DRAFT Geotechnical Report

US 68/KY 80 over Lake Barkley

11/20/2014

This document contains a current DRAFT of the Geotechnical Report for the Lake Barkley Bridge and includes the report body only and not Appendices. The report is currently undergoing internal reviews but this DRAFT is being provided to bidders as a courtesy. The Department intends to post the final report including appendices when it is available. However, the Department does not currently intend to post a “track changes” version since the report is not a contract document.

Geotechnical Engineering Report

Lake Barkley Bridge
US 68/KY 80 Bridge over Lake Barkley
Item No. 1-180.60
Lake Bridges Project
Trigg County, Kentucky

November 20, 2014
Terracon Project Number: N1115097

Prepared for:
Kentucky Transportation Cabinet
Frankfort, Kentucky

Prepared by:
H.C. Nutting, a Terracon Company
Cincinnati, Ohio

DRAFT

terracon.com
Terracon
Environmental ■ Facilities ■ Geotechnical ■ Materials

November 20, 2014

Kentucky Transportation Cabinet
1236 Wilkinson Boulevard
Frankfort, KY 40601

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

Re: Geotechnical Engineering Report for Lake Barkley Bridge - DRAFT
US 68/KY 80 Bridge over Lake Barkley
Item No. 1-180.60
Lake Bridges Project
Trigg County, Kentucky
HCN/Terracon Project No. N1115097

Dear Mr. Beckett:

H. C. Nutting, a Terracon Company (HCN), in conjunction with Florence & Hutcheson (F&H, now ICA), has completed the geotechnical engineering services for the above-referenced project. 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 6, 2011, and May 14, 2013. The purpose of this work was to perform geotechnical field, and laboratory testing to obtain site-specific data for the project site and to perform static and seismic geotechnical analyses for the proposed Lake Barkley Bridge.

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.
Senior Geotechnical Engineer

Ronald J. Ebelhar, P.E.
Senior Principal

Aaron J. Muck, P.E.
Principal

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EXECUTIVE SUMMARY

A geotechnical study has been performed for the proposed Kentucky Lake Bridge, Item No. 1-180.60 as part of the US68/KY 80 Bridge over Lake Barkley in Trigg County, Kentucky.

- An extensive field and laboratory test program was performed to study the subsurface soil and bedrock conditions for the proposed Lake Barkley construction for the referenced project. The field and laboratory program was performed in two phases. The initial phase was performed in 2011. The final phase was performed in 2014, once the proposed bridge pier locations and foundation type were known. The 2011 and 2014 programs were supplemented with previous boring programs performed in 1997 and 1999 by KYTC and in 2010 by Florence & Hutcheson and H.C. Nutting, a Terracon Company.
- The field test program included Standard Penetration Tests and Cone Penetration Tests. At selected boring locations, Pressuremeter Tests and PS Suspension Logging were performed.
- Field test program results indicate that soils within the previous flood plain of the Cumberland River consist of cohesive or cohesionless alluvial deposits underlain by granular alluvial and residual deposits with various amounts of chert pieces and fragments and then limestone bedrock. Borings performed in the previous channel of the Cumberland River indicate that soils consist of granular alluvial deposits with various amounts of chert and then limestone bedrock.
- Within the previous flood plain areas of the Cumberland River, weak and liquefiable soils were encountered beginning at the existing ground surface or mudline elevation to depths of up to 50 feet. These liquefiable layers will impact the Lake Barkley Bridge foundation design.
- Seismic response spectra have been developed and presented for the proposed Lake Barkley Bridge structure. In addition, the potential for liquefaction, liquefaction settlement and lateral spread has been evaluated for the foundation soils at the Lake Barkley substructure units.
- Drilled shaft foundation recommendations have been developed under both static and seismic conditions for support of the proposed Lake Barkley Bridge pier and End Bent 2 locations, including evaluation of axial resistance, tensile resistance, and estimated downdrag.
- Driven H-pile supported on limestone bedrock foundation recommendations have been developed under both static and seismic conditions for support of the proposed Lake Barkley Bridge End Bent 1 location, including evaluation of axial resistance, tensile resistance, driveability, settlement of pile foundations, settlement of pile foundation soils, and estimated downdrag.
- Based on relatively high axial and lateral loads at the bridge pier locations, foundation support of the bridge piers on drilled shaft foundations bearing in the limestone bedrock have been selected by the design team. Due to potential for karst conditions in the limestone bedrock at the End Bent 1 location, the design team has selected H-piles driven to practical refusal on bedrock to support End Bent 1. The number of H-piles will be increased in an effort to

overcome any loss of foundation support due to karst conditions at End Bent 1. At End Bent 2, the design team has selected drilled shafts bearing in the limestone bedrock, since it was determined that the depth of bedrock was such that shallow foundations were not considered economical and practical.

- Karst bedrock conditions encountered in the east approach span areas will impact the construction of the proposed drilled shaft foundations at the proposed Piers 3 to 5 of the Lake Barkley Bridge. Due to the encountered karst conditions in the limestone bedrock, additional rock core borings are recommended to be performed by the bridge contractor during construction of the bridge foundations to further explore the presence of, depth of and plan location of the voids/cavities within the limestone bedrock at Piers 1 to 6 of the proposed bridge.

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.

GEOTECHNICAL ENGINEERING REPORT - DRAFT
LAKE BARKLEY BRIDGE
US 68/KY 80 BRIDGE OVER LAKE BARKLEY
ITEM NO. 1-180.60
LAKE BRIDGES PROJECT
TRIGG COUNTY, KENTUCKY

Terracon Project No. N1115097
November 20, 2014

1.0 INTRODUCTION

A geotechnical engineering study has been completed for the proposed Lake Barkley Bridge, Item No. 1-180.60 as part of US68/KY 80 in Trigg County, Kentucky. This work was completed to supplement previous studies performed by KYTC and other consultants. To date, design information for the Lake Barkley Bridge has been provided to the design team in an incremental manner. This report compiles and expands upon the previously provided incremental information, but does not include any new design or construction recommendations.

1.1 Previous Reports

The following reports have previously been completed:

KYTC No.	DESCRIPTION	DATE	SOURCE
SA-022-2007	Report of Seismic Geotechnical Engineering Analyses Conducted in Support of the Bridge Type Study, US 68/KY 80, Proposed Bridge over Lake Barkley	February 6, 2009	Stantec Consulting Services, Inc. (formerly FMSM)
S-035-2006	Seismic Assessment of the Bridge Section, US 68/KY 80 Bridge over Lake Barkley	December 15, 2005	QORE, Inc.
R-005-1999	Geotechnical Engineering Roadway Report, US 68 (Cadiz-Aurora Road) (includes seismic analyses of the west causeway performed by QORE, Inc.)	January 14, 1999	KYTC Geotechnical Branch

1.2 Current Study

The Kentucky Transportation Cabinet (KYTC) has proposed bridge replacements for existing crossings of Kentucky Lake and Lake Barkley. These crossings follow the existing US 68/KY 80 highway corridor. The existing bridges are multi-span structures served by causeways on both ends of each structure.

The proposed Lake Barkley Bridge is located between US 68/KY80 Stations 3147+95 to 3186+00 across Lake Barkley (See Exhibit A-1), which is located approximately 82 to 140 feet north of the centerline of the existing bridge. The bridge crosses between the new west causeway (which was partially constructed in 2010) and the new End Bent 2 location of Lake Barkley. Lake Barkley has a Summer Pool Elevation of 359.0 feet and a Winter Pool Elevation of 354.0 feet. The west causeway is located from the west bank of Lake Barkley and extends into the lake to the west end bent (End Bent 1) for the proposed Lake Barkley replacement bridge. The east end bent (End Bent 2) is located above the bluff on the east lake shore. The east causeway for the existing bridge will not be expanded or used as support for the new bridge alignment.

Test borings (SPT) and Cone Penetration Test (CPT) soundings have been performed at the site between 1997 and 2014. The subsurface data was initiated with forty-five existing SPT borings with rock core and/or auger borings with rock core performed by KYTC in 1997 and 1999. Eighteen (18) Standard Penetration Test (SPT) borings with rock core were performed in 2010 and 2011 for the Lake Barkley Bridge ranging in depth below existing ground surface from 34.9 feet to 145.6 feet. Thirty-one (31) combined Cone Penetration Test (CPT) and SPT borings or just CPT soundings were performed in 2010 and 2011 to depths of 10.9 feet to 116.5 feet below existing grades. Three (3) rock core soundings were performed in 2010 and 2011 for the Lake Barkley Bridge ranging in depth below existing ground surface from 36.4 feet to 137.7 feet. Thirteen (13) Standard Penetration Test (SPT) borings with rock core were performed in 2014 for the Lake Barkley Bridge proposed drilled shaft locations ranging in depth below existing ground surface from 21.5 feet to 139.3 feet. Seven (7) combined Cone Penetration Test (CPT) and SPT borings with rock core were performed in 2014 for the proposed drilled shaft locations to depths of 87.6 feet to 162.9 feet below existing grades. In 2014, Fifty-seven (57) auger borings with rock core were performed at the proposed drilled shaft locations for the Lake Barkley Bridge ranging in depth below existing ground surface from 32.0 feet to 197.6 feet. The work scope included laboratory testing of selected soil samples, engineering analyses and recommendations for the design and construction of the bridge structure.

Prior to publishing this report, correspondence regarding foundation design and causeway global stability, both static and seismic, were provided to the design team within individual letters. Copies of selected correspondence are included in Appendix B of this report.

2.0 PROJECT INFORMATION

2.1 Lake Barkley Bridge

The proposed Lake Barkley Bridge has a length of 3805 feet and will support four lanes of vehicle traffic. The bridge deck width will generally be 75 feet – 4 inches, including a 10-foot-wide sidewalk. All information in **Section 2.0** is based upon the Stage 2 Final drawings dated November 7, 2014.

2.2 Lake Barkley Bridge Approaches

The approach spans are currently planned to consist of welded steel-plate girder construction. The west approach will be divided into seven spans and the east approach will be divided into six spans. The length of the proposed approach spans range from 211.25 feet to 290.0 feet. Expansion type end bents are planned due to the anticipated thermal movements at the end bent locations. The approach span piers will be supported on three columns connected to a single beam support. The following table outlines the end bent and pier locations along the proposed approaches relative to the US 68/KY 80 Centerline. The required nominal resistance for each foundation member, based on the pile structural resistance for End Bent 1 and a resistance factor of 0.5 for both end bearing and side resistance within the bedrock socket for the drilled shafts supporting the remaining piers and End Bent 2, was reported on the November 7, 2014 plans.

Lake Barkley Bridge Substructure Locations	Station	Maximum Required Nominal Resistance per Foundation Member (kips)
West End Bent (End Bent 1)	3147+95.00	550
Pier 1	3150+19.75	12850
Pier 2	3152+89.75	12900
Pier 3	3155+59.75	14390
Pier 4	3158+29.75	14730
Pier 5	3161+19.75	14280
Pier 6	3163+69.75	14120
Pier 9	3173+68.75	13200
Pier 10	3176+23.75	13060
Pier 11	3178+78.75	11420
Pier 12	3181+33.75	11390
Pier 13	3183+88.75	10760
East End Bent (End Bent 2)	3186+00.00	1464

Based upon the Stage 2 Final Plans provided by Michael Baker Jr. and Palmer Engineering, between approximately 10 and 32 feet of new embankment fill is proposed at the End Bent 1 location. The approach causeway embankment up to approximately Station 3147+25 has been placed and compacted previously. The embankment fill from approximately Station 3147+25 to the proposed End Bent 1 location will be placed and compacted as part of the proposed bridge construction. The Stage 2 Final Plans indicate that between 8 to 10 feet of new embankment is proposed at the End Bent 2 location. End Bent 1 will be pile-supported and constructed so the pavement elevations at End Bent 1 will be at approximate Elevation 382 feet on the new causeway. End Bent 2 will be supported using drilled shaft foundations bearing in the limestone bedrock, the pavement elevation at End Bent 2 will be approximately Elevation 402 feet. The

spans will be supported on piers with drilled shaft foundations. Mudline elevations for the approach span piers generally range from Elevation 336 to 358 feet.

2.3 Main Span

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 3166+01.00 to Station 3171+51.00) and support four traffic lanes. In the plans the west pier is referred to as Pier 7, and east pier is referred to as Pier 8. The Main Span will span the navigation channel of Kentucky Lake and will be constructed using structural steel. The Main Span will be supported on 3 column piers having column diameters of 12 feet. The final plans indicate that the required nominal resistance for each foundation member (drilled shaft socketed into limestone bedrock) is 8,960 kips based on resistance factors of 0.5 for both end bearing and side resistance in the bedrock socket. Mudline elevations range from approximate Elevations 308 to 332 feet at the Main Span pier locations.

3.0 SUBSURFACE CONDITIONS

3.1 Lake Barkley Bridge

As mentioned in **Section 1.0**, field work performed in 2010, 2011 and 2014 for this study included twenty-eight (28) soil test borings with rock core by F&H, KYTC, and HCN; thirty-six (36) cone penetration test (CPT) soundings with some having rock core by HCN; and thirty-seven (37) auger borings with rock core performed by F&H, KYTC and HCN. This test boring data was supplemented with data from previous studies performed by KYTC in 1997 and 1999. Exhibits A-2 to A-9 show the locations of the borings performed for this study as well as historical borings performed by others in the vicinity. The Subsurface Data sheets are shown on Exhibits A-10 to Exhibit A-46 along with terminology and symbols used on the CPT sounding logs. Exhibits A-2 through A-46 were prepared by Florence & Hutcheson (now ICA Engineering).

3.1.1 Geology

The geology of the site is influenced by the Illinois Basin. Mississippian rock strata dip generally northward into the basin with some variation due to faulting. Normal faults are northeast trending forming horst and graben type faulting. The movement along the faults likely ceased prior to pre-Cretaceous deposition. Rock formations dip at an estimated 1 to 1.5 degrees southwest.

The Mississippian Warsaw formation is typically exposed east side of Lake Barkley in a bluff. The Warsaw formation is typically composed of medium to coarse grain, bioclastic limestone with some cherty beds. The formation also contains solution enhanced fractures and cavities, and some vuggy texture. The fractures and vugs can be clay filled. Styolites and larger scale voids are also possible in the formation. Within the flooded Cumberland River valley, the Warsaw is overlain by residual soils (10 to 24 feet thick) and/or relatively thick alluvium (50 feet to 70 feet thick).

Oftentimes the alluvium contains chert fragments, which have been derived from the surrounding formations and deposited by river action. Some hilltops contribute rounded chert pebbles and cobbles derived from the Tuscaloosa Formation.

The encountered soil conditions across the proposed Lake Barkley Bridge site have been influenced by the flooded Cumberland River. The east and west sides of the proposed bridge alignment contain flood deposits and upper granular alluvial deposits. The flood deposits and upper alluvial deposits have been eroded within the previous Cumberland River channel in the proposed bridge alignment.

3.1.2 Submerged West Cumberland River Flood Plain Typical Profile

Based on the results of the subsurface exploration program, subsurface conditions at the project site can be generalized in four areas. The first area, which represents the submerged west flood plain of the Cumberland River, including the west bank, is from approximately Station 3147+00 to 3167+50. The second area, which represents the submerged channel of the Cumberland River, is from approximately Station 3167+50 to 3171+00. The third area, which represents the submerged east flood plain of the Cumberland River including the east bank, is from approximately Station 3171+00 to 3185+00. The fourth area represents the limestone bluff area just east of the east shore of Lake Barkley and is from approximately Station 3185+00 to 3186+00.

Based on the results of the subsurface investigation program, subsurface conditions between Stations 3147+00 to 3167+50 can be generalized as follows:

Stratum	Approximate Depth to Bottom of Stratum (feet)	Material Encountered	Consistency/Density
Stratum 1	15 to 20 (STA 3147+00 to 3147+75)	Embankment fill, silty gravel to poorly graded sand with grave, occasional clay seams	Loose to medium dense
Stratum 2	20 to 70	Silty clay, lean clay, silt, silty sand flood plain deposits	Soft to stiff and very loose to medium dense
Stratum 3	48 to 108	Silty sand, poorly-graded sand with silt, poorly-graded sand, well-graded sand with silt, silty gravel, poorly-graded gravel with silt, trace clay seams, trace chert pieces and fragments	Medium dense to very dense
Stratum 4	84 to 198 (borings and CPT/borings with rock core terminated in this stratum)	Limestone bedrock, fine to scattered coarse grained with some chert nodules and occasional isolated air or clay filled voids	Hard to very hard (in terms of rock hardness)

Ten (10) of the borings between the proposed End Bent 1 and the submerged Cumberland River encountered voids or cavities in the rock core performed in the borings. The voids/cavities in the bedrock were encountered starting at between 0.6 to 34.0 feet below the top of encountered bedrock in these borings. The bottom of the lowest encountered voids/cavities was approximately 1.3 to 61.9 feet below the top of encountered bedrock in these borings. The following table reports the boring numbers, top of mudline elevation, top of bedrock elevation and the top and bottom of the cavity/void encountered within the boring. The voids/cavities encountered were oftentimes not continuous between top elevation and the bottom elevation reported in the below table, please refer to the boring logs and the subsurface data sheets for additional information regarding the encountered voids in the test borings.

Boring	Mudline Elevation (feet)	Top of Rock Elevation (feet)	Top Elevation Highest Encountered Void (feet)	Bottom Elevation Lowest Encountered Void (feet)
4013	371.7	263.3	250.5	240.5
5009	348.3	244.1	243.5	226.0
13A	350.8	257.4	254.0	250.2
5011	349.0	250.1	249.3	237.1
5014W	348.5	258.0	237.3	211.5
5014B	347.4	257.4	234.6	214.9
5014A	346.0	258.3	235.4	196.4
5014	347.8	258.7	234.7	210.4
5015	347.5	257.6	244.7	235.4
5014E	347.1	258.2	224.2	208.5

3.1.3 Submerged Cumberland River Channel Typical Profile

The submerged Cumberland River Channel has been divided into the west and east piers, since the west main span pier is located closer to the top of the submerged west river bank, whereas the east main span pier is located closer to the toe of the submerged east river bank. The east riverbank slope is somewhat steeper than the west riverbank slope in the vicinity of the proposed bridge. Based on the results of the borings, subsurface conditions between at the west and east main span piers can be generalized as follows:

3.1.3.1 West Main Span Pier

Stratum	Approximate Depth to Bottom of Stratum (feet)	Material Encountered	Consistency/Density
Stratum 1	20 to 27	Silty clay, lean clay or silt, flood plain deposits	Soft to medium stiff
Stratum 2	65 to 70 (some borings and CPT/borings were terminated in this stratum)	Poorly-graded sand with silt, poorly-graded sand, well-graded sand, poorly-graded gravel, well-graded gravel, contains pieces and fragments of chert	Medium dense to very dense, occasional loose seams
Stratum 3	108.2 to 145.6 (not the bottom of the strata, some borings terminated in this strata)	Limestone bedrock, fine to scattered coarse grained with some chert nodules	Limestone - Hard to very hard (in terms of rock hardness)

In Test Boring 5027, a possible cavity in the bedrock was encountered from 0.4 to 1.3 feet below the top of bedrock elevation. The encountered cavity could have been due to misinterpretation of a boulder or limestone floater located above the actual top of bedrock.

