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Final Structure Geotechnical Report

BRIDGE REPLACEMENT
ILLINOIS 13 OVER BIG MUDDY RIVER
JACKSON COUNTY, ILLINOIS

PTB 147, ITEM 41
ROUTE: FAP 331 (IL RTE 13)
SECTION: 12-1B-1
CONTRACT NO.: 78056
PROJECT NO.: P-99-027-08
STRUCTURE NO.: EXISTING 039-0013 (EB), PROPOSED 039-0075
EXISTING 039-0049 (WB), PROPOSED 039-0076

Thomas J. Casey, P.E.
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June 18, 2010
Revised July 15, 2010

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SCI ENGINEERING, INC.
CONSULTANTS IN DEVELOPMENT, DESIGN AND CONSTRUCTION
GEOTECHNICAL
ENVIRONMENTAL
NATURAL RESOURCES
CULTURAL RESOURCES
CONSTRUCTION SERVICES

July 15, 2010

Mr. Bruce Schopp, P.E., S.E.
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RE: Final Structure Geotechnical Report
Bridge Replacement
Illinois 13 over Big Muddy River
Jackson County, Illinois
PTB 147, Item 41
Route: FAP 331 (IL Rte 13)
Section: 12-1B-1
Contract No.: 78056
Project No.: P-99-027-08
Structure No.: 039-0013 (EB), 039-0075 proposed
039-0049 (WB), 039-0076 proposed
SCI No.: 2008-3043.50

Dear Mr. Schopp:

Enclosed is our *Final Structure Geotechnical Report (SGR)* dated June 2010 and revised July 15, 2010. This revision is per request from the Illinois Department of Transportation Foundations and Geotechnical Unit. This report should be read in its entirety, and our recommendations considered in the design and construction of the proposed bridge replacement. Please call if you have any questions.

Respectfully,

SCI ENGINEERING, INC.

Julie A. Miller

Julie A. Miller, P.E.
Project Engineer

Thomas J. Casey, P.E.
Senior Engineer

JAM/TJC/tlw

Enclosure

One additional copies and one electronic copy submitted.

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JACKSON COUNTY, ILLINOIS
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1.0 PROJECT DESCRIPTION

The geotechnical study summarized in this report was performed for the proposed replacement of the bridges that carry Illinois Route 13 over the Big Muddy River in Jackson County, Illinois. The location of the site is shown on the *Vicinity and Topographic Map*, Figure 1. The purpose of our study was to review and further explore the subsurface conditions and develop design and construction recommendations for the project. Additionally, we were tasked with reevaluating the stability issues related to long-term and seismic conditions. It is our understanding that insufficient factors of safety were calculated based on a letter from IDOT dated April 22, 2005 that was provided to SCI with available data. In addition to the stability concerns, it appears that excessive settlement may be influencing the overall condition of the bridge approach slabs and embankments.

Illinois 13 is a four-lane divided highway in the Murphysboro area. Two parallel structures, east-bound (EB) structure (039-0013) and west-bound (WB) structure (039-0049), carry Illinois 13 over the Big Muddy River. The three-span structures are approximately 416.5 feet in length with a skew of 27 degrees, 30 minutes. The EB structure was built in 1954 and reconstructed in 1980. The bearings were adjusted in 1959, 1974, and 1986. In 2002, the joint was removed from the EB structure. The WB structure was constructed in 1974 and has never been reconstructed. The bearings on this structure are over-extended and the joints are no longer functional. The bridge beams for both structures have been cut off to make room for expansion/contraction of the steel beams. The gap has since closed.

As described above, the bridges have been in need of repair for several years due to problems with the subsurface conditions beneath the bridge. Based on the IDOT *Bridge Condition Report* provided to SCI, excessive settlement has resulted in distress to the bridge abutments and at both structures the approach pavements have undergone excessive deflection over the lifetime of the structures.

The two existing structures will be removed and replaced with two longer 5-span structures with a planned back-of-abutment to back-of-abutment length of approximately 466 feet. The longer structures will move the abutments and approach slabs further back allowing the bridge to span more of the settlement prone soils. The longer span will also facilitate flattening of the end slope to a 3H:1V on both abutments. This change is dictated by the stability analysis and will also further serve to decrease the loading of the poor soils underlying the abutments. Each proposed structure will carry two lanes of traffic in each direction and the skew of the bridge will remain the same.

2.0 SUBSURFACE EXPLORATION

2.1 Area Geology

The project site is located approximately ¾ of a mile east of Murphysboro, Illinois. The soils in the immediate area were formed in alluvial deposits classified as the Belknap, Colp, and Hurst groups. The Belknap silt loam is frequently flooded and is somewhat poorly drained. These soils are located closest to the river. The Colp and Hurst groups comprise the slopes and the upland soils. These soils consist of silt loam to silty clay loam and rarely flood.

2.2 Exploration Procedures

In October 2004, a subsurface exploration and laboratory tests were performed by IDOT. Six standard penetration test (SPT) borings, designated B-1S through B-6S were drilled near the existing bridge abutments and piers. A summary of the subsurface conditions encountered in these borings is included in the *Subsurface Conditions* section and was used in our engineering analyses. In July 2009, SCI drilled three additional borings B-101 near the east abutment, and B-103 and B-104 near the west abutment. Table 2.2 below presents a summary of the borings and pertinent information. The boring locations were selected and staked in the field by SCI using approximate distances referenced from existing site features. The station and offset at each boring location was estimated by SCI from the as-built plans dated November 20, 1973 provided by Oates Associates, Inc. (Oates). The elevations were estimated based on measurements from the as-built plans and a USGS survey point located on the east abutment of the southern structure. The field exploration was performed in general accordance with procedures outlined in the *1999 IDOT Geotechnical Manual*. A geologist from SCI was with the drill rig to supervise drilling, log the borings, and perform field unconfined compressive strength tests.

For the most recent exploration, a CME-750 drill rig with hollow-stem augers was used to advance the borings. At IDOT's request, Shelby tube samples were collected at near continuous intervals from a depth of 20 feet to 34 feet in B-101 and at selected locations between a depth of 22 feet and 47 feet in

B-103. In B-104, SPTs were performed with a split-spoon sampler at 2½-foot intervals to a depth of 30 feet and then at 5-foot intervals thereafter. Unconfined compressive strengths of cohesive split-spoon samples were measured with a Rimac testing apparatus or a pocket penetrometer when the sample was not conducive to Rimac testing.

Table 2.1 – Summary of Soil Test Borings

Boring	Type	Date	Depth	Elevation	STA	Offset	
1-S	Abutment	October-04	95.5	383.2	338+68	28	lt wb cl
B-101	Abutment	July-09	36	363.0	339+28	75	lt eb cl
2-S	Pier	October-04	65	354.4	339+62	36	rt wb cl
3-S	Pier	October-04	75	353.7	339+94	30	lt wb cl
4-S	Pier	September-04	75	351.7	341+61	31	rt wb cl
5-S	Pier	October-04	63	351.6	341+88	22	rt eb cl
B-103	Abutment	July-09	47	383.0	343+09	14	rt wb cl
6-S	Abutment	October-04	102.5	383.0	343+17	22	rt eb cl
B-104	Abutment	July-09	99	383.0	343+61	14	rt eb cl

2.3 Subsurface Conditions

Detailed information regarding the nature and thickness of the soils and rock encountered during both explorations, and the results of the field sampling and laboratory testing are shown on the Boring Logs in Appendix A. A *Site Plan* showing the boring locations with respect to the proposed structures is shown on *Figure 2*. The generalized soil profiles for both recent explorations are included as *Subsurface Profile,* Figures 3.

