S-134-2007
(Consultant)

M E M O R A N D U M

TO: Mark Hite, P.E.
Division of Structural Design

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

DATE: May 21, 2009

SUBJECT: Greenup County
Report of Geotechnical Exploration
KY 2541Bridge Replacement Over
Little Sandy River
Item No. 9-1038.0
Mars No. 6739701D

The geotechnical engineering report for this structure has been completed by H.C. Nutting. We have reviewed and concur with the recommendations as presented in this report.

A copy of the report is attached. If you have any questions, please contact this office at 502-564-2374.

Attachment:

CC: R. Powell
D. Eldridge
R. Stull
D. Miller
A. Ross
J. Simpson
B. Bryant
B. Greene
D. Moses
J. Kenkel (H.C. Nutting)

GEOTECHNICAL ENGINEERING REPORT

GREENUP COUNTY, KENTUCKY
FD 50 045 2541 001- 002
BRZ 093 101
MARS NO. 6739701D
KY 2541 BRIDGE REPLACEMENT OVER LITTLE SANDY RIVER
ITEM NO. 09-1038.0
STATION 8+50 TO STATION 20+05

FOR

VAUGHN & MELTON CONSULTING ENGINEERS
MIDDLESBORO, KENTUCKY

April 2009

H. C. NUTTING

A Terracon COMPANY

April 6, 2009

W.O. 71503.009 / N1085515

Mr. Bart Asher, P.E.
Kentucky Transportation Cabinet
Division of Structural Design
1236 Wilkinson Blvd.
Frankfort, Kentucky 40601-1200

RE: Geotechnical Engineering Report
Greenup County, Kentucky
FD 50 045 2541 001-002
BRZ 093 101
Mars No. 6739701D
KY 2541
Bridge Replacement over Little Sandy River
Item No. 09-1038.0
Station 8+50 to 20+05

Dear Mr. Asher:

We are pleased to submit herewith our report of geotechnical study for the reference project. The geotechnical study included subsurface exploration drilling, laboratory testing, engineering analysis, meetings with the design team, and submittal of this geotechnical engineering report and drawings.

We appreciate the opportunity to be of service. Please contact us if you have any questions.

Respectfully submitted,
H. C. Nutting Company

Jerome B. Kenkel, P.E.
Principal Geotechnical Engineer

Henry Mathis, P.E.
Senior Consultant

Cc: Mr. Ken Corder, P.E., Vaughn & Melton

GEOTECHNICAL ENGINEERING REPORT
GREENUP COUNTY, KENTUCKY
FD 50 045 2541 001-002
BRZ 093 101, MARS NO. 6739701D
KY 2541 BRIDGE REPLACEMENT OVER LITTLE SANDY RIVER
ITEM NO. 09-1038.0
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TABLE OF CONTENTS

Section **Page No.**

1.0 LOCATION AND DESCRIPTION OF PROJECT 3
2.0 TOPOGRAPHY AND DRAINAGE 3
3.0 GEOLOGY 4
4.0 DRILLING AND SAMPLING 4
4.1 General 4
4.2 Rock Core Borings 5
4.3 Disturbed Sample Borings 6
4.4 Undisturbed Sample Borings 6
5.0 LABORATORY TESTING AND RESULTS 6
5.1 General 6
5.2 Slake Durability Index (SDI) and Jar Slake Testing (JS) 6
5.3 Disturbed Soil Testing 7
5.4 Undisturbed Soil Testing 7
5.4.1 Unconfined Compressive Strength Testing 8
5.4.2 Unconsolidated-Undrained Triaxial Strength Testing 8
5.4.3 Consolidated-Undrained Triaxial Testing with Pore Pressure Measurements. 8
5.4.4 One-Dimensional Consolidation Testing 8
6.0 ENGINEERING ANALYSES 9
6.1 Embankment Slope Stability Analyses 9
6.2 Settlement Analyses 14
6.3 Negative Skin Friction Analyses 16
6.4 Pile Foundation Driveability Analyses 17
6.4.1 West Abutment (End Bent 1) – Sta. 13+43.00 17
6.4.2 East Abutment (End Bent 2) – Sta. 18+03.00 19
6.5 Drilled Shaft Design 21
7.0 LATERAL LOAD ANALYSIS 33
7.1 Bridge Pier 1 Sta. 15+73.00 33
8.0 GEOTECHNICAL NOTES 34
9.0 DESIGN RECOMMENDATIONS 36

APPENDICES
Appendix A Project Location Map
Appendix B Coordinate Data Submission Form
Appendix C Cone Penetrometer Testing Logs
Appendix D Engineering Drawings

GEOTECHNICAL ENGINEERING REPORT
GREENUP COUNTY, KENTUCKY
FD 50 045 2541 001-002
MARS NO. 6739701D
KY 2541 BRIDGE REPLACEMENT OVER LITTLE SANDY RIVER
ITEM NO. 09-1038.0
STATION 8+90 TO STATION 20+05

1.0 LOCATION AND DESCRIPTION OF PROJECT

The project involves a bridge replacement of the existing steel truss bridge over the Little Sandy River (KY 2541) in Greenup County, Kentucky near the confluence with the Ohio River, which is located approximately 400 to 500 feet left of the centerline. The northwest end of Ky. 2541 ties into existing US 23 north of Greenup, which is approximately 800 feet to the southeast of the project beginning. This alignment will replace the bridge at its existing location. A 20 M.P.H. design speed will be utilized; with two twelve foot wide lanes and four-foot shoulders.

This alignment begins south of the existing railroad crossing, tying into the existing roadway without changes to the crossing. The roadway then diverges to the west of the existing pavement to allow an improved approach to the bridge, crossing the Little Sandy River at the same location as the existing. The new roadway will tie into the existing pavement before its intersection with Cherry Street and Main Street of Greenup. Vertical alignment of the bridge will be raised so that the existing opening can be maintained. The new bridge configuration will be a two span arrangement, with the pier being out within the main channel. The beginning of the bridge is at station 13+25.00, with the centerline of the river pier at station 15+75.00, and the end of the bridge at station 18+25.00.

The project begins at Station 8+90, and ends at 20+05 for a project length of 0.2 miles. Additionally, portions of the intersecting road, which serves as an entrance to a logging facility for loading barges, will be realigned. This road realignment will occur from Station 48+00 to 50+00, where it intersects the mainline left of station 12+95. The proposed roadway alignment can be seen on the Project Location Map in Appendix A of this report.

According to the design plans there is only a very small amount of roadway excavation on the project, therefore, approximately 12,000 cubic yards of borrow will be necessary to construct the embankments.

2.0 TOPOGRAPHY AND DRAINAGE

The Little Sandy River empties into the Ohio River just to the left of the project centerline in Greenup. Relatively level alluvial valleys characterize the topography. Embankment material will be required to establish the proposed grades, as there are very minor excavations on the project. The largest embankment for the project will occur near the spill-thru slope for End Bent 2 near station 18+25, where a total fill height of approximately 19 ft. is planned at the left shoulder.

3.0 GEOLOGY

Bedrock formations underlying the Quaternary-aged alluvial deposits of the Little Sandy River have been mapped as Mississippian-aged Borden Formation siltstone, sandstone, and shale.

Siltstone in the Borden formation bedrock is typically found to be greenish gray to light gray, weathering to speckled brownish gray. Often, there is no weakness between bedding planes in this material. Sandstone is typically found to be very light gray to light greenish gray, fine grained, and occurring in 1 to 3 ft. thick layers, parted by 1 to 2 inch shale partings to thin layers. Shale is typically light greenish gray, and present as partings to thin layers between sandstone and/or siltstone layers.

Embankments across the site will be constructed over existing alluvial deposits. Some foundation problems are expected within the recent very soft, unconsolidated alluvial deposits.

Drainage at the site is to the Little Sandy River and Ohio River, which intersect north of the existing and proposed bridge.

Faults are not anticipated on this project. The regional dip is less than 0.5% per mile, west to East.

4.0 DRILLING AND SAMPLING

4.1 General

A subsurface investigation plan was developed by H. C. Nutting Company, following review of the project plans and cross-sections provided by Vaughn & Melton. The plan was submitted to the KYTC Geotechnical Branch on April 12, 2006 for review. The plan was approved with minor changes and a preliminary geotechnical meeting was held at the site on May 3, 2006. The subsurface exploration includes undisturbed and disturbed sample borings, rock cores, and rock soundings. Drilling was performed using a track-mounted drill rig. Drilling began on July 6, 2006 on a barge for the center Pier

No. 1 using a track mounted 45 C drilling rig. Since the tug boat pushing the barge did not have clearance under the bridge, holes to the right of centerline in the bay area where the west abutment spill thru embankment slope will toe out into the river, were not accessible. This area was composed of very soft unconsolidated river sediments under two feet of water at the time of the drilling. Due to the weak saturated unconsolidated soils encountered within the riverbed, we did not obtain much information from the standard testing and sampling procedures, therefore, the decision was to perform Cone Penetrometer Tests (CPT) from the existing bridge deck.

The CPT soundings were completed utilizing a truck-mounted Geoprobe rig. A Cone Penetration Test (CPT) involves pushing an instrumented 10-cm^2 pieozocone into the ground and measuring cone tip resistance, friction sleeve resistance, and excess pore pressure continuously as the cone is advanced, in accordance with ASTM Standard Method D5778.

Cone Penetrometer Tests (CPT) was performed in October 18, 2006, through the existing bridge deck at stations 14+00 and 14+46. On November 21, 2006 we returned to the site at the east abutment spill through slope, where similar weak saturated soils existed, and performed CPT 25' Lt. of station 17+70 and 60' Lt. of station 17+75. All borings, and CPT locations were staked by KTC, District 9, Design Office who provided us with the ground surface elevations, Latitude and Longitude information. The drilling operations were monitored by a Project Engineer and/or Project Geologist from H. C. Nutting Company.

4.2 Rock Core Borings

Rock core borings were performed at each of the end bent locations for the proposed bridge replacement, as well as the proposed pier location. Upon completion of the core holes, the rock core was transported to a storage facility at the H. C. Nutting office and logged by a geologist. The geologist determined the rock types, and the rock disintegration zone (RDZ) for each of the rock core borings. Additionally, the percent recovery and rock quality designation (RQD) was determined for each rock core run. Per the KTC standards, the Kentucky RQD is determined by the sum of all rock core pieces longer than 4 inches that are not easily broken by hand pressure divided by the total length of the core run. Representative samples of the rock core were tested for jar slake durability (JS), slake durability index (SDI) and unconfined compression strengths.

The rock types as determined by the project geologist were predominate siltstones, sandstones, and shales. The detailed bedrock descriptions are presented on the geotechnical drawings located in Appendix B.

4.3 Disturbed Sample Borings

Disturbed sample borings were performed along the roadway alignment at approximate 150 to 200 ft. intervals to determine the overburden thickness, moisture and soil type. These disturbed sample borings were preformed with a track-mounted drill rig, using 4-inch diameter continuous flight augers. The auger cuttings were monitored by the project engineer with particular attention to the soil type, color, texture, moisture content, and consistency of the material brought to the surface. Representative bag samples of each soil type were obtained and submitted to the H. C. Nutting geotechnical laboratory in Cincinnati for testing as specified in the KTC Geotechnical Manual. Jar samples of the soil overburden were also obtained to determine the natural moisture content.

4.4 Undisturbed Sample Borings

Undisturbed soil samples were obtained using thin-walled tubes and Standard Penetration Tests (SPT) at critical embankment locations. The purpose of the undisturbed samples is to evaluate the actual in-situ soil strength parameters to aid in the engineering analyses. Selected thin-walled tube samples were subject to consolidated-undrained triaxial strength tests, unconsolidated-undrained triaxial strength tests, unconfined compressive strength tests, engineering classification, and consolidation tests. Classification tests or wash gradations were performed on SPT samples.

5.0 LABORATORY TESTING AND RESULTS

5.1 General

The laboratory tests were conducted in our Cincinnati Laboratory by H.C. Nutting Company utilizing the specified AASHTO or Kentucky Methods. The results of the laboratory tests are depicted graphically on the soil profile, embankment stability sections, and subsurface data sheets located in Appendix C.

5.2 Slake Durability Index (SDI) and Jar Slake Testing (JS)

SDI and Jar Slake tests were performed on representative bedrock samples to provide an indication of the weathering potential of the bedrock. Both jar slake and SDI tests were performed on shale or siltstone samples. The JS and SDI tests were performed on 5 to 10 ft. intervals throughout the bedrock column. The Kentucky Transportation Cabinet separates shale into four categories for design purposes, depending on the SDI and Jar Slake values. The bedrock categories are listed in Table 2 below.

Table 2 – KYTC Shale Classifications		
Classification	**SDI (%)**	**Jar Slake Index**
Durable	95 to 100	6
Non-Durable, Class I	80 to 94	4 or 5
Non-Durable, Class II	50 to 79	3 or 4
Non-Durable, Class III	0 to 49	1 or 2

5.3 Disturbed Soil Testing

Three representative bag samples obtained from the disturbed soil borings were subject to soil classification including grain size distributions with hydrometer, Atterberg limits and specific gravity. Both the Unified Soil Classification System (USCS) and the AASHTO classification system were applied to the soil samples. According to the design plans there is only a very small amount of excavation on the project, therefore, disturbed soil bag samples were treated as fill samples and CBR tests were not performed.

The overburden soils within the project area are generally cohesive and classified as CL low plasticity clay, per the Unified Soil Classification System (USCS). Two of the three tested soil samples classified as A-6, and the remaining bag sample was classified as an A-4. The soil classifications are consistent with those identified in the USDA Soil Survey. The overburden soils are typically moist to saturated, with natural moisture contents ranging from 23 to 33%, averaging 27%.

The soils on the project are very silty in nature and therefore, very moisture sensitive. Subgrade construction problems are anticipated to occur throughout the shallow fill areas. In general, the natural moisture contents are above or near the Plasticity Limit. The topsoil thickness is generally less than 12 inches.

5.4 Undisturbed Soil Testing

Undisturbed Shelby Tube Samples and/or Standard Penetration Tests (SPT) were obtained from critical embankment sections and subjected to strength testing. The undisturbed soil samples were extruded, trimmed into six inch samples, visually classified and a pocket penetrometer performed to represent the sampling interval. A laboratory testing program was assigned to the thin-walled tubes and approved by the KTC Geotechnical Branch. The laboratory testing program consisted of unconfined compression tests, consolidated-undrained triaxial testing, unconsolidated-undrained triaxial testing and engineering classification. The results of the undisturbed soil testing are presented on the appropriate embankment stability and subsurface data sheets located in Appendix C.

Classification testing was performed on each of the undisturbed thin-walled tube samples. Classifications and/or wash gradations were performed on the SPT samples. The samples from the thin-walled tubes were classified as follows: 12 CL's, 2 ML's, and a CL-ML. as per the Unified Soil Classification System (USCS). The samples from the SPT were classified as follows: 7 SP-SM's, 9 SM's, 5 SP's, 4 CL's, 2 GP-GM's, and 2 ML's.

5.4.1 Unconfined Compressive Strength Testing

Nine (9) unconfined compressive strength tests were performed on thin-walled tube samples from critical embankment foundation areas. The purpose of the unconfined compressive strength testing is to estimate the short-term (undrained) shear strength of the soil. The unconfined compression tests yielded strengths ranging from 0.66 tsf to 3.20 tsf. The results are located in Appendix C on the embankment stability sections and subsurface data sheets.

5.4.2 Unconsolidated-Undrained Triaxial Strength Testing

Three (3) unconsolidated-undrained triaxial compression strength tests were performed on thin-walled tube samples from critical embankment foundation areas. These tests are typically performed on soils with low cohesion (high silt, low clay content), samples obtained below the water table and at depths greater than 20'. The purpose of these tests is to estimate the short-term (undrained) shear strength of the soil. Test results yielded strengths of 0.35 tsf, 1.12 tsf, and 1.14 tsf.

5.4.3 Consolidated-Undrained Triaxial Testing with Pore Pressure Measurements

In addition to the unconfined compressive testing, two (2) consolidated-undrained triaxial tests with pore pressure measurements were performed on representative thin-walled tube samples at embankment foundation locations. The consolidated-undrained triaxial tests are utilized to determine the long-term effective stress parameters of the overburden soils for the embankment slope stability analyses after the excess pore pressures have dissipated. The results of the consolidated-undrained triaxial testing are presented in Appendix C on the embankment stability section sheets.

5.4.4 One-Dimensional Consolidation Testing

Two (2) consolidation tests were performed on overburden soil samples in order to evaluate the settlement potential under the proposed embankment loadings.

These overburden soil test samples were obtained at the proposed east bridge abutment approach embankment and embankment station 12+70. The results of the consolidation tests are summarized in Table 1 below.

Table 1: Consolidation Test Results

Station	Offset	Depth (ft.)	Classification USCS/AASHTO	p_o' (tsf)	p_c' (tsf)	C_c	C_r	e_o
12+70	12.4' Lt.	27.5-29.5	CL A-4 (7)	1.2	3.66	0.25	0.05	0.860
18+00	60' Lt.	7-9	CL A-6 (13)	0.47	5.4	0.27	0.04	0.841

6.0 ENGINEERING ANALYSES

6.1 Embankment Slope Stability Analyses

Slope Stability Analyses were performed on the West Approach Embankment (Station 13+50 and on the East Approach Embankment (Station 18+00) in February 2007.
Two embankment sections, as shown on Table II, were analyzed for short-term, long-term and long term rapid drawdown global slope stability. The analyses were conducted using the computer program PCSTABL5 (computer analysis for general slope stability problems) and STED, version 6.5. The PCSTABL5 program was developed at Purdue University and utilizes the method of slices to perform slope stability analyses. The modified Bishop method was chosen to compute factors of safety for rotational failure surfaces. The slope stability program was used to investigate three cases: Case A – short-term (end-of-construction) conditions with total strength parameters, and Case B – long-term conditions with effective strength parameters and Case C – long term rapid drawdown.

The soil and bedrock parameters used in the stability analyses were determined from the laboratory test data and our experience with similar site soils/bedrock. The embankments for the project will be constructed with off-site soils and limestone rock borrow. The strength parameters for the new roadway embankment were estimated from Section GT-601-2 of the KTC Geotechnical Manual.

The KYTC Geotechnical Manual presents target factors of safety for embankment stability in Section GT601-3. Table 2 below outlines the required minimum safety factor for slope stability.

Table 2: KTC Embankment Slope Stability Target Safety Factors			
	Short Term	**Long Term**	**Rapid Drawdown**
Roadway Embankments	1.1-1.3	1.4-1.6	1.0-1.2

In performing the embankment slope stability analyses for this project, the following methodology was followed:

1. The subsurface strata and profile was developed, and then strength parameters were assigned at the section by referencing and using the following:
    a. Boring logs and subsurface description
    b. Natural moisture content
    c. Classification testing
    d. Strength testing and standard penetration test results
    e. Determination of water table, 50 and 100 year flood elevations
    f. Determination of bedrock surface
    g. Pocket penetrometer readings
    h. From long term experience

2. Foundation strength parameters were assigned for short term (total stress) and long term (effective stress) based on items 1a through 1h.

3. Embankment strength parameters were assigned for short term and long term based on the lab test data, long term experience, and per review of several geotechnical publications and books, which included item GT-601-2 of the Kentucky Geotechnical Manual. These publications and books, along with experience were also used in assigning parameters to the foundation soils, when laboratory test data was insufficient.

The strength parameters used for the embankments and for the bedrock for the analyses performed in February 2007 are as follows:

	Short Term			**Long Term**		
	Y (PCF)	**C (PCF)**	**Φ°**	**Y (PCF)**	**C (PCF)**	**Φ°**
Soil Embankment	120	1000	0	120	200	20
Rock Embankment	130	0	35	130	0	38
In Situ Bedrock	140	4000	0	140	4000	0

Table 3 below summarizes the results of the embankment stability analyses.
Stability analyses performed in February 2007, assuming different slope configuration and different types of embankment materials, indicated the slopes will not be stable as designed.

Table 3: Slope Stability Analyses

Station	Slope Configuration	Height of Fill	Type of Fill Material	Factor of Safety		
				Short Term	Long Term	Long Term Rapid Drawdown
13+50 West Approach Spill Thru Slope	2H:1V	16'	Rock	1.0	1.5	0.6
Spill Thru Slope	3H:1V	16'	Rock	0.9	2.1	1.0
End Bent No.I	2H:1V	16'	Rock	1.2	1.9	1.0
18+00 East East Approach Spill Thru Slope	2H:1V	20'	Soil	1.7	1.5	0.8
Spill Thru Slope	2H:1V	20'	Rock	1.6	1.4	0.6
Spill Thru Slope	3H:1V	20'	Rock	1.6	2.2	1.0
End Bent No. 2	3H:1V	20'	Rock	1.2	1.6	1.0
KTC Target Safety Factors				1.2-1.4	1.6-1.8	1.0-1.2

After reviewing our analyses, the KTC held a meeting on May 4, 2007, to discuss the options in order to stabilize the slopes. The following options were proposed: ground modification, breast wall abutments, excavation of soft materials and replacement with rock, bridge approach slab, and lengthen the bridge. After obtaining estimates for all of the options, the KTC decided to lengthen the bridge. Therefore, the contract with Vaughn and Melton, and H.C. Nutting Co. was modified to include the extra work for revising design plans. The contract modification was approved on August 29, 2008 and the bridge was extended approximately 25' on each end. This bridge extension of the east and west abutments allowed the spill – thru slopes to toe out on dry land in-lieu of the river where the soft unconsolidated saturated soil was located.

Additional drilling operations were begun in September 2008. These included undisturbed sample borings at station 12+70, 12.4' Lt., and station 17+90, 31' Lt., for the embankment slope stability analyses. Undisturbed soil borings and rock cores for the revised end bent locations were obtained at station 13+25 9.5' Lt., and station 18+25 7' Lt. of centerline. Rockline soundings were obtained for the revised end bent locations at stations 13+25 20' Rt. and station 18+24 6' Rt. of centerline. The testing as previously described in sections 4.0 and 5.0 was performed on the soil and rock samples. Refer to Appendix C for the embankment stability sections and subsurface data sheets.

Slope stability analyses methodology, as described previously in this section, was followed for stations 12+ 50 and the east abutment embankment spill – thru slope at station 18+00. Slope stability analyses were not performed for the west abutment embankment spill – thru slope because due to the bridge extension the embankment height (approximately 6') was not sufficient to warrant any analyses.

The strength parameters used for the embankments and for the bedrock for the analyses performed in December 2008 are as shown below. Refer to the embankment stability section sheets in Appendix C for the foundation soil parameters. The drawdown was from the 100 year flood elevation 542.6 to the normal pool of 514.0.

	Short Term			Long Term & Rapid Drawdown		
	Y (PCF)	C (PCF)	Φ°	Y (PCF)	C (PCF)	Φ°
Soil Embankment	130	1500	0	130	200	28
Rock Embankment	130	0	35	130	0	38
In Situ Bedrock	155	8000	0	155	8000	0

Table 4 below summarizes the results of the embankment stability analyses performed in December 2008.

Table 4: Slope Stability Analyses Results

Station	Slope Configuration	Height of Fill	Type of Fill Material	Factor of Safety		
				Short Term	Long Term	Long Term Rapid Drawdown
12+50	2H:1V	13’	Soil	3.7	2.1	N.A.
18+00 East Approach Spill Thru Slope	2H:1V	19’	Soil	2.5	1.8	0.999
KTC Target Safety Factors				1.2-1.4	1.6-1.8	1.0-1.2

Since the factor of safety for the long term rapid drawdown at the east approach embankment spill – thru slope was below the KTC target value additional slope stability analyses were performed only for this section.