3.1.3.2 East Main Span Pier

Stratum	Approximate Depth to Bottom of Stratum (feet)	Material Encountered	Consistency/Density
Stratum 1	3 to 15	Silty clay, lean clay or silt, flood plain deposits	Soft to medium stiff
Stratum 2	8 to 19	Silty sand, poorly graded sand with silt and silt	Loose to medium dense, occasional dense seams
Stratum 3	49 to 61 (some borings and CPT/borings were terminated in this stratum)	Poorly-graded sand, well-graded sand, poorly-graded gravel, well-graded gravel, contains pieces and fragments of chert	Medium dense to very dense
Stratum 3	90.8 to 136.5 (not the bottom of the strata, some borings terminated in this strata)	Limestone bedrock, fine to scattered coarse grained with some chert nodules	Limestone - Hard to very hard (in terms of rock hardness)

3.1.4 Submerged East Cumberland River Flood Plain Typical Profile

Based on the results of the borings, subsurface conditions between Stations 3171+00 to 3185+00 can be generalized as follows:

Stratum	Approximate Depth to Bottom of Stratum (feet)	Material Encountered	Consistency/Density
Stratum 1	5 to 30 (Borings along existing causeway only)	Embankment fill, lean clay	Soft to stiff
Stratum 2	16 to 66	Silty clay, lean clay, fat clay and silt, flood plain deposits	Soft to very stiff
Stratum 3	36 to 84	Silty sand, poorly-graded sand with silt, trace clay seams	Loose to medium dense
Stratum 4	61 to 85 (some borings and CPT/borings terminated in this stratum)	Silty sand, poorly-graded sand with silt, well-graded sand with silt or clay, poorly-graded sand, silty gravel, trace clay seams, trace chert pieces and fragments	Medium dense to dense, some very dense seams and occasional loose seams
Stratum 5	85 to 99 (not the bottom of the strata, some borings terminated in this strata)	Limestone bedrock, fine to scattered coarse grained with some chert nodules and occasional isolated air or clay filled voids	Hard to very hard (in terms of rock hardness)

At approximately Station 3180+50, the surface of the bedrock started increasing in elevation. The bedrock elevation increased from approximately Elevation 261 feet near Station 3180+50 to approximately Elevation 298 feet near Station 3183+50.

3.1.5 Limestone Bluff (east side Lake Barkley) Typical Profile

Based on the results of the borings, subsurface conditions between Stations 3185+00 to 3186+00 can be generalized as follows:

Stratum	Approximate Depth to Bottom of Stratum (feet)	Material Encountered	Consistency/Density
Stratum 1	10 to 32	Silty clay, lean clay, elastic silt, residual soil	Stiff to very stiff
Stratum 2	36.4 to 67.6 (not the bottom of the strata, some borings terminated in this strata)	Limestone bedrock, fine to scattered coarse grained with some chert nodules	Hard to very hard (in terms of rock hardness)

The top of bedrock elevation varied from approximately Elevation 381.5 to 359.1 feet. The variation in top of bedrock elevation could be an indication of pinnacles and valleys at the top of bedrock surface, which would indicate karst conditions. At Borings 5066 and 5066A, which were drilled approximately 18 inches from one another in plan dimension, encountered bedrock at Elevations 362.3 feet and 381.4 feet, respectively, which indicates the potential for abrupt changes

in the top of bedrock elevation within a relatively short lateral distance. In addition, voids/cavities were encountered at four of the test borings drilled at or near the proposed End Bent 2 location. The top of the encountered voids/cavities were encountered between 2.4 to 5.4 feet below the top of encountered bedrock. The bottom of the lowest void or cavity in these borings was encountered between 3.4 to 8.3 feet below the top of the encountered bedrock elevation. The thickness of the encountered voids/cavities ranged from 0.5 to 5.9 feet thick. The following table summarizes the voids/cavities encountered within the test borings performed at the proposed End Bent 2 location.

Boring	Mudline Elevation (feet)	Top of Rock Elevation (feet)	Top Elevation Highest Encountered Void (feet)	Bottom Elevation Lowest Encountered Void (feet)
5055	391.2	366.0	363.2	362.6
5056	390.9	369.4	366.2	365.3
5060	390.7	371.3	365.9	365.4
5069	391.9	365.9	363.5	357.6

Conditions encountered at each boring location are indicated on the individual boring logs. Stratification boundaries on the boring logs represent the approximate location of changes in soil types; in-situ, the transition between materials may be gradual. Details for each of the borings can be found on the logs in Appendix A of this report.

3.1.6 Groundwater

Due to the introduction of drilling fluid to facilitate casing advancement in all borings in the existing causeways and the remaining borings being performed from barges on the lake, groundwater levels were not obtained while drilling, or after the completion of drilling in these borings. Considering the low permeability of some of the soils encountered in the borings on the existing causeways, a relatively long period of time would be necessary for a groundwater level to develop and stabilize in a borehole in these materials. Long-term observations in piezometers or observation wells sealed from the influence of surface water are often required to define groundwater levels in materials of this type. It is anticipated that the static groundwater readings would closely mimic the static pool elevations of Lake Barkley. The Summer pool elevation of Lake Barkley is reported as Elevation 359.0 feet. The winter pool elevation of Lake Barkley is reported as Elevation 354.0 feet.

Groundwater level fluctuations in the causeway soils surrounding the Lake Barkley bridge site occur due to seasonal variations in the amount of rainfall, runoff, the varying Lake Barkley pool levels and possibly other factors not evident at the time the borings were performed. Based on this, long-term groundwater monitoring was determined to be unnecessary. Therefore, groundwater levels during construction or at other times in the life of the structure may be higher or lower than the levels indicated on the stability cross-sections. The possibility of groundwater

level fluctuations should be considered when developing the design and construction plans for the project.

4.0 RECOMMENDATIONS FOR DESIGN AND CONSTRUCTION

This section presents recommendations for the Lake Barkley Bridge. Based on the information obtained from the field and laboratory study and our analyses, the following key geotechnical considerations for the Lake Barkley Bridge were identified.

- Settlement of the foundation soils surrounding the foundations at the end bents
- Global Stability of the End Bent embankments (static and seismic conditions)
- Axial foundation support of the bridge
- Installation of the drilled shaft foundations
- Lateral response of bridge foundations
- Seismic site response of the bridge foundation soils
- Liquefaction potential of the bridge foundation soils
- Seismic settlement of the bridge foundation soils
- Seismic axial foundation support
- Seismic lateral response of the bridge foundations

Recommendations and Special Notes for drilled shaft construction for the proposed Lake Barkley Bridge were also developed as part of this work.

In December of 2012, the design earthquake was changed for the design of the Lake Barkley Bridge. Initially the design earthquake was based upon a 2500-year return period. In December of 2012, the design earthquake was reduced to a 1000-year return period. As a result, all geotechnical analyses involving seismic events that had initially assumed a 2500-year design earthquake event were reanalyzed using a 1000-year design earthquake.

4.1 Development of Design Soil Parameters for Lake Barkley Bridge

The soil borings and CPT soundings provided the needed data and intact samples for testing in the upper lean clay and silt soils, as well as the underlying granular alluvial soils. Chert gravel was encountered in granular soil samples below approximately Elevation 300 feet. Based on the grain-size analyses, the gravel portion of the granular soils is less than 3-inches in diameter and less than 10% of soil matrix has diameters between ¾ and 3 inches. Occasionally, refusal to split-barrel sampling was encountered within the granular soils which may be an indication of the presence of granular particles with diameters greater than 2 inches or the presence of cobble-size particles.

We used the results of our field study and previous work by others to develop idealized soil profiles for geotechnical analyses at each of the end bent and pier locations. We estimated engineering properties from the results of the field work and laboratory testing for input into analyses for settlement, axial driven pile resistance, lateral deformation, site response, liquefaction, seismic settlement, axial seismic pile resistance and lateral seismic response. Additional details of the parameters used are discussed below.

Where available, results of consolidated-undrained (CU) triaxial shear tests and unconsolidated-undrained triaxial (UU) shear tests were used to estimate the effective (long-term) soil shear strength parameters and the total (short-term) soil strength parameters in the fine-grained soil samples collected in the test borings. Data from these tests were also used to estimate the unit weight parameters of the fine-grained soils at the site. This data was supplemented with data from the CPT soundings. One-dimensional consolidation test data was used to estimate the soil consolidation data and time-rate factors for the fine-grained foundation soils at End Bent 1. Constrained Modulus data interpreted from the CPT soundings was also used to estimate soil consolidation parameters and the stress history of the foundation soils at End Bent 1, which confirmed the parameters determined from the one-dimensional consolidation test results. Refer to Appendix G for the Laboratory Test Results.

Soil shear strength and unit weight were estimated for the existing fill (embankment) soils from laboratory testing on samples from previous studies.

The soil shear strength parameters (effective strength parameters) of the granular soils were estimated using the SPT N-value data from the test borings. The estimated internal angle of friction values were limited to between 34 and 36 degrees, since it was believed some of the N-values were being skewed upward by a small percentage of gravel or chert gravel in the granular soils as discussed above. Where SPT N-values were obtained that did not appear to be skewed by the gravel or chert gravel, the shear strength parameters of the granular soils were estimated using published AASHTO correlations. Unit weights of some of the granular soils were based upon previous triaxial consolidated-undrained shear test results.

Consolidation parameters were derived from one-dimensional consolidation test data from Borings B-4007, B-4009, and B-4013 performed near End Bent 1. A one-dimensional consolidation test was also performed on a soil sample from B-4022 located at End Bent 2. Compression and recompression indices and overconsolidation ratios (OCR) were also estimated from CPT data, where no consolidation test data was available. The parameters derived from CPT C-4006 and C-2104 were compared to the one-dimensional consolidation data from B-4007 and B-4009. Where laboratory data or CPT data were not available, corrected N-values from SPT borings were used to estimate compression and recompression indices.

Results of the pressuremeter testing (PMT) were used to select lateral soil parameters that were provided to the structural design team for use in their lateral foundation analyses. In addition, the SPT N-values and laboratory shear strength testing was used to select lateral soil response parameters, based upon published correlations. A table summarizing the pressuremeter testing results is included in Appendix I.

From the acceleration profiles developed by the enhanced site response analyses, appropriate seismic coefficients were selected using guidance outlined in FHWA HI-99-012, with updates as appropriate by Dr. Ed Kavazanjian, for use in pseudo-static seismic slope stability analyses. Residual or reduced strength performance of soil after seismic loadings for site-specific soils may also influence slope stability considerations for embankments, existing or proposed causeways, and deep foundation design parameters. We calculated residual strengths for the liquefied layers using our CPT tests and relationships proposed by Olson and Johnson (2008). We compared these results to post-cyclic static strength values obtained from the direct simple shear tests, found the CPT values to be conservative and used them for the post-liquefaction static analyses.

Groundwater in steady-state, short-term and seismic analyses was assumed to be at the Summer Pool Elevation = 359.0 feet for global stability and foundation analyses. For global stability, rapid drawdown analyses of the end bent side slopes and end slopes, the following two cases were assumed:

1. The difference between maximum flood at Elevation 375 feet to normal pool at Elevation 359 feet.

2. The difference between normal pool at Elevation 359 feet to loss of lake at Elevation 300 feet.

4.2 Lake Barkley Bridge Seismic Evaluation

The soil borings provided good quality, intact samples for geotechnical laboratory testing which provided site-specific soil parameters under dynamic loading. The CPT and SCPT soundings provided high-resolution stratigraphic profiling, soil resistance data and shear wave velocity data to use in conjunction with data developed from laboratory testing and from previous studies.

We used the results of our field study and previous work by others to develop idealized soil profiles for seismic geotechnical analyses. We estimated engineering properties from the results of the field work and laboratory testing for input into the site response, liquefaction, seismic settlement and embankment deformation predictions. Additional details of the parameters used are discussed below.

Field sampling and laboratory cyclic shear testing with post-cyclic static strength tests were performed on samples from representative embankment and foundation soil and the results

were used in more rigorous and detailed slope stability analyses. After performing liquefaction analyses, lateral displacement analyses for lateral spread were also performed.

Executive Summaries of the Lake Barkley Response Spectra and Liquefaction/Seismic Settlements using the 1000-year design criteria were provided to the design team at various dates. The design memorandums are divided into the end bents, the east approach spans, the west approach spans and the main span piers and are included in Appendix B.

As previously mentioned, in December 2012, the design seismic event was changed from a 2500-year return period to a 1000-year return period. The seismic evaluation for the Lake Barkley Bridge was initially performed using the 2500-year design earthquake. Results of the Lake Barkley Response Spectra and Liquefaction and Seismic Settlement using the 1000-year design event are reported in Appendix B in our design memorandums.

4.2.1 Equivalent Linear Site Response Analyses

Site response analyses were performed to characterize the impact of seismic accelerations on the subsurface soils and bedrock supporting the Lake Barkley Bridge. This information is used as input for liquefaction analyses and slope stability analyses. The equivalent linear site response analyses model the soil and bedrock as horizontally layered, linear visco-elastic material characterized by an initial low-strain shear modulus (shear wave velocity) and an equivalent viscous damping ratio. The non-linear, strain-dependent behavior of the soil and bedrock layers are modeled using modulus reduction and damping curves which represent the variation of these with increasing shear strain. The FHWA Publication HI-99-012 indicates that the one-dimensional response analysis approach provides a reasonable approximation of two-dimensional response.

The seismic time histories in bedrock for the design earthquake events are also required inputs for the equivalent linear site response analyses. A seismic ground motion study was performed by Pacific Engineering in 2008 (included as part of KYTC Reference No. SA-023-2007). The Pacific Engineering report used the most current attenuation relations available at the time. The US Nuclear Regulatory Commission (NRC), in collaboration with the US Geological Survey (USGS) and national experts, is funding an initiative to develop Next Generation Attenuation Relationships for the Central and Eastern US. We contacted the Pacific Earthquake Engineering Research Center for the latest attenuation relationships to evaluate the impact of the relationships on the site response analyses and learned that this research will not be complete until 2014. Although some of the researchers released their updated relationships in June 2011, we consider it premature to alter the Pacific Engineering report. As part of their work, Pacific Engineering developed six (6) three-dimensional bedrock time histories. Of these six (6) time histories, three (3) were “natural” time histories measured from actual events for a similar fault mechanism and three (3) were “synthetic” time histories. This was done since there are no actual recorded time histories available for NMSZ events approaching the magnitude of the design events. The bedrock time histories were modeled as events occurring within a bedrock layer having an average shear wave velocity of 8500 feet per second. Each of the six

(6) three-dimensional time histories included two (2) orthogonal horizontal (H1 & H2) and one (1) vertical (V) record for each recurrence interval (500-, 1000- and 1000- and 2500-year return period) evaluated. KYTC Reference No. SA-023-2007 shows the horizontal time histories for each of the 2500-year return period events used for this study.

Shear wave velocity profiles were developed using a combination of the previously performed crosshole seismic tests and shear wave reflection survey and the downhole seismic tests performed for this project. The crosshole and downhole seismic tests were used for the upper 75 to 140 feet of each soil profile and the Bay Geophysical shear wave reflection surveys and Geovision P-S Suspension Logs (See Appendix H) were used where appropriate for the remainder of the soil profile to the top of bedrock. As described earlier, the bedrock layer is assumed to have a shear wave velocity of 8500 feet per second to be consistent with the values used and recommendations in the Pacific Engineering report and results in a significant impedance contrast.

As indicated earlier, we performed drilling and sampling to obtain intact samples at selected locations. Laboratory testing included Index (limits/gradations), Resonant Column tests, Static and Cyclic Direct Simple Shear Tests with post-cyclic static test to failure on selected representative samples. The resonant column and cyclic simple shear tests were used to develop shear modulus and damping variations with strain for each major soil unit at each location.

Equivalent linear site response analyses were performed for existing soil conditions using the suite of time histories developed by Pacific Engineering in 2008 as the input motions. A set of 6 time histories for each horizontal direction (designated H1 and H2) were used. The approach spans were divided into ranges, based upon similar soil conditions and site response behavior. The main span piers were divided into the east and west piers, and the end bents were divided into End Bent 1 and End Bent 2. Analyses performed using SHAKE2000 indicated that peak horizontal ground accelerations at the Lake Barkley Bridge ground surface at the end bents, approach spans and main span piers for the design 1000-year earthquake event will range from about 0.24 to 0.54g's for the locations analyzed.

Please refer to the table below for the average peak ground accelerations at the ground surface. In the table the substructures at the west and east approaches have been divided into three or four ranges each by Stations.

Area	Range Description	Station Ranges	Average Peak Ground Accelerations at Ground Surface (g)
End Bent 1		3147+95	0.24
West Approaches	Range 1	3148+00 to 3152+01	0.27
West Approaches	Range 2	3152+01 to 3158+00	0.28
West Approaches	Range 3	3158+00 to 3165+00	0.24
West Main Span Pier		3166+01	0.36

Area	Range Description	Station Ranges	Average Peak Ground Accelerations at Ground Surface (g)
East Main Span Pier		3171+51	0.54
East Approaches	Range 1	3172+00 to 3177+00	0.33
East Approaches	Range 2	3177+00 to 3181+50	0.28
East Approaches	Range 3	3182+90 to 3183+10	0.34
East Approaches	Range 4	3184+40 to 3184+60	0.31
End Bent 2		3186+00	0.38
End Bent 2		3186+00	0.15 (bedrock)

Generally the response analyses showed two distinct period ranges at the main span piers and the east abutment (End Bent 2) at which significant spectral acceleration was occurring at the ground surface. The following table outlines the distinct period ranges at the main span piers and End Bent 2.

Range Description	Station Ranges	Period Range 1 (seconds)	Period Range 2 (seconds)
West Main Span Pier	3165+75 to 3166+25	0.08 to 0.20	0.39 to 0.50
East Main Span Pier	3170+75 to 3171+25	0.08 to 0.16	0.27 to 0.34
End Bent 2	3186+00	0.10 to 0.12	0.20 to 0.30

Generally the response analyses showed three distinct period ranges at End Bent 1, the west approach spans and the east approach spans at which significant spectral acceleration was occurring at the ground surface. The following table outlines the distinct period ranges at the east and west approach spans and End Bent 1.