Fill consisting of silty clay (A-6 and A-7), clay (A-7-6), sandy clay (A-4 and A-6), and clayey silt (A-4) was encountered at the surface in Borings B-103 and B-104. The fill was most likely placed during construction of the bridge abutments. The fill extended to depths of 22 feet (El. 352) in B-103 and 28 feet in B-104 (El. 355). The fill appeared to be medium stiff to stiff with blow counts ranging from 6 to 15 blows per foot and moisture contents ranging from 7 to 29 percent.

For the remaining borings, which were not drilled through the embankment soils, natural soils were encountered at the surface. The natural soils in each of the borings consisted of interbedded alluvial soils consisting of low plastic silty clay (A-6 and A-7), low plastic silty clay loam (A-6), sandy clay (A-6), clayey silt (A-4), silt loam (A-4), sandy silt (A-4), and clay (A-7 and A-7-6). The cohesive soils were encountered to depths of boring termination at 36 feet (El. 327) and 47 feet (El. 327) in Borings B-101 and B-103 respectively. In the remaining borings, the cohesive soils extended to depths ranging from 59.5 feet (El. 292.1) to 91.5 feet (El. 302.5). Sand and gravel layers were interbedded throughout the profile. More significant layers of sand and gravel were encountered at the elevations listed in Table 2.2.

Table 2.2 – Summary of Sand and Gravel Layers

Boring	Elevation (feet)
1S	293.2 – 288.2
2S	314.4 – 309.4
	299.9 – 289.9
3S	321.7 – 314.2
4S	297.2 – 288.7
5S	302.1 – 297.1
6S	303.5 – 291.0
B-104	342.0 – 339.0
	311.0 – 306.0
	303.5 – 294.0

The cohesive soils were generally medium stiff to stiff in consistency. However, very soft to medium stiff layers of cohesive soils were encountered predominantly between the elevations of 347 and 317. However, the soft soils were encountered as shallow as El. 357 and extended as deep as El. 302 in some of the borings. The SPT N-values within the soft clay layers ranged from 0 to 6 blows per foot, while the unconfined compressive strength values ranged from 0.2 to 1.5 tons per square foot (tsf). A one-dimensional consolidation test was performed on a relatively undisturbed sample collected within this soil stratum.

Most of the sand and gravel deposits encountered were medium to dense; however, isolated lenses of very loose to loose sand was encountered in thin layers across the site. The N-values within the deposits of loose sand ranged from 0 bpf to 10 bpf.

Rock consisting of clayey shale and coal was encountered in each of the deep borings, which is summarized in Table 2.3. The top of rock ranged from El. 288 to El. 292 across the project site. The shale and coal extended to the termination of the deepest boring elevation wise at an El. of 277 in 4-S.

Table 2.3 – Summary of Groundwater Table and Rock Elevations

Boring	GWT	GWT Elevation	Depth to Rock	Rock Elevation
1-S	28.2	355.0	95	288
B-101	18	345.0	-	-
2-S	4.8	349.6	64.5	290
3-S	3.3	350.4	64.5	289
4-S	7	344.7	63	289
5-S	5.2	346.4	62	290
B-103	45	338.0	-	-
6-S	25.5	357.5	92	291
B-104	40	343.0	91.5	292

Groundwater observations summarized in Table 2.3 were recorded during drilling or within 24 hours of drilling. The groundwater level is subject to seasonal and climatic variations, and other factors; and may be present at different elevations in the future. In addition, without extended periods of observation, prediction of the groundwater level may not be possible.

2.4 Laboratory Test Results

Laboratory testing was performed on select undisturbed samples to include consolidation testing, unconsolidated-undrained, and consolidated-undrained triaxial tests along with index property testing. The results of these tests are summarized in Tables 2.4 and 2.5.

Table 2.4 – Summary of Atterberg Limits and Triaxial Test Data for Shelby Tubes

Boring	Depth (feet)	Elevation (feet)	Atterberg Limits		Unconsolidated – Undrained Triaxial Compressive Strength (psf)	Consolidated-Undrained Total Stress Results		Consolidated-Undrained Effective Stress Results	
			LL	PI		Cohesion (psf)	Phi (degrees)	Cohesion (psf)	Phi (degrees)
B-101	20 – 22	343 – 341	35	18	2,000	--	--	--	--
	22 – 24	341 – 339	--	--	2,400	--	--	--	--
	24 – 26	339 – 337	--	--	2,200	--	--	--	--
	26 – 28	337 – 335	--	--	2,200	--	--	--	--
	28 – 30	335 – 333	25	7	--	600	21	420	30
	32 – 34	331 – 329	--	--	6,000	--	--	--	--
B-103	22 – 24	352 - 350	--	--	4,600	--	--	--	--
	27 – 29	347 – 345	47	30	--	530	18	640	22
	34 –-36	340 – 338	--	--	2,600	--	--	--	--
	45 – 47	329 – 327	--	--	6,200	--	--	--	--

Table 2.5 – Summary of Consolidation Testing

Boring	Depth (feet)	Elevation (feet)	Atterberg Limits		Compression Index, Cc	Recompression Index, Cr	Secondary Compression Index, Cα	Coefficient of Consolidation, Cυ (in2/min)
			LL	PI				
B-101	20 – 22	343 – 341	35	18	0.208	0.018	0.011	0.120

3.0 GEOTECHNICAL EVALUATIONS

In order to provide the most economical and feasible solution to the soil issues affecting the structures, we evaluated the amount of settlement that has likely occurred based on all available data collected and reviewed at the time of this report. This information includes subsurface investigations performed by IDOT and SCI, as–built plans, a bridge condition report, preliminary TS&L plans, and verbal communications with IDOT personnel familiar with the project. By evaluating the existing conditions and the previous settlement, SCI hopes to determine the primary mechanisms influencing the deformation and provide recommendations to mitigate the issues.

3.1 Settlement

Approximately 20 to 28 feet of fill was added to create the east and west embankments. If one is to assume the soft soils were normally consolidated at the time of construction, a maximum settlement of 26 inches would have occurred due to the weight of the fill. This amount of settlement would vary with different heights of the embankment fill as well as with variations of the thickness of the highly compressible soils. Based on the limited amount of laboratory test data, approximately 25 percent of this settlement may have occurred as long as 3+ years after completion of construction. This may have been anticipated since sand drains were placed under the west abutment of the westbound lanes. If one considers the potential secondary compression of these very soft soils, large post-construction deformations are possible.

To reduce the effects of settlement on the new bridge, the spans have been lengthened to move both abutments further away from the soft soil zones. This also allows the riverside slope to be flattened to 3 horizontal to 1 vertical (3H:1V) from the existing 2H:1V. Flattening the slopes will allow for removal of some of the overburden material, which will reduce the load imposed and the potential for further settlement.

3.1.1 Ancillary Settlement Mitigation Techniques

The following recommendations should be considered as low cost alternatives that will help reduce the effects of settlement after replacement of the two structures. Since the replacement will be staged construction, where one bridge will be utilized while the other is completely replaced, there will be time available to further pre-load the very soft soils underlying the existing embankment. This could be accomplished by adding surplus fill in the approach areas during construction. Once construction of the structure is complete, the excess fill could be removed and used to perform any final grading or redressing of the embankment slopes. For this preload, we recommend approximately 5 feet of fill be placed from the new abutment to STA 343+8. It should be sloped to promote drainage of surface water away from the surcharge and provide access for working on the structure. Although it will be difficult to quantify the direct results of the preloading program, this mitigation technique combined with the abutment setback, will reduce the long-term maintenance issues the bridges currently experience. If the preloading is utilized, we recommend that two settlement plates be installed within each approach abutment area to monitor the rate of settlement.

Based on conversations with District 9 personnel, we understand that IDOT had previously utilized lightweight fill in a similar situation within the district. Lightweight fill works to reduce the amount of settlement of soft soils by decreasing the weight applied to them relative to conventional fill. An important note, when using certain lightweight fills it should not be placed below the high groundwater level due to buoyancy concerns. Based on observations by IDOT personnel concerning high water marks within the creek, the use of lightweight fill should be limited to the upper two or three feet of the finished subgrade. More information can be provided with regards to the geofoam option if this method needs to be further evaluated.

Based on information provided, it appears that the planned western approach slab on the eastbound lane will extend from the bridge to STA 343+66. This should reduce the effects of future differential settlements anticipated. Based on available data, it appears that the very soft soils begin to taper to less than 5 feet in thickness near STA 343+60, which represents the western extent of data. We would recommend extending the planned west abutment approach slab for the westbound lane in a similar manner (approximately 20 feet beyond the existing approach slab terminus.) It may be possible to utilize a second approach slab with the use of a "sleeper" slab to control differential settlement between the two slabs instead of constructing a single long approach slab. Costs of the two options should be evaluated to determine the more economical route.

There are several other more expensive options that could be considered. One such option that was reviewed was ground improvement using rammed aggregate piers (RAP). This type of solution works by modifying the strength of the underlying soils. By improving the compressibility and shear strength of the deposit, the soil mass is less prone to deformation and provides additional resistance for slope stability. Generally, the effective depth for these systems is approximately 40 feet beneath finished grade. For this site where the existing fill is approximately 25 feet on average, the RAP would only be able to treat the upper 15 feet of the very soft soils.

We had discussions with a representative from Geopier Foundations, LLC (Geopier) about potential costs for ground improvement. They stated it would be possible to utilize a "replacement" method from the bottom elevation of the settlement prone soils up to the depth at which point they could install the geopiers. The replacement would include augering a cased hole down though the soft soils and backfilling with a crushed stone to an approximate depth of 40 feet below finished grade, where the traditional RAP would be constructed through the remaining soft soils. This would strengthen the soils to a depth greater than just the RAP piers alone. Preliminary budget numbers provided by Geopier indicated this option would cost approximately $200,000 to $375,000. One additional benefit of this alternative is that the end slope could be increased to 2H:1V as originally constructed, which may result in some savings. Additional design and cost information would be needed to determine if this alternative is viable or not.

3.2 Bridge Approach Slab

The bridge approach slabs should be designed to bear on existing embankment fill or newly placed low plastic structural fill. In evaluating the bearing resistance of the slabs, we recommend using a modulus of subgrade reaction of 150 pounds per square inch per inch of deflection (pci).

3.3 Seismic Considerations

3.3.1 Design Earthquake

Ground shaking at the foundation of structures and liquefaction of the soil under the foundation are the principle seismic hazards to be considered in design of earthquake-resistant structures. Soil liquefaction is possible within loose sand and silt deposits below the groundwater table. Liquefaction occurs when a rapid buildup in water pressure, caused by the ground motion, pushes sand particles apart, resulting in a loss of strength and later densification as the water pressure dissipates. This loss of strength can cause bearing capacity failure while the densification can cause excessive settlement. Potential earthquake damage can be mitigated by structural and/or geotechnical measures or procedures common to earthquake resistant design.

For the purposes of seismic design the bridge has been classified as *Regular* and *Essential*. According to the Illinois Department of Transportation Bridge Manual 2008 edition, the structure should be designed to a design earthquake with a 7 percent Probability of Exceedance (PE) over a 75-year exposure period (i.e. a 1,000-year design earthquake). The 1,000-year design earthquake has a Moment Magnitude (Mw) of 6.9 and a Peak Ground Acceleration (PGA) of 0.33g, as determined from data provided by the United States Geological Survey (USGS) National Seismic Hazard Mapping Project and procedures outlined in the Bridge Manual.

3.3.2 Site Class Determination

The seismic site soil classification for the bridge site was determined from the design earthquake data, the subsurface data, and the procedures described in AGMU Memo 09.1, *Seismic Site Class Definition*, of the IDOT Bridge Manual Design Guides. The Site Class was evaluated using methods defined as the B and C, which include evaluating the SPT N-values and undrained shear strength, S_u. The following results were calculated:

Method $\overline{B}$ using N: 7 to 10 bpf (Site Class E)

Method $\overline{C}$ using S_u: 1120 psf to 1740 psf (Site Class D)

The AGMU states for bridges with individual span lengths less than 200 feet and overall lengths less than 750 feet, the stronger soil determination can be used for design. Based on the span and overall bridge lengths and the guidelines in the AGMU, we recommend that Site Class D be used for the project. Based on Table 3.15.2-1 the Seismic Performance Zone is 3. Seismic design parameters for the site are as follows:

$F_a = 1.29$, $F_v = 2.15$, $S_{DS} = 0.82g$, and $S_{D1} = 0.35g$.

3.3.3 Liquefaction Potential Analysis

The liquefaction potential analysis for the site was conducted using field and laboratory data and the techniques outlined in the National Center for Earthquake Engineering (NCEER) Technical Report NCEER-97-0022. The analysis was used to supplement the previous work performed by IDOT that was provided to SCI by Oates and is included with the new liquefaction analysis. For the seismic hazard evaluation, it is generally not prescribed to assume that earthquakes would coincide with other extreme loading events, (i.e. reoccurring flood events) unless the structure is considered critical, at which time engineering judgment may be used to provide additional conservatism to the analysis, if necessary. The groundwater elevation was estimated from the end of boring conditions. Sands located above the groundwater table are not susceptible to liquefaction.

Based on our analyses, the majority of the soils have sufficient strength and/or a fines content that make the threat of liquefaction minimal during the design earthquake. However, isolated relatively thin (< 5 feet) layers of loose sands encountered at various depths ranging from El. 345 to El. 296 feet at the western abutment and at the pier locations are susceptible to liquefaction. Potentially liquefiable soils were not encountered beneath the eastern abutment. The results of the recent and original IDOT liquefaction analyses are presented in Appendix B as well as summarized below.

While the amount of the seismically-induced settlement is dependent on the magnitude and distance from the seismic event, we estimate that the settlements from the design earthquake will be negligible and relatively uniform in nature so liquefaction mitigation techniques are not required. This evaluation is based on the depth of the liquefiable soils and the isolated nature of the liquefiable layers. Research performed by Youd and Garris[1] (1995) indicated similar results for liquefaction sites where liquefiable layers are overlain by more than 10 feet of non-liquefiable soils. For the effects of the seismic loading on embankment stability, refer to the following section entitled *3.4 Slope Stability*. Additionally, unbraced

[1] Youd, T.L, and Garris, C.T. (1995), "Liquefaction-Induced Ground-Surface Disruption". *Journal of Geotechnical Engineering*. Vol. 121, No.11, pp. 805-809.

length of the piles during liquefaction should not be a concern as the potentially liquefiable layers are relatively thin (less than 5 feet thick) and do not uniformly occur across the site. For the effects of liquefaction on axial pile capacity, refer to Section *3.7 Bridge Foundations* below.

3.4 Slope Stability

We conducted slope stability analyses for the new bridge abutment embankments. These analyses were conducted using the commercially available software programs PCSTABL (developed by Purdue University) and STEDwin 2.74 (developed and marketed by Annapolis Engineering Software), engineering soil properties from the subsurface exploration data, the given slope geometries, the peak ground acceleration (PGA) from the design earthquake, and the procedures for seismic slope stability outlined in Federal Highway Administration (FHWA) publication FHWA-HI-99-012 *Geotechnical Earthquake Engineering*. Although LRFD was specified for the project, to our knowledge there is currently no accepted methodology for utilizing this design philosophy in slope stability analysis. Therefore, we have utilized traditional Allowable Stress Design analyses using Factor of Safety (FS) values presented in the Bridge Manual.

The embankments were evaluated for end slopes at 2H:1V and 3H:1V for the eastern and western abutments, respectively and for side slopes at 2H:1V for the west abutments. The side slopes for the east abutment were not evaluated since the west abutment side slopes worked and were considered to contain the worse soil conditions. For the static, long-term slope stability analyses, effective stress values were used in a simplified soil profile developed for the bridge embankments. For the short-term analyses, total stress values were used. Because undisturbed samples were taken and strength testing performed on select samples, minimum FS values of 1.3 were used for both short-term and long-term scenarios. In each case, the embankments achieved the minimum factors of safety for the static conditions.

For the seismic and yield analyses, reduced total stress values were utilized in the phased process. The first phase (Phase 1) considers a minimum FS of 1.0 while using ½ of the full PGA value for each embankment for seismic loading. According to FHWA-HI-99-012, this approach typically results in approach embankment deformations of less than 12 inches, which is generally considered acceptable for non-critical structures. For profiles that achieved a FS less than 1.0, an analysis of permanent displacement using the yield acceleration was performed (Phase 2). A maximum displacement of 6 inches was calculated using this methodology. This amount of deflection is less than the allowable deformation typically prescribed for embankments of bridge structures. In addition, the failure arc passes through the abutment, which indicates that this analysis is very conservative as the abutment piles would act as a shear key during the event which is not considered within the model. This shear key would likely limit deformations even more than predicted. The third and last phase is to evaluate the affects of liquefaction, which is generally agreed to occur after the earthquake loading has stopped. For this analysis, we consider reduced total stress values for the fine-grained materials and a residual undrained shear strength for the liquefiable layers estimated from the N-values after Seed and Harder, 1990. The post-earthquake liquefaction analysis considers a FS of 1.0. The individual output graphics from the analyses are presented in the report Appendix C.

Subsequently to this analysis, it was determined to flatten both abutment slopes similarly based on discussions with the district. The analyses were not reevaluated since they worked as is within the current guidelines. Future slope stability analysis will be performed as part of the ground improvement design as requested by the district.

3.4.1 Additional Slope Considerations

Portions of the embankment side slopes are severely eroded. These areas should be repaired and appropriate erosion and sediment control measures should be used during construction. These erosion measures include proper contouring during site grading activities, the installation of siltation fences, and/or inlet protection to minimize the loss of ground and the transportation of sediment onto adjacent properties or into waterbodies. Depending on the length of time the subgrade is exposed and the amount of siltation that occurs, it may be necessary to periodically remove materials collected by the sediment control systems. Timely sodding and/or seeding of sloped surfaces will help reduce this potential problem from reoccurring in the future.

3.5 Scour

Based on the TS&L received in February 2010, the design scour elevations for a 100-year event were given for the abutment and piers. Table 3.1 presents the design scour elevations; however these elevations are subject to change in the final design.

Table 3.1 – Summary of Design Scour Elevation

Abutment/ Pier	Design Scour Elevation (ft)
East Abutment	373.8
Pier 1	349.1
Pier 2	318.0
Pier 3	318.0
Pier 4	345.0
West Abutment	373.8

3.6 Mining Activity

According to the Illinois State Geological Survey, there are several former coal mines that are situated within Jackson County. However, there is no visual evidence of subsurface mining activity at the site and according to the Murphysboro Quadrangle, Directory of Coal Mines in Illinois map, dated July 2008, the site was not undermined. In addition, the subject site is more than 500 feet away from the nearest mapped mine, which is generally accepted as the distance at which additional analysis of undermining needs to be evaluated. It should be noted that the borings that encountered coal did not encounter any voids that would indicate undermining activities. Therefore, a study of the effects of mining activity on the project is not considered necessary.

3.7 Bridge Foundations

The foundation supporting the proposed bridge must provide sufficient support to resist dead and live loads, including seismic loads. Several potential foundation options were considered for supporting the new bridge structure that include driven steel H-piles, metal shell piles, and drilled shaft foundations. As detailed in the previous sections, liquefiable zones were identified throughout the subsurface profiles. As such, we have recommended that the foundations bear below the liquefiable zones. This would result in terminal elevations within close proximity to bedrock. Due to this required length, we are recommending that the driven H-pile and drilled shaft foundation options extend to and bear on or within bedrock. Skin friction design values are provided for metal shell piles at the abutments only. Skin friction at the pier locations is limited by depth and the presence of potentially liquefiable soils and end bearing H-piles would likely provide a more economical solution between the driven pile options.

For either driven pile foundation option, we recommend a minimum of two test piles be installed to verify length of piles. One test pile should be installed at an end abutment and an interior bent, this will help determine the average penetration of the driven pile into the underlying bedrock. Test piles will also better define the differences in depth to bedrock across the site as it relates to driving/installing piles. Recommendations for all the potential foundation options are provided below.

3.7.1 Driven Steel Piles

The structural capacity of driven piles depends on the allowable stress and cross sectional areas of steel and concrete. Per the Bridge Manual, the Maximum Nominal Required Bearing ($R_{N\ max}$) for the recommended pile section is shown in Table 3.2. Steel H-piles should conform to ASHTO M270 Grade 50 (ASTM 709 Gr 50) or equivalent with a minimum yield stress of 50 kips per square inch (ksi). Metal shell piles should conform to ASTM A252 grade 3 (or equivalent) with a minimum yield stress of 45 ksi.

For either of the driven steel piles, cofferdams will likely be required for the two interior most piers in order to construct the pile caps. The cofferdam should be properly designed and submitted for review by IDOT prior to construction. The cost of the cofferdams may indicate that drilled shaft foundations may be more economical for the interior piers for this project. However, recommendations are provided for H-piles at the pier foundations and both piles for the abutments.

We recommend a minimum driven pile center to center spacing of three pile diameters, as recommended by the All Geotechnical Manual Users (AGMU) 10.2. The maximum spacing shall be limited to 3.5 times the effective footing thickness plus 1 foot, but not to exceed 8 feet. Once the final spacing is determined, the piles should be evaluated for group effects.

Table 3.2 – Maximum Nominal Required Bearing (R_N max) vs. Pile Section

Pile Description	R_N max (kips)
HP 12×53	419
HP 12×63	497
HP 14×73	578
HP 14×89	705 1
Metal Shell 12" OD w/ 0.25" walls	355
Metal Shell 14" OD w/ 0.25" walls	416
Metal Shell 14" OD w/ 0.312" walls	516

Note 1 – Wave equation analysis required during driving

3.7.1.1 H-Pile Recommendations

For H-piles driven into rock, skin friction was not considered during the design, only end bearing. Based on the IDOT AGMU 10.2, a geotechnical resistance factor (φ_G) of 0.55 was used for the design of the H-pile foundations. Geotechnical losses were not considered necessary in the static pile design. A summary of the design capacities, or factored resistance available (R_F) is presented in the following tables for each H-pile type. "Hard driving" conditions are not likely to occur, therefore, pile shoes are not required. During the seismic event the Bridge Manual allows the use of a Geotechnical Resistance Factor (φ_G) of 1.0 instead of 0.55 for the static analysis.

Even though the H-piles are driven into bedrock and are designed as purely end-bearing piles, the effects of liquefaction were considered during the design process. First, the piles are founded well below any potentially liquefiable layer eliminating any bearing capacity issues. Second, the seismic capacities are presented in Tables 3.3, 3.4 and 3.5 and include the reduction due to liquefaction where appropriate. This reduction should be considered as a worst case scenario as it assumes settlement of the non-liquefiable soils above the liquefied zone resulting in down-drag. As previously mentioned, research has shown that generally this type of isolated liquefaction does not result in surface disruption, which would indicate that settlement of the entire upper non-liquefied soil column is unlikely to occur.

Table 3.3 – Summary of Western Abutment H-Pile Design Capacity

Pile Description	Geotechnical Resistance Factor, φ_G	Factored Resistance Available, R_F (kips)	Geotechnical Seismic Resistance Factor, φ_G	Factored Seismic Resistance Available[2], R_F (kips)
HP 12×53	0.55	230	1.0	105
HP 12×63	0.55	270	1.0	180
HP 14×73	0.55	320	1.0	205
HP 14×89	0.55	390[1]	1.0	330

Note 1 – Wave equation analysis required during driving
Note 2 – Considers down-drag due to liquefaction.

Table 3.4 – Summary of Eastern Abutment H-Pile Design Capacity

Pile Description	Geotechnical Resistance Factor, φ_G	Factored Resistance Available, R_F (kips)	Geotechnical Seismic Resistance Factor, φ_G	Factored Seismic Resistance Available, R_F (kips)
HP 12×53	0.55	230	1.0	420
HP 12×63	0.55	270	1.0	498
HP 14×73	0.55	320	1.0	578
HP 14×89	0.55	390[1]	1.0	706

Note 1 – Wave equation analysis required during driving

Table 3.5 – Summary of Pier H-Pile Design Capacity

Pile Description	Geotechnical Resistance Factor, φ_G	Factored Resistance Available, R_F (kips)	Geotechnical Seismic Resistance Factor, φ_G	Factored Seismic Resistance Available[2], R_F (kips)
HP 12×53	0.55	230	1.0	220
HP 12×63	0.55	270	1.0	295
HP 14×73	0.55	320	1.0	340
HP 14×89	0.55	390[1]	1.0	466

Note 1 – Wave equation analysis required during driving
Note 2 – Considers effects due to liquefaction.

For uplift resistance calculations, an allowable adhesion of 0.4 ksf may be used for the H-piles in soil below the planned scour elevation, if any. The uplift due to the embedment in the shale/coal should be neglected.

The pile lengths, as shown in Table 3.6, were estimated from the encountered bedrock elevations and the top elevations estimated from existing and preliminary TS&Ls. The pile lengths also assume on average 3 feet of embedment into the rock. Although variations in bedrock within the borings is minimal, the estimated pile lengths should be adjusted based on the test pile results.

Table 3.6 – Summary of H-Pile Lengths

Pile Locations	Assumed Pile Top Elevation (ft)	Estimated Pile Tip Elevation (ft)	Estimated Pile Length (ft)
East Abutment	377	285	92
Pier 1	377	286	91
Pier 2	320	286	34
Pier 3	320	286	34
Pier 4	377	286	91
West Abutment	377	288	89

3.7.1.2 Metal Shell Pile Recommendations

As previously discussed, due to limited frictional resistance available at the pier locations and the relatively shallow bedrock, metal shell piles are likely not economically feasible. For the abutment locations, there appears to be sufficient frictional capacity deeper than the potentially liquefiable soils before encountering bedrock. Based on the IDOT AGMU 10.2, a geotechnical resistance factor (φ_G) of

0.55 was used for the design of the metal shell pile foundations. During the seismic event the Bridge Manual allows the use of a Geotechnical Resistance Factor (φ_G) of 1.0 instead of 0.55 for the static analysis, thereby doubling the capacity available.

Also when considering the seismic event, SCI evaluated the potential densification of the potentially liquefiable soils underlying the western abutment due to the installation of the displacement piles. Lin, et al[2] (2004) indicates an average improvement index increase of 84 percent in strength of sands below the groundwater table for displacement piles installed on a pile center to center spacing of 4.0 pile diameters. This reflects an increase of 96 percent inside the pile grouping and an increase of 71 percent on the edge of the pile group. The existing bridge foundations use a range of pile center to center spacings of 2.6 to 4.0 diameters depending on substructure unit. Using the lower bound value of 71 percent the potentially liquefiable soils strength increases enough to resist the potential to liquefy during the design event. Based on this analysis no reduction of capacity was taken during the extreme seismic loading condition in Table 3.7 below for the western abutment. As with the H-piles previously discussed, potentially liquefiable soils were not encountered underlying the eastern abutment. Therefore, liquefaction and down-drag was neglected during the extreme seismic loading event presented in Table 3.7 for the eastern abutment as well.

We also recommend a maximum pile center to center spacing of 4.0 pile diameters to take advantage of the densification effect. Should a larger spacing be required, SCI can provide additional values to use for seismic pile design.

The recommended pile tip and top elevations are shown in Table 3.7 with the R_F for metal shell pile sizes. The top elevations of the piles were estimated from the preliminary TS&L. The tip elevations were estimated from the depth to bedrock in the case of the piers and available factored resistance at the abutment locations.

[2] Effects of Post Driven Pile Soil Densification on Liquefaction Resistance", G. Lin, T. J. Casey, and W. Yang, Proceedings: 11th International Conference on Soil Dynamics and Earthquake Engineering, 2004

Table 3.7 – Summary of Metal Shell Pile Lengths vs. Factored Resistance Available (RF)

Pile Locations	Estimated Pile Top Elevation (ft)	Estimated Pile Tip Elevation (ft)	Estimated Pile Length (ft)	R_F (12-inch) (kips)	Seismic R_F (12-inch) (kips)	14-inch OD			
						R_F (0.25 walls) (kips)	Seismic R_F (0.25 walls) (kips)	R_F (0.31 walls) (kips)	Seismic R_F (0.31 walls) (kips)
East Abutment & Pier 1	377	327	50	140	260	170	310	170	310
	377	317	60	180	320	200	370	200	370
	377	307	70	190	340	220	400	220	400
Pier 2	320	289	31	190	230	230	270	280	370
Pier 3	320	289	31	190	230	230	270	280	370
West Abutment & Pier 4	377	327	50	130	230	140	260	140	260
	377	317	60	140	250	160	290	160	290
	377	307	70	170	310	200	370	200	370

3.7.2 Drilled Shaft Recommendations

Based on the cost implications of the need to use cofferdams at the pier locations, we understand that drilled shafts may provide an economical alternative for founding the piers. As previously discussed, based on the potential for liquefaction at the pier locations, drilled shafts would need to be founded within the bedrock formation. This will allow the drilled shaft to develop sufficient resistance to resist downdrag during and immediately following the seismic event.

3.7.2.1 Drilled Shafts Bearing in Rock

As the anticipated loads for the bridge are not known at the time of this report, it is difficult to determine the depth of rock sockets that will be required. If rock sockets deeper than the depth of the investigation are required, additional drilling will be needed to verify the rock strength values. For the purpose of determining the feasibility of drill shafts we have provided the following values to be used in the design and should be considered estimates. Drilled shafts should be spaced no closer than three shaft diameters, center to center.

Drilled shafts which extend at least two pier diameters into the competent bedrock should be designed for a factored tip resistance of 36 ksf and factored skin friction of 0.75 ksf in order to generate the factored resistance available (R_f). These values reflect a geotechnical resistance factor (φ_G) of 0.5 for strength limit design. For seismic considerations, a (φ_G) of 1.0 should be used to calculate the seismic factored resistance available (R_{fseis}), which results in tip and side resistance values of 72 ksf and 1.5 ksf, respectively. The down-drag acting on the shaft should be calculated using the following formula:

$$DD = (1\ ksf) \times (\pi d) \times (55\ ft),$$

where, d is the diameter of the shaft in feet. The down-drag should be subtracted from R_{fseis} calculated above.

3.7.2.2 Drilled Shaft Uplift Capacity

The uplift capacity of the drilled shaft foundations should be computed as the sum of the weight of the foundation element and the frictional resistance (adhesion) between the pier shaft and the adjacent soil, and rock within the rock socket. An allowable adhesion of 0.4 ksf may be used through the soil portion of the shaft, and 0.55 ksf may be used for the rock. The pier excavations should be observed using a shaft inspection device, and the rock sockets measured in the field to confirm that the estimated uplift capacities are present.

3.7.2.3 Drilled Shaft QA/QC and Construction Considerations

If drilled shafts are used for bridge support, a construction method using a polymer slurry in addition to casing will be required. The drilling rig will need to be at an elevation at least 10 feet above the estimated surface water elevation at the time of drilling to provide room for movement above the casing height. In order to keep casing height to a minimum, each shaft should be drilled, rebar set, and concrete poured as quickly as possible. This will help minimize the shafts from being flooded and require additional clean-out if the river should rise above the casing.

The auger cuttings should be observed as the shafts are drilled to document that competent materials are present. QA/QC for the drilled shafts should include a combination of using a shaft inspection device (SID camera) and/or Crosshole Sonic Logging (CSL) testing to ensure the bottom is clean and verify the integrity of the concrete. This will also verify that the estimated uplift capacities are present.

3.8 Lateral Pile Response

A representation of the shaft response under lateral loading is required for design of the bridge superstructure. The lateral response can be developed by modeling the soil/shaft interaction with the computer program LPILE. Discrete elements are used in LPILE to represent the shaft and non-linear soil using springs. The non-linear soil springs are commonly referred to as P-Y curves.

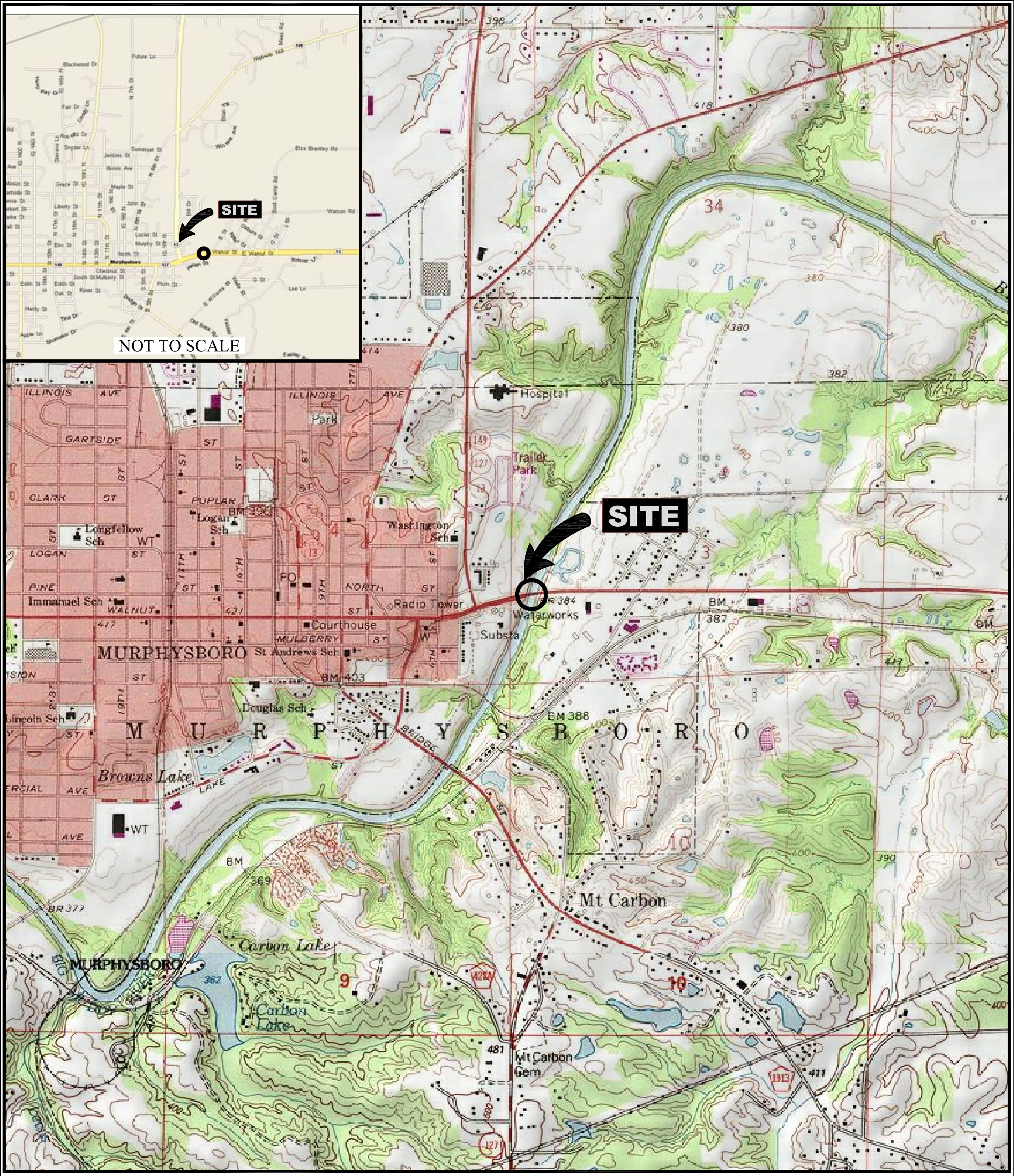
Based on the encountered subsurface conditions, tables for 1-S, 2-S, 3-S, 4-S, 5-S, 6-S, and B-104 summarizing appropriate soil modulus parameters (k), phi angles, cohesion, effective unit weights, and values of strain at 50 percent of the maximum stress (E_{50}) for the LPILE analyses are included in Appendix D (Reference: LPILE User's Manual, Ensoft, Inc., October 2000). When shaft design details and load information are available, LPILE analyses can then be performed.

4.0 CONSTRUCTION CONSIDERATIONS

We understand that one structure will be shut down, demolished and rebuilt while the other structure stays in service. Temporary cantilevered sheetpile walls may be needed to support the roadway remaining open while construction on the closed portion commences. Depending on the location of the proposed abutment piles with respect to the existing bridge foundations, care should be taken during pile driving to avoid the existing footings, or the existing footings should be pre-cored. The construction activities should be performed in accordance with the current *IDOT Standard Specifications for Road and Bridge Construction* and any pertinent special provisions or policies.

5.0 LIMITATIONS

The recommendations provided herein are for the exclusive use of our client and IDOT. They are specific only to the project described, and are based on subsurface information obtained at nine boring locations within the bridge area, our understanding of the project as described herein, and geotechnical engineering practice consistent with the standard of care. No other warranty is expressed or implied. SCI should be contacted if conditions encountered during construction are not consistent with those described.



PROJECT NAME	General Notes/Legend
ILLINOIS 13 OVER BIG MUDDY RIVER PTB 147, ITEM 41 JACKSON COUNTY, ILLINOIS	USGS TOPOGRAPHIC MAP MURPHYSBORO, ILLINOIS QUADRANGLE DATED 1976 PHOTO REVISED 1978 10' CONTOURS POMONA, ILLINOIS QUADRANGLE DATED 1991 PHOTO REVISED 1996 20' CONTOURS MICROSOFT STREETS AND TRIPS 2005
VICINITY AND TOPOGRAPHIC MAP	

DRAWN BY	DKM	DATE	JOB NUMBER
CHECKED BY	JAM	06/2010	2008-3043.50

N
W E
S

SCALE 1" = 2000'

FIGURE 1

PROJECT NAME

ILLINOIS 13 OVER BIG MUDDY RIVER
PTB 147, ITEM 41
JACKSON COUNTY, ILLINOIS

SITE PLAN

SCALE 1" = 100'

JOB NUMBER 2008-3043.50

DATE 06/2010

DRAWN BY DKM

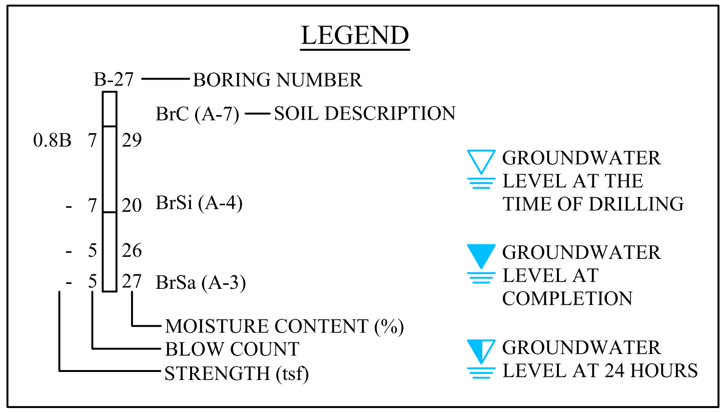
CHECKED BY JAM

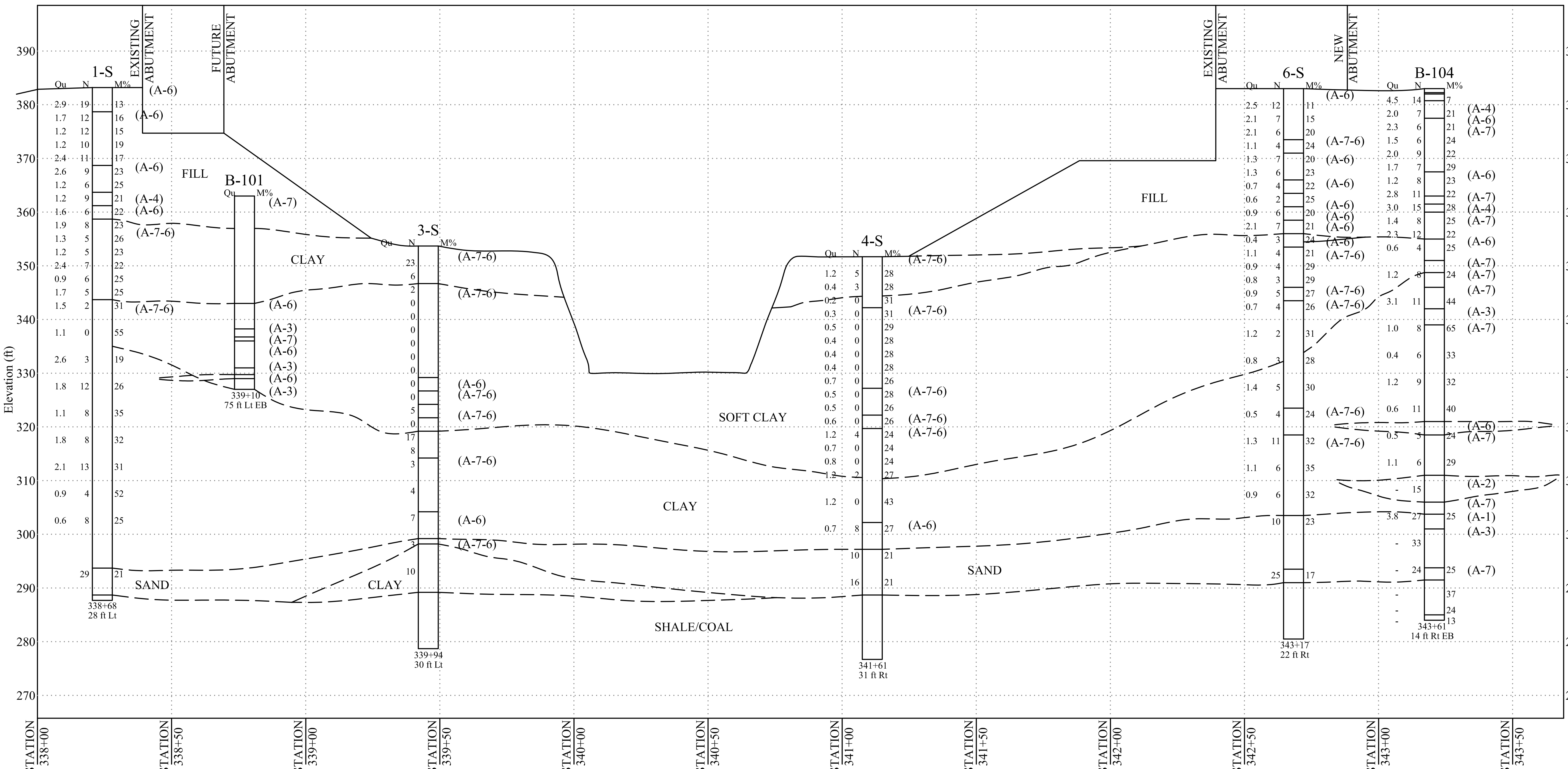
FIGURE 2

General Notes/Legend

- INDICATES APPROXIMATE SOIL BORING LOCATIONS DRILLED 07/2009 BY SCI ENGINEERING
- INDICATES APPROXIMATE SOIL BORING LOCATIONS DRILLED 10/2004 BY IDOT

BASED ON UNDATED PLAN PROVIDED ELECTRONICALLY ON 09/02/2009 FROM OATES ASSOCIATES, INC.. DIMENSIONS AND LOCATIONS ARE APPROXIMATE; ACTUAL MAY VARY. DRAWING SHALL NOT BE USED OUTSIDE THE CONTEXT OF THE REPORT FOR WHICH IT WAS GENERATED.





Soil profile is for illustrative purposes only. Actual conditions will vary.

Distance Along Baseline (ft)

Qu values in tons per square foot.

General Notes/Legend

PROJECT NAME
ILLINOIS 13 OVER BIG MUDDY RIVER
PTB 147, ITEM 41
JACKSON COUNTY, ILLINOIS

SOIL BORING PROFILE
BORINGS 1-S, B-101,3-S, 4-S, 6-S, & B-104

SCALE	VERT. 1" = 40' HOR. 1" = 20'
JOB NUMBER	2008-3043.50
DATE	06/2010
DRAWN BY	DKM
CHECKED BY	JAM
FIGURE	3

Appendix A

ILLINOIS DEPARTMENT OF TRANSPORTATION
District Nine Materials

Bridge Foundation
Boring Log

ILL 13 Over Big Muddy River

Route: ILL 13 Structure Number: 039-0013 and 039-0049 Date: 10/20/2004

Section

Bored By: Bryan Keller

County: Jackson Location: E. Edge of Murphysboro Checked By: Rob Graeff

Boring No 1-S
Station 338+68
Offset 28' LT WB CL
Ground Surface 383.2 Ft

Surf Wat Elev: 339.7
Ground Water Elevation
when Drilling 343.7
At Completion 351.4
At: 24 Hrs: 355.0

	DEPTH	BLOW	Qu tsf	W%
Very stiff, damp, brown, Silty Clay A-6				
		5		
		9	2.9B	13
		10		
378.7				
Stiff, moist, brown, Silty Clay Loam A-6	5.0	2		
		5	1.7B	16
		7		
		1		
		5	1.2S	15
		7		
	10.0	1		
		5	1.2S	19
		5		
371.2				
Very stiff, moist, brown, Silty Clay Loam A-6		1		
		5	2.4S	17
		6		
368.7				
Very stiff, moist, brown mottled grey, Silty Clay A-6	15.0	1		
		4	2.6B	23
		5		
366.2				
Stiff, very moist, brown mottled grey, Silty Clay A-6		1		
		3	1.2B	25
		3		
363.7				
Stiff, moist, grey, Silt Loam A-4	20.0	1		
		5	1.2B	21
		4		
361.2				
Stiff, moist to very moist, grey, Silty Clay A-6		1		
		2	1.6B	22
		4		
358.7				
	25.0	1		

	DEPTH	BLOW	Qu tsf	W%
Stiff, very moist, brown mottled grey, Silty Clay to Clay A7-6		3	1.9B	23
		5		
		1		
		2	1.3B	26
		3		
353.7				
Stiff, very moist, brown, Clay to Silty Clay A7-6	30.0	1		
		2	1.2B	23
		3		
351.2				
Very stiff, very moist, brown mottled, grey, Clay to Silty Clay A7-6		1		
		3	2.4B	22
		4		
348.7				
Medium, very moist, brown mottled grey, Clay to Silty Clay A7-6	35.0	WH		
		3	0.9B	25
		3		
346.2				
Stiff, very moist, brown mottled grey, Clay to Silty Clay A7-6		1		
		2	1.7B	25
		3		
343.7				
Stiff, very moist, grey, Silty Clay Loam A-6	40.0	1		
		1	1.5P	31
		1		
338.7				
Stiff, wet, grey, Clay A7-6	45.0	WH		
		1	1.1B	55
		WH		
333.7				
	50.0	WH		

N-Std Pentr Test: 2" OD Sampler,
140# Hammer, 30" Fall (Type Fail. B-Bulge S-Shear E-Estimated P-Penetrometer)

Route: ILL 13 Date: 10/20/2004

Section:

County: Jackson

Boring No: 1-S

Station: 338+68

Offset: 28' LT WB CL

Ground Surface: 383.2 Ft

Description	DEPTH	BLOW	Qu tsf	W%
Very stiff, moist, brown, Clay A7-6 with Sand Layers		1	2.6B	19
		2		
328.7				
Stiff, very moist, grey, Clay A7-6 with Sand and Silt Loam Layers	55.0	1		
		3	1.8S	26
		9		
	60.0	1		
		2	1.1B	35
		6		
318.7				
Stiff, very moist, brown, Clay A7-6	65.0	2		
		4	1.8S	32
		4		
313.7				
Very stiff, very moist, brown, Clay A7-6	70.0	3		
		6	2.1B	31
		7		
308.7				
	75.0	WH		

Description	DEPTH	BLOW	Qu tsf	W%
Medium, wet, brown, Clay A7-6		1	0.9B	52
		3		
303.7				
Medium, very moist, brown, Clay A7-6 with Sand Layers	80.0	WH		
		3	0.6B	25
		5		
	85.0			
293.7				
Medium, wet, grey, fine to medium Sand 93% Sand 3% Silt 4% Clay	90.0	10		
		13		21
		16		
288.7				
Hard, dry, grey, Clay Shale over Coal 287.7	95.0	100/11"		
Bottom of hole = 95.5 ft Free water observed at 39.5 ft. Elevation referenced to Bk .E. Abut Sta 338+89 Elevation = 383.47 ft. To convert "N" values to "N60" valuesmultiply by 1.25.				
	100.0			

N-Std Pentr Test: 2" OD Sampler,

140# Hammer, 30" Fall (Type Fail B-Bulge S-Shear E-Estimated P-Penetrometer)

ILLINOIS DEPARTMENT OF TRANSPORTATION
District Nine Materials

Bridge Foundation Boring Log

ILL 13 WB Over Big Muddy River

Route: ILL 13 Structure Number: 039-0013 and 039-0049 Date: 10/7/2004

Section

Bored By: Bryan Keller

County: Jackson Location: E. Edge of Murphysboro Checked By: Rob Graeff

Boring No 2-S
Station 339+62
Offset 36' RT CL
Ground Surface 354.4 Ft

Surf Wat Elev: 338.5
Ground Water Elevation
when Drilling 322.4
At Completion
At: 24 Hrs: 349.6

Description	DEPTH	BLOW	Qu tsf	W%
Very stiff, moist, brown, Silty Clay A-6		2		
		5	2.6B	17
		8		
349.9				
Stiff, moist, brown, Silty Clay A-6	5.0	2		
		4	1.7S	20
		5		
347.4				
Medium, very moist, grey, Clay A7-6		1		
		2	0.7B	39
		2		
344.9				
Soft to medium, very moist, grey, Clay to Silty Clay A7-6	10.0	WH		
		1	0.5B	31
		1		
342.4				
Very soft, very moist, grey, Clay to Silty Clay A7-6		WH		
		WH	0.2B	32
		WH		
339.9				
Soft, very moist, grey, Clay to Silty Clay A7-6	15.0	WH		
		WH	0.3B	32
		WH		
337.4				
Soft to medium, very moist, grey, Clay A7-6		WR		
		WH	0.5B	29
		WH		
334.9				
Soft, very moist, grey, Clay A7-6	20.0	WH		
		WH	0.4B	29
		WH		
		WH		
		WH	0.4B	29
		WH		
	25.0	WR		

Description	DEPTH	BLOW	Qu tsf	W%
		WH	0.4B	27
		WH		
327.4				
Medium, very moist, grey, Clay A7-6		WR		
		WH	0.7B	24
		WH		
324.9				
Medium, very moist, brown, Clay A7-6	30.0	1		
		2	0.7B	28
		2		
322.4				
Soft, wet, brown, Clay A7-6 with Sand layers		WH		
		1	0.4B	29
		1		
319.9				
Medium, wet, brown, Clay to Silty Clay A7-6	35.0	WH		
		3	0.8B	26
		5		
317.4				
Stiff, very moist, brown, Clay A7-6 with some Gravel		1		
		2	1.3S	34
		2		
314.9				
Dense, wet, grey, Sandy Gravel 33% Sand 17% Silt 6% Clay 44% Gravel	40.0	1		
		20		14
		21		
309.9				
Stiff, wet, brown, Clay A7-6	45.0	WH		
		1	1.1S	43
		2		
304.9				
	50.0	1		

N-Std Pentr Test: 2" OD Sampler,
140# Hammer 30" Fall (Type Fail B-Bulge S-Shear E-Estimated P-Penetrometer)

Route: ILL 13 **Date:** 10/7/2004

Section:

County: Jackson

Boring No: 2-S

Station: 339+62

Offset: 36' RT CL

Ground Surface: 354.4 **Ft**

Description	Elev.	DEPTH	BLOW	Qu tsf	W%
Stiff, very moist, grey, Clay A7-6 with Sand layers			1	1.3B	28
			2		
	299.9				
Very loose, wet, grey, very fine Silty Sand 92% Sand 2% Silt 6% Clay		55.0	WR		
			WR		25
			WR		
	294.9				
Medium, wet, grey, very fine Silty Sand with some Gravel and Clay layers 79% Sand 4% Silt 8% Clay 9% Gravel		60.0	4		
			5		21
			7		
	289.9				
Hard, dry, black, Clay Shale	289.4	65.0	100/8"		
		70.0			
		75.0			

Bottom of hole = 65.0 ft.

Free water observed at 32.0 ft.

Elevation referenced to Bk East Abutment Sta 338+65 Elevation = 383.64 ft.

To convert "N" values to "N60" values multiply by 1.25.

DEPTH	BLOW	Qu tsf	W%
80.0			
85.0			
90.0			
95.0			
100.0			

N-Std Pentr Test: 2" OD Sampler,
140" Hammer 30" Fall (Type Fail B=Bulge S=Shear E=Estimated P=Penetrometer)

ILLINOIS DEPARTMENT OF TRANSPORTATION
District Nine Materials

Bridge Foundation
Boring Log

ILL 13 EB Over Big Muddy River

Sheet 1 of 2

Route: ILL 13 | Structure Number: 039-0013 and 039-0049 | Date: 10/5/2004

Section | Bored By: Bryan Keller

County: Jackson | Location: E. Edge of Murphysboro | Checked By: Rob Graeff

Boring No 3-S
Station 339+94
Offset 30' LT CL
Ground Surface 353.7 Ft

Surf Wat Elev: 338.5
Ground Water Elevation
when Drilling 324.2
At Completion
At: 24 Hrs: 350.4

Description	DEPTH	BLOW	Qu tsf	W%
Hard, damp, brown, Silty Clay A7-6		2 8 15	4.5P	17
349.2				
Very stiff, damp to moist, brown, Silty Clay A7-6	5.0	2 3 3	2.5P	16
346.7				
Stiff, moist, brