MEMORANDUM

TO: Mike McGregor, P.E. cc: Kyle Poat, P.E.
Chief District Engineer Mike Oliver, P.E.
District 1 Bryan Black, P.E.

FROM: Bart Asher, P.E., P.L.S. David Martin, P.E.
Geotechnical Branch Manager

TO: Kyle Poat, P.E.
cc: Mike Oliver, P.E.
Bryan Black, P.E.
David Martin, P.E.

DATE: November 20, 2013

SUBJECT: Marshall & Trigg Counties
US 68 / KY 80 over Kentucky Lake
Item No. 1-180.70 & 1-180.75
CID No. 13-1305
DRAFT Pile Load Test Program Summary Report

The attached DRAFT report has been completed by Terracon/HCN and is related to the Kentucky Lake test piles (K-1 through K-6). This report is being distributed for the information of the structural design team and the construction & testing team for the advance construction contract (CID 13-1305) under which the load test program was performed. Additionally, it will be posted for the information of prospective bidders for the bridge contract (CID 13-1212). The FINAL report will distributed upon its completion.

If there are any questions or if any additional information is needed, please contact this branch.

Attachment: Referenced Document dated November 19, 2013
Pile Load Test Program Summary Report
Kentucky Lake Bridge Pile Load Test Program
US 68/KY 80 Bridge over Kentucky Lake – Advance Contract
CID 131305
Lake Bridges Project
Marshall and Trigg Counties, Kentucky
November 19, 2013
Terracon Project Number: N1115097

Prepared for:
Kentucky Transportation Cabinet
Frankfort, Kentucky

Prepared by:
Cincinnati, Ohio
November 19, 2013

Kentucky Transportation Cabinet
1236 Wilkinson Boulevard
Frankfort, KY  40601

Attn: Mr. Darrin Beckett, P.E.
    Lake Bridges Geotechnical Project Manager

Re:    Pile Load Test Program Summary Report - Draft
       Kentucky Lake Bridge Pile Load Test Program
       US 68/KY 80 Bridge over Kentucky Lake – Advance Contract
       CID 131305
       Lake Bridges Project
       Marshall and Trigg Counties, Kentucky
       HCN/Terracon Project No. N1115097

Dear Mr. Beckett:

H. C. Nutting, a Terracon Company (HCN), in conjunction with Florence & Hutcheson (F&H),
has completed the geotechnical engineering services for the above-referenced project.
Reviews and independent analyses on the project were provided by Sam Paikowsky, P.E.,
Jerome DiMaggio, P.E. and GRL Engineers, Inc.  This study was performed in general
accordance with our original contract dated June 15, 2010, notice to proceed dated June 24,
2010 and our authorized contract modification documents dated April 5, 2011, and May 14,
2013.  The purpose of this work was to observe and analyze static axial load testing, pseudo-
static axial and lateral load testing and dynamic testing of large-diameter steel pipe piles as
outlined in the Special Notes for the referenced project.

We appreciate the opportunity to be of service to you on this project.  If you have any questions
concerning this report, or if we may be of further service, please contact us.

Sincerely,

H.C. Nutting, A Terracon Company

Jeffrey D. Dunlap, P.E.  Ronald J. Ebelhar, P.E.
Senior Geotechnical Engineer  Senior Principal
TABLE OF CONTENTS

EXECUTIVE SUMMARY ........................................................................................................ i

1.0 INTRODUCTION ............................................................................................................... 1

2.0 PROJECT INFORMATION ................................................................................................... 1
  2.1 Dynamic Load Testing .................................................................................................... 2
  2.2 Static Load Test ............................................................................................................. 3
  2.3 Axial Pseudo-Static Load Tests .................................................................................... 4
  2.4 Lateral Pseudo-Static Load Test .................................................................................. 4
  2.5 Proposed Bridge Axial Foundation Loading Information ............................................ 5

3.0 SUBSURFACE CONDITIONS ............................................................................................... 6
  3.1 Shallow Water Test Area .............................................................................................. 6
  3.2 Deep Water Test Area .................................................................................................. 6
  3.3 Groundwater ................................................................................................................ 7

4.0 LOAD TEST RESULTS ....................................................................................................... 7
  4.1 Dynamic Pile Load Testing ........................................................................................... 7
  4.2 Static Load Test – Test Pile K-1 .................................................................................. 9
  4.3 Axial Pseudo-Static Tests – Test Piles K-2 and K-3 ....................................................... 9
  4.4 Lateral Pseudo-Static Tests – Test Pile K-3 ................................................................. 10

5.0 DATA ANALYSIS .............................................................................................................. 11
  5.1 Dynamic Pile Load Testing ......................................................................................... 11
  5.2 Static Load Testing ....................................................................................................... 12
  5.3 Pseudo-Static Load Testing ......................................................................................... 13
  5.4 Lateral Pseudo-Static Load Testing ............................................................................ 13
  5.5 Overall Evaluation ........................................................................................................ 14
    5.5.1 Setup Factors ....................................................................................................... 14
    5.5.2 LRFD Resistance Factors .................................................................................... 16

6.0 CONCLUSIONS AND RECOMMENDATIONS .................................................................. 17
### APPENDIX A – PILE LOAD TEST PROGRAM

<table>
<thead>
<tr>
<th>Exhibit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-1</td>
<td>Exhibit A-1: Pile Load Test Program Locations</td>
</tr>
<tr>
<td>A-2</td>
<td>Exhibit A-2: Pile Load Test Program Locations</td>
</tr>
<tr>
<td>A-3 to A-15</td>
<td>Exhibits A-3 to A-15: Test Pile Schematic Drawings</td>
</tr>
<tr>
<td>A-16 to A-21</td>
<td>Exhibits A-16 to A-21: Dynamic Load Testing Data Summary Tables</td>
</tr>
<tr>
<td>A-31</td>
<td>Exhibit A-31: Time, dates and elapsed time for driving, restrike and testing of piles</td>
</tr>
<tr>
<td>A-33</td>
<td>Exhibit A-33: Shallow Water – 72-Inch Diameter Pipe Pile</td>
</tr>
<tr>
<td>A-34</td>
<td>Exhibit A-34: Deep Water – 72-Inch Diameter Pipe Pile</td>
</tr>
<tr>
<td>A-35</td>
<td>Exhibit A-35: Load-Displacement data reported for Pile K-1 static load test and its interpretation</td>
</tr>
<tr>
<td>A-36</td>
<td>Exhibit A-36: Side Shear vs. Displacement (t-z curve) Test Pile K-2</td>
</tr>
<tr>
<td>A-38</td>
<td>Exhibit A-38: Side Shear vs. Displacement (t-z curve) Test Pile K-3</td>
</tr>
<tr>
<td>A-40</td>
<td>Exhibit A-40: Derived static top of pile response from the Statnamic test compared with the L-Pile model results</td>
</tr>
<tr>
<td>A-41</td>
<td>Exhibit A-41: Displacement profiles measured from the Statnamic test compared with the L-Pile model results</td>
</tr>
<tr>
<td>A-42</td>
<td>Exhibit A-42: Normalized Setup after Intial Drive – 72-inch-diameter Pipe Pile – 8511 kips Static Nominal Resistance (Statnamic)</td>
</tr>
<tr>
<td>A-44</td>
<td>Exhibit A-44: Observed Test Pile Set Up Characteristics</td>
</tr>
<tr>
<td>A-45</td>
<td>Exhibit A-45: Estimated Axial Pile Resistances at Pier Locations Using Superposition Methods</td>
</tr>
</tbody>
</table>

### APPENDIX B – RECOMMENDATIONS FOR PILE DESIGN AND CONSTRUCTION
EXECUTIVE SUMMARY

A pile load test program has been performed for the proposed Kentucky Lake Bridge, as part of the US68/KY 80 Bridge over Kentucky Lake Advance Contract (CID 131305) in Marshall and Trigg Counties, Kentucky. The following list provides our summary comments of the principal findings.

- An extensive pile load test program was performed to study the constructability and design parameters for large-diameter steel pipe piles.
- The pile load test program included static axial pile load tests, axial pseudo-static pile load tests, a lateral pseudo-static pile load test and dynamic pile testing in general accordance with the special notes for the referenced project.
- The pile load test program confirmed that the proposed 72-inch-diameter steel pipe piles and 48-inch-diameter pipe piles with wall thicknesses ranging from 1.0 to 2.0 inches can be installed to the anticipated target nominal resistances and respective elevations at the site using a Menck MHU800S hydraulic hammer.
- The pile load test program indicated that the proposed steel constrictor plates should be installed within the pipe piles to increase the bearing resistance of the pipe piles. The pile load test program also helped the design team to gather information regarding viable design of the constrictor plates and how to locate the constrictor plates relative to the pile tip location. The pile load test also allowed the design team to gather information concerning what soil strata the constrictor plates should bear within to mobilize additional bearing resistance in the pile design.
- The lateral pseudo-static pile load testing confirmed the lateral soil response parameters used in the initial foundation design phase of the project.
- Axial static and pseudo-static pile load testing indicated that the skin resistance values used in the pile design phase were valid and likely somewhat conservative. However, the axial pile load testing indicated that the 48-inch-diameter piles were more likely to achieve a plugged condition with the steel constrictor plates located 98 feet above the design pile tip than the 72-inch-diameter piles at the test locations. When the piles were driven to or near the design pile tip elevation at the test locations, the 72-inch-diameter piles did not achieve a fully plugged condition with the steel constrictor plates located 98 feet above the pile tip.
- Dynamic testing indicated that both the side resistance and end bearing resistance of the pipe piles at the site will increase with time, requiring up to 25 days or more to show correlation with static / pseudo-static test results.
- Dynamic testing also indicated that design pile nominal resistances can be achieved and confirmed in the field.
- Dynamic testing indicated that 2.0-inch-thick wall pile thickness for the specified 72-inch-diameter steel pipe piles will likely be required to avoid potential pile damage during installation.
The dynamic testing also indicated that the specified pipe piles can likely be installed to Elevation 168 feet at the site without exceeding specified driving stresses.

- The dynamic testing indicated that a nearly- or completely-plugged condition can be achieved in the 6-foot-diameter piles, provided the steel constrictor plates are located such that the constrictor plates bear within a relatively dense granular soil strata (i.e. near Elevation 267 feet).

- In the plugged piles, a smaller or lower-energy pile hammer than the one used during the test program will not develop adequate energy to confirm the bearing resistance of the piles and may not be able to drive the 72-inch-diameter piles to the target elevations.

- The vibration monitoring program indicates that the pile driving activities will not likely generate damaging vibration levels at the existing bridge structural elements. However, vibration monitoring is recommended during construction to provide real-time indications of vibration levels should conditions vary from those encountered during this test program.

- Static and Statnamic testing indicate that dynamic testing performed during bridge construction may not be capable of confirming the long-term static axial resistance of the large diameter piles due to dynamic testing typically underestimating long-term static resistance. It will be necessary to drive the pipe piles to resistances measured using dynamic testing methods that are lower than the required axial nominal resistance.

- Recommendations for resistance factors for the Kentucky Lake Bridge pier piles are provided based upon the testing performed in the load test program and the variability of soil conditions encountered in the test borings.

This summary should be used in conjunction with the entire report for design purposes. It should be recognized that details were not included or fully developed in this section, and the report must be read in its entirety for a comprehensive understanding of the items contained herein. The section titled GENERAL COMMENTS should be read for an understanding of the report limitations.
1.0 INTRODUCTION

A pile load test program has been completed for the proposed Kentucky Lake Bridge Advance Contract (CID 131305) in Marshall and Trigg Counties, Kentucky. The work was performed in general accordance with the Special Notes for Pile Load Test Program, Dynamic Pile Testing and Static and Pseudo-Static Pile Testing, which were included in the contract documents for CID 131305. This report summarizes the observations made in the field and the analysis of data collected during the pile load test program. Applied Foundation Testing (AFT) was the pile testing consultant for Massman Construction who installed the test piles and constructed the platforms and pile load test frames. Massman Construction was a subcontractor to Jim Smith Contracting, who was the prime contractor for the Advance Contract.

2.0 PROJECT INFORMATION

The pile load test program was performed in general accordance with the Special Note for Pile Load Test Programs and the Special Notes for Dynamic Pile Testing, and Static and Pseudo-Static Pile Testing. The pile load test program consisted of testing six test piles designated as K-1 through K-6. The test piles consisted of both 48- and 72-inch-diameter steel pipe piles having wall thicknesses ranging from 1.0 inches to 2.0 inches. Constrictor plates were placed in the interior of the piles in an effort to force a plugged condition to occur in the test piles, in order to take advantage of higher axial pile resistances in the piles. The constrictor plate was located 98 feet above the tip of Piles K-1 to K-5 and 76 feet above the tip of Pile K-6, which was lower due to questions about drivability to highest allowable tip elevations required for lateral loading conditions. The test piles were manufactured using ASTM A572 Grade 50 steel. The test piles were fabricated by Greens Bayou Pipe Mill in Houston, Texas. The test piles were fabricated using steel plates shaped into half-circles and welded together to created 10-foot-long round pipe sections by welding the half-circle sections along the longitudinal seams. Individual round pipe sections were then welded together, end-to-end, to create the pipe piles. Pile shoes flush on the outside of the pile section were installed on each test pile.
Five of the six test piles were installed in the vicinity of the relatively shallow water (20 feet deep or less) near the north toe of the proposed west causeway, near the location of the proposed west end bent location of the Kentucky Lake Bridge. One test pile was installed in the deeper water areas (up to 70 feet deep) near the location of the east end of the proposed Main Span of the Kentucky Lake Bridge. Drawings are included in Appendix A (Exhibits A-1 and A-2) showing the two proposed pile load test areas for Kentucky Lake. The following table outlines the locations of the installed test piles by referencing the US68/KY80 Centerline, the pipe pile diameter and the pipe wall thickness.

<table>
<thead>
<tr>
<th>Test Pile</th>
<th>Station and Offset</th>
<th>Pipe Pile Diameter (inches)</th>
<th>Pipe Wall Thickness (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>K-1</td>
<td>984+50, 100 feet Left</td>
<td>48</td>
<td>1.5</td>
</tr>
<tr>
<td>K-2</td>
<td>988+50, 100 feet Left</td>
<td>48</td>
<td>1.5</td>
</tr>
<tr>
<td>K-3</td>
<td>988+00, 100 feet Left</td>
<td>72</td>
<td>2.0</td>
</tr>
<tr>
<td>K-4</td>
<td>985+00, 100 feet Left</td>
<td>48</td>
<td>1.0</td>
</tr>
<tr>
<td>K-5</td>
<td>984+00, 100 feet Left</td>
<td>72</td>
<td>1.5</td>
</tr>
<tr>
<td>K-6</td>
<td>1009+40, Centerline</td>
<td>72</td>
<td>2.0</td>
</tr>
</tbody>
</table>

As mentioned previously, the scope of the pile load test program was performed in general accordance with the Special Notes for the Advance Contract. However, two exceptions to the scope were authorized and directed by the Department. The first scope change involved the driving criteria of the piles. The initial estimated pile tip elevation in the shallow water test area was changed from Elevation 220 feet (Table 1.4.1 of the Special Note for Pile Load Test Program) to Elevation 210 feet, and the initial estimated pile tip elevation in the deep water test area was changed from Elevation 200 feet (Table 1.4.1 of the Special Note for Pile Load Test Program) to Elevation 190 feet. This change was made after Pile K-1 was driven to an approximate pile tip of Elevation 211 feet, and Pile K-4 was driven to an approximate tip of Elevation 192 feet.

The second scope change authorized by the Department was to splice a pile section on to Pile K-3 for a second extended driving sequence. The pile section for the splice was obtained by cutting the portion of Pile K-6 that was above the mudline. Once the splice was made to K-3, Pile K-3 was driven to a tip elevation of approximately Elevation 168.7 feet.

### 2.1 Dynamic Load Testing

Dynamic load testing was performed on each test pile during initial driving, extended driving and during a second extended drive (when applicable). The dynamic load testing was performed on each test pile by Applied Foundation Testing between August 3, 2013 and September 25, 2013. Individual reports of each driving sequence and associated restrikes and final summary reports were prepared and submitted by Applied Foundation Testing for each test pile. The dynamic load testing was performed in general accordance with the Special Note for Dynamic Load Testing. The following table summarizes the dynamic testing performed on each test pile.
Please refer to Exhibits A-3 to A-15 in Appendix A of this report for schematics showing the pile tip and steel plate elevations on the interior of the test piles at the end of initial drive, extended drive and/or second extended drive. Additional information provided in the schematics show the approximate mudline elevation at the test pile location, the water elevation and the date it was recorded, the interpreted soil strata from the static pile resistance calculations, the soil parameters used in the static API pile resistance calculations, and the elevation of the soil material within the pile above the steel plate.

<table>
<thead>
<tr>
<th>Test Pile</th>
<th>Initial Drive</th>
<th>Extended Drive</th>
<th>2nd Extended Drive</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>End of Drive</td>
<td>Approximate 48-hour Restrike</td>
<td>Approximate 72-hour Restrike</td>
</tr>
<tr>
<td>K-1</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>K-2</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>K-3</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>K-4</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>K-5</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>K-6</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

Notes: 1. 24-hour restrike after second extended drive.
2. 358.8-hour restrike after second extended drive.

### 2.2 Static Load Test

A static load test was performed on Test Pile K-1. At the time of the static load test, the pile tip elevation was at approximately Elevation 209 feet. The load test frame was designed by Genesis Structures, a subconsultant to Massman Construction and Jim Smith Contracting. Monitoring of the test was performed by Applied Foundation Testing. The static load test frame was designed to safely apply a load of 6000 kips at the head of Pile K-1 along the vertical axis of the pile. The static load test was performed in general accordance with the Special Note for Static and Pseudo-Static Pile Testing. The test pile was instrumented using vibrating wire strain gages at ten depth intervals along four vertical lines equally spaced around the perimeter of the exterior of the pipe pile. The strain gages were protected during driving by welding a steel angle over the gages and associated wires. In addition, three telltales were installed along the exterior of the pile and terminated near the toe of Pile K-1. The top of pile movement was measured using four displacement transducers mounted on a reference beam. Backup pile head measurements and measurements of reaction pile movements were measured using survey methods. An Enerpac Synchronous Lift System provided the applied load and hydraulic jack elongation using six – 600-ton hydraulic jacks. The static pile load test was performed between September 3 and 4, 2013. Please refer to Applied Foundation Testing’s static load test report dated September 23, 2013, for additional details regarding pile load test set-up, the pile instrumentation, the procedures used to perform the test and information regarding the collected data.
2.3 Axial Pseudo-Static Load Tests

Axial pseudo-static (Statnamic) load testing was performed on Test Piles K-2 and K-3. At the time of the pseudo-static load testing, the K-2 pile tip elevation was at approximately Elevation 209.9 feet and the K-3 pile tip elevation was at approximately Elevation 210.8 feet. The load tests were designed by AFT based upon requirements provided in the Special Note for Static and Pseudo-Static Load Testing. The frames holding the Statnamic loading equipment was designed in conjunction with AFT and Massman Construction Company. Monitoring of the tests was performed by Applied Foundation Testing. The axial pseudo-static load tests were performed in general accordance with the Special Note for Static and Pseudo-Static Pile Testing.

The test piles were instrumented using half-bridge resistance strain gages at ten depth intervals along four vertical lines equally spaced around the perimeter of the exterior of the pipe pile. The strain gages were protected during driving by welding a steel angle over the gages and associated wires. The top of pile displacement was measured using capacitive accelerometers. The applied load was measured using a load cell. A data acquisition system capable of recording 5,000 samples per second was used to collect the data. The K-2 axial pseudo-static pile load test was performed on August 23, 2013, and the K-3 axial pseudo-static pile load test was performed on August 29, 2013. Please refer to Applied Foundation Testing’s Statnamic load test reports for K-2 and K-3, dated September 11, 2013, and September 20, 2013, respectively, for additional details regarding pile load test set-up, the pile instrumentation, the procedures used to perform the tests and information regarding the collected data.

2.4 Lateral Pseudo-Static Load Test

A lateral pseudo-static (Statnamic) load test was performed on Test Pile K-3. At the time of the testing, the K-3 pile tip elevation was at approximately Elevation 210.8 feet. The load test was designed by AFT based upon requirements provided in the Special Note for Static and Pseudo-Static Load Testing. Monitoring of the tests was performed by Applied Foundation Testing. The lateral pseudo-static load test was performed in general accordance with the Special Note for Static and Pseudo-Static Pile Testing.

The test pile was instrumented using half-bridge resistance strain gages at ten depth intervals along four vertical lines equally spaced around the perimeter of the exterior of the pipe pile. The strain gages were protected during driving by welding a steel angle over the gages and associated wires. The lateral pile displacement was measured using accelerometers, two mounted near the point of lateral load application and one mounted approximately 2 feet below the point of load application. A string of eight subsurface accelerometers were installed in a grooved inclinometer casing on the exterior of the pipe pile. A load cell was used to measure the applied Statnamic lateral load. A data acquisition system capable of recording 5,000 samples per second was used to collect the data. The K-3 lateral pseudo-static pile load test was performed on September 10, 2013, by applying four separate lateral load increments. Please refer to Applied Foundation Testing’s Lateral Statnamic load test report K-3, dated September 21, 2013, and revised October
8, 2013, for additional details regarding pile load test set-up, the pile instrumentation, the procedures used to perform the tests and information regarding the collected data.

### 2.5 Proposed Bridge Axial Foundation Loading Information

While performing the pile load test program, revised estimated nominal resistances per pile were received from the structural design team. Using the 90% design plans for the Kentucky Lake Bridge, the following table outlines the location of the proposed bridge end bents and piers, the proposed foundation type and size, the number of piles supporting the substructure and the estimated required nominal resistance in each foundation member. The required nominal resistance is based upon a memorandum provided by the structural designers and the 90% design plans. Based on information from the structural design team on October 28, 2013, the static Strength I and Strength IV load cases control the estimated axial loads. The required nominal loads per foundation member are based upon foundation resistance factors recommended in Section 5.5.2 of this report.

<table>
<thead>
<tr>
<th>Substructure</th>
<th>Station</th>
<th>Foundation Type / Quantity</th>
<th>Diameter (inches)</th>
<th>Estimated Required Nominal Resistance per Foundation Member (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>End Bent 1</td>
<td>988+91.50</td>
<td>Steel Pipe Pile / 28</td>
<td>30</td>
<td>1810</td>
</tr>
<tr>
<td>Pier 1</td>
<td>992+27.50</td>
<td>Steel Pipe Pile / 3</td>
<td>72</td>
<td>8864</td>
</tr>
<tr>
<td>Pier 2</td>
<td>995+92.50</td>
<td>Steel Pipe Pile / 3</td>
<td>72</td>
<td>9123</td>
</tr>
<tr>
<td>Pier 3</td>
<td>999+57.50</td>
<td>Steel Pipe Pile / 3</td>
<td>72</td>
<td>9219</td>
</tr>
<tr>
<td>Pier 4</td>
<td>1003+00.00</td>
<td>Steel Pipe Pile / 14</td>
<td>72</td>
<td>5500</td>
</tr>
<tr>
<td>Pier 5</td>
<td>1008+50.00</td>
<td>Steel Pipe Pile / 14</td>
<td>72</td>
<td>5500</td>
</tr>
<tr>
<td>Pier 6</td>
<td>1011+48.50</td>
<td>Steel Pipe Pile / 5</td>
<td>72</td>
<td>6940</td>
</tr>
<tr>
<td>Pier 7</td>
<td>1015+02.50</td>
<td>Steel Pipe Pile / 3</td>
<td>72</td>
<td>9121</td>
</tr>
<tr>
<td>Pier 8</td>
<td>1018+56.50</td>
<td>Steel Pipe Pile / 3</td>
<td>72</td>
<td>9003</td>
</tr>
<tr>
<td>Pier 9</td>
<td>1022+10.50</td>
<td>Steel Pipe Pile / 3</td>
<td>72</td>
<td>8661</td>
</tr>
<tr>
<td>End Bent 2</td>
<td>1025+02.50</td>
<td>Steel Pipe Pile / 28</td>
<td>30</td>
<td>1600</td>
</tr>
</tbody>
</table>

The proposed Kentucky Lake Bridge has a length of 3611 feet and will support four lanes of traffic. The pavement width will generally be 64 feet – 4 inches with a 10-foot wide sidewalk.

The approach spans are currently planned to consist of welded steel plate girder construction. The proposed Main Span will consist of a basket-handle tied arch having a length of 550 feet from center-to-center of the bearings (Station 1003+00 to Station 1008+50).
3.0 SUBSURFACE CONDITIONS

Several test borings and cone penetration tests (CPT) were performed in 2011 and 2012 near the shallow water test area and the deep water test area. The soil test borings were performed by F&H, KYTC, and HCN/Terracon and the cone penetration test (CPT) soundings were performed by KYTC and HCN/Terracon. The following sections provide a brief description of the soil conditions at the shallow water and deep water test areas. Additional details regarding soil conditions and laboratory test results in these areas are provided in the October 2012 geotechnical report prepared for the Lagoon Bridge and Causeway construction and the final geotechnical report prepared for the Kentucky Lake Bridge.