Range Description	Station Ranges	Period Range 1 (seconds)	Period Range 2 (seconds)	Period Range 3 (seconds)
End Bent 1	3147+95	0.1 to 0.15	0.2 to 0.3	0.6 to 0.85
West Approaches Range 1	3148+00 to 3152+01	0.05 to 0.1	0.15 to 0.25	0.45 to 0.62
West Approaches Range 2	3152+01 to 3158+00	0.08 to 0.14	0.2 to 0.25	0.55 to 0.65
West Approaches Range 3	3158+00 to 3165+00	0.08 to 0.15	0.2 to 0.25	0.6 to 0.68
East Approaches Range 1	3172+00 to 3177+00	0.1 to 0.14	0.22 to 0.27	0.44 to 0.62
East Approaches Range 2	3177+00 to 3181+50	0.1 to 0.13	0.25 to 0.3	0.43 to 0.64
East Approaches Range 3	3182+90 to 3183+10	0.1 to 0.13	0.16 to 0.21	0.42 to 0.57
East Approaches Range 4	3184+40 to 3184+60	0.07 to 0.11	0.19 to 0.31	No peak

Equivalent linear site response analyses were performed for existing soil conditions with proposed fill to planned grades at the west causeway expansion locations using the suite of time histories developed by Pacific Engineering in 2008 as the bedrock input motions. A set of 6 time histories for each horizontal direction (designated H1 and H2) were used.

The equivalent linear analyses, performed using the computer program SHAKE2000, indicated that average peak horizontal ground accelerations at the ground surface at the West Causeway for the design 1000-year earthquake event will range from about 0.24 to 0.28 g's as shown in the table

below. Average embankment accelerations shown below were determined from the site response analyses using the shear stresses and total normal stresses in the causeway embankment. The seismic coefficient for use in pseudo-static slope stability analyses is calculated as one-half the average embankment acceleration shown below.

Range Description	Station Ranges	Average Peak Ground Accelerations at Ground Surface (g)	Average Embankment Acceleration (g)	Seismic Coefficient
West Causeway	3141 to 3147	0.28	0.169	0.084
End Bent 1	3147+95	0.24	0.144	0.072

The site response analyses generated response spectra at the ground surface and selected depths below ground surface at each of the end bents, the approach spans and for the main span piers. For the ground surface at each location, the spectral ordinates corresponding to the short period (0.2 seconds) and 1-second period from each of the two horizontal sets (H1 and H2) were averaged and truncated design spectra were developed using the three-point method in the AASHTO LRFD Bridge Design Specifications 6th Edition (2012) for each location.

For the ground surface acceleration response spectra at each location, the spectral ordinates corresponding to the short period (0.2 seconds) and the 1-second period from each of the two horizontal sets of ground motion (H1 and H2) were averaged and truncated design spectra were developed using the three-point method in the AASHTO LRFD guidance for each location. The truncated three-point method did not accurately model the response. Therefore, the truncated spectra were adjusted as appropriate to capture the overall response from all the H1 and H2 analyses for each of the East and West Approach Span station ranges and the Main Span Pier Locations. The interpreted AASHTO-type horizontal response spectra are shown on Exhibits C-1 to C-3 for the ground surface for the East and West Approach Span station ranges and the Main Span Pier Locations.

Due to the shallow bedrock conditions at the end bent/abutment locations (which are not accounted for using the prescriptive AASHTO 3-point spectrum), the truncated three-point method did not accurately model the responses. Therefore, the truncated spectra were adjusted as appropriate to capture the overall response from all the H1 and H2 analyses for the Abutments. The interpreted AASHTO-type horizontal response spectra are shown on Exhibit C-4 for the ground surface for the End Bents/Abutments and bedrock for the East Abutment.

In accordance with FHWA guidance (NHI-11-032), the vertical response spectra was calculated from the Vertical to Horizontal (V/H) spectral ratios for the bedrock input motions provided by Pacific Engineering as 2/3 of the horizontal spectra for the periods considered here. In Appendix C, Exhibits C-5 to C-8 present the vertical response spectra for the West Approach Spans, East Approach Spans, Main Span Piers and the end bents/abutments. In Appendix C, Exhibits C-9 to C-11 present the individual response spectra of the H1-component horizontal site response analyses for the West Approach Spans, and Exhibits C-12 to C-15 present the individual response spectra of the H1-component horizontal site response analyses for the East Approach Spans.

Exhibits C-16 and C-17 in Appendix C present the individual response spectra of the H1-component horizontal site response analyses for the West and East Piers of the Main Span. Exhibits C-18 and C-19 in Appendix C present the individual response spectra of the H1-component horizontal site response analyses at the End Bents 1 and 2, respectively. Exhibit C-20 in Appendix C presents the response spectra of the H1-component horizontal site response analyses at the End Bents 2/East Abutment at the bedrock surface. In Appendix C, Exhibits C-21 to C-23 present the individual response spectra of the H2-component horizontal site response analyses for the West Approach Spans, and Exhibits C-24 to C-27 present the individual response spectra of the H2-component horizontal site response analyses for the East Approach Spans. Exhibits C-28 and C-29 in Appendix C present the individual response spectra of the H2-component horizontal site response analyses for the West and East Piers. Exhibits C-30 and C-31 in Appendix C present the individual response spectra of the H1-component horizontal site response analyses at the End Bents 1 and 2, respectively. Exhibit C-32 in Appendix C presents the response spectra of the H2-component horizontal site response analyses at the End Bents 2/East Abutment at the bedrock surface.

Acceleration and displacement time histories for H1, H2 and the vertical component (at 10-foot elevation intervals and the top of bedrock for approach span Pier 11 and the East Main Span Pier) were developed for one earthquake record. The data files were compiled and sent to the structural designer of the approach spans for use in their structural models. After providing this information to the structural design team, the design team concluded that time history records at 6 drilled shaft diameters below the mudline were required for all substructure units for all 6 available seismic records.

For the purposes of the liquefaction analyses discussed below, at a given location, the cyclic stress profile was calculated using the average of the shear stress profiles for all the time histories developed from SHAKE2000 for both horizontal (orthogonal) records. This means a total of twelve time histories were averaged and considered for each location. Similarly, for the pseudo-static analyses discussed below, the average acceleration in the embankment for all twelve time histories was calculated for each location.

4.2.2 Non-Linear Site Response Analyses

There are more comprehensive site response analyses that could be considered, including non-linear, effective-stress based analytical approaches (DESRA/D-MOD, etc.) that include pore pressure generation models to predict the shear stresses induced in the soil profile. D-MOD analyses were performed for the 2500-year design event at the Kentucky Lake Main Span Piers. The results of the analyses were very similar to the equivalent linear analyses. Since the SHAKE analyses were not significantly different, it was decided that DESRA / D-MOD analyses not be performed for the 1000-year site response analyses for the Lake Barkley Bridge.

4.2.3 Liquefaction Analyses

The use of Standard Penetration Test (SPT) blow count data often leads to overly conservative interpretations of liquefaction potential and seismic settlement. SPT blow counts may be heavily impacted by borehole disturbance in saturated soils, and due to the intermittent sampling intervals. The Cone Penetration Test (CPT) provides a continuous record of soil resistance (i.e., where the SPT gives a single value for a 2.5- to 5-ft depth increment, the CPT will represent the soil resistance by 25 to 40 data values in that same increment at a higher precision and resolution) and has no borehole disturbance effects to influence the results. The CPT was used as an enhanced tool on this project to allow for more comprehensive evaluation of the liquefaction potential of the soil profile. We performed seismic CPT soundings through the existing embankments and the subsurface soil at locations where SPT borings from previous studies indicated the potential for liquefaction.

The primary geotechnical inputs to the liquefaction analyses were based on the CPT data (or SPT data), and the laboratory tests to evaluate soil type (plasticity and gradation), and empirical estimates/measurements unit weight. This information was used to calculate the Cyclic Resistance Ratio (CRR).

The Cyclic Stress Ratios (CSR's) calculated from the shear stress profiles developed from our site response analyses as discussed above were used. Shear stress values were averages of shear stress profiles developed from six (6) sets of time histories used to model a given seismic event (1000-year return periods). Since two independent horizontal records were considered for each time history, a total of twelve time histories were actually used to develop the CSR.

Liquefaction analyses have been completed at selected CPT and SPT boring locations in the End Bents, approach span ranges and Main Span locations using the cyclic stress ratio profile obtained from the SHAKE analyses. The CPT analyses were performed using methods described by Robertson (2009). The SPT analyses were performed using methods described by Boulanger and Idriss (2004). Based on these analyses, there is a significant potential for liquefaction in the range of the End Bent 1, Approach Span stations, and the Main Span Piers using the 1000-year event. The liquefaction analyses indicate a range of 5 to 32 feet of liquefiable/softened soils in this range of Stations. Liquefaction is estimated to result in up to 20 inches of vertical settlement and up to 345 inches of lateral displacement if no mitigation is performed at these locations. Note that these settlement and lateral displacement estimates are made using empirical correlations and are for preliminary design purposes only.

The highest estimated vertical and horizontal displacements are located in the east and west approach span areas and in portions of the Main Span piers. Note that these settlement and lateral displacement estimates are made using empirical correlations and are for information only. The lateral spreading estimates, in particular, should be interpreted as giving only a rough order of magnitude estimate of lateral displacements, particularly when the calculated displacement is large. In addition, the lateral displacement estimate is for lateral spreading of near level ground and not a sloping ground surface. A summary of the estimated liquefiable

layer thickness, seismic settlements and lateral displacements are shown below for selected boring/sounding locations:

Nominal Station	Location	Softened / Liquefiable Zone Thickness (ft)	Bottom Elevation of Softened/ Liquefiable Soils (ft)	Estimated Seismic Settlement (inches)	Estimated Lateral Spread Displacement (inches)**
End Bent 1 (also represents the West Causeway)					
3147+64	C2104S	3.4	296	1.2	2.4
3147+95	C4006	0.4	321	0.8	0.1
3147+95	B4013	3.0	316.7	1.7	7.0
3149+29	C4007	19	281.2	5	14
West Approach Spans					
3149+29	C4007	19	281.2	5	14
3150+64	C4008	7	278.8	2	2
3152+01	C4009	17	275.9	4	14
3153+48	C4010	14	278.5	4	19
3154+79	C4011	16	268.8	5	20
3156+25	C4012	15	312.1	6	35
3157+49	C4013	20	268.2	7	32
3158+88	C4014	9	267.9	3	5
3160+25	C4015	7	316.8	2	20
3161+62	C4016	16	310.9	7	59
3163+06	C4017	11	286.5	4	36
3164+44	C4018	9	315.9	3	47
Main Span Piers					
3164+44	C4018	9	293	2.8	48
3165+87	B4014	0	N/L*	0.2	0**
3165+83	B4015	19	292	11.6	102
3166+16	B4016	0	N/L	0	0
3166+01	C4020	6	262	1.9	15
3166+17	B4017	3	276	0.9	0**
3171+36	B4018	3	283	0.9	0**
3171+37	B4019	3	295	1.1	14
3171+66	B4020	0	N/L	0	0
3171+53	C4021	6	279	3.6	38
3171+66	B4021	9	280	2.0	14
3172+68	C4022	30	311	19.7	345
East Approach Spans					
3172+68	C4022	32	266.6	20.0	345
3173+84	C4023	5	267.4	1.8	18
3175+27	C4024	22	279.6	7.2	65
3176+67	C4025	25	279.7	7.0	62
3178+09	C4026	11	270.9	3.0	22
3179+31	C4027	14	270.9	3.0	20
3180+58	C4028	33	281.4	14.4	70
3181+95	C4029	13	287.8	3.2	15
3183+31	C4030	14	301.2	5.0	30
End Bent 2/Abutment 2					
3186+10	B4022	0	N/L	N/L	N/L
3186+00	B4023	0	N/L	N/L	N/L
3186+00	B4024	0	N/L	N/L	N/L

Nominal Station	Location	Softened / Liquefiable Zone Thickness (ft)	Bottom Elevation of Softened/ Liquefiable Soils (ft)	Estimated Seismic Settlement (inches)	Estimated Lateral Spread Displacement (inches)**
3186+01	B4025	0	N/L	N/L	N/L
3186+04	C4032	0	N/L	N/L	N/L

* N/L indicates no liquefiable zones were identified method discussion below

** liquefiable/softened zones are below river channel bottom elevation and will not experience lateral spread

Additional analyses were performed using yield accelerations back-calculated from pseudo-static slope stability analyses and the Newmark method of calculating seismic lateral displacements from the acceleration time histories at the base of the liquefied zone. The following table provides expected seismic lateral displacements that can be used to evaluate the kinematic loads. The Newmark lateral displacements are more rigorous and replace the lateral spread displacement estimates in the table above for the purposes of kinematic load analyses. The table below reports the results of the Newmark analyses.

Station Ranges	Base Elevation for Lateral Displacements (ft)	Seismic Lateral Displacements-Newmark Method (inches)
End Bent 1 (3H:1V)		
3147+95 (spill-through)	317	<0.1
3147+50 (sideslope)	348	<0.25
West Approach Spans		
3148+00 to 3164+50	304	<0.25
3164+50 to 3165+75	304	<1.0
Main Span Piers		
3165+75 to 3166+25	304	1.0
3171+25 to 3171+75	304	7.5
East Approach Spans		
3172+00 to 3173+00	304	1.5
3173+00 to 3173+50	304	1.0
3173+50 to 3174+00	304	0.5
3174+00 to 3181+50	304	<0.5
End Bent 2/Abutment 2		
3186+00	N/A	0

Residual strengths of liquefied zones were also estimated based on correlations in the literature and laboratory testing results. Stability analyses conducted using the residual shear strengths indicated that flow failures are not expected even if the soil does liquefy. These liquefiable layer thicknesses, seismic settlement, and lateral spreading estimates are considerably less than previous studies have suggested.

4.3 End Bent Settlement

Embankment fill is proposed at each of the end bent locations. The embankment fill is being placed at the beginning of the construction of the Lake Barkley Bridge. Construction of the embankment fill will result in settlement of the embankment foundation soils. We understand the construction sequence will be performed in this order: 1) construct embankment up to the

design finish elevation, 2) allow at least 90% of consolidation settlement to occur, 3) excavate to the bottom of pile cap elevation 4) drive end bent piles, 5) construct the end bent structure, and 6) construct the bridge superstructure.

At End Bent 1, between 10 to 33 feet of new embankment fill is proposed. The stress increase in the foundation soils due to the new embankment fill will result in settlement of the embankment foundation soils. For the existing soil conditions (no ground improvement), estimated settlement at the End Bent 1 location due to the anticipated embankment fill is on the order of 7 to 8.5 inches. It is estimated that approximately 90% of the anticipated settlement will occur in approximately 75 to 100 days following construction of the proposed embankment fill.

Approximately 10 feet of new embankment fill is proposed at End Bent 2. Significant foundation soil settlement is not anticipated due to the new embankment construction.

These estimates for End Bent 1 are based upon the estimated preconsolidation pressures, initial void ratio and recompression indices determined from one-dimensional consolidation test data from collected undisturbed soil samples collected in the soil borings. Where one-dimensional consolidation test data was not available, correlations between constraint modulus from the CPT sounding data, and compression indices and overconsolidation ratio estimates from the CPT data were used to estimate consolidation data in fine-grained soils. Settlement of granular soil layers were estimated using SPT N_{60} values and published relationships by Hough.

It is recommended at least 90% of the foundation soil settlement be allowed to occur prior to driving piles. This will allow the pile design to ignore the load effects of downdrag. In addition, allowing foundation soil settlement to occur will allow the use of battered piles without special considerations. As a method to confirm that the foundation soil settlement has slowed to an acceptable rate or stopped, we recommend that after the embankment is constructed at End Bent 1 that a minimum of two (2) telltales be installed through the embankment fill and the proposed rock embankment fill and be grouted into the foundation soil immediately underlying the rock embankment fill. Due to the proposed dumped rock embankment fill and the presence of the lake water, installation of settlement plates is not practical for the End Bent 1 location, nor are vibrating wire methods of monitoring settlement practical due to the required length of cable for a readout box location beyond the area impacted by the new causeway foundation soil settlement.

The telltales should be installed through the pile core fill exclusion zone (further details regarding Telltales are provided in **Section 4.8** of this report and on Exhibit D-4 in Appendix D). We recommend that telltale casing be 4 to 6 inches in diameter and be set 2 feet below the bottom of embankment elevation. A pilot hole should be drilled approximately 2 feet below the bottom of casing and a steel telltale rod with a grout tube attached would then be placed in the pilot hole. The bottom 2 feet would then be grouted into place. Centralizers should be placed around the telltale bars to prevent the bars from touching the sides of the casing. The bars could then be read at regular intervals for changes in elevation. Readings should be obtained beginning with the completion of embankment construction and the start of the bridge end bent construction. It should

be noted that the settlement of the foundation soils which occurs as the embankment is constructed will not get captured using this method. The telltales will provide data regarding the rate of foundation settlement, after the telltales have been installed. They also will not capture any vertical deformations above the telltale elevation.

One telltale should be installed near the centerline of the causeway and the second should be installed near the left edge of the proposed abutment location (left edge of pavement). A benchmark will need to be established that is not impacted by settlement of the embankment foundation soils, which may be on the mainland, on the adjacent causeway or the Henry Lawrence Bridge. Alternatively, pre-manufactured extensiometers could also be used instead of the described telltales, provided they are installed per the manufacturer's recommendations.

4.4 Lake Barkley Bridge Global Stability

Slope stability analyses were performed at seven (7) locations (Stations 3141+50, 3143+50, 3146+00 and 3147+50 for the embankment side slopes, through the spill-through abutment at End Bent 1, and the east and west banks of the existing Cumberland River) corresponding to the west causeway, End Bent 1 and submerged Cumberland River bank locations. Slope stability analyses were performed for both static and seismic conditions. The static condition analyses included short-term (construction), long-term (steady-state) conditions and several rapid-drawdown conditions.

4.4.1 West Causeway and End Bent 1 Global Stability

For the proposed west causeway sections as well as the spill-through abutment slope at Lake Barkley End Bent 1, the slope models included a bottom dredging limit of Elevation 346 feet. This dredging limit bottom elevation was provided to ICA by the structural design team. The proposed slopes at each section were evaluated for both static and seismic loading condition. The static loading condition evaluated includes short-term (construction), long-term (steady-state), and rapid-drawdown conditions. Seismic (pseudo-static) loading conditions were analyzed using the 1000-year seismic coefficients which ranged from 0.072g to 0.084g.

