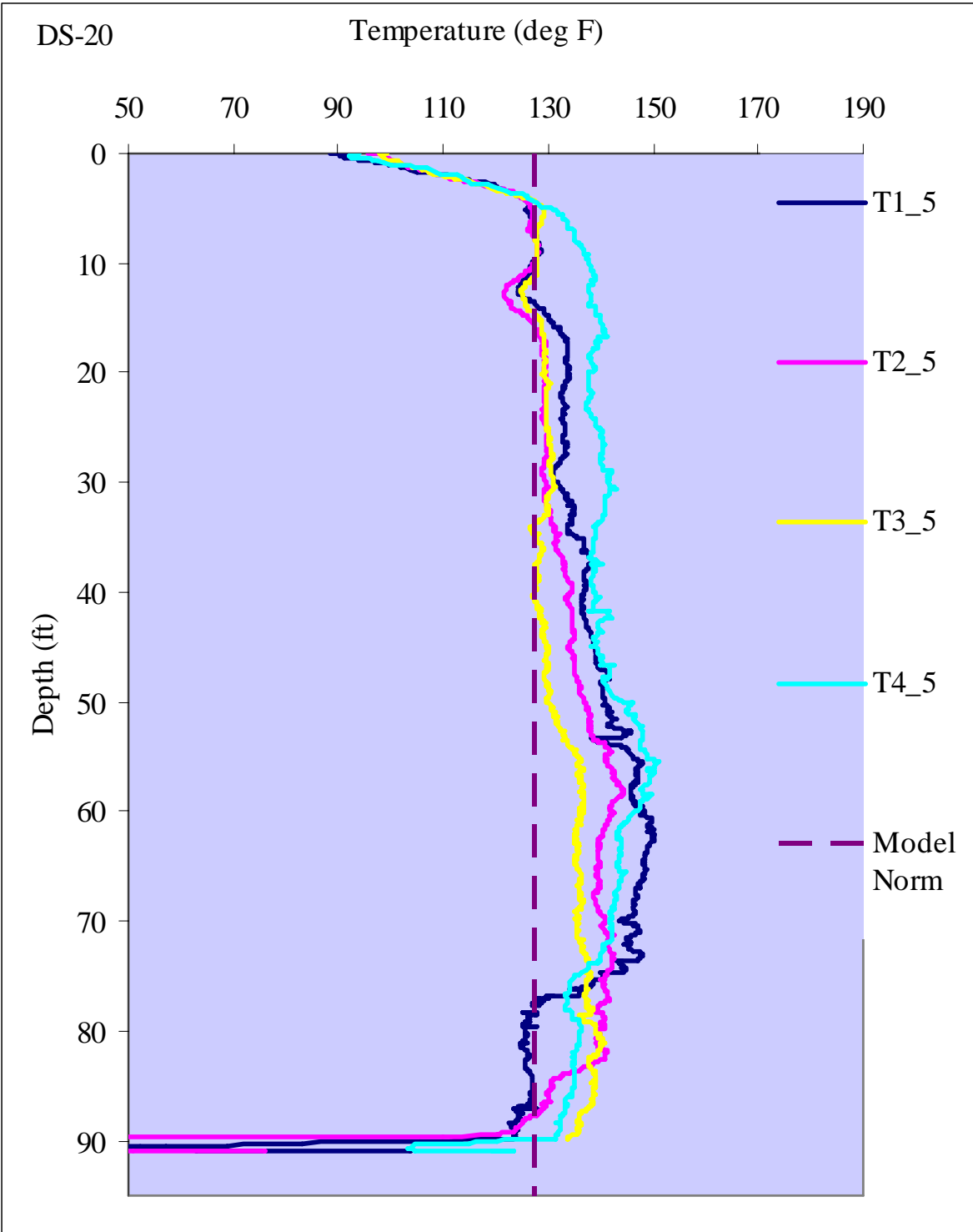


Figure 3-31. DS-20 Measured temperature traces for Tubes 1 through 4 compared to the model norm.

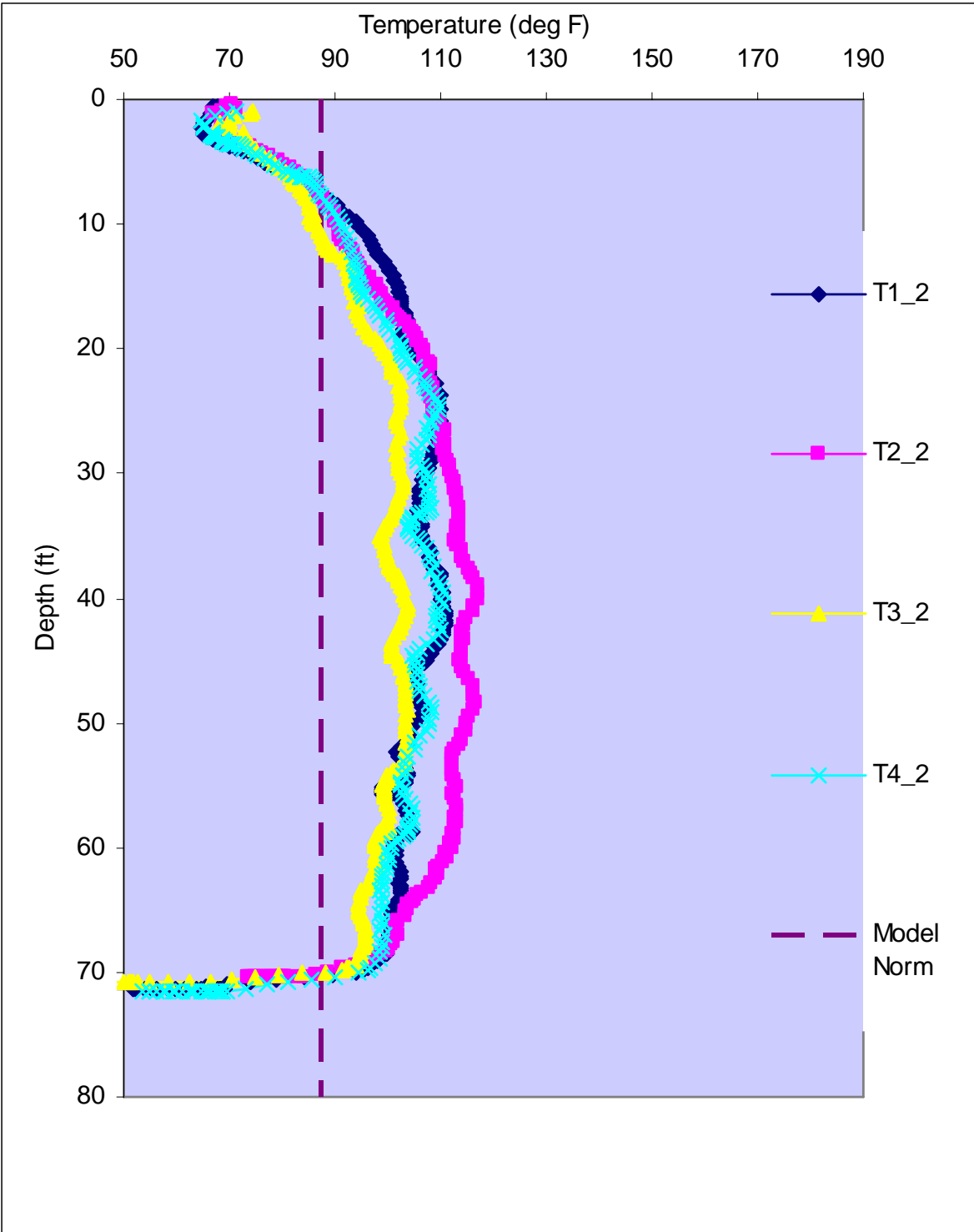


Figure 3-32. DS-23 Measured temperature traces for Tubes 1 through 4 compared to the model norm.

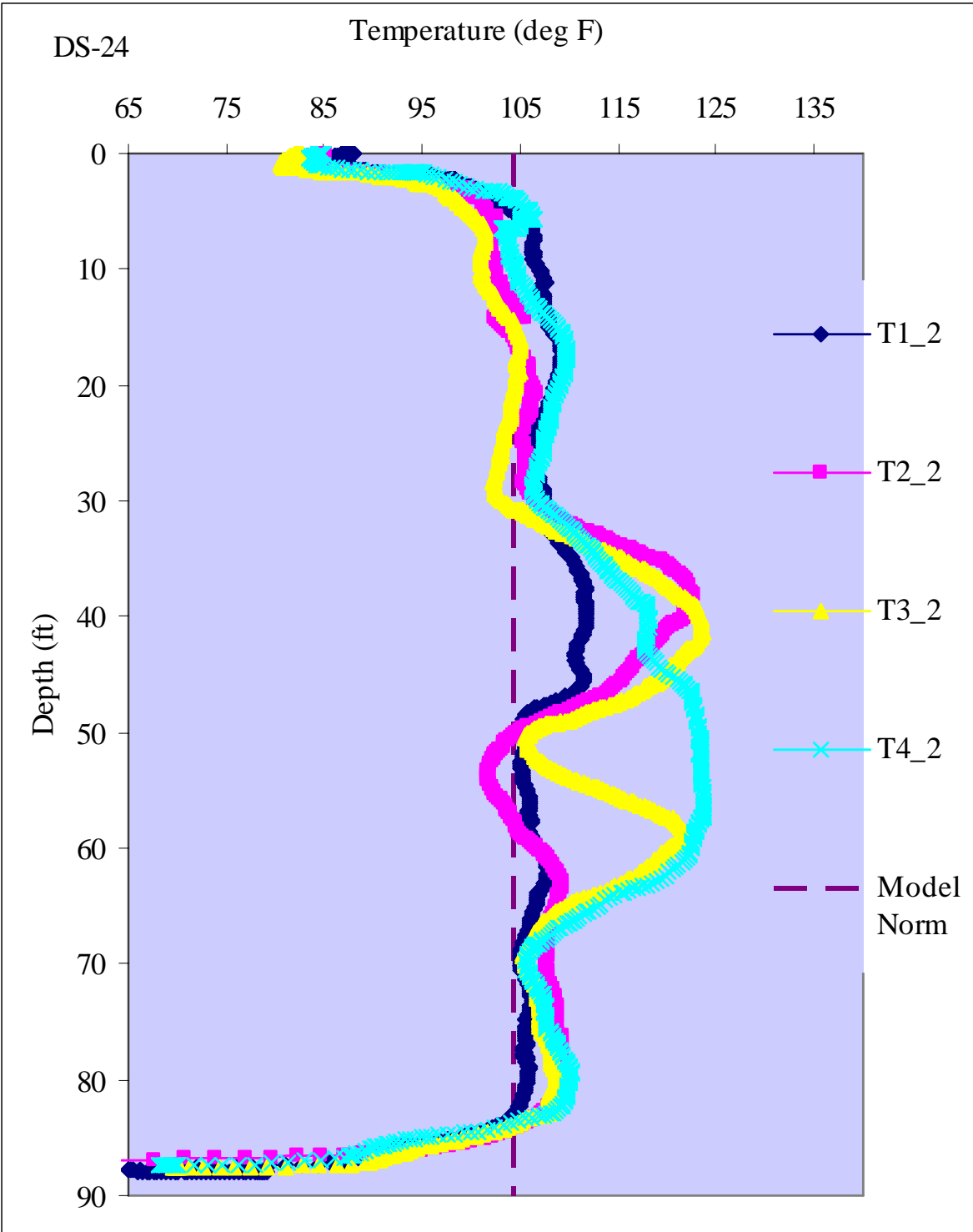


Figure 3-33. DS-24 Measured temperature traces for Tubes 1 through 4 compared to the model norm.

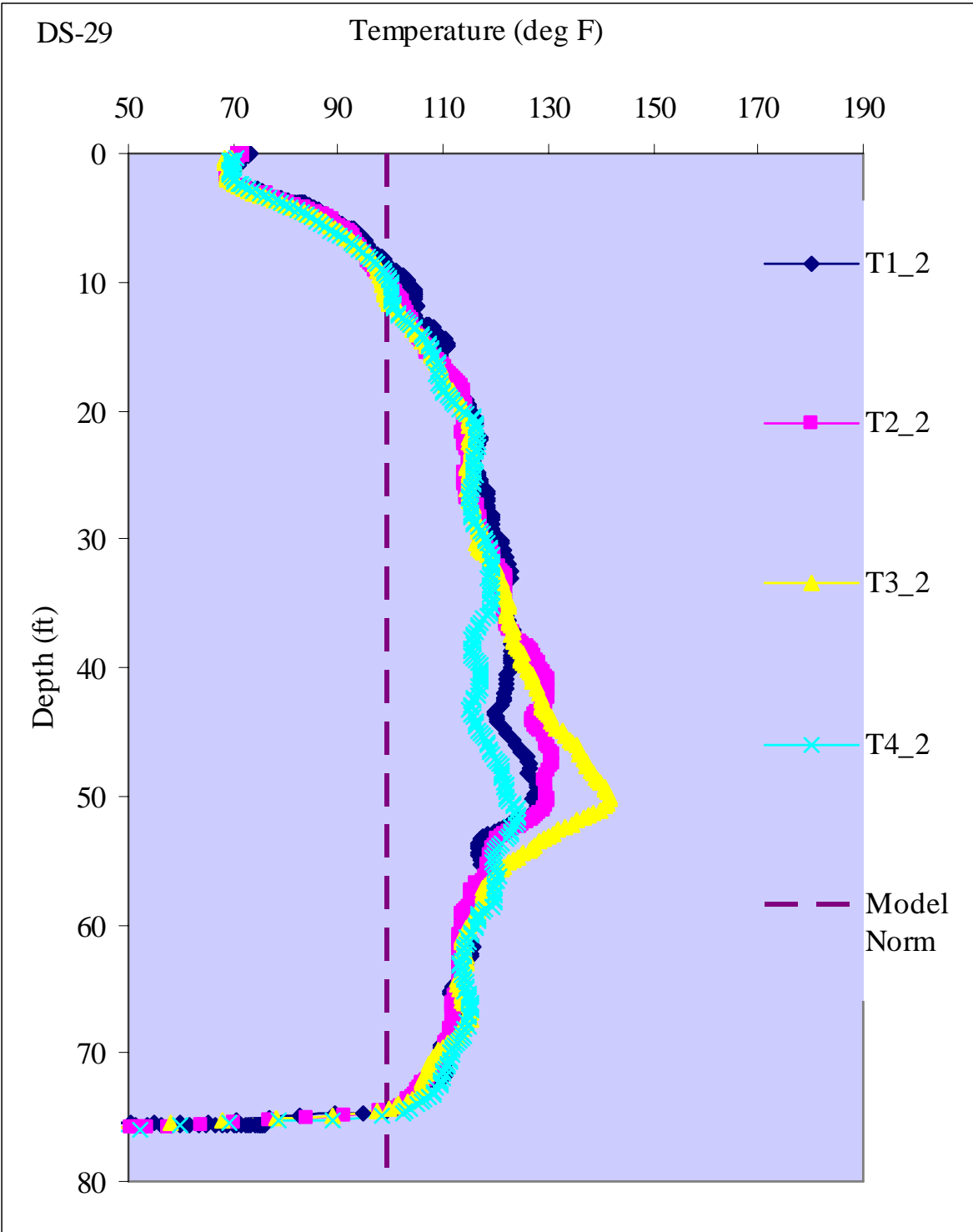


Figure 3-34. DS-29 Measured temperature traces for Tubes 1 through 4 compared to the model norm.

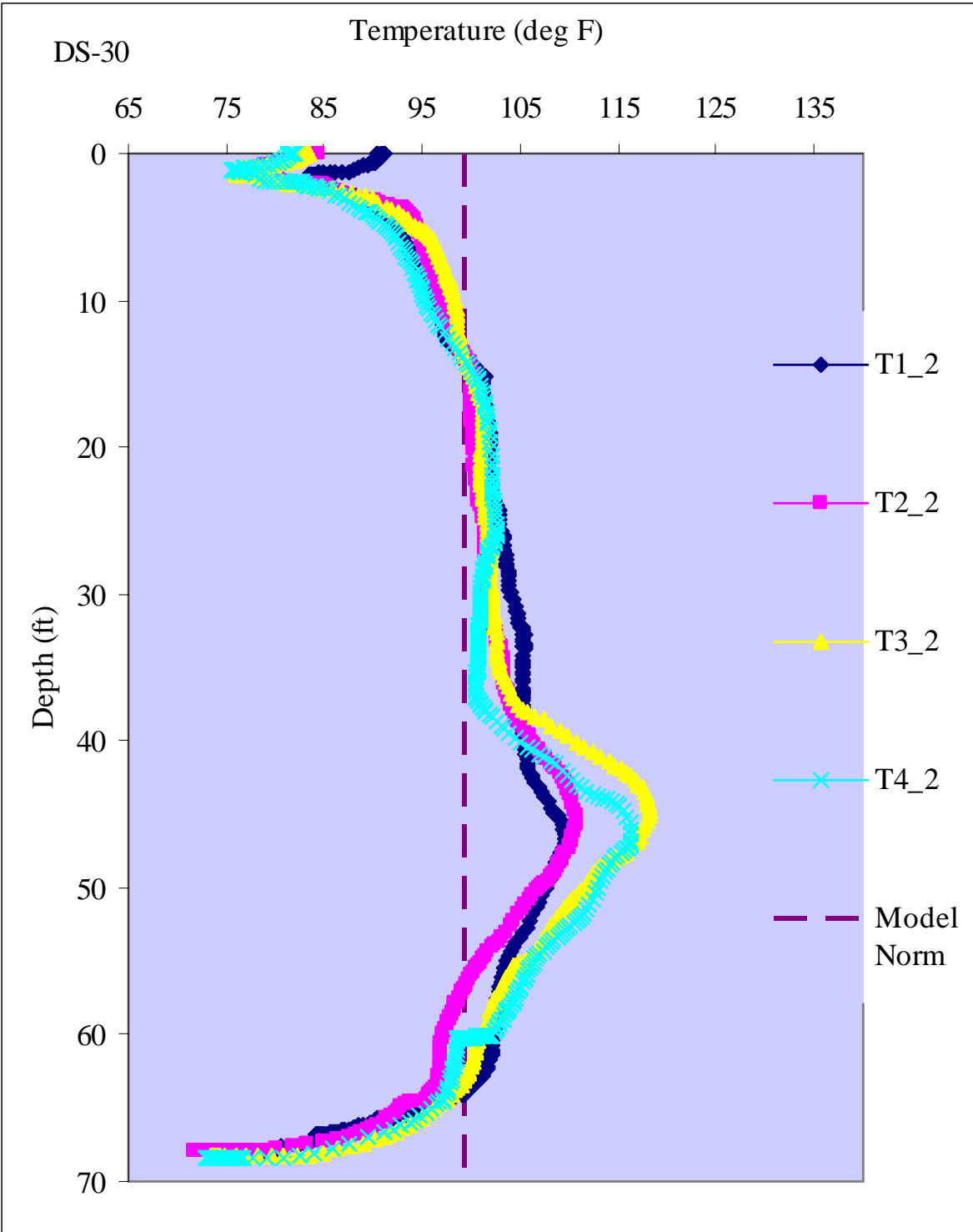


Figure 3-35. DS-30 Measured temperature traces for Tubes 1 through 4 compared to the model norm.

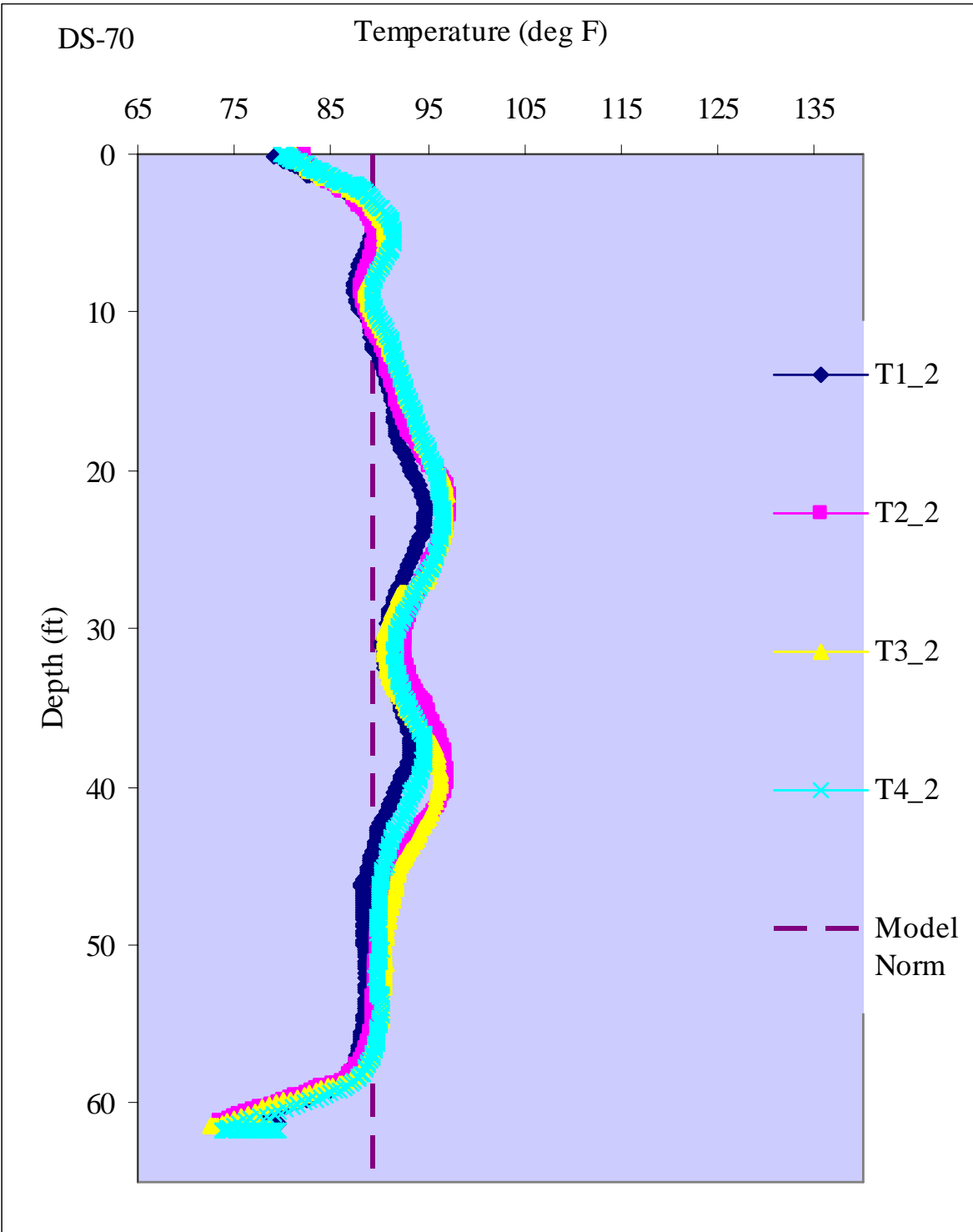


Figure 3-36. DS-70 Measured temperature traces for Tubes 1 through 4 compared to the model norm.

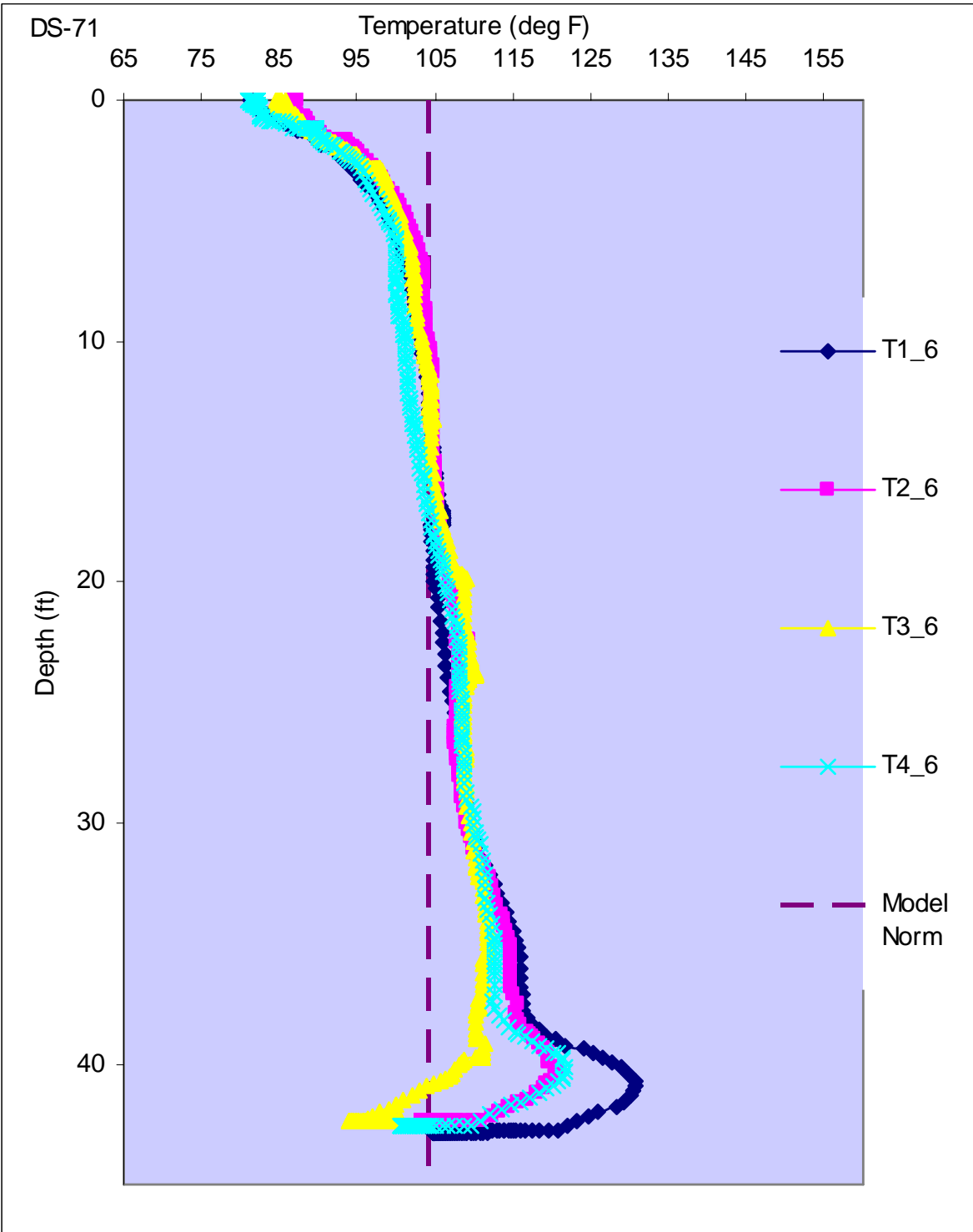


Figure 3-37. DS-71 Measured temperature traces for Tubes 1 through 4 compared to the model norm.

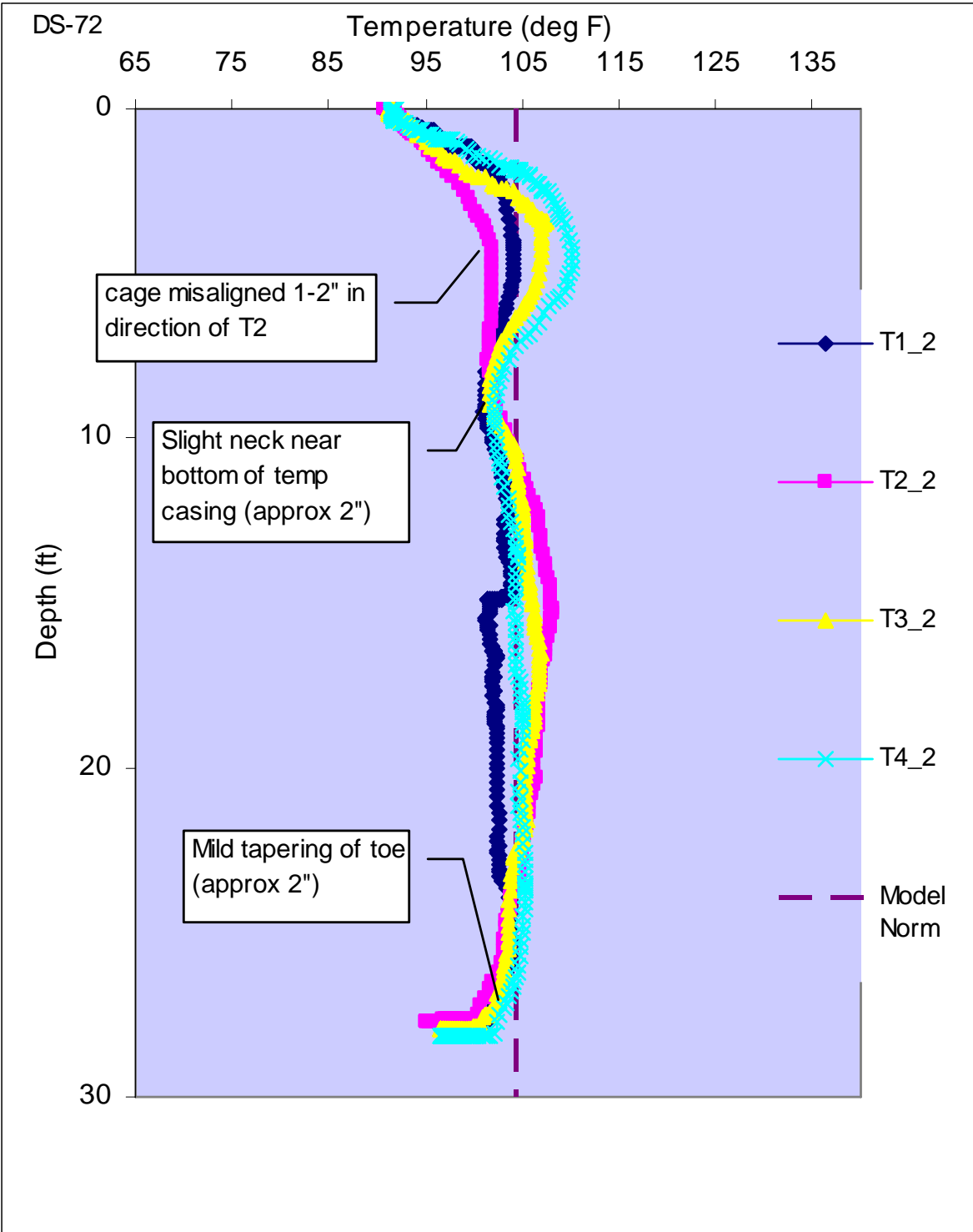


Figure 3-38. DS-72 Measured temperature traces for Tubes 1 through 4 compared to the model norm.

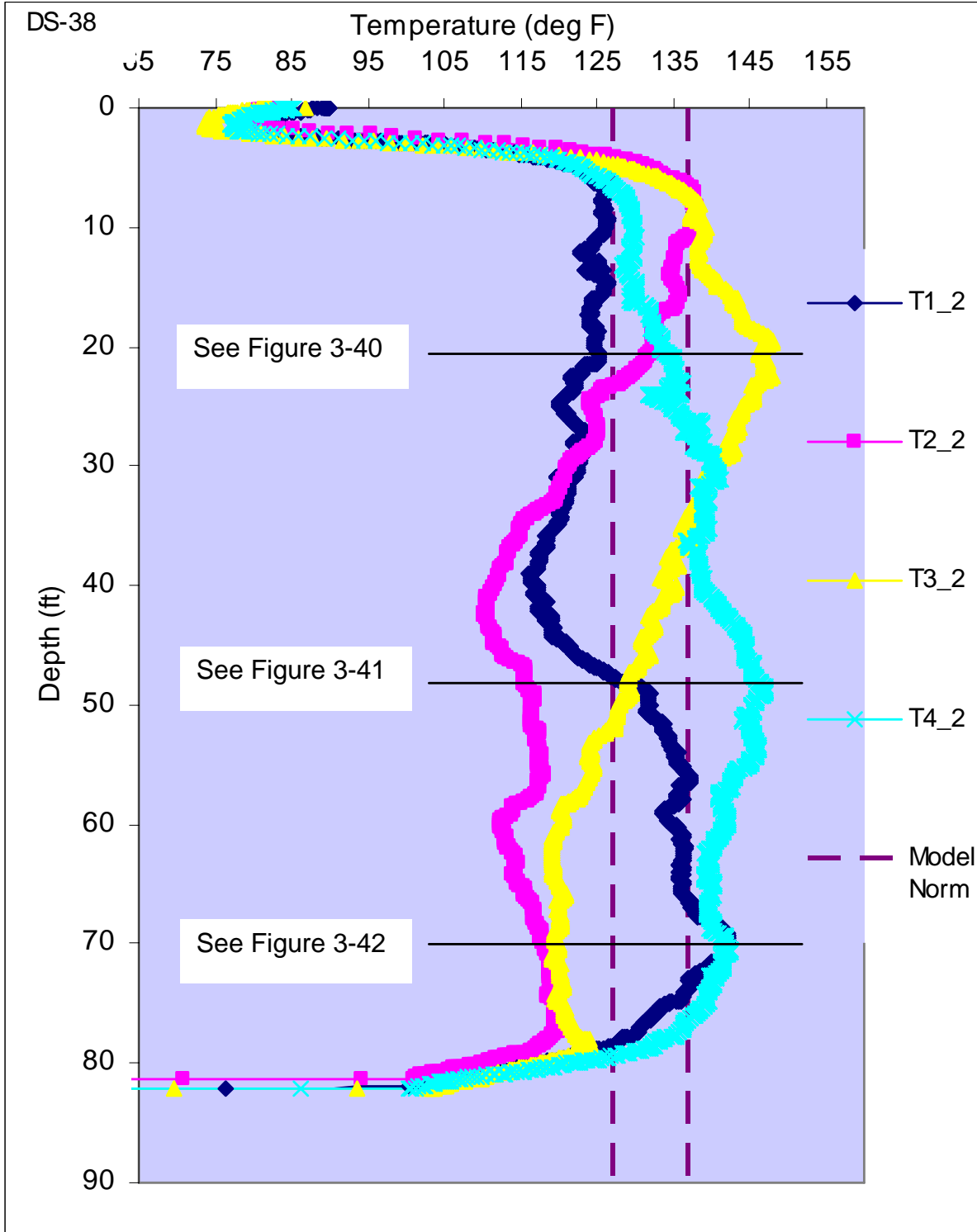


Figure 3-39. DS-38 Measured temperature traces for Tubes 1 through 4 compared to the model norm.

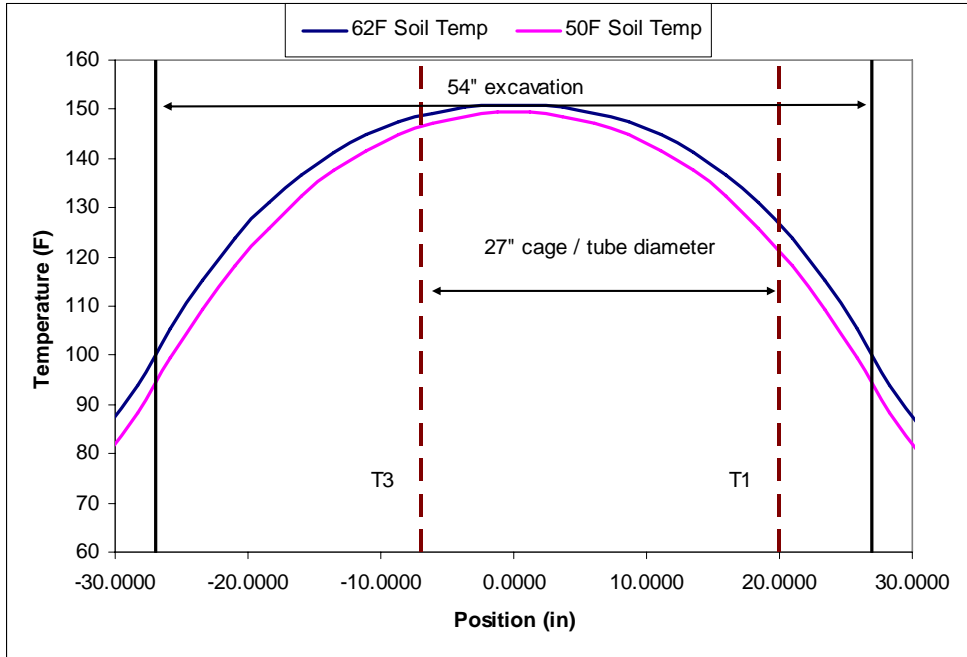


Figure 3-40. Cage alignment in 54" excavation at a depth of 21 ft.

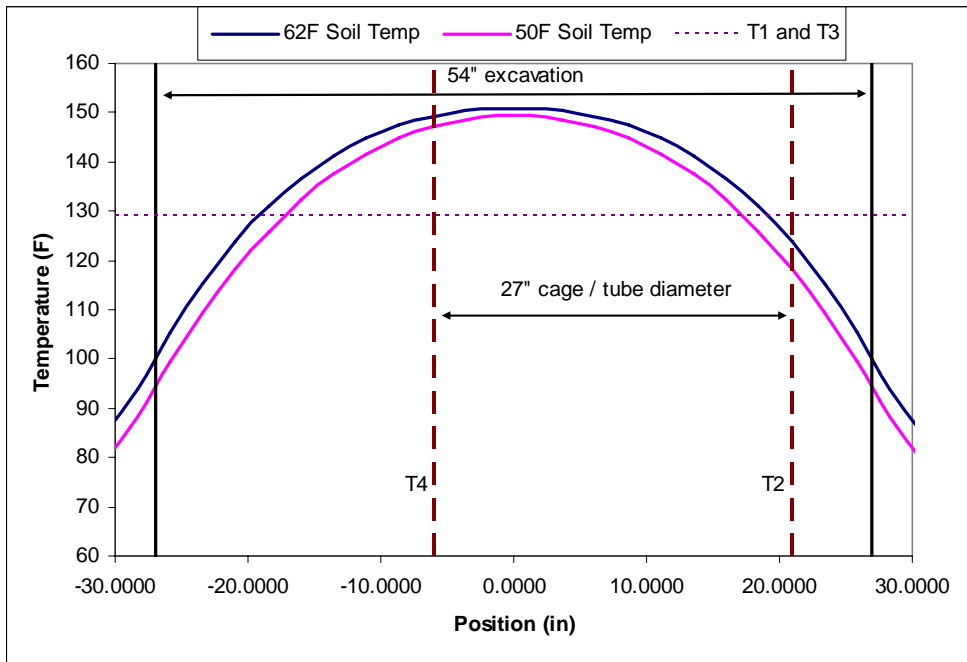


Figure 3-41. Cage alignment in 54" excavation at a depth of 48 ft.

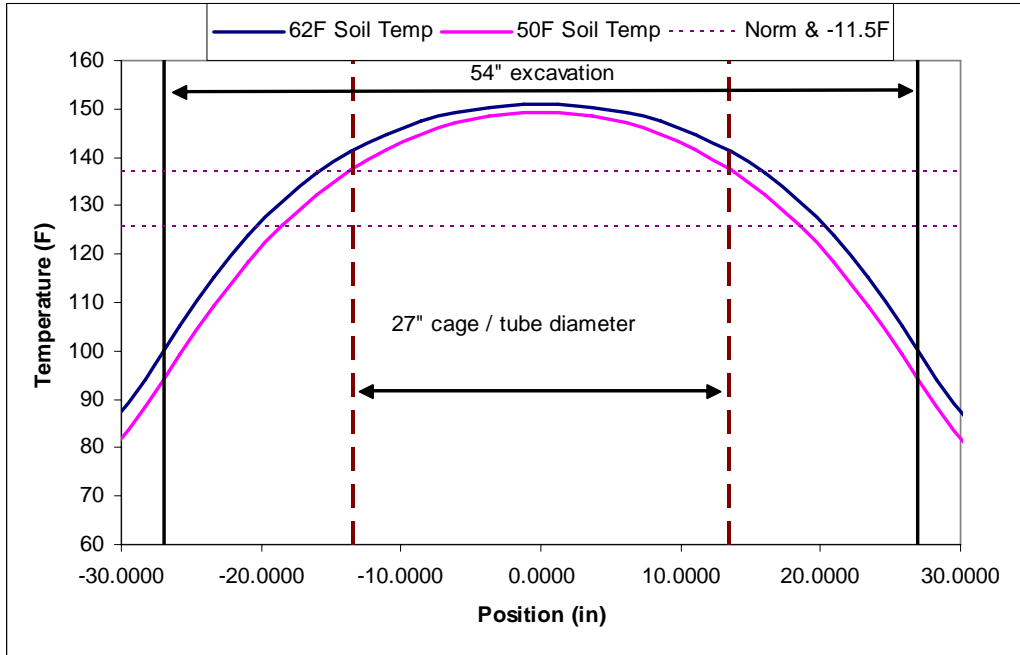


Figure 3-42. Cage alignment in 54" excavation at a depth of 70 ft.

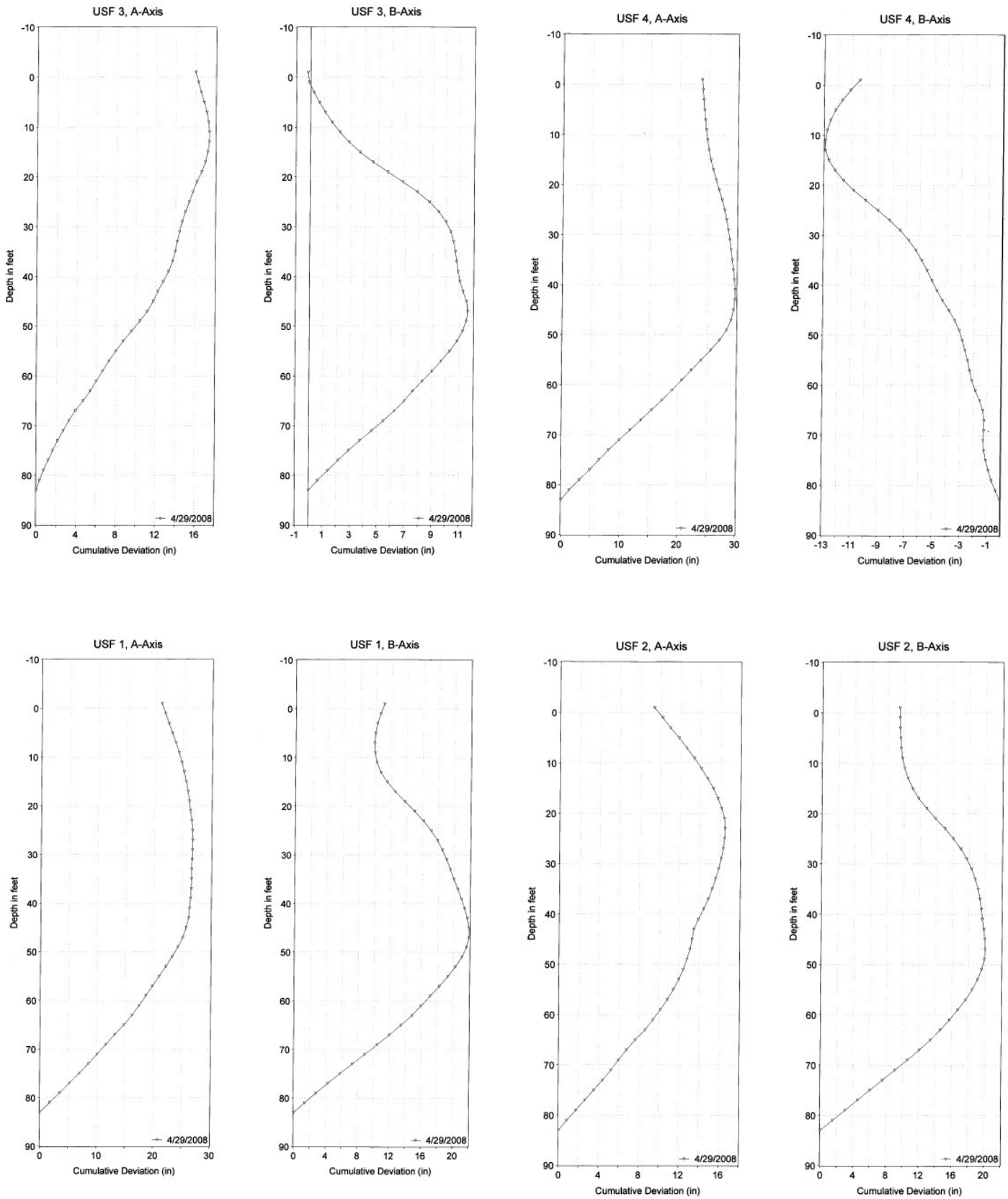


Figure 3-43. Inclinometer results for shaft DS-38.

Cage Alignment

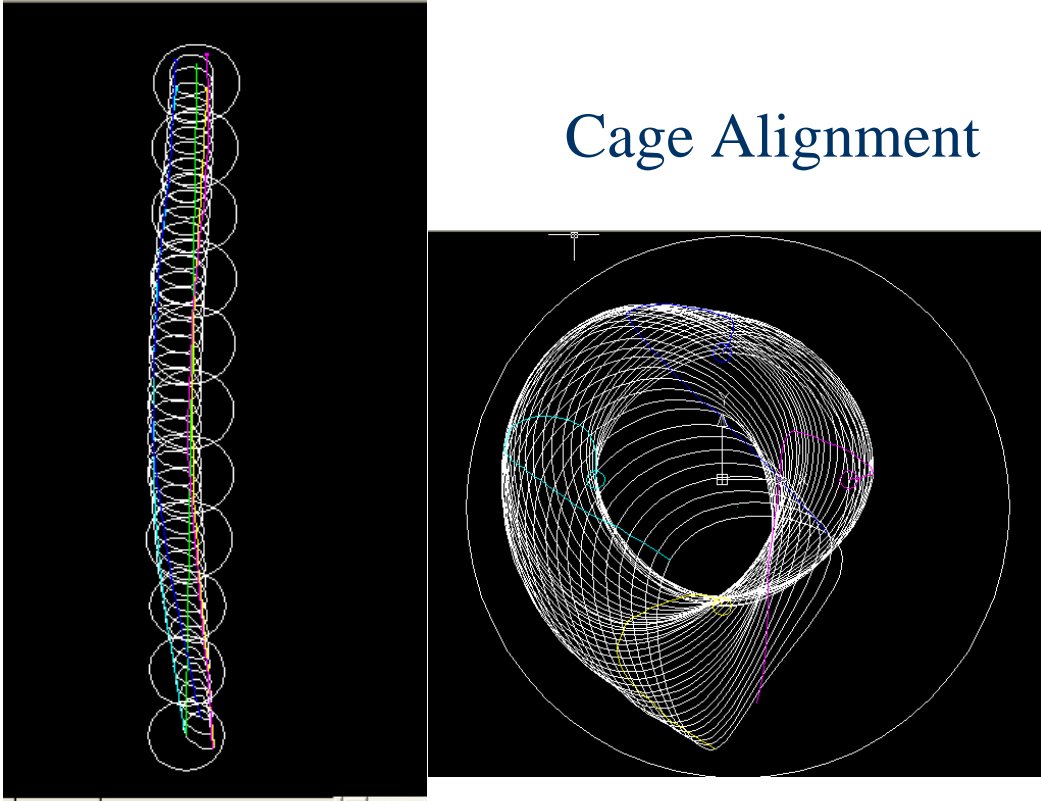


Figure 3-44. Inclinometer results plotted in 3D relative to the excavation.

Table 3-2. UF-290 Shaft Testing Details

Shaft I.D.	Date/Time Test Performed	Date/Time Cast	Casting Air Temp		Casting Conc Temp		Hydrati on Time (hrs)	Diam (in)	Tube Length (ft)	Rock Socket Length (ft)	Vol. Theory (cy)	Vol. Field (cy)	Vol.	Casing Depth (ft)	Casing O.D. (in)
			C	F	C	F							(%)		
152	10/22/08 17:30	10/21/08 10:35	25	77	30	86	30.9	48	48	14.3	24	29	123	15	55
154	10/22/08 18:20	10/21/08 10:00	23	73	29	84	32.3	48	51.5	13.5	26	31	119	15	55
171	10/22/08 16:30	10/21/08 14:00	27	80	33	92	26.5	48	52.7	16.1	27	38	142	15	55

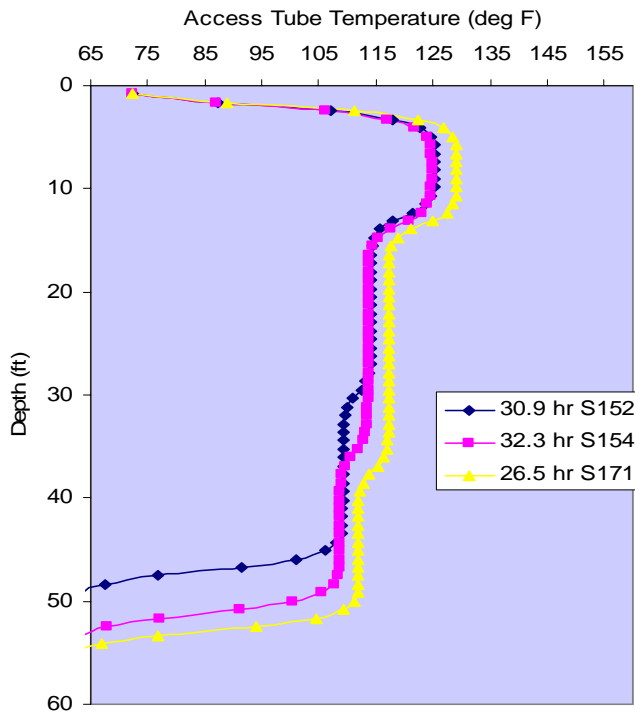


Figure 3-45. Normal access tube temperatures for the shafts tested.

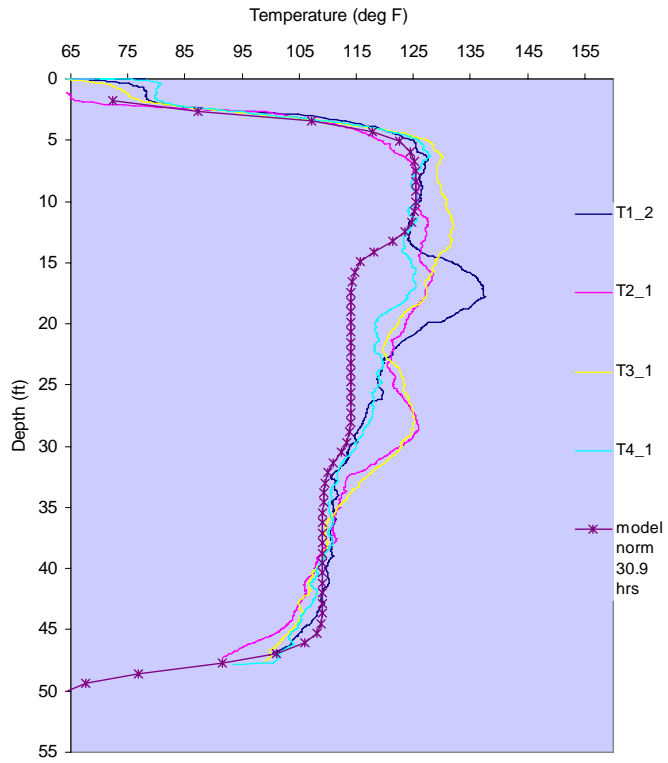


Figure 3-46. Thermal results for Shaft 152.

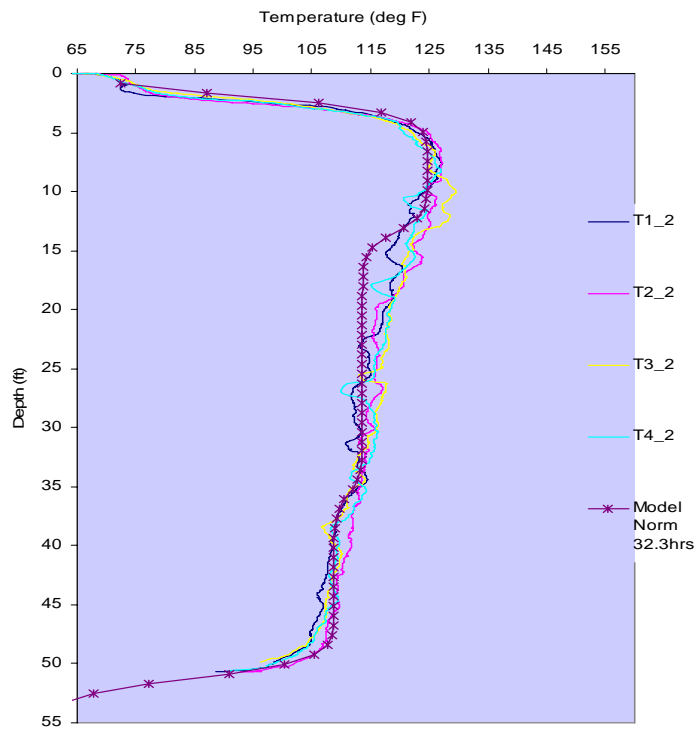


Figure 3-47. Thermal results for Shaft 154.

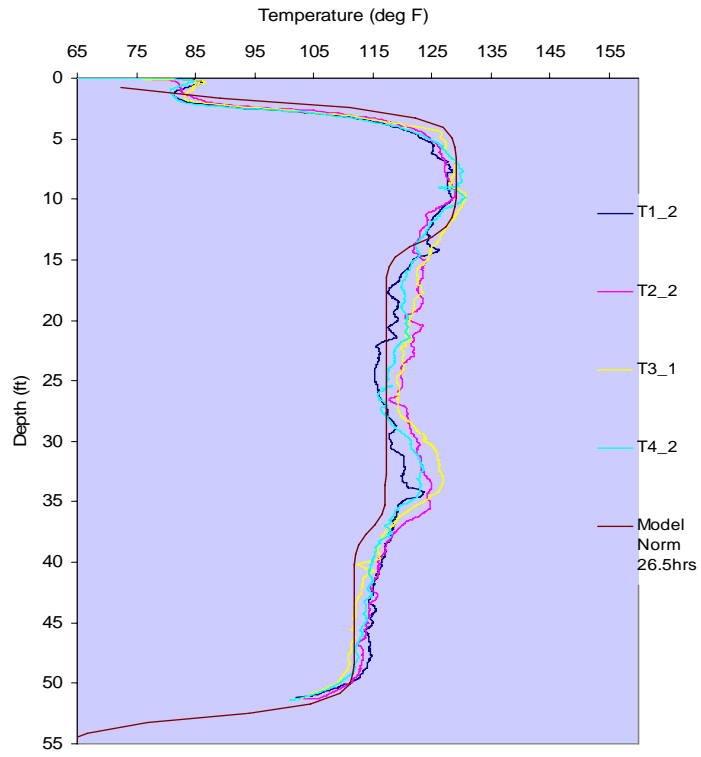


Figure 3-48. Thermal results for Shaft 171.

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Chapter Four: Voided Shaft Construction

4.1 Background of Concept

Large prestressed piles (2 – 3ft) are often cast with a cylindrical void aligned with the longitudinal axis of the pile to minimize construction weight while also reducing concrete cost. Larger diameter post-tensioned cylinder piles (3 – 6ft) develop enormous axial and bending capacity with only a 6 to 8in thick annular ring of concrete (concrete pipe pile). An even larger version is plausible in the form of a voided drilled shaft (Figure 4-1). Aside from obvious constructability issues, benefits include reducing concrete volume and pouring time which in turn would relax concrete supplier issues as well as reducing hydration heat generation.

4.1.1 Construction Considerations

Construction of drilled shafts, simply stated, involves excavating a hole deep in the ground with rotary type augers (hence the name drilled), inserting reinforcing steel into the excavation in the form of a cylindrical cage, and filling the hole with wet/liquid concrete which occupies the space from which the soil was excavated. To construct a shaft with a central void would involve normal excavation of the shaft's outer diameter followed by the insertion of a centralized steel casing (or similar) that can adequately maintain its position during the concreting of the annular volume. This may require that additional excavation of the smaller diameter to allow the inner casing to adequately seal below the bottom of the outer shaft diameter (Figure 4-2).

Concrete placement can be carried out with any method (full length pump truck hose or tremie) provided it can be easily moved during concreting to maintain constant concrete flow levels around the inner casing. Use of new high performance shaft concrete would certainly be advantageous. Alternately, placement from multiple tremies may be an option.

Inner casing installation, alignment, and overcoming potential buoyancy forces are perhaps the most significant obstacles to constructing voided shafts. The physics of buoyancy forces only provide a problem if the concrete can form a pressure face beneath the casing causing an upward force (Figure 4-2). Lateral concrete pressure will not induce buoyancy but rather will require sufficient casing stiffness such that it will not collapse. As there is little surface area on which upward pressure could act (open ended casing), the real issue is assuring concrete will not flow underneath and fill the inner casing. Therefore, the casing must form a seal with the bottom of the excavation in spite of the upward drag force that accompanies concreting.

One method of sealing the casing is socketing it beneath the toe of the voided shaft. This socket is not required to develop significant side shear with the inner casing but must provide a reasonable seal. Advancing the inner casing into the underlying strata could be performed by duplex drilling (drilling beneath the casing while advancing), vibratory, or

oscillatory installation. When slurry stabilization is to be used, duplex drilling would likely be preferred as vibratory installation could disrupt the integrity of the excavation walls. In most cases, cuttings would not need to be removed (or at least not completely) from the inner casing during its installation, nor would it be necessary to perform clean-out processes within the inner casing. When full length temporary casing is employed to stabilize the hole, duplex, vibratory, oscillatory, or a combination of these installation methods would suffice to install the inner casing.

An alternate method of providing a seal between the inner casing and the excavation bottom might include a flange at the base of the casing (rigid, flexible, or combination thereof) that would both center the casing at the toe and provide a flat surface on which the self weight of the shaft concrete would secure the seal (Figure 4-3). A combination of flange and socketing may be found most suitable in certain circumstances.

Centering the inner casing as well as the reinforcement cage is also important and can be achieved by attaching a simple frame work to the inner casing. If a flange assembly is used, the frame work is extended from and/or incorporated into the flange. Struts attached to this frame to provide the necessary stiffness serve dual purpose by providing cage centering via properly dimensioning their connection locations (Figure 4-4). This provides better assurance of the cage placement than the presently used plastic spacers which often are found floating to the top during concreting.

4.1.2 Strength Considerations

Strength reduction caused by the reduced cross-sectional area has little effect on the structural performance of the foundation element in that the soil resistance is typically the limiting parameter. Therein, the geotechnical capacity is only affected by the reduction in end bearing area which is not typically considered a significant capacity contributor in large diameter shafts. However, if the inner casing was initially plugged or plated this capacity could be regained.

Structurally, a centrally voided shaft would exhibit a reduction in axial capacity roughly proportional to the loss in cross-sectional area. In general, load cases involving lateral loads and overturning moments produce far more severe stresses but would only be mildly affected by the presence of the void. For example, a 9 ft (2.75 m) diameter shaft with a 4 ft (1.22 m) diameter void, 1% 60 ksi (410 MPa) steel, and $f'c$ of 5 ksi (34 MPa) would result in a axial capacity reduction in the range of 17% whereas the bending capacity would be reduced less than 1%. This is due to the minimal contribution to the moment of inertia and the associated bending strength provided by the more centrally located concrete material. Further, the 1% reduction does not consider the gain in bending capacity associated with the inner steel casing if permanent. If the axial capacity must be maintained, slightly increasing $f'c$ to 6 ksi (41 MPa) would suffice. Figure 4-5 shows the interaction diagram for the above example shafts as well as the 6 ksi voided option.

4.1.3 Curing Temperature Maintenance

The numerically modeled temperature responses of a 9 ft (2.75 m) diameter shaft with and without a 4 ft (1.22) diameter void are shown in Figure 4-6. The model parameters simulated summer ambient conditions (changing winter to summer) but used concrete heat parameters similar to the Ringling Causeway Bridge. Note that under those conditions the peak temperature increase in the non-voided shaft (Figure 4-6) is related to the difference in ambient temperature and the lack of thermal convection in saturated soil.

The voided shaft was modeled with the void (center of casing) filled with slurry which in turn attained the same peak temperature. This was well less than the recommended safe temperature, and temperature differentials momentarily approach but do not exceed 20 deg C. Recent unpublished results, using published cement heat parameters, also indicate that supplanting 50% cement with ground granulated blast furnace slag does not diminish either peak or differential temperatures in large diameter shafts, but increases the centroidal peak time lag.

Although the accuracy of the model has been verified with field data that supports the non-voided shaft temperature response, a voided shaft had not been constructed prior to the onset of this study and was therefore a primary focus.

4.1.4 Cost Effectiveness

Preliminary cost comparisons between the permanent steel casing required to maintain the void during concreting and the central concrete that would be displaced (not required) showed the concept to be cost effective even without the savings associated with the now unnecessary cooling system. Figure 4-7 shows that for void diameters greater than about 4.3 ft (1.3m) the cost savings from concrete not used offsets the cost of the steel casing. This assumes that the casing is permanent and no innovative method of inner formwork extraction has been devised.

For 9 ft (2.75m) shafts, voids larger than 4 to 5 ft (1.2 to 1.5 m) are not likely to be considered as an annular thickness of 2.5 ft (0.75 m) is envisioned to be the practical lower limit for construction. This leaves approximately 2 ft (0.6 m) between the inner casing and the reinforcement cage for a pump truck hose or tremie to negotiate the concrete placement process. As a result, the Figure 4-7 results show a breakeven cost for 4.3 ft (1.3 m) voids which would be reasonable for 9 ft (2.75 m) shafts. For larger diameter voids (larger shafts) cost savings can be realized with additional savings from no required cooling system. Further benefits accompany voiding shafts in the assurance of long-term durability.

4.2 Full Scale Demonstration Construction

The logistics of constructing a voided shaft were fleshed out in a full scale demonstration project conducted as part of this study. This was made possible in part with the cooperation of a local drilled shaft contractor who provided a site, personnel and

equipment. A thorough review of the construction process and the findings of its effectiveness in controlling mass concrete effects in large diameter drilled shafts is presented herein.

4.2.1 Preparation

The time-line for the voided shaft construction was planned to allow for the worst case peak temperature conditions (mid to late summer). With this in mind, and ground temperature conditions at the hottest in September (Figure 4-8), construction was scheduled accordingly. Preparation for the construction involved: scheduling with contractor (R. W. Harris, Inc.), site layout, fabrication of both the reinforcement cage and central casing, ground and cage access/monitoring tube installation, and installation of thermocouples.

Scheduling was based on a mid to late September pour when an appropriate drill rig and crew were between projects. The test site was located at R.W. Harris, Inc., 12300 44th St. N. in Clearwater, FL (Figure 4-9). As a result, all preparation was done sufficiently in advance such that the research team was ready at a moment's notice of an available crew.

The reinforcement cage consisted of 36 longitudinal bars equally distributed inside 83" diameter #5 stirrups with a spacing of 12 inches on center. The reinforcement cage was outfitted with 9 - 26ft long, 2 inch Schedule 80 PVC pipe for thermal testing and cross-hole logging. Thermocouples were placed on three of the monitoring tubes (120 degree separation) at the top, middle, and bottom of the tubes. Figure 4-10 shows the reinforcement cage with monitoring tubes. The central casing selected has a 46 inch outer diameter with ½ inch wall thickness and was 30.5 ft long.

Ground monitoring tubes (4 total) and a well-point were installed by FDOT District I drill crew. The spacings of the monitoring tubes were positioned at 1/4, 1/2, 1, and 2 diameters from the proposed edge of shaft. Figure 4-11 shows the drilling of the ground monitoring tubes. Thermocouples were placed on the ground monitoring tubes located at the proposed midpoint of the shaft (12.5 ft depth).

The central casing was equipped with thermocouples and an access tube centered using struts welded to the inside (Figures 4-12 and 4-13).

4.2.2 Excavation and Concreting

Construction of the voided shaft took place on September 25, 2007 at the R.W. Harris test site. The entire procedure was broadcast via webcam from the USF geotechnical webpage for those unable to attend/visit the site. Records of the construction sequence, thermal testing and long-term temperature monitoring were also posted and updated every 15 minutes to <http://geotech.eng.usf.edu/voided.html>. The ability to post information to a web-page allows for a daily progress report of the construction, data collection, and overall performance of the testing.

Both webcam and time lapse photography were/are archived on that site which show the hour by hour construction. The following time-lines can be seen from archival footage shown on-line at <http://geotech.eng.usf.edu/timelapse.html> and <http://geotech.eng.usf.edu/RWHwebcam.html>

Voided Shaft Construction Time Lapse Photos / WebCam Archives (on line)

- *7:30-9:00am Setup / Excavation*
- *9-10:00am Excavation / Clean-out*
- *10-11:00am Clean-out / Settle / Final Clean-out*
- *11-12:00pm Cage Placement*
- *12-1:00pm Central Casing Placement / Concreting*
- *1-2:00pm Concreting / Surface Casing Removal*
- *2-3:00pm Clean up / Instrumentation*
- *3-4:00pm Instrumentation*

4.2.2.1 Excavation. The general excavation process entailed: installing a slightly oversized surface casing (10 ft diameter, 8 ft long, and embedded 7 ft); dry excavation with a 9ft diameter auger for the first several feet; after which, polymer slurry was introduced to stabilize the excavation; slurry stabilized excavation proceeded without issue down to a depth of 25ft; followed by a multi-stage clean out process (bucket was used to scrape the bottom of debris immediately after final auger depth and then again after a 30 minute wait period). Figure 4-14 shows this process.

4.2.2.2 Cage Placement. The reinforcing cage was picked at two locations to avoid excessive bending (Figure 4-15). Locking wheel cage spacers (12” diameter) were placed at the top, bottom, and middle of the reinforcing cage to provide 6 inches of clear-cover (Figure 4-16). The reinforcing cage was hung in-place so that the finished concrete would be level with the top of the cage.

4.2.2.3 Central Casing (Full Length Void). The 46 inch outer diameter steel casing (30.5ft long) was set into the center of excavation with a crane (Figure 4-17). The self-weight of the steel casing penetrated the soil between 3-6 inches. The penetration of the casing into the soil prevents concrete from entering the void area (one of the sealing options presented above). To prevent the top of the casing from shifting around during the initial concrete pour, a back-hoe bucket was used to hold the top of the casing (in practice, struts would be welded between the surface and central casing to assure concentric location (Figure 4-18).

4.2.2.4 Concrete Placement. A double tremie system was used to place concrete on opposite sides of the annular portion of the excavation (Figure 4-19). The concrete specifications called for a standard 4000 psi mix with 8 inch slump using #57 stone mix design. This was felt to be the most representative and perhaps the least flowable shaft concrete. During the concrete placement, the concrete level at three points around the shaft were measured to ensure concrete was flowing around the void and through the reinforcing cage.

4.2.2.5 Surface Casing Removal. The temporary surface casing was removed after final concrete placement. Two boom trucks were used to remove the casing (Figure 4-20).

4.2.3 Long-Term Monitoring

As soon as the site was clear of construction equipment all thermocouples were attached to a Campbell Scientific data logger (Figure 4-21). The data logger allowed for real-time remote monitoring of the temperature. The system updated a data file every fifteen minutes via a Verizon cellular uplink. The data was processed and posted by the host server at USF after every upload and could be viewed on-line at the research web page.

Thermocouples installed in the drilled shaft were monitored via a real-time cellular data collection system. Figure 4-22 shows the data as shown on the USF website. The battery voltage was also being monitored real-time to ensure the system voltage did not drop below 11.6V (Figure 4-23). Once the voltage drops below 11.6 volts, the data collection has approximately 8 hours of life. The Campbell Scientific data collection system is equipped with a solar panel to help sustain the battery voltage. However, the cellular uplink requires a large amount of power to communicate. Therein, two trips were made over the duration of the monitoring to the field to charge the battery cells. The data collection power system was later optimized prior to going to the I35W site discussed in Chapter 2.

4.3 Post Construction Testing

Aside from the long-term monitoring program implemented immediately upon shaft construction, a series of post construction tests were initiated using the Thermal Integrity Test system using the access tubes installed in both the shaft cage and the surrounding ground. These two series of tests were referred to as Thermal Integrity Testing and Ground Temperature Profiling. Although the procedure for both test series is identical, the review of the data has two distinct purposes: shaft integrity evaluation and ground/soil thermal property evaluation.

Thermocouples installed in the drilled shaft were monitored with the Campbell Scientific cellular data collection system and an Omega 220 data logger. The Campbell Scientific data system was collecting real-time thermocouple data every 15 minutes which can be downloaded remotely as previously discussed. The Campbell Scientific system monitored 25 thermocouples within the shaft and surrounding soil while the Omega 220 data logger collected the remaining 2 soil thermocouples (1D & 2D).

Figure 4-24 annotates much of the data from Figure 4-22 according to gage locations. Note that ground temperature measurements show elevated temperatures for months afterwards in spite of the cooler weather. The soil temperature 2D away from the shaft showed a consistent temperature profile and can be considered the datum or outer edge of the zone of influence.

4.4 Results

Prediction of the peak temperature in the shaft where made prior to the field construction (Johnson and Mullins, 2006); wherein the predicted temperature would peak at 138 deg F (Figure 4-6) at approximately 24hrs. The measured temperature for the voided shaft (Figures 4-22 and 4-24) confirmed those model results.

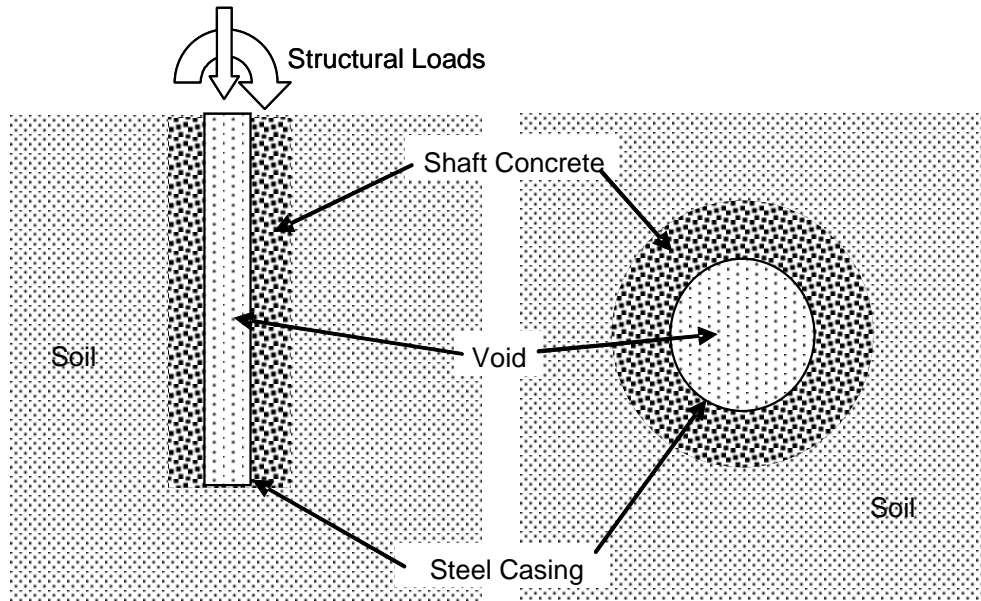


Figure 4-1. Conceptual schematic of a voided drilled shaft, in profile view (left) and plan view (right).

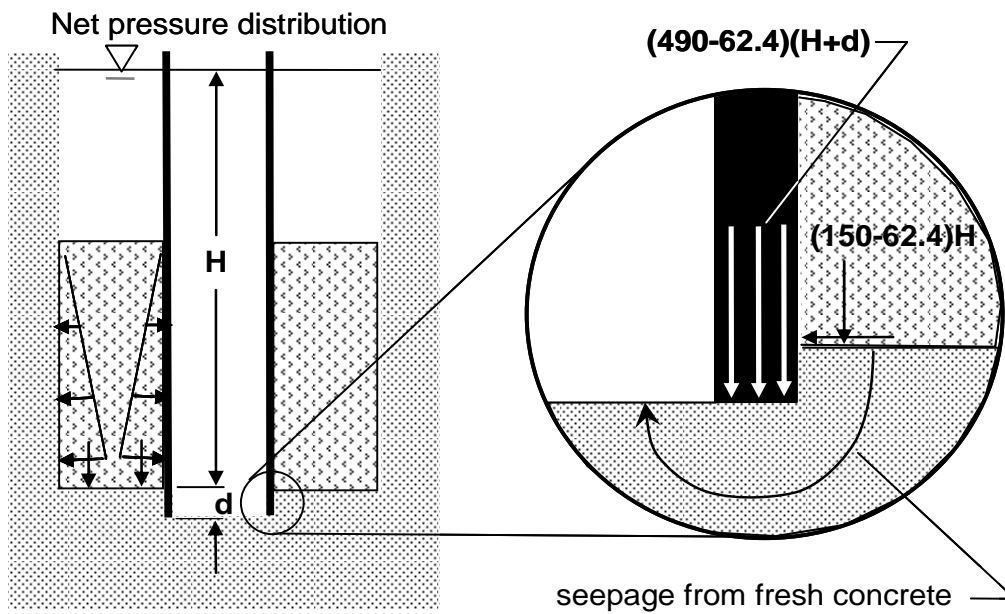


Figure 4-2. Net hydrostatic pressure distribution during construction

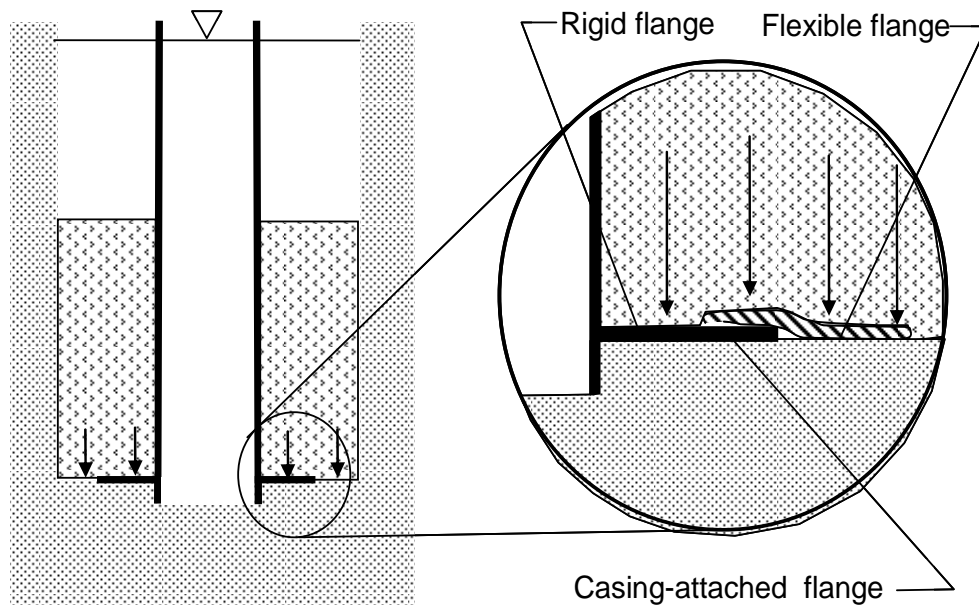


Figure 4-3. Figure 8. Rigid and flexible combination sealing flange is attached to the void casing, and engaged by the slurry and concrete load.

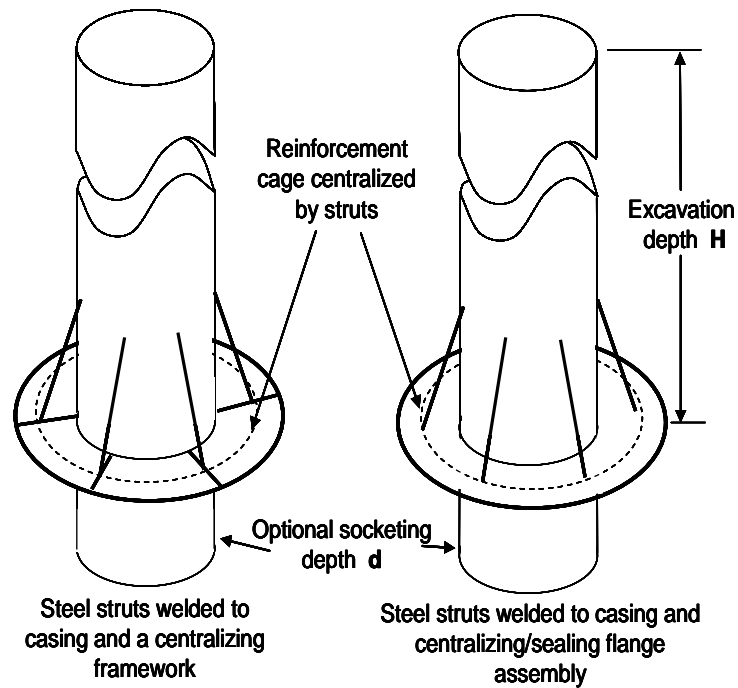


Figure 4-4 Figure 9. Void (inner) casing and reinforcement cage piloting framework.

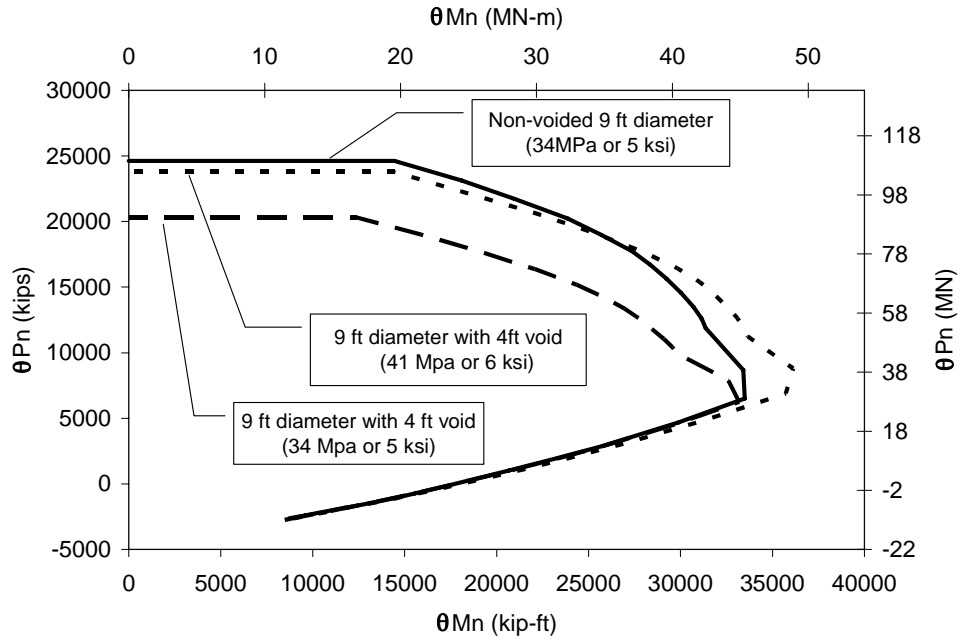


Figure 4-5. Interaction diagram of 9ft (2.75m) diameter voided and non-voided shafts.

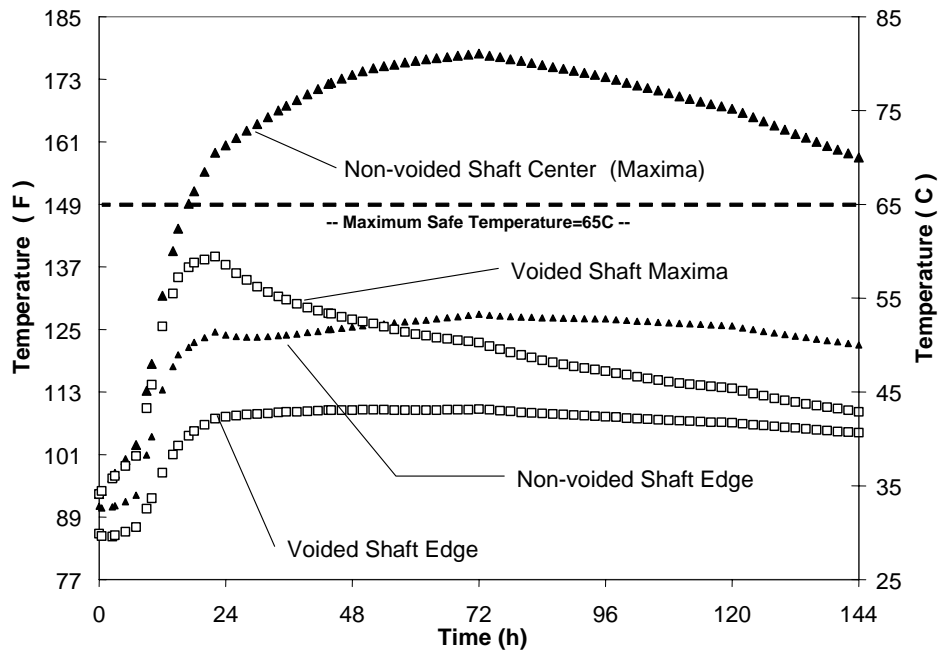


Figure 4-6. Calculated temperatures for 9ft (2.75m) voided and non-voided drilled shafts in saturated sands, with summer installation (“Clearwater” case).

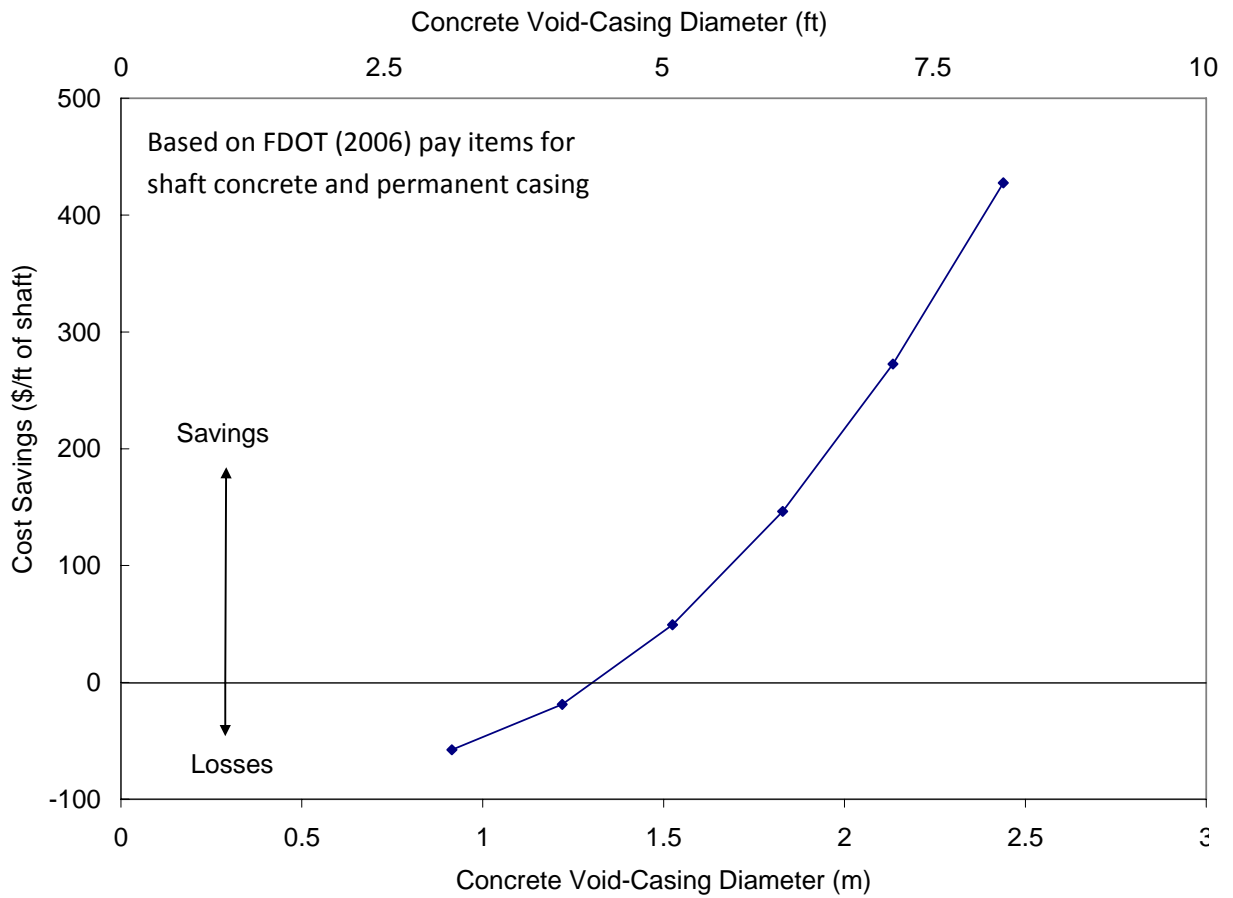


Figure 4-7. Cost savings per ft of void diameter from unused concrete, including the inner casing, which is assumed permanent.

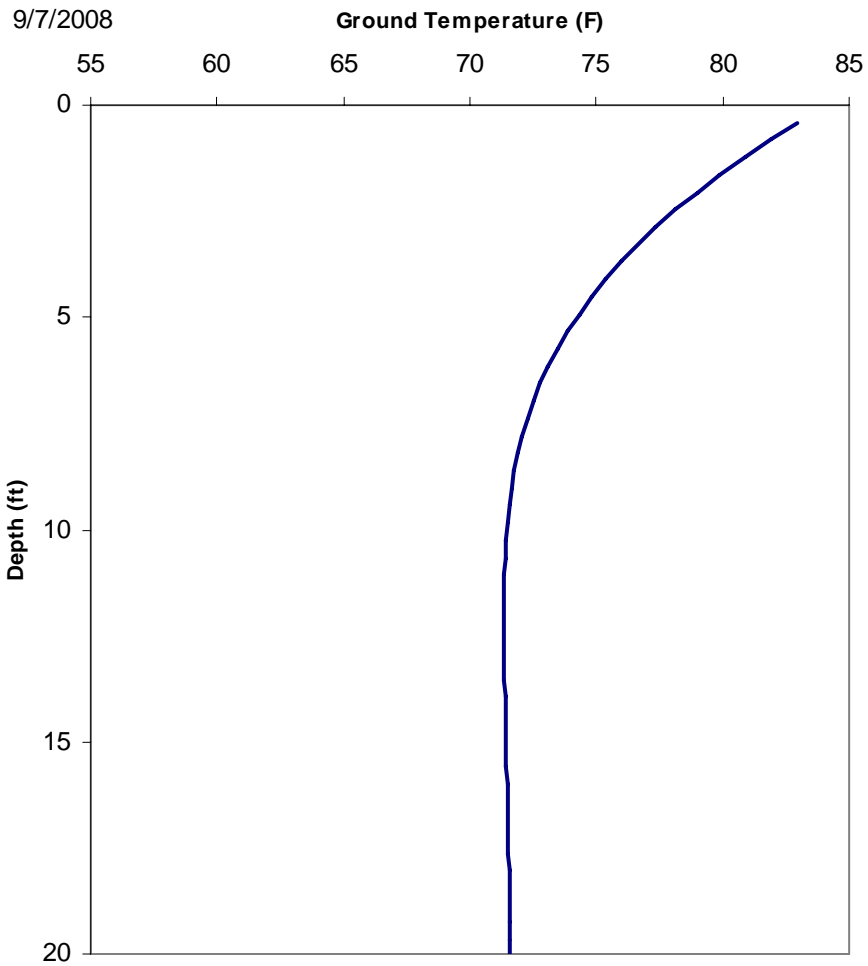
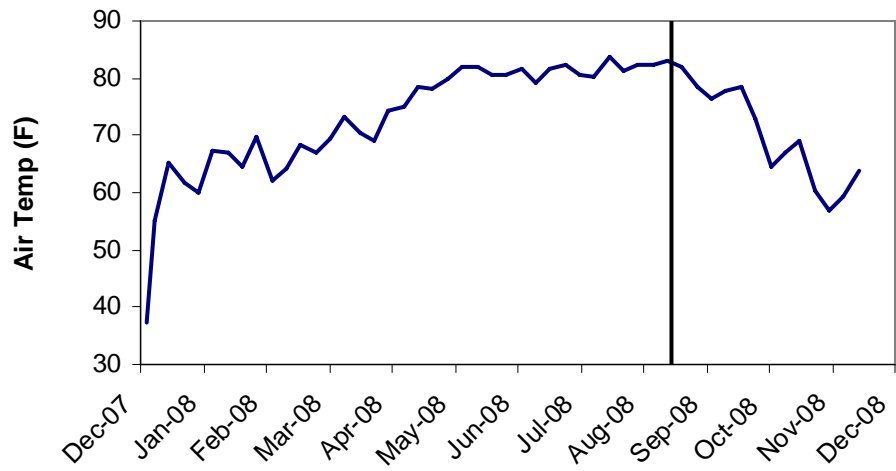


Figure 4-8. Hottest summer conditions occur in September for both the air and soil.

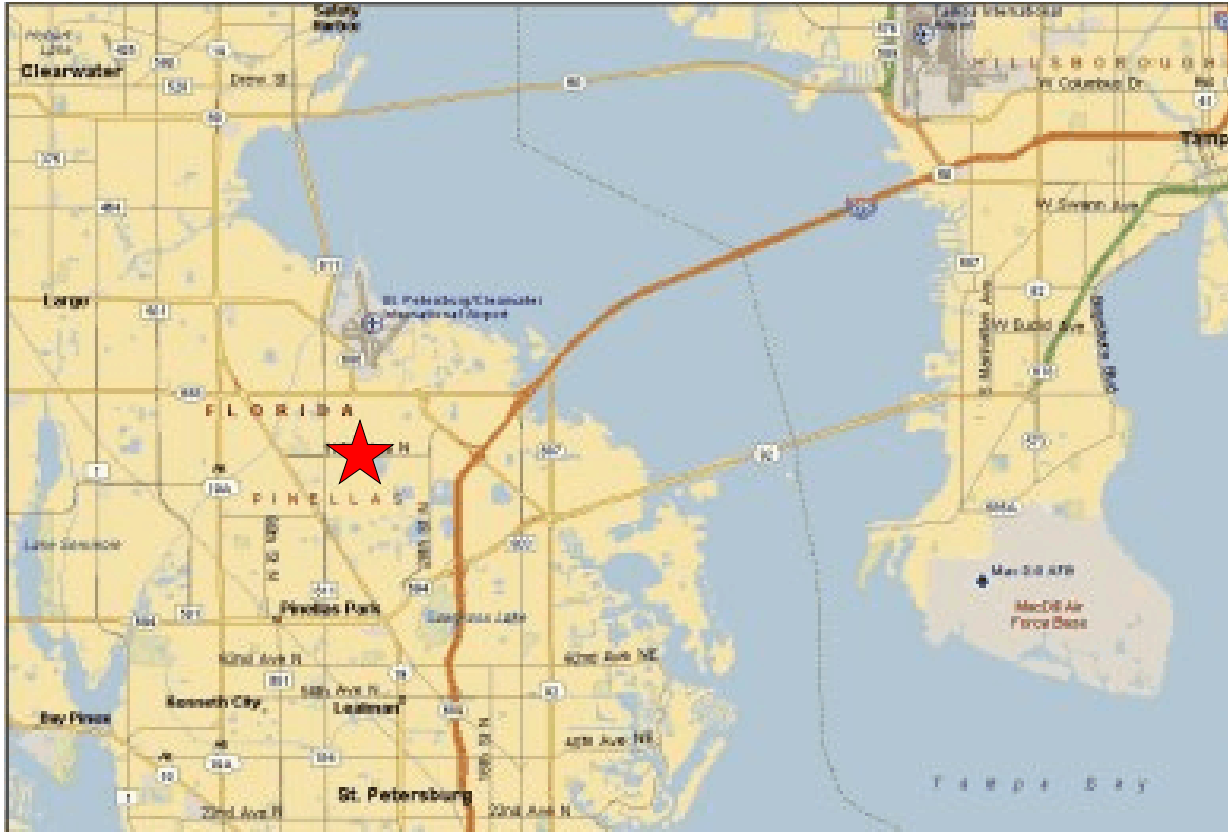


Figure 4-9. Site location in Clearwater, Florida.



Figure 4-10. Reinforcement cage with monitoring tubes and thermocouples.



Figure 4-11. Installation of ground monitoring tubes by FDOT District I personnel.



Figure 4-12. Voided shaft center casing access tube supports.



Figure 4-13. Thermocouples and access tube installed in center casing.



Figure 4-14. Drilling and clean-out of 9ft diameter excavation.



Figure 4-15. Lifting reinforcement cage.



Figure 4-16. Placement of reinforcement cage with 12in diameter wheel spacers.



Figure 4-17. Lifting central casing.



Figure 4-18. Placing and securing central casing.



Figure 4-19. Concrete placement using two tremies.



Figure 4-20. Temporary casing extraction.



Figure 4-21. Finish shaft with data acquisition system in place.

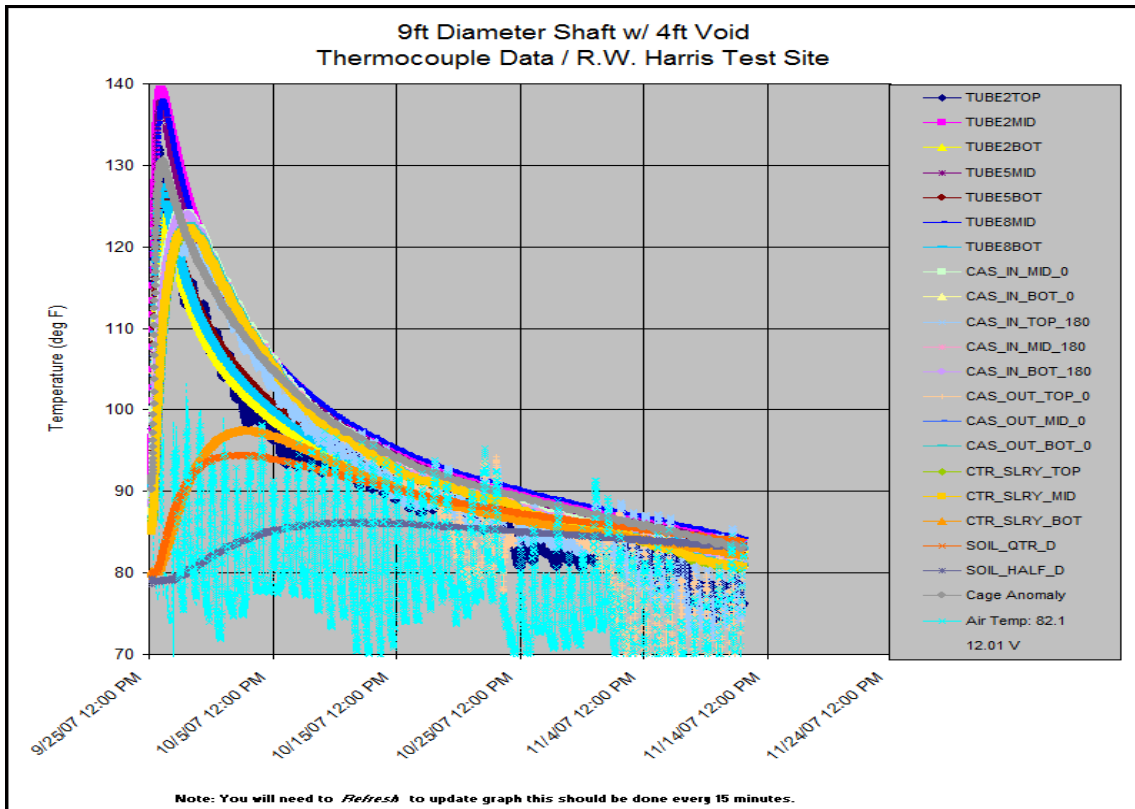


Figure 4-22. Realtime thermocouple data as posted on USF webpage.

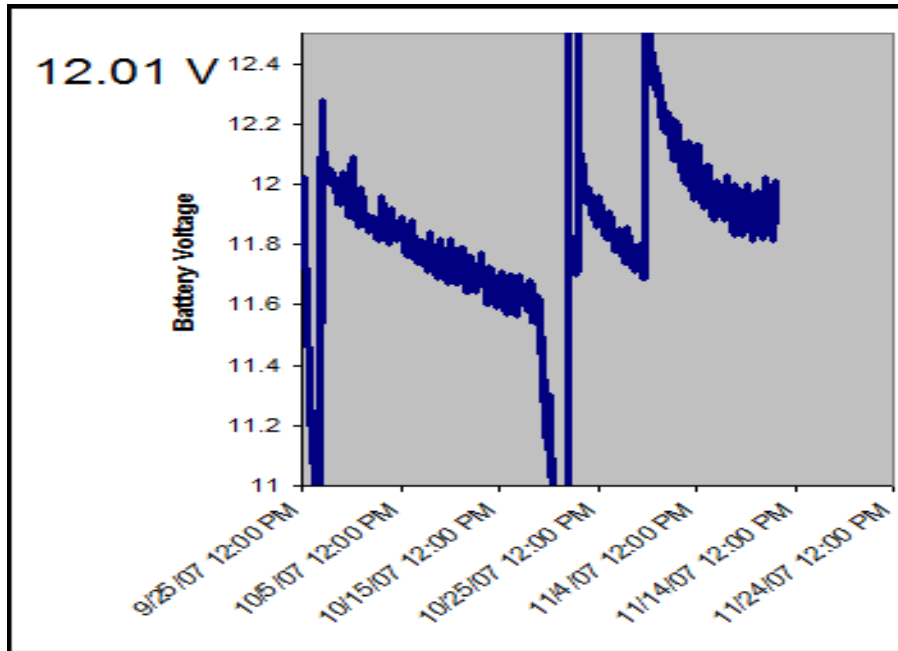


Figure 4-23 On-line battery voltage monitoring to assure remote system remained viable throughout the monitoring period.

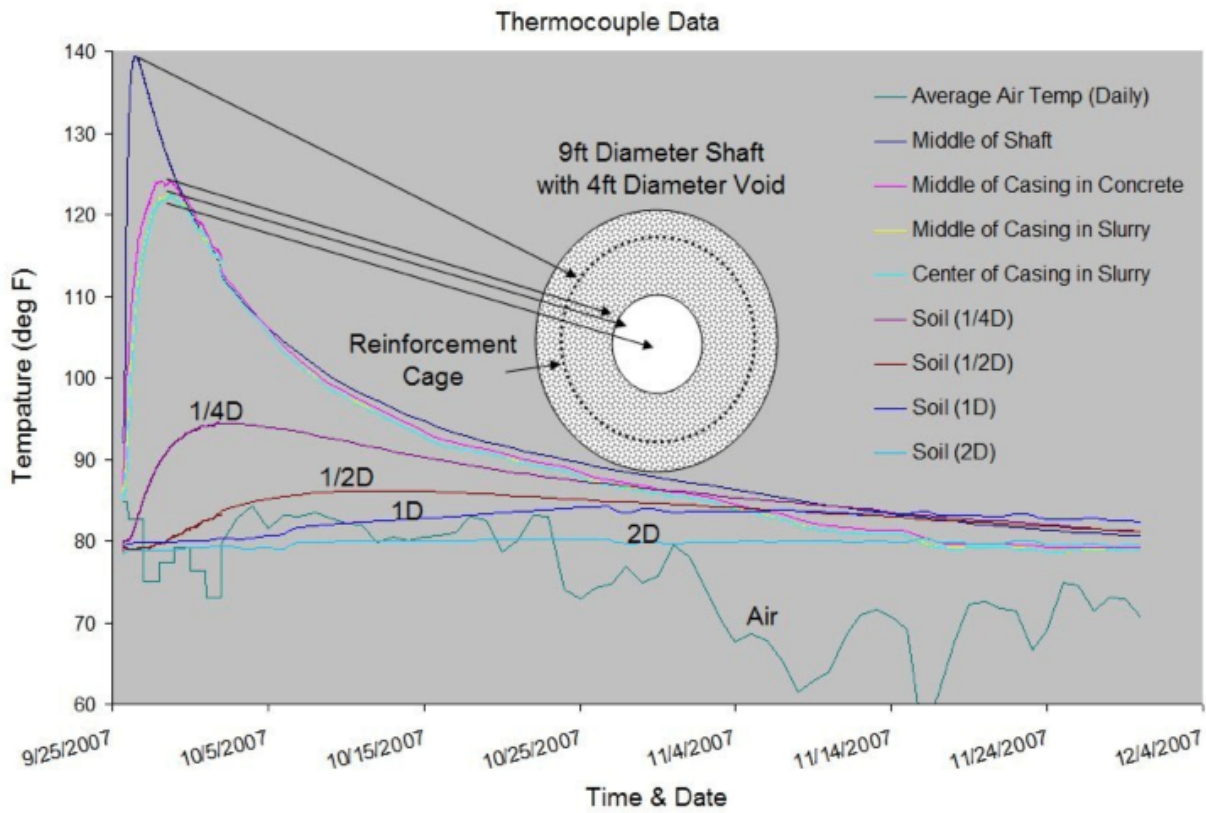


Figure 4-24 Annotated data from the voided shaft and surrounding vicinity.

Chapter Five: Numerical Modeling

Numerical modeling can be used to both assess mass concrete potential and evaluate thermal integrity results. Regardless of application the hydration energy production and rate of production are just as important as the surrounding environmental conditions and boundary conditions used to simulate those conditions. This chapter will begin by discussing ways of predicting the energy production and the soil temperature distribution that strongly affect the dissipation of the energy.

Finite difference algorithms chosen to produce internal temperature predictions were developed in a previous study (Mullins et al., 2007) packaged in a software named T3DModel. This software was specifically developed for analyzing drilled shaft integrity, but showed great application for all concrete hydration induced temperature distribution problems.

5.1 Energy Production

The primary algorithms used to predict the hydration energy production (rate and magnitude) stem from Schindler (2006) wherein a set of closed form solutions were presented defining three terms α , β , and τ . The terms define both the degree of hydration (Eq 5-1) as well as the rate of hydration (Eq 5-2); α , β , and τ are defined in terms of the fraction of cementing materials or cement constituents in equations 5-3, 5-4, and 5-5, respectively. The Blaine may be given in either mm^2/g or m^2/kg but must be converted to m^2/kg for introduction into the following equations where necessary.

$$\alpha(t_e) = \alpha_u \exp\left(-\left[\frac{\tau}{t_e}\right]^\beta\right) \quad (5-1)$$

$$Q_H(t) = H_u C_c \left(\frac{\tau}{t_e}\right)^\beta \left(\frac{\beta}{t_e}\right) \alpha(t_e) \frac{E}{R} \left(\frac{1}{273+T_r} - \frac{1}{273+T_c}\right) \quad (5-2)$$

$$\alpha_u = \frac{1.031w/c}{0.194 + w/c} + 0.5p_{FA} + 0.3p_{SLAG} \leq 1.0 \quad (5-3)$$

$$\beta = 181.4p_{C_3A}^{0.146} p_{C_3S}^{0.227} Blaine^{-0.535} p_{SO}^{0.558} \exp(-0.647p_{SLAG}) \quad (5-4)$$

$$\tau = 66.78 p_{C_3A}^{-0.154} p_{C_3S}^{-0.401} Blaine^{-0.804} p_{SO}^{-0.758} \exp(2.187 p_{SLAG} + 9.5 p_{FA} p_{FA-C_3O}) \quad (5-5)$$

The total amount of energy produced by the mix is proportional to the total cement content and/or supplementing cementing materials, SCM (e.g. flyash, slag, etc.). This is quantified by the total heat of hydration from cement or SCMs using the following two equations where p represents the fraction of the various cement constituents or the fraction of cementing materials as appropriate.

$$H_{cem} = 500 p_{C_3S} + 260 p_{C_2S} + 866 p_{C_3A} + 420 p_{C_4AF} + 624 p_{SO_3} + 1186 p_{freeCAO} + 850 p_{MgO} \quad (5-6)$$

$$H_u = H_{cem} p_{cem} + 461 p_{SLAG} + h_{FA} p_{FA} \quad (5-7)$$

Sample mix designs are included in the appendix wherein the constituents are itemized along with the various percentages used in the above equations.

The T3DModel houses a library of concrete mix designs and can be updated as needed by inputting the percentages of the various cement and flyash constituents. Figure 5-1 shows the new input screen to input user defined mix designs.

Input parameters include percentages of the following constituents for cement: MgO, C2S, C3A, C3S, SO3, C4AF, and CaO as well as Blaine (m^2/kg). For flyash the SO3 and CaO percentages are needed.

Using the inputted heat source parameters and by running various models for a given site, baseline norms can be established for quick assessment of a thermal integrity test. Figure 5-2 shows a family of curves for a given site and mix design for various shaft diameters and testing times. For instance, if thermal integrity testing is performed on a 42 inch diameter shaft, 30 hours after casting, the expected tube temperature would be approximately 122 deg F.

5.2 Ground Temperature Distribution (Boundary Conditions)

Boundary conditions for any thermal model are essentially based on the temperature variations that exist either in the soil as a function of depth or the air temperature as a function of time. The first is important for defining the gradient the soil will experience during hydration which was previously unknown. The second can easily be obtained based on archival temperature data from a number of government or private internet sources. As a result, a unique temperature profile (with depth) is present at the time of concreting for shafts or footings that can be approximated either by modeling or insitu measurements. Previous case studies performed during this and previous studies have been evaluated to identify reliable methods of predicting these temperature profiles that in turn make the predictive models more robust. This is especially helpful with regard to thermal integrity testing of drilled shafts and comparing field results to predicted norms.

By running long-term T3DModel runs that incorporate heat transfer in and out of the soil for periods of months prior to concreting, the exact soil profile at the time of concreting can be directly inputted into the model to more accurately predict a perfect shaft temperature profile. The amount of time is somewhat insensitive provided that a long-enough timeframe has been selected prior to the time of concreting. For example, annual temperature fluctuations (as shown by monthly or weekly averages) do not vary significantly from year to year. But, in the event they do, the exact recent air temperature history of a specific site can be used as an input time series (air boundary condition) to precondition the soil profile. Given that the deep soil temperature in Florida stay reasonably constant at 72 deg F, an extended air temperature above 80 deg F in summer climates will produce a vertical temperature profile like that shown in Figure 5-3.

This data was collected two years apart and several feet apart (one in the soil and the other in a “cold” shaft). The significance is that soil temperature is not constant throughout the year but tends to return to the same profile at the same time of the year.

As air temperature data is available for most major cities on a daily and/or hourly basis, ground temperature profiles can be established for almost any location with reasonable accuracy. This affords modeling the ability to provide refined boundary conditions. Figure 5-4 shows archival data for the Tampa / St. Petersburg area showing essentially reproducible temperature variations throughout a 14 year period.

For a given concreting date of a drilled shaft or mass concrete element, the exact conditions can be obtained that led up to the surrounding environment’s temperature distribution. Figure 5-5 shows data over a one year period from which model predictions of the soil temperature profile were produced.

Figures 5-6a through 5-6c show the ground temperature profiles determined from long-term model runs incorporating the air temperature time series from Figure 5-5. Notice there are two times a year when the ground temperature is relatively uniform, April and October / November. At all other times the ground is either heating or cooling and lagging behind the air temperature trends.

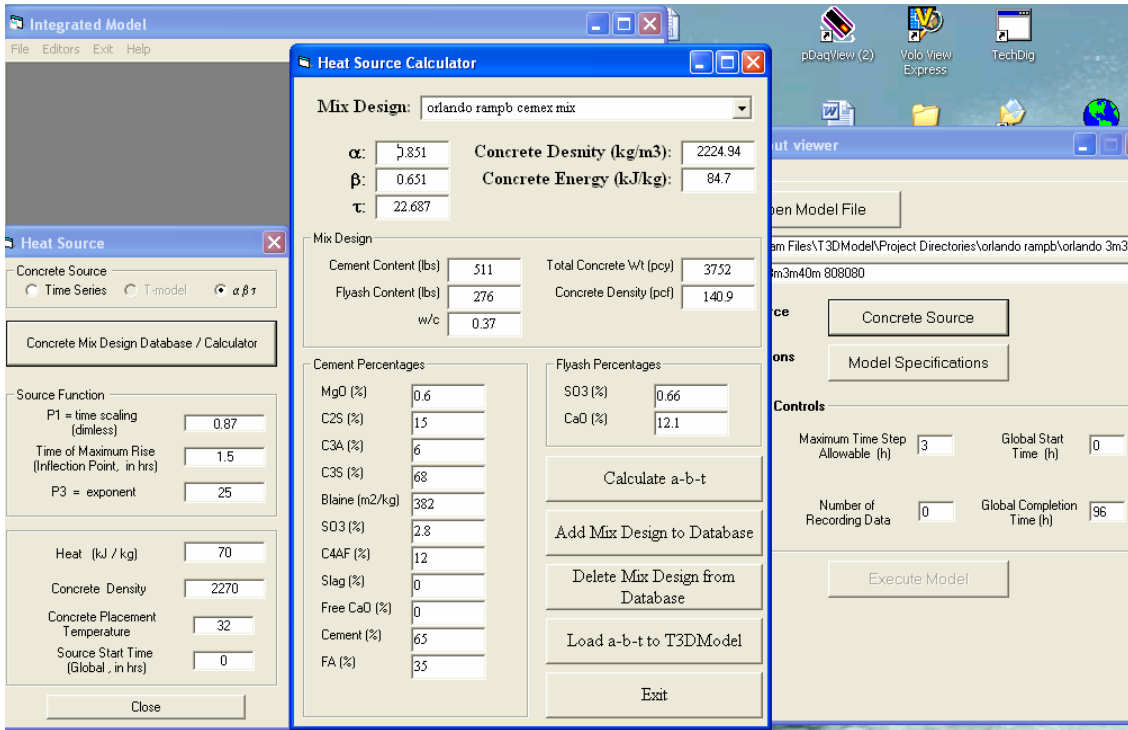


Figure 5-1. Heat source calculator used to input new mix design parameters.

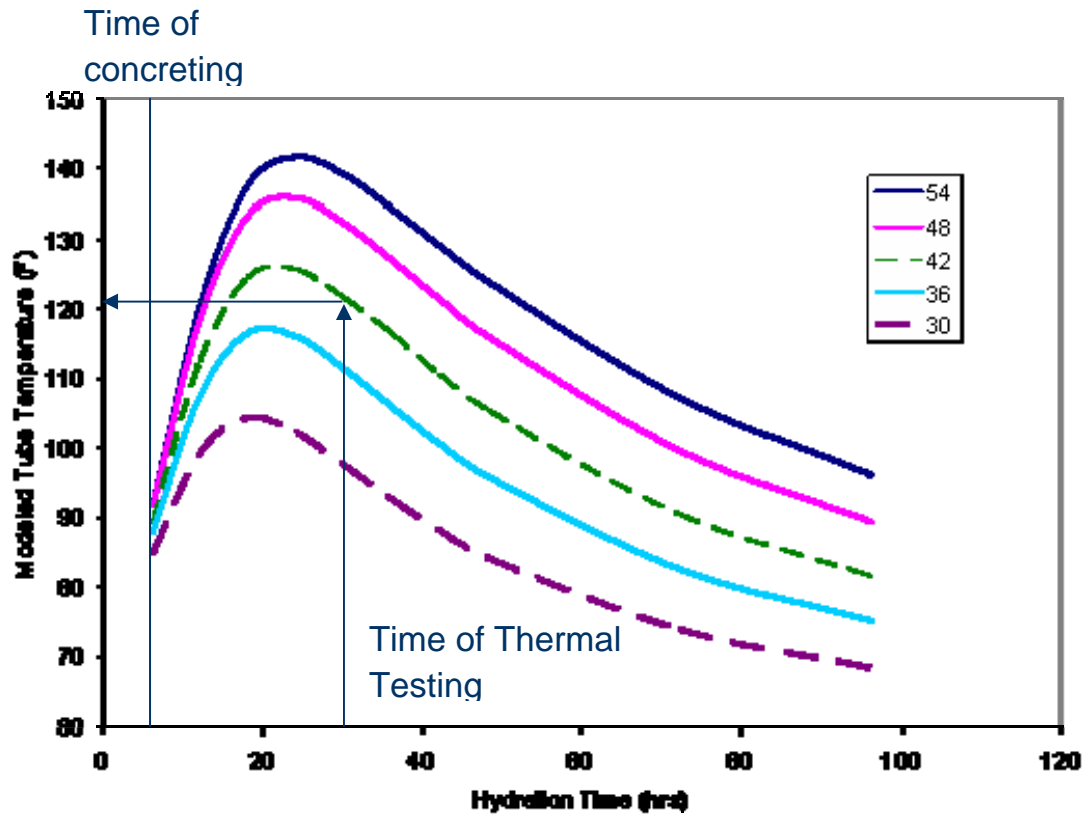


Figure 5-2. Expected access tube temperatures for various sizes of shaft and testing times.

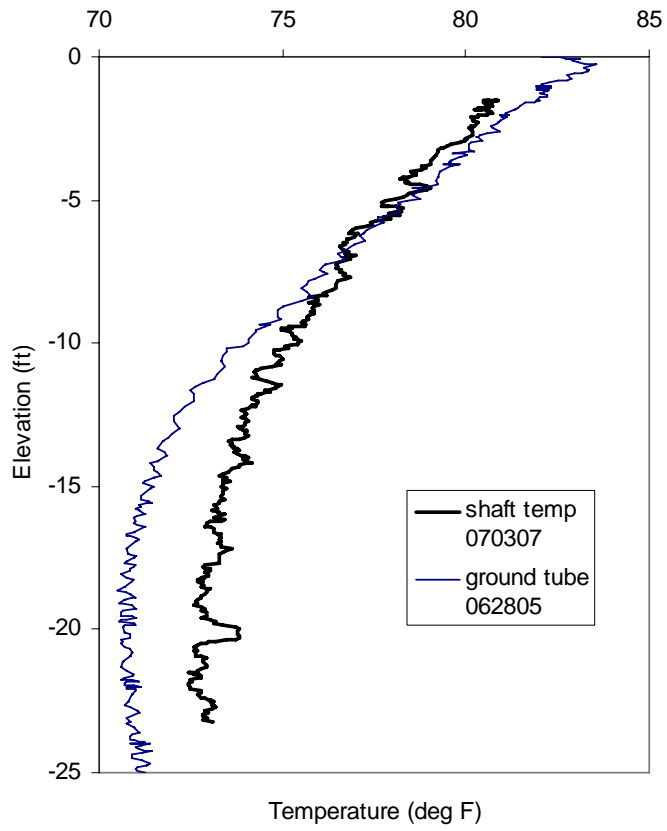


Figure 5-3. Soil temperature profile at Clearwater test site.

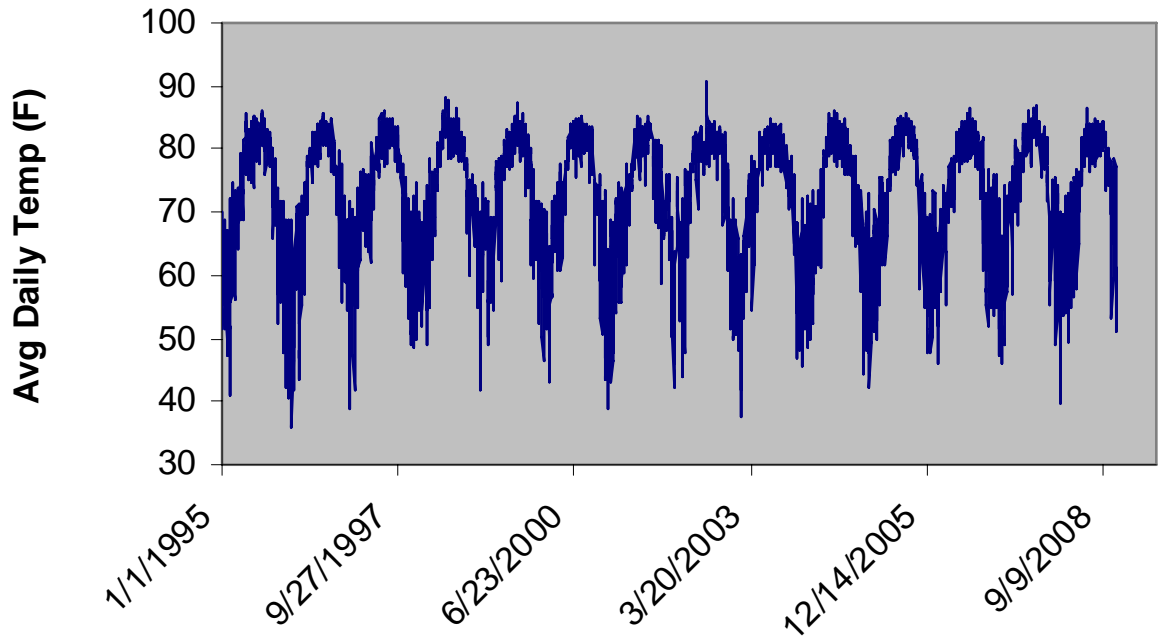


Figure 5-4. Average daily temperature for the Tampa / St. Petersburg, FL area.

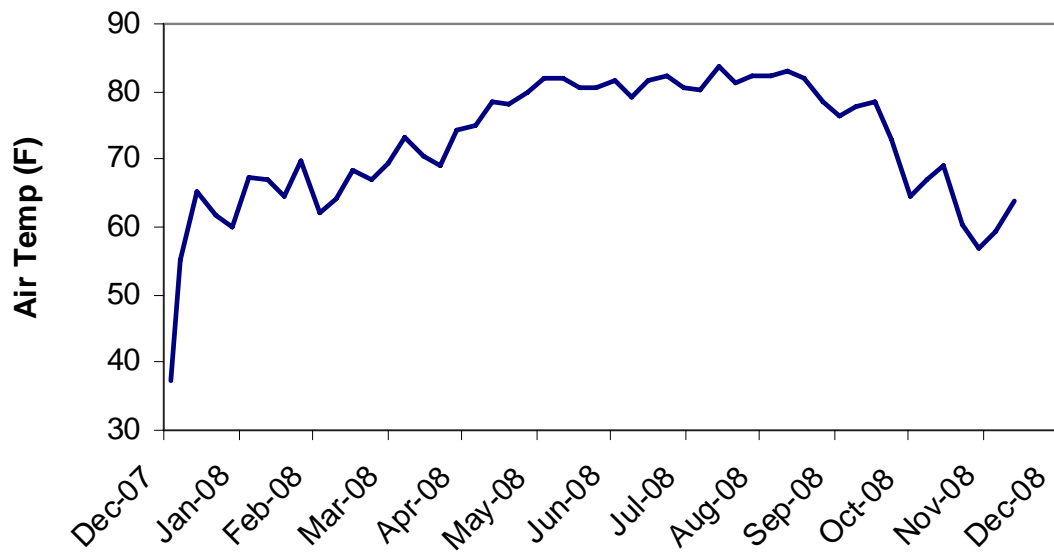


Figure 5-5. Average weekly temperature variations over one year period (Clearwater, FL).

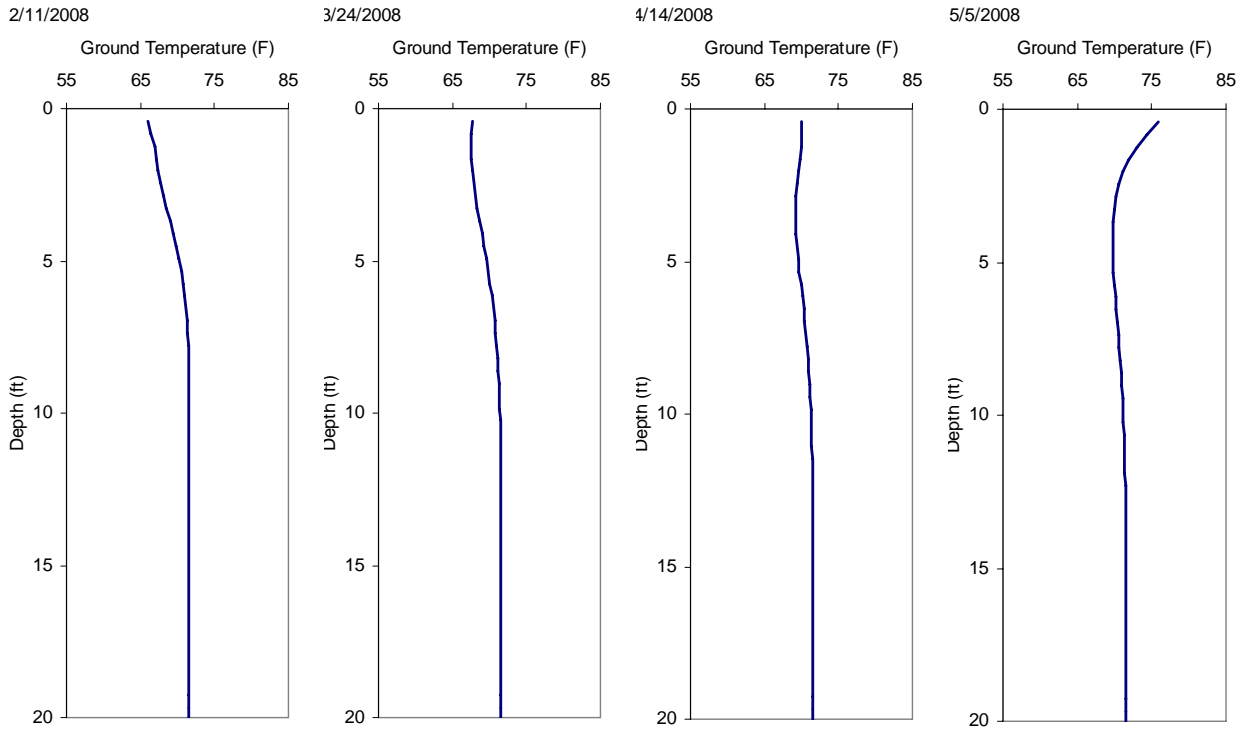


Figure 5-6a. Modeled ground temperature profile (Feb to May).

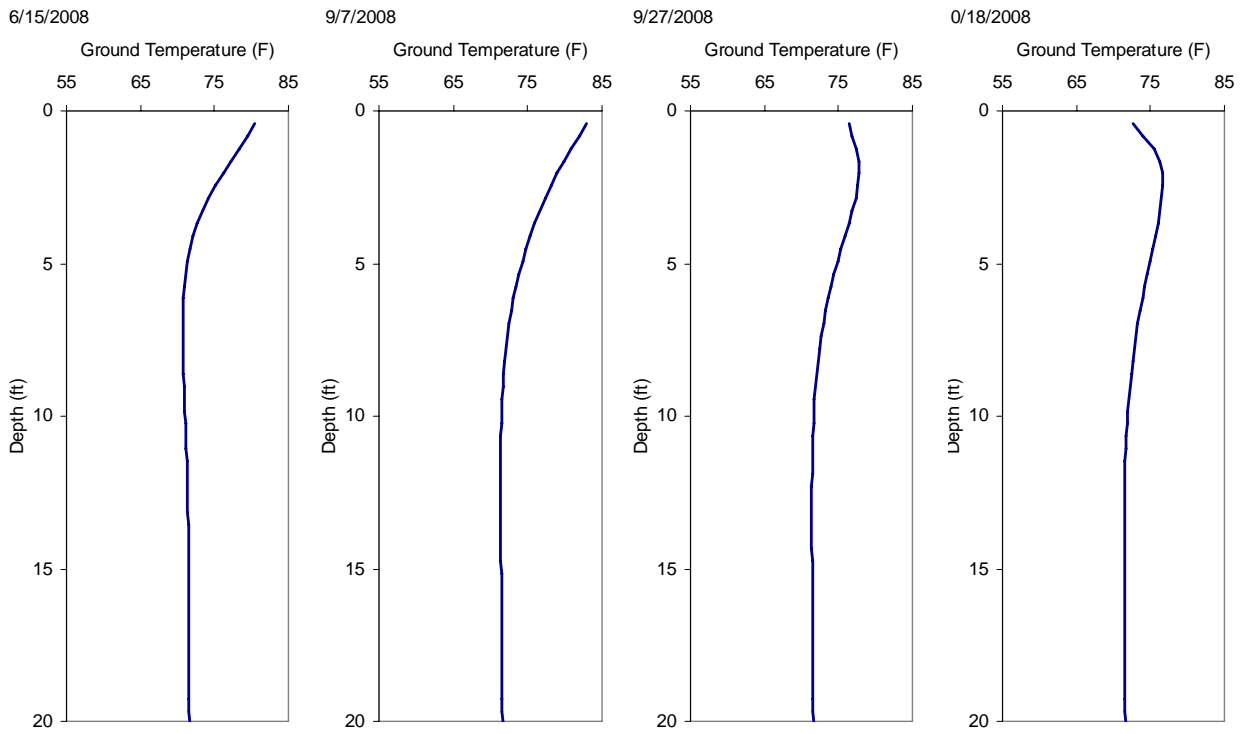


Figure 5-6b. Modeled ground temperature profile (June to October).

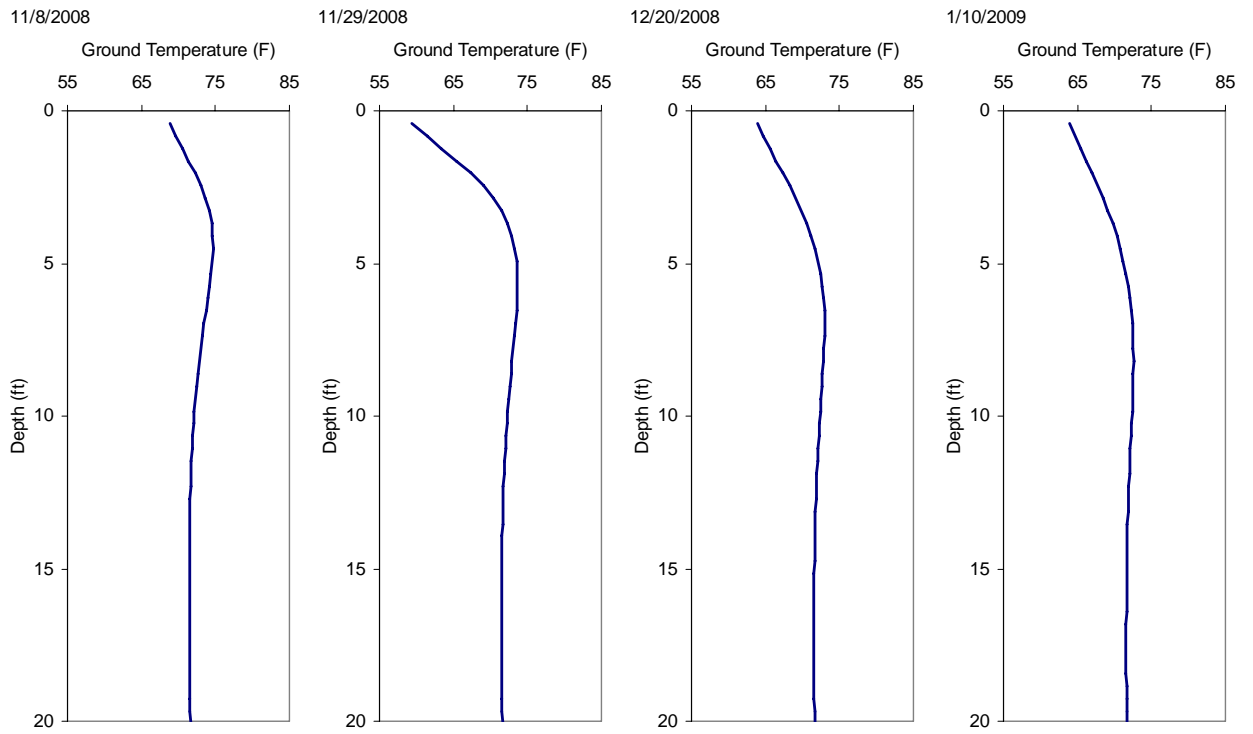


Figure 5-6c. Modeled ground temperature profile (November to January).

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Chapter Six: Summary and Conclusions

This study addressed factors affecting mass concrete conditions for all large-sized concrete elements with specific focus on drilled shafts. The heat energy produced by curing concrete caused by hydration has both advantageous and detrimental effects. The advantages arise from the reproducible and predictable temperature signature from curing concrete that can be compared to field measurement via infrared measurements (thermal integrity testing of drilled shafts). The detrimental aspects come from either immediate differential temperature-induced cracking or long-term durability reduction from delayed ettringite formation (DEF).

The construction of a voided drilled shaft (essentially a cast-in-place pipe pile) was demonstrated to verify that not only the internal temperature of large diameter shafts could be controlled by geometry changes but also that the practical aspects of constructing such as shaft were not overlooked. As a result, the logistics of the voided shaft construction were conducive to standard shaft construction, did not add needless complication, and in fact reduced concreting time and concrete quantity. The removal of the central core of concrete from the 9ft diameter demonstration shaft drastically reduced internal temperatures (both peak and differential) such that it did not exhibit mass concrete conditions. This was in stark contrast with the 4ft diameter shaft constructed under identical conditions in an earlier study that exceeded both the peak and differential temperature limits.

Extensive thermal integrity testing was performed to increase confidence in the suitability of the test method as well as the field testing protocols. In each case, field testing was compared to modeled results to verify numerical modeling refinements that were undertaken throughout the study.

Outcomes of the study can be summarized in recommendations for guidelines or specifications for the use of *Thermal Integrity Evaluation* and *Mass Concrete Identification* (included in the Appendix).

6.1 Recommendations for Thermal Integrity Evaluation

Guidelines for the use of *Thermal Integrity Evaluation* of drilled shafts must incorporate the overall concept as well as definitive pieces of information that must be obtained during concrete placement, shaft excavation, or general construction processes. As cited in results from Chapter 3, the thermal integrity technician should note or obtain from the shaft inspector the following items: time of testing, time of concreting, concrete mix design, concrete casting temperature, length and diameter of the shafts, construction logs detailing the method of construction (casing length, casing diameter, etc), concreting logs, and a boring log. Table 6-1 shows an example shaft detail listing.

Additional information which is helpful for accurate thermal modeling of the site includes the excavation/spoil temperature as the material is spun from the tool at the surface. This is not required of all shafts but rather a representative number to map the vertical soil temperature distribution. This information is best obtained by the shaft inspector and can be obtained simply using a hand-held infrared temperature gun (Figure 6-1).

Criteria for acceptance of a shaft on the basis of the temperature profile are dependent on the needs of the shaft with regards to either structural capacity or durability. Therein, a simple reduction in wave speed or increased arrival times typical of Cross Hole Sonic Logging is not sufficient. Rather, a definite criterion based on either loss of structural capacity or cover must be adopted. Since a means of field testing is available to ascertain cover around drilled shaft reinforcing steel, the engineer can design for a specific cover based on project and site specific applications.

Likewise, the structural (axial and bending) capacity of most shafts exceeds that required for most regions of the shaft (perhaps excluding the upper most portions near the cap). If a cross-section shows a reduction that does not impinge on the cover criterion (discussed above) then it is likely to be acceptable. To this extent, the actual needs of the structure must be known to best determine the acceptance of an imperfect shaft. Figure 6-2 shows an example shaft usage from top to bottom relative to the 0.75% steel provided broken into 15 equal lengths (plotted points). The full shaft section should not be compromised in the upper portions (fifth from the top being the critical section). However, lower portions of the shaft need not develop full capacity and could likely exhibit significant cross sectional reductions without concern (again provided that cover was sufficient). This does not mean that substandard shafts should be accepted because anomalies detected are in a non-critical region. However, full knowledge of the as-built shaft dimensions may provide a more precise estimate of a malformed shaft capacity. This study makes no recommendations with regard to a course of action to be taken by the state if a shaft demonstrates an anomaly that has no adverse affects on the serviceability of the shaft.

Thermal integrity evaluation requires knowledge of the normally anticipated temperature of the shaft prior to deciding on the actual as-built shape. This relates to the normal heat signature of a curing shaft based on shaft diameter, the surrounding environment (e.g. casing, water, soil, rock or combination thereof), the concrete mix design, and the time at which the thermal scans are conducted. Series of normal temperatures for various shaft sizes can be prepared in advance of the testing and used to quickly ascertain the shaft conditions at the time of testing. Figure 6-3 shows an example of the anticipated temperatures of unaltered shafts of various diameters for a given set of site conditions. All temperatures correspond to the normal radial location of the access tubes with proper cover.

A recommended time frame for thermal testing (Figure 6-4) can be established on the basis of shaft diameter, the time to peak temperature, and the drop in temperature related to the greatest achieved change in temperature during the curing (peak minus initial concrete temperature). The predicted response is dependent on mix design, soil conditions, ambient air temperature, and concrete placement temperature. In general, the

optimal time to test is between the peak temperature and that time where the drop in temperature is equivalent to 25% of the greatest temperature change. Testing times later than optimal will still provide reasonable results provided the concrete (at the location of the tube) to soil temperature change is no less than 60 deg F (or 120 deg F for a 60 deg F soil temperature).

Due to the amount of information derived from *Thermal Integrity Evaluation* a more precise rationale for accepting or rejecting shafts can be adopted that incorporates the actual shaft cross section and capacity (as determined from test results). This requires that the moment and axial load distribution down the length of the shaft must be available. This information should have already been prepared when in design.

A sample technical specification has been prepared and included in the Appendix.

6.2 Sensitivity of Thermal Modeling for Thermal Integrity Evaluation

The ability to detect and evaluate anomalies is dependent on both the field temperature measurement resolution and the model sensitivity. Field measurements are equipment dependent but at present the infrared sensors can detect temperature variations down to the nearest 0.2 deg F.

Thermal modeling shows the sensitivity to have sub-inch resolution as shown in Figures 6-5 and 6-6. Figure 6-5 shows the effect of necking on 4, 6, 8, and 10ft diameter shafts with sectional reductions ranging from 1 to 6 inches. This translates into temperature reductions ranging from 4 to 25 deg F for radial reductions of 1 to 6 inches, respectively (Figure 6-6). Interestingly, the temperature reduction noted is relatively independent of shaft diameter. From a field interpretation standpoint a 1.0 deg F temperature reduction can be interpreted to have a ¼” radial reduction (for the Ocala model parameters). Given the device resolution (0.2 deg F), field measurements have a theoretical anomaly detection of 0.05 inch radially (bulge or neck).

Table 6-1 Shaft Testing Details

Shaft I.D.	Date/Time Test Performed	Date/Time Cast	Casting Air Temp		Casting Conc. Temp		Hydration Time (hrs)	Diam. (in)	GWT (ft)	Length (ft)	Rock Socket Length (ft)	Vol. Theory (cy)	Vol. Field (cy)	Vol. (%)	General Comments
			C	F	C	F									
DS-26	4/3/2008 13:09	4/2/2008 11:02	27	81	27	80	26.1	42	N/A	80	32	28.507	76	267	No concerns Casing set to 78' concrete fell 50' and 42' in two pours
DS-27	4/3/2008 12:00	4/1/2008 10:52	24	75	26	79	49.1	54	N/A	85	41	50.069	72	144	No concerns drilled 33' w/54 set casing to 76ft 48" to bottom



Figure 6-1. Economical hand-held infrared thermometer.

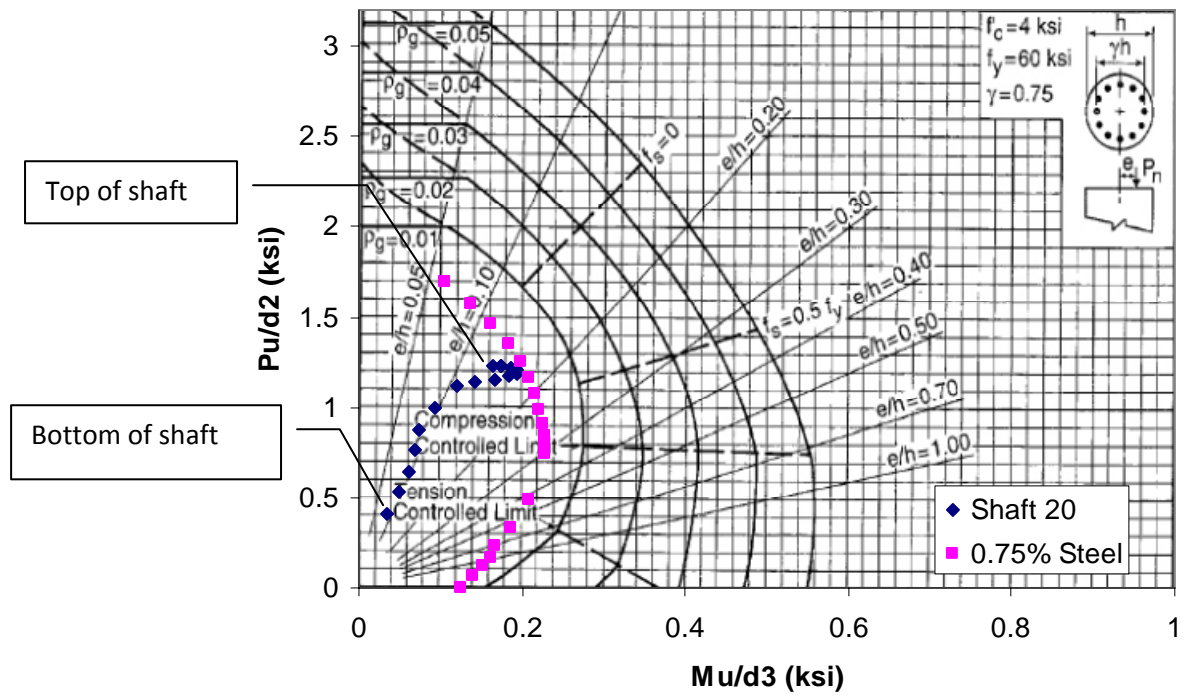


Figure 6-2. Example shaft capacity usage as a function of position in shaft.

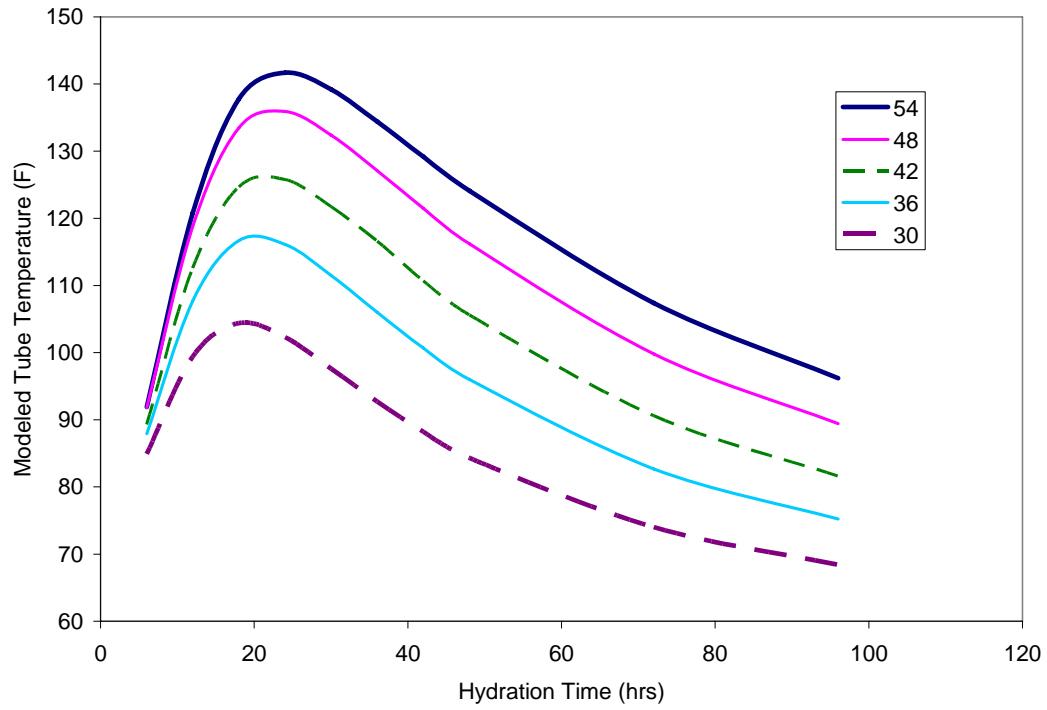


Figure 6-3. Normal integrity tube temperatures as a function of shaft diameter and hydration time.

Thermal Testing Timeframe

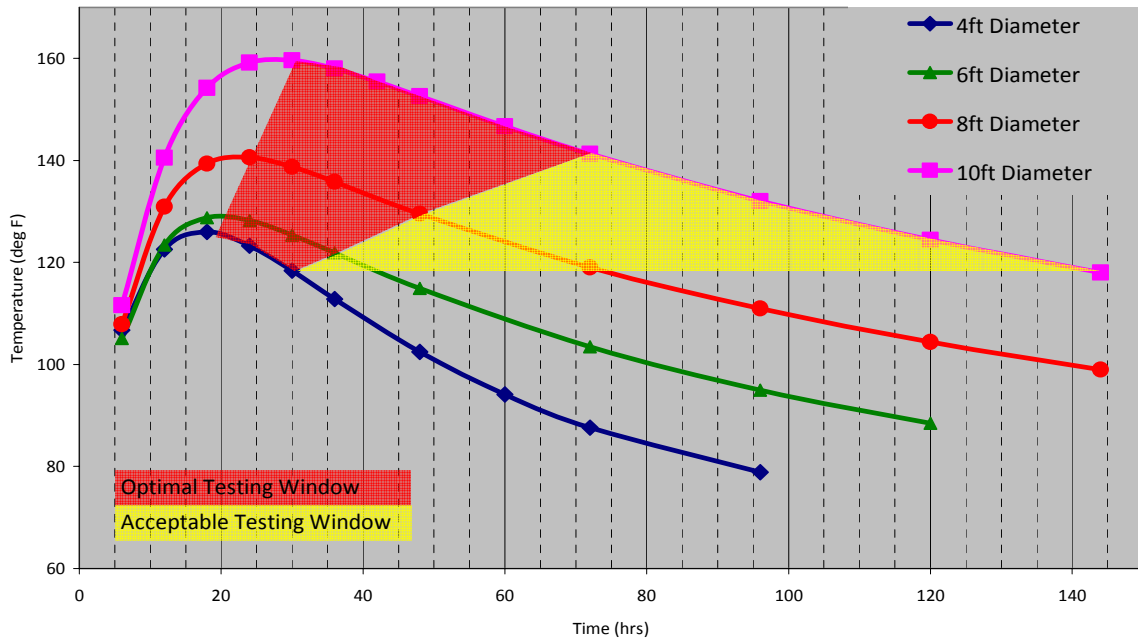


Figure 6-4. Recommended testing timeframes for various shaft sizes.

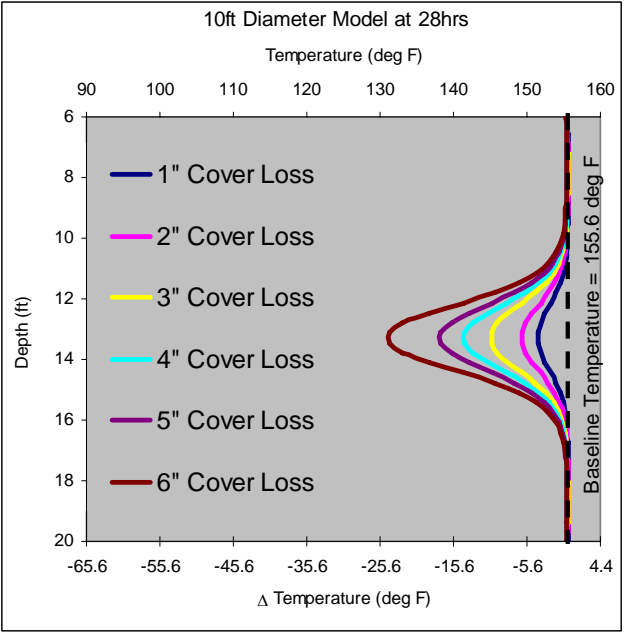
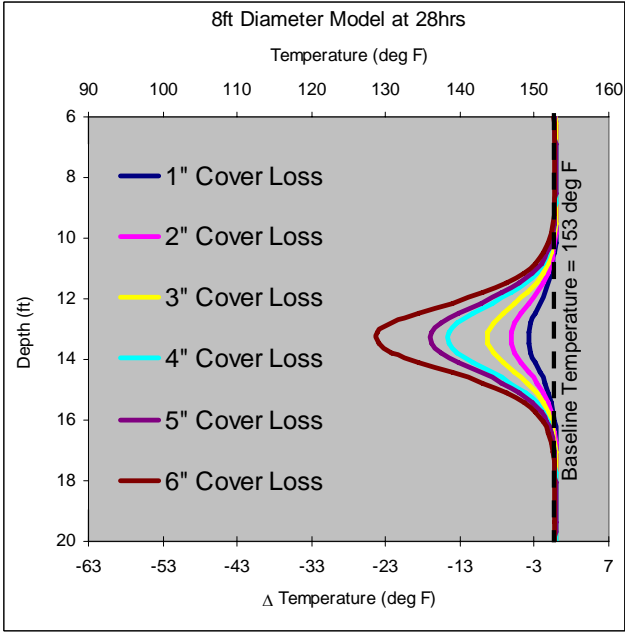
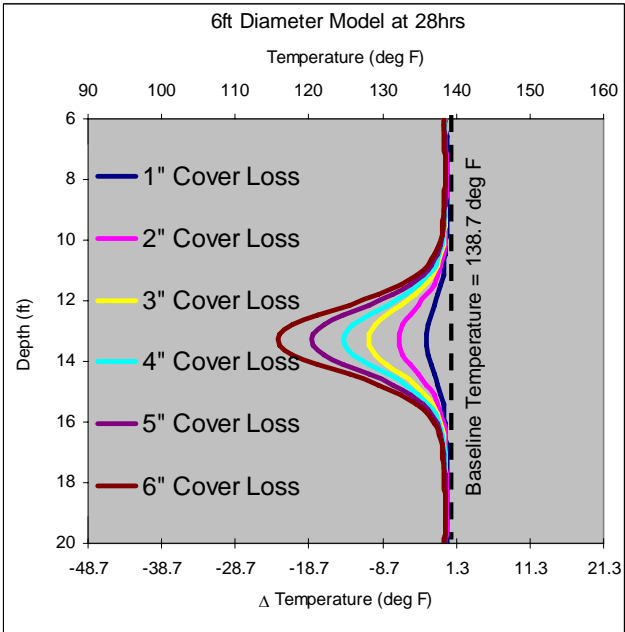
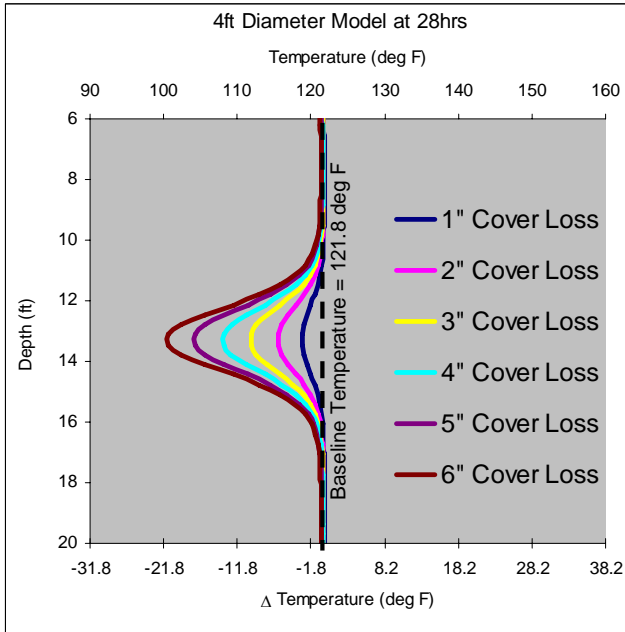


Figure 6-5. Model response for cover loss for various size shafts.

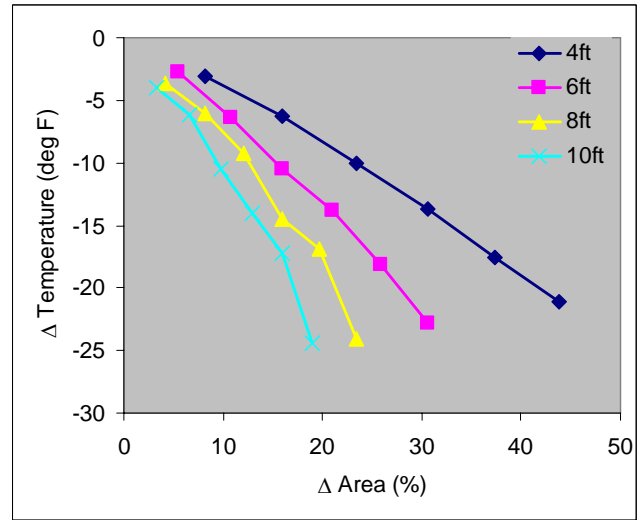
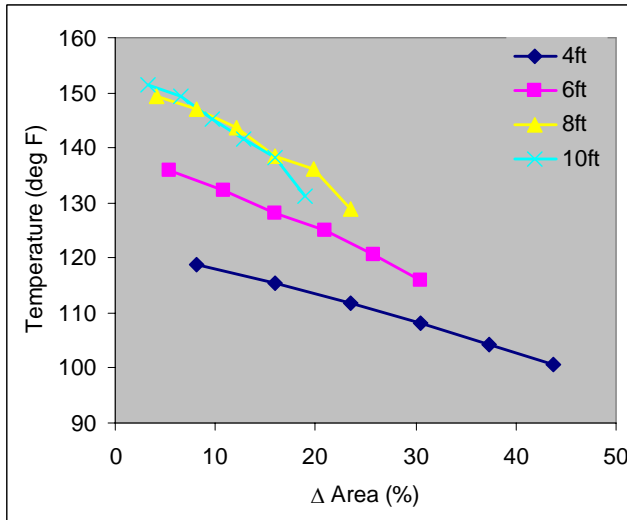
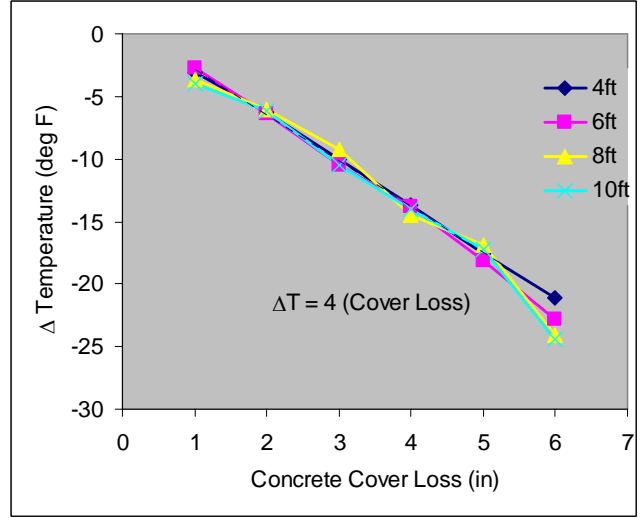
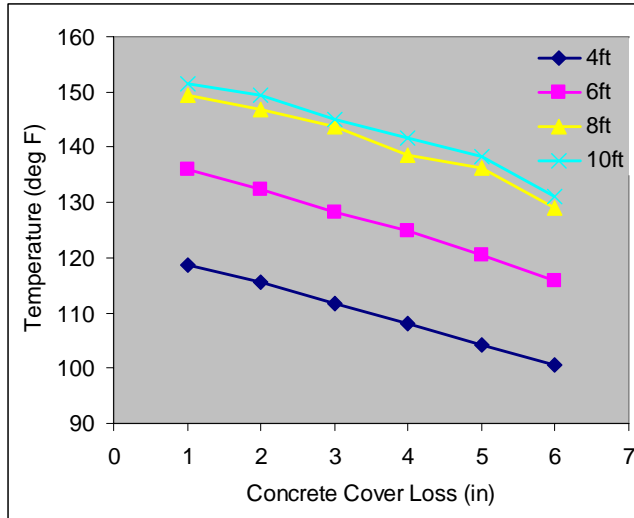


Figure 6-6. Effects of dimensions on temperature for various shaft sizes.

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Appendix A: Concrete Mix Designs

*Ringling Replacement
Drill Shaft*

Tom J. Jiles
ISSUED : T BILLOW
REVIEWED : J R WARD
DATE : 08/07/00
MIX NO. : 01-M2024

CONCRETE MIX DESIGN

CONCRETE SUPPLIER: SOUTHDOWN, INC
ADDRESS: 9117 FLORIDA MINING BLVD TAMPA FL
PLANT LOCATION: BRADENTON
FOOT ASSIGNED PLANT NO.: 113-106
DATE: 08/01/00

TELEPHONE NO: 813/384-3447
PROJECT NO: 196042-15201

CLASS CONCRETE: IV DRILL SHAFT (28) 4000 PSI

SOURCE OF MATERIALS

COARSE AGGREGATE :	FLORIDA ROCK IND	GRADE :	37 S.G. (SSD): 2.450
FINE AGGREGATE :	VULCANICA	F.M. :	3.10 S.G. (SSD): 2.630
PIT NO. (COARSE) :	12-250	TYPE :	CRUSHED LIMESTONE
PIT NO. (FINE) :	16-081	TYPE :	SILICA SAND
CEMENT :	SOUTHDOWN INC	SPEC :	AASHTO M-85 TYPE II
AIR ENR. ADMIX :	DAREX	W R GRACE	SPEC: AASHTO M-154
1ST ADMIX :	WRDA-64	W R GRACE	SPEC: AASHTO M-194 TYPE D
2ND ADMIX :	----		SPEC: ----
3RD ADMIX :	----		SPEC: ----
FLY ASH :	FLORIDA FLYASH	SPEC :	ASTM C-618 CLASS F

HOT WEATHER MIX
AGGREGATE CORR: 0.6

CEMENT	kg:	291.00	SLUMP RANGE	:	175.00 TO 225.00	mm
COARSE AGG	kg:	955.00	AIR CONTENT	:	0.00 TO 6.00	%
FINE AGG	kg:	680.00	UNIT WEIGHT (WET)	:	2264.00	kg/m3
AIR ENR ADMX	ml:	155.00	W/C RATIO (PLANT)	:	0.41	kg/kg
1ST ADMIXTURE	ml:	1137.00	W/C RATIO (FIELD)	:	0.41	kg/kg
2ND ADMIX	ml:	0.00	THEO YIELD	:	1.00	m3
3RD ADMIX	ml:	0.00				
WATER	L:	183.00				
WATER	kg:	183.00				
FLY ASH	kg:	155.00				

- 490.477 16/43

- 261.2506 16/43

CC:
DIST 1 & 7
FILE

PRODUCER TEST DATA

CHLORIDE CONT :	0.08	kg/m3	
SLUMP :	215.00	mm	
AIR CONTENT :	2.00	%	
TEMPERATURE :	37.00	DEG C	
COMPRESSIVE STRENGTH MPA			
7 -DAY-	31.70	28 -DAY-	38.60
-DAY-		-DAY-	

Figure A-1. Ringling Causeway Mix Design.



17-August-2011

Mix ID Number: 150929

Strength: 4,000 PSI

Alternate ID: 10929

Ginco Code: 7-400-8-C-C8-79-3-2-2EA

Description: 4000 #57 Drill Shaft

Use:

Material	Description	ASTM	Weight (lb)	oz/yd	Specific Gravity	Absolute Volume
Type I-II	Cement	C-150	564		3.15	2.87
Flyash F	Flyash Class F	C-618	142		2.17	1.05
Fine Aggregate	Sand	C-33	1319		2.63	8.03
Coarse Aggregate	#57 Stone	C-33	1448		2.45	9.47
Water Reducer	Water Reducer	C-494		37.60	1.00	0.00
Potable	Water		317		1.00	5.08
TOTAL CEMENTITIOUS MATERIAL PER ASTM C 595			706 lb			
Design Percent Air "Entrapped/Entrained"			2.0 %			0.54
Slump Range (From Mixer Discharge):			7.00 To 9.00 "		Absolute Volume 27.04 cu.ft.	
Air Content (From Mixer Discharge):			0.50 To 3.50 %			
Plastic Unit Weight:			140.2 lbs./cu.ft.			
W/C + P Ratio: Lbs/Lbs			0.45			
Total Weight per Cubic Yard:			3790 lb			

NOTES:

Cemex has no knowledge or authority regarding mix placement. It is the responsibility of the project architect/engineer, and/or contractor to ensure that the proper mix is ordered for the application. Cementitious content is stated as a minimum and Cemex reserves the right to increase it. Chemical admixtures are added in accordance with the manufacturer's recommendations, and Cemex reserves the right to adjust admixture dosages as needed, depending upon ambient conditions and jobsite demands. If aggregate specific gravity varies from original design, Cemex reserves the right to make adjustments as needed to ensure proper yield. Mix is designed to achieve design strength after 28 days curing time, and is designed for use when ambient temperatures are between 70 and 100 degrees. Please refer to ACI 305 (hot weather) and/or ACI 306 (cold weather) for additional information, as proper curing methods are necessary to ensure best results. Engineer's written approval of submittal required prior to initial shipment. Please fax a copy of approval to 813-684-2892 or mail to 311 Falkenburg Rd South, Tampa, FL 33619, ATTN: Technical Services.

Pursuant to ASTM C-94, section 14.4, the concrete supplier is to be provided with copies of all compressive strength tests. Please ask your testing lab to send reports to 311 Falkenburg Rd. South, Tampa, FL 33619, ATTN: Technical Services.

Our concrete strength guarantee per ASTM C94 will not be effective unless the field sampling has been done per ASTM C172 and ASTM C31 and unless the laboratory testing fully complies with ASTM C39 and ASTM C1077 & Certified through CMEC or Equal. "CMEC"

COMMENTS:

* 150929-Insufficient/no historical data available for this mix. Mix 64165 contains less total cementitious materials (cement-564 flyash=142) and same w/c ratio as 150929. Mix 64165 is submitted as historical reference data for mix 150929.

Figure A-2 Concrete mix design for voided shaft.



TAMPA-LARGO-354
2195 LAKE AVE. SE
LARGO, FL 33771

DATE 09/25/2007	TICKET NO. 35424756
P.O.#	PROJECT NO. 31172775

BILL TO				SHIP TO			
30056615 R W HARRIS INC 12300 44TH STREET N PINELLAS				34405171 4470 122ND AVE N CLEARWATER 4470 122ND AVE N CLEARWATER			
LOAD	LEAVE PLANT	ARRIVE JOB SITE	START DISCHARGE	FINISH DISCH.	LEAVE JOB SITE	ARRIVE PLANT	STAND-BY TIME
1202	1207	1220					
SLUMP	PLANT NO.	TRUCK NO.	DRIVER		MAP ZONE		
9.00 in.	7-9"	354	5078	BARNES, KENNETH L		002	
ORDER	PRODUCT CODE	REFERENCE	USAGE				
50173101	7-400-8-C-C8-79-3-2 74008CC87	15054020	DRILL SHAFT				
LOAD QTY	CUMULATIVE QUANTITY	ORDERED QUANTITY	U/M	PRODUCT CODE	PRODUCT DESCRIPTION	UNIT	EXTENSION
9.00	36.00	63.00	Yd ³	150929	DS4000PSI 1"LS_AF_		
					74008CC87932		
1.00		EA			FUEL SURCHARGE RM		
9.00		Y3			ENVIRONMENTAL PER		
9.00		UA			YARD		
					RETARDER(2)		
Last Service Sent : 3 Tampa Largo #354 - Truck 5043						07 SEP 25 PM 12:02	
Last Service Sent : 3 Tampa Largo #354 - Truck 5043							
SAFETY FIRST!!!							
SPECIAL INSTRUCTIONS #2 RETARDER**49TH ST NORTH OF 118TH AVE N TO EAST ON 122ND AVE N / CLEARWATER CODY - 407 709 1114						SALES TAX	
						TICKET TOTAL	
						SUB-TOTAL	
						STAND-BY TIME	
						TOTAL	

All terms and conditions contained in the Material Quote and Credit Application and Agreement are hereby incorporated into this document and made a part hereof. This document is subject to the terms and conditions appearing on the reverse side hereof.
NOTICE: THE STRENGTH OF CONCRETE DESCRIBED IS BASED UPON A SLUMP OF 3 TO 5 INCHES, OR AS SPECIFIED BY THE APPROVED MIX DESIGN WHEN TESTED IN ACCORDANCE WITH ASTM C-94. THE CUSTOMER IS RESPONSIBLE FOR REQUIREMENTS OF ACI 308A AND 308.

THE UNDERSIGNED AUTHORIZED ADDITION OF _____ GALLONS OF WATER. SLUMP TAKEN: YES NO TEST CYL TAKEN: YES NO

Plant Manager
Ernie Riddlebarger Jr.

REASON FOR STANDBY: _____
CUSTOMER ASSUMES RESPONSIBILITY AND/OR LIABILITY FOR ANY PERSONAL OR PROPERTY DAMAGE WITHIN THE CURB AND PROPERTY LINE. ALL CLAIMS MUST BE MADE AT TIME OF DELIVERY.

RECEIVED AND ACCEPTED BY: *[Signature]*

CUSTOMER 1 COPY 4783220

700D001

4783214

Figure A-2b Concrete Truck Ticket for Voided Shaft Demonstration.

Subject: REQUEST FOR CONCRETE MIX DESIGN APPROVAL

Requested By: Kevin Heindel Phone 651-686-4233
 Firm Name: Cemstone Products Co.
 Agency Engineer/Inspector Kevin Western (MnDOT) SP # (I-35W Bridge)

Proposed Aggregate Sources					
	CA #1	CA #2	CA #3	CA #4	Sand
Pit Number	<u>82001</u>	<u>73006</u>			<u>82001</u>
Pit Name	<u>Grey Cloud</u>	<u>Martin Marietta</u>			<u>Grey Cloud</u>
Nearest Town	<u>Newport</u>	<u>St. Cloud</u>			<u>Newport</u>
Size	<u>3/8" (CA-80)</u>	<u>3/4" (CA-50)</u>			<u>Sand</u>
Sp.G. ¹	<u>2.66</u>	<u>2.72</u>			<u>2.62</u>
Absorption ¹	<u>0.013</u>	<u>0.004</u>			<u>0.010</u>

¹ Provided by Mn/DOT

Proposed Cementitious Sources			
	Cement	Fly Ash	Slag
Manufacturer/Distributor	<u>Lehigh</u>	<u>Headwaters</u>	<u>Holcim</u>
Mill/Power Plant	<u>Mason City, IA</u>	<u>Coal Creek, ND</u>	<u>Chicago, IL</u>
Type/Class	<u>Type I</u>	<u>Class F</u>	<u>Grade 100</u>
Specific Gravity	<u>3.15</u>	<u>2.55</u>	<u>2.89</u>

Proposed Mix Designs			
	Drilled Shafts		
Type of Work	<u>ITF5035C</u>		
Mix Number	<u>ITF5035C</u>		
Water (lbs/C.Y.)	<u>270</u>		
Cement (lbs/C.Y.)	<u>242</u>		
Fly Ash (lbs/C.Y.)	<u>108</u>		
Slag (lbs/C.Y.)	<u>359</u>		
W/CM Ratio	<u>0.38</u>		
Sand (Oven Dry, lbs/C.Y.)	<u>1350</u>		
CA #1 (Oven Dry, lbs/C.Y.)	<u>410</u>		
CA #2 (Oven Dry, lbs/C.Y.)	<u>1280</u>		
CA #3 (Oven Dry, lbs/C.Y.)			
CA #4 (Oven Dry, lbs/C.Y.)			
%Air Content	<u>2.0</u>		
Maximum Spread (3" Range)	<u>20" to 23"</u>		
VMA (oz/100 #CM) BASF-358	<u>6.0</u>		
HRWRA (oz/100 #CM) 7500	<u>5.0</u>		
AEA (oz/100 #CM) Daravair			

The above mixes are approved for use, contingent upon satisfactory site performance and continuous acceptability of all materials sources, by:

Requested By		Date
Mn/DOT Reviewer		Date
Reviewed by: Mn/DOT Concrete Office		Date

Comments: Mix ITF5035C is for information purposes only and has been created by adjusting the aggregate proportions of mix ITF5035B so that the JMF may be met. No new JMF for mix ITF5035C has been created.

Written by: <u>FMM</u>	Revised by:	Approved by:
Date: <u>10.08.07</u>	Date:	Date:

Figure A-3 Mix design for the I35W drilled shafts.



SUPPLIER : **CEMEX** 13-Jan-09
 MIX CODE NO. : **1389153** Mix Code Number must be used when ordering concrete
 DESCRIPTION : 4000 PSI IN 28 DAYS - REGULAR WITH HRWR (GRANITE), RADIATION SHIELDING
 CLIENT :
 PROJECT :

RMD,400,RAD,6903,2:GAP-HRW

MATERIAL	SOURCE	DESCRIPTION	SPECIFIC GRAVITY	WEIGHT (lbs/cy)
CEMENT	CEMEX	ASTM C-150.....TYPE I CEMENT	3.15	775
FINE AGG.	CEMEX	ASTM C-33.....NATURAL SAND	2.63	1222
COARSE AGG.	CONRAD YELVINGTON	ASTM C-33.....#57 GRANITE	2.66	1900
WATER		ASTM C-94 30.5 GALS.	1.00	254
ADMIXTURE 1	W.R. GRACE	ASTM C-494,D.....RECOVER		
ADMIXTURE 2	W.R. GRACE	ASTM C-494 A,D.....WRDA 60		
ADMIXTURE 3	W.R. GRACE	ASTM C-494,F.....ADVA 120		
TOTALS				4151

DESIGNED SLUMP : 6" to 9" DESIGNED UNIT WEIGHT: 153.7 lbs./c.f.
 DESIGNED AIR : 2.0% Entrapped DESIGNED W/C RATIO: 0.33

NOTES:

CEMEX has no knowledge or authority regarding where this mix is to be placed. Therefore, it is the responsibility of the project architect/engineer, and or contractor to insure that the above designed mix parameters of compressive strength, water cement ratio, cement content, and air content are appropriate for the anticipated environmental conditions (ie, ACI-318-02 Chapter 4, and the local Building Codes).

Chemical admixtures are added in accordance with the manufacturers recommendations.

Designed mix cementitious content is stated as a minimum, and CEMEX reserves the right to increase cementitious content. CEMEX also reserves the right to adjust aggregate weights to maintain design aggregate volumes.

Superplasticizer (Admixture 3) will be added to a target slump of 2-3" at the batching plant to achieve a 9" maximum slump at the jobsite.

For and on behalf of;

CEMEX

Wm. "Tag" Herring

William "Tag" Herring
 Mix Design Specialist

Florida Region
 5325 SR 64 East, Bradenton, FL 34208, USA, Cell (941) 504-0265

Figure A.4a Mix design for USF nuclear vault project.



Brooksville South Plant
 10311 CEMENT PLANT ROAD
 Brooksville, FL 34601
 Phone (352) 799-7881 / FAX (352) 799-6068

CEMENT MILL TEST REPORT

Cement Identified as: AASHTO M85; ASTM C150 TYPE I/II
 Plant: Cemex Brooksville Cement
 Location: Brooksville, FL
 Production Date: 11/01/08 to 11/30/08

Date of Report: 12/2/08
 Silo 2,4 & 5

STANDARD CHEMICAL REQUIREMENTS (ASTM C114)	SPECIFICATIONS	ASTM C-150	ASTM C-150	ASTM C-1157	AASHTO M-85	AASHTO M-85	TEST RESULTS
		TYPE I Low alkali	TYPE II Low alkali	GU	TYPE I Low alkali	TYPE II Low alkali	
Silicon Dioxide (SiO2) %	Minimum	---	---	---	---	20.0	20.7
Aluminum Oxide (Al2O3) %	Maximum	---	6.0	---	---	---	4.8
Ferric Oxide (Fe2O3) %	Maximum	---	6.0	---	---	6.0	4.9
Calcium Oxide (CaO) %	---	---	---	---	---	---	64.3
Magnesium Oxide (MgO) %	Maximum	6.0	6.0	---	6.0	6.0	0.6
Sulfur Trioxide (SO3) % **	Maximum	3.5	3.0	---	3.5	3.0	3.2
Loss on Ignition (LOI) %	Maximum	3	3	---	3	3	2.3
Insoluble Residue (IR) %	Maximum	0.75	0.75	---	0.75	0.75	0.38
Alkalies (Na2O equivalent) %	Maximum	0.60	0.60	---	0.60	0.60	0.37
Carbon Dioxide in cement (CO2) %	---	---	---	---	---	---	0.51
Limestone % in cement (ASTM C150 A1)	Maximum	5	5	---	5	5	1.3
CaCO3 in limestone % (2.274 x %CO2 LS)	Minimum	70	70	---	70	70	89
Tricalcium Silicate (C3S) %	Maximum	---	---	---	---	---	58
Dicalcium Silicate (C2S) %	---	---	---	---	---	---	18
Tricalcium Aluminate (C3A) %	Maximum	---	8	---	---	8	6
Tetra calcium Aluminoferrite (C4AF) %	---	---	---	---	---	---	12
(C3S + 4.75 C3A) (C4AF + 2C3A) or (C4AF + C2F) %	Maximum	---	100	---	---	100	85
	Maximum	---	---	---	---	---	24
PHYSICAL REQUIREMENTS							
(ASTM C204) Blaine Fineness, cm2/g	Minimum	2800	2800	---	2800	2800	3860
(ASTM C204) Blaine Fineness, cm2/g	Maximum	---	4200	---	---	4200	3860
(ASTM C430) -325 Mesh %	---	---	---	---	---	---	98.0
(ASTM C191) Time of Setting (Vicat)							
Initial Set, minutes	Minimum	45	45	45	45	45	79
Final Set, minutes	Maximum	375	375	420	375	375	180
(ASTM C185) Air Content of Mortar %	Maximum	12	12	---	12	12	7.6
(ASTM C151) Autoclave Expansion %	Maximum	0.80	0.80	0.80	0.80	0.80	-0.030
(ASTM C187) Normal Consistency %	---	---	---	---	---	---	24.8
(ASTM C1538) Expansion in Water %	Maximum	0.02	0.02	0.02	0.02	0.02	0.011
(ASTM C184) 7 day Heat of Hydration cal/g	Max. If specified	---	---	---	---	70	78
(ASTM C109) Compressive Strength, psi (Mpa)							
1 Day	---	---	---	---	---	---	1390 (13.7)
3 Days	Minimum	1740 (12.0)	1450 (10.0)	1450 (10.0)	1740 (12.0)	1450 (10.0)	3580 (24.7)
7 Days	Minimum	2760 (19.0)	2470 (17.0)	2465 (18.0)	2760 (19.0)	2470 (17.0)	4890 (32.3)
28 Days	Minimum	---	---	---	---	---	6432 (44.4)

** The performance of Cemex Type I/II has proven to be improved with sulfur trioxide levels in excess of the 3.0% limit for Type II.
 As per footnote D (Table One ASTM C150) our Optimum SO3 is 3.51 % (ASTM C583-04). Cement tested according to C1028-4 allowed .011 exp at 3.75% SO3

The data shown above is typical of the cement produced at Brooksville, FL, and currently being shipped from this silo.
 This Cement contains Limestone.

Cemex hereby certifies that this cement meets or exceeds the chemical and physical Specifications of:

- ASTM C-150 for Type I and Type I low alkali
- ASTM C-150 for Type II and Type II low alkali
- ASTM C-1157 GU
- AASHTO M-85 for Type I and Type I low alkali
- AASHTO M-85 for Type II and Type II low alkali

Oliver H. Sohn
 Oliver H. Sohn
 Quality Control Manager

Cemex is not responsible for the improper use or workmanship that may be associated with the use of this cement.

Figure A.4b Mix design for USF nuclear vault project (continued).

CONCRETE TEST REPORT

CLIENT: **Precise Construction**
5026 Trenton Street
Tampa, Florida 33619
Attn: Mr. Rod Reinhold

DATE: **4/2/2009**
 JOB NO: **6111-08-097 A**

PROJECT: **Bruce B. Downs Medical Center**
Office Building & Vault

LABORATORY NO:
 SET NO: **06**

FIELD INFORMATION				FIELD TEST DATA			
DATE SAMPLED:	3/5/2009	BY:	N. SUAREZ	TEMPERATURE: AIR (° F):	75	TEMP CONCRETE (° F):	78
CONCRETE SUPPLIER:	CEMEX	PLANT:	4062	0 GALS WATER ADDED TO CY			
TRUCK NO:		TICKET NO:	30682270	ADD'L WATER AUTH. BY:			
TIME MIXED:	12:55 PM	TIME SAMPLED:	1:40 PM	MEASURED		SPECIFIED	
DESIGN MIX NO:	1389153	SIZE OF LOAD (CY):	8	SLUMP (IN.)	7 1/2	6 - 9	
DESIGN COMP. STR. AT 28 DAYS (PSI):			4000	AIR CONTENT (%):			
DATE REC'D N LAB:			6/6/2009	UNIT WEIGHT (PCF):	153.6	153.7	
LOCATION OF POUR:							
VAULT PIT FOOTING				FIELD REMARKS			
				THE UNIT WEIGHT TEST NOTED ABOVE IS THE PLASTIC CONCRETE WEIGHT AS			
				PER ASTM C-138. THE AIR DRY UNIT WEIGHT BASED ON THE WEIGHT OF ONE			
				TEST CYLINDER DRIED IN AN AIR CONDITIONED SPACE FOR 28 DAYS.			

LABORATORY TEST DATA								
CYL NO.	DATE OF TEST	AGE (DAYS)	LOAD (lbs)	Diameter (inches)	AREA (sq. in.)	COMPRESSIVE STRENGTH (psi)	TESTED BY	FRACTURE TYPE
A	3/12/09	7	101,580	4.05	12.88	7890	CM	1
B	4/2/09	28	122,970	4.05	12.88	9550	PH	5
C	4/2/09	28	123,550	4.05	12.88	9590	PH	5
D	4/2/09	AIR DRY	UNIT WEIGHT			151.5 PCF		

SAMPLES ARE 4" DIAMETER AND 8" LENGTH UNLESS OTHERWISE NOTED
 SAMPLES CURED IN ACCORDANCE WITH ASTM C-31 UNLESS OTHERWISE NOTED
 FRACTURE TYPE AS PER ASTM C-39-2004

REMARKS:

Figure A.4c Concrete test results for USF nuclear vault project.



Date Issued: 8/7/2008

SUPPLIER : CEMEX Gainesville
 MIX CODE : 1345348 *Mix Code must be used when ordering concrete.*
 DESCRIPTION : 5000 psi in 28 days - Seal Mix
 CLIENT : Case Atlantic
 PROJECT : Southwest Parking Garage Complex
 RMS,5000,SEAL,8",3%,T1A20%,

Material	Source	Description	Spec. Grav.	Weight (lb)
Type I-II	CEMEX	ASTM C-150, Type I/II Cement	3.15	633
Flyash F	CEMEX	ASTM C-618, Class F Flyash	2.45	158
Fine Aggregate	CEMEX	ASTM C-33, Natural Sand	2.63	1,181
Coarse Aggregate	CEMEX	ASTM C-33, #57 Limestone	2.45	1,477
Air Entrainer	W.R. Grace	ASTM C-260, Type AEA, Darex		
Water Reducer & Retarder	W.R. Grace	ASTM C-494, Type D, WRDA 60		
Potable		ASTM C-94, Potable 38.0 Gal.	1.00	317
TOTAL				3766

Specified Slump 7.00 To 9.00 in.
 Specified Air: 1.50 To 4.50 %

Designed Unit Weight 139.6 lbs./cu.ft.
 Designed W/C Ratio: 0.40

NOTES:

CEMEX has no knowledge or authority regarding where this mix is to be placed. Therefore, it is the responsibility of the project architect/engineer, and or contractor to insure that the above designed mix parameters of compressive strength, water cement ratio, cement content, and air content are appropriate for the anticipated environmental conditions (ie, ACI-318-96 Chapter 4, and the local Building Codes).

Chemical admixtures are added in accordance with the manufacturers recommendations.

Designed mix cementitious content is stated as a minimum, and CEMEX reserves the right to increase cementitious content. CEMEX also reserves the right to adjust aggregate weights to maintain design aggregate volumes.

COMMENTS:

For and on behalf of:
 Rinker Materials of Florida, Inc., d/b/a CEMEX

Scott Emerson
 Mix Design Specialist

Florida Region
 1557 Jupiter Park Drive, Suite 1, Jupiter, FL 33458, USA, (561) 745-5240 Fax (561) 746-1727

Figure A-5a. UF-290 Project Mix Design



ASTM C 618 TEST REPORT

Project Name: August QC
Sample Number: 6121-01

Report Date: 11/1/2006
Sample: Crystal River
Tested By: JT/JX

TESTS	RESULTS	ASTM C 618 CLASS F/C	AASHTO M 295 CLASS F/C
Crystal River Monthly QC			
CHEMICAL TESTS			
Silicon Dioxide (SiO ₂), %	55.59		
Aluminum Oxide (Al ₂ O ₃), %	25.49		
Iron Oxide (Fe ₂ O ₃), %	5.93		
Sum of SiO ₂ , Al ₂ O ₃ , Fe ₂ O ₃ , %	87.01	70.0/50.0 min.	70.0/50.0 min.
Calcium Oxide (CaO), %	2.54		
Magnesium Oxide (MgO), %	1.18		
Sulfur Trioxide (SO ₃), %	0.38	5.0 max.	5.0 max.
Sodium Oxide (Na ₂ O), %	0.64		
Potassium Oxide (K ₂ O), %	2.48		
Total Alkalies (as Na ₂ O), %	2.27		
PHYSICAL TESTS			
Moisture Content, %	0.01	3.0 max.	3.0 max.
Loss On Ignition, %	4.00	6.0 max.	5.0 max.
Amount Retained on No. 325 Sieve, %	25.00	34 max.	34 max.
Specific Gravity	2.23		
Autoclave Soundness, %	-0.01	0.8 max.	0.8 max.
SAI, with Portland Cement at 7 days, % of Control	75.5	75 min.*	75 min.*
SAI, with Portland Cement at 28 days, % of Control	79.3	75 min.*	75 min.*
Water Required, % of Control	98.8	105 max.	105 max.

Meets ASTM C618, AASHTO M295, FDOT Section 929, SCDHPT and MDOT specifications for Class F Fly Ash.

*Meeting the 7 day or 28 day strength activity index will indicate specification compliance.

Approved By:

Diana Bertfield
Diana Bertfield
QC Specialist

Approved By:

Brian Shaw
Brian Shaw
Materials Testing Manager

Figure A-5b. UF-290 Project Mix Design (continued).



Brooksville South Plant
 10311 CEMENT PLANT ROAD
 Brooksville, FL 34601
 Phone (352) 799-7881 / FAX (352) 799-6088

CEMENT MILL TEST REPORT

Cement Identified as: AASHTO M85; ASTM C150 TYPE I/II
 Plant: Cemex Brooksville Cement
 Location: Brooksville, FL
 Production Date: 04/01/08 to 04/30/08

Date of Report: 05/07/08

Silo 2

STANDARD CHEMICAL REQUIREMENTS (ASTM C114)	SPECIFICATIONS	ASTM C-150	ASTM C-150	ASTM C-1157	AASHTO M-85	AASHTO M-85	TEST RESULTS
		TYPE I Low alkali	TYPE II Low alkali	GU	TYPE I Low alkali	TYPE II Low alkali	
Silicon Dioxide (SiO ₂) %	Minimum	---	---	---	---	20.0	20.4
Aluminum Oxide (Al ₂ O ₃) %	Maximum	---	6.0	---	---	6.0	4.9
Ferric Oxide (Fe ₂ O ₃) %	Maximum	---	6.0	---	---	6.0	3.7
Calcium Oxide (CaO) %		---	---	---	---	---	64.1
Magnesium Oxide (MgO) %	Maximum	6.0	6.0	---	6.0	6.0	0.8
Sulfur Trioxide (SO ₃) % **	Maximum	3.5	3.0	---	3.5	3.0	3.1
Loss on Ignition (LOI) %	Maximum	3	3	---	3	3	2.9
Insoluble Residue (IR) %	Maximum	0.75	0.75	---	0.75	0.75	0.50
Alkalies (Na ₂ O equivalent) %	Maximum	0.60	0.60	---	0.60	0.60	0.36
Carbon Dioxide in cement (CO ₂) %		---	---	---	---	---	1.40
Limestone % in cement (ASTM C150 A1)	Maximum	5	5	---	5	5	3.6
CaCO ₃ in limestone % (2.274 x %CO ₂ LS)	Minimum	70	70	---	70	70	89
Tricalcium Silicate (C ₃ S) %	Maximum	---	---	---	---	---	59
Dicalcium Silicate (C ₂ S) %		---	---	---	---	---	14
Tricalcium Aluminate (C ₃ A) %	Maximum	---	8	---	---	8	7
Tetracalcium Aluminoferrite (C ₄ AF) %		---	---	---	---	---	11
(C ₃ S + 4.78 C ₃ A)	Maximum	---	100	---	---	100	92
(C ₄ AF + 2C ₃ A) or (C ₄ AF + C ₂ F) %	Maximum	---	---	---	---	---	25
PHYSICAL REQUIREMENTS							
(ASTM C204) Blaine Fineness, cm ² /g	Minimum	2800	2800	---	2800	2800	3933
(ASTM C204) Blaine Fineness, cm ² /g	Maximum	---	4200	---	---	4200	3933
(ASTM C430) -325 Mesh %		---	---	---	---	---	94.4
(ASTM C191) Time of Setting (Vicat)							
Initial Set, minutes	Minimum	45	45	45	45	45	116
Final Set, minutes	Maximum	375	375	420	375	375	216
(ASTM C185) Air Content of Mortar %	Maximum	12	12	---	12	12	10.0
(ASTM G151) Autoclave Expansion %	Maximum	0.80	0.80	0.80	0.80	0.80	-0.010
(ASTM C187) Normal Consistency %		---	---	---	---	---	24.7
(ASTM C1038) Expansion in Water %	Maximum	0.02	0.02	0.02	0.02	0.02	---
(ASTM C109) Compressive Strength, psi (Mpa)							
1 Day		---	---	---	---	---	1930 (13.3)
3 Days	Minimum	1740 (12.0)	1450 (10.0)	1450 (10.0)	1740 (12.0)	1450 (10.0)	3590 (24.8)
7 Days	Minimum	2760 (19.0)	2470 (17.0)	2465 (15.0)	2760 (19.0)	2470 (17.0)	4860 (33.5)
28 Days	Minimum	---	---	---	---	---	6394 (44.1)

** The performance of Cemex Type III has proven to be improved with sulfur trioxide levels in excess of the 3.0% limit for Type II.
 Note D in ASTM C-150 allows for additional sulfate, provided expansion as measured by ASTM C-1038 does not exceed 0.020%

The data shown above is typical of the cement produced at Brooksville, FL and currently being shipped from this silo.
 This Cement contains Limestone.

Cemex hereby certifies that this cement meets or exceeds the chemical and physical specifications of:

- ASTM C-150 for Type I and Type II low alkali
- ASTM C-150 for Type II and Type II low alkali
- ASTM C-1157 GU
- AASHTO M-85 for Type I and Type I low alkali
- AASHTO M-85 for Type II and Type II low alkali

Eduardo Caballero Aguilar
 Corporate Quality Control Manager

Cemex is not responsible for the improper use or workmanship that may be associated with the use of this cement.

Figure A-5c. UF-290 Project Mix Design (continued).



1800 N.W. 58th Lane
 Ocala, FL 34475
 Phone: 352-854-0169
 Fax: 352-854-1209



CLIENT: Reliable Constructors
 PROJECT: Marion county Judicial Center
 DATE: January 10, 2008

Project:
 Reviewed
 Reviewed As Noted
 Revise & Resubmit
 Rejected
 By: JAM Date: 1/13/08

MIX NO. 1F054025

Drilled Shaft
 CLASS: 5,000 psi

SOURCE OF MATERIALS

Cement:	Suwannee	Type	VII	3.15	S.G.	ASTM C 150
Flyash:	Boral	Class	F	2.25	S.G.	ASTM C 618
Coarse Aggregate : #1	Limestone	# 67	@	2.33	S.G.	ASTM C 33
Coarse Aggregate : #2	Limestone		@	2.34	S.G.	ASTM C 33
Fine Aggregate :	Silica Sand	F.M.	2.2 @	2.63	S.G.	ASTM C 33
Screenings:	None	F.M.	2.4 @	2.33	S.G.	ASTM C 33
Retarder/Water reducer	WRDA 60 by W.R. Grace	Type		A/D		ASTM C 494
Admixture #1:	N/A	Type		A/F		ASTM C 494
Admixture #2:	N/A	Type				ASTM C 494
Admixture #3:	Recover	Type		D		ASTM C 494
Air Entrainment Admix:	DAREX by W.R. Grace					ASTM C 260

MIX DESIGN

MATERIALS PER CUBIC YARD
 (Saturated Surface Dry Aggregates)

TOTAL CEMENT, Per ASTM C595	785
CEMENT, LBS.	585
FLYASH, LBS.	200
COARSE AGGREGATE #1, LBS.	1414
COARSE AGGREGATE #2, LBS.	0
FINE AGGREGATE, LBS.	1147
SCREENINGS :	0
RETARDER/WATER REDUCER	35.1/52.7
ADMIXTURE (#1), OZS.	0
ADMIXTURE (#2), OZS.	0
ADMIXTURE (#3), OZS.	11.7
AIR ENTR. ADMIX, OZS.	0.0
WATER, GAL.	38.0
WATER, LBS.	316.5
TOTAL WEIGHT, Per Cu. Yard, Lbs.	3663
Sand Aggregate Ratio	41.8%

ABSOLUTE VOLUMES

	2.98	Cu.Ft.
	1.42	Cu.Ft.
	9.73	Cu.Ft.
	0.00	Cu.Ft.
	6.99	Cu.Ft.
	0.00	Cu.Ft.
	0.00	Cu.Ft.
	0.81	Cu.Ft.
	5.07	Cu.Ft.

THEORETICAL YIELD

	27.00	Cu.Ft.
--	-------	--------

TEST DATA

SLUMP RANGE	7.0	TO	9.0
AIR CONTENT:	1.5	TO	4.5
UNIT WEIGHT:	135.65	P.C.F.	
WATER/CEMENT RATIO:	0.40	(lbs./lbs.)	

Our concrete strength guarantee per ASTM C94 will not be effective unless the field sampling has been done per ASTM C172 and ASTM C31 and unless the laboratory testing fully complies with ASTM C39 and ASTM C1077 & Certified through CMEC or Equal. "CMEC" "Concrete Materials Engineering Council"

Figure A-5d. Concrete mix design for Marion County Judicial Center.

groups, subgroups were established by varying the water to binder ratio. Additional details may be found in attached Tables A-1 and A-2.

Table 1. HBI Mix Design Details and Test Results

Mix ID	Binder Dosage (C+S) (kg/m ³)*	Water/Binder Ratio (%)	Grout to Soil Ratio by Volume (%)	28 Day Average UCS (psi)	14 Day Average Permeability (cm/sec)
B0.5	90	250	25.8	303	4.0E-08
B0.7	90	300	30.3	208	2.1E-07
B0.9	130	200	30.8	401	2.0E-08
B1	130	250	37.3	310	2.6E-08
B2	130	300	43.8	196	9.8E-08
B4	180	200	42.7	454	2.0E-08
B5	180	250	51.7	337	1.3E-08
B6	180	300	60.7	239	1.0E-07
B8	230	200	54.5	470	1.8E-08
B9	230	250	66.0	354	2.0E-08

All mixes contain Bentonite at a proportion of 0.088 x the weight of the binder
 * kg of binder per m³ of original soil

14-day permeability test results (ASTM D 5084) are included and meet or exceed project requirements

Figure A-5e. Lake Okeechobee Mix Design.

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Appendix B: TSP for Thermal Evaluation

Specification for Thermal Integrity Testing of Drilled Shafts

Thermal Integrity Evaluation of a foundation utilizes the thermal signature generated during the hydration phase of the concrete curing process. Deviations in the thermal signature from a gradient predicted by modeling of the concrete mix design and soil profile can indicate anomalies in the shaft cross-section. A decrease in the measured temperature may indicate a decrease in shaft cross-section, whereas an increase in measured temperature may be indicative of a bulge or increase in the shaft cross-section. Advantages of the Thermal Integrity Evaluation method include the use of access tubes typical of current integrity testing methods, early detection of potential anomalies, anomaly detection outside the reinforcing cage and a reusable temperature measuring system.

Frequency of Testing

Perform all TI testing in bridge bents or piers containing one column supported by one or two drilled shafts, or two columns with one or more of the columns supported by only one drilled shaft. For all other drilled shafts, perform TI testing only on drilled shafts selected by the Engineer, or as shown in the plans. The minimum number of shafts tested is the number of shafts indicated in the plans. At their discretion, the Engineer may increase the number shafts tested as deemed necessary. Engage a qualified Specialty Engineer to perform the TI testing. The qualified TI Specialty Engineer must have a Licensed Professional Engineer supervising the collection and interpretation of data. The Contractor shall provide all necessary assistance to the TI Specialty Engineer to satisfactorily perform the testing.

All tubes in a tested shaft shall be profiled with the TI equipment.

Tube Requirements

Thermal Integrity (TI) Tubes: Install TI access tubes full length in all drilled shafts from the tip of shaft to a point high enough above top of shaft to allow TI testing, but not less than 30 inches above the top of the drilled shaft, ground surface or water surface, whichever is higher. Equally space tubes around circumference of drilled shaft. Securely tie access tubes to the inside of the reinforcing cage and align tubes to be parallel to the vertical axis of the center of the cage. Access tubes must be Schedule 80 PVC with a nominal diameter of 2.0 inches. Couple tubes as required with couplers, such that inside of tube remains flush. Seal the bottom and top of the tubes with threaded caps. The tubes, joints and bottom caps shall be watertight. Seal the top of the tubes with lubricated, threaded caps sufficient to prevent the intrusion of foreign materials. Stiffen the cage sufficiently to prevent damage or misalignment of access tubes during the lifting and installation of the cage. Repair or replace any unserviceable tube prior to concreting. Exercise care in removing the caps from the top of the tubes after installation so as not to apply excess torque, hammering or other stress which could break the bond between the tubes and the concrete. Provide the following number and configuration of TI access tubes in each drilled shaft based on the diameter of the shaft.

Shaft Diameter (inches)	Number of Tubes Required	Configuration around the inside of Circular Reinforcing cage
36 to 48	4	90 degrees apart
54 to 60	5	72 degrees apart
66 to 72	6	60 degrees apart
78 to 84	7	51.5 degrees apart
90 to 96	8	45 degrees apart

Insert simulated or mock probes in each cross-hole-sonic access tube prior to concreting to ensure the serviceability of the tube. Fill access tubes with clean potable water and recap prior to concreting. Repair or replace any leaking, misaligned or damaged tubes as in a manner acceptable to the Engineer prior to concreting. Immediately after concreting, check the water levels in the CSL access tubes and refill as necessary. If tubes become unserviceable, core new holes in the drilled shaft as directed by the Engineer. For the purposes of Thermal Integrity Evaluation, the contractor shall provide reasonable access to the shaft for a time period up to two weeks after concreting or as directed by the Engineer.

After acceptance of production shafts by the Engineer, remove all water from the access tubes or core holes and fill the tubes or core holes with a structural non-shrink grout approved by the Engineer.

Equipment

Furnish Thermal Integrity testing equipment as follows:

- (1) Include thermal probe equipped with a minimum of three (2) Infrared Thermocouples sensors capable of being lowered into a 2.0 inch nominal diameter tube.
- (2) Include a microprocessor based data acquisition system for display, storage, and transfer of data.
- (3) Electronically measure and record the relative position (depth) of the probe within the tube with each TI signal.
- (4) Print the TI logs for report presentation.
- (5) Provide report quality plots of TI measurements that identify each individual test.
- (6) Electronically store each TI log in digital format, with shaft identification, date, time and test details.

Procedure

Field measurements for the purpose of Thermal Integrity Evaluation shall be conducted at a time that corresponds as closely as is practical to the peak temperature generation in the shaft as

directed by the Engineer. A Concrete Mix Design shall be submitted to the Testing Engineer a minimum of ten (10) days prior to shaft construction, in order to determine the proper testing time. The drilled shaft concrete mix design shall be tested for heat production by the State Materials Office or by their designated party. Results from said testing will be modeled to ascertain the time vs. temperature response of the specified shaft geometry and soil profile prior to concreting. In addition, furnish information regarding the shaft, tube lengths and depths, construction dates, and other pertinent shaft installation observations and details to the Specialty Engineer at the time of testing.

- (1) Remove all water immediately prior to performing the testing.
- (2) Lower the probe starting from the top of the tubes, over an electronic depth measuring device.
- (3) Continuously record TI signals at depth intervals of 2.5 inches or less from the top of the tubes to the bottom of each shaft.
- (4) Assure the probe is hanging free without interference of the cable prior to lowering to provide accurate depth measurements in the TI records.
- (5) Report any anomalies indicated by the TI signals to the Engineer and conduct further analyses or tests as required to evaluate the extent of possible defects.

Reporting and Evaluation

Present the TI testing and analysis results to the Engineer in a report. Include TI logs with analyses of the temperature reading of each sensor versus depth for each tube and the average temperature readings of each tube versus depth for all tubes. Identify any readings which indicate an anomaly on the logs and as a discussion item in the report.

The Engineer will evaluate the TI test results and determine whether or not the drilled shaft construction is acceptable. TI test results with deviations greater than 5 degrees over a 1 ft length shall be further evaluated using Signal Matching Analyses to determine the possible shaft cross-section loss of measured anomalies. A 3-D rendering of the shaft shall be included along with the Signal Matching Analyses graphical results, when this analysis is required.

Evaluation of Unacceptable Shafts

If the Engineer determines a drilled shaft is unacceptable based on the TI results, core the shaft to allow further evaluation and repair, or replace the shaft. If coring to allow further evaluation of the shaft and repair is chosen, one or more core samples shall be taken from each unacceptable shaft for full depth of the shaft or to the depth directed by the Engineer. The Engineer will determine the number, location, and diameter of the cores based on the results of TI testing and analysis. Keep an accurate log of cores. Properly mark and place the cores in a crate showing the shaft depth at each interval of core recovery. Transport the cores, along with five copies of the coring log to the Engineer. Perform strength testing by an AASHTO certified lab on portions of

the cores that exhibit questionable concrete as determined by the Engineer. If the drilled shaft TI testing, analyses and coring indicate the shaft is defective, propose remedial measures for approval by the Engineer. Such improvement may consist of, but is not limited to correcting defective portions of the shaft, providing straddle shafts to compensate for capacity loss, or providing a replacement shaft. Repair all detected defects and conduct post repair integrity testing using TI testing as described herein. Submit all results to the Engineer within five days of test completion for approval.

SECTION T455

THERMAL INTEGRITY TESTING OF DRILLED SHAFTS

T455-1 Description: This work consists of installing access tubes and providing safe and secure access assistance to the Engineer for the purpose of evaluating drilled shaft integrity via internal temperature measurements using the Thermal Integrity Test method as described herein. The Thermal Integrity Test method is based on measuring the heat generation of hydrating cement. The analysis of measured temperature profiles requires knowledge of the concrete mix used and soil profile for the purposes of determining heat generation and soil insulation parameters. For typical shaft concrete mixes, thermal testing should be carried out between one and two days after shaft concreting.

The Contractor is not required to provide Cross-Hole Sonic Logging (CSL) Tests.

T455-2 Thermal Integrity Testing Access : Install 1.5 inch ID black iron CSL access tubes full length in all drilled shafts in accordance with Section 455-16.4.

Provide access assistance to the Engineer in testing the shafts within 4 hours of the peak temperature generation. Peak temperature generation is expected to occur between 24 and 48 hrs after shaft concrete placement. The Engineer will test all drilled shafts in bridge bents or piers considered non-redundant in the plans. Based on the observations during drilled shaft construction, the Engineer may test one or all drilled shafts in bridge bents or piers considered redundant in the plans. For all other drilled shafts, only drilled shafts selected by the Engineer will be tested.

T455-3 Evaluation of Thermal Integrity Testing: The Engineer will evaluate the observations during drilled shaft construction and the Thermal Integrity Test results to determine whether or not the drilled shaft construction is acceptable within three working days of testing the shaft. Drilled shafts with either insufficient cover or a 5 degree Fahrenheit reduction from the model norm over a length of shaft at least 2ft in length will not be accepted without an engineering analysis. If the Contractor determines at any time during the non-destructive testing and evaluation of the drilled shaft that the drilled shaft should be replaced, no further testing or evaluation of that shaft is required.

T455-4 Coring and/or Repair of Drilled Shafts: If the Engineer determines a drilled shaft is unacceptable based on the Thermal Integrity Testing, or observes problems during drilled shaft construction, core the shaft to allow further evaluation and repair, or replace the shaft. If coring to allow further evaluation of the shaft and repair is chosen, one or more core samples shall be taken from each unacceptable shaft for full depth of the shaft or to the depth directed by the Engineer. The Engineer will determine the number, location, and diameter of the cores based on the results of 3-D tomographic analysis of Thermal Integrity Testing data. Keep an accurate log of cores. Properly mark and place the cores in a crate showing the shaft depth at each interval of core recovery. Transport the cores, along with five copies of the coring log to the Engineer. Perform strength testing by an AASHTO certified lab on portions of the cores that exhibit questionable concrete as determined by the Engineer.

If the drilled shaft Thermal Integrity Testing, 3-D tomographic analyses and coring indicate the shaft is defective, propose remedial measures for approval by the Engineer. Such measures may consist of, but are not limited to correcting defective portions of the shaft, providing straddle shafts to compensate for capacity loss, or providing a replacement shaft. Repair all detected defects and assist the Engineer in retesting the shaft(s) as described herein.

Perform all remedial work described in this Section at no additional compensation, and with no increase in contract time.

T455-5 Basis of Payment: Include all costs associated with assisting the Engineer with Thermal Integrity Testing in the costs of the Drilled Shafts.