3.1 Shallow Water Test Area

The shallow water test area was once located in the flood plain of the former Tennessee River, prior to damming of the Tennessee River that created the current Kentucky Lake. Test borings in the area of the shallow water test area indicate that the soils in the area beginning at the mudline (around Elevation 346 feet at the test piles) generally consist of approximately 30 feet of alluvial clay and silt soils to approximately Elevation 315 feet. Generally the alluvial clay and silt soils were classified as ML, CL-ML or ML soils. The alluvial clay and silt is underlain by about 35 feet of fine silty sand to fine sand to approximately Elevation 280 feet, which were classified as SM or SP-SM and some SP soils. Beginning around Elevation 280 feet or 55 feet below the mudline, sand and gravel to silty sand and gravel soils (GM, GW-GM, GP-GM, GC, and SM soils, with occasional CL soils) were encountered to an elevation of around Elevation 235 feet. Beginning at approximately Elevation 235 feet or around 110 feet below the mudline elevation, sand and gravel with chert layers or silty sand and gravel with chert layers was encountered to the termination depth of the test borings performed in the area of the test borings in and around the shallow water test area. The soils below Elevation 235 feet were classified as GM soils with occasional seams of SM, SC-SM or ML. The soils encountered below Elevation 235 feet have been interpreted as residual soils formed from the underlying Ft. Payne formation.

3.2 Deep Water Test Area

The deep water test area is the channel of the former Tennessee River, prior to damming of the Tennessee River that created the current Kentucky Lake. Test borings in the area of the deep water test area indicate that the soils in the area beginning at the mudline (around Elevation 285 feet at Test Pile K-6) generally consist of approximately 10 feet of alluvial silty sand soils to approximately Elevation 275 feet. Generally the alluvial silty sand soils were classified as SM soils. The alluvial silty sand is underlain by about 20 feet of silty sand and gravel to sand and gravel to approximately Elevation 255 feet, which were classified as GM, GW, SP or SW soils. Beginning around Elevation 255 feet or 30 feet below the mudline, silty sand and gravel to fine sand soils with some clay lenses (GM, SW, SP, SW, and SP-SM soils) were encountered to an elevation of around Elevation 235 feet. Beginning at approximately Elevation 235 feet or
around 50 feet below the mudline elevation, sand and gravel with chert layers or silty sand and gravel with chert layers was encountered to the termination depth of the test borings performed in the area of the test borings in and around the shallow water test area. The soils below Elevation 235 feet were classified as GM, GP-GM, GP, SP or SW soils with occasional seams of SM, CL-ML or ML. The soils encountered below Elevation 235 feet have been interpreted as residual soils formed from the underlying Ft. Payne formation.

3.3 Groundwater

The shallow water and deep water test areas were both located below the water surface of Kentucky Lake. It is anticipated that the static groundwater readings would closely mimic the static pool elevations of Kentucky Lake. The Summer pool elevation of Kentucky Lake is reported as Elevation 359 feet. The winter pool elevation of Kentucky Lake is reported as Elevation 354 feet.

4.0 LOAD TEST RESULTS

The following sections briefly summarize the results of the load test program. Detailed results can be found in the reports prepared by Applied Foundation Testing (AFT) that were submitted to the Department upon completion of the load test program.

4.1 Dynamic Pile Load Testing

As discussed in Section 2.1, the piles were driven to an initial drive tip elevation and then driven to an extended drive tip elevation. In addition, two piles, K-3 and K-5 had a second extended drive performed, where the pile tips were extended to an even deeper tip elevation. The following tables provide a brief summary of the dynamic pile testing results including the CAPWAP results and the approximate pile tip for the end of drive (EOD) and beginning of restrike (BOR) conditions for the initial drive, the extended drive and the second extended drive, when applicable. Additional tables summarizing the dynamic load testing of the test piles are presented as Exhibits A-16 to A-21. These tables indicate the Case resistance and the CAPWAP resistance for each test pile at each of the associated end of drive (EOD) and restrike condition for all of the initial drive, extended drive and second extended drive conditions. The tables in Appendix A also include the elapsed time between the EOD and BOR conditions.

<table>
<thead>
<tr>
<th>Test Pile</th>
<th>Diameter (inches)</th>
<th>Tip Elevation (feet)</th>
<th>Date</th>
<th>Drive Condition</th>
<th>Reported CAPWAP Total Resistance (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>K-1</td>
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</table>

Responsive ■ Resourceful ■ Reliable
### Initial Drive of Test Piles

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<th>Test Pile</th>
<th>Diameter (inches)</th>
<th>Tip Elevation (feet)</th>
<th>Date</th>
<th>Drive Condition</th>
<th>Reported CAPWAP Total Resistance (kips)</th>
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</tr>
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<td>≈72-hour Restrike</td>
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</tr>
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### Extended Drive of Test Piles

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<th>Test Pile</th>
<th>Diameter (inches)</th>
<th>Tip Elevation (feet)</th>
<th>Date</th>
<th>Drive Condition</th>
<th>Reported CAPWAP Total Resistance (kips)</th>
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</thead>
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<td>K-1</td>
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<td>K-3</td>
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</table>

### Second Extended Drive of Test Piles

<table>
<thead>
<tr>
<th>Test Pile</th>
<th>Diameter (inches)</th>
<th>Tip Elevation (feet)</th>
<th>Date</th>
<th>Drive Condition</th>
<th>Reported CAPWAP Total Resistance (kips)</th>
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</thead>
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<tr>
<td>K-3</td>
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<td>≈358-hour Restrike</td>
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</table>
During the extended and second extended drive of Test Pile K-5, which was a 72-inch diameter steel pipe pile with a 1.5-inch-thick wall, some evidence of potential damage near the tip of the pile was indicated by the dynamic pile testing. Based on the return wave being reflected above the toe of the pile (5 to 6 feet above the toe of the pile), some damage to the pile may have occurred during installation. During the second extended drive of Test Pile K-5, the wave reflection remained at an approximate constant distance from the pile toe, indicating that the damage likely did not propagate up the length of the pile and the damage was likely isolated in a small area of the pile. Based on the dynamic load test data and the fact that the resistance of the pile increased during the second extended drive, the data suggests that the tip of the pile did not experience crushing, but driving stresses may have been exceeded in a small portion of the pile. Despite the maximum driving stress in the pile indicated by the CAPWAP analyses not exceeding the yield stress of the steel, it is possible that the yield stress of the steel was exceeded in an isolated area of the pile wall near the toe of the pile, likely due to concentrated stresses from an encountered chert layer or cobble.

4.2 Static Load Test – Test Pile K-1

A static load test was performed on Test Pile K-1. At the time the static load test was performed, the pile tip was at approximate Elevation 211 feet. The pile was statically loaded to a maximum test load of 6,025 kips. The maximum pile head vertical displacement during the test was 1.452 inches and the permanent displacement upon removing the test load was 0.022 inches. The telltales located approximately 5 feet above the pile tip, indicated a maximum pile tip movement of approximately 0.15 inches under the full test load and a permanent displacement of approximately 0.06 inches upon complete removal of the test load. The deflection measured at the pile head during the pile load test never exceeded the calculated elastic shortening of the test pile. Thus, the FHWA failure criteria for a 48-inch diameter steel pipe pile was not achieved at the maximum applied test load of 6,025 kips.

The measured skin resistance of the test pile using the vibrating wire strain gages ranged from 0.9 to 3.4 ksf in the upper 110 feet of the test pile and ranged from 4.8 to 11.1 ksf in the lower 27 feet of the test pile for the portion bearing in the sand and gravel with chert layers. The estimated mobilized end bearing on the full end bearing area of the test pile is approximately 96 ksf. Based upon the pile head deflection being less than the calculated elastic shortening of the test pile, it is anticipated that the full skin resistance and end bearing were not mobilized during the static load test.

4.3 Axial Pseudo-Static Tests – Test Piles K-2 and K-3

Axial pseudo-static load tests were performed on the 48-inch-diameter Test Pile K-2 and the 72-inch-diameter Test Pile K-3. Please refer to Exhibits A-22 and A-23 which show the load versus vertical displacement curves for the two axial pseudo-static load tests. Exhibits A-22 and A-23 are from the AFT final axial Static load test reports. Please refer to the AFT final Static load test reports.
load test reports for further collected data from the axial Statnamic load tests, which also include estimated t-z and Q-z plots developed using data from the Statnamic load tests.

The estimated static resistance of Test Pile K-2 having a tip elevation of approximately Elevation 209.9 feet was 6,592 kips. The maximum pile vertical displacement was reported as 4.13 inches and the permanent displacement was reported as 2.2 inches. Using the strain gage data mounted to the exterior of Test Pile K-2, the measured maximum skin resistance along the exterior of the test pile ranged from 1.55 to 2.61 ksf in the upper 110 feet portion of the test pile below the mudline. In the lower 26-foot portion of the test pile, the maximum skin resistance was measured between 4.35 to 7.56 ksf. The measured end bearing across the entire cross-sectional area of the test pile was approximately 182.3 ksf.

The estimated static resistance of Test Pile K-3 having a tip elevation of approximately Elevation 210.8 feet was 8,511 kips. The maximum pile vertical displacement was reported as 2.646 inches and the permanent displacement was reported as 1.464 inches. Using the strain gage data mounted to the exterior of Test Pile K-3, the measured mobilized maximum skin resistance along the exterior of the test pile ranged from 1.13 to 1.76 ksf in the upper 110 feet portion of the test pile below the mudline. In the lower 26-foot portion of the test pile, the maximum skin resistance was measured between 4.21 to 5.92 ksf. The measured end bearing across the entire cross-sectional area of the test pile was approximately 105.9 ksf. Based on the lower values of both skin resistance and end bearing and the low total and permanent measured vertical pile displacements measured during the test on Test Pile K-3, and that the vertical pile displacement did not exceed the FHWA failure criteria for Test Pile K-3, it appears that Test Pile K-3 was not loaded to failure during the axial pseudo-static test.

4.4 Lateral Pseudo-Static Tests – Test Pile K-3

A lateral pseudo-static load test, consisting of four lateral load increments, was performed on the 72-inch diameter Test Pile K-3. P-y data was developed by back-fitting the results using the L-Pile software. AFT performed the data back-fitting to develop the p-y data. The p-y data derived from the Statnamic data and the data back-fitting matched very well with the predicted lateral soil response developed by the geotechnical design team.

The Statnamic data also indicated that negligible lateral displacement occurred below a pile embedment of 46 feet below the mudline. The pile behaved in a linear-elastic manner within applied lateral load range. Exhibit A-24 in Appendix A shows the displacement versus elevation for four applied Statnamic lateral loads, which was taken from the AFT final report of the lateral Statnamic report.
5.0 DATA ANALYSIS

5.1 Dynamic Pile Load Testing

Based on review of the dynamic pile load testing, the proposed large-diameter pipe piles driven at the site will gain resistance with time. This is evident by the increase pile resistances reported by the dynamic pile testing and the CAPWAP analyses performed at the end of drive conditions and the subsequent restrikes performed on the test piles. In general, the pile resistance gain occurs more rapidly in the deep water test area as compared to the shallow water test area. In the deep water test area (Test Pile K-6) the soil strata consists of granular soils that have relatively high permeability that allows excess pore pressures developed around the pile and within the interior of the pile to dissipate more rapidly, as compared to the finer-grained, alluvial soils encountered in the upper 65 to 70 feet of the soil strata in the shallow water areas. However, in the shallow water area at Test Pile K-3 when the pile was driven to a depth where the interior constrictor plate was located within the more permeable granular soils located below Elevation 280 feet, the pile resistance due to dissipation of pore pressures below the constrictor plate within the pile and the granular soils along the exterior of the pile increase more rapidly, than when the interior constrictor plate was located at elevations above Elevation 280 feet and was bearing in the fine sand to fine silty sand alluvial soils. Exhibits A-25 to A-30 in Appendix A show plots of dynamic test resistance versus time.

Exhibits A-25 to A-30 also include data points from the static pile load test and the axial pseudo static to show how the dynamic load test data compares. Typically the dynamic load test data underestimate the static resistance indicated by the static load test performed on Test Pile K-1 bearing at a similar pile tip elevation and the estimated static resistance estimated from the axial Statnamic tests performed on Test Piles K-2 and K-3 bearing at a similar pile tip elevation.