Seismic wedge analyses were also analyzed for the post-liquefaction case by restricting the wedge/block to slip through the liquefied layers. The residual strengths for the "liquefied" or softened" layers were estimated from results of CPT and/or SPT tests for granular layers or using 80% of the undrained shear strength values for cohesive layers.

Based on our analysis for the west causeway (left side), the proposed 2H:1V slopes are acceptable from the multi-use path down to the causeway toe from Station 3143+00 to the beginning of the bridge. Back of Station 3143+00 (left side), it is recommended the slopes from the multi-use path down to the causeway toe be placed on 2.5H:1V slopes. We also understand that 3H:1V slopes will be used from the multi-use path up to the roadway in the entire west causeway. In addition, 3H:1V slopes will be used for the End Bent 1 spill-

through abutment slope. The following table presents the complete list of slope configurations used in the west causeway side slope and End Bent 1 spill-through slope analyses.

Location		Proposed Slope	
		Left Side	Right Side
3141+50		3H:1V/2.5H:1V	3H:1V
3143+50		3H:1V/2H:1V	2H:1V
3146+00		3H:1V/2H:1V	3H:1V/2H:1V
3147+50		3H:1V/2H:1V	3H:1V/2H:1V
Lake Barkley End Bent 1	Slope A (from Top to Elev. 366 feet)	3H:1V	
	Slope B (from Elev. 363 feet to toe)		

The tables below summarize the results from the analyses performed based on static conditions and the 1000-year event for the seismic portion. Plots of the analyses are included in Appendix D.

Spill-Through Abutment Slope –Lake Barkley Bridge End Bent 1 (Slope A)	
Condition	**Factors of Safety**
Construction	1.8
Steady-State	1.8
Pseudo-Static Seismic (1000-year, k_s-0.072g)	1.5
Rapid Drawdown - Max Flood to Normal Pool (3 Stage)	1.8
Rapid Drawdown - Normal Pool to Loss of Lake (3 Stage)	1.8
Rapid Drawdown - Max Flood to Normal Pool (Simple Effective)	1.0
Rapid Drawdown - Normal Pool to Loss of Lake (Simple Effective)	1.6

Spill-Through Abutment Slope –Lake Barkley Bridge End Bent 1 (Slope B)	
Condition	**Factors of Safety**
Construction	2.1
Steady-State	2.1
Pseudo-Static Seismic (1000-year, k_s-0.072g)	1.4
Rapid Drawdown - Max Flood to Normal Pool (3 Stage)	2.1
Rapid Drawdown - Normal Pool to Loss of Lake (3 Stage)	2.3
Rapid Drawdown - Max Flood to Normal Pool (Simple Effective)	1.7
Rapid Drawdown - Normal Pool to Loss of Lake (Simple Effective)	1.6

Station 3141+50 (Left Side)	
Condition	**Factors of Safety**
Construction	1.6
Steady-State	1.6
Pseudo-Static Seismic (1000-year, k_s-0.084g)	1.2
Rapid Drawdown - Max Flood to Normal Pool (3 Stage)	1.5
Rapid Drawdown - Normal Pool to Loss of Lake (3 Stage)	1.5
Rapid Drawdown - Max Flood to Normal Pool (Simple Effective)	1.0
Rapid Drawdown - Normal Pool to Loss of Lake (Simple Effective)	1.1

Station 3141+50 (Right Side)	
Condition	**Factors of Safety**
Construction	1.3
Steady-State	1.9
Pseudo-Static Seismic (1000-year, k_s-0.084g)	1.0
Rapid Drawdown - Max Flood to Normal Pool (3 Stage)	1.8
Rapid Drawdown - Normal Pool to Loss of Lake (3 Stage)	1.8
Rapid Drawdown - Max Flood to Normal Pool (Simple Effective)	1.0
Rapid Drawdown - Normal Pool to Loss of Lake (Simple Effective)	1.2

Station 3143+50	
Condition	**Factors of Safety**
Construction	1.4
Steady-State	1.6
Pseudo-Static Seismic (1000-year, k_s=0.084g)	1.0
Rapid Drawdown - Max Flood to Normal Pool (3 Stage)	1.4
Rapid Drawdown - Normal Pool to Loss of Lake (3 Stage)	1.5
Rapid Drawdown - Max Flood to Normal Pool (Simple Effective)	1.0
Rapid Drawdown - Normal Pool to Loss of Lake (Simple Effective)	1.0

Station 3146+00 (Left Side)	
Condition	**Factors of Safety**
Construction	1.3
Steady-State	1.9
Pseudo-Static Seismic (1000-year, k_s=0.084g)	1.0
Rapid Drawdown - Max Flood to Normal Pool (3 Stage)	1.8
Rapid Drawdown - Normal Pool to Loss of Lake (3 Stage)	1.8
Rapid Drawdown - Max Flood to Normal Pool (Simple Effective)	1.0
Rapid Drawdown - Normal Pool to Loss of Lake (Simple Effective)	1.2

Station 3146+00 (Right Side)	
Condition	**Factors of Safety**
Construction	1.5
Steady-State	2.2
Pseudo-Static Seismic (1000-year, k_s=0.084g)	1.1
Rapid Drawdown - Max Flood to Normal Pool (3 Stage)	2.0
Rapid Drawdown - Normal Pool to Loss of Lake (3 Stage)	2.0
Rapid Drawdown - Max Flood to Normal Pool (Simple Effective)	1.5
Rapid Drawdown - Normal Pool to Loss of Lake (Simple Effective)	1.4

Station 3147+50 (Left Side)	
Condition	**Factors of Safety**
Construction	1.3
Steady-State	1.9
Pseudo-Static Seismic (1000-year, k_s=0.072g)	1.0
Rapid Drawdown - Max Flood to Normal Pool (3 Stage)	1.8
Rapid Drawdown - Normal Pool to Loss of Lake (3 Stage)	1.9
Rapid Drawdown - Max Flood to Normal Pool (Simple Effective)	1.0
Rapid Drawdown - Normal Pool to Loss of Lake (Simple Effective)	1.7

Station 3147+50 (Right Side)	
Condition	**Factors of Safety**
Construction	1.5
Steady-State	2.2
Pseudo-Static Seismic (1000-year, k_s=0.072g)	1.1
Rapid Drawdown - Max Flood to Normal Pool (3 Stage)	1.6
Rapid Drawdown - Normal Pool to Loss of Lake (3 Stage)	1.8
Rapid Drawdown - Max Flood to Normal Pool (Simple Effective)	1.7
Rapid Drawdown - Normal Pool to Loss of Lake (Simple Effective)	1.9

Seismic Wedge Analysis –Lake Barkley Bridge End Bent 1 (1000-Year)	
Condition	**Factors of Safety**
PL-1	2.3
PL-2	2.4
PL-3	2.3
PL-4	1.7
PL-5	1.3
PL-6	2.6
PL-7	3.3

Seismic Wedge Analysis – Station 3141+50 Side Slope (1000-Year)	
Condition	**Factors of Safety**
PL-1 (Right Side)	2.7
PL-2	1.5
PL-2 (Right Side)	2.6
PL-3	2.4
PL-4	2.5
PL-5	2.3
PL-6	2.5
PL-7	1.9

Seismic Wedge Analysis – Station 3143+50 Side Slope (1000-Year)	
Condition	**Factors of Safety**
PL-1	1.5
PL-2	1.4
PL-3	1.1
PL-4	1.6
PL-5	1.3

Seismic Wedge Analysis – Station 3146+00 Side Slope (1000-Year)	
Condition	**Factors of Safety**
PL-1	1.6
PL-2	1.4
PL-2 (Right Side)	1.5
PL-3	1.3
PL-3 (Right Side)	1.5
PL-4	1.6
PL-5	1.5
PL-6	1.7

Seismic Wedge Analysis – Station 3147+50 Side Slope (1000-Year)	
Condition	**Factors of Safety**
PL-1	1.5
PL-2	1.6
PL-3	1.4
PL-4	1.2
PL-4 (Right Side)	1.3
PL-5	1.2
PL-6	1.1

The factors of safety as presented above for the Lake Barkley Bridge End Bent 1 spill-through abutment slope as well as the west causeway side slopes were equal to or greater than the minimum required values. Ground modification is not required to satisfy static or seismic slope stability. However, if in the bridge design, the loads due to the lateral displacement of the upper soil profile cannot be resisted by the pile or drilled shaft group and/or the resulting foundation displacements do not meet the displacement performance objectives, some form of mitigation may still be necessary.

4.4.2 Existing Submerged Cumberland River Banks Global Stability

The existing slopes on both the west bank and the east bank of the Cumberland River near the centerline of the bridge alignment were evaluated for the static and seismic loading conditions. The proposed slopes at each section were evaluated for both static and seismic loading condition. The static loading condition evaluated includes long-term (steady-state), and rapid-drawdown conditions. Seismic (pseudo-static) loading conditions were analyzed using the 1000-year seismic coefficients which ranged from 0.098g for the wet banks to 0.143g for the east bank.

Seismic wedge analyses were also analyzed for the post-liquefaction case by restricting the wedge/block to slip through the liquefied layers. The residual strengths for the “liquefied” or softened” layers were estimated from results of CPT and/or SPT tests for granular layers or using 80% of the undrained shear strength values for cohesive layers.

The tables below summarize the results from the analyses performed based on static conditions and the 1000-year event for the seismic portion. Plots of the analyses are included in Appendix D.

Existing Submerged West Cumberland River Bank	
Condition	**Factors of Safety**
Steady-State	>4
Pseudo-Static Seismic (1000-year, k_s=0.098g)	2.3
Rapid Drawdown - Max Flood to Normal Pool (3 Stage)	>4
Rapid Drawdown - Normal Pool to Loss of Lake (3 Stage)	3.3
Rapid Drawdown - Normal Pool to Loss of Lake (Simple Effective)	2.7
Existing Submerged East Cumberland River Bank	
Condition	**Factors of Safety**
Steady-State	2.0
Pseudo-Static Seismic (1000-year, k_s=0.143g)	1.0
Rapid Drawdown - Max Flood to Normal Pool (3 Stage)	1.9
Rapid Drawdown - Normal Pool to Loss of Lake (3 Stage)	1.0
Rapid Drawdown - Normal Pool to Loss of Lake (Simple Effective)	1.0

Seismic Wedge Analysis – Existing West Cumberland River Bank (1000-Year)	
Condition	**Factors of Safety**
PL-1	>4
PL-2	>4
PL-3	>4
PL-4	>4
PL-5	>4
PL-6	>4
PL-7	>4
PL-8	3.0
PL-9	2.6
PL-10	2.5

Seismic Wedge Analysis – Existing East Cumberland River Bank (1000-Year)	
Condition	**Factors of Safety**
PL-1	>4
PL-2	2.7
PL-3	>4
PL-4	>4
PL-5	3.5
PL-6	2.8
PL-7	2.6
PL-8	1.0
PL-9	1.0
PL-10	1.0
PL-11	1.7

The factors of safety as presented above for the existing Cumberland River banks were equal to or greater than the minimum required values for all cases except for one rapid drawdown condition at the east bank. At the East Cumberland River bank, for the rapid drawdown loading condition from normal pool to loss of lake, the factor of safety was less than the minimum required of 1.1 for the 3-stage method. Using SIGMA/W, the estimated displacement at the toe of the east river bank due to this condition was calculated to be approximately 3 inches. This lateral displacement value is less than the anticipated lateral

spread displacement value, indicating the kinematic loading due to a rapid drawdown failure is less than the value due to lateral spread from a design seismic event. At this existing slope, there are zones within the soil profile where lateral displacement is expected to mobilize peak shear resistance in the slope.

Ground modification is not required to satisfy static or seismic slope stability. However, if in the bridge design, the loads due to the lateral displacement of the upper soil profile cannot be resisted by the pile or drilled shaft group and/or the resulting foundation displacements do not meet the displacement performance objectives, some form of mitigation may still be necessary.

4.5 Lake Barkley Bridge Pile Foundations – End Bent 1

Based upon Boring B4013 and potentially in the upper 5 feet of the rock core obtained from CPT sounding C4006, there is evidence of karst conditions within the limestone bedrock at End Bent 1. Due to the depth of bedrock and potential cost over runs the karst could cause during drilled shaft installation, the design team decided during a meeting at KenLake lodge held in March 2014 to use driven H-pile foundations driven to practical refusal on bedrock to support End Bent 1 of the Lake Barkley Bridge.

4.5.1 Static Axial Design Recommendations

At the bridge End Bent 1, end bearing H-piles on bedrock were recommended by the geotechnical team and selected by the design team over drilled shafts. Compatibility of the H-pile supported end bent with an adjacent pier supported on drilled shafts will need to be checked by the structural designer.

End bearing H-piles with reinforced pile points driven to the limestone bedrock can be considered. The top of bedrock elevation ranged from Elevation 263.3 to 271.1 feet at the test boring locations. Pile points are required to help break through potential limestone slabs above competent bedrock or broken bedrock at the bedrock surface, based upon the evidence of karst at the End Bent 1 location. The use of pile points for piles bearing on bedrock is standard practice on KYTC projects. Due to the high shear strength of the encountered bedrock, the nominal bearing resistance of the steel H-piles driven to bedrock would be based on the structural limit state per AASHTO Section 10.7.3.2.3. Due to karst conditions, the pile tips may be driven to refusal at varying elevations due to potential pinnacles and valleys in the bedrock surface. Other resistance factors may need to be applied to the nominal axial resistance for specific load cases. Due to the karst conditions at End Bent 1, we recommend increasing the number of piles by at least 33% beyond what is required by the design for additional redundancy. Minimum pile spacing should still meet the requirements established by AASHTO.

It is anticipated that the pile settlement would approach the elastic shortening of the piles due to shear strength of the bedrock and the stress-strain relationships observed during laboratory testing of the bedrock samples.

Embankment fill is proposed at the End Bent 1 location. Construction of the embankment fill will result in settlement of the embankment foundation soils. The embankment will be constructed during the initial construction of the bridge. We understand the construction sequence will be performed in this order: 1) construct embankment up to the design finish elevation, 2) allow at least 90% of consolidation settlement to occur, 3) excavate to the bottom of pile cap elevation 4) drive end bent piles, 5) construct the end bent structure, and 6) construct the bridge superstructure.

For the existing soil conditions (no ground improvement), estimated settlement at the End Bent 1 location due to the anticipated embankment fill is on the order of 7 to 8.5 inches. This will cause downdrag forces on the piles at End Bent 1. The neutral axis of the piles is located at approximately Elevation 280 feet, which is approximately 89 feet below the assumed bottom of pile cap elevation. The neutral axis was defined as the depth at which 1 inch or less of relative movement occurs between the piles and the surrounding soil, assuming the abutment piles would compress approximately 0.5 inches under axial load. It is estimated that approximately 90% of the anticipated settlement will occur in approximately 75 to 100 days following construction of the proposed embankment fill. The following table presents the estimated downdrag loads for the evaluated H-pile sections at End Bent 1. The alpha method (Tomlinson Method) was used for the fine-grained soil analyses and the recommended load factor is 1.4, based upon current AASHTO LRFD specifications.

Pile Size	Estimated Downdrag Load (kips)	Estimated Neutral Axis Elev. (feet)
HP14x89	520	280
HP14x117	585	280
HP18x157	750	280
HP18x204	825	280

Note: The downdrag loads only need to be considered for design if the piles are driven prior to completion of 90% of the foundation soil settlement. If 90% of the foundation soil settlement is not allowed to occur, the estimated downdrag load will need to be added to the axial load on the pile foundations.

It is our understanding that the estimated downdrag loads are almost equivalent to the structural resistance of the piles. As a result, the structural design team cannot include the downdrag load in the design of the End Bent 1 piles. It is recommended that if the construction schedule allows, at least 90% of the foundation soil settlement should be allowed to occur prior to H-pile installation. In addition, allowing foundation soil settlement to occur will allow the use of battered H-piles without special considerations. We recommend instrumentation monitoring to confirm that the foundation soil settlement has slowed to an acceptable rate or stopped. We recommend that tell tales be installed at the End Bent 1 location soon after the design embankment surface grades are achieved.

Wave equation analyses using the GRLWEAP software indicates the proposed H-pile sizes can be driven to the anticipated bearing depths, assuming the allowable compressive and tensile stresses are 90% of the steel yield stress for the H-piles. The analyses assume the yield strength is 50 ksi for the H-piles. Additionally, the wave equation analyses suggest that hammer blows in the final 15 to 20 feet may exceed 140 blows per foot for the hammers selected, which will increase the installation time of the piles. Wave equation analyses, performed for the selected hammer, indicate that a hammer with a rated energy of 71,700 ft.-lbs. or higher will be required to drive the H-piles to the refusal on bedrock and avoid overstressing the piles. Results of the GRLWEAP analyses are presented in Exhibit E-1.

Due to the variation in bedrock surface elevation, potential for pinnacles and valleys from karst conditions, and encountered chert pieces in the overburden soils, pile driving points are recommended at the tips of both the H-piles. Pile points for the H-Piles should be provided in accordance with Section 604.02.06 of the KYTC Standard Specifications.

4.5.2 Seismic Axial Design Recommendations

For end bearing H-piles supported on the limestone bedrock, the seismic axial resistance of the H-piles would be determined using AASHTO LRFD Bridge Design Specifications Section 6.5.4.2 and 6.5. Compatibility of the H-pile supported end bent with an adjacent pier supported on drilled shafts will need to be checked by the structural designer.

For the existing soil conditions, estimated seismic settlement at the CPT C4006 location near End Bent 1 due to the liquefaction/cyclic softening of soil is estimated at approximately 2 inches. This will cause downdrag forces on the H-piles at End Bent 1 during a design seismic event. The neutral axis is typically defined as the depth at which 0.5 inch or less of relative movement occurs between the drilled shaft and the surrounding soil. Soil above the neutral axis will result in downdrag loads on the piles. These calculated downdrag loads consider the thickness of liquefied soil and estimated residual soil shear strengths of the liquefied soils and consider grout to soil interaction parameters. The following table outlines the estimated unfactored downdrag loads for each H-pile size at the selected representative location.

H-Pile Size	Unfactored Seismic Downdrag Load Per Pile (kips)
HP14x89	255
HP14x117	260
HP18x157	305
HP18x204	310

4.6 Lake Barkley Bridge Drilled Shaft Foundations – Piers and End Bent 2

Due to the larger lateral and axial loads at the piers and the relatively shallow depth to bedrock at the pier locations, drilled shaft foundations socketed into bedrock are recommended at the bridge

pier locations. Foundation recommendations have been developed for support of the proposed West Approach Spans from approximately Station 3148+00 to 3165+00 (Piers 1 to 6), the East Approach Spans from Approximately Station 3172+00 to 3185+00 (Piers 9 to 13), the west Main Span Pier at Station 3166+01 (Pier 7) and the east Main Span Pier at Station 3171+51 (Pier 8). At End Bent 2, we understand that the structural design team prefers drilled shaft foundations socketed into bedrock over shallow footings bearing on bedrock to avoid a 20 to 25 feet high foundation wall.

According to the project plans, the approach span drilled shafts will have a diameter of 84-inches within the soil strata and a rock socket diameter of 78-inches. The plans also indicate that the drilled shafts supporting the main span piers will have a diameter of 72-inches in the soil strata with a rock socket diameter of 66-inches. The proposed End Bent 2 drilled shafts will have a diameter of 48-inches within the soil strata and a rock socket diameter of 42-inches.

The drilled shaft design approach is based upon the methods outlined in the LRFD Drilled Shaft Manual (FHWA-NHI-10-016) and the Sixth Edition of the AASHTO LRFD Bridge Design Manual. For construction purposes and per the direction of the Department, the analyses assume that a permanent steel casing will be extended from the bottom of the pier cap to some distance (1 to 2 feet) into the encountered limestone bedrock to seal the rock socket from caving soils. It is our understanding that the structural design team is considering the permanent steel casing in the lateral design of the drilled shafts.

Steel casing will need to be installed to depths of between 10 to 110 feet below the existing mudline or ground surface elevation in order to seal the casing into the limestone bedrock. Potential methods of installing the casing include vibrating the casing to the top of bedrock and then either sealing the casing by rotating the casing into bedrock or possibly excavating the bedrock and then sealing the casing using concrete. The concrete and the underlying bedrock could then be excavated using smaller diameter drilling tools. Another method to advance and seal the casing into the bedrock may be to use rotator/oscillator equipment. At a nearby site on the Tennessee River, temporary casing was advanced to the bedrock and then a permanent casing was installed within the temporary casing, which allowed the permanent casing to be spun into the bedrock to seal the casing. Due to the depth of the encountered soil at the site and the hard encountered bedrock at the site, installing and sealing the required steel permanent casing may prove difficult. Since the foundation design includes lateral resistance of the drilled shafts within the soil strata, any annulus created between temporary steel casing and permanent steel casing will need to be filled with grout or other approved materials to ensure contact between the permanent casing and the soil.

Design parameters for both axial end bearing and side resistance for rock-socketed drilled shafts are provided. Shafts will also need to be evaluated for lateral resistance which may control rock socket embedment depths. Strain compatibility when using side and end bearing would need to be evaluated as well as group settlement, as part of final design when the drill shaft geometry and layout are finalized. Drilled shaft performance is strongly related to the effectiveness of the

construction technique in preserving the integrity of the bearing materials and ensuring the structural integrity of the reinforced concrete shaft element.

The drilled shaft design parameters for axial loading were developed based on the test borings, detailed review of rock cores, laboratory testing, and review of published literature. Design of the drilled shafts can include both base resistance and side resistance in the bedrock. An estimate of the total scour should be performed to determine what side resistance is available from the overburden soils. We have ignored the side resistance in overburden soils for axial compressive resistance computations. The load-displacement relationship (strain compatibility) between base and side resistance should be considered in the design since the maximum side resistance typically occurs at a much lower displacement than the maximum base resistance.

Reasons cited in published literature for neglecting side resistance of rock sockets include; (1) possibility of strain-softening behavior of the sidewall interface (2) possibility of degradation of material in the borehole wall in argillaceous rock, (3) uncertainty regarding the roughness of the sidewall. Site-specific laboratory testing has not been performed to determine load-deformation behavior on the rock/concrete interface. Based on published literature on similar bedrock material as those encountered for this project, strain softening is not commonly observed and therefore strain compatibility should not be a factor in combining side resistance and base resistance. This tendency is likely related to the dilatancy of the shaft/rock interface. Field load testing along with careful quality control during construction to confirm sidewall conditions should be performed to confirm and justify our assumption that side resistance can be used in combination with base resistance. Laboratory testing can also be performed in addition to field testing if strain softening is a concern.

The design guidelines for geotechnical strength are based on limiting the displacement at nominal resistance to 2% of shaft diameter, considering that larger diameter shafts (6 feet or greater diameter) will be used. We have limited the nominal base resistance to 375 ksf to satisfy the above-discussed criterion, for the limestone bedrock at depths lower than 10 feet below the top of bedrock. At Piers 1, 2, 6, 9, 10, 11, 12 and 13, the recommended minimum rock socket below the top of the encountered bedrock is the greater of 1.5B (B=drilled shaft socket diameter) or 12 feet, whichever is greater. At Piers 7 and 8, the recommended minimum rock socket below the top of the encountered bedrock is the greater of 1.5B (B=drilled shaft socket diameter) or 10 feet, whichever is greater. At Piers 3, 4 and 5 and End Bent 2 where voids in the bedrock are anticipated, the recommended minimum rock socket below any casing used to seal voids is the greater of 1.5B (B=drilled shaft socket diameter) or 12 feet below the casing used to seal any voids (10 feet at End Bent 2). At Piers 3, 4 and 5 and End Bent 2, if the contractor elects to pump concrete to seal any voids and then redrill the rock socket, the rock socket should extend to the criteria recommended for Piers 1, 2 and 6 or at least 3 feet below the last void where concrete was pumped, whichever is deeper. The recommended value also considers disturbance and construction considerations which has a significant impact on design performance of drilled shafts. Also, for nominal resistance greater than 375 ksf, creep movements are likely due to the presence of calcite-filled joints and potentially clay-filled seams if karst conditions exist. The limiting of base

resistance also appears reasonable considering the need to ensure strain compatibility between side resistance and end bearing and limiting the overall foundation movement to less than 0.5 inch for the service limit state and less than 2% of shaft diameter for the strength limit state. Project-specific load testing should be performed to help determine load displacement data and modify design values, as needed.

Based on our understanding from the structural design team, all the drilled shaft rock socket lengths in the October 3, 2014 drawings were controlled by the minimum rock socket length (10 feet at the main span piers and End Bent 2 and 12 feet at the approach spans) and not design axial load. As a result, load testing of the drilled shafts is not recommended. However, a technique drilled shaft (used to demonstrate construction methods) is recommended in the east approach area at Station 3180+00 at the project centerline, and a second technique drilled shaft is recommended in the west approach area at approximately Station 3160+79 at the project centerline. No drilled shafts should be installed on the project until the technique drilled shaft at Station 3180+00 is installed successfully and accepted by KYTC. No drilled shafts in Piers 1 to 6 should be installed until the technique drilled shaft at Station 3160+79 is successfully installed and accepted by KYTC.

The rock socket should be designed having a diameter that is 6 inches less than the diameter in the overburden soil. The reinforcing steel should have a minimum cover of 6 inches of concrete within the rock socket.

4.6.1 Static Axial Design Recommendations – West Approach Spans

Foundation recommendations have been developed for support of the proposed West Approach Spans from approximately Station 3148+00 to 3165+00 (Piers 1 to 6). Based on the design drawings, it is proposed for the drilled shafts to have an 84-inch diameter from the pier cap to the top of bedrock and then have a 78-inch diameter rock socket.

At Bridge Piers 3, 4 and 5, several test boring performed at or near the proposed drilled shaft locations encountered voids in the bedrock that were either air-, clay- or rubble-filled, due to karst conditions. The following table outlines the borings were voids were encountered, the elevation of the lowest encountered void and the bridge pier impacted by the encountered voids. Sonic caliper testing should be performed on all the completed rock sockets to confirm the location of any voids or horizontal solution cavities in the bedrock, prior to placing any steel casing across any voids or concrete within the encountered voids. Please note that Test Borings 13A, 5014W, 5014B, 5014A, 5014B and 5015 are not located at or adjacent to proposed drilled shafts, since Bridge Pier 5 was relocated 20 feet east of its original position to its current position, based upon the karst conditions encountered in Borings 5014B, 5014A, 5014, and 5015 and since steel drill tools were lost within Boring 5014 during the subsurface exploration.

Boring	Top Elevation of Highest Encountered Void (feet)	Elevation of Lowest Encountered Void (feet)	Bridge Pier Associated with Boring
5009	243.5	226.0	3
5011	249.3	237.1	4
13A	254.0	250.2	4
5014W	237.3	211.5	5
5014B	234.6	214.9	5
5014A	235.4	196.4	5
5014	234.7	210.4	5
5015	244.7	235.4	5
5014E	224.2	208.5	5

Where borings indicate voids (clay or air filled) greater than 1-foot-thick, permanent steel casing will likely need to be placed across the void to limit loss of concrete in the shaft excavation. The shaft side resistance should be neglected down to at least the bottom of the steel casing. The casing will be embedded some depth into the limestone below the bottom of the lowest void and the bottom of the void will likely be irregular, thus the top of the rock socket elevation has been assumed 3 feet below the bottom of the lowest encountered void at locations where voids were encountered in the borings. For estimating purposes, we recommend that all top of rock socket elevations be lowered at bridge piers where one of the borings encountered voids.

Where the borings indicate voids that are 1-foot-thick or less, the voids can likely be filled with concrete or grout by filling the excavated rock socket with grout/concrete to a height of at least 1.5 feet above the void. Once the concrete sets, the concrete can be drilled out and the rock socket extended to the design tip elevation. The rock socket can then be designed using the recommended side resistance values, provided that the rock socket extends at least 3 feet below the void to be below any weathered bedrock for end bearing resistance (this is the case at Pier 4, i.e. bottom of lowest void is Elevation 237.1 feet, so the design drilled shaft tip elevation should be Elevation=234.0 feet).

For the limestone bedrock 3 feet and greater below the top of bedrock or below the bottom of the lowest encountered void, the recommended nominal side resistance is 20 ksf. For the limestone bedrock 12 feet and greater below the top of bedrock the nominal end bearing resistance is 375 ksf. These values assume a minimum 28-day concrete compressive strength of 3,500 psi, since the side resistance is limited by the concrete shear strength, not the bedrock strength.

Per AASHTO Table 10.5.5.2.4-1, the resistance factor for axial compression in bedrock is 0.5 and 0.4 for uplift in bedrock. If applied to a single shaft supporting a bridge pier, then the resistance factors should be reduced by 20 percent (per AASHTO Section 10.5.5.2.4). A Drilled Shaft Resistance Table (Exhibit E-2 in Appendix E) is provided for design drilled shaft rock socket diameter of 6.5 feet for the approach span drilled shaft foundations in Piers 1 to 6. We have attached Exhibit E-3 that includes the encountered mudline elevation, encountered rock elevation, bottom of any encountered weathered bedrock, bottom elevation of the lowest encountered void and the recommended top of rock socket elevation the west approach spans. The zero depth in

Exhibit E-2 corresponds to the top of rock socket elevation provided in Exhibit E-3 for the west approach span drilled shafts, except at Pier 4 where the recommended highest drilled shaft tip elevation is Elevation 234.0 feet.

4.6.2 Static Axial Design Recommendations – East Approach Spans

Static drilled shaft resistance values for the bedrock socket portions of the drilled shafts were developed for the Lake Barkley East Approach Span Pier locations between Stations 3172+00 to 3185+00 (Piers 9 to 13). Based on the design drawings, it is proposed for the drilled shafts to have an 84-inch diameter from the pier cap to the top of bedrock and then have a 78-inch diameter rock socket.

For the limestone bedrock 3 feet and greater below the top of bedrock, the recommended nominal side resistance is 20 ksf. For the limestone bedrock 12 feet and greater below the top of bedrock the nominal end bearing resistance is 375 ksf. These values assume a minimum 28-day concrete compressive strength of 3,500 psi, since the side resistance is limited by the concrete shear strength, not the bedrock strength.

Per AASHTO Table 10.5.5.2.4-1, the resistance factor for axial compression in bedrock is 0.5 and 0.4 for uplift in bedrock. If applied to a single shaft supporting a bridge pier, then the resistance factors should be reduced by 20 percent (per AASHTO Section 10.5.5.2.4). A Drilled Shaft Resistance Table (Exhibit E-2 in Appendix E) is provided for design drilled shaft rock socket diameter of 6.5 feet for the approach span drilled shaft foundations in Piers 9 to 13 (Exhibit E-2 applies to both the west and east approach spans). Exhibit E-3 is located in Appendix E, which includes the encountered mudline elevation, encountered rock elevation, bottom of any encountered weathered bedrock, bottom elevation of the lowest encountered void and the recommended top of rock socket elevation the east approach spans. The zero depth in the Exhibit E-2 corresponds to the top of rock socket elevation provided in Exhibit E-3 for the east approach span drilled shafts.

4.6.3 Static Axial Design Recommendations – Main Span Piers

Static drilled shaft resistance values for the bedrock socket portions of the drilled shafts have been developed for the Lake Barkley Main Span Pier locations. The West and East Main Span Piers (Piers 7 and 8) are approximately centered at Lake Barkley Stations 3166+01 and 3171+51, respectively, based upon current information. Based on the design drawings, it is proposed for the drilled shafts to have a 72-inch diameter from the pier cap to the top of bedrock and then have a 66-inch diameter rock socket. Boring 5027 encountered a 0.9-foot thick void beginning approximately 1.3 feet below the top of the encountered bedrock at this location. No evidence of karst conditions were encountered within the remaining borings performed to date at the Lake Barkley Main Span pier locations.

For the limestone bedrock, the recommended nominal side resistance is 21 ksf and the nominal end bearing resistance is 375 ksf. These values assume a minimum 28-day concrete compressive

strength of 3,500 psi, since the side resistance is limited by the concrete shear strength, not the bedrock strength.

Per AASHTO Table 10.5.5.2.4-1, resistance factors in bedrock are 0.5 for axial compression and 0.4 for uplift. A Drilled Shaft Resistance Table (Exhibit E-4) is provided for design drilled shaft rock socket diameter of 5.5 feet for the main span drilled shaft foundations. Exhibit E-5 in Appendix E includes the encountered mudline elevation, encountered rock elevation, bottom of any encountered weathered bedrock, bottom elevation of the lowest encountered void and the recommended top of rock socket elevation for both Piers 7 and 8 of the Main Span. The zero depth in Exhibit E-4 corresponds to the top of rock socket elevation provided in Exhibit E-5. At Pier 7 were the void was encountered just below the top of bedrock elevation, this was treated as weathered rock for the purpose of Exhibit E-5, since it coincides with the top of rock with the other Pier 7 drilled shaft locations.

4.6.4 Static Axial Design Recommendations – End Bent 2

Static drilled shaft resistance values for the bedrock socket portions of the drilled shafts have been developed for the Lake Barkley End Bent 2 location. The End Bent 2 centerline is located at Station 3186+00, based upon current information. Based on the design drawings, it is proposed for the drilled shafts to have a 48-inch diameter from the pier cap to the top of bedrock and then have a 42-inch diameter rock socket.

The top of bedrock elevation varied from approximately Elevation 381.5 to 359.1 feet. The variation in top of bedrock elevation could be an indication of pinnacles and valleys at the top of bedrock surface, which would indicate karst conditions. At Borings 5066 and 5066A, which were drilled approximately 18 inches from one another in plan dimension, encountered bedrock at Elevations 362.3 feet and 381.4 feet, respectively, which indicates that abrupt changes in top of bedrock elevation within a relatively short lateral distance. In addition, voids/cavities were encountered at four of the test borings drilled at or near the proposed End Bent 2 location (Borings 5055, 5056, 5060, and 5069). The top of the encountered voids/cavities were encountered between 2.4 to 5.4 feet below the top of encountered bedrock. The bottom of the lowest void or cavity in these borings was encountered between 3.4 to 8.3 feet below the top of the encountered bedrock elevation. The thickness of the encountered voids/cavities ranged from 0.5 to 5.9 feet thick.

At Borings B5055, B056, and B5060 (Drilled Shafts 55, 56 and 60) where less than 1-foot-thick voids were encountered in the upper 6 feet below the top of encountered bedrock, the voids can likely be filled with concrete or grout by filling the excavated rock socket with grout/concrete to a height of at least 1.5 feet above the void. Once the concrete sets, the concrete can be drilled out and the rock socket extended to the design tip elevation.
Boring B5069 located at Drilled Shaft 69 indicated voids (clay or air filled) up to about 3.5-feet-thick in the upper 8.3 feet of the encountered bedrock, and permanent steel casing will likely need to be placed across these voids to limit loss of concrete in the shaft excavation. The shaft side resistance should be neglected down to at least the bottom of the steel casing. The casing will be

embedded some depth into the limestone below the bottom of the lowest void and the bottom of the void will likely be irregular, thus the top of the rock socket elevation has been assumed 3 feet below the bottom of the lowest encountered void at locations where voids were encountered in the borings.

For the limestone bedrock, the recommended nominal side resistance is 21 ksf and the nominal end bearing resistance is 375 ksf. These values assume a minimum 28-day concrete compressive strength of 3,500 psi, since the side resistance is limited by the concrete shear strength, not the bedrock strength.

Per AASHTO Table 10.5.5.2.4-1, resistance factors in bedrock are 0.5 for axial compression and 0.4 for uplift. A Drilled Shaft Resistance Table (Exhibit E-6) is presented in Appendix E for a design drilled shaft rock socket diameter of 3.5 feet for the End Bent 2 drilled shaft foundations. The zero depth in Exhibit E-6 corresponds to the top of rock socket elevation provided in Exhibit E-7. Exhibit E-7 includes the encountered mudline elevation, encountered rock elevation, bottom of any encountered weathered bedrock, bottom elevation of the lowest encountered void and the recommended top of rock socket elevation for the End Bent 2 drilled shafts.

4.6.5 Seismic Axial Design Recommendations – West Approach Spans

Seismic foundation recommendations have been developed for support of the proposed West Approach Spans (Piers 1 to 6) from approximately Station 3148+00 to 3165+00 based on a 1000-year design seismic event. The seismic axial resistance in soils and lateral soil response parameters for non-liquefied soils consider 90% of the static undrained shear strength values for fine-grained soils and internal angle of friction for granular soils [$0.9S_u$ or $\tan^{-1}(0.9\tan\Phi)$] as compared to the static soil parameters estimated for the Approach Spans. Seismic skin resistance values along the portion of the drilled shaft within the soil above bedrock were estimated using methods outlined in the 2012 AASHTO LRFD Bridge Design Specification. The unit side resistance values for seismic conditions within the cohesionless soil portion of the soil profile was reduced to 90% of the static values to be consistent with the shear strength loss of granular soils observed in the cyclic shear test results.