Revision No. 1

Utilizing a rock embankment with the foundation soil strength parameters the same as above, drawdown was from the 100 year flood elevation 542.6 to the normal pool of 514.0 (Normal Pool), and the embankment parameters are as follows:

	Short Term			Long Term & Rapid Drawdown		
	Y (PCF)	C (PCF)	Φ°	Y (PCF)	C (PCF)	Φ°
Rock Embankment	140	0	38	140	0	38
In Situ Bedrock	155	8000	0	155	8000	0

Please note that all of the safety factors decreased. In particular the long term rapid drawdown safety factor decreased from the original factor of safety of 0.999 to 0.908 for

revision no. 1. Table 5 below summarizes the results of the embankment stability analyses using the rock embankment for revision number 1.

Table 5: Slope Stability Analyses Results

Station	Slope Configuration	Height of Fill	Type of Fill Material	Factor of Safety		
				Short Term	Long Term	Long Term Rapid Drawdown
18+00 East Approach Spill Thru Slope	2H:1V	19'	Rock	2.0	1.7	0.908
KTC Target Safety Factors				1.2-1.4	1.6-1.8	1.0-1.2

Revision No. 2

Utilizing a rock embankment with the foundation soil strength parameters the same as above, drawdown was from the 50 year flood elevation 540.0 to the normal pool of 514.0 and the embankment parameters are as follows:

Short Term				**Long Term & Rapid Drawdown**		
	Y (PCF)	C (PCF)	Φ°	Y (PCF)	C (PCF)	Φ°
Rock Embankment	140	0	45	140	0	45
In Situ Bedrock	155	8000	0	155	8000	0

Please note that the rapid drawdown safety factor 0.908 for revision 1 increased to 0.988 using the higher phi angle of 45 degrees for the rock embankment.

Table 6 below summarizes the results of the embankment stability analyses for revision number 2.

Table 6: Slope Stability Analyses Results

Station	Slope Configuration	Height of Fill	Type of Fill Material	Factor of Safety		
				Short Term	Long Term	Long Term Rapid Drawdown
18+00 East Approach Spill Thru Slope	2H:1V	19'	Rock	2.0	2.1	0.988
KTC Target Safety Factors				1.2-1.4	1.6-1.8	1.0-1.2

Revision No. 3

Utilizing a rock embankment and a three (3') undercut and rock replacement of the soil embankment beginning at the toe of the spill –thru and extending 30' up station with the foundation soil strength parameters the same as above, drawdown from the 50 year flood elevation 540.0 to the normal pool of 514.0 and the embankment parameters are as follows:

	Short Term			Long Term & Rapid Drawdown		
	Y (PCF)	C (PCF)	Φ°	Y (PCF)	C (PCF)	Φ°
Rock Embankment	140	0	45	140	0	45
3' X 30' Undercut Toe of Embankment	140	0	45	140	0	45
In Situ Bedrock	155	8000	0	155	8000	0

Please note that the rapid drawdown safety factor 0.988 of revision 2 increased to 1.088 using the 3' undercut and rock replacement. Table 7 below summarizes the results of the embankment stability analyses using the 3' undercut and rock replacement .

Table 7: Slope Stability Analyses Results

Station	Slope Configuration	Height of Fill	Type of Fill Material	Factor of Safety		
				Short Term	Long Term	Long Term Rapid Drawdown
18+00 East Approach Spill Thru Slope	2H:1V	19'	Rock	1.9	2.2	1.088
KTC Target Safety Factors				1.2-1.4	1.6-1.8	1.0-1.2

Refer to Table 8 for a comparison of the factors of safety with the revisions, for the slope stability analysis at station 18+00.

Table 8: Comparison of Slope Stability Analyses

Type of Embankment	Short Term	Long Term	Rapid Drawdown
Soil Embankment	2.5	1.8	0.999
Rock Embankment Rev. 1 Phi= 38deg.	2.0	1.7	0.908
Rock Embankment Rev. 2 Phi= 45 deg	2.0	2.1	0.988
Rock Embankment Rev. 3 3' Undercut	1.9	2.2	1.088

Per review by KTC Geotechnical Section, it is recommended that a rock embankment with a 3 ft. undercut and replacement at the toe, with rock replacement, be utilized for the embankment at End Bent 2.

6.2 Settlement Analyses

Settlement analyses were performed at stations 12+50, and 18+00 (east abutment approach embankment). Settlement analyses were not performed for the west abutment approach embankment due to the bridge extension the embankment height (approximately 6') was not sufficient to warrant any analyses.
To perform the analyses, the subsurface materials were arranged into stratas in consideration of soil type and classification, natural moisture, and strength. Compression indices were assigned to each stratum, and the potential amount of settlement to occur due to the new embankment loading, was computed.

The consolidation parameters and soil strength parameters were obtained from the laboratory test results performed within the project, per data available in geotechnical publications and reference books, and per our long term experience. Two consolidation tests were performed on representative samples obtained in borings 20 and 24. The consolidation test results per the samples obtained in borings 20 and 24 are shown in the following table.

Consolidation Test Results

Hole No.	Depth (ft.)	LL	PI	γ_{dry} (pcf)	W%	P_c (tsf)	C_c	C_r	e_{on}
24	7-9	36	14	92.9	28.3	5.4	0.27	0.04	0.841
20	27.5-29.5	30	8	92.7	29.7	3.6	0.25	0.05	0.860

Time rate of settlement analyses were also performed at station 12+50 and station 18+00, to estimate the amount of time required for 90% consolidation to occur. Coefficient of consolidation parameters were obtained from laboratory test results, and per published data.

The time rate of settlement computation is highly dependent on the length or number of drainage paths. The length or number of drainage paths is typically difficult to determine accurately, and as a minimum, 2 way drainage is assumed as a starting point in the analysis. It has been our long term experience that 90% consolidation occurs much more rapidly than the values which are computed.

At station 12+50, it is presumed that the existing embankment will be left in place. At centerline and at the left shoulder, the new embankment height will be approximately 12 ft. At the right shoulder it is presumed that the new embankment will be constructed over the existing embankment. The new embankment height will be about 4 ft. at the right shoulder.

Settlement analysis was performed to determine the approximate amount of settlement to occur at centerline, under an embankment loading of 12 ft., and at the right shoulder, where the new embankment height will be about 4 ft.

The estimated settlement to occur due to the new embankment loadings at centerline to the right shoulder is 3.9” and 0.9”, respectively. The estimated time to achieve 90% consolidation was computed to be 2 years, assuming double drainage.
Settlement stakes/pins could be placed every 25 to 50 ft. along the left and right shoulder and centerline within a distance of 100 ft. each way of station 12+50, following embankment construction. Elevations of the top of the pins would be determined at least once a week. Paving operations would not begin until the measured settlement is equal to or less than 1/8 inch over a 7 day week, for 3 consecutive weeks, or as determined by the engineer.

At station 18+00, we performed settlement analysis to determine the amount of settlement to occur at centerline and at the left shoulder. A portion of the new embankment will be constructed over the existing embankment. We have assumed that at least 5 ft. of the existing embankment, with the assumption that the existing embankment has marginal backfill. The height of the new embankment at centerline (with a 5 ft. undercut) and at the left shoulder (no undercut necessary) will be 11’ and 19 ft., respectively.

The computed settlement at centerline is 2.3 inches, and 3.1 inches at the left shoulder. At the right shoulder, the computed settlement to occur is 1.7 inches.

A time rate of settlement analysis was performed at station 18+00, to calculate the estimated time required for about 90% of the settlement to occur. Our analysis indicates that 90% of the settlement will occur in about 2.3 months, or essentially during the construction period.

6.3 Negative Skin Friction Analyses

At the east bridge abutment, Station 18+00, settlement analysis was performed, and the computed settlement ranged from 2.3” to 3.1” from centerline to the shoulder. The time rate of settlement for 90% consolidation to occur was computed to be 2 to 3 months. Due to consolidation of the foundation soils resulting from the placement of new fill, and groundwater fluctuations, including flood events, the soil moves down with respect to the pile, and the resulting side shear acts downward on the pile. This is negative skin friction, also called “downdrag”.

The principal effect of downdrag is to increase the axial load on the pile. Review of published literature suggests that the amount of relative settlement between the soil and

pile that is necessary to mobilize negative skin friction is about ½ in. Settlements associated with consolidation of foundation soils will occur as new fill embankments are being built and for some period of time after their completion. There is an effect of time in all negative skin friction problems. If the pile is driven before soil settlement begins, downdrag will have its maximum effect. Conversely, if all settlement is complete before the pile is driven, there will be minimal loss of capacity (or increase in pile movement) due to downdrag.

Per our analysis, the computed downdrag for an HP 12X53 and HP 14X73 H-pile is 38 tons and 45 tons respectively.

The foundation design consists of H-piles bearing on bedrock; therefore the structural capacity of the pile will govern. The allowable structural design capacity is 0.25 Fy, which means that a safety factor of 4.0 is used The allowable structural capacity for grade 50 steel for the HP 12X53 and HP 14X73 pile is 780 kips (390 tons) and 1075 kips (537 tons). The ultimate structural capacity is 1560 tons and 2148 tons. These safety factors also provide a suitable margin of safety in the event that some negative skin friction was to develop. Conversely, a time waiting period of about 3.0 months could occur following embankment construction and prior to pile driving to let most of the settlement to occur, therefore, reducing the downdrag on the piles.

6.4 Pile Foundation Driveability Analyses

6.4.1 West Abutment (End Bent 1) – Sta. 13+43.00

Wave equation analyses were performed at the west abutment locations in which new piles will be driven for the bridge improvement. The wave equation analyses were performed to determine the necessary hammer energy to drive the piles to bedrock without over-stressing the piles or encountering excessive blow counts. The analyses were performed utilizing the GRL-WEAP program version 98-2. The wave equation analyses were performed for both 12x53 and 14x73 H-piles, grade 50 ksi steel. The driving stresses in compression or tension should not exceed $0.9F_y$, where F_y is the yield strength of the steel. Thus, the driving stress in the piles should not exceed 45 ksi.

At the west abutment, borings 11 and 21 and soundings 10 and 22 were used to determine the subsurface profile. Bedrock is encountered at approximately 56 ft. below existing grade, near elevation 476+/-. Refer to the Geotechnical Drawings in Appendix C. for the subsurface data sheets. It was assumed in the analysis that the top of pile was 6 ft. below the ground surface, whereby a 50 ft. long pile is required to be driven to bedrock.

It is recommended that H-piles, driven to practical refusal, bearing on the top of bedrock, be used to support the west abutment. Both HP 12x53 and HP 14x73 piles were considered. The allowable capacity per pile will be based on the allowable structural capacity of the steel pile. Allowable load capacities of 97 tons and 134 tons can be developed for HP 12x53 and HP 14x73 piles respectively, on the basis of an allowable stress of 12.5 ksi ($0.25F_y$, with F_y = 50 ksi) acting over the cross sectional area.

A pile capacity analysis was performed to determine the supporting capacity of the subsurface soils and bedrock. The analysis was performed using the computer program "Driven." The results of this analysis are attached, which confirm that the piles need to be driven to bear on bedrock.

In performing the Wave Equation Analysis, the following assumptions were made to perform the drivability analysis:

- Efficiency of the Pile Hammer (driving system) = 80%
- Shaft Resistance = 10%
- Pile Driving Refusal Criteria = 20 blows per inch.

We consulted KTC concerning the size of the pile hammer typically used. It is our understanding that KTC typically has experienced hammers in the rated energy range of 18 ft. – kips to 40 ft. – kips as being suitable. We have performed 7 case studies, which include 2 case studies using an HP12x53 pile, and 5 case studies using an HP14x73 pile for various hammer sizes. The acceptability of the hammer will depend on:

1. Maximum driving stress is less than 45 ksi.
2. Maximum blow count for practical refusal is 20 blows per inch.
3. Hammer will drive pile to ultimate capacity. The allowable structural capacities for HP12x53 and HP14x74 are 194 kips and 268 kips respectively. It is assumed that the minimum ultimate resistance to be developed for each pile during driving will be at least twice (SF = 2.0) the design capacity, or 388 kips (HP12x53), and 536 kips (HP14x74).

The following are the GRLWEAP analysis results for the purpose of making a preliminary size estimate of the pile hammer/energy rating.

HP 12X53 EST. ULTIMATE STRUCTURAL CAPACITY = 388 KIPS							
Hammer ID	**Hammer Make**	**Type**	**Energy/Power (ft.-kips)**	**Ultimate Capacity**	**Maximum Stress (ksi)**	**Blow Count B/Inch**	**Remarks**

121	ICE 422	CED	23.120	400	32.518	16.5	
123	ICE 520	CED	30.369	400	31.476	14.5	

HP 14X73 EST. ULTIMATE STRUCTURAL CAPACITY = 563 KIPS							
Hammer ID	**Hammer Make**	**Type**	**Energy/ Power**	**Ultimate Capacity**	**Maximum Stress (ksi)**	**Blow Count B/Inch**	**Remarks**
122	ICE 440	CED	18.560	300	20.179	15.88	Capacity Not Achieved
				400	21.073	79.58	Blows Too High/Capacity Not Achieved
121	ICE 422	CED	23.120	500	29.697	45.9	Blows Too High/Capacity Not Achieved
123	ICE 520	CED	30.369	500	28.791	62.0	Blows Too High/Capacity Not Achieved
124	ICE 640	CED	40.620	500	33.026 33.450	9.5 39.5	This Hammer Will Work
125	ICE 660	CED	51.627	500	38.404	7.48	This Hammer Will Work
125	ICE 660	CED	51.627	600	39.180	21.3	This Hammer Will Work

6.4.2 East Abutment (End Bent 2) – Sta. 18+03.00

Wave equation analyses were performed at the east abutment locations in which new piles will be driven for the bridge improvement. The wave equation analyses were performed to determine the necessary hammer energy to drive the piles to bedrock without over-stressing the piles or encountering excessive blow counts. The analyses were performed utilizing the GRL-WEAP program version 98-2. The wave equation analyses were performed for both 12x53 and 14x73 H-piles, grade 50 ksi steel. The driving stresses in compression or tension should not exceed $0.9F_y$, where F_y is the yield strength of the steel. Thus, the driving stress in the piles should not exceed 45 ksi.

At the east abutment, borings 16 and 25 and soundings 15 and 26 were used to determine the subsurface profile. Bedrock is encountered at approximately 45 ft. below existing grade, near elevation 489+/-. Refer to the Geotechnical Drawings Appendix C for the subsurface data sheets. It was assumed in the analysis that the top of pile was 6 ft. below the ground surface, whereby a 39 ft. long pile is required to be driven to bedrock.

It is recommended that H-piles, driven to practical refusal, bearing on the top of bedrock, be used to support the east abutment. Both HP 12x53 and HP 14x73 piles were considered.

A pile capacity analysis was performed to determine the supporting capacity of the subsurface soils and bedrock. The analysis was performed using the computer program "Driven." The results of this analysis are attached, which confirm that the piles need to be driven to bear on bedrock.

In performing the Wave Equation Analysis, the following assumptions were made to perform the drivability analysis.

- Efficiency of the Pile Hammer (driving system) = 80%
- Shaft Resistance = 10%
- Pile Driving Refusal Criteria = 20 blows per inch.

We consulted KTC concerning the size of the pile hammer typically used. It is our understanding that KTC typically has experienced hammers in the rated energy range of 18 ft. – kips to 40 ft. – kips as being suitable. We have performed 7 case studies, which include 4 case studies using an HP12x53 pile, and 3 case studies using an HP14x73 pile for various hammer sizes. The acceptability of the hammer will depend on:

4. Maximum driving stress is less than 45 ksi.
5. Maximum blow count for practical refusal is 20 blows per inch.
6. Hammer will drive pile to ultimate capacity. The allowable structural capacities for HP12x53 and HP14x74 are 195 kips and 268 kips respectively. It is assumed that the minimum ultimate resistance to be developed for each pile during driving will be at least twice (SF = 2.0) the design capacity, or 390 kips (HP12x53), and 536 kips (HP14x73).

The following are the GRLWEAP analysis results for the purpose of making a preliminary size estimate of the pile hammer/energy rating.

HP 12X53 EST. ULTIMATE STRUCTURAL CAPACITY = 390 KIPS

Hammer ID	Hammer Make	Type	Energy/ Power (ft.-kips)	Ultimate Capacity	Maximum Stress (ksi)	Blow Count B/Inch	Remarks
121	ICE 422	CED	23.120	400	34.9	11.6	This Hammer Will Work
123	ICE 520	CED	30.369	400	31.476	14.5	This Hammer Will Work
122	ICE 440	CED	18.560	300	24.5	31	Capacity Not Achieved
124	ICE 640	CED	40.620	500	41.8	16	This Hammer Will Work

HP 14X73 EST. ULTIMATE STRUCTURAL CAPACITY = 563 KIPS							
Hammer ID	Hammer Make	Type	Energy/ Power	Ultimate Capacity	Maximum Stress (ksi)	Blow Count B/Inch	Remarks
123	ICE 520	CED	30.369	500	29.9	37.0	Blows Too High/Capacity Not Achieved
124	ICE 640	CED	40.620	500 600	35.3 35.9	10.7 42.0	This Hammer Will Work
125	ICE 660	CED	51.627	600	41.0	21.2	This Hammer Will Work

6.5 Drilled Shaft Design

The interior bridge pier will be located within the river, at station15+75 CL. Two borings were drilled at the pier location in 2006, namely Hole No. 12 and Hole No. 14 at station 15+82, 16'LT and station 15+87, 24'RT, respectively.

The water depth at the borings was approximately 25 and 22.4 ft., considering pool stage at elevation 514. The overburden thickness above bedrock was 16.5 and 16.7 ft, consisting of very weak alluvial soils. Top of bedrock was encountered between elevations 474.1 to 476.3.

The bedrock consists primarily of siltstone and shale. Percent rock core recovery ranged from 96% to 100%. SDI test results ranged from 61 to 100. Unconfined compression tests ranged from 186 to 795 tsf.

It is recommended that the interior bridge pier be supported on a straight drilled shaft foundation system bearing within the bedrock. The shafts should be socketed several feet into the bedrock, below any upper weathered zones, to satisfy axial and lateral loadings. The piers should also be socketed deep enough in consideration of long term

scour effects on the bedrock. Consideration of long term scour effects on bedrock must also be considered in determining the bottom elevation of the drilled shafts.

The bearing elevation must take long term scour and erodability of the bedrock into concern. It is recommended that the shafts be placed sufficiently deep in the rock, so that they are not affected by scour over the long term. Scour is the process of displacing streambed materials as a result of the erosive action of water. For the usual conditions, the greatest scour should be expected to occur during the largest flood.

We have reviewed and researched several publications concerning scour at bridges. We have found very little information in the literature concerning scour of bedrock. Per publication “Evaluating Scour at Bridges” February 1993, by the U.S. Department of Commerce, the following is stated: “The current equations and methods for estimating scour at bridges are based primarily on laboratory research. Very little field data have been collected to verify the applicability and accuracy of the various design procedures for the range of soil conditions, stream flow conditions, and bridge designs encountered throughout the United States.” Per “ Bridge Substructure and Foundation Design”---“ scour is the most common cause of failure of highway bridges in the United States.”

The bedrock can become soft, scour and erode under the action of the river current. We do not know how much or the rate of scour of the bedrock will occur over the long term. In addition, there is an unknown of how deep local scour may become at the bridge pier by the formation of vortexes at the pier. The formation of vortexes results from the pileup of water at the upstream face of the pier and subsequent acceleration of flow around the nose of the pier. This vortex which develops at the upstream face is termed the horseshoe vortex. In addition to the horseshoe vortex around the base of the pier, there are vertical vortices downstream of the pier called the wake vortex. Both vortices remove material from the pier base region.

Erosion of rock is a process of progressive dislodgement. Bedrock consists of layers of material with joints and cracks between them. Water penetrating the joints and cracks creates hydrostatic pressure between pieces of rock. The water flowing over the rough boundary is accompanied by turbulence and eddies, resulting in fluctuating pressures at the boundary. The fluctuating pressures at the boundary, and the hydrostatic pressures in the cracks and joints causes a tugging and pulling at the rock.

These fluctuating net forces progressively dislodge the rock. Once a piece of rock is dislodged by the power of the flowing water, it is displaced. Fluctuating, pulsating forces cause the erosion of rock. These types of forces occur around bridge piers during flooding. We believe that the safe measure is to support the interior pier on a drilled shaft foundation well below the top of rock elevation.

The bedrock consists primarily of siltstone and shale. At boring 12 (S-3), drilled at Sta. 15+82, 16' Lt., the slake durability index results (SDI) performed on 10 samples ranged from 65 to 88, with the lowest SDI = 65, occurring within the top 2 ft. of the bedrock core. The average of the 10 SDI results is 79. The RQD ranges from 74 to 100, with the lowest RQD within the top few feet of core. Rock core recovery was consistently at 100%.

Within boring 14 (S-2), drilled at Sta. 15+87, 24' Rt., the SDI results performed on 9 samples ranged from 78 to 84. The RQD ranges from 61 to 97, with the two lowest results of 78 and 61 occurring within the top few feet of core. Rock core recovery ranged from 96% to 100%.

The percent of core recovery and the RQD suggest that scour may not be of concern; however, the SDI results indicate that the bedrock is scour susceptible, per the guidelines presented in the KTC Geotechnical Manual. It is our recommendation that the effects of scour be considered within the upper 5 ft. of the bedrock, within the drilled shaft foundation design.

The drilled shafts were evaluated for axial loading. Detailed analysis was performed in accordance with Publication No. IF 99-029 "Drilled Shafts: Construction Procedures and Design Methods", per the Federal Highway Administration. The detailed analysis is attached. The drilled shafts were evaluated for axial loading, and the resulting capacities and resistances are provided for Allowable Stress Design (ASD) in the attached design tables on pages 8 through 16. The total allowable axial capacity and allowable uplift capacity are provided. The capacities and resistances consider shaft diameters ranging from 4 ft to 8 ft., with analysis provided in 6 inch increments, that is, at 4 ft., 4.5ft., 5.0ft., etc.

The allowable uplift capacities provided within the tables do not include the weight of the drilled shaft.

The unconfined compressive strength per 18 tests run on the rock core obtained from the two river borings ranged from 186 to 795 tsf, for an average strength of 573 tsf. The three lowest test results were 186.4, 336.8, and 395.5 tsf. In our analysis to determine axial and uplift capacities, an unconfined compressive strength of 400 tsf was used.

Permanent casing will be required in the construction of the drilled shafts when penetrating into the bedrock. The plan details indicate that the cased portion of the shafts is to be 6-inches larger in diameter than the rock sockets. The top of bedrock was at elevation 474.1 and 476.3 within the river boring hole number 12 and hole number 14 respectively. No frictional resistance should be considered in the cased

portion of the shaft. The drilling contractor should use a casing which is 6” larger than the specified drilled shaft diameter, to allow for passage of the drilling tool of proper diameter during final excavation of the drilled shaft. For example, if the design diameter of the drilled shaft is 48”, then a 54” casing will likely be used.

The rigid casing should be sealed into the bedrock. This may require welded hardened steel teeth on the bottom of the casing to help achieve the permanent seal into the hard bedrock.

Allowable Stress Design (ASD)

DRILLED SHAFT AXIAL CAPACITY TABLE

Greenup County Bridge Over The Little Sandy River Ky. 2641
Pier No. 1

Rock Socket Diameter = 4.0 feet
Rock Socket Diameter = 48 inches

2/27/2009

Depth Below Permanent Casing FT.	Ultimate Unit Side Shear q_{ss} (ksf)	Ultimate Unit End Bearing q_{eb} (ksf)	Ultimate Side Shear Q_{ss} (kips)	Ultimate End Bearing Q_{eb} (kips)	Total Ultimate Axial Capacity Q_{ut} (kips)	Total Allowable Axial Capacity Q_{at} (kips)	Total Allowable Uplift Capacity Q_{up} (kips)
0.0							
1.0	22.6	190	284	2386	2670	890	66
2.0	22.6	190	568	2386	2954	985	133
3.0	22.6	190	852	2386	3238	1079	199
4.0	22.6	190	1135	2386	3521	1174	265
5.0	22.6	190	1419	2386	3805	1268	331
6.0	22.6	190	1703	2386	4089	1363	397
7.0	22.6	190	1987	2386	4373	1458	464
8.0	22.6	190	2271	2386	4657	1552	530
9.0	22.6	190	2555	2386	4941	1647	596
10.0	22.6	190	2839	2386	5225	1742	662
11.0	22.6	190	3122	2386	5508	1836	728
12.0	22.6	190	3406	2386	5792	1931	795
13.0	22.6	190	3690	2386	6076	2025	861
14.0	22.6	190	3974	2386	6360	2120	927
15.0	22.6	190	4258	2386	6644	2215	994
16.0	22.6	190	4542	2386	6928	2309	1060
17.0	22.6	190	4826	2386	7212	2404	1126
18.0	22.6	190	5109	2386	7495	2498	1192
19.0	22.6	190	5393	2386	7779	2593	1258
20.0	22.6	190	5677	2386	8063	2688	1325
						D (ft.) =	4.0
						FS =	3.0

Allowable Stress Design (ASD)

DRILLED SHAFT AXIAL CAPACITY TABLE

Greenup County Bridge Over The Little Sandy River Ky. 2641
Pier No. 1

Rock Socket Diameter = 4.5 feet
Rock Socket Diameter = 54 inches

2/27/2009

Depth Below Permanent Casing FT.	Ultimate Unit Side Shear q_{ss} (ksf)	Ultimate Unit End Bearing q_{eb} (ksf)	Ultimate Side Shear Q_{ss} (kips)	Ultimate End Bearing Q_{eb} (kips)	Total Ultimate Axial Capacity Q_{ut} (kips)	Total Allowable Axial Capacity Q_{at} (kips)	Total Allowable Uplift Capacity Q_{up} (kips)
0.0							
1.0	22.6	190	319	3019	3338	1113	74
2.0	22.6	190	639	3019	3658	1219	149
3.0	22.6	190	958	3019	3977	1326	224
4.0	22.6	190	1277	3019	4296	1432	298
5.0	22.6	190	1597	3019	4616	1539	373
6.0	22.6	190	1916	3019	4935	1645	447
7.0	22.6	190	2235	3019	5254	1751	522
8.0	22.6	190	2555	3019	5574	1858	596
9.0	22.6	190	2874	3019	5893	1964	671
10.0	22.6	190	3193	3019	6212	2071	745
11.0	22.6	190	3513	3019	6532	2177	820
12.0	22.6	190	3832	3019	6851	2284	894
13.0	22.6	190	4151	3019	7170	2390	969
14.0	22.6	190	4471	3019	7490	2497	1043
15.0	22.6	190	4790	3019	7809	2603	1118
16.0	22.6	190	5109	3019	8128	2709	1192
17.0	22.6	190	5429	3019	8448	2816	1267
18.0	22.6	190	5748	3019	8767	2922	1341
19.0	22.6	190	6067	3019	9086	3029	1416
20.0	22.6	190	6387	3019	9406	3135	1490
						D (ft.) =	4.5
						FS =	3.0

Allowable Stress Design (ASD)

DRILLED SHAFT AXIAL CAPACITY TABLE

Greenup County Bridge Over The Little Sandy River Ky. 2641
Pier No. 1

Rock Socket Diameter = 5.0 feet
Rock Socket Diameter = 60 inches

2/27/2009

Depth Below Permanent Casing FT.	Ultimate Unit Side Shear q_{ss} (ksf)	Ultimate Unit End Bearing q_{eb} (ksf)	Ultimate Side Shear Q_{ss} (kips)	Ultimate End Bearing Q_{eb} (kips)	Total Ultimate Axial Capacity Q_{ut} (kips)	Total Allowable Axial Capacity Q_{at} (kips)	Total Allowable Uplift Capacity Q_{up} (kips)
0.0							
1.0	22.6	190	355	3728	4083	1361	83
2.0	22.6	190	710	3728	4438	1479	166
3.0	22.6	190	1064	3728	4792	1597	248
4.0	22.6	190	1419	3728	5147	1716	331
5.0	22.6	190	1774	3728	5502	1834	414
6.0	22.6	190	2129	3728	5857	1952	497
7.0	22.6	190	2484	3728	6212	2071	580
8.0	22.6	190	2839	3728	6567	2189	662
9.0	22.6	190	3193	3728	6921	2307	745
10.0	22.6	190	3548	3728	7276	2425	828
11.0	22.6	190	3903	3728	7631	2544	911
12.0	22.6	190	4258	3728	7986	2662	994
13.0	22.6	190	4613	3728	8341	2780	1076
14.0	22.6	190	4967	3728	8695	2898	1159
15.0	22.6	190	5322	3728	9050	3017	1242
16.0	22.6	190	5677	3728	9405	3135	1325
17.0	22.6	190	6032	3728	9760	3253	1407
18.0	22.6	190	6387	3728	10115	3372	1490
19.0	22.6	190	6742	3728	10470	3490	1573
20.0	22.6	190	7096	3728	10824	3608	1656
						D (ft.) =	5.0
						FS =	3.0

Allowable Stress Design (ASD)

DRILLED SHAFT AXIAL CAPACITY TABLE

Greenup County Bridge Over The Little Sandy River Ky. 2641
Pier No. 1

Rock Socket Diameter = 5.5 feet
Rock Socket Diameter = 66 inches

2/27/2009

Depth Below Permanent Casing FT.	Ultimate Unit Side Shear q_{ss} (ksf)	Ultimate Unit End Bearing q_{eb} (ksf)	Ultimate Side Shear Q_{ss} (kips)	Ultimate End Bearing Q_{eb} (kips)	Total Ultimate Axial Capacity Q_{ut} (kips)	Total Allowable Axial Capacity Q_{at} (kips)	Total Allowable Uplift Capacity Q_{up} (kips)
0.0							
1.0	22.6	190	390	4511	4901	1634	91
2.0	22.6	190	781	4511	5292	1764	182
3.0	22.6	190	1171	4511	5682	1894	273
4.0	22.6	190	1561	4511	6072	2024	364
5.0	22.6	190	1951	4511	6462	2154	455
6.0	22.6	190	2342	4511	6853	2284	546
7.0	22.6	190	2732	4511	7243	2414	637
8.0	22.6	190	3122	4511	7633	2544	728
9.0	22.6	190	3513	4511	8024	2675	820
10.0	22.6	190	3903	4511	8414	2805	911
11.0	22.6	190	4293	4511	8804	2935	1002
12.0	22.6	190	4684	4511	9195	3065	1093
13.0	22.6	190	5074	4511	9585	3195	1184
14.0	22.6	190	5464	4511	9975	3325	1275
15.0	22.6	190	5854	4511	10365	3455	1366
16.0	22.6	190	6245	4511	10756	3585	1457
17.0	22.6	190	6635	4511	11146	3715	1548
18.0	22.6	190	7025	4511	11536	3845	1639
19.0	22.6	190	7416	4511	11927	3976	1730
20.0	22.6	190	7806	4511	12317	4106	1821
						D (ft.) =	5.5
						FS =	3.0

Allowable Stress Design (ASD)

DRILLED SHAFT AXIAL CAPACITY TABLE

Greenup County Bridge Over The Little Sandy River Ky. 2641
Pier No. 1

Rock Socket Diameter = 6.0 feet
Rock Socket Diameter = 72 inches

2/27/2009

Depth Below Permanent Casing FT.	Ultimate Unit Side Shear q_{ss} (ksf)	Ultimate Unit End Bearing q_{eb} (ksf)	Ultimate Side Shear Q_{ss} (kips)	Ultimate End Bearing Q_{eb} (kips)	Total Ultimate Axial Capacity Q_{ut} (kips)	Total Allowable Axial Capacity Q_{at} (kips)	Total Allowable Uplift Capacity Q_{up} (kips)
0.0							
1.0	22.6	190	426	5369	5795	1932	99
2.0	22.6	190	852	5369	6221	2074	199
3.0	22.6	190	1277	5369	6646	2215	298
4.0	22.6	190	1703	5369	7072	2357	397
5.0	22.6	190	2129	5369	7498	2499	497
6.0	22.6	190	2555	5369	7924	2641	596
7.0	22.6	190	2981	5369	8350	2783	696
8.0	22.6	190	3406	5369	8775	2925	795
9.0	22.6	190	3832	5369	9201	3067	894
10.0	22.6	190	4258	5369	9627	3209	994
11.0	22.6	190	4684	5369	10053	3351	1093
12.0	22.6	190	5109	5369	10478	3493	1192
13.0	22.6	190	5535	5369	10904	3635	1292
14.0	22.6	190	5961	5369	11330	3777	1391
15.0	22.6	190	6387	5369	11756	3919	1490
16.0	22.6	190	6813	5369	12182	4061	1590
17.0	22.6	190	7239	5369	12608	4203	1689
18.0	22.6	190	7664	5369	13033	4344	1788
19.0	22.6	190	8090	5369	13459	4486	1888
20.0	22.6	190	8516	5369	13885	4628	1987
						D (ft.) =	6.0
						FS =	3.0

Allowable Stress Design (ASD)

DRILLED SHAFT AXIAL CAPACITY TABLE

Greenup County Bridge Over The Little Sandy River Ky. 2641
Pier No. 1

Rock Socket Diameter = 6.5 feet
Rock Socket Diameter = 78 inches

2/27/2009

Depth Below Permanent Casing FT.	Ultimate Unit Side Shear q_{ss} (ksf)	Ultimate Unit End Bearing q_{eb} (ksf)	Ultimate Side Shear Q_{ss} (kips)	Ultimate End Bearing Q_{eb} (kips)	Total Ultimate Axial Capacity Q_{ut} (kips)	Total Allowable Axial Capacity Q_{at} (kips)	Total Allowable Uplift Capacity Q_{up} (kips)
0.0							
1.0	22.6	190	461	6300	6761	2254	108
2.0	22.6	190	922	6300	7222	2407	215
3.0	22.6	190	1384	6300	7684	2561	323
4.0	22.6	190	1845	6300	8145	2715	431
5.0	22.6	190	2306	6300	8606	2869	538
6.0	22.6	190	2767	6300	9067	3022	646
7.0	22.6	190	3228	6300	9528	3176	753
8.0	22.6	190	3689	6300	9989	3330	861
9.0	22.6	190	4151	6300	10451	3484	969
10.0	22.6	190	4612	6300	10912	3637	1076
11.0	22.6	190	5073	6300	11373	3791	1184
12.0	22.6	190	5534	6300	11834	3945	1291
13.0	22.6	190	5995	6300	12295	4098	1399
14.0	22.6	190	6457	6300	12757	4252	1507
15.0	22.6	190	6918	6300	13218	4406	1614
16.0	22.6	190	7379	6300	13679	4560	1722
17.0	22.6	190	7840	6300	14140	4713	1829
18.0	22.6	190	8301	6300	14601	4867	1937
19.0	22.6	190	8763	6300	15063	5021	2043
20.0	22.6	190	9224	6300	15524	5175	2152
						D (ft.) =	6.5
						FS =	3.0

Allowable Stress Design (ASD)

DRILLED SHAFT AXIAL CAPACITY TABLE

Greenup County Bridge Over The Little Sandy River Ky. 2641
Pier No. 1

Rock Socket Diameter = 7.0 feet
Rock Socket Diameter = 84 inches

2/27/2009

Depth Below Permanent Casing FT.	Ultimate Unit Side Shear q_{ss} (ksf)	Ultimate Unit End Bearing q_{eb} (ksf)	Ultimate Side Shear Q_{ss} (kips)	Ultimate End Bearing Q_{eb} (kips)	Total Ultimate Axial Capacity Q_{ut} (kips)	Total Allowable Axial Capacity Q_{at} (kips)	Total Allowable Uplift Capacity Q_{up} (kips)
0.0							
1.0	22.6	190	497	7307	7804	2601	116
2.0	22.6	190	993	7307	8300	2767	232
3.0	22.6	190	1490	7307	8797	2932	348
4.0	22.6	190	1987	7307	9294	3098	464
5.0	22.6	190	2483	7307	9790	3263	579
6.0	22.6	190	2980	7307	10287	3429	695
7.0	22.6	190	3477	7307	10784	3595	811
8.0	22.6	190	3973	7307	11280	3760	927
9.0	22.6	190	4470	7307	11777	3926	1043
10.0	22.6	190	4967	7307	12274	4091	1159
11.0	22.6	190	5463	7307	12770	4257	1275
12.0	22.6	190	5960	7307	13267	4422	1389
13.0	22.6	190	6457	7307	13764	4588	1505
14.0	22.6	190	6954	7307	14261	4754	1621
15.0	22.6	190	7450	7307	14757	4919	1738
16.0	22.6	190	7947	7307	15254	5085	1854
17.0	22.6	190	8444	7307	15751	5250	1968
18.0	22.6	190	8940	7307	16247	5416	2084
19.0	22.6	190	9437	7307	16744	5581	2200
20.0	22.6	190	9934	7307	17241	5747	2316
						D (ft.) =	7.0
						FS =	3.0

Allowable Stress Design (ASD)

DRILLED SHAFT AXIAL CAPACITY TABLE

Greenup County Bridge Over The Little Sandy River Ky. 2641
Pier No. 1

Rock Socket Diameter = 7.5 feet
Rock Socket Diameter = 90 inches

2/27/2009

Depth Below Permanent Casing FT.	Ultimate Unit Side Shear q_{ss} (ksf)	Ultimate Unit End Bearing q_{eb} (ksf)	Ultimate Side Shear Q_{ss} (kips)	Ultimate End Bearing Q_{eb} (kips)	Total Ultimate Axial Capacity Q_{ut} (kips)	Total Allowable Axial Capacity Q_{at} (kips)	Total Allowable Uplift Capacity Q_{up} (kips)
0.0							
1.0	22.6	190	532	8388	8920	2973	124
2.0	22.6	190	1064	8388	9452	3151	248
3.0	22.6	190	1596	8388	9984	3328	372
4.0	22.6	190	2129	8388	10517	3506	497
5.0	22.6	190	2661	8388	11049	3683	621
6.0	22.6	190	3193	8388	11581	3860	745
7.0	22.6	190	3725	8388	12113	4038	869
8.0	22.6	190	4257	8388	12645	4215	993
9.0	22.6	190	4790	8388	13178	4393	1118
10.0	22.6	190	5322	8388	13710	4570	1242
11.0	22.6	190	5854	8388	14242	4747	1366
12.0	22.6	190	6386	8388	14774	4925	1490
13.0	22.6	190	6918	8388	15306	5102	1614
14.0	22.6	190	7451	8388	15839	5280	1737
15.0	22.6	190	7983	8388	16371	5457	1861
16.0	22.6	190	8515	8388	16903	5634	1985
17.0	22.6	190	9047	8388	17435	5812	2111
18.0	22.6	190	9579	8388	17967	5989	2235
19.0	22.6	190	10112	8388	18500	6167	2359
20.0	22.6	190	10644	8388	19032	6344	2484
						D (ft.) =	7.5
						FS =	3.0

Allowable Stress Design (ASD)

DRILLED SHAFT AXIAL CAPACITY TABLE

Greenup County Bridge Over The Little Sandy River Ky. 2641
Pier No. 1

Rock Socket Diameter = 8.0 feet
Rock Socket Diameter = 96 inches

2/27/2009

Depth Below Permanent Casing FT.	Ultimate Unit Side Shear q_{ss} (ksf)	Ultimate Unit End Bearing q_{eb} (ksf)	Ultimate Side Shear Q_{ss} (kips)	Ultimate End Bearing Q_{eb} (kips)	Total Ultimate Axial Capacity Q_{ut} (kips)	Total Allowable Axial Capacity Q_{at} (kips)	Total Allowable Uplift Capacity Q_{up} (kips)
0.0							
1.0	22.6	190	567	9545	10112	3371	132
2.0	22.6	190	1135	9545	10680	3560	265
3.0	22.6	190	1703	9545	11248	3749	397
4.0	22.6	190	2271	9545	11816	3939	530
5.0	22.6	190	2838	9545	12383	4128	662
6.0	22.6	190	3406	9545	12951	4317	795
7.0	22.6	190	3974	9545	13519	4506	926
8.0	22.6	190	4541	9545	14086	4695	1060
9.0	22.6	190	5109	9545	14654	4885	1192
10.0	22.6	190	5677	9545	15222	5074	1323
11.0	22.6	190	6244	9545	15790	5263	1455
12.0	22.6	190	6812	9545	16357	5452	1587
13.0	22.6	190	7380	9545	16925	5642	1720
14.0	22.6	190	7948	9545	17493	5831	1852
15.0	22.6	190	8515	9545	18060	6020	1984
16.0	22.6	190	9083	9545	18628	6209	2117
17.0	22.6	190	9651	9545	19196	6399	2249
18.0	22.6	190	10218	9545	19763	6588	2381
19.0	22.6	190	10786	9545	20331	6777	2513
20.0	22.6	190	11354	9545	20899	6966	2646
						D (ft.) =	8.0
						FS =	3.0

7.0 LATERAL LOAD ANALYSIS

7.1 Bridge Pier I Station 15+73.00

The interior bridge pier, which is located at station 15+75, will be located within the river channel. Holes 12(S-3) and hole 14(S-5) were drilled in 2006 for this pier. The foundation will consist of drilled shafts, socketed into bedrock. The riverbed was at approximate elevation 492+/-. Approximately 17 ft. of weak alluvial soil overlies bedrock. Bedrock was encountered at about elevation475+/-.

The bedrock consists primarily of hard siltstone. Eighteen unconfined compression tests were performed on the bedrock. The range of compressive strengths is 186tsf to 795 tsf, with an average strength of 573 tsf.

It is our recommendation that at least a 4 ft. permanent casing be used in developing the rock socket. The following parameters can be considered in performing the Lateral Load Analysis within the bedrock.

γ =155 pcf = Unit Weight
q_u = 400 tsf= 5555 psi = Unconfined Compressive Strength
c= 200 tsf= 2777psi = Cohesive Strength
q_{eb} =190 ksf = Ultimate Unit End Bearing
f_s = 22.6 ksf = Ultimate Unit Side Shear Resistance
E = 500,000 psi = Modulus of Elasticity
e_{50} =0.00005 =Strain@ 50% of maximum stress
k_s =2500pci =Static Horizontal Subgrade Modulus

It is recommended that lateral load analysis for the drilled piers be performed utilizing the computer program “LPILE PLUS 4.0”, developed at the University of Texas at Austin. The computer program “LPILE” yields the response (deflection, bending moment, and shear forces) of a laterally loaded single pier in a non-linear soil and rock medium, which is modeled as a series of springs (P-y curves) and treats the pier as a set of discreet elements. The method of solution is finite difference approximation to the governing fourth order differential equation. The tabulated rock parameters can be used to generate P-y curves in lateral analysis, and are based on boring log information, unconfined compression tests, and per suggestions provided in the LPILE and program technical manual.

We can provide the design analysis for the drilled pier system, as necessary. We will need the lateral shear force and bending moment acting at the top of the foundation, the vertical load acting on a pier, and the amount of uplift force, to provide the final design.

To perform the lateral load drilled pier analysis, we would need the exact loading (axial and shear forces and bending moments and rotational restraints), their point of application, and boundary conditions (free head or fixed head) at the pier/pile head.

8.0 GEOTECHNICAL NOTES

1. In accordance with Section 206 of the Standard Specifications for Road and Bridge Construction, current edition, the moisture content of embankment material shall not vary from the optimum moisture content as determined by KM 64-511 by more than +2 percent or less than -2 percent. This moisture content requirement shall have equal weight with the density requirement when determining the acceptability of embankment construction. Refer to the Family of Curves for moisture/density correlations.

2. All soils, whether from roadway or borrow, may require manipulation to obtain proper moisture content prior to compaction. Direct payment shall not be permitted for rehandling, hauling, stockpiling, and/or manipulating soils.

3. Excavation of surface ditches and channel changes adjacent to embankment areas shall be performed prior to the placement of the adjacent embankments. The material excavated for the channel changes and surface ditches is suitable for embankment construction if dried to proper moisture content in accordance with Section 206 of the Standard Specifications for Road and Bridge Construction, current edition. Direct payment shall not be permitted for rehandling, hauling, stockpiling, and/or manipulating soils.

4. The contractor is responsible for conducting any operations necessary to excavate the cut areas to the required typical section. These operations shall be incidental to the unit bid price for roadway excavation or embankment in-place.

5. In order to provide a working platfom for embankment construction, granular embankment, in accordance with the current edition of Section 805 of the Standard Roadway and Bridge Construction Specification, except the 21/2 inch maximum size limit is waived and the material shall be classified as non-erodible, shall be placed over all soft and/or saturated foundation areas that may be detected during construction, as directed by the Engineer. The granular embankment material shall be wrapped by Type IV Geotextile Fabric in accordance with Section 214 & 843 of the Standard Roadway and Bridge Construction Specification, current edition. The required thickness is

estimated to be 2.0 ft. but the actual thickness and locations of this treatment shall be determined by the Engineer during construction and may depend on

seasonal fluctuations in the water table. For quantity estimation purposes, this shall include the low wet areas for the following stations:

Stations 8+50 to Stations 12+50 Left of centerline
Stations 18+25 to Stations 18+75 Left of centerline

6. Some soil horizons and slopes on the project are subject to erosion. Necessary procedures in accordance with Sections 212 and 213 of the Standard Roadway and Bridge Construction Specification current edition shall be followed on construction.

7. Foundation embankment benches shall be placed in accordance with the Standard Drawing RGX-010 at the location listed below and/or as directed by the Engineer.
   Stations 9+75 to 12+25

8. Transverse benching and perforated pipe underdrains shall be installed at the following approximate locations and any others designated by the Engineer. Contrary to Standard Drawing RDP-006, the transverse benches and pipe underdrains shall be installed on both the upgrade and downgrade cut to fill transitions.
   Station 9+75.

9. The pile cores for the bridge end-bent structure shall be constructed in accordance with Kentucky Standard RGX -100 and RGX -105 meeting the material requirements of the current edition of Special Provision 69. A granular pile core is anticipated and quantities shall be calculated for such. The final design shall meet the approval of the Engineer.

10. Prior to pile driving operations on the east abutment (End Bent II) the existing concrete abutment, piles, limestone slabs on the outside of the embankment shall be removed. The existing embankment from station 18+05 to station 18+35 shall be undercut three (3') deep and the width of existing embankment. The backfill material for this undercut area shall be granular embankment in accordance with Section 805 of the Standard Roadway and Bridge Construction Specification, current edition.

11. The entire embankment from station 18+05 to station 19+00 shall be constructed of granular embankment, in accordance with Section 805 of the Standard Roadway and Bridge Construction Specification, current edition.

9.0 DESIGN RECOMMENDATIONS

1. An average soil shrinkage value of two (2) percent is estimated for this project. This value should be applied to the formula for calculating the Apparent Shrinkage as outlined in the Design Manual.

2. According to the design plans there is only a very small amount of excavation on the project, therefore, disturbed soil bag samples were treated as fill samples and CBR tests were not performed on the bag samples. An estimated CBR design value of 2.0 is suggested for the subgrade.

   As an alternate, since we are using granular embankment material to stabilize any soft and wet areas, rock subgrade may be considered. A CBR design value of 11 is recommended for the Granular Embankment and a CBR design value of 2 shall be used for the soil beneath this rock. The recommended thickness of the rock roadbed is 1 ft.

3. Use drilled shafts to support the interior bridge pier, constructed in accordance with the Special Note for Drilled Shafts, current edition.

4. Embed the shafts into unweathered bedrock. The drilled shafts should have a minimum uncased rock socket length into unweathered bedrock of 2.5 times the rock socket diameter. Consideration must be given to the long term effects of scour, in setting the bottom elevations of the shaft.

5. Perform lateral load analyses using the geotechnical parameters provided in the attached Lateral Load Analysis section of the report. These parameters may be used to perform analyses using LPILE Plus. Some of the parameters may not be required to be input, depending on the version of the program being used.

6. Evaluate the allowable axial capacities using the attached Drilled Shaft Capacity Tables. Longer sockets may be required to satisfy axial or lateral load design criteria.

7. Permanent casing is required in the overburden. It should be noted in the plans that the permanent casing should be incidental to the unit bid price for Drilled Shaft, Common.

8. Detail the common portion of the shafts showing diameters larger than the rock socket diameters.

9. A hammer with a minimum energy rating of 23,000 ft.-lbs. is suitable to drive the HP 12X53 H piles to refusal at the east and west abutment. A hammer with an energy rating of 40,000 ft.-lbs. is required for a HP14X73 pile. Approval of the pile driving system, by the Engineer will be subject to satisfactory field performance of the pile driving procedures.

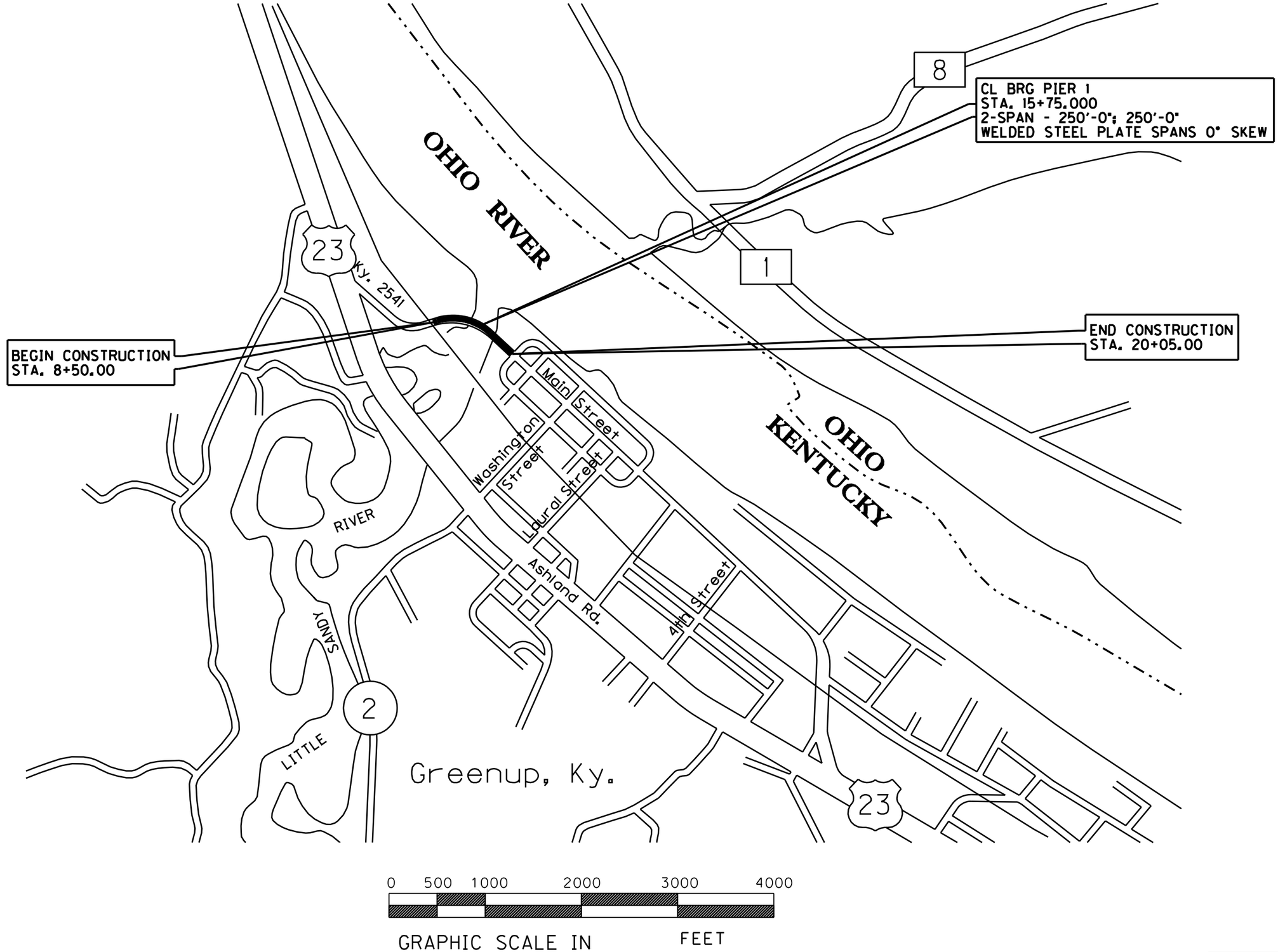
10. It is recommended that End Bent I Station 13+43.00 and End Bent II Station 18+03.00 be supported on H-piles driven to refusal bearing on bedrock. The estimated pile tip for End Bent I is 476.0 and for End Bent II is 487.0.

APPENDIX A

PROJECT LOCATION MAP

COUNTY OF	ITEM NO.	SHEET NO.
GREENUP	9-1038.00	

APPROVED BY _____ DATE _____



LOCATION MAP

APPENDIX B

COORDINATE DATA SUBMISSION FORM

COORDINATE DATA SUBMISSION FORM

KYTC DIVISION OF STRUCTURAL DESIGN -- GEOTECHNICAL BRANCH

Greenup Co. **Date: February 2009**

Bridge over The Little Sandy River Ky. 2561

Vaughn Melton Consulting Engineers

Contact Person Ken Corder, P.E.

Item # 09-1038.0

Mars #6739701D

Project # FD 50 045 2541 001- 002 BRZ 0903 101

Notes:

Mainline roadway, entrance road Lt. 12+95, and structure borings.

(circle one)

Elevation Datum **Sea Level** **Assumed**

HOLE NUMBER	LATITUDE (Decimal Degrees)	LONGITUDE (Decimal Degrees)	HOLE NUMBER	STATION	OFFSET	ELEVATION (ft)
1	38.5807605821	82.8414505154	1	10+00	C.L.	534.5
2	38.5809955412	82.8412140842	2	11+07	21' Lt.	521.8
3	38.5812126853	82.8407537856	3	12+50	14' Lt.	525.1
4	38.5816950627	82.8403481381	4	48+00	50' Lt.	526.8
5	38.5813332956	82.8407215986	5	49+50	23' Rt.	525.3
6	38.5797903804	82.8388557731	6	20+00	C.L.	537.9
10	38.5810126702	82.8405128692	10	13+50	7' Lt.	532.8
11	38.5809783101	82.8405531218	11	13+50	10' Rt.	532.1
12	38.5805996492	82.8398941787	12	15+82	16' Lt.	515.6
14	38.5805095115	82.8399760184	14	15+87	24' Rt.	515.4
15	38.5801536389	82.8393331543	15	18+10	5' Lt.	533.2
16	38.5800835488	82.8393962604	16	18+14	26' Rt.	533.7
20	38.5811722788	82.8407060761	20	12+70	12.4' Lt.	524.7
21	38.5810641905	82.8405713193	21	13+25	9.5'Lt.	533.5
22	38.5810045668	82.8406411726	22	13+25	20' Rt.	532.2

HOLE NUMBER	LATITUDE (Decimal Degrees)	LONGITUDE (Decimal Degrees)	HOLE NUMBER	STATION	OFFSET	ELEVATION (ft)
23	38.5802433622	82.8393230875	23	17+90	31' Lt.	517.9
24	38.5802833868	82.8392286684	24	18+00	60' Lt.	520.7
25	38.5801297998	82.8392897970	25	18+25	7' Lt.	533.6
26	38.5801053835	82.8393231526	26	18+24	6' Rt.	533.9

APPENDIX C

CONE PENTROMETER TESTING LOGS

CONE PENETRATION TEST

Summary of Test Method

The cone penetration test is performed in general accordance with the procedures as set forth in the "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils" (ASTM D 5778). In general, the resistance to penetration of a conical-pointed penetrometer is measured as it is advanced into subsurface soils at a slow, steady rate. Soundings performed using this test method provide a detailed record of the cone resistance, sleeve resistance, piezocone pore pressure (when recorded) and friction ratio as they vary with depth. The cone sounding log contains the following symbols and data:

Cone resistance, qc (ksf) - A penetrometer tip with a conical point having a 60° cone apex angle and a cone base area of 10 cm^2 is advanced through the soil at a constant rate of 20 mm/sec. The force on the conical point (cone) required to penetrate the soil is measured by electrical methods, at a minimum of every 50 mm of penetration. Stress on the cone is calculated as the measured force divided by the cone base area to obtain the cone resistance.

Friction sleeve resistance, fs (ksf) - A friction sleeve is present on the penetrometer immediately behind the cone tip, and the force exerted on the friction sleeve is measured by electrical methods at a minimum of every 50 mm of penetration. Stress on the sleeve is calculated by dividing the measured force on the surface area of the friction sleeve to determine friction sleeve resistance.

Piezocone pore pressure, u (ksf) - A pressure transducer may be present with its porous element located between the cone and the friction sleeve, and piezocone pore pressure induced during advancement of the penetrometer is measured. The dissipation of either positive or negative excess pore water pressure can be monitored by stopping penetration, unloading the push rod and recording pore pressure as a function of time. When the pore pressure value becomes constant, it is measuring the equilibrium value or piezometric level at that depth.

Friction ratio, Rf (%) - The friction ratio is the ratio of the friction sleeve resistance (fs) to cone resistance (qc) measured where the friction sleeve and cone point are at the same depth and is expressed as a percentage.

Interpretation

The cone resistance measurement may be used to evaluate site stratigraphy, homogeneity and depth to firm layers, voids or cavities, and other discontinuities. The use of a friction sleeve and pore pressure element can provide an estimate of soil classification, and correlations with engineering properties of soils. When requested, the typical engineering properties analyzed may include some or all of the following: soil classification, undrained shear strength of cohesive soils, relative density and friction angle of cohesionless soils, and an interpreted SPT-CPT correlation (termed the SPT Energy Ratio).

Soil Classification - The classification method is described in Robertson et. al. (1986). Material types are determined as a function of the corrected total cone resistance (corrected for water pressure acting above the cone) and friction ratio. Numbers in parentheses correspond to the material types tabulated in the method.

Undrained shear strength, Su (ksf) - The undrained shear strength of cohesive soils is estimated using methods described by Lunne and Kleven (1981). The strength is estimated from the cone resistance, the effective overburden pressure and an empirical cone factor N_k.

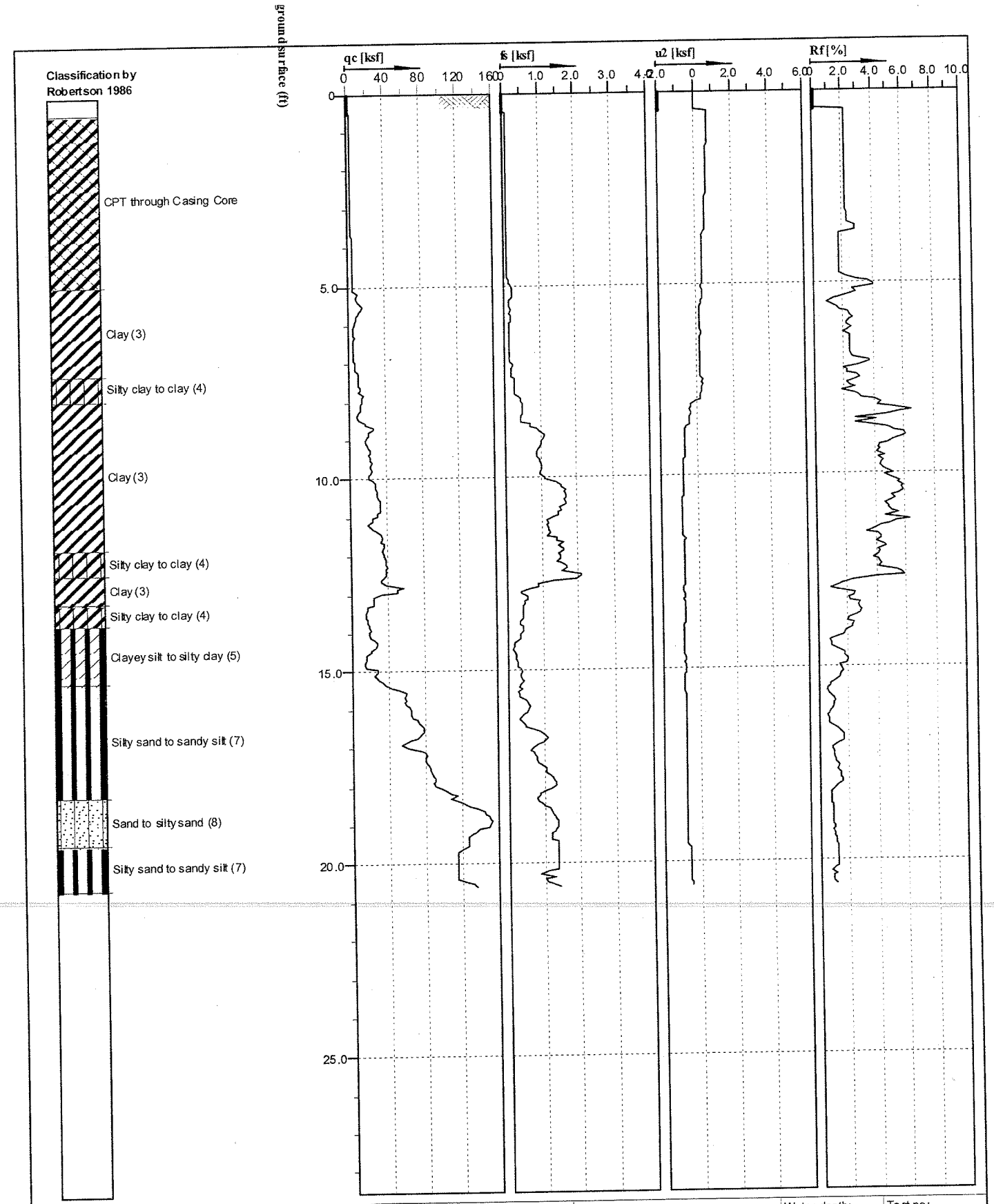
Relative Density, ID (%) - The relative density of cohesionless soils is estimated using methods described by Jamioloski et al (1985) for normally consolidated, uncemented sands. The relative density is estimated from the cone resistance and effective overburden pressure.

Friction angle, Fi (°) - The friction angle of cohesionless soils is estimated using methods described by Robertson and Campanella (1983). The friction angle is estimated using the cone resistance and the vertical effective overburden pressure.

SPT Energy Ratio, N60 () - The SPT Energy Ratio is estimated using methods described by Robertson et. al. (1986). The SPT N-value is estimated using the cone resistance and an empirical SPT-CPT factor dependent on the soil classification as determined above.

WEST APPROACH EMBANKMENT

STATION 14+00



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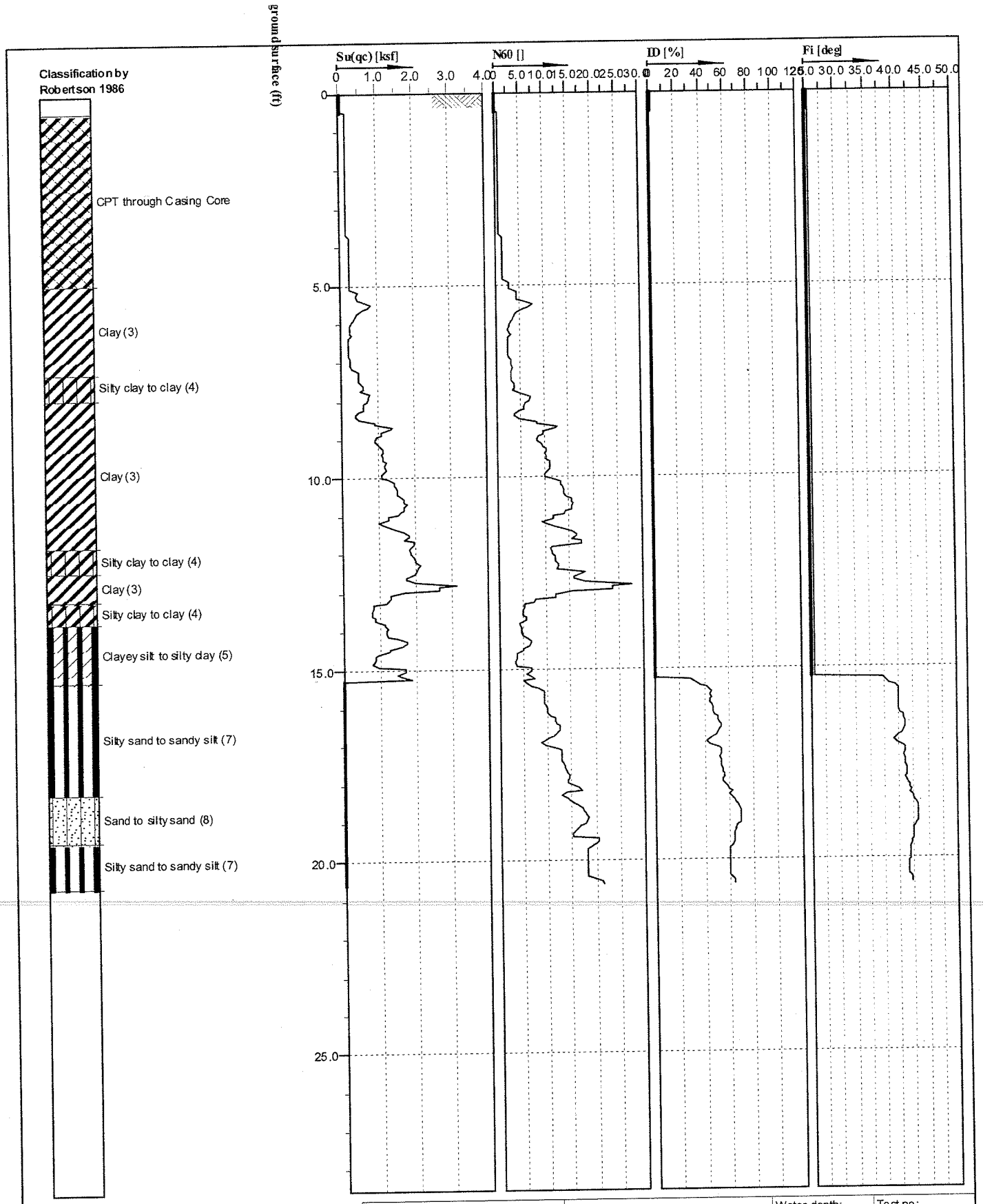
Project ID: 71503.009	Client: Kentucky Transportation Cabinet	Date: 10/18/2006	Scale: 1 : 40
Project KY 2541-Little Sandy River Bridge		Page: 1/1	Fig:
LOG OF CONE PENETRATION TEST		File: c1400.cpd	

H. C. NUTTING CO.
P.O. BOX C
CINCINNATI, OH
45025
513.321.5816

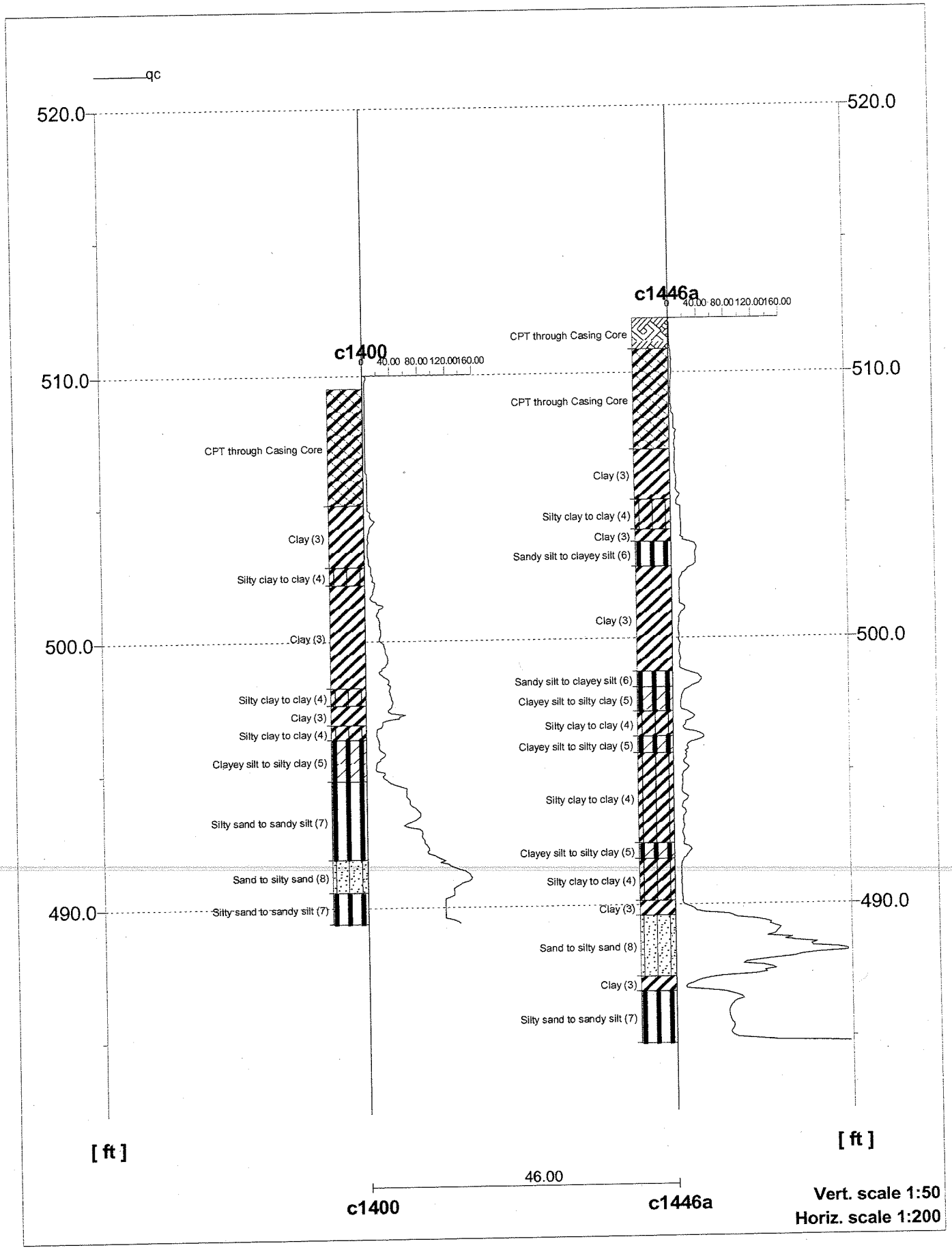
Cone No: 3389
Tip area [cm2]: 10
Sleeve area [cm2]: 150