Previous work by Sam Paikowsky on the Sakonnet River Bridge in Rhode Island has indicated that CAPWAP and other one-dimensional wave matching models can underestimate the static resistance of large-diameter pipe piles where a steel interior constrictor plate is used to increase the end bearing component of the pile. As a result, Sam Paikowsky analyzed the collected dynamic pile testing data from the Kentucky Lake Bridge Advance Contract in an effort to produce dynamic load testing results that better match the estimated static pile resistance from both the static load test and the axial Statnamic load tests that were performed on selected test piles during the pile load test program. An energy approach was applied to the dynamic load testing data in an effort to better match the static and pseudo-static load testing results. The results of the energy approach are shown in the table below, Exhibit A-31 in Appendix A, which also includes the calculated CAPWAP resistance of the test piles, as well. The following table provides a summary of the energy method results, CAPWAP results and the associated static load test and pseudo-static results.
Pile Load Test Program Summary Report - Draft
Kentucky Lake Bridge – Advance Contract ■ Trigg and Marshall County, Kentucky
November 19, 2013 ■ Terracon Project No. N1115097

<table>
<thead>
<tr>
<th>Test Pile</th>
<th>Tip Elevation (feet)</th>
<th>EOD CAPWAP Resistance (kips)</th>
<th>72-Hour Restrike CAPWAP Resistance (kips)</th>
<th>Energy Approach (kips)</th>
<th>Estimated Static Resistance (kips)</th>
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</thead>
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<td>3800</td>
<td>5200</td>
<td>8105</td>
<td>8511(^2)</td>
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</table>

Notes: 1. Projected static resistance from static pile load test, Sam Paikowsky
2. Estimated static resistance from Stanamic load test, AFT

Base on the results of the energy approach results and comparing them to the CAPWAP and estimated static resistance of Test Piles K-1 to K-3, the energy approach appears to underestimate the static resistance of the 4-foot diameter piles but may be a better estimate of the static resistance of the 72-inch-diameter piles than CAPWAP. Past research by Paikowsky and Stenersen indicates that static resistance estimated using beginning of restrike CAPWAP typically underestimates the actual static resistance by approximately 15%. The inclusion of the steel constrictor plate further increases the underestimation of the static pile resistance by CAPWAP due to theoretical limitations in the CAPWAP model according to Paikowsky and Chernauskas.

Exhibits A-32 to A-34 show plots of dynamic testing, pseudo-static load test, static load test, calculated static resistance using API RP2A and calculated tensile static resistance using API RP2A versus Elevation. The plots demonstrate that the CAPWAP pile resistance at beginning of restrike at a given pile tip elevation will likely underestimate the static pile resistance.

5.2 Static Load Testing

As indicated earlier, Test Pile K-1 did not experience failure prior to reaching the maximum applied test load of 6,025 kips. Sam Paikowsky has developed methods for extrapolating the static resistance of piles subjected to static load testing without experiencing failure. Using these methods, the estimated static resistance of Test Pile K-1 is between 9,000 kips to 11,100 kips, and the recommended value was 9,550 kips. As a result, it appears the static load test performed on Pile K-1 only mobilized between 54% and 67% of the full static resistance of Test Pile K-1. Please refer to Exhibit A-35 in Appendix A that shows the static load test data, the calculated elastic shortening of the test pile, the Davisson failure criterion line and the extrapolated static resistance of Test Pile K-1. Referring to Exhibit A-32, the minimal vertical movement resulted in only partial mobilization of the side resistance and the end bearing of the test pile. As a result, plotting the measured load resistance during the test versus elevation appears to show that the calculated static geotechnical pile resistance is unconservative. However, the maximum test load plots almost on the calculated geotechnical resistance plot at a pile tip of Elevation 210 feet, and the test pile did not experience failure. Therefore, it appears that the geotechnical resistance calculations are likely conservative compared to the actual pile behavior.
5.3 Pseudo-Static Load Testing

From the pseudo-static load testing (Statnamic load testing), t-z and Q-z curves were developed by AFT. These curves are presented in Appendix A as Exhibits A-36 through A-39. Based upon the vertical displacement that occurred during testing, the Statnamic test on Test Pile K-2 appears to have loaded the pile to failure, whereas K-3 was likely not loaded to failure. As a result, the t-z and Q-z curves developed from the K-2 Statnamic load test are likely a more realistic estimate of the actual side shear versus vertical displacement as compared to the t-z curves developed from the K-3 Statnamic test. AFT has taken into account that the K-3 pile was not loaded to failure in the Q-z plot (Exhibit A-39) by extrapolating the vertical load at the pile tip to a vertical displacement of 5% of the pile diameter. The Q-z curve developed for the K-3 pile from the Statnamic is recommended to be used for design. Examining the t-z curve from K-2 (Exhibit A-36), it is also apparent that sufficient vertical deformation did not occur during the static load test performed on Test Pile K-1 to fully mobilize the skin resistance nor the end bearing resistance at the applied test load.

It is possible that the difference between the static load test resistance and the static resistance estimated using the Statnamic tests was poor due to the time differences after end of drive at which the tests were performed. The static load test was performed approximately 29 days after end of driving Pile K-1; whereas the Statnamic test performed on Pile K-2 was performed 11 days after end of pile driving, and the Statnamic test performed on Pile K-3 was performed 8 days after end of pile driving.

5.4 Lateral Pseudo-Static Load Testing

After performing the lateral pseudo-static (Statnamic) load test on Test Pile K-3, consisting of four individual lateral load increments, AFT back-analyzed the load deflection response of the pile using L-Pile. The lateral pile-soil response parameters were varied to determine the p-y data for the soils in contact with the pile. The following table shows the soil strata, the AFT calculated lateral soil response parameters and the lateral soil response parameters originally recommended to the design team. As can be seen, the recommended parameters match very well with the back-analyzed parameters calculated by AFT. It should be noted that the actual soil parameters in the upper cohesive alluvial soils vary by location and some variation between actual soil conditions and the recommended and back-calculated soil parameters should be expected. Exhibit A-40 shows the derived top of pile lateral response compared to the L-Pile model results and indicates the back analyzed L-Pile parameters model the Statnamic test results very well. Exhibit A-41 is a plot of displacement profiles measured from the Statnamic test compared to the L-Pile model results.
5.5 Overall Evaluation

5.5.1 Setup Factors

We evaluated the soil strength gain versus time as compared to the end of drive (EOD) dynamic test data. The results indicate that the total pile resistance at EOD was about 0.7 (setup factor of 1.4) of the long-term estimated nominal resistance determined by static load test and Statnamic load test results. The beginning of restrike (BOR) for 72 hours (or other time durations) was 0.75 to 0.8 (setup factor of 1.25 to 1.33). The estimated 168-hour BOR of the estimated nominal resistance is projected to be about 0.85 (setup factor of 1.18). This relationship is illustrated in Exhibit A-42 which shows the CAPWAP resistance of the 72-inch-diameter piles normalized against the static resistance estimated from the Statnamic test performed on Pile K-3. This relationship is also consistent with the results shown in Exhibit A-43 which presents the CAPWAP resistances of the 48-inch-diameter piles normalized against the static load test results from Pile K-1. Piles K-4 and K-5, which have smaller pipe wall thicknesses, exhibited lower EOD and BOR resistances than the thicker-walled pipe piles.

The summary of setup factors for piles from EOD to BOR at the initial drive, extended drive and second extended drive conditions are provided in Exhibit A-43 in Appendix A. At the initial drive pile tip elevations for the 72-inch-diameter piles, the minimum setup factor is approximately 1.35.
Generally, as the pile tips were driven to lower (deeper) elevations, the setup factors became smaller. This may be due to additional engagement of the constrictor plate, which may cause the set per blow upon restrike to be insufficient to fully engage the side resistance and/or the end resistance of the piles.

Based on the data presented in Exhibits A-42 to A-44, it is unlikely that the signal matching analyses (CAPWAP) will be capable of accurately predicting the true long-term static resistance of the 72-inch-diameter pipe piles. Previous studies have indicated that the static resistance calculated using dynamic testing methods generally underestimate the actual static resistance from static load tests by approximately 15%. The load test data at this site, indicates that CAPWAP results at longer-term restrikes (168-hours or longer) may underestimate the actual static resistance of the proposed pipe piles by as much as 15%.

Based on the setup calculated using CAPWAP values from EOD to BOR, the amount of setup generally decreases as the constrictor plate engages more end bearing within the denser granular soil strata. As a result, it is possible that decreasing the distance from the pile tip to the constrictor plate could result in lower values of setup than those observed during the load test program. However, it is anticipated that engaging the constrictor plate within the dense granular deposits will result in higher nominal pile resistances that may not be indicated by dynamic testing methods. This effect will occur because the increased end bearing on the constrictor plates will result in reduced vertical pile movement upon restrike, and the inability to mobilize peak end bearing or side resistance of the pile.

One of the team member consultants, GRL, reviewed selected dynamic pile testing records to consider other methods of improving the match quality and the estimation of static pile resistance. This work was in addition to the signal-matching work performed by AFT. Since the piles generally achieved a relatively high end bearing value, particularly when the constrictor plate bears in a dense granular material, and the constrictor plate within the piles results in a plugged or nearly-plugged condition, GRL applied radial or radiation damping models to the signal matching analysis. The radiation model assumes some energy is radiated away from the pile tip instead of being completely confined to static and dynamic responses of the soil shear along the pile and at the pile toe. For the dynamic records where radiation damping was applied, the model generally resulted in a significantly better signal match quality, indicating the radiation damping allows CAPWAP to better model the signals recorded by the dynamic pile testing equipment. The pile resistances calculated with CAPWAP using the radiation damping model also generally produced higher end bearing resistance values than the CAPWAP models without the radiation damping.

It appears that the radiation damping model is better suited for estimating the end bearing component of the piles when less pile set is experienced per hammer blow. This is the case when the constrictor plates are engaged on the dense granular soils. At Pile K-6 at the 72 hour restrike, the use of the radiation damping model resulted in approximately 5,100 kips of end bearing resistance; whereas the use of the second or double toe model resulted in approximately 4,500 kips of end bearing which is the combined resistance at the pile toe and the second toe at the
constrictor plate location. Currently, the plan for the production piles is to locate the constrictor plates such that they bear on or within the dense granular soils so as to engage additional end bearing resistance. The following table presents the results of the CAPWAP static resistance estimates with and without radiation damping, where the Smith damping factor used in the radiation model was limited to 0.4 s/ft. Please note that the GRL analyses used a single-toe pile model, and the AFT analyses used a double-toe model. Based upon the energy method results provided by Sam Paikowsky and the static and Statnamic load test results, it appears that the use of the radiation damping model results in conservative estimates of the long-term static resistance of the tested piles.

<table>
<thead>
<tr>
<th>Pile</th>
<th>Condition</th>
<th>CAPWAP Result (kips) / Match Quality</th>
<th>CAPWAP Low Radiation (kips) / Match Quality</th>
</tr>
</thead>
<tbody>
<tr>
<td>K-1</td>
<td>BOR Extended Drive</td>
<td>5000 / 5.51</td>
<td>6350 / 1.24</td>
</tr>
<tr>
<td>K-3</td>
<td>BOR Extended Drive</td>
<td>8202 / 5.76</td>
<td>7500 / 1.85</td>
</tr>
<tr>
<td>K-5</td>
<td>2nd Extended Drive Restrike</td>
<td>10000 / 4.54</td>
<td>8640 / 1.62</td>
</tr>
<tr>
<td>K-6</td>
<td>72-Hour Restrike</td>
<td>7927 / 5.59</td>
<td>8600 / 2.15</td>
</tr>
</tbody>
</table>

Note: 1. Double-toe model with no low radiation damping

### 5.5.2 LRFD Resistance Factors

The static load testing and the Statnamic load testing confirmed the estimates of available long-term geotechnical axial resistance. Based on the static and Statnamic load testing that was performed in the shallow water test areas, we believe there is justification to increase the LRFD resistance factor ($\phi$) to 0.7 (compared to 0.65) for Piers 1 to 3 and Piers 7 to 9, only. Based on successful performance of the static load test and the Statnamic load tests per site condition, the AASHTO LRFD Bridge Design Specifications allow for a resistance factor for driven piles up to 0.80. However, based on our assessment of the site, we do not recommend increasing the driven pile resistance factor to the full value of 0.80 for the following reasons:

1. The soil conditions are similar between the west and east approach areas, but are not exactly the same.
2. The static load test and Statnamic load tests were performed west of the proposed End Bent 1, which is not in the approach span area.
3. Some variation was observed in the test borings at the individual piers from north to south.

Based on this, we believe it is appropriate to increase the resistance factor from 0.65 to 0.7 at Piers 1 to 3 and Piers 7 to 9. Increasing the resistance from 0.65 to 0.70 would allow the previously recommended pile tip elevations to be used at Piers 1 to 3 and Piers 7 to 9. This assumes the production pile driving criteria would still be established using dynamic pile testing during construction.

At Piers 4, 5 and 6 in the deep water area, where no static or Statnamic load testing was performed, there is no justification for increasing the resistance factor above 0.65, which is
based upon pile driving criteria established by dynamic testing. The following table outlines the anticipated required nominal resistances based upon the recommended resistance factors.

<table>
<thead>
<tr>
<th>Substructure</th>
<th>Station</th>
<th>Foundation Type / Quantity</th>
<th>Diameter (inches)</th>
<th>Required Nominal Resistance(^1) per Foundation Member (kips)</th>
<th>Recommended Driven Pile Resistance Factor ((\phi))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pier 1</td>
<td>992+27.50</td>
<td>Steel Pipe Pile / 3</td>
<td>72</td>
<td>8864</td>
<td>0.7</td>
</tr>
<tr>
<td>Pier 2</td>
<td>995+92.50</td>
<td>Steel Pipe Pile / 3</td>
<td>72</td>
<td>9123</td>
<td>0.7</td>
</tr>
<tr>
<td>Pier 3</td>
<td>999+57.50</td>
<td>Steel Pipe Pile / 3</td>
<td>72</td>
<td>9219</td>
<td>0.7</td>
</tr>
<tr>
<td>Pier 4</td>
<td>1002+97.75</td>
<td>Steel Pipe Pile / 14</td>
<td>72</td>
<td>5500</td>
<td>0.65</td>
</tr>
<tr>
<td>Pier 5</td>
<td>1008+52.25</td>
<td>Steel Pipe Pile / 14</td>
<td>72</td>
<td>5500</td>
<td>0.65</td>
</tr>
<tr>
<td>Pier 6</td>
<td>1011+48.50</td>
<td>Steel Pipe Pile / 14</td>
<td>72</td>
<td>6940</td>
<td>0.65</td>
</tr>
<tr>
<td>Pier 7</td>
<td>1015+02.50</td>
<td>Steel Pipe Pile / 3</td>
<td>72</td>
<td>9121</td>
<td>0.7</td>
</tr>
<tr>
<td>Pier 8</td>
<td>1018+56.50</td>
<td>Steel Pipe Pile / 3</td>
<td>72</td>
<td>9003</td>
<td>0.7</td>
</tr>
<tr>
<td>Pier 9</td>
<td>1022+10.50</td>
<td>Steel Pipe Pile / 3</td>
<td>72</td>
<td>8661</td>
<td>0.7</td>
</tr>
</tbody>
</table>

1. Provided by Baker/Palmer Design Team

6.0 CONCLUSIONS AND RECOMMENDATIONS

An extensive pile load test program was performed to study the constructability and design parameters for large-diameter steel pipe piles for the replacement bridge over Kentucky Lake foundation support. The pile load test program included static axial pile load test, axial pseudo-static pile load tests, a series of lateral pseudo-static pile load tests and dynamic pile testing in general accordance with the special notes for the referenced project.

The pile load test program confirmed that the proposed 72-inch-diameter steel pipe piles and 4-foot-diameter pipe piles with wall thicknesses ranging from 1.0 to 2.0 inches can be installed at the site using a Menck MHU800S hydraulic hammer. Some of the data from the dynamic pile testing indicated that Test Pile K-5 may have been damaged during driving. Test Pile K-5 had a pipe wall thickness of 1.5 inches, whereas Test Piles K-3 and K-6 had 2-inch thick pipe walls. As a result, dynamic testing data indicated that 2.0-inch-thick wall pile thickness for the specified 72-inch-diameter steel pipe piles will likely be required to avoid potential pile damage during installation. The dynamic testing performed on Test Piles K-3 and K-6 indicated that the specified 2.0-inch-thick pipe piles can likely be installed to at least Elevation 168 feet at the site without exceeding specified driving stresses.

The pile load test program indicated that the proposed steel constrictor plates should be installed within the pipe piles to increase the bearing resistance of the pipe piles. Based on the dynamic pile load test data, the interior constrictor plates must be driven into the medium dense to very dense
sand and gravel or silty sand and gravel soil strata (constrictor plate approximately Elevation 267 feet or lower) that is located below the fine-grained alluvial soils and the fine sand to fine silty sand alluvial soils in order to develop plugged or nearly plugged conditions within the 72-inch-diameter pipe piles. The dynamic pile testing also indicated that fully engaging or almost fully engaging the resistance from installation of the interior constrictor plates will be necessary to mobilize enough pile resistance to achieve the estimated pile resistance required for the piles at each bridge pier substructure. As a result, the distance between the bottom of the interior constrictor plates and the tip of the 72-inch-diameter pipe piles will need to be reduced to achieve higher pile resistances at current planned pile tip elevations (estimated to be between Elevation 200 to 220 feet). Appendix B includes a table of estimated pile tip elevations and recommended constrictor plate locations.

Dynamic testing indicated that the side resistance and end bearing resistance of the pipe piles at the site will increase with time, requiring up to 25 days or more to show a reasonable correlation with static / pseudo-static test results. Dynamic testing also indicated that design pile nominal resistances can be achieved and confirmed in the field at elevations below the design tip elevations or at extended restrike time durations. Using super-position methods that use the end bearing components of Test Pile K-6 during the initial drive and K-3 during the second extended drive, along with the side resistance component of the 72-inch-diameter piles near the planned pile tip elevations, it is our opinion that dynamic testing methods are capable of confirming the required nominal pile resistances used for design. However, the energy of the pile hammer will need to be adequate to confirm the design required nominal pile resistances. As a result, we recommend using a pile hammer during installation of the production 72-inch-diameter steel pipe piles with interior steel constrictor plates that has a similar energy rating as the Menck MHU 800S hammer that was used during the pile load test program.

Axial static and pseudo-static pile load testing indicated that the skin resistances used in the pile design phase were valid and likely somewhat conservative. However, the axial pile load testing indicated that the 48-inch-diameter piles were more likely to achieve a plugged condition with the steel constrictor plates located 98 feet above the pile tip at or near the original design pile tip elevation. The 72-inch-diameter piles did not achieve a fully plugged condition with the steel constrictor plates located 98 feet above the pile tip at or near the design pile tip elevation.

We recommend that the production piles be driven at least to 75% of required nominal resistance at EOD conditions. At 72-hour and 168-hour restrikes, we recommend that the piles have at least 85% and 90% of the required nominal resistance, respectively. These recommendations assume that the recommended highest allowable pile tip elevation indicated in Appendix B is achieved. Exhibit A-45 shows the estimated pile axial resistances based upon superposition methods using the estimated required nominal resistance values calculated with the recommended resistance factors derived from the load test data. The superposition was performed using the end bearing CAPWAP values from either test piles K-3 or K-6, depending on whether the area of interest is in the shallow water or deep water areas, and CAPWAP skin resistance values from various test piles bearing at or near the recommended estimated pile tip elevations. The resistance values in Exhibit A-45, show the estimated 72-hour restrike resistances are generally 85% and higher. The
available 550-hour restrike skin resistance value indicates the piles are capable of providing the required long-term axial resistance.

Based on results of the static and Statnamic vertical load tests when compared to the CAPWAP results, the estimates of static load resistance from the CAPWAP analyses generally underestimate the long-term static resistance of the pile. The use of the radiation damping model in conjunction with the single-toe model appears to better estimate the pile’s long-term static resistance, when compared to the static and Statnamic test results. As a result, we recommend that a single-toe signal matching model capable of considering radiation damping be used to analyze the 72-inch-diameter, steel pipe piles containing the constrictor plates during construction. The maximum Smith damping factor for the radiation damping should be limited to 0.4 s/ft.

Considering the estimated mobilized resistance of the piles versus elevation and comparing these values to the t-z curves developed from the Statnamic axial load tests, the pile resistance was only partially mobilized during the static pile load test at Test Pile K-1 and the axial Statnamic load test at Test Pile K-3. Exhibits A-32 and A-33 in Appendix A show plots of the 48- and 72-inch-diameter estimated geotechnical nominal pile resistance (estimated using the API RP2A methodology), the estimated pile resistance from the static and axial Statnamic load tests and predicted axial pile resistance. Comparing the results of the estimated calculated geotechnical pile resistance, the test results and the predicted actual pile resistance from the static load test data with the developed t-z curves, it appears that the calculated geotechnical nominal pile resistances are likely conservative.

During driving of Test Pile K-6, the vibration monitoring instrumentation located on the existing bridge pier indicated increased levels of vertical accelerations as compared to the background vibrations that were recorded prior to and after pile driving in the deep water area. The vibration monitoring program indicates that the pile driving activities will not likely generate damaging vibration levels at the existing bridge structural elements. However, vibration monitoring is recommended during the proposed bridge construction to provide real-time indications of vibration levels should conditions vary from those encountered during this test program.
APPENDIX A

PILE LOAD TEST PROGRAM
PILE K-1 - INITIAL DRIVE
1.5-INCH WALL, 48-INCH DIAMETER
PILE LENGTH = 185 FEET

ELEV. (FT.)

370
ASSUMED SOIL PARAMETERS
IN STATIC PILE ANALYSES

346
MUDLINE

330
CLAY
Cu = 1,000 psf
γ = 67.6pcf

308
TOP OF SOIL IN PILE ELEV. 337

290
FINE SAND TO SILTY SAND
φ = 32°
γ = 62.2 pcf
Fmax = 2.2 ksf

280
Nq = 50
qmax = 100 ksf

250
SILTY SAND AND GRAVEL
TO SAND AND GRAVEL
φ = 34°
γ = 62.6 pcf
Fmax = 2.2 ksf

235
Nq = 40
qmax = 200 ksf

210
SAND AND GRAVEL OR SILTY
SAND AND GRAVEL WITH
CHERT LAYERS
φ = 36°
γ = 67.6 pcf
Fmax = 2.4 ksf
Nq = 50
qmax = 250 ksf

210 EOD 8-6-2013
CUTTING SHOE
FLUSH ON OUTSIDE
OF PILE

170

EOD = END OF DRIVE
TOP OF SOIL ELEVATION
WITHIN PILE AND MUDLINE
PROVIDED BY OTHERS

KENTUCKY LAKE BRIDGE-ADVANCE CONTRACT
US 68 / KY 80 BRIDGE OVER KENTUCKY LAKE
MARSHALL AND TRIGG COUNTIES
KENTUCKY
PILE K-1 - EXTENDED DRIVE
1.5-INCH WALL, 48-INCH DIAMETER
PILE LENGTH = 183.5 FEET

ELEV. (FT.)

370
ASSUMED SOIL PARAMETERS
IN STATIC PILE ANALYSES

355.8
346

TOP OF SOIL IN PILE ELEV. 342

330
CLAY
C_u=1,000 psf
γ'=67.8pcf

315

BOTTOM STEEL PLATE
ELEV. 291.8 9-19-2013

290
FINE SAND TO SILTY SAND
φ'=32°
γ'=62.2 pcf
F_max=2 ksf
N_q=50
q_max=100 ksf

250
SILTY SAND AND GRAVEL TO SAND AND GRAVEL
φ'=34°
γ'=62.6 pcf
F_max=2.2 ksf
N_q=40
q_max=200 ksf

210
SAND AND GRAVEL OR SILTY SAND AND GRAVEL WITH
CHERT LAYERS
φ'=36°
γ'=67.6 pcf
F_max=2.4 ksf
N_q=50
q_max=250 ksf

193.8 EOD 9-19-2013

170
CUTTING SHOE
FLUSH ON OUTSIDE OF PILE

EOD= END OF DRIVE
TOP OF SOIL ELEVATION
WITHIN PILE AND MUDLINE
PROVIDED BY OTHERS
NOTE: 1.5 FT. REMOVED
FROM SPliced SECTION
EXHIBIT A-5
CID 131305
KENTUCKY LAKE BRIDGE-ADVANCE CONTRACT
US 68 / KY 80 BRIDGE OVER KENTUCKY LAKE
MARSHALL AND TRIGG COUNTIES
KENTUCKY

TEST PILE SCHEMATIC

PILE K-2 - INITIAL DRIVE
1.5-INCH WALL, 48-INCH DIAMETER
PILE LENGTH = 185 FEET

ELEV. (FT.)

370
ASSUMED SOIL PARAMETERS
IN STATIC PILE ANALYSES
370

359

345

MUDLINE

330

TOP OF SOIL IN PILE
ELEV. 329.8-14-2013
330

FINE SAND TO SILTY SAND
ϕ=32°
γ' = 62.6pcf
Fmax=2 ksf
Nq=20
qmax=100 ksf

290

BOTTOM STEEL PLATE
ELEV. 307.89 8-12-2013
290

Silty Sand and Gravel
To Sand and Gravel
ϕ=34°
γ' = 62.6pcf
Fmax=2.2 ksf
Nq=40
qmax=200 ksf

250

235

SAND AND GRAVEL TO SILTY SAND AND GRAVEL
WITH CHERT LAYERS
ϕ=36°
γ' = 67.6pcf
Fmax=2.4 ksf
Nq=50
qmax=250 ksf

210

209.89 EOD 8-12-2013
210

CUTTING SHOE
FLUSH ON OUTSIDE
OF PILE
170

EOD= END OF DRIVE
TOP OF SOIL ELEVATION
WITHIN PILE AND MUDLINE
PROVIDED BY OTHERS
PILE K-2 - EXTENDED DRIVE
1.5-INCH WALL, 48-INCH DIAMETER
PILE LENGTH = 171 FEET

ELEV. (FT.)
370
ASSUMED SOIL PARAMETERS
IN STATIC PILE ANALYSES

365
345

MUDLINE
CLAY
Cu=1,000 psf
γ'w=67.6 pcf

330
TOP OF SOIL IN PILE
ELEV. 329.7 9-19-2013

315
BOTTOM STEEL PLATE
ELEV. 300.89 9-19-2013

290
FINE SAND TO SILTY SAND
φ'=32°
γ'w=62.6 pcf
Fmax=2 ksbf
Nq=20
qmax=100 ksbf

280

250
SILTY SAND AND GRAVEL
TO SAND AND GRAVEL
φ'=34°
γ'w=62.6 pcf
Fmax=2.2 ksbf
Nq=40
qmax=200 ksbf

235

210
SAND AND GRAVEL TO SILTY
SAND AND GRAVEL WITH
CHERT LAYERS
φ'=36°
γ'w=67.6 pcf
Fmax=2.4 ksbf
Nq=50
qmax=250 ksbf

202.89 EOD 9-19-2013

170
CUTTING SHOE
FLUSH ON OUTSIDE
OF PILE

EOD= END OF DRIVE
TOP OF SOIL ELEVATION
WITHIN PILE AND MUDLINE
Provided by others
EXHIBIT A-7
CID 131305
KENTUCKY LAKE BRIDGE-ADVANCE CONTRACT
US 68 / KY 80 BRIDGE OVER KENTUCKY LAKE
MARSHALL AND TRIGG COUNTIES
KENTUCKY

PILE K-3 - INITIAL DRIVE
2-INCH WALL, 72-INCH DIAMETER
PILE LENGTH = 185 FEET

ELEV. (FT.)

360
MUDLINE

320
CLAY
Cu=1,000 psf
Y'v=67.6 pcf

308.8 8-21-2013
BOTTOM STEEL PLATE

FINE SAND TO SILTY SAND
ϕ'=32°
Y'v=62.6 pcf
Fmax=2 ksf
Nq=20
qmax=100 ksf

315

SILTY SAND AND GRAVEL TO SAND AND GRAVEL
ϕ'=34°
Y'v=62.6 pcf
Fmax=2.2 ksf
Nq=40
qmax=200 ksf

235

SAND AND GRAVEL TO SILTY
SAND AND GRAVEL WITH CHERT LAYERS
ϕ'=36°
Y'v=67.6 pcf
Fmax=2.4 ksf
Nq=50
qmax=250 ksf

200

357.4

346

TOP OF SOIL IN PILE ELEV.
331.8 8-21-2013

EOD= END OF DRIVE
TOP OF SOIL ELEVATION
WITHIN PILE AND MUDLINE
PROVIDED BY OTHERS

CUTTING SHOE
FLUSH ON OUTSIDE
OF PILE
PILE K-3 - EXTENDED DRIVE
2-INCH WALL, 72-INCH DIAMETER
PILE LENGTH = 171 FEET

ELEV. (FT.)

360
MUDLINE
CLAY
Cu=1,000 psf
Y'v=67.6 pcf

320
FINE SAND TO SILTY SAND
ϕ=32°
Y'v=62.6 pcf
Fmax=2 ksf
Nq=20
qmax=100 ksf

280
SILTY SAND AND GRAVEL TO SAND AND GRAVEL
ϕ=34°
Y'v=62.6 pcf
Fmax=2.2 ksf
Nq=40
qmax=200 ksf

240
SAND AND GRAVEL TO SILTY SAND AND GRAVEL WITH CHERT LAYERS
ϕ=36°
Y'v=67.6 pcf
Fmax=2.4 ksf
Nq=50
qmax=250 ksf

200

CUTTING SHOE
FLUSH ON OUTSIDE
OF PILE

TOP OF SOIL IN PILE
ELEV. 328.5  9-17-2013

315
BOTTOM STEEL PLATE
ELEV. 300.0  9-13-2013

346
202 EOD  9-13-2013

EOD= END OF DRIVE
TOP OF SOIL ELEVATION
WITHIN PILE AND MUDLINE
PROVIDED BY OTHERS

ASSUMED SOIL PARAMETERS
IN STATIC PILE ANALYSES (API)
PILE K-3 - SECOND EXTENDED DRIVE
2-INCH WALL, 72-INCH DIAMETER
PILE LENGTH = 263.2 FEET

ELEV. (FT.)

360
MUDLINE

320
FINE SAND TO SILTY SAND
\( \phi = 32^\circ \)
\( Y' = 62.6 \text{ pcf} \)
\( F_{\text{max}} = 2 \text{ ksf} \)
\( N_q = 20 \)
\( q_{\text{max}} = 100 \text{ ksf} \)

315
TOP OF SOIL IN PILE
ELEV. 321 9-24-2013

300
SILTY SAND AND GRAVEL TO SAND AND GRAVEL
\( \phi = 34^\circ \)
\( Y' = 62.6 \text{ pcf} \)
\( F_{\text{max}} = 2.2 \text{ ksf} \)
\( N_q = 40 \)
\( q_{\text{max}} = 200 \text{ ksf} \)

280
BOTTOM STEEL PLATE
ELEV. 266.7 9-24-2013

240
SAND AND GRAVEL TO SILTY SAND AND GRAVEL WITH CHERT LAYERS
\( \phi = 36^\circ \)
\( Y' = 67.6 \text{ pcf} \)
\( F_{\text{max}} = 2.4 \text{ ksf} \)
\( N_q = 50 \)
\( q_{\text{max}} = 250 \text{ ksf} \)

200

168.7 EOD 9-24-2013

160
CUTTING SHOE
FLUSH ON OUTSIDE
OF PILE

EOD = END OF DRIVE
TOP OF SOIL ELEVATION
WITHIN PILE AND MUDLINE
PROVIDED BY OTHERS

ASSUMED SOIL PARAMETERS
IN STATIC PILE ANALYSES (API)

CLAY
\( C_u = 1,000 \text{ psf} \)
\( Y' = 67.6 \text{ pcf} \)

Scale: 1"=40'V

EXHIBIT A-9

CID 131305
KENTUCKY LAKE BRIDGE-ADVANCE CONTRACT
US 68 / KY 80 BRIDGE OVER KENTUCKY LAKE
MARSHALL AND TRIGG COUNTIES
KENTUCKY
EXHIBIT A-10

CID 131305
TEST PILE SCHEMATIC

KENTUCKY LAKE BRIDGE-ADVANCE CONTRACT
US 68 / KY 80 BRIDGE OVER KENTUCKY LAKE
MARSHALL AND TRIGG COUNTIES
KENTUCKY

PILE K-4 - INITIAL DRIVE
1-INCH WALL, 48-INCH DIAMETER
PILE LENGTH = 185 FEET

ELEV. (FT.)
370
357
346
330
TOP OF SOIL IN PILE
ELEV. 331 8-5-2013
315
290
BOTTOM STEEL PLATE
ELEV. 290 8-3-2013
280
250
SILTY SAND AND GRAVEL
TO SAND AND GRAVEL
ϕ=34°
γ'=62.6 pcfs
Fmax=2.2 ksf
Nq=40
qmax=200 ksf
235
210
SAND AND GRAVEL OR SILTY
SAND AND GRAVEL WITH
CHERT LAYERS
ϕ=36°
γ'=67.6 pcfs
Fmax=2.4 ksf
Nq=50
qmax=250 ksf
210
192 EOD 8-6-2013
170
CUTTING SHOE
FLUSH ON OUTSIDE
OF PILE

EOD= END OF DRIVE
TOP OF SOIL ELEVATION
WITHIN PILE AND MUFLINE
PROVIDED BY OTHERS

ASSUMED SOIL PARAMETERS
IN STATIC PILE ANALYSES
CLAY
Cu=1,000 psf
γ'=67.6 pcfs
20
qmax=100 ksf

FINE SAND TO SILTY SAND
ϕ=32°
γ'=62.6 pcfs
Fmax=2 ksf

NER: 20
qmax=100 ksf
PILE K-4 - EXTENDED DRIVE
1-INCH WALL, 48-INCH DIAMETER
PILE LENGTH = 185 FEET

ELEV. (FT.)

370
ASSUMED SOIL PARAMETERS
IN STATIC PILE ANALYSES

359  8-13-2013
357  8-9-2013
346

MUCLINE

CLAY
Cu=1,000 psf
γ' =67.6 pcf

330
TOP OF SOIL IN PILE
ELEV. 331  8-5-2013

290
FINE SAND TO SILTY SAND
φ'=32°
γ' =62.6 pcf
Fmax=2 ksf
Nq=20
qmax=100 ksf

250
SILTY SAND AND GRAVEL
TO SAND AND GRAVEL
φ'=34°
γ' =62.6 pcf
Fmax=2.2 ksf
Nq=40
qmax=200 ksf

210
SAND AND GRAVEL OR SILTY
SAND AND GRAVEL WITH
CHERT LAYERS
φ'=36°
γ' =67.6 pcf
Fmax=2.4 ksf
Nq=50
qmax=250 ksf

186.5 EOD 8-9-2013

170
CUTTING SHOE
FLUSH ON OUTSIDE
OF PILE

EOD= END OF DRIVE
TOP OF SOIL ELEVATION
WITHIN PILE AND MUCLINE
PROVIDED BY OTHERS
EXHIBIT A-12
CID 131305
KENTUCKY LAKE BRIDGE-ADVANCE CONTRACT
US 68 / KY 80 BRIDGE OVER KENTUCKY LAKE
MARSHALL AND TRIGG COUNTIES
KENTUCKY

PILE K-5 - INITIAL DRIVE
1.5-INCH WALL, 72-INCH DIAMETER
PILE LENGTH = 185 FEET

ASSUMED SOIL PARAMETERS
IN STATIC PILE ANALYSES (API)

ELEV. (FT.)

360
MUDLINE

320
CLAY
Cu=1,000 psf
γ'=67.6 pcf

280
FINE SAND TO SILTY SAND
ϕ'=32°
γ'=62.6 pcf
Fmax=2 ksf
Nq=20
qmax=100 ksf

240
SILTY SAND AND GRAVEL
TO SAND AND GRAVEL
ϕ'=34°
γ'=62.6 pcf
Fmax=2.2 ksf
Nq=40
qmax=200 ksf

200
SAND AND GRAVEL TO SILTY SAND AND GRAVEL WITH
CHERT LAYERS
ϕ'=36°
γ'=67.6 pcf
Fmax=2.4 ksf
Nq=50
qmax=250 ksf

TOP OF SOIL IN PILE
ELEV. 339.7  8-21-2013

347
358
360

315
320

280
280

240
235

210 EOD  8-17-2013

EOD= END OF DRIVE
TOP OF SOIL ELEVATION
WITHIN PILE AND MUDLINE
PROVIDED BY OTHERS

CUTTING SHOE
FLUSH ON OUTSIDE
OF PILE
PILE K-5 - EXTENDED DRIVES
1.5-INCH WALL, 72-INCH DIAMETER
PILE LENGTH = 185 FEET

ASSUMED SOIL PARAMETERS
IN STATIC PILE ANALYSES (API)

360
MUDLINE

320
CLAY
Cu=1,000 psf
V=67.6 pcf

280
FINE SAND TO SILTY SAND
\( \phi=32^\circ \)
V=62.6 pcf
Fmax=2.2 ksf
Nq=20
qmax=100 ksf

240
SILTY SAND AND GRAVEL
TO SAND AND GRAVEL
\( \phi=34^\circ \)
V=62.8 pcf
Fmax=2.2 ksf
Nq=40
qmax=200 ksf

200
SAND AND GRAVEL TO SILTY
SAND AND GRAVEL WITH
CHERT LAYERS
\( \phi=36^\circ \)
V=67.6 pcf
Fmax=2.4 ksf
Nq=50
qmax=250 ksf

188 EOD 8-27-2013

160
CUTTING SHOE
FLUSH ON OUTSIDE
OF PILE

204.2 EOD 8-23-2013

TOP OF PILE ELEV. 373
END OF EXTENDED DRIVE 8-27-2013

TOP OF SOIL IN PILE AFTER EXTENDED DRIVE
ELEV. 337 8-27-2013

BOTTOM STEEL PLATE
ELEV. 302.2 8-23-2013 EXTENDED DRIVE

BOTTOM STEEL PLATE
ELEV. 286 8-27-2013

EOD = END OF DRIVE
TOP OF SOIL ELEVATION
WITHIN PILE AND MUDLINE
PROVIDED BY OTHERS
PILE K-6 - INITIAL DRIVE
2-INCH WALL, 72-INCH DIAMETER
PILE LENGTH = 210 FEET

ASSUMED SOIL PARAMETERS
IN STATIC PILE ANALYSES (API)

ELEV. (FT.)
360
320
280
240
200
160

MUDLINE
SILTY SAND
φ=32°
γ'=57.6 pcf
Fmax=2 ksf
Nq=20
qmax=100 ksf

TO SILET SAND AND GRAVEL
φ=36°
γ'=62.6 pcf
Fmax=2.2 ksf
Nq=40
qmax=200 ksf

SAND AND GRAVEL TO SILET SAND
φ=33°
γ'=60.6 pcf
Fmax=2.2 ksf
Nq=30
qmax=160 ksf

SAND AND GRAVEL TO SILET SAND AND GRAVEL WITH
CHERT LAYERS
φ=36°
γ'=67.6 pcf
Fmax=2.4 ksf
Nq=50
qmax=250 ksf

TOP OF SOIL IN PILE
ELEV. 286.25 9-11-2013
285.2
275
255
235
200

BOTTOM STEEL PLATE
ELEV. 267 9-11-2013

191 EOD 9-11-2013

CUTTING SHOE
FLUSH ON OUTSIDE
OF PILE

EOD= END OF DRIVE
TOP OF SOIL ELEVATION
WITHIN PILE AND MUDLINE
PROVIDED BY OTHERS
PILE K-6 - EXTENDED DRIVE
2-INCH WALL, 72-INCH DIAMETER
PILE LENGTH = 210 FEET

ELEV. (FT.)

360

ASSUMED SOIL PARAMETERS
IN STATIC PILE ANALYSES (API)

356.1

SILTY SAND

ϕ=32°

γ=57.6 pcf

Fmax=2 ksf

Nq=20

qmax=100 ksf

320

TOP OF SOIL IN PILE

ELEV. 285.35  9-14-2013

285.2

MUDLINE

280

SAND AND GRAVEL

TO SILTY SAND AND GRAVEL

ϕ=36°  γ=62.6 pcf  Fmax=2.2 ksf

Nq=40  qmax=200 ksf

275

240

SAND TO SILTY SAND

ϕ=33°  γ=60.6 pcf  Fmax=2.2 ksf

Nq=30  qmax=160 ksf

255

SAND AND GRAVEL TO SILTY

SAND AND GRAVEL WITH

CHERT LAYERS

ϕ=36°

γ=67.6 pcf

Fmax=2.4 ksf

Nq=50

qmax=250 ksf

235

200

172.35 EOD  9-14-2013

160

CUTTING SHOE

FLUSH ON OUTSIDE

OF PILE

EOD= END OF DRIVE
TOP OF SOIL ELEVATION
WITHIN PILE AND MUDLINE
PROVIDED BY OTHERS
<table>
<thead>
<tr>
<th>Pile Tip Elevation (ft)</th>
<th>Description</th>
<th>Date</th>
<th>Time</th>
<th>Elapsed Time (hrs)</th>
<th>CASE Resistance (kips)</th>
<th>CASE Jc</th>
<th>CAPWAP (kips)</th>
<th>K-1 CASE % of Static</th>
<th>K-1 CAPWAP % of Static</th>
<th>Plate Elevation (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>210</td>
<td>EOID</td>
<td>08/06/13</td>
<td>16:07</td>
<td>0</td>
<td>3304</td>
<td>0.45</td>
<td>3301</td>
<td>0.55</td>
<td>0.55</td>
<td>308</td>
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<td>209.8</td>
<td>48-hr</td>
<td>08/08/13</td>
<td>11:32</td>
<td>43</td>
<td>4514</td>
<td>0.45</td>
<td>4500</td>
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<td>0.75</td>
<td>307.8</td>
</tr>
<tr>
<td>209.6</td>
<td>72-hr</td>
<td>08/09/13</td>
<td>13:28</td>
<td>69</td>
<td>5070</td>
<td>0.45</td>
<td>5000</td>
<td>0.84</td>
<td>0.83</td>
<td>307.6</td>
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<td>209.6</td>
<td>K-1 Static</td>
<td>09/04/13</td>
<td>9:32</td>
<td>689</td>
<td>6025</td>
<td></td>
<td></td>
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<td></td>
<td>307.6</td>
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<td>209.6</td>
<td>PS-ED-BOD</td>
<td>09/19/13</td>
<td>15:11</td>
<td>1055</td>
<td>4627</td>
<td>0.4</td>
<td>5000</td>
<td>0.77</td>
<td>0.83</td>
<td>307.6</td>
</tr>
<tr>
<td>193.8</td>
<td>PS-EOED</td>
<td>09/19/13</td>
<td>15:54</td>
<td>0</td>
<td>5732</td>
<td>0.4</td>
<td>5717</td>
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<td>291.8</td>
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<td>193.8</td>
<td>PS-ED+96</td>
<td>09/23/13</td>
<td>12:44</td>
<td>93</td>
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<td>7366</td>
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<td>291.8</td>
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### Exhibit A-17  Test Pile K-2 Dynamic Pile Testing Data

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<th>Pile Tip Elevation (ft)</th>
<th>Description</th>
<th>Date</th>
<th>Time</th>
<th>Elapsed Time (hrs)</th>
<th>CASE Resistance (kips)</th>
<th>CASE Jc</th>
<th>CAPWAP (kips)</th>
<th>K-2 CASE % of Static</th>
<th>K-2 CAPWAP % of Static</th>
<th>Plate Elevation (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>209.9</td>
<td>EOID</td>
<td>08/12/13</td>
<td>11:42</td>
<td>0.1</td>
<td>3246</td>
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<td>08/14/13</td>
<td>9:01</td>
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<td>4500</td>
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<td>08/15/13</td>
<td>8:55</td>
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<td>0.79</td>
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<td>K-2 Statnamic</td>
<td>08/23/13</td>
<td>20:05</td>
<td>272.4</td>
<td>6953</td>
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<td>0.76</td>
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<td>09/19/13</td>
<td>11:06</td>
<td>911.4</td>
<td>5023</td>
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<td>0.1</td>
<td>5451</td>
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<td>09/23/13</td>
<td>8:49</td>
<td>93</td>
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<td>6402</td>
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<td>300.9</td>
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## Exhibit A-18  Test Pile K-3 Dynamic Pile Testing Data

<table>
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<th>Description</th>
<th>Date</th>
<th>Time</th>
<th>Elapsed Time (hrs)</th>
<th>CASE Resistance (kips)</th>
<th>CASE JC</th>
<th>CAPWAP (kips)</th>
<th>K-3 CASE % of Static</th>
<th>K-3 CAPWAP % of Static</th>
<th>Plate Elevation (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>210.8</td>
<td>EOID</td>
<td>08/21/13</td>
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<td>0</td>
<td>2715</td>
<td>0.45</td>
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<td>0.32</td>
<td>0.45</td>
<td>308.8</td>
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<tr>
<td>210.7</td>
<td>48-hr</td>
<td>08/23/13</td>
<td>15:47</td>
<td>51.2</td>
<td>4643</td>
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<td>4200</td>
<td>0.55</td>
<td>0.49</td>
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<td>210.6</td>
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<td>1.00</td>
<td>1.00</td>
<td>308.6</td>
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<td>09/13/13</td>
<td>11:02</td>
<td>550.5</td>
<td>7624</td>
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<td>8202</td>
<td>0.90</td>
<td>0.96</td>
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<td>Pa1-ED</td>
<td>09/13/13</td>
<td>11:20</td>
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</tr>
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<td>11:36</td>
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Exhibit A-19  Test Pile K-4 Dynamic Pile Testing Data

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<th>Pile Tip Elevation (ft)</th>
<th>Description</th>
<th>Date</th>
<th>Time</th>
<th>Elapsed Time (hrs)</th>
<th>CASE Resistance (kips)</th>
<th>CASE Jc</th>
<th>CAPWAP Resistance (kips)</th>
<th>CAPWAP % of Static</th>
<th>K-4 CASE % of Static</th>
<th>K-4 CAPWAP % of Static</th>
<th>Plate Elevation (feet)</th>
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<td>4193</td>
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<td>08/06/13</td>
<td>14:39</td>
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<td>Pile Tip Elevation (ft)</td>
<td>Description</td>
<td>Date</td>
<td>Time</td>
<td>Elapsed Time (hrs)</td>
<td>CASE Resistance (kips)</td>
<td>CASE Jc</td>
<td>CAPWAP (kips)</td>
<td>CASE % of Static</td>
<td>CAPWAP % of Static</td>
<td>Plate Elevation (feet)</td>
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</tr>
<tr>
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<td>3890</td>
<td>0.45</td>
<td>3930</td>
<td>0.39</td>
<td>0.39</td>
<td>302.20</td>
<td></td>
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<tr>
<td>204.2</td>
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<td>08/27/13</td>
<td>16:49</td>
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<td>7587</td>
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<tr>
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<td>Pile Tip Elevation (ft)</td>
<td>Description</td>
<td>Date</td>
<td>Time</td>
<td>Elapsed Time (hrs)</td>
<td>CASE Resistance (kips)</td>
<td>CASE Jc</td>
<td>CAPWAP (kips)</td>
<td>K-6 AvgRst % of Static</td>
<td>K-6 CAPWAP % of Static</td>
<td>Plate Elevation (feet)</td>
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</tr>
<tr>
<td>------------------------</td>
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<td>------------------------</td>
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<tr>
<td>191</td>
<td>EOID</td>
<td>11-Sep</td>
<td>10:46</td>
<td>0.0</td>
<td>5986</td>
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<td>6090</td>
<td>0.77</td>
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<td>267.00</td>
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<tr>
<td>191</td>
<td>48-hr</td>
<td>13-Sep</td>
<td>14:16</td>
<td>51.5</td>
<td>6989</td>
<td>0.35</td>
<td>7001</td>
<td>0.90</td>
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<tr>
<td>191</td>
<td>72-hr</td>
<td>14-Sep</td>
<td>9:01</td>
<td>70.3</td>
<td>7792</td>
<td>0.35</td>
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<td>1.01</td>
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<td>0.0</td>
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<td>1.01</td>
<td>267.00</td>
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</tr>
<tr>
<td>172.4</td>
<td>EOED</td>
<td>14-Sep</td>
<td>11:08</td>
<td>0.0</td>
<td>7510</td>
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<td>7741</td>
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<td>1.00</td>
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<tr>
<td>172.4</td>
<td>EOED+96 hrs</td>
<td>18-Sep</td>
<td>10:11</td>
<td>95.1</td>
<td>9948</td>
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<td>9904</td>
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<td>1.28</td>
<td>248.40</td>
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</table>
Figure 1: Applied Foundation Testing, Inc.

Load vs. Displacement from Statnamic Test
Test Pile K-3, Kentucky Lake Site

From AFT Report Dated September 20, 2013

Exhibit A-23

Applied Foundation Testing, Inc.
Displacements vs Elev for Kentucky Lake Test Pile

Exhibit A-24 - *Displacement versus Elevation for Loads 1 through 4*

From AFT Report Dated September 21, 2013
Dynamic Test Resistance vs. Time
Test Pile K-1

Axial Resistance (kips)

Elapsed Time after End of Initial Drive (hours)
Dynamic Test Resistance vs. Time
Test Pile K-2

Axial Resistance (kips)

Elapsed Time after End of Initial Drive (hours)
Dynamic Test Resistance vs. Time
Test Pile K-3

Axial Resistance (kips)

Elapsed Time after End of Initial Drive (hours)
Dynamic Test Resistance vs. Time
Test Pile K-4

Axial Resistance (kips)

Elapsed Time after End of Initial Drive (hours)
Dynamic Test Resistance vs. Time
Test Pile K-5

Axial Resistance (kips)

Elapsed Time after End of Initial Drive (hours)
Dynamic Test Resistance vs. Time
Test Pile K-6

Axial Resistance (kips)
Elapsed Time after End of Initial Drive (hours)

- CASE-Initial Drive
- CAPWAP-Initial Drive
- CASE-Extended Drive
- CAPWAP-Extended Drive
## Table 2. Time, dates and elapsed time for driving, restrike and testing of piles

<table>
<thead>
<tr>
<th>Pile</th>
<th>State</th>
<th>Date and Time for SOD</th>
<th>Elapsed Time Since SOD</th>
<th>CAPWAP or Tested Capacity (kips)</th>
<th>Energy Approach for CAPWAP Blow (kips)</th>
<th>Blow Number</th>
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</thead>
<tbody>
<tr>
<td>K-1</td>
<td>EOD</td>
<td>8/6/13, 1:08 PM</td>
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<td>8/8/13, 11:32 AM</td>
<td>46 hrs, 24 min</td>
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<td>5535</td>
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<td>8/9/13, 1:18 PM</td>
<td>72 hrs, 20 min</td>
<td>5000</td>
<td>5990</td>
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<td>Static LT</td>
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<td>1,079 hrs, 3 min</td>
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<td>1,079 hrs, 47 min</td>
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<td>9/23/13, 12:44 PM</td>
<td>1,172 hrs, 36 min</td>
<td>7366</td>
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<td>69 hrs, 1 min</td>
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<td>5870</td>
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</tr>
<tr>
<td></td>
<td>Statnamic</td>
<td>8/23/2013, 12:00* PM</td>
<td>264 hrs, 18 min</td>
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<td>N/A</td>
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<td>910 hrs, 27 min</td>
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<td>6105</td>
<td>1 of 199</td>
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<td>911 hrs, 22 min</td>
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<td>1,004 hrs, 21 min</td>
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<td>8505</td>
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<td>K-3</td>
<td>EOD</td>
<td>8/21/13, 9:16 AM</td>
<td>3 hrs, 18 min</td>
<td>3800</td>
<td>10.905</td>
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<td>54 hrs, 31 min</td>
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<td>11.835</td>
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<td>8/24/13, 2:09 PM</td>
<td>76 hrs, 52 min</td>
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<td></td>
<td>BOR ED</td>
<td>9/13/13, 11:02 AM</td>
<td>550 hrs, 28 min</td>
<td>8202/8006</td>
<td>14105</td>
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<td></td>
<td>EOD ED</td>
<td>9/13/13, 12:03 PM</td>
<td>551 hrs, 29 min</td>
<td>6005</td>
<td>16350</td>
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<td>655 hrs, 58 min</td>
<td>6800</td>
<td>14150</td>
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<tr>
<td></td>
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<td>9/24/13, 3:16 PM</td>
<td>822 hrs</td>
<td>6920</td>
<td>10240</td>
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<td>823 hrs, 47 min</td>
<td>10,530</td>
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<tr>
<td></td>
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<td>847 hrs, 42 min</td>
<td>10,620</td>
<td>13845</td>
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</tr>
<tr>
<td>K-4</td>
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<td>8/1/13, 6:31 PM</td>
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<td>91 hrs, 45 min</td>
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<td>5430</td>
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<tr>
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<td></td>
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<td>8/13/13, 4:22 PM</td>
<td>286 hrs, 1 min</td>
<td>5600/5827</td>
<td>5155</td>
<td>4 of 5</td>
</tr>
</tbody>
</table>

Notes:  
- **SOD** – Start of Drive  
- **EOD** – End of Drive  
- **ED** – Extended Drive  
- **BOR** – Beginning of Restrike  
- **XDR** – X Day Restrike, e.g. 2DR = 2 Day Restrike  
- **RS** – Restrike  
- *Assumed test start time  
- CAPWAP results as reported by AFT. Two sets of results refer to early reporting/final reporting values, respectively
Exhibit A-31 (cont’d)  Time, dates and elapsed time for driving, restrike and testing of piles

<table>
<thead>
<tr>
<th>Pile</th>
<th>State</th>
<th>Date and Time for SOD</th>
<th>Elapsed Time Since SOD</th>
<th>CAPWAP or Tested Capacity (kips)</th>
<th>Energy Approach for CAPWAP Blow (kips)</th>
<th>Blow Number</th>
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<tbody>
<tr>
<td>K-5</td>
<td>EOD</td>
<td>8/17/13, 2:49 PM</td>
<td>2 hrs, 19 min</td>
<td>2530/2845</td>
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<td>47 hrs, 27 min</td>
<td>5790</td>
<td>6620</td>
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<tr>
<td></td>
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<td>8/21/13, 4:38 PM</td>
<td>97 hrs, 48 min</td>
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<tr>
<td></td>
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<td>8/23/13, 8:36 AM</td>
<td>137 hrs, 47 min</td>
<td>6800/7452</td>
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<td>5 of 1161</td>
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<td>8/27/13, 4:50 PM</td>
<td>242 hrs, 43 min</td>
<td>5800/7183</td>
<td>9535</td>
<td>1159 of 1161</td>
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<tr>
<td></td>
<td>BOR 2nd ED RS</td>
<td>9/11/13, 4:18 PM</td>
<td>599 hrs, 8 min</td>
<td>10000</td>
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</tr>
<tr>
<td>K-6</td>
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<td>53 hrs, 48 min</td>
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<td>9465</td>
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<tr>
<td></td>
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<td>13,315</td>
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</tbody>
</table>

Notes: SOD – Start of Drive     EOD – End of Drive     ED – Extended Drive
BOR – Beginning of Restrike     XDR – X Day Restrike, e.g. 2DR = 2 Day Restrike
RS – Restrike                  *Assumed test start time
**Requires extrapolation and evaluation. Provided values are the maximum applied loads.
CAPWAP results as reported by AFT. Two sets of results refer to early reporting/final reporting values, respectively.
Shallow Water - 48-Inch Diameter Pipe Pile
Load Test Resistance and Calculated Geotechnical Static Resistance (API RP 2A Method)

Mudline
Elevation 346 ft.
Shallow Water - 72-Inch Diameter Pipe Pile
Pseudo-Static Test Resistance and Calculated Static Geotechnical Resistance (API RP 2A Method)

Mudline Elevation 346 ft.
Initial Drive Plate Elevation = 308.8 feet
Extended Drive Plate Elevation = 300 feet
Second Extended Drive Plate Elevation = 266.7 feet

- Calculated Tensile (API RP2A)
- Calculated Resistance (API RP2A)
- Pseudo-Static Test (Load Distribution partially mobilized)
- Pile K-3 Initial Drive Dynamic Testing
- Pile K-3 Extended Drive Dynamic Testing
- Pile K-3 Second Extended Drive Dynamic Testing
Deep Water - 72-Inch Diameter Pipe Pile
Calculated Static Geotechnical Resistance (API RP 2A Method)

Mudline Elevation 285.2 ft.

Initial Drive Plate Elevation = 267 feet

Extended Drive Plate Elevation = 248.4 feet

Nominal Geotechnical Resistance (kips)

- Calculated Tensile (API RP2A)
- Calculated Resistance (API RP2A)
- Pile K-6 Initial Drive Dynamic Testing
- Pile K-6 Extended Drive Dynamic Testing

EOD

72-hour Restrike

96-hour Restrike

Exhibit A-34
Exhibit A-35  Load displacement data reported for pile K-1 static load test and its interpretation.

From Sam Paikowsky Correspondence October 9, 2013
Side Shear vs Displacement (TZ Curves)
Test Pile K-2
Kentucky Lake Site

From AFT Report Dated September 11, 2013
Applied Foundation Testing
Figure 3 Applied Foundation Testing, Inc.

Load vs. Toe Displacement
Toe Segment El 214.6ft to 209.6ft
Pile K-2
Kentucky Lake Site

Load (kips)

Displacement (inches)
Figure 2: Applied Foundation Testing

Side Shear vs Displacement (TZ Curves)
Test Pile K-3
Kentucky Lake Site

Side Shear (ksf)
Segment Midpoint Displacement (inches)
Segment 1
Segment 2
Segment 3
Segment 4
Segment 5
Segment 6
Segment 7

From AFT Report Dated September 20, 2013
Exhibit A-38
Applied Foundation Testing
Load vs. Toe Displacement
Toe Segment EI 215.8ft to 210.8ft
Pile K-3
Kentucky Lake Site

From AFT Report Dated September 20, 2013
Applied Foundation Testing, Inc.
Exhibit A-40  -  Derived Static Top of Shaft Response from the Statnamic Test Compared with the LPILE Model Results

From AFT Report Dated September 21, 2013
Exhibit A-41 - *Displacement profiles measured from the Statnamic Test Compared with the LPILE Model Results*

From AFT Report Dated September 21, 2013
Normalized Setup After Initial Drive - 72-inch-diameter Pipe Pile - 8511 kips
Static Nominal Resistance (Statnamic)

Measured Resistance to Static Nominal Resistance (8511 kips)

Hours After End of Drive

K-3 CASE
K-3 CAPWAP
K-5 (72-inch, 1.5-inch wall) CASE @ El. 210 ft.
K-5 (72-inch, 1.5-inch wall) CAPWAP @ El. 210 ft.
K-5 (72-inch, 1.5-inch wall) CASE @ El. 204.2 ft.
K-5 (72-inch, 1.5-inch wall) CAPWAP @ El. 204.2 ft.
Normalized Setup After Initial Drive - 48-inch-diameter Pipe Pile - 6025 kips

Static Nominal Resistance (Statnamic/Static)

- K-1 CASE
- K-1 CAPWAP
- K-2 CASE
- K-2 CAPWAP
- K-4 CASE
- K-4 CAPWAP

Measured Resistance to Static Nominal Resistance (6025 kips)

Hours After End of Drive
### Exhibit A-44  Observed Test Pile Set Up Characteristics

<table>
<thead>
<tr>
<th>Pile</th>
<th>Pile Diameter (inches)</th>
<th>Initial Drive Tip Elevation (feet)</th>
<th>EOD Initial Drive Tip Elevation (feet)</th>
<th>Initial Drive Tip Elevation (feet)</th>
<th>EOD Initial Drive Tip Elevation (feet)</th>
<th>Extended Drive Tip Elevation (feet)</th>
<th>EOD Extended Drive Tip Elevation (feet)</th>
<th>2nd Extended Drive Tip Elevation (feet)</th>
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<td>Shallow Water Average</td>
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**Notes:**
* 24-hr restrike
** 360-hr restrike
### Exhibit A-45: Estimated Axial Pile Resistances at Pier Locations Using Superposition Methods

<table>
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<tr>
<th>Pier</th>
<th>72-hour CAPWAP End Brearing Plate Resistance (kips)</th>
<th>72-hour CAPWAP Side Resistance (kips)</th>
<th>550-hour CAPWAP Side Resistance (kips)</th>
<th>Required Nominal Resistance Factor (φ)</th>
<th>72-Hour Superposition CAPWAP Total Resistance (kips)</th>
<th>Superposition CAPWAP % Required (%)</th>
<th>72-Hour Superposition API Total Resistance (kips)</th>
<th>Superposition API % Required (%)</th>
<th>Superposition CAPWAP &amp; API Total Resistance (kips)</th>
<th>Superposition % Required (%)</th>
<th>Superposition CAPWAP &amp; API Total Resistance (kips)</th>
<th>Superposition % Required (%)</th>
<th>Long-Term Superposition CAPWAP Total Resistance (kips)</th>
<th>Superposition % Required (%)</th>
<th>Long-Term Superposition API Total Resistance (kips)</th>
<th>Superposition % Required (%)</th>
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**Note:** at K-5, 72-hr BOR Rs = 4500 kips at Elev 210 feet
APPENDIX B
RECOMMENDATIONS FOR PILE DESIGN AND CONSTRUCTION
ESTIMATED PILE TIP ELEVATIONS / RECOMMENDED CONSTRICTOR PLATE LOCATIONS

Assuming steel constrictor plates are installed and welded within the interior of the piles to force plugs to form in the open-ended piles and assuming full end bearing conditions occur a depth of 2 to 3 pile diameters into the bedded chert soil layer and the steel constrictor plate engages at approximately 2 to 2.5 pile diameters into at least dense to medium dense soils, the following table outlines the maximum elevations, the estimated elevations and the anticipated lowest elevations that are recommended for pile tip elevations for each pier location of the Kentucky Lake Bridge. Please note that the actual elevation at which full-plugged conditions occur within the interior of the piles may vary from the elevations reported in the table below. In addition, the tip elevations are not necessarily based upon achieving the estimated nominal resistance per pile (the resistances at the selected elevations may be higher than required), but they also consider the behavior of the piles with the constrictor plates observed during the pile load test program performed between August 2013 and September 2013.

To correlate dynamic testing to the actual static pile nominal resistance, a static pile load testing program was recommended and performed prior to beginning production pile driving as part of the Lagoon Bridge construction. The testing was used to confirm the design assumptions and to correlate static resistance with dynamic pile-soil testing behavior and aid in production pile driving. The highest recommended pile tip elevations below are based upon observations made during the pile load test program and subsurface conditions indicated in the test borings. The recommended highest allowable tip elevations consider lateral load and information provided by the structural design team.

The following table also recommends the recommended constrictor plate elevation based upon the recommended highest allowable pile tip elevation and the top elevation of the silty sand and gravel to sand and gravel with silt strata encountered in the test borings. The pile load test program indicated that the SM or SP-SM (fine sand to fine silty sand) soil layer was not capable of engaging adequate (150 ksf or higher) end bearing values using the constrictor plates. Based upon observations made during the pile load testing program, we anticipate that the constrictor plate will penetrate approximately two pile diameters into the silty sand and gravel strata prior to developing significant end bearing resistance on the bottom side of the constrictor plate allowing the pile to behave in a plugged manner for the portion of the pile beneath the constrictor plate.
<table>
<thead>
<tr>
<th>Pier/ Stations</th>
<th>Pile Diameter (inches)</th>
<th>Top of Bedded Chert Elevation (feet)</th>
<th>Highest Allowable Pile Tip Elevation (feet)</th>
<th>Estimated Pile Tip Elevation (feet)</th>
<th>Lowest Anticipated Tip Elevation (feet)</th>
<th>Recommended Distance from Pile Tip to Bottom of Constrictor Plate (feet)</th>
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