Design parameters for both axial end bearing and side resistance for rock-socketed drilled shafts under seismic loading conditions are provided in tabular form in attached Exhibit E-8. The zero depth in Exhibit E-8 corresponds to the top of rock socket elevation provided in Exhibit E-3 for the west approach span drilled shafts, except at Pier 4 where the recommended highest drilled shaft tip elevation is Elevation 234.0 feet. Per AASHTO Section 10.5.5.3.3, seismic resistance factors in bedrock are 1.0 for axial compression and 0.8 for uplift. The drilled shaft design approach is based upon the methods outlined in the LRFD Drilled Shaft Manual (FHWA-NHI-10-016) and the Sixth Edition of the AASHTO LRFD Bridge Design Manual.

For analyses assumptions related to construction of drilled shafts, it should be assumed that a permanent steel casing will be required through the soil and embedded some distance into the underlying limestone bedrock to seal the rock socket from caving soils. The permanent casing can

be considered in the structural resistance of the drilled shaft, provided that a sacrificial portion of the casing thickness is neglected due to the potential for corrosion.

For the existing soil conditions at the West Approach Spans, estimated seismic settlement at the existing CPT soundings and test boring locations range from 2 to 7 inches. At Pier 1, the soil layers with liquefaction potential which can result in settlement exceeding 0.5 inches during a seismic event extending to Elevation 275.9 feet. At Piers 2 and 3, the soil layers with liquefaction potential which can result in settlement exceeding 0.5 inches during a seismic event extend to Elevation 268.2. At Piers 4 to 6, the soil layers with liquefaction potential which can result in settlement exceeding 0.5 inches during a seismic event extend to Elevation 267.9 feet.

For the existing soil conditions at the West Approach Spans, estimated seismic settlement at the existing CPT soundings and test boring locations range from about 2 to 7 inches. At Pier 1, the soil layers with liquefaction potential, which extend to a bottom elevation of about Elevation 275.9 feet, can result in settlement exceeding about 0.5 inches during a seismic event. At Piers 2 to 4 and Piers 5 to 6, the soil layers with liquefaction potential which can result in settlement exceeding 0.5 inches during a seismic event extend to between Elevations 268.2 and 267.9 feet, respectively. Due to presence of deeper soil layers with liquefaction potential, the magnitude of the estimated downdrag loads due to liquefaction/soil softening settlement are higher at Piers 2 to 6 than the value for Pier 1.

The estimated settlement will cause downdrag forces on the drilled shafts at the West Approach Spans during a design seismic event. The neutral axis is typically defined as the depth at which 0.5 inch or less of relative movement occurs between the drilled shaft and the surrounding soil. Soil above the neutral axis will result in downdrag loads on the shafts. The neutral axis elevations are estimated at approximately Elevation 276 feet at Pier 1 and approximately Elevation 268 feet at Piers 2 through 6. These calculated downdrag loads consider the thickness of liquefied soil and estimated residual soil shear strengths of the liquefied soils and consider steel casing to soil interaction parameters. Lower values could be considered if the final construction method results in a double permanent steel casing situation where a void exists between the inner and outer casings.

The following table outlines the estimated unfactored downdrag loads per drilled shaft for each shaft diameter at the selected representative locations for the West Approach Span Piers. Per AASHTO Table 3.4.1-2, load factors of 1.25 should be applied to the unfactored downdrag loads for both cohesive and granular soils.

Bridge Pier Number	Drilled Shaft Diameter (feet)	Unfactored Downdrag Load Per Drilled Shaft (kips)
1	7	820
2 to 4	7	1390
5 and 6	7	1350

4.6.6 Seismic Axial Design Recommendations – East Approach Spans

Seismic drilled shaft resistance values for the bedrock socket portions of the drilled shafts were developed for the Lake Barkley East Approach Span Pier locations (Piers 9 to 13) between Stations 3172+00 to 3185+00 for the design 1000-year seismic event. The seismic axial resistance in soils and lateral soil response parameters for non-liquefied soils consider 90% of the static undrained shear strength values for fine-grained soils and internal angle of friction for granular soils [0.9Su or tan-1(0.9tanΦ)] as compared to the static soil parameters estimated for the Approach Spans. These parameters were selected by considering published literature and the cyclic shear tests performed on selected soil samples. Seismic skin resistance values along the portion of the drilled shaft within the soil above bedrock were estimated using methods outlined in the 2012 AASHTO LRFD Bridge Design Specification. The unit side resistance values for seismic conditions within the cohesionless soil portion of the soil profile was reduced to 90% of the static values to be consistent with the shear strength loss of granular soils observed in the cyclic shear test results.

Design parameters for both axial end bearing and side resistance for rock-socketed drilled shafts under seismic loading conditions at Piers 9 to 13 are provided in in Exhibit E-8 in Appendix E. The zero depth in Exhibit E-8 corresponds to the top of rock socket elevation provided in Exhibit E-3 for the west approach span drilled shafts. Per AASHTO Section 10.5.5.3.3, seismic resistance factors in bedrock are 1.0 for axial compression and 0.8 for uplift. The drilled shaft design approach is based upon the methods outlined in the LRFD Drilled Shaft Manual (FHWA-NHI-10-016) and the Sixth Edition of the AASHTO LRFD Bridge Design Manual.

For analyses assumptions related to construction of drilled shafts, it should be assumed that a permanent steel casing will be required through the soil and embedded some distance into the underlying limestone bedrock to seal the rock socket from caving soils. The permanent casing can be considered in the structural resistance of the drilled shaft, provided that a sacrificial portion of the casing thickness is neglected due to the potential for corrosion.

For the existing soil conditions at the East Approach Spans, estimated seismic settlement at the existing CPT soundings and test boring locations range from about 0.5 to 20 inches. At Piers 9 and 10, the soil layers with liquefaction potential, which extend to a bottom elevation of about Elevation 266.6 feet, can result in settlement up to about 20 inches during a seismic event. At Pier 11, the soil layers with liquefaction potential, which extend to a bottom elevation of about Elevation 281.4 feet, can result in settlement exceeding about 14 inches during a seismic event. At Piers 13 and 14, the soil layers with liquefaction potential, which extend to bottom elevations of about Elevation 301.2 and Elevation 332.8 feet, respectively, can result in settlement exceeding about 0.5 inches during a seismic event. Due to presence of deeper soil layers with liquefaction potential, the magnitude of the estimated downdrag loads due to liquefaction/soil softening settlement are much higher Piers 9 and 10 than the values at Piers 11 to 13.

The estimated settlement at these locations will cause downdrag forces on the drilled shafts at the East Approach Spans during a design seismic event. The neutral axis is typically defined as the depth at which 0.4 to 0.5 inches or less of relative movement occurs between the drilled shaft and the surrounding soil. Soil above the neutral axis will result in downdrag loads on the shafts. The neutral axis elevations are estimated at approximately Elevation 267 feet at Piers 9 and 10, approximately Elevation 297 feet at Pier 11, approximately Elevation 301.5 feet at Pier 12 and approximately Elevation 336 feet at Pier 13. Please note that the lowermost liquefiable layer in CPT Sounding C4031 was moved up in elevation to just above the top of bedrock elevation, since the selected soil/bedrock model had a top of bedrock elevation just above the lowermost liquefiable layer in CPT Sounding C4031. These calculated downdrag loads consider the thickness of liquefied soil and estimated residual soil shear strengths of the liquefied soils and consider steel casing to soil interaction parameters. Lower values could be considered if the final construction method results in a double permanent steel casing situation where a void exists between the inner and outer casings.

The following table outlines the estimated unfactored downdrag loads per drilled shaft for each shaft diameter at the selected representative locations for the East Approach Span Piers, respectively. Per AASHTO Table 3.4.1-2, load factors of 1.25 should be applied to the unfactored downdrag loads for both cohesive and granular soils.

Bridge Pier Number	Drilled Shaft Diameter (feet)	Unfactored Downdrag Load Per Drilled Shaft (kips)
9 and 10	7	990
11	7	320
12	7	300
13	7	180

4.6.7 Seismic Axial Design Recommendations – Main Span Piers

Seismic foundation recommendations have been developed for support of the proposed Main Span Piers located at approximate Stations 3166+01 (Pier 7) and 3171+51 (Pier 8) based on a 1000-year design seismic event. The seismic axial resistance in soils and lateral soil response parameters for non-liquefied soils consider 90% of the static undrained shear strength values for fine-grained soils and internal angle of friction for granular soils [$0.9S_u$ or $\tan^{-1}(0.9\tan\Phi)$] as compared to the static soil parameters estimated for the Approach Spans. Seismic skin resistance values along the portion of the drilled shaft within the soil above bedrock were estimated using methods outlined in the 2012 AASHTO LRFD Bridge Design Specification. The unit side resistance values for seismic conditions within the cohesionless soil portion of the soil profile was reduced to 90% of the static values to be consistent with the shear strength loss of granular soils observed in the cyclic shear test results.

At the West Main Span Pier (Pier 7), seismic evaluation results from the SPT Boring B4015 (which included CPT sounding intervals, as well) was used in our analyses. Currently, seismic analyses and evaluation indicate that the potential liquefiable soil zone thickness is 19 feet with a bottom elevation of liquefiable soil at Elevation 292 feet in B4015. At the East Main Span Pier (Pier 8), seismic evaluation results from the SPT Boring B4021 (which did not include CPT sounding intervals) with data from CPT Sounding C4021 were used in our analyses. Seismic analyses and evaluation indicate that the potential liquefiable soil zone thickness is 9 feet with a bottom elevation of liquefiable soil at Elevation 280 feet in B4021.

The seismic soil shear strength parameters for non-liquefied soils consider 90% of the static undrained shear strength values for fine-grained soils and internal angle of friction for granular soils [0.9Su or tan-1(0.9tanΦ)] as compared to the static soil parameters estimated for the Main Span Piers. These parameters were selected by considering published literature and the cyclic shear tests performed on selected soil samples. Seismic skin resistance values along the portion of the drilled shaft within the soil above bedrock were estimated using methods outlined in the 2012 AASHTO LRFD Bridge Design Specification. The unit side resistance values for seismic conditions within the cohesionless soil portion of the soil profile was reduced to 90% of the static values to be consistent with the shear strength loss of granular soils observed in the cyclic shear test results.

Design parameters for both axial end bearing and side resistance for rock-socketed drilled shafts under seismic loading conditions are provided in tabular form in Exhibit E-9 in Appendix E. Per AASHTO Section 10.5.5.3.3, seismic resistance factors in bedrock are 1.0 for axial compression and 0.8 for uplift. The drilled shaft design approach is based upon the methods outlined in the LRFD Drilled Shaft Manual (FHWA-NHI-10-016) and the sixth Edition of the AASHTO LRFD Bridge Design Manual.

For analyses assumptions related to construction of drilled shafts, it should be assumed that a permanent steel casing will be required through the soil and embedded some distance into the underlying limestone bedrock to seal the rock socket from caving soils. The permanent casing can be considered in the structural resistance of the drilled shaft, provided that a sacrificial portion of the casing thickness is neglected due to the potential for corrosion.

For the existing soil conditions at Pier 7, estimated seismic settlement at the CPT/SPT B4015 location due to the liquefaction/cyclic softening of soil is estimated at approximately 11.6 inches. For the existing soil conditions at Pier 8, estimated seismic settlement at the CPT C4021 location due to the liquefaction/cyclic softening of soil is estimated at approximately 3.6 inches. The estimated settlement at these locations will cause downdrag forces on the drilled shaft at the approach spans during a design seismic event. The neutral axis is typically defined as the depth at which 0.5 inch or less of relative movement occurs between the drilled shaft and the surrounding soil. Soil above the neutral axis will result in downdrag loads on the piles. These calculated downdrag loads consider the thickness of liquefied soil and estimated residual soil shear strengths of the liquefied soils and consider grout to soil interaction parameters. The following table outlines

the estimated unfactored downdrag loads per drilled shaft for each shaft diameter at the selected representative locations for Piers 7 and 8, respectively.

Bridge Pier Number	Drilled Shaft Diameter (feet)	Unfactored Downdrag Load Per Drilled Shaft (kips)
7	6	260
8	6	550

4.6.8 Seismic Axial Design Recommendations – End Bent 2

Seismic foundation recommendations have been developed for support of the proposed End Bent 2 located at approximate Stations 3186+00 based on a 1000-year design seismic event. The seismic axial resistance in soils and lateral soil response parameters for non-liquefied soils consider 90% of the static undrained shear strength values for fine-grained soils and internal angle of friction for granular soils [$0.9S_u$ or $\tan^{-1}(0.9\tan\Phi)$] as compared to the static soil parameters estimated for the Approach Spans. Seismic skin resistance values along the portion of the drilled shaft within the soil above bedrock were estimated using methods outlined in the 2012 AASHTO LRFD Bridge Design Specification. The unit side resistance values for seismic conditions within the cohesionless soil portion of the soil profile was reduced to 90% of the static values to be consistent with the shear strength loss of granular soils observed in the cyclic shear test results.

Design parameters for both axial end bearing and side resistance for rock-socketed drilled shafts under seismic loading conditions are provided in tabular form in Exhibit E-10 in Appendix E. Per AASHTO Section 10.5.5.3.3, seismic resistance factors in bedrock are 1.0 for axial compression and 0.8 for uplift. The drilled shaft design approach is based upon the methods outlined in the LRFD Drilled Shaft Manual (FHWA-NHI-10-016) and the sixth Edition of the AASHTO LRFD Bridge Design Manual.

No liquefaction potential is anticipated within the overburden soils at End Bent 2, based upon our liquefaction analyses. As a result, no downdrag due to liquefaction settlement is anticipated for the drilled shafts at End Bent 2.

4.6.9 Additional Construction Considerations

Several of the test borings performed near the proposed drilled shafts for Bridge Piers 3, 4 and 5 and at End Bent 2, encountered voids/cavities within the limestone bedrock. Evidence of cavities/voids were also encountered in borings performed near End Bent 1, as well as potentially a minor void near the encountered bedrock surface in one of the borings performed at Pier 7. Additional cavities/voids could be encountered at other drilled shaft locations throughout the proposed bridge site. The thickness of the cavities varied from less than 1-foot to tens of feet thick. In addition, the cavities/voids were encountered between a few feet below the top of encountered bedrock to as deep as 24 feet below the top of encountered bedrock. The voids/cavities are likely filled with either clay, sand, or rubble/rock fragments, but some of the voids may not be filled with anything but air or water.

Two potential methods of installing the drilled shafts through the encountered voids/cavities are proposed. The following methods are proposed in this report; however the Contractor may propose additional installation methods that will require acceptance by the Department prior to implementation. Any alternate methods proposed by the Contractor to install the drilled shafts through encountered voids in the bedrock would need to be submitted as part of the Contractor's drilled shaft installation plan.

1. Method 1 – Cavity Stabilization – Where the thickness of voids is less than approximately 1-foot or the cavity is filled with clay or other materials that allow the cavity to be excavated without collapse, the excavation through the cavity and to some distance above the top of the cavity can be backfilled with concrete. After the concrete is allow to set for some period of time, the green concrete can be re-excavated and the rock excavation can be continued below the stabilized cavity.

2. Method 2 – Drilled Shaft Cavity Remediation – Where thicker cavities or cavities not infilled with soil, sand, etc., the cavity can be sealed by extending the permanent steel casing through the cavity and sealing the casing into the bedrock at the bottom of the cavity. This will likely require installation of a temporary steel casing through the soil column and sealing the temporary casing at the top of bedrock. The permanent casing would then be advanced into the oversized bedrock excavation and sealed at the bottom of the encountered cavity. The proposed rock socket excavation would then be performed below the encountered cavity/void.

Since only one test boring was performed at or near each drilled shaft location, an additional two rock core borings are recommended at each of the proposed drilled shaft locations at Bridge Piers 1 to 6. The purpose of the borings will be to better define the presence of, the thickness of, the plan extent of, and the general geometry of voids/cavities in the bedrock at the proposed drilled shaft locations for support of Piers 1 to 6. Based on the conditions encountered in the contractor's rock core borings, the contractor may need to modify his drilled shaft installation plan after performing the additional rock core borings. The quantities of drilled shaft common, rock and special excavation for voids would also need to be modified after the Contractor's rock core borings are completed.

Also, a boring could not be performed at the Drilled Shaft 54 location during the design phase boring program. We recommend that the Contractor also perform one rock core boring at the proposed Drilled Shaft 54 location prior to constructing drilled shafts at the proposed End Bent 2 location.

During the design exploratory test boring program, drilling tools were lost in three abandoned boring locations. Descriptions of the type and size and depth of the lost drilling tools are provided on the subsurface data sheets, which are included in Appendix A of this report. Borings where drilling tools were lost include Borings B5006, B5014 and B5066. Please note that after drilling

Boring B5014, the location of Bridge Pier 5 was relocated 20 feet to the east to the current Pier 5 location. As a result, the drilling tools lost in Boring B5014 should not be an issue during the installation of Drilled Shaft 14 at Pier 5. The Contractor will need to consider the presence of these tools during the installation of Drilled Shafts 6 and 66.

The structural design of the drilled shafts requires a steel casing from the top of the drilled shafts to the top of bedrock. It is assumed the permanent steel casing would penetrate some distance into the bedrock to form a seal at the bottom of the steel casing. This will require installing steel casing up to approximately 110 feet in length. This will require either vibrating the casing to the top of bedrock and turning the casing to get it sealed, using a rotator/oscillator to install the casing or some other method selected by the Contractor to install and seal the casing. At a previous project located at the north end of Lake Barkley and Kentucky Lake, a temporary casing installed to bedrock was required to install and seal a permanent casing into bedrock with approximately 50 to 60 feet of overburden soil below the mudline. The contractor excavated the soil from within the oversized temporary casing, set the permanent casing and then spun the permanent casing into the limestone bedrock to seal the permanent casing into the limestone bedrock.

Preliminary wave equation analyses using the GRLWEAP software indicates 86-inch diameter steel casing can be installed to the top of bedrock using vibratory methods. The analyses assume the yield strength is 45 ksi for the steel casing. The analyses assumed a variation of side and toe resistance in the deeper granular deposits to evaluate vibratory hammers producing various frequencies and equivalent energies. The analyses indicated that vibratory hammers similar to APE Tandem 600 or APE Tandem 400 hammers with cross-beam attachments will likely be required to advance the casing to bedrock. Results of the GRLWEAP analyses are presented in Exhibit E-11.

The limestone bedrock has unconfined compression strengths ranging from about 5,300 psi to over 33,000 psi, with average values around 16,000 psi. In addition, the limestone bedrock contains chert inclusions and nodules. Rock excavation in drilled shafts at other projects within the vicinity of this project have required reverse circulation drilling methods to successfully excavate the limestone bedrock that contains various sizes and distribution of chert inclusions. Methods of seating the permanent steel casing into the limestone bedrock should consider the limestone with chert inclusions encountered at all drilled shaft locations and the potential for vertical crevices and undulating bedrock surface at drilled shaft foundations.

4.7 Foundation Lateral Resistance

Due to the larger lateral and axial loads at the piers and the relatively shallow depth to bedrock at the pier locations, drilled shaft foundations socketed into bedrock are recommended at the bridge pier locations. Foundation recommendations have been developed for support of the proposed West Approach Spans from approximately Station 3148+00 to 3165+00, the East Approach Spans from Approximately Station 3172+00 to 3185+00, the east Main Span Pier at Station 3166+01 and the west Main Span Pier at Station 3171+51. At End Bent 2, we understand that the structural

design team prefers drilled shaft foundations socketed into bedrock over shallow footings bearing on bedrock to avoid a 20 to 25 feet high foundation wall. At End Bent 1, driven H-piles driven to practical refusal on the limestone bedrock are recommended.

Lateral soil resistance parameters for the encountered soils have been developed for both static and seismic conditions at the Kentucky Lake end bents, approach span piers and the main span piers. The lateral soil parameters have been provided for the L-Pile and GROUP software packages for the end bents and the approach span piers, per the request of the structural designer. Lateral soil parameters for the FB-MultiPier software have been developed for the main span piers, per the request of the structural designer.

4.7.1 Static Lateral Soil Resistance –End Bent 1

At End Bent 1, CPT soundings C4006 and B5101 and Boring B4013 were used to develop the representative soil profile. Since the H-piles are end bearing on bedrock and little to no penetration of the H-Piles into the limestone bedrock is anticipated, no lateral resistance parameters have been provided for the limestone bedrock at End Bent 1. Using the For the H-pile foundations at End Bent 1, the unit skin resistance and end bearing of the soil was calculated using the FHWA DRIVEN software. A table, Exhibit F-1, is attached in Appendix F outlining the soil stratigraphy, elevations of the soil layers and the recommended soil parameters for the static lateral pile foundation analyses. H-piles could be battered to help resist lateral loads, as well. The designer will need to consider the thickness of the anticipated bridge beams and the bottom of pile cap in selecting the top of Layer 1 elevation in Exhibit F-1.

4.7.2 Static Lateral Soil Resistance – Bridge Piers and End Bent 2

Using the subsurface data from the test borings, CPT and laboratory testing, recommended static parameters were selected using the technical manual for the L-Pile and GROUP software for the end bent and approach span pier locations. At the main span piers for the FB-MultiPier software, the Geovision PS suspension velocity data from Test Borings 4014 and 4021 were also used to help develop the Poisson's Ratio parameters for the deeper subsurface soils (See Appendix H). The pressuremeter data from Test Borings 4016 and 4020 (See Appendix I) was considered in selection of the Young's Modulus values in the granular soils, in addition to the subsurface data from the borings, CPT and laboratory testing.

Tables are included in Appendix F outlining the soil stratigraphy, elevations of the soil layers and the recommended soil parameters for the lateral pile foundation analyses. The unit skin resistance and unit end bearing values reported in the tables are based upon the methods outlined in the LRFD Drilled Shaft Manual and the Sixth Edition of the AASHTO LRFD Bridge Design Manual.

The ground surface elevation at each end bent will need to be selected based upon the bottom of pile cap elevation. The designer will need to consider the effects of scour on the selection of a mudline elevation in the lateral analyses. If an elevation is selected for the mudline that is higher than or lower than the Layer 1 mudline elevation on the tables, the designer should keep the bottom elevation of Layer 1 constant and use the recommended parameters from the bottom of Layer 1 elevation to the selected mudline elevation.

Exhibits F-2 to F-13 correspond to the west and east approach piers. Exhibits F-14 to F-19 correspond to the main span piers. The lateral soil parameter exhibits are direction sensitive for the main span piers and the correct exhibit for the loading direction will need to be selected.

Exhibits F-20 to F-25 correspond to End Bent 2. End Bent 2 has been divided into the front abutment wall, the north wing wall and the south wing wall. Based on variations in encountered bedrock elevation at the test borings, the abutment wall was subdivided into two idealized soil and bedrock profiles. The south wing wall was subdivided into three idealized soil and bedrock profiles based upon the observed variations in top of bedrock elevation at each of the three drilled shaft locations in the south wing wall structure.

Water level elevation for the maximum regulated flood in Lake Barkley is Elevation 375 feet. However, this is a short-term condition that would likely not saturate the anticipated cohesive embankment materials at the end bents and the shear strength of the durable rock fill would remain constant during the flood event. All soils below Elevation 359 feet should be assumed as saturated, due to the pool elevation of the lake. Therefore, it is our opinion that evaluation of the short-term flood event is not warranted.

4.7.3 Seismic Lateral Soil Resistance – End Bent 1

Seismic parameters for use in the L-Pile or GROUP software and seismic axial foundation resistance were developed for the Lake Barkley End Bent 1 location at Station 3147+95. The seismic lateral soil response parameters for non-liquefied soils consider 90% of the static undrained shear strength values for fine-grained soils and internal angle of friction for granular soils [$0.9Su$ or $\tan^{-1}(0.9\tan\Phi)$] as compared to the static soil parameters estimated for the Approach Spans. In addition, 90% of the k (p-y) values are recommended for the design seismic event. These parameters were selected by considering published literature and the cyclic shear tests performed on selected soil samples.

For the H-pile foundations at End Bent 1, the unit skin resistance of the soil was calculated using the FHWA DRIVEN software and the seismic shear strength parameters described earlier. A table, Exhibit F-26, is attached in Appendix F outlining the soil stratigraphy, elevations of the soil layers and the recommended soil parameters for the seismic lateral pile foundation analyses. The designer will need to consider the thickness of the anticipated bridge beams and the top of the anticipated drilled shaft elevation or the bottom of pile cap in selecting the top of Layer 1 elevation in Exhibit F-26.

Based upon liquefaction analyses at End Bent 1, several soil layers have the potential for liquefaction or softening from the existing ground surface to approximate Elevation 316.7 feet at the idealized section. For liquefied/softened soils, it is recommended that they be modeled as soft clay using the residual shear strength of the liquefied soil and the major strain (ε_{50}) reported in the following table. Please note that the soil layers with liquefaction potential have been simplified and reduced to 3 layers in the table. The lateral parameters for soil above, below and in between the liquefied layers are equal to the values for either Layers 1, 2 or 3 (based upon elevation) reported in Exhibit F-26.

Top Elevation (ft.)	Bottom Elevation (ft.)	Average Residual Shear Strength (psf)	Major Strain ε_{50}
End Bent 1			
348.5	348	70	0.02
321	321.5	210	0.02
318.2	316.7	170	0.02

At the End Bent 1 location, two seismic load cases will need to be considered. One case is where the individual soil layers liquefy and non-liquefied soil layers above the liquefied soil layers translate laterally during the seismic event. This load case would occur during the seismic event. Soil Parameters from Exhibit F-26 would be used for the non-liquefied soil parameters.

The second case considers where the soil above a liquefied or softened layer near the toe elevation of the submerged Cumberland River slope moves laterally, resulting in kinematic loading of the piles due to sliding of the stable soil mass above the lowermost liquefiable or softened layer. The lowest elevation of a liquefiable soil layer was Elevation 316.7 feet in Boring B4013, which is approximately 12.5 feet higher than the bottom of the nearby submerged Cumberland River channel. The kinematic loading case should be considered for soil above the lowest liquefiable layer, which is at approximately Elevation 316 feet. The kinematic load could be calculated using the recommended Young's Modulus values in Exhibit F-26 for Layers 1, 2 and 3 and the simplified pile analysis for kinematic loading recommended in Chapter 10 of the FHWA document LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations, Publication Number FHWA-NHI-11-032, dated August 2011. This load case would occur immediately after the seismic event. Below Elevation 316 feet, the soil parameters in Exhibit F-26 should be used for design.

4.7.4 Seismic Lateral Soil Resistance – Bridge Piers

Using the subsurface data from the test borings, CPT and laboratory testing, recommended seismic parameters were selected using the technical manual for the L-Pile and GROUP software for the approach span pier locations for the design team. At the main span piers seismic FB-MultiPier software parameters were developed for use by the design team. Exhibits F-27 through F-44 in Appendix F outline the soil stratigraphy, elevations of the soil layers and

the recommended soil parameters for the seismic lateral pile foundation analyses at the approach span piers and main span piers.

The seismic lateral soil response parameters for non-liquefied soils consider 90% of the static undrained shear strength values for fine-grained soils and internal angle of friction for granular soils [$0.9S_u$ or $\tan^{-1}(0.9\tan\Phi)$] as compared to the static soil parameters estimated for the Approach Spans. In addition, 90% of the k (p-y) values are recommended for the design seismic event. These parameters were selected by considering published literature and the cyclic simple shear tests performed on selected soil samples. The lateral soil resistance parameters are directionally sensitive at the main span locations. The designer will need to consider the effects of scour on the selection of a mudline elevation in the lateral analyses. If an elevation is selected for the mudline that is higher than or lower than the Layer 1 mudline elevation on the tables, the designer should keep the bottom elevation of Layer 1 constant and use the recommended parameters from the bottom of Layer 1 elevation to the selected mudline elevation.

Based upon our liquefaction analyses at the approach spans and main span, several soil layers have the potential for liquefaction or softening from the existing mudline to between approximate Elevations 268 to 276 feet at the approach span idealized sections and between approximate Elevations 280 to 292 feet at the main span idealized sections. For liquefied/softened soils, it is recommended that they be modeled as soft clay using the residual shear strength of the liquefied soil and the major strain (ε_{50}) reported in the following table. Please note that the soil layers with liquefaction potential have been simplified and reduced to 4 to 6 layers in the table. The lateral parameters for soil above, below and in between the liquefied layers are equal to the values for either Layers 1, 2, 3, 4, 5, 6 or 7 (based upon elevation and the idealized profile being used) reported in Exhibits F-27 to F-44 in Appendix F for the end bents and approach spans. If the actual mudline at a given pier elevation is lower than any of the liquefied soil layers in the table, the liquefied soil layers above the mudline should be removed.

Approximate Top Elevation (feet)	Approximate Bottom Elevation (feet)	Estimated Average Residual Shear Strength (psf)	Major Strain ε_{50}
CPT C4009 – Pier 1			
339	336	10	0.02
326.5	322	50	0.02
319	315	100	0.02
310	307	210	0.02
287	284	430	0.02
277	276	180	0.02

Approximate Top Elevation (feet)	Approximate Bottom Elevation (feet)	Estimated Average Residual Shear Strength (psf)	Major Strain ε_{50}
CPT C4013 - Piers 2, 3 and 4			
347	344	10	0.02
336	333	60	0.02
332	323	120	0.02
319	317	190	0.02
306	305	280	0.02
270	268	270	0.02
CPT C4014 – Piers 5 and 6			
331	329	70	0.02
319	317	170	0.02
305	303	280	0.02
271	268	400	0.02
CPT C4022 – Piers 9 and 10			
345	340	5	0.02
338	326.5	15	0.02
326.5	315	50	0.02
314	312	80	0.02
268.5	267	590	0.02
CPT C4028 – Pier 11			
344	341	5	0.02
333	313	65	0.02
312	308	160	0.02
297	294	225	0.02
CPT C4030 – Pier 12			
338.5	337.5	25	0.02
329.5	328.5	70	0.02
325	321	95	0.02
317	315	120	0.02
CPT/SPT B4015 – West Main Span Pier – Pier 7			
323.2	319.6	320	0.02
315.9	307.3	700	0.02
306.4	303.0	115	0.02
302.2	301.2	90	0.02
294.9	292.4	210	0.02

Approximate Top Elevation (feet)	Approximate Bottom Elevation (feet)	Estimated Average Residual Shear Strength (psf)	Major Strain ε_{50}
CPT 4021/SPT B4021 – East Main Span Pier – Pier 8			
313.0	310.5	5	0.02
308.0	305.0	20	0.02
299.3	297.6	310	0.02
286.6	284.6	490	0.02
281.1	279.6	600	0.02

At the approach span and main span locations, two seismic load cases will need to be eventually considered. One case is where the individual soil layers liquefy and non-liquefied soil layers above the liquefied soil layers translate laterally during the seismic event. This load case would occur during the seismic event. Soil Parameters from Exhibits F-27 through F-44 would be used for the non-liquefied soil parameters.

The second case considers where the soil above a liquefied or softened layer near the toe elevation of the submerged Cumberland River slope moves laterally after the seismic event, resulting in kinematic loading of the piles due to sliding of the stable soil mass above the lowermost liquefiable or softened layer. The lowest elevation of the liquefiable soil layers in the representative West Approach Span profiles range from approximately Elevations 268 to 276 feet, which is approximately 28 to 36 feet lower than the bottom of the adjacent submerged Cumberland River channel. The lowest elevation of the liquefiable soil layers in the representative East Approach Span profiles range from approximately Elevations 267 to 336 feet, which is approximately 37 feet lower than to 32 feet higher than the bottom of the adjacent submerged Cumberland River channel.

The kinematic loading case should be considered for soil above the submerged Cumberland River channel elevation, which is at approximately Elevation 304 feet. The kinematic load could be calculated using the recommended Young's Modulus values in Exhibits F-27 to F-38 for Layers 1, 2, 3 and/or 4 and the simplified pile analysis for kinematic loading recommended in Chapter 10 of the FHWA document LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations, Publication Number FHWA-NHI-11-032, dated August 2011. This load case would occur immediately after the seismic event. Below Elevation 304 feet, except the liquefied/strain softened intervals below Elevation 304 feet, the soil parameters in Exhibits F-27 to F-38 should be used for design.

At the West Main Span Pier (Pier 7), the kinematic loading case should be considered for soil above the submerged Cumberland River channel elevation, which is at approximately Elevation 304 feet. This case will apply to the west main span pier location, which is adjacent to the submerged west Cumberland River bank. The kinematic load could be calculated using the recommended Young's Modulus values in Exhibit F-39 for Layers 1 and 2 and the simplified pile

analysis for kinematic loading, as recommended in Chapter 10 of the FHWA document LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations, Publication Number FHWA-NHI-11-032, dated August 2011. This load case would occur immediately after the seismic event. Below Elevation 301.2 feet and except for the interval having liquefaction potential from Elevations 294.9 to 292.4 feet, the soil parameters in Exhibit F-39 should be used for design, since the layers below the presented elevations do not appear to have liquefaction potential.

At the East Main Span Pier (Pier 8), the kinematic loading case should be considered for soil above the submerged Cumberland River channel elevation, which is at approximately Elevation 304 feet. This case will apply to the east main span pier location, which is adjacent to the submerged east Cumberland River bank. The kinematic load could be calculated using the recommended Young's Modulus values in Exhibit F-42 for Layers 1 and 2 and the simplified pile analysis for kinematic loading, as recommended in Chapter 10 of the FHWA document LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations, Publication Number FHWA-NHI-11-032, dated August 2011. This load case would occur immediately after the seismic event. Below Elevation 305 feet and except for the intervals having liquefaction potential from Elevations 299.3 to 297.5 feet, Elevations 286.6 to 284.6 feet and Elevations 281.1 to 279.6 feet, the soil parameters in Exhibit F-42 should be used for design, since the layers below the presented elevations do not appear to have liquefaction potential.

Alternately, the kinematic load from seismic lateral displacement can be modeled using offset p-y curves for the soil mass above the lowest liquefiable layer. The p-y curves for the non-liquefied mass are used to load the pile by "offsetting" the p-y curves relative to the pile axis a distance equal to the estimated seismic lateral displacement. The corresponding p-value for that displacement is used to load the pile. For the zones above and below the lowest liquefied layer, the soil parameters from Exhibits F-27 to F-44 would apply. For the liquefied layer, the soil is treated as a soft clay using the average residual strength value and major strain values shown in the table on pages 49 and 50 of this report.

Lateral spreading analyses were performed using yield accelerations back-calculated from pseudo-static slope stability analyses, the residual shear strengths in the liquefied zones, and the Newmark method of calculating seismic lateral displacements from the acceleration time histories at the base of the liquefied zone. Liquefaction Analyses and the results from the Newmark analyses (reported in the table below) should be used in the kinematic loading case.

Station Ranges	Base Elevation for Lateral Displacements (feet)	Seismic Lateral Displacements-Newmark Method (inches)
3148+00 to 3164+50	304	<0.25
3164+50 to 3165+75	304	<1.0
3172+00 to 3173+00	304	1.5
3173+00 to 3173+50	304	1.0
3173+50 to 3174+00	304	0.5
3174+00 to 3184+00	304	<0.5

4.7.5 Seismic Lateral Soil Resistance – End Bent 2

Using the subsurface data from the test borings, CPT and laboratory testing, recommended seismic parameters were selected using the technical manual for the L-Pile and GROUP software for the End Bent 2 location for the design team. End Bent 2 has been divided into the front abutment wall, the north wing wall and the south wing wall. Based on variations in encountered bedrock elevation at the test borings, the front abutment wall was subdivided into two idealized soil and bedrock profiles. The south wing wall was subdivided into three idealized soil and bedrock profiles based upon the observed variations in top of bedrock elevation at each of the three drilled shaft locations in the south wing wall structure. Exhibits F-45 through F-50 in Appendix F outline the soil stratigraphy, elevations of the soil layers and the recommended soil parameters for the seismic lateral pile foundation analyses at End Bent 2.

The seismic axial resistance in soils and lateral soil response parameters for non-liquefied soils consider 90% of the static undrained shear strength values for fine-grained soils and internal angle of friction for granular soils [0.9Su or $\tan^{-1}(0.9\tan\Phi)$] as compared to the static soil parameters estimated for End Bent 2. In addition, 90% of the k (p-y) values are recommended for the design seismic event. These parameters were selected by considering published literature and the cyclic shear tests performed on selected soil samples.

Based on the bedrock properties, the vertical movement of the drilled shafts embedded within the limestone bedrock may not be sufficient to mobilize full axial side resistance within the soil. Considering the relatively thin soil thickness encountered (about 9 to 31 feet) at the End Bent 2 location and potential for small vertical drilled shaft displacements under axial load, no skin resistance in the soil column has been provided in Exhibits F-45 through F-50. At the End Bent 2 location, no liquefaction or lateral spread is anticipated based upon the encountered subsurface conditions encountered in the test borings at End Bent 2.

4.7.6 Passive Pressure at Integral Abutment

We understand that the designers may wish to consider the transfer of seismic loads to the soil behind the end bent abutment walls. Per AASHTO Guide Specifications for LRFD Seismic Bridge Design and assuming that Item 805.11 (Granular Structural Backfill) is used as backfill behind the abutment, a passive earth pressure coefficient of 8.1 is recommended for seismic design. This value assumes a minimum phi angle of 33 degrees (angle of internal friction) for

seismic conditions (36 degrees for static conditions) with a delta value of 2/3 phi between the concrete wall and the soil backfill. The granular structural backfill should extend the height of the abutment wall horizontally back from the bottom of the pile cap (i.e. if the abutment wall is 10 feet tall from the bottom of the pile cap to the proposed road grade, the granular structural backfill should extend 10 feet back horizontally from the back of the pile cap), then extend up at a 45 degree angle to the vertical away from the abutment.

4.8 Roadway Geotechnical Notes

The following notes are recommended for construction of the proposed embankments at the Lagoon Bridge and the Causeway. The following Roadway Geotechnical Notes were prepared by Florence and Hutcheson (now ICA Engineering) and are included in the project Roadway Plans.

1. Clearing and grubbing of embankment areas shall be completed in accordance with Section 202 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction.

2. Removal of existing structures and other obstructions shall be completed in accordance with Section 203 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction.

3. Based upon available soil testing performed along this project, some soil horizons are considered to be erodible. Procedures shall be performed as required to control erosion and water pollution in accordance with Section 212 and 213 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction.

4. All water wells and/or cisterns within the limits of construction, whether shown on the plans or not, shall be plugged in accordance with Section 708 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction.

5. All channel changes and special ditches shall be constructed prior to placement of any embankment materials adjacent to them in accordance with Section 206 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction. Materials excavated from these areas may be utilized in construction of the embankments, but may require aeration to the proper moisture contents prior to compaction operations. No extra payment shall be permitted for re-handling, hauling, stockpiling and/or manipulating these materials.

6. In accordance with Section 206 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge construction, the moisture content of embankment and subgrade materials shall not vary from the optimum moisture content, as determined by KM 64-511, by more than plus or minus two percent. This moisture content requirement shall have equal weight with the density requirement when determining the acceptability of embankment and subgrade construction. Embankment materials that are constructed above Elevation 363 feet shall be compacted to at least 95 percent of the standard proctor maximum dry density as determined by KM 64-511. Refer to the family of curves for moisture-density relationships.

7. The Contractor is responsible for conducting any operations necessary to excavate the cut areas to the required typical sections. These operations shall be incidental to Roadway Excavation.

8. All soils, whether from roadway excavation or borrow, may require manipulation to obtain proper moisture contents prior to compaction. Direct payment shall not be permitted for re-handling, hauling, stockpiling and/or manipulating soils.

9. Existing stone riprap in the footprint of the proposed embankment above Elevation 359 feet shall be removed and/or stockpiled prior to any embankment construction being performed. The pay item for removing riprap shall be “Special Excavation” (CY).

10. Any embankment from the existing ground line to a minimum elevation of 363 feet shall be constructed of Granular Embankment meeting the requirements of Section 805.10 of the Standard Specifications for Road and Bridge Construction, current edition. As per Section 805.03.04 of the Standard Specifications for Road and Bridge Construction, the granular embankment shall be non-erodible and stable material is required to be quarry-processed limestone. Also, the Granular Embankment gradation shall be in accordance with the following table:

GRADATION – GRANULAR EMBANKMENT	
Sieve Size	**Allowable Percent Passing (%)**
12”	100
3”	0-100
2 ½”	0-100
2”	0-90
1 ½”	0-30
¾”	0-10
No. 200	0-7

The Granular Embankment a minimum of one foot above the water elevation at the time of construction or as directed by the Engineer, shall be constructed by end-dumping and grading the material into the lakes edge until a firm working platform is constructed above the water upon which normal embankment construction can take place. Any portion of the Granular Embankment that is not constructed by the end-dumping method shall be constructed in proper lifts in accordance with Section 206 of the Standard Specifications for Road and Bridge Construction. The outside geometry of this portion of embankment should be such that the toe of the construction berm should extend to the approximate location of the projected roadway embankment toe, resulting in an effective geometry from top of slope to toe of slope. Prior to placement of granular embankment, Type III geotextile fabric shall be placed beneath the granular embankment, and at the interfaces of the granular embankment and existing ground line. Also, in accordance with Section 214.03.03 of the Standard Specification for Road and Bridge Construction, it will be required to limit the drop height of the lowest 2 feet of granular embankment to no greater than 3 feet to avoid damaging the Type III geotextile fabric. Prior to placement of the embankment materials above Elevation 363 feet and after placement of the granular embankment, Type IV geotextile fabric shall be placed on the granular embankment to serve as a separator. Both Type III and Type IV geotextile fabrics shall be in accordance with Section 213 and 843 of the current Kentucky Department of Highways Standard Specifications. This work shall be done as shown on the project cross-sections and as directed by the Engineer.

11. All fill slopes shall be protected with a minimum two feet of Cyclopean Stone Riprap meeting the requirements of Section 805 of the Standard Specifications for Road and Bridge Construction, current edition. The riprap shall be placed as an integral part of the embankment and not as an additional thickness on the outside face of the final embankment geometries. The riprap shall extend from the toe of the embankments upwardly to Elevation 375 feet. The riprap shall be placed in accordance with Section 703 of the current standard specifications for Road and Bridge Construction and as directed by the Engineer. A Type I Geotextile Fabric, meeting the requirements of Section 214 and 843 of the Standard Specifications for Road and Bridge Construction, current edition, shall be placed between the embankment above Elevation 363 feet and the slope protection.

12. The contractor shall construct embankment foundation benches and transverse benches as indicated on the plans or as directed by the Engineer, prior to placement of embankment in areas requiring such benches. The benches shall be constructed one at a time beginning with the lowest bench. Each bench shall be backfilled prior to excavation of the next bench. This procedure shall be followed to help maintain stability of the existing slopes.

13. Materials used for embankment construction above Elevation 363 feet shall have the following minimum effective strength parameter:

    Friction Angle (Φ') = 30^0

    The friction angle of 30° as shown above is an absolute minimum for the embankment materials above Elevation 363 feet. This minimum strength parameter shall be verified through the use of laboratory tests on collected samples to obtain source approval. The Contractor is responsible for obtaining the necessary samples through the use of either test pits or auger borings, performing the required laboratory tests and presenting these results in advance for KYTC's approval prior to beginning construction of the embankment above Elevation 363 feet. Laboratory testing on these samples shall include Consolidated Undrained (CU) triaxial in accordance with AASHTO T297 for determining the material's strength. Prior to performing the CU triaxial tests, the lab samples are to be compacted to a minimum of 95% standard proctor maximum dry density as determined by KM 64-511. Also, the moisture content of the lab samples shall not vary from the optimum moisture content, as determined by KM 64-511, by more than plus or minus two percent. It is required that all necessary CU triaxial tests be performed and approved by KYTC prior to beginning construction of the embankment above Elevation 363 feet. A minimum of one (1) set of triaxial tests shall be completed for up to 20,000 cubic yards of embankment or for every change in either material type or borrow source. One (1) CU set constitutes three (3) individual triaxial tests or points. After completion of the CU triaxial tests, index testing shall also be performed. One (1) index test will be required for each CU set and the sample selected for the index test should be representative of the entire CU set which includes three (3) individual triaxial tests or points. These index tests will include Atterberg Limits in accordance with AASHTO T89 and T90 and Gradation in accordance with AASHTO T88. Soils that classify as non-plastic (NP) will not require Atterberg Limits. Laboratory testing needs to be performed by a firm pre-qualified by KYTC for Geotechnical Laboratory Testing and AASHTO Materials Reference Laboratory (AMRL) Accredited for AASHTO T297, T88, T89 and T90. Prior to beginning testing, consult the Geotechnical Branch to ensure that a lab is accredited or certified. The cost of obtaining the samples and performing the laboratory tests shall be incidental to the unit price for Embankment in Place.

14. In addition to the required CU triaxial testing, index testing shall also be performed on the proposed materials used for embankment construction above Elevation 363 feet. These index tests shall include Atterberg Limits in accordance with AASHTO T89 and T90 and Gradation in accordance with AASHTO T88. Soils which classify as non-plastic (NP) will not require Atterberg Limits. A minimum of one (1) set of index testing shall be completed for up to 5,000 cubic yards of embankment or for every change in either material type or borrow source. The Contractor shall randomly sample from the borrow source and the samples for index testing must come from the 5,000 cubic yards of embankment that the samples represent. As mentioned in the Geotechnical Note Number 13, there will be one (1) index test completed for each CU set where the one

(1) index test is representative of the three (3) individual triaxial tests or points. This one (1) index test completed with each CU set can be counted and used toward the one (1) set of index testing requirement for every 5,000 cubic yards of embankment material. If the embankment material is placed prior to receiving the results of the index testing and the index tests show that the material is unacceptable, then the Contractor may have to remove that material at no expense to the Department. Soils that classify in accordance with the Unified Soil Classification System as GW, GP, GM, GC, SW, SP, SM, SC, CL, or ML will be accepted as embankment material above Elevation 363 feet. Also, in accordance with the Unified Soil Classification System, soils that classify as CH, MH, OH, OL, or Pt will not be accepted to be used as embankment material above Elevation 363 feet. If the classification of the samples is different than the classifications of the triaxial samples, additional CU triaxial testing may be required at the discretion of the Department and at no additional cost to the Department. In addition, CU triaxial and index testing may be waived on embankment materials above Elevation 363 feet if granular embankment meeting the requirements in Geotechnical Note Number 10 is utilized higher than the required minimum elevation. Laboratory testing needs to be performed by a firm pre-qualified by KYTC for Geotechnical Laboratory Testing and AASHTO Materials Reference Laboratory (AMRL) Accredited for AASHTO T88, T89, and T90. Prior to beginning testing, consult the Geotechnical Branch to ensure that a lab is accredited or certified. The cost of obtaining the samples and performing the laboratory tests shall be incidental to the unit price for Embankment in Place.

15. The bridge approach embankment at End Bent 1 shall include ‘granular pile cores’ to facilitate installation of the foundation systems. Construction of the pile cores shall be in accordance with KYTC Special Provision No. 69, Standard Drawing Nos. RGX-100 and RGX-105 (with the exceptions described below), and Section 206 of the current Kentucky Department of Highways Standard Specifications for Road and Bridge Construction. Erodible materials shall not be used in construction of pile cores. An exception to the KYTC Standard Drawings RGX-100 and RGX-105 is the structure granular backfill. Behind the abutments of the Lake Barkley End Bents, this structure granular backfill shall extend 16 feet horizontally back into the embankment from the bottom of the pile/shaft cap then extend up a 1H:1V slope vertically away from the abutment. This work shall be done as shown on the project plans and as directed by the Engineer.

16. The contractor shall not excavate either temporary or permanent to slopes steeper than shown on the project plans without approval by the Engineer. This includes excavation within the existing causeway.

17. Settlement monitoring in the form of telltales will be required near End Bent 1 of the Lake Barkley Bridge. The telltales shall be installed and constructed in accordance with the following guidelines:

a. Within two (2) weeks of the completion of Granular Embankment to Elevation 363 feet at each of the two (2) locations described below, install telltale assemblies through the granular embankment. Telltale assembly holes may be installed using an air drill or other method approved by the Engineer. At Station 3147+80, install one telltale 15 feet right of the causeway centerline and the second 20 feet left of the causeway centerline.

See the Telltale Detail for more information (Appendix D, Exhibit D-4).

Proposed Telltale Locations and Elevations		
Station	**Offset (feet)**	**Grouted Bottom of Telltale Elevation Range (feet)**
3147+80	+/- 20 Left	345-347
3147+80	+/- 15 Right	357-359

b. Grout the telltale rod at least 4 feet below the original grade elevation, using a neat cement grout. The length of the grouted zone should be at least 2 feet. The steel telltale rod should have a nominal diameter of 3/4 inches. Install a threaded nut, coupler or tack weld a steel washer at the bottom end of the telltale rod to increase bond between grout and telltale rod. The rod shall extend to between 1 to 4 feet above Elevation 363 feet. Contractor shall provide details of proposed telltale assembly to Engineer for approval prior to installation.

c. Provide a casing around the telltale rod, ensuring no bond between the casing and the grout or between the telltale rod and the casing. Any backfill around the casing is incidental to the unit bid price of Settlement Platform.

d. Provide centralizers between the casing and the telltale rod, unless the annulus between the casing and the telltale rod is less than 1/2 inch.

e. As the embankment in place is constructed, stop work at any location where telltales are disturbed or damaged, and make necessary repairs or replacement at no additional cost to the Department. As necessary, add sections of the telltale rod and casing to the assembly to maintain a rod height of 1 to 4 feet above the surrounding ground surface. Telltale rod extensions and casing should be secured in a manner so that future readings are not affected. Take elevation readings immediately before and after each extension is added. Cover the top of the casing as work progresses to prevent material from falling into casing. Extend the casing and telltale rod to the end of grading contract causeway grade +/-6 inches.

f. Provide either a flush mount or above grade protective cover upon reaching construction finish grade.

The installation and extension of telltales shall be included in the pay item, "Settlement Platform", including all labor and materials for the telltale assembly, installation of the

assembly and telltale readings taken for the extension of rods. Standard Drawing RGX-015-02 and Section 216 of the Standard Specification will not apply and there will be no separate payment for steel pipes. The quantity is 2 Settlement Platforms.

18. Perform all necessary surveying under the general supervision of a Professional Engineer or Licensed Land Surveyor. Within 2 days after initial installation of the telltales, the top of the telltale rod shall be surveyed for vertical elevation using a surveying method having vertical precision of 0.01 feet. During the Embankment in Place construction and for the first 10 weeks after reaching finished construction grade at each telltale location, the reading frequency of each telltale shall be once every 2 weeks. The reading frequency shall then be reduced to once every 4 weeks, continuing until the Contractor is notified to cease telltale surveying. Provide the surveyed elevation data for each telltale on a monthly basis to the Engineer. The pay items for the readings shall be:

    a. "Telltale Surveying", "Each", including all labor, travel, materials and reports associated with this task for all installed telltales for each set of readings except those incidental to rod extensions. The Engineer will determine when the Telltale survey readings are no longer required. The estimated quantity is 20 readings.

APPENDIX A

SUBSURFACE DATA SHEETS AND FIELD EXPLORATION

Field Exploration Description

The boring and sounding locations for the existing causeway study were staked by Hall-Harmon Surveying prior to our arrival onsite for drilling activities. Ground surface elevations indicated on the boring logs were recorded from information provided on the boring stakes. For the new alignment of the Lake Barkley Bridge, survey control was provided by Florence & Hutcheson (ICA) for both marine and land locations. The locations and elevations of the borings should be considered accurate only to the degree implied by the means and methods used to define them. The Coordinate Data Submission Form for the project SPT borings and CPT soundings is attached in Appendix J.

Drilling / Standard Penetration Test / Sampling

Borings for the Lake Barkley Bridge were performed by KYTC, Florence & Hutcheson (ICA) and HCN/Terracon. KYTC used a CME-550 rotary drill rig to perform borings on Lake Barkley from a spud barge. The Florence & Hutcheson/ICA borings were drilled with a CME-55 track-mounted rotary drill rig in 2014 and with a CME-45 track-mounted rotary drill rig in 2010. HCN/Terracon borings were drilled using a CME-550 rotary drill rig for land and marine borings. All borings were performed using a 4-inch-diameter casing advancer. Samples of the soils encountered in the borings were obtained using split-barrel and thin-walled tube procedures.

In the split-barrel sampling procedure, the number of blows required to advance a standard 2-inch O.D. split-barrel sampler the last 12 inches of the typical total 18-inch penetration by means of a 140-pound hammer with a free fall of 30 inches, is the standard penetration resistance value (SPT-N). This value is used to estimate the in-situ relative density of cohesionless soils and consistency of cohesive soils. When the 3-inch O.D. split-barrel sampler was used, the N-values were corrected for the oversized sampler.

A CME automatic SPT hammer was used to advance the split-barrel sampler in the borings. A significantly greater efficiency is achieved with automatic hammers compared to the conventional safety hammer operated with a cathead and rope. This higher efficiency has an appreciable effect on the SPT-N value. Terracon performed SPT hammer energy calibrations in the field for each of the drill rigs used on this project.

In the thin-walled tube sampling procedure, a thin-walled, seamless steel tube with a sharp cutting edge is pushed hydraulically into the soil to obtain a relatively undisturbed sample. The samples were tagged for identification, sealed to reduce moisture loss, and taken to our laboratory for further examination, testing, and classification. Information provided on the boring logs attached to this report includes soil descriptions, consistency evaluations, boring depths, sampling intervals, and groundwater conditions. The borings were backfilled with auger cuttings prior to the drill crew leaving the site.

At selected locations where bedrock was encountered, core samples of bedrock were obtained using dual-tube rock coring techniques. Cores were visually logged in the field and stored in core boxes for additional logging and transport.

A field log of each boring was prepared by a field geologist, engineer or geotechnical technician. These logs included visual classifications of the materials encountered during drilling as well as the engineer's interpretation of the subsurface conditions between samples.

Cone Penetration Test (CPT) Soundings

The CPT soundings were made by KYTC and HCN/Terracon. Kentucky Transportation Cabinet performed their soundings using the CME-550 rig. HCN/Terracon used both the CME-550 rig or a Geoprobe 5400 mounted on a Ford F450 four-wheel drive truck. KYTC used a Hogentogler 10-cm^2 cone while HCN/Terracon used a Geotech AB NOVA 10-cm^2 cone system to obtain CPT data. A continuous record was obtained wherever possible from the ground surface or the minimum depth to establish the casing. Where premature refusal was met, a geotechnical drill rig was used to set casing down approximately 5 feet past the refusal depth or until the driller determined that suitable material for CPT was encountered. If no suitable material for CPT was encountered, an SPT sample was obtained to document the drilled material. At this point, the cone system was deployed through the casing and continued to the target depth or refusal, whichever occurred first.

Downhole Seismic Tests

The downhole seismic test was deployed using the CPT system by adding an accelerometer package to the cone rod string. The CPT soundings were paused at approximate 5-foot intervals to perform the downhole seismic test. The test consists of triggering a polarized shear-wave source at the ground surface and recording the arrivals of the shear wave at the cone tip. All data was collected by logging software on a laptop computer on site. The collected data was interpreted and is presented in this Appendix as Exhibits A-48 to A-58.

PS Suspension Logging

See Appendix I