H. C. NUTTING CO.
P.O. BOX C
CINCINNATI, OH
45226
513.321.5816
EMPLOYEE OWNED

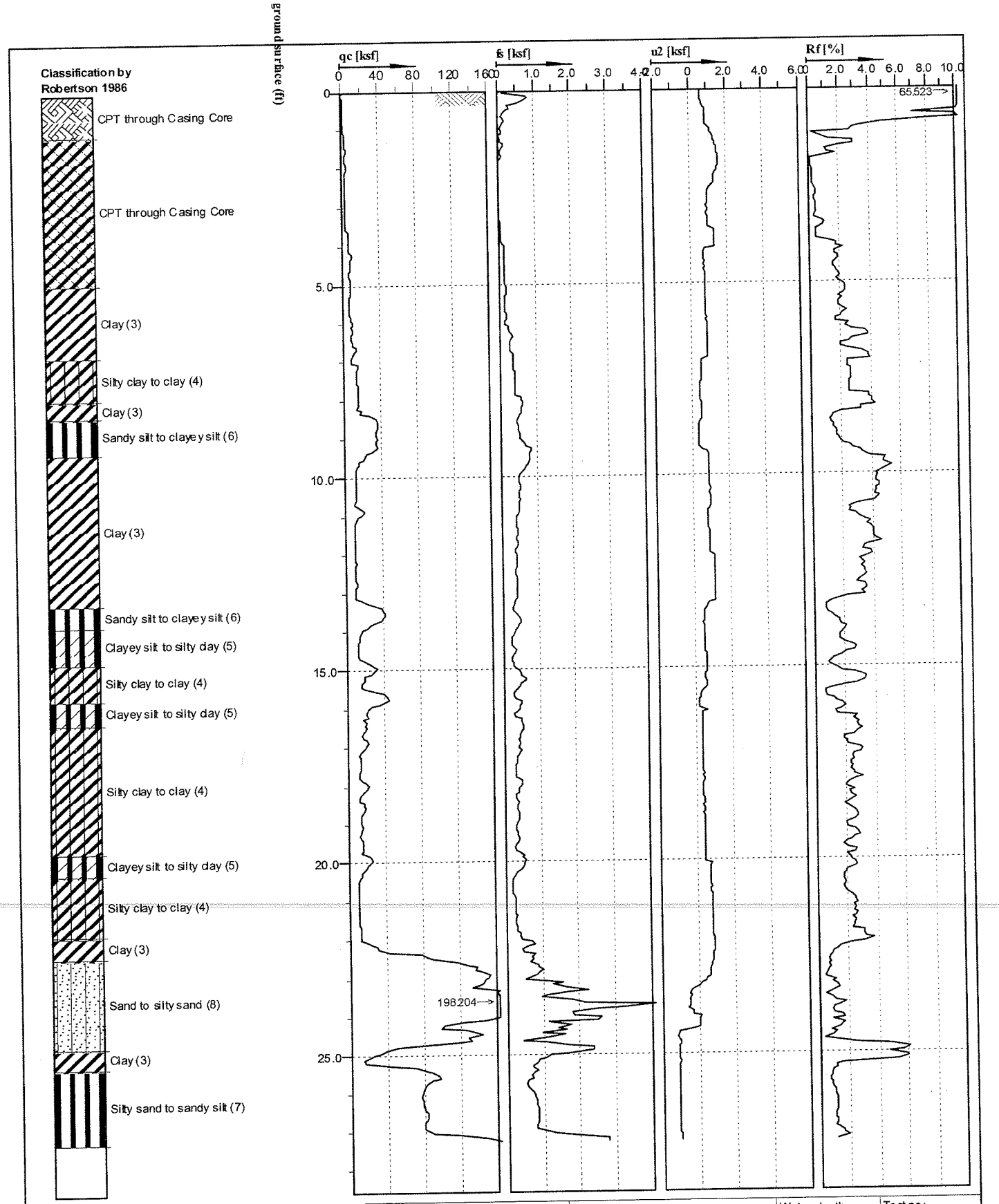
Cone No: 3389
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Greenup, Greenup Co, KY	Position: X: 1400.00 ft, Y: 0.00 ft	Water depth: 0.49	Test no: C1400
Project ID: 71503.009	Client: Kentucky Transportation Cabinet	Date: 10/18/2006	Scale: 1 : 40
Project KY 2541-Little Sandy River Bridge		Page: 1/1	Fig:
LOG OF CONE PENETRATION TEST		File: c1400.cpd	


qc
520.0
510.0
500.0
490.0
c1400
0 40.00 80.00 120.00 160.00
CPT through Casing Core
Clay (3)
Silty clay to clay (4)
Clay (3)
Silty clay to clay (4)
Clay (3)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Silty sand to sandy silt (7)
Sand to silty sand (8)
Silty sand to sandy silt (7)
c1446a
0 40.00 80.00 120.00 160.00
CPT through Casing Core
CPT through Casing Core
Clay (3)
Silty clay to clay (4)
Clay (3)
Sandy silt to clayey silt (6)
Clay (3)
Sandy silt to clayey silt (6)
Clayey silt to silty clay (5)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Silty clay to clay (4)
Clay (3)
Sand to silty sand (8)
Clay (3)
Silty sand to sandy silt (7)
[ft]
[ft]
46.00
c1400
c1446a
Vert. scale 1:50
Horiz. scale 1:200

WEST APPROACH EMBANKMENT

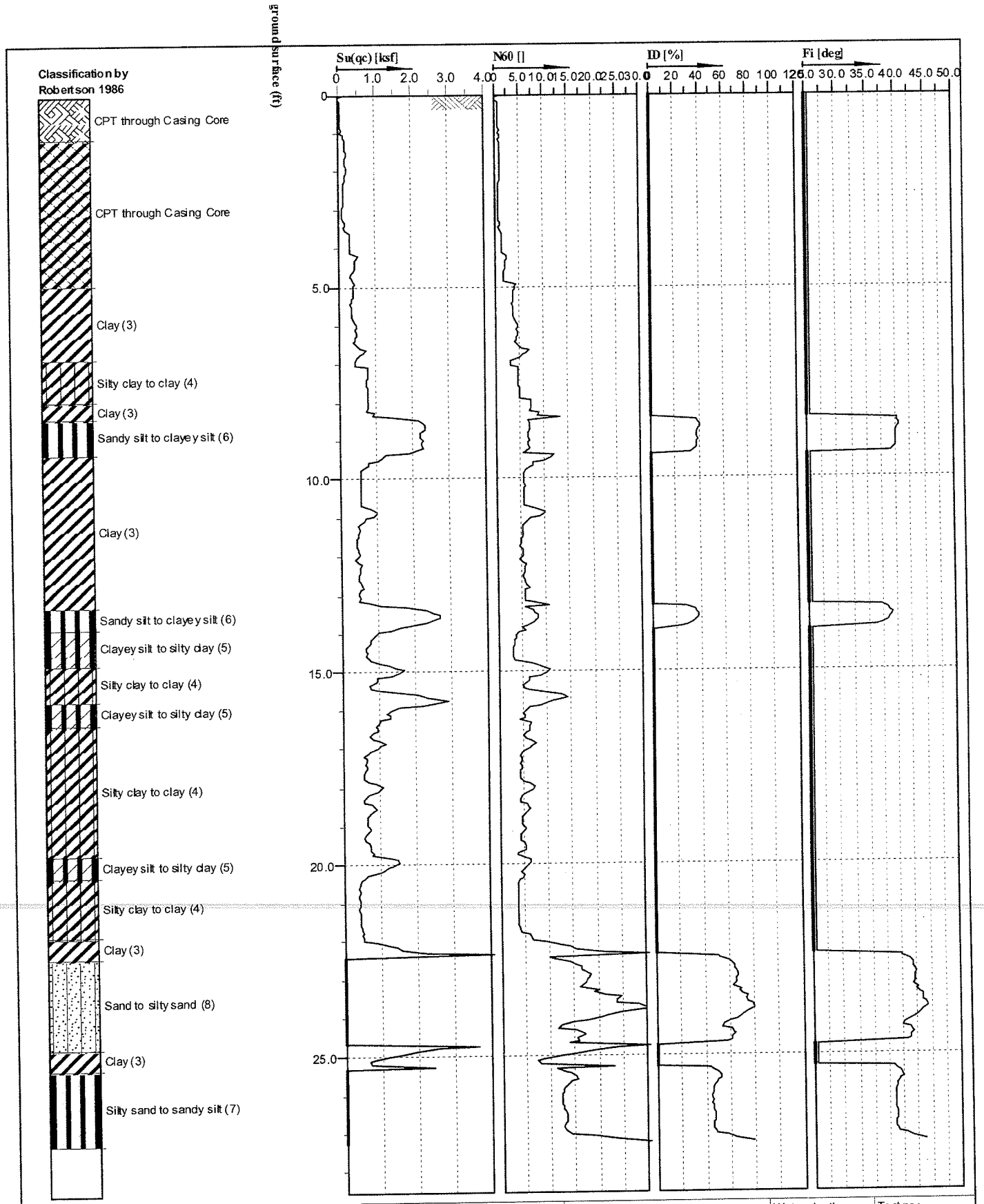
STATION 14+46



Location: Greenup, Greenup Co. KY	Position: X: 1446.00 ft, Y: 0.00 ft	Water depth: 0.49	Test no: C1446A
Project ID: 71503.009	Client: Kentucky Transportation Cabinet	Date: 10/18/2006	Scale: 1 : 40
Project KY 2541-Little Sandy River Bridge		Page: 1/1	Fig:
LOG OF CONE PENETRATION TEST		File: c1446a.cpd	

H. C. NUTTING CO.
P.O. BOX C
CINCINNATI, OH
45225
513.321.5816

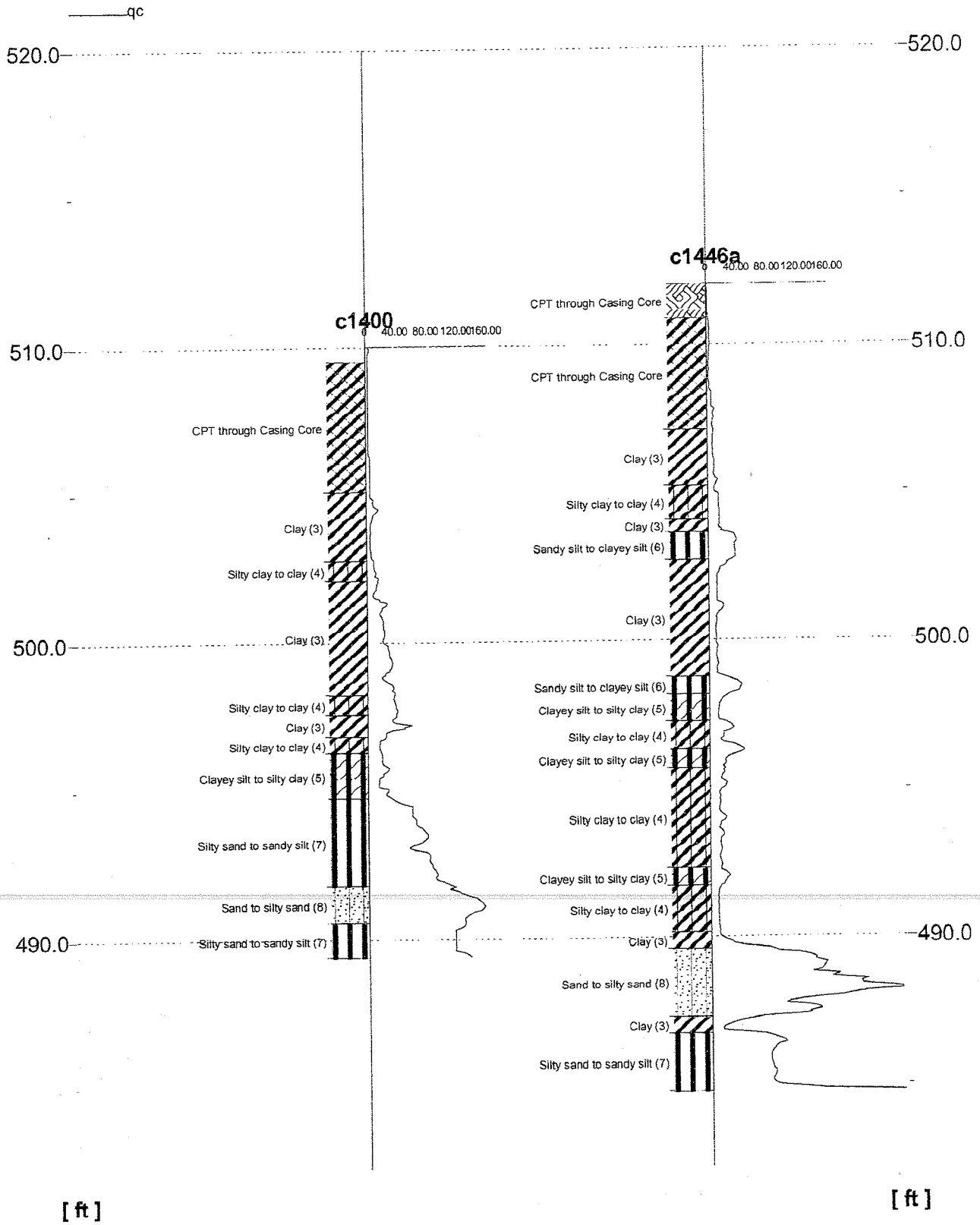
Cone No: 3389
Tip area [cm2]: 10
Sleeve area [cm2]: 150



H. C. NUTTING CO.
P.O. BOX C
CINCINNATI, OH
45225
513.321.5816
EMPLOYEE OWNED

Cone No: 3389
Tip area [cm2]: 10
Sleeve area [cm2]: 150

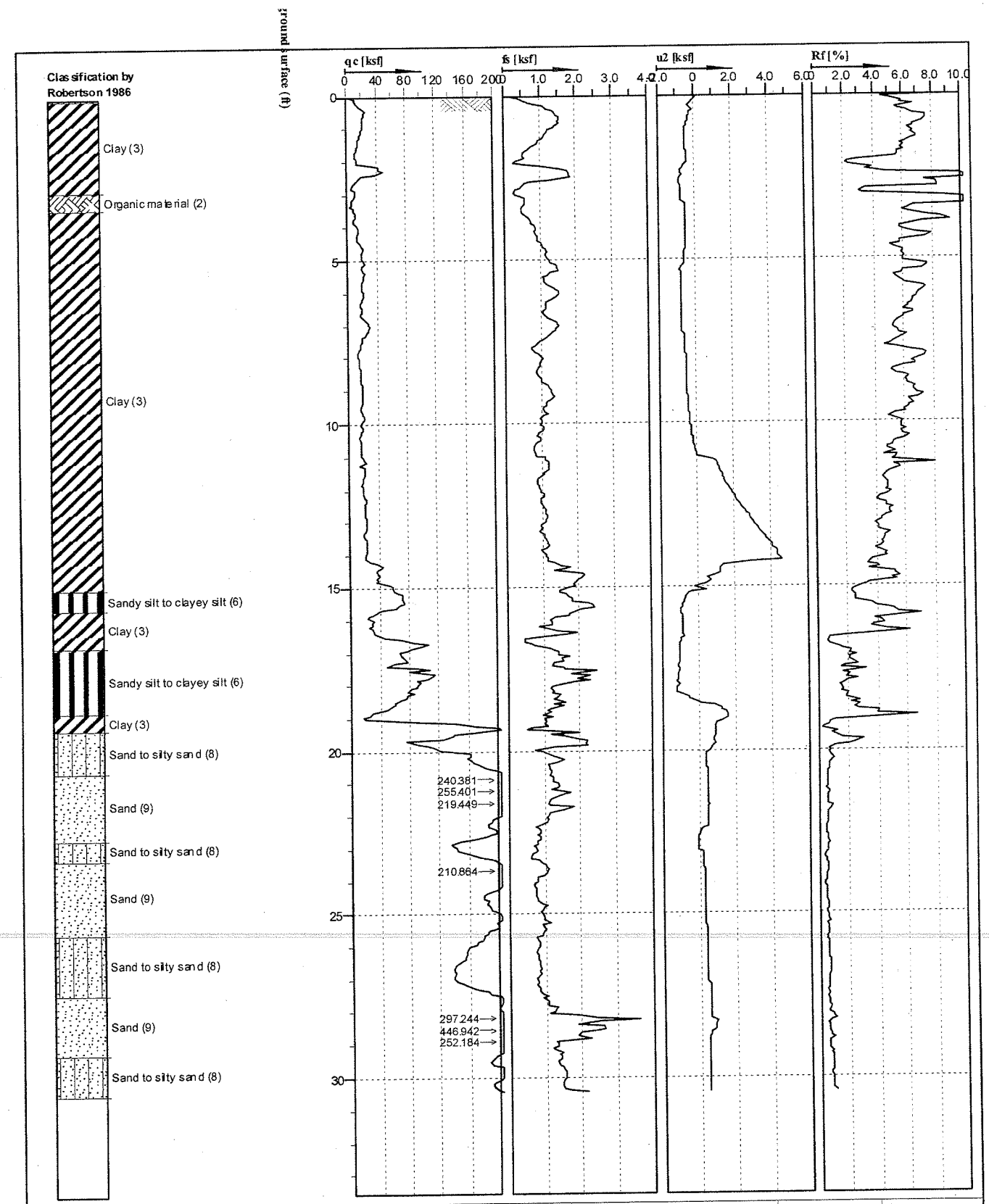
Location: Greenup, Greenup Co, KY	Position: X: 1446.00 ft, Y: 0.00 ft	Water depth: 0.49	Test no: C1446A
Project ID: 71503.009	Client: Kentucky Transportation Cabinet	Date: 10/18/2006	Scale: 1 : 40
Project KY 2541-Little Sandy River Bridge		Page: 1/1	Fig:
LOG OF CONE PENETRATION TEST		File: c1446a.cpd	



Vert. scale 1:50
Horiz. scale 1:200

EAST APPROACH EMBANKMENT

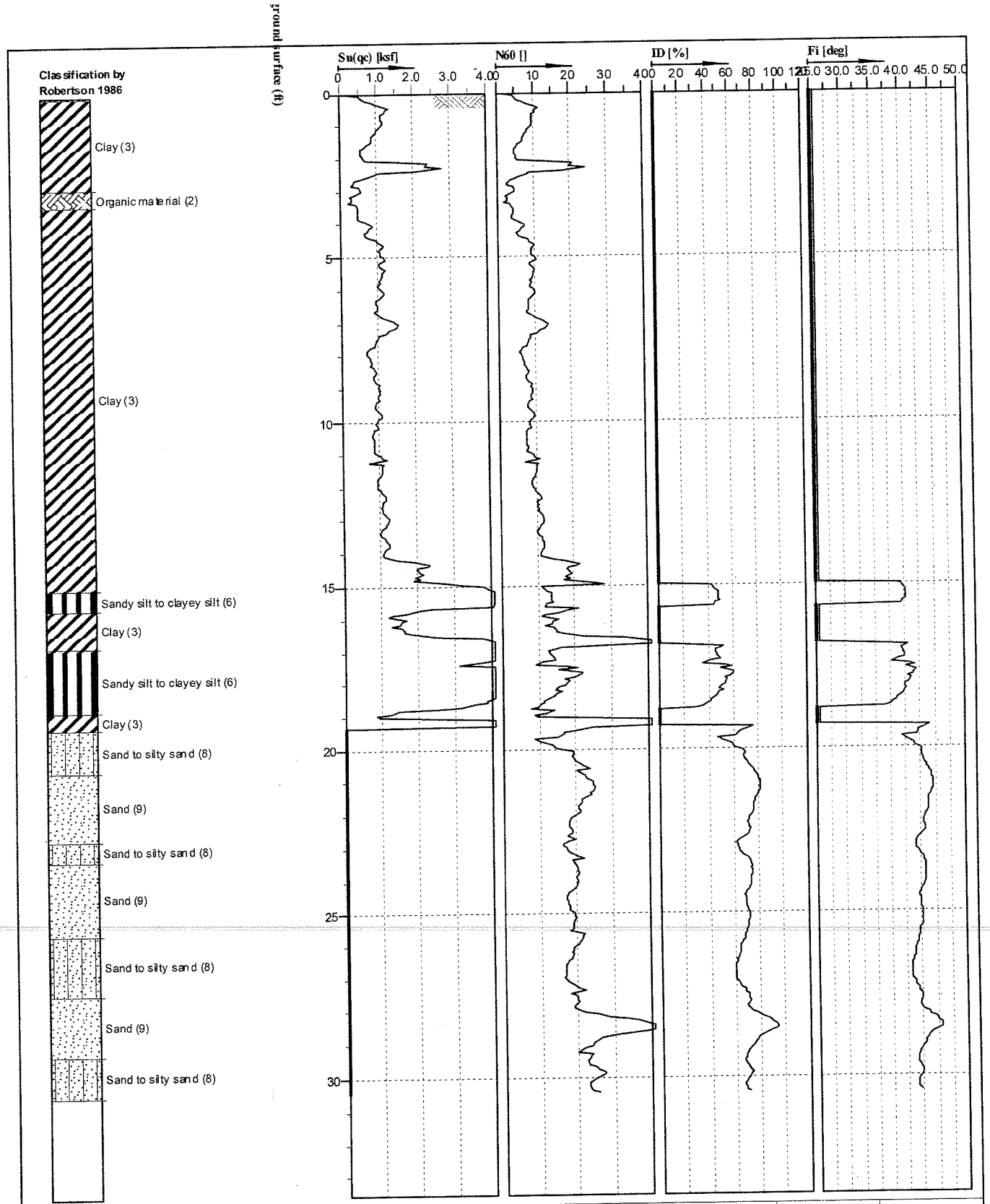
STATION 17+70



Location: Greenup, Greenup County, Kentucky	Position: X: 1770.01 ft, Y: 25.98 ft	Ground level: 516.99	Test no: 1
Project ID: 71503.009	Client: Kentucky Transportation Cabinet	Date: 11/21/2006	Scale: 1 : 47
Project: KY 2541-Little Sandy River Bridge		Page: 1/1	Fig:
LOG OF CONE PENETRATION TEST		File: B-1.cpd	

H. C. NUTTING CO.
P.O. BOX C
CINCINNATI, OH
45225
513.321.5816

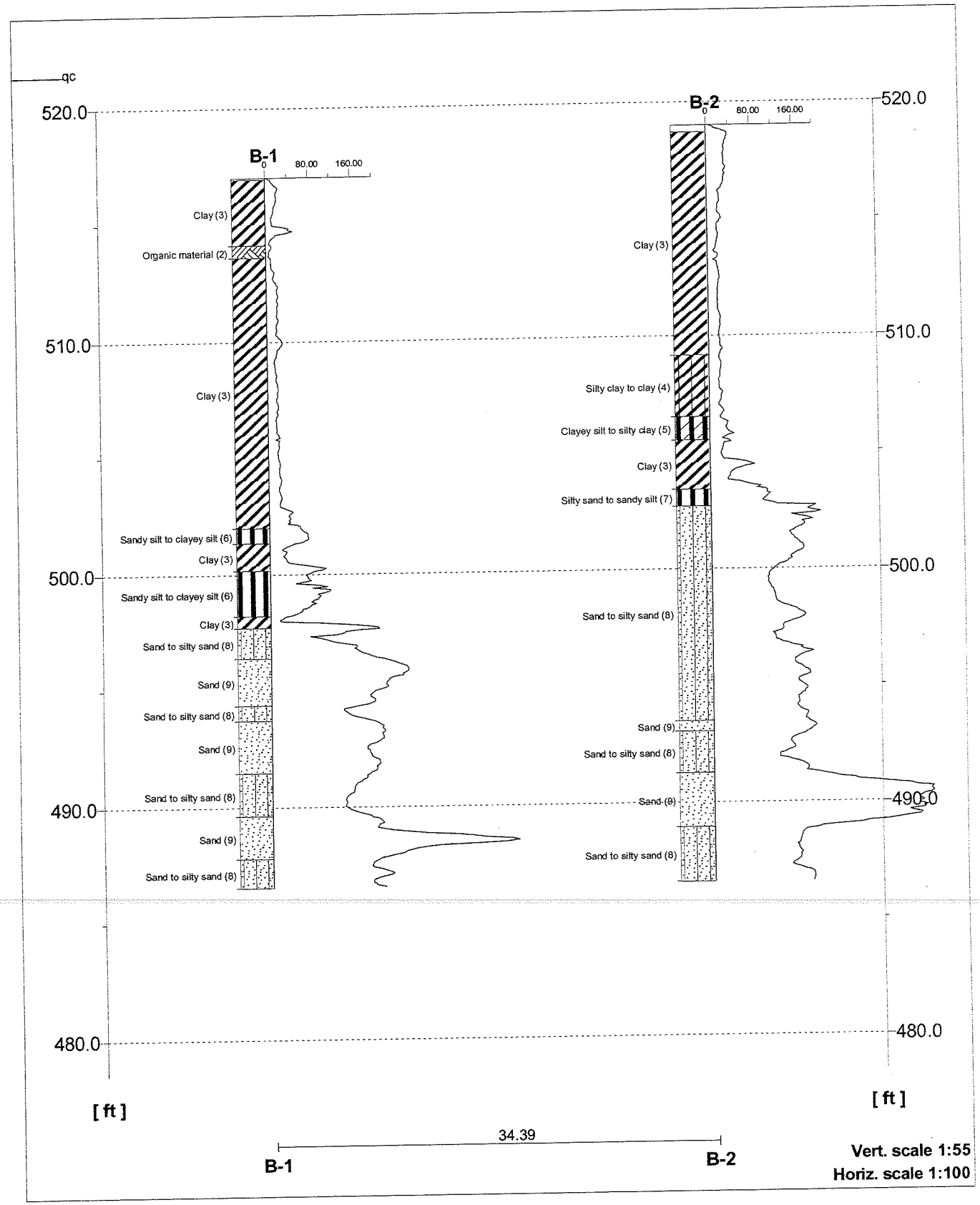
Cone No: 3389
Tip area [cm2]: 10
Sleeve area [cm2]: 150



H. C. NUTTING CO.
P.O. BOX C
CINCINNATI, OH
45222
513.321.5816
GEOTECHNICAL

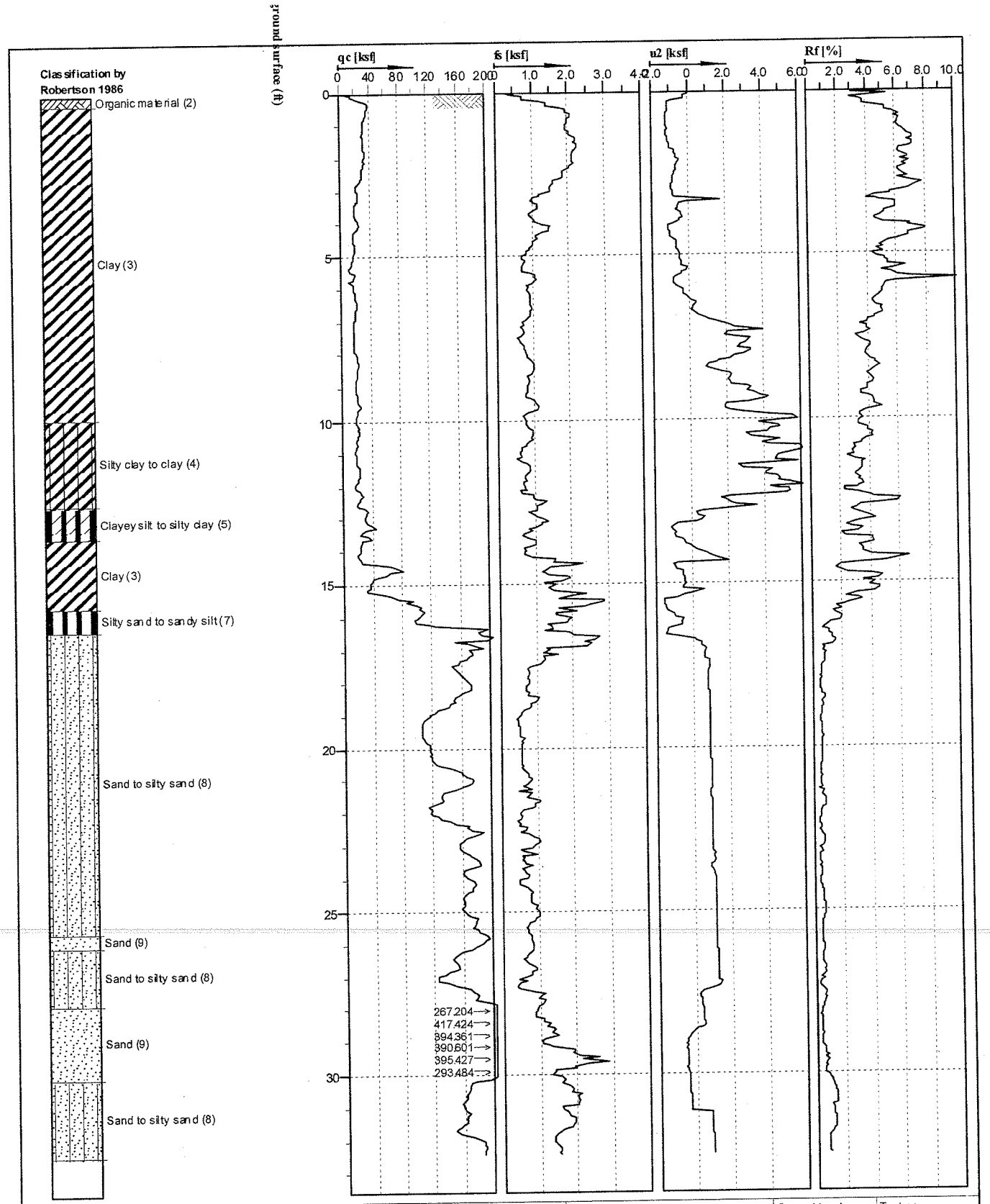
Cone No: 3389
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Greenup, Greenup County, Kentucky	Position: X: 1770.01 ft, Y: 25.98 ft	Ground level: 516.99	Test no: 1
Project ID: 71503.009	Client: Kentucky Transportation Cabinet	Date: 11/21/2006	Scale: 1 : 47
Project: KY 2541-Little Sandy River Bridge		Page: 1/1	Fig:
LOG OF CONE PENETRATION TEST		File: B-1.cpd	


qc
520.0
510.0
500.0
490.0
480.0
B-1
0
80.00
160.00
Clay (3)
Organic material (2)
Clay (3)
Sandy silt to clayey silt (6)
Clay (3)
Sandy silt to clayey silt (6)
Clay (3)
Sand to silty sand (8)
Sand (9)
Sand to silty sand (8)
Sand (9)
Sand to silty sand (8)
Sand (9)
Sand to silty sand (8)
B-2
0
80.00
160.00
Clay (3)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Clay (3)
Silty sand to sandy silt (7)
Sand to silty sand (8)
Sand (9)
Sand to silty sand (8)
Sand (9)
Sand to silty sand (8)
[ft]
[ft]
34.39
B-1
B-2
Vert. scale 1:55
Horiz. scale 1:100

EAST APPROACH EMBANKMENT

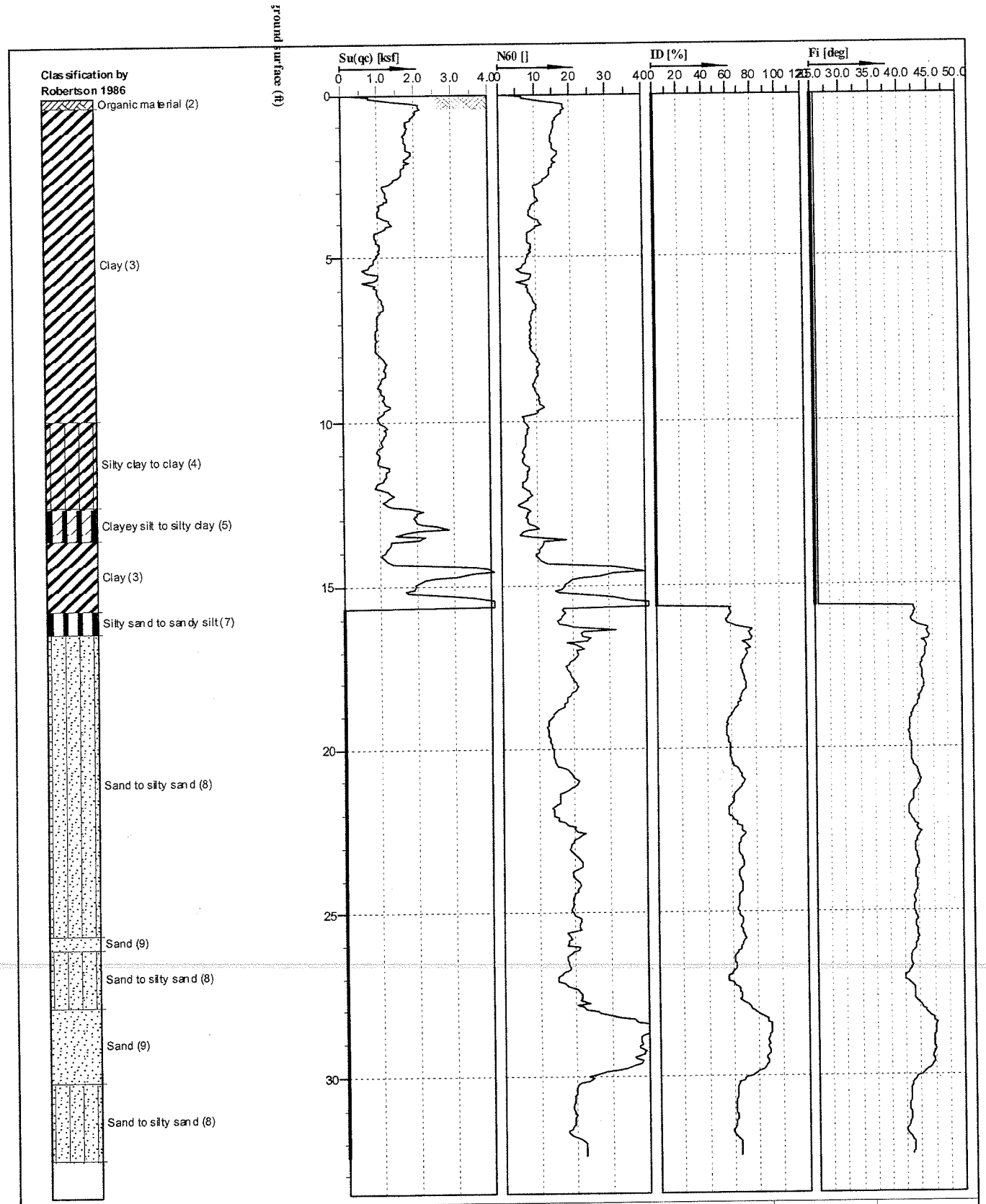
STATION 17+75



Location: Greenup, Greenup County, Kentucky	Position: X: 1775.00 ft, Y: 60.01 ft	Ground level: 519.00	Test no: 2
Project ID: 71503.009	Client: Kentucky Transportation Cabinet	Date: 11/21/2006	Scale: 1 : 47
Project: KY 2541-Little Sandy River Bridge		Page: 1/1	Fig:
LOG OF CONE PENETRATION TEST		File: B-2.cpd	

H. C. NUTTING CO.
P.O. BOX C
CINCINNATI, OH
45225
513.321.5816

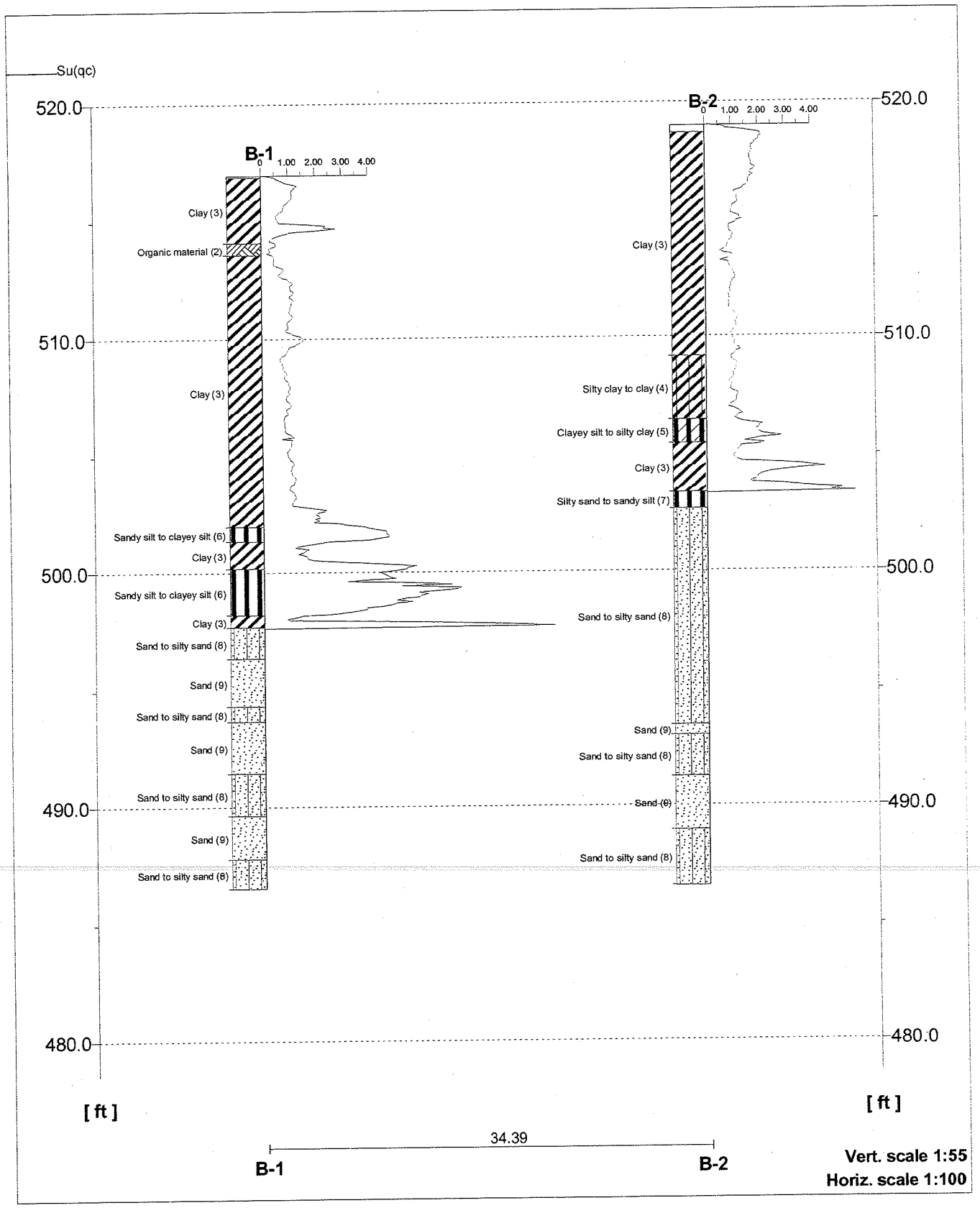
Cone No: 3389
Tip area [cm2]: 10
Sleeve area [cm2]: 150



Location: Greenup, Greenup County, Kentucky	Position: X: 1775.00 ft, Y: 60.01 ft	Ground level: 519.00	Test no: 2
Project ID: 71503.009	Client: Kentucky Transportation Cabinet	Date: 11/21/2006	Scale: 1 : 47
Project: KY 2541-Little Sandy River Bridge		Page: 1/1	Fig:
LOG OF CONE PENETRATION TEST		File: B-2.cpd	

H. C. NUTTING CO.
P.O. BOX C
CINCINNATI, OH
45225
513.321.5816

Cone No: 3389
Tip area [cm2]: 10
Sleeve area [cm2]: 150


Su(qc)
520.0
510.0
500.0
490.0
480.0
B-1
0
1.00
2.00
3.00
4.00
Clay (3)
Organic material (2)
Clay (3)
Sandy silt to clayey silt (6)
Clay (3)
Sandy silt to clayey silt (6)
Clay (3)
Sand to silty sand (8)
Sand (9)
Sand to silty sand (8)
Sand (9)
Sand to silty sand (8)
Sand (9)
Sand to silty sand (8)
B-2
0
1.00
2.00
3.00
4.00
Clay (3)
Silty clay to clay (4)
Clayey silt to silty clay (5)
Clay (3)
Silty sand to sandy silt (7)
Sand to silty sand (8)
Sand (9)
Sand to silty sand (8)
Sand (9)
Sand to silty sand (8)
[ft]
[ft]
34.39
B-1
B-2
Vert. scale 1:55
Horiz. scale 1:100

APPENDIX D

ENGINEERING DRAWINGS

COUNTY OF	ITEM NO.	SHEET NO.
SPRINGFIELD	13-765.00	

GEOTECHNICAL SYMBOL SHEET

AASHTO Classification of Soils and Soil-Aggregate Mixtures

General Classification	Granular Materials (35% or less passing 0.075 mm)							Silt-Clay Materials (More than 35% passing 0.075 mm)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis, Percent Passing											
2.00 mm (No. 10)	50 max	- - -	- - -	- - -	- - -	- - -	- - -	- - -	- - -	- - -	- - -
0.425 mm (No. 40)	30 max	50 max	51 min	- - -	- - -	- - -	- - -	- - -	- - -	- - -	- - -
0.075 mm (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of Fraction Passing 0.425 mm (No. 40)											
Liquid Limit	- - -		- - -	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min

Unified Soil Classifications

MAJOR DIVISIONS		SYMBOL	NAME
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well graded sands or gravelly sands, little or no fines.
		SP	Poorly graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS LL IS LESS THAN 50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays silty clays, lean clays.
	SILTS AND CLAYS LL IS GREATER THAN 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
UNCLASSIFIED MATERIAL		NONE	Non-classified material(i.e. overburden, pavement, slag, etc.) Include visual description.

PI	Plasticity Index
AI	Activity Index
LI	Liquidity Index
S+C	Silt + Clay (% finer than No.200 Sieve)
○	Rockline Soundings
◕	Disturbed Sample Boring
◎	Undisturbed Sample Boring
⊙	Undisturbed Sample Boring & Rock Core
●	Rock Core
	Slope Inclinometer Installation typical applications:
OW	Observation Well
▼ (Date)	Water Elevation
VS (psf)	Field Vane Shear Strength
■	Thin-walled Tube Sample
<	Standard Penetration Test Sample
N	Penetration Resistance
Qu (psf)	Unconfined Compressive Strength
UU (psf)	Unconsolidated Undrained Triaxial Strength
w%	Moisture Content
KY RQD	Rock Quality Designation (Kentucky Method)
Std RQD	Rock Quality Designation (Standard Method)
SDI(JS)	Slake Durability Index (Jar Slake Test)
REC	Core Recovery
ϕ	Angle of Internal Friction (Total Stress)
$\bar{\phi}$	Angle of Internal Friction (Effective Stress)
c (psf)	Cohesion (Total Stress)
$\bar{c}$ (psf)	Cohesion (Effective Stress)
γ (pcf)	Total Unit Weight
RDZ	Rock Disintegration Zone
OB	Overburden Bench
IB	Intermediate Bench
R	Refusal
NR	Refusal Not Encountered

LIMESTONE

SANDSTONE

DURABLE SHALE (SDI ≥ 95)

NONDURABLE SHALE (SDI < 95)

COAL

TALUS, MINE WASTE, FILL MATERIAL, BOULDERS, & ETC.

GRANULAR EMBANKMENT

STRUCTURE GRANULAR BACKFILL

SLOPE PROTECTION

GEOTECHNICAL SYMBOL SHEET

PREPARED BY ____ DATE ____
CHECKED BY ____ DATE ____
APPROVED BY ____ DATE ____

Cell Library: kyta.cel
Cell Name: sspl
DD-MMM-YYYY HH:MM

COUNTY OF	ITEM NO.	SHEET NO.
Greenup	9-1038.00	

APPROVED BY _____ DATE _____

Geotechnical Notes

1.) In accordance with Section 206 of the Standard Specifications for Road and Bridge Construction, current edition, the moisture content of embankment material shall not vary from the optimum moisture content as determined by KM 64-511 by more than +2 percent or less than -2 percent. This moisture content requirement shall have equal weight with the density requirement when determining the acceptability of embankment construction. Refer to the Family of Curves for moisture/density correlations.

2.) All soils, whether from roadway or borrow, may require manipulation to obtain proper moisture content prior to compaction. Direct payment shall not be permitted for rehandling, hauling, stockpiling, and/or manipulating soils.

3.) Excavation of surface ditches and channel changes adjacent to embankment areas shall be performed prior to the placement of the adjacent embankments. The material excavated for the channel changes and surface ditches is suitable for embankment construction if dried to proper moisture content in accordance with Section 206 of the Standard Specifications for Road and Bridge Construction, current edition. Direct payment shall not be permitted for rehandling, hauling, stockpiling, and/or manipulating soils.

4.) The contractor is responsible for conducting any operations necessary to excavate the cut areas to the required typical section. These operations shall be incidental to the unit bid price for roadway excavation or embankment in-place.

5.) In order to provide a working platfom for embankment construction, granular embankment, in accordance with the current edition of Section 805 of the Standard Roadway and Bridge Construction Specification, except the 21/2 inch maximum size limit is waived and the material shall be classified as non-erodible, shall be placed over all soft and/or saturated foundation areas that may be detected during construction, as directed by the Engineer. The granular embankment material shall be wrapped by Type IV Geotextile Fabric in accordance with Section 214 & 843 of the Standard Roadway and Bridge Construction Specification, current edition. The required thickness is estimated to be 2.0 ft. but the actual thickness and locations of this treatment shall be determined by the Engineer during construction and may depend on seasonal fluctuations in the water table. For quantity estimation purposes, this shall include the low wet areas for the following stations:

Stations 8+50 to Stations 12+50 Left of centerline
Stations 18+25 to Stations 18+75 Left of centerline

6.) Some soil horizons and slopes on the project are subject to erosion. Necessary procedures in accordance with Sections 212 and 213 of the Standard Roadway and Bridge Construction Specification, current edition, shall be followed on construction.

7.) Foundation embankment benches shall be placed in accordance with the Standard Drawing RGX-010 at the location listed below and/or as directed by the
Engineer.
Stations 9+75 to 12+25

8.) Transverse benching and perforated pipe underdrains shall be installed at the following approximate locations and any others designated by the Engineer. Contrary to Standard Drawing RDP-006, the transverse benches and pipe underdrains shall be installed on both the upgrade and downgrade cut to fill transitions.
Station 9+75

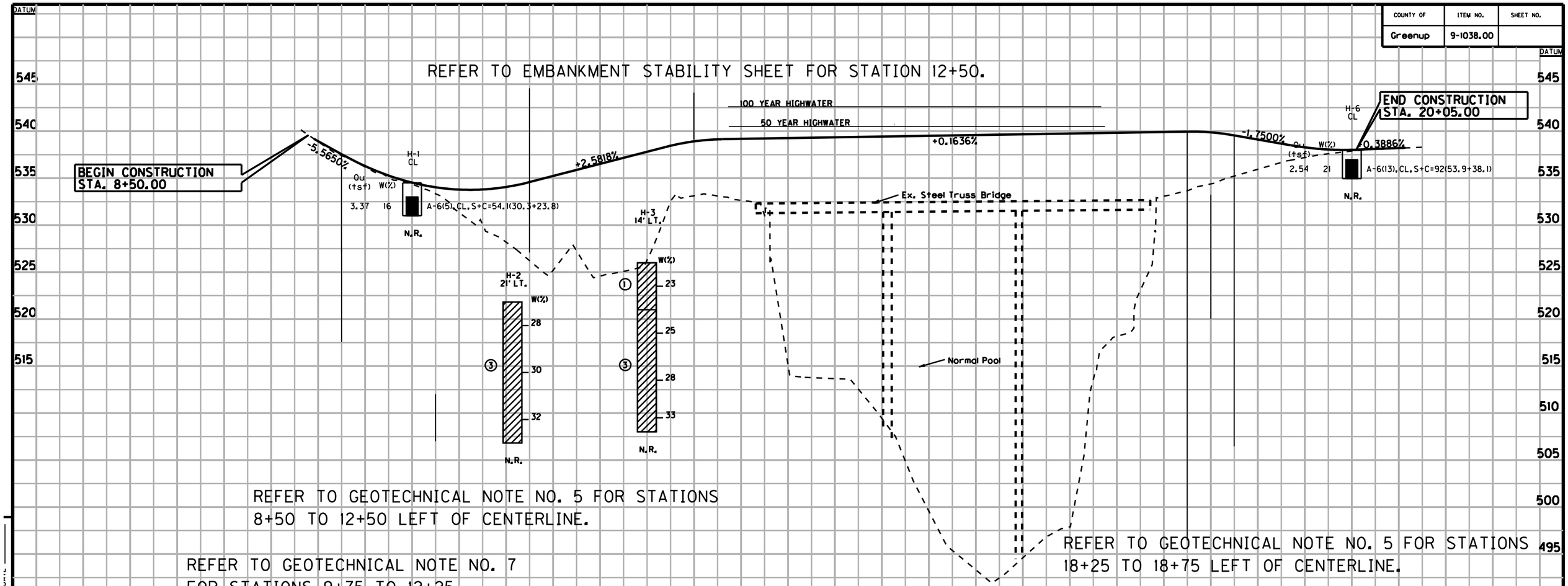
9.) The pile cores for the bridge end-bent structure shall be constructed in accordance with Kentucky Standard RGX -100 and RGX -105 meeting the material requirements of the current edition of Special Provision 69. A granular pile core is anticipated and quantities shall be calculated for such. The final design shall meet the approval of the Engineer.

10.) Prior to pile driving operations on the east abutment (End Bent II) the existing concrete abutment, piles, limestone slabs on the outside of the embankment shall be removed. The existing embankment from station 18+05 to station 18+35 shall be undercut three (3') deep and the width of existing embankment. The backfill material for this undercut area shall be granular embankment in accordance with Section 805 of the Standard Roadway and Bridge Construction
Specification, current edition.

11.) The entire embankment from station 18+05 to station 19+00 shall be constructed of granular embankment, in accordance with Section 805 of the Standard Roadway and Bridge Construction Specification, current edition.

GEOTECHNICAL NOTES

COUNTY OF	ITEM NO.	SHEET NO.
Greenup	9-1038.00	



REFER TO GEOTECHNICAL NOTE NO. 5 FOR STATIONS 8+50 TO 12+50 LEFT OF CENTERLINE.

REFER TO GEOTECHNICAL NOTE NO. 5 FOR STATIONS 18+25 TO 18+75 LEFT OF CENTERLINE.

REFER TO GEOTECHNICAL NOTE NO. 7 FOR STATIONS 9+75 TO 12+25.

REFER TO GEOTECHNICAL NOTE NO. 8 FOR STATION 9+75.

REFER TO EMBANKMENT STABILITY FOR STATION 18+25 SPILL-THROUGH EAST ABUTMENT SLOPE REVISIONS 1, 2, AND 3.

REFER TO GEOTECHNICAL NOTE NO. 10 FOR STATIONS 18+05 TO 18+35.

REFER TO GEOTECHNICAL NOTE NO. 11 FOR STATIONS 18+05 TO 19+00.

HOLE NO.		H-4	H-4	H-5
SAMPLE NO.		1	2	3
STATION		48+00	48+00	49+50
OFFSET		50' LT.	50' LT.	23' RT.
DEPTH		1.0-5.0	5.0-8.0	10.0-15.0
COMPOSITION OF TOTAL SAMPLE	GRAVEL (- 3" + NO. 10)	0.1	0.0	0.0
	SAND (- NO 10 + NO. 200)	30.3	15.4	11.6
	SILT (- 0.075 mm + 0.002 mm)	35.6	47.2	51.8
	CLAY (- 0.002 mm)	34.0	37.4	36.6
LIQUID LIMIT		28	32	32
PLASTIC LIMIT		19	20	20
PLASTICITY INDEX		9	12	12
ACTIVITY INDEX		0.26		
SPECIFIC GRAVITY			2.71	2.67
AASHTO CLASSIFICATION		A-4(4)	A-6(9)	A-6(10)
UNIFIED CLASSIFICATION		CL	CL	CL
CALIFORNIA BEARING RATIO				
MAXIMUM DRY DENSITY (pcf)				
OPTIMUM MOISTURE (%)				
% +4.75mm MATERIAL IN CBR & MOISTURE-DENSITY TESTS				
MOISTURE CONTENT		24	24	27
LIQUIDITY INDEX		0.55	0.33	0.58

SCALE: 1" = 50' HORIZONTAL
1" = 5' VERTICAL

537.2 538.28 537.9 538.02 538.2 538.21

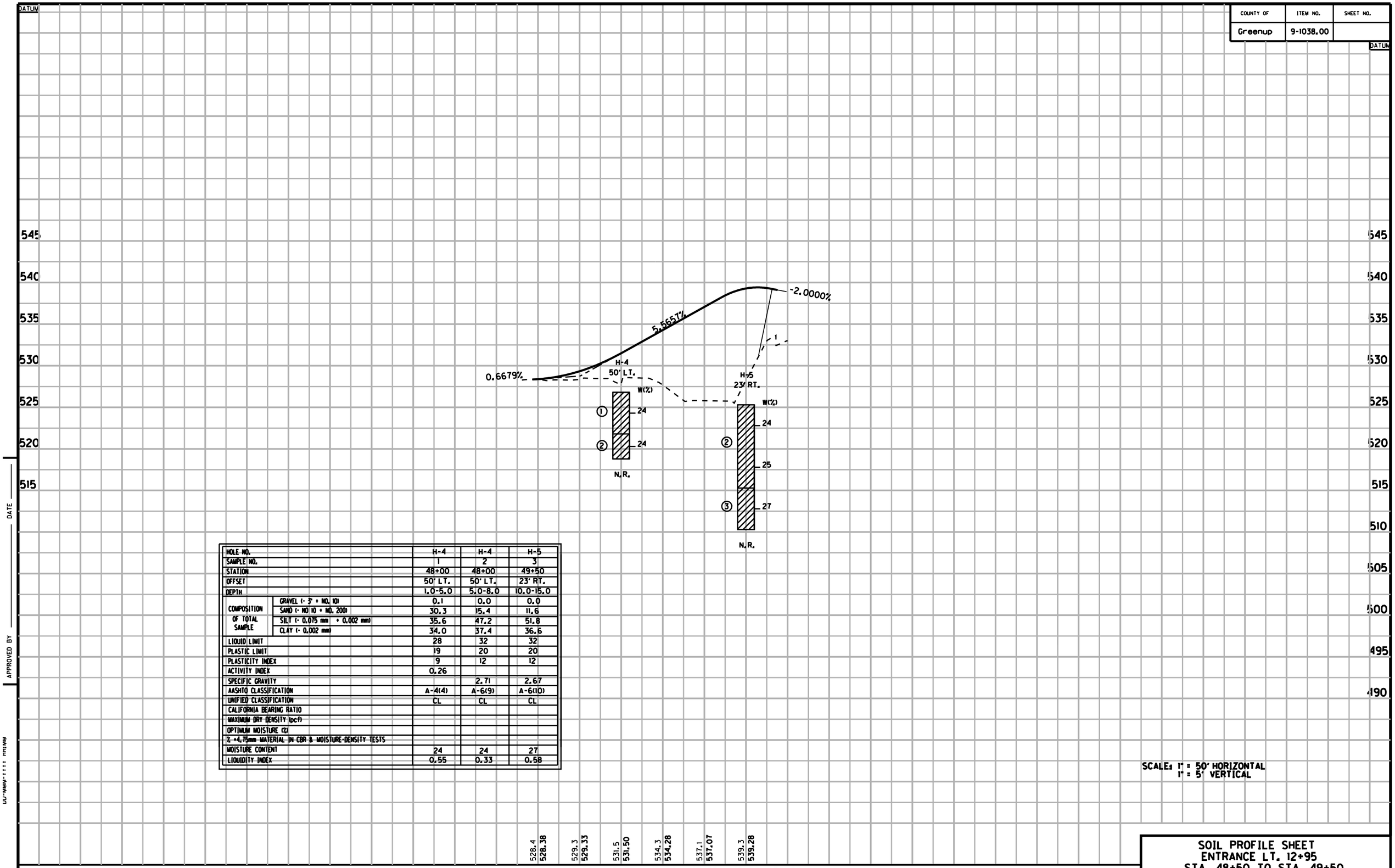
20+00 21+00

538.9 538.96 536.2 536.30 534.4 534.54 531.4 533.79 528.2 534.06 525.2 535.23 524.6 536.52 526.2 537.81 533.1 538.92 532.7 539.18 515.3 539.26 513.7 539.35 509.5 539.43 499.6 539.51 493.3 539.59 494.6 539.67 497.9 539.75 518.2 539.84 533.1 539.92 534.4 539.88 535.9 539.13

9+00 10+00 11+00 12+00 13+00 14+00 15+00 16+00 17+00 18+00 19+00

SOIL PROFILE SHEET
STA. 8+50.0 to STA. 20+05.00

COUNTY OF	ITEM NO.	SHEET NO.
Greenup	9-1038.00	



HOLE NO.		H-4	H-4	H-5
SAMPLE NO.		1	2	3
STATION		48+00	48+00	49+50
OFFSET		50' LT.	50' LT.	23' RT.
DEPTH		1.0-5.0	5.0-8.0	10.0-15.0
COMPOSITION OF TOTAL SAMPLE	GRAVEL (- 3" + NO. 10)	0.1	0.0	0.0
	SAND (- NO 10 + NO. 200)	30.3	15.4	11.6
	SILT (- 0.075 mm + 0.002 mm)	35.6	47.2	51.8
	CLAY (- 0.002 mm)	34.0	37.4	36.6
LIQUID LIMIT		28	32	32
PLASTIC LIMIT		19	20	20
PLASTICITY INDEX		9	12	12
ACTIVITY INDEX		0.26		
SPECIFIC GRAVITY			2.71	2.67
AASHTO CLASSIFICATION		A-4(4)	A-6(9)	A-6(10)
UNIFIED CLASSIFICATION		CL	CL	CL
CALIFORNIA BEARING RATIO				
MAXIMUM DRY DENSITY (pcf)				
OPTIMUM MOISTURE (%)				
% +4.75mm MATERIAL IN CBR & MOISTURE-DENSITY TESTS				
MOISTURE CONTENT		24	24	27
LIQUIDITY INDEX		0.55	0.33	0.58

SOIL PROFILE SHEET
ENTRANCE LT. 12+95
STA. 48+50 TO STA. 49+50

DATE

APPROVED BY

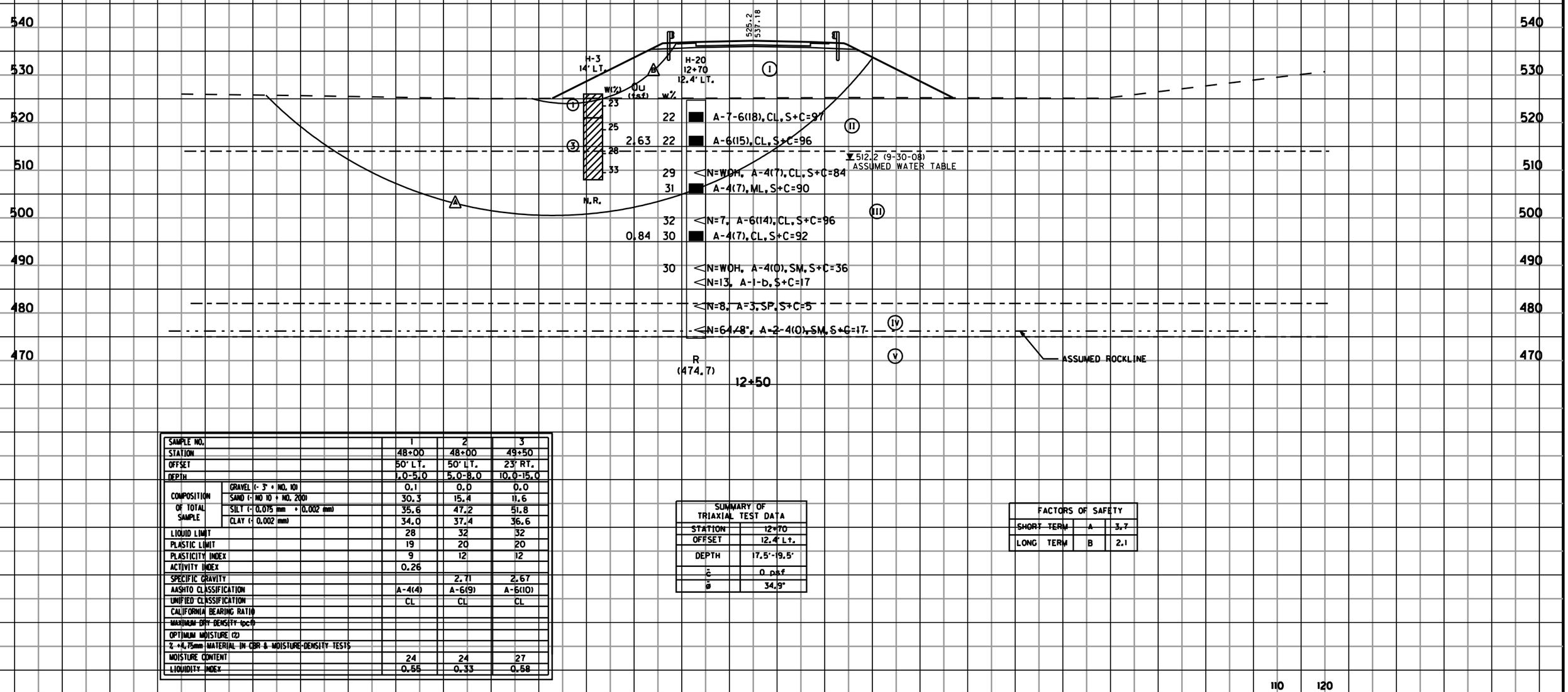
UU-MMM-1111 HHMM

COUNTY OF	ITEM NO.	SHEET NO.
GREENUP	9-1038.0	x3

DATE

APPROVED BY

ASSUMED SOIL STRENGTH PARAMETERS					
SOIL	I	II	III	IV	V
SHORT TERM	γ = 130 c = 1500 ϕ = 0	γ = 125 c = 1500 ϕ = 0	γ = 125 c = 800 ϕ = 0	γ = 122 c = 0 ϕ = 30	γ = 155 c = 8000 ϕ = 0
LONG TERM	γ = 130 $\bar{c}$ = 200 $\bar{\phi}$ = 28	γ = 125 $\bar{c}$ = 0 $\bar{\phi}$ = 33	γ = 125 $\bar{c}$ = 0 $\bar{\phi}$ = 33	γ = 122 $\bar{c}$ = 0 $\bar{\phi}$ = 34	γ = 155 c = 8000 ϕ = 0



SAMPLE NO.		1	2	3
STATION		48+00	48+00	49+50
OFFSET		50' LT.	50' LT.	23' RT.
DEPTH		1.0-5.0	5.0-8.0	10.0-15.0
COMPOSITION OF TOTAL SAMPLE	GRAVEL (- 3" + NO. 10)	0.1	0.0	0.0
	SAND (- NO 10 + NO. 200)	30.3	15.4	11.6
	SILT (- 0.075 mm + 0.002 mm)	35.6	47.2	51.8
	CLAY (- 0.002 mm)	34.0	37.4	36.6
LIQUID LIMIT		28	32	32
PLASTIC LIMIT		19	20	20
PLASTICITY INDEX		9	12	12
ACTIVITY INDEX		0.26		
SPECIFIC GRAVITY			2.71	2.67
AASHTO CLASSIFICATION		A-4(4)	A-6(9)	A-6(10)
UNIFIED CLASSIFICATION		CL	CL	CL
CALIFORNIA BEARING RATIO				
MAXIMUM DRY DENSITY (pcf)				
OPTIMUM MOISTURE (%)				
% +4.75mm MATERIAL IN CBR & MOISTURE-DENSITY TESTS				
MOISTURE CONTENT		24	24	27
LIQUIDITY INDEX		0.55	0.33	0.58

SUMMARY OF TRIAXIAL TEST DATA	
STATION	12+70
OFFSET	12.4' Lt.
DEPTH	17.5'-19.5'
$\bar{c}$	0 psf
$\bar{\phi}$	34.9°

FACTORS OF SAFETY		
SHORT TERM	A	3.7
LONG TERM	B	2.1

SCALE: 1" = 10' HORIZONTAL
1" = 10' VERTICAL

EMBANKMENT STABILITY SECTION
MAINLINE
STA. 12+50

E-SHEET 1 10APR6.

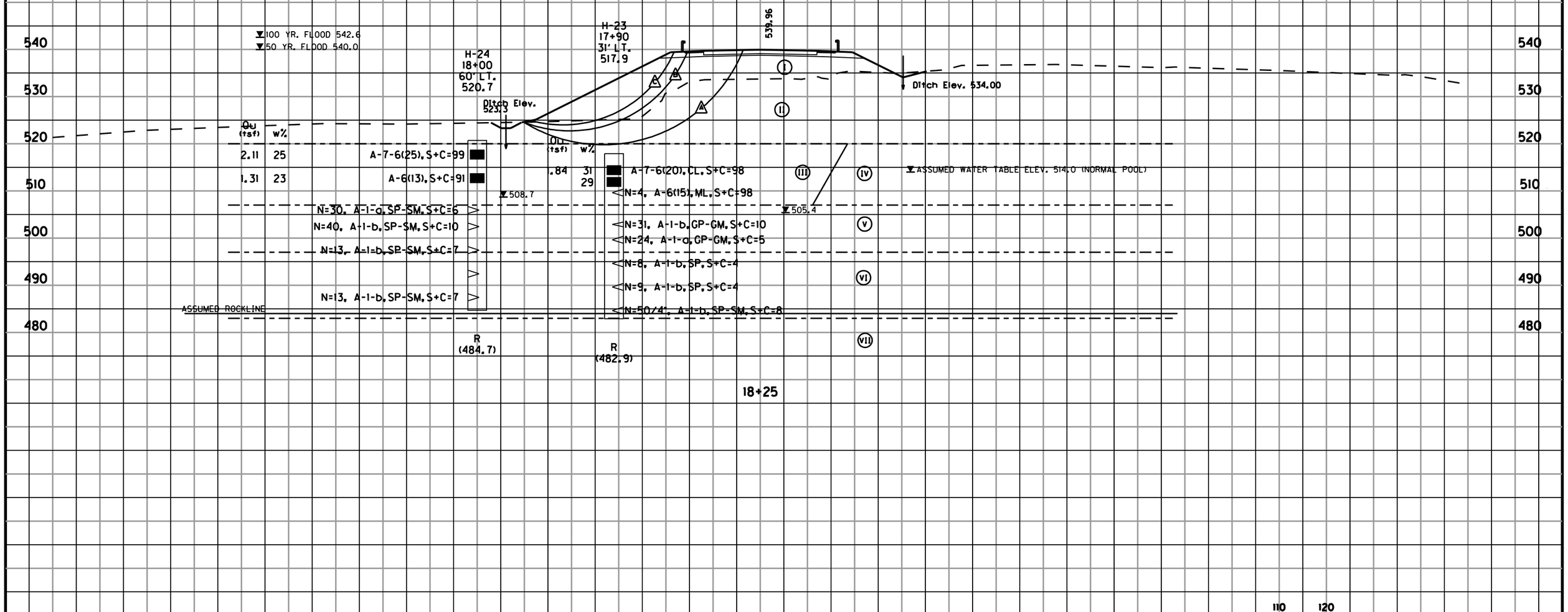
COUNTY OF	ITEM NO.	SHEET NO.
GREENUP	9-1038.0	X3

APPROVED BY ____ DATE ____

ASSUMED SOIL STRENGTH PARAMETERS																			
SOIL	I		II		III		IV		V		VI		VII	
SHORT TERM	γ =	130	γ =	122	γ =	122	γ =	115	γ =	125	γ =	120	γ =	155
	c =	1500	c =	700	c =	1000	c =	0	c =	0	c =	0	c =	8000
	φ =	0	φ =	0	φ =	0	φ =	28	φ =	39	φ =	32	φ =	0
LONG TERM	γ =	130	γ =	122	γ =	122	γ =	115	γ =	125	γ =	120	γ =	155
	c̄ =	200	c̄ =	0	c̄ =	0	c̄ =	0	c̄ =	0	c̄ =	0	c̄ =	8000
	φ̄ =	28	φ̄ =	33	φ̄ =	33	φ̄ =	28	φ̄ =	39	φ̄ =	32	φ̄ =	0
LONG TERM RAPID DRAWDOWN	γ =	130	γ =	122	γ =	122	γ =	115	γ =	125	γ =	120	γ =	155
	c̄ =	200	c̄ =	0	c̄ =	0	c̄ =	0	c̄ =	0	c̄ =	0	c̄ =	8000
	φ̄ =	28	φ̄ =	33	φ̄ =	33	φ̄ =	28	φ̄ =	39	φ̄ =	32	φ̄ =	0

SUMMARY OF TRIAXIAL TEST DATA	
STATION	17+90
OFFSET	31' Lt.
DEPTH	5'-7'
c̄	0 psf
φ̄	30.1°

FACTORS OF SAFETY		
SHORT TERM	A	2.5
LONG TERM	B	1.8
LONG TERM RAPID DRAWDOWN	C	0.99



SCALE: 1" = 10' HORIZONTAL
1" = 10' VERTICAL

SPILL THROUGH EAST ABUTMENT SLOPE
STA. 18+00

COUNTY OF	ITEM NO.	SHEET NO.
GREENUP	9-1038.0	X3

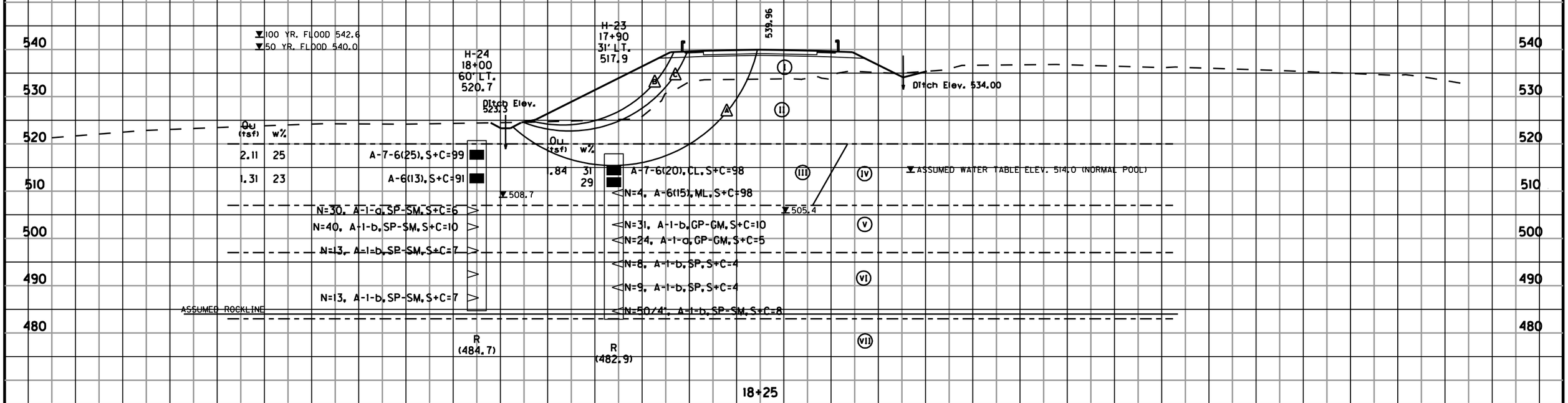
DATE

APPROVED BY

ASSUMED SOIL STRENGTH PARAMETERS							
SOIL	I	II	III	IV	V	VI	VII
SHORT TERM	γ = 140 c = 0 ϕ = 38	γ = 122 c = 700 ϕ = 0	γ = 122 c = 1000 ϕ = 0	γ = 115 c = 0 ϕ = 28	γ = 125 c = 0 ϕ = 39	γ = 120 c = 0 ϕ = 32	γ = 155 c = 8000 ϕ = 0
LONG TERM	γ = 140 $\bar{c}$ = 0 $\bar{\phi}$ = 38	γ = 122 $\bar{c}$ = 0 $\bar{\phi}$ = 33	γ = 122 $\bar{c}$ = 0 $\bar{\phi}$ = 33	γ = 115 $\bar{c}$ = 0 $\bar{\phi}$ = 28	γ = 125 c = 0 ϕ = 39	γ = 120 c = 0 ϕ = 32	γ = 155 c = 8000 ϕ = 0
LONG TERM RAPID DRAWDOWN	γ = 140 $\bar{c}$ = 0 $\bar{\phi}$ = 38	γ = 122 $\bar{c}$ = 0 $\bar{\phi}$ = 33	γ = 122 $\bar{c}$ = 0 $\bar{\phi}$ = 33	γ = 115 $\bar{c}$ = 0 $\bar{\phi}$ = 28	γ = 125 c = 0 ϕ = 39	γ = 120 c = 0 ϕ = 32	γ = 155 c = 8000 ϕ = 0

SUMMARY OF TRIAXIAL TEST DATA	
STATION	17+90
OFFSET	31' Lt.
DEPTH	5'-7'
$\bar{c}$	0 psf
$\bar{\phi}$	30.1°

FACTORS OF SAFETY		
SHORT TERM	A	2.0
LONG TERM	B	1.7
LONG TERM RAPID DRAWDOWN	C	0.908



REVISION NO. 1

SCALE: 1" = 10' HORIZONTAL
1" = 10' VERTICAL

SPILL THROUGH EAST ABUTMENT SLOPE
STA. 18+00 - REVISION NO. 1

COUNTY OF	ITEM NO.	SHEET NO.
GREENUP	9-1038.0	X3

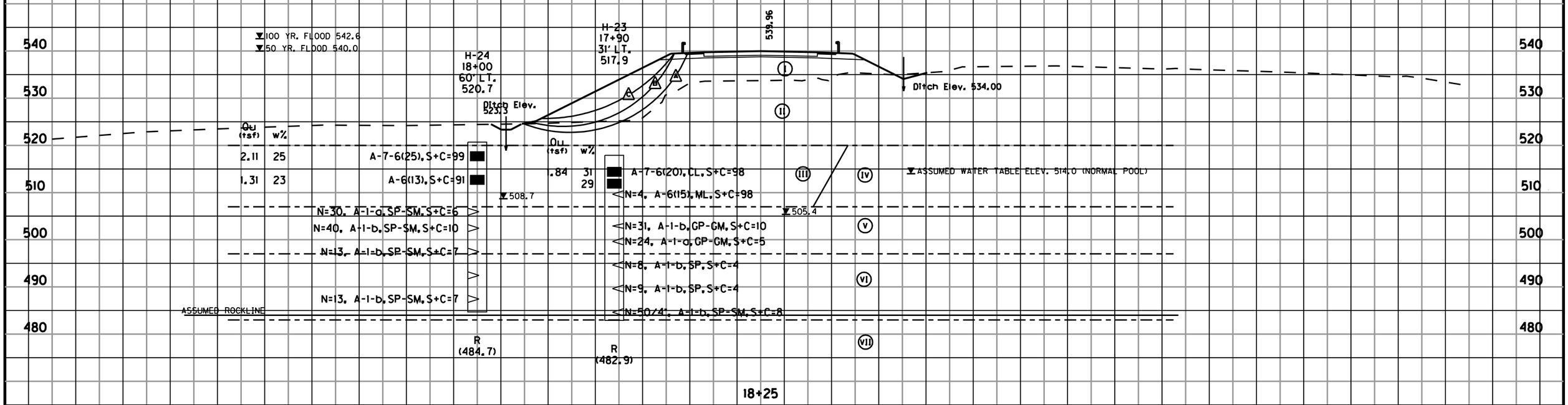
DATE

APPROVED BY

ASSUMED SOIL STRENGTH PARAMETERS																							
SOIL	I		II		III		IV		V		VI		VII	
SHORT TERM	γ =	140	γ =	122	γ =	122	γ =	115	γ =	125	γ =	120	γ =	155
	c =	0	c =	700	c =	1000	c =	0	c =	0	c =	0	c =	8000
	ø =	45	ø =	0	ø =	0	ø =	28	ø =	39	ø =	32	ø =	0
LONG TERM	γ =	140	γ =	122	γ =	122	γ =	115	γ =	125	γ =	120	γ =	155
	c̄ =	0	c̄ =	0	c̄ =	0	c̄ =	0	c̄ =	0	c̄ =	0	c̄ =	8000
	ø̄ =	45	ø̄ =	33	ø̄ =	33	ø̄ =	28	ø̄ =	39	ø̄ =	32	ø̄ =	0
LONG TERM RAPID DRAWDOWN	γ =	140	γ =	122	γ =	122	γ =	115	γ =	125	γ =	120	γ =	155
	c̄ =	0	c̄ =	0	c̄ =	0	c̄ =	0	c̄ =	0	c̄ =	0	c̄ =	8000
	ø̄ =	45	ø̄ =	33	ø̄ =	33	ø̄ =	28	ø̄ =	39	ø̄ =	32	ø̄ =	0

SUMMARY OF TRIAXIAL TEST DATA	
STATION	17+90
OFFSET	31' Lt.
DEPTH	5'-7'
c̄	0 psf
ø̄	30.1°

FACTORS OF SAFETY		
SHORT TERM	A	2.0
LONG TERM	B	2.1
LONG TERM RAPID DRAWDOWN	C	0.988



REVISION NO. 2

SCALE: 1" = 10' HORIZONTAL
1" = 10' VERTICAL

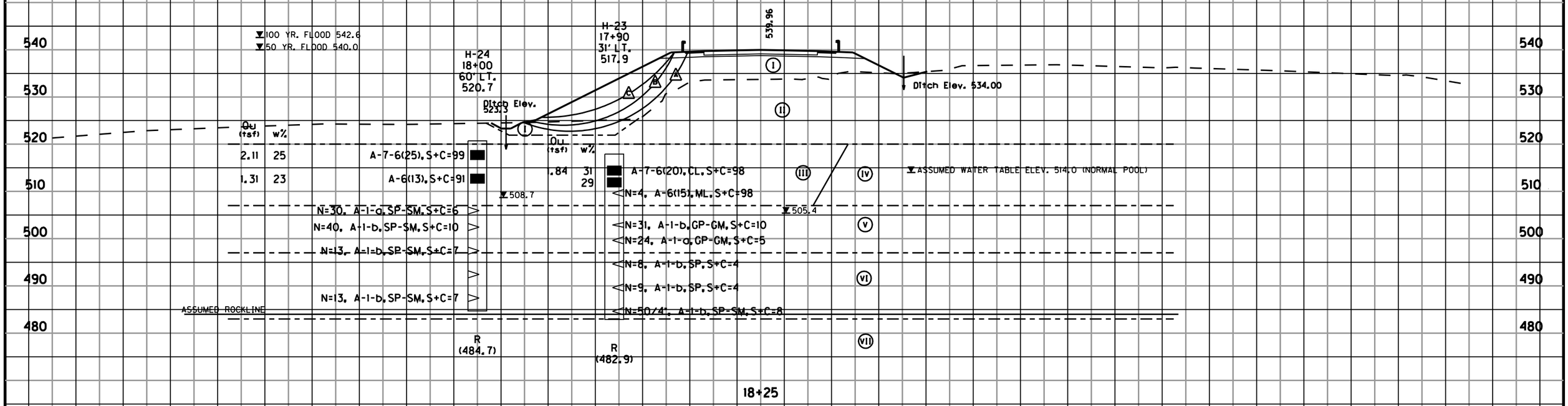
SPILL THROUGH EAST ABUTMENT SLOPE
STA. 18+00 - REVISION NO. 2

COUNTY OF	ITEM NO.	SHEET NO.
GREENUP	9-1038.0	X3

ASSUMED SOIL STRENGTH PARAMETERS							
SOIL	I	II	III	IV	V	VI	VII
SHORT TERM	γ = 140 c = 0 ø = 45	γ = 122 c = 700 ø = 0	γ = 122 c = 1000 ø = 0	γ = 115 c = 0 ø = 28	γ = 125 c = 0 ø = 39	γ = 120 c = 0 ø = 32	γ = 155 c = 8000 ø = 0
LONG TERM	γ = 140 c̄ = 0 ø̄ = 45	γ = 122 c̄ = 0 ø̄ = 33	γ = 122 c̄ = 0 ø̄ = 33	γ = 115 c̄ = 0 ø̄ = 28	γ = 125 c = 0 ø = 39	γ = 120 c̄ = 0 ø̄ = 32	γ = 155 c = 8000 ø = 0
LONG TERM RAPID DRAWDOWN	γ = 140 c̄ = 0 ø̄ = 45	γ = 122 c̄ = 0 ø̄ = 33	γ = 122 c̄ = 0 ø̄ = 33	γ = 115 c̄ = 0 ø̄ = 28	γ = 125 c = 0 ø = 39	γ = 120 c = 0 ø = 32	γ = 155 c = 8000 ø = 0

SUMMARY OF TRIAXIAL TEST DATA	
STATION	17+90
OFFSET	31' Lt.
DEPTH	5'-7'
c̄	0 psf
ø̄	30.1°

FACTORS OF SAFETY		
SHORT TERM	A	1.9
LONG TERM	B	2.2
LONG TERM RAPID DRAWDOWN	C	1.23



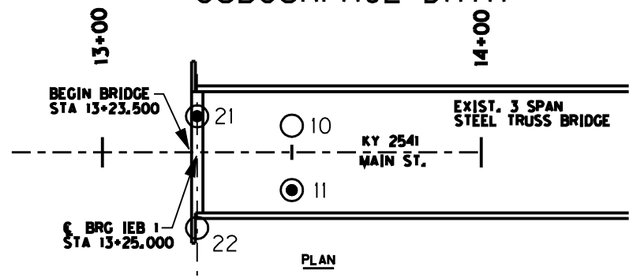
REVISION NO. 3

SCALE: 1" = 10' HORIZONTAL
1" = 10' VERTICAL

SPILL THROUGH EAST ABUTMENT SLOPE
STA. 18+00 - REVISION NO. 3

SUBSURFACE DATA

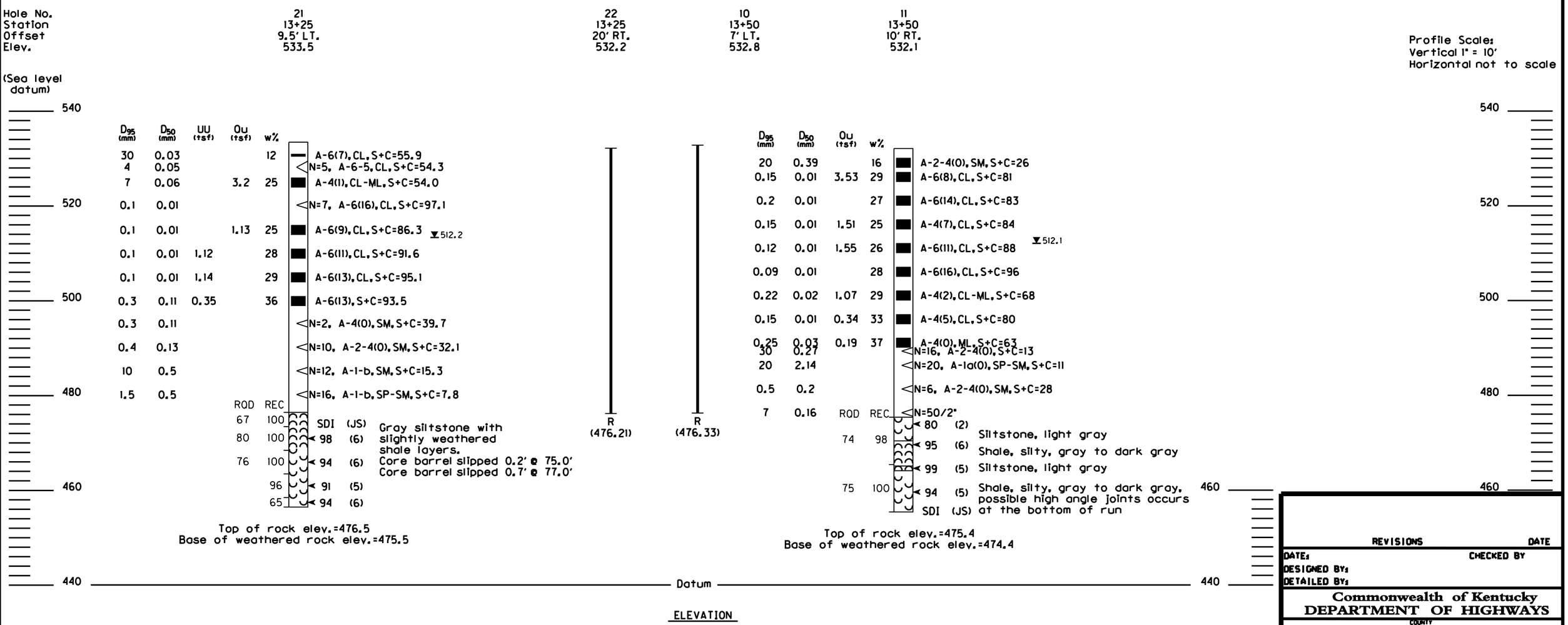
Plan Scale 1" = 20'



PLAN

END BENT #1 STATION 13+25.00
APPROXIMATE ROADWAY GRADE
ELEVATION = 539.1

Profile Scale:
Vertical 1" = 10'
Horizontal not to scale



ELEVATION

REVISIONS	DATE

DATE:
DESIGNED BY:
DETAILED BY:
CHECKED BY

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY: GREENUP

ROUTE: KY 2541
CROSSING: LITTLE SANDY RIVER

SUBSURFACE DATA

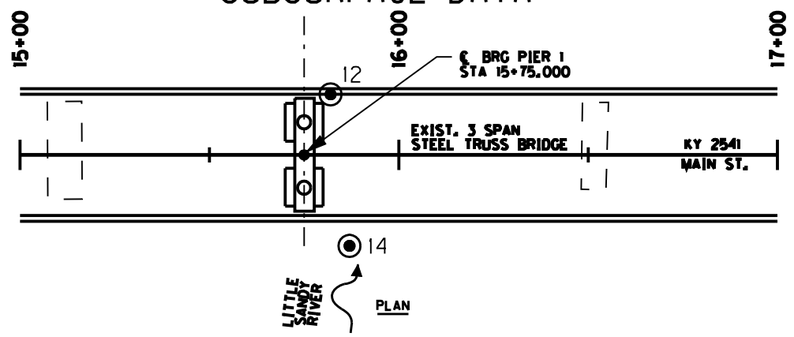
H. C. NUTTING
A Terracon COMPANY

SHEET NO.
DRAWING NO.

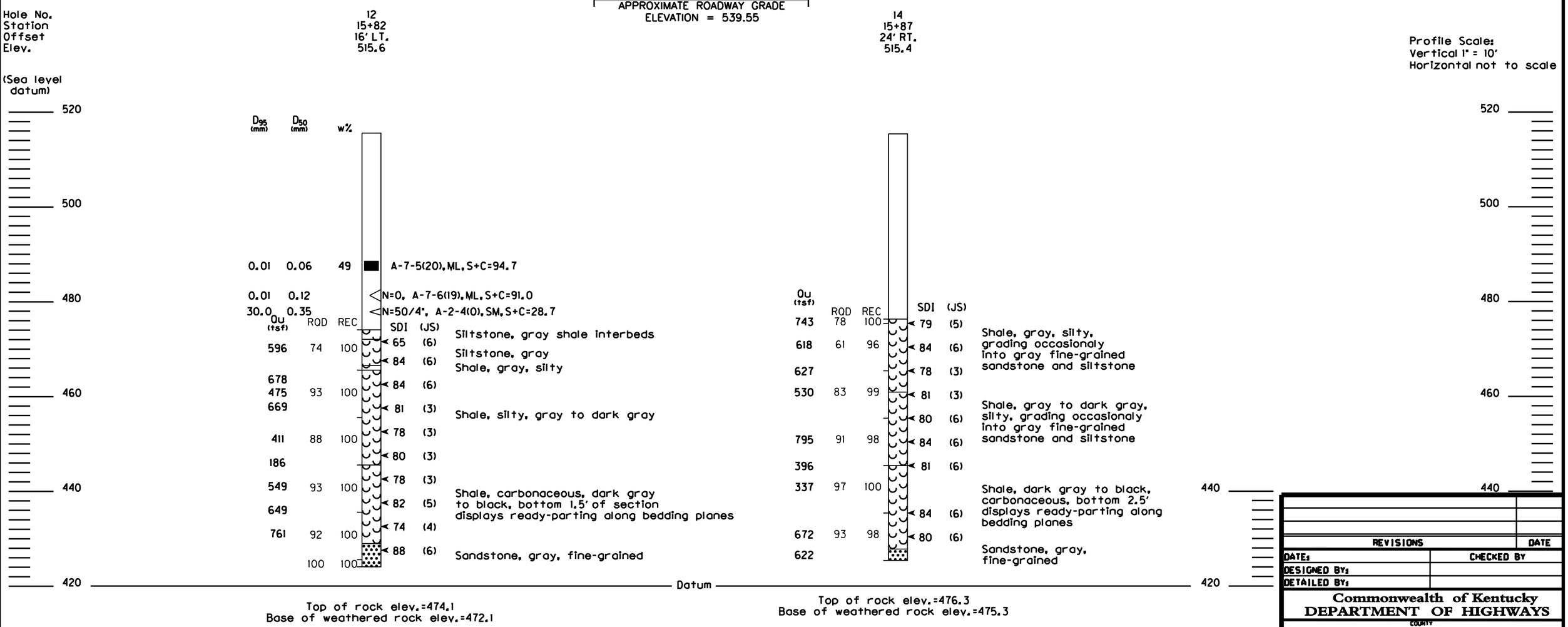
ITEM NO.
9-1038.00

SUBSURFACE DATA

Plan Scale 1" = 20'



Profile Scale:
Vertical 1" = 10'
Horizontal not to scale



ELEVATION

REVISIONS	DATE

DATE:	CHECKED BY
DESIGNED BY:	
DETAILED BY:	

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY: GREENUP

ROUTE: KY 2541 | CROSSING: LITTLE SANDY RIVER

SUBSURFACE DATA

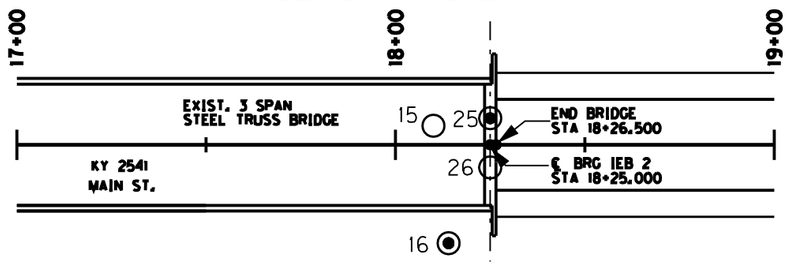
H. C. NUTTING
A Terracon COMPANY

SHEET NO.
DRAWING NO.

ITEM NO.
9-1038.00

SUBSURFACE DATA

Plan Scale 1" = 20'



END BENT #2 STATION 18+25.00
APPROXIMATE ROADWAY GRADE
ELEVATION = 539.90

PLAN

Profile Scale:
Vertical 1" = 10'
Horizontal not to scale

Hole No.	15	16	25	26
Station	18+10	18+14	18+25	18+24
Offset	5' LT.	26' RT.	7' LT.	6' RT.
Elev.	533.2	533.7	533.6	534.0

(Sea level datum)

540 / 520 / 500 / 480 / 460 / 440

Hole No. 16

D_{95} (mm)	D_{50} (mm)	w%		
0.1	0.01	21		A-6(16), CL, S+C=92
3	0.01	23		A-7-6(12), CL, S+C=68
0.2	0.03	27		A-4(3), CL-ML, S+C=72
0.2	0.03	25		A-4(3), CL-ML, S+C=72 ▼518.7
0.25	0.1	29	<	N=5, A-4(0), SM, S+C=42
0.25	0.08	30	<	N=3, A-4(0), SM, S+C=50
19	0.92	16	<	N=45, A-1-b(0), SM, S+C=14
25	4.44	18	<	N=44, A-1-b(0), GP-GM
6	0.53	23	<	N=8, A-1-b(0), SP-SM, S+C=6
14	0.44	18	<	N=25, A-1-b(0), SM, S+C=17

Heaving sands encountered at 30'. Water introduced to hole below 35'

R (488.74)

RQD	REC	SDI	(JS)	
		91	(6)	Shale, sandy, dark gray, trace fossiliferous, (carbonized plant remains); bottom 0.5' of core brecciated; top 0.3' indurated clay
76	98	91	(6)	
92	100	96	(6)	Siltstone, light gray, grading occasionaly into gray, silty shale
99	100	96	(6)	Shale, gray, silty, grading occasionaly into light gray siltstone
		98	(6)	

Top of rock elev.=488.7
Base of weathered rock elev.=487.7

Hole No. 25

D_{95} (mm)	D_{50} (mm)	Q_u (tsf)	w%		
2	0.01	0.66	26		A-6(9), CL, S+C=79
0.25	0.05	0.74	20		A-4(0), S+C=62
0.25	0.08			<	N=4, A-4-0, SM, S+C=49
0.25	0.08			<	N=7, A-4(0), SM, S+C=9
2	0.08			<	N=7, A-4(0), SM, S+C=49 ▼513.1
18	1			<	N=26, A-1-b, SM, S+C=9
22	3.36			<	N=53, A-1-b, SP-SM, S+C=8
9.5	1.9			<	N=13, A-1-b, SP, S+C=3
5	0.52			<	N=19, A-1-b, SP-SM, S+C=7

R (490.99)

RQD	REC	SDI	(JS)	
				Siltstone, gray
48	100	98	(6)	Shale, gray, seams of approx. 6.8" intervals, highly weathered at 45.7'-46'
				Shale, gray, highly weathered
72	100	98	(6)	Siltstone, gray, with highly weathered gray shale seams every 6"-8"
96	100	95	(5)	Siltstone, gray, with highly weathered gray shale seams
97	100	95	(6)	

Top of rock elev.=488.6
Base of weathered rock elev.=486.6

Datum

ELEVATION

REVISIONS	DATE

DATE:	CHECKED BY
DESIGNED BY:	
DETAILED BY:	

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY: GREENUP

ROUTE: KY 2541 — CROSSING: LITTLE SANDY RIVER

SUBSURFACE DATA

H. C. NUTTING
A Terracon COMPANY

SHEET NO.
DRAWING NO.

ITEM NO. 9-1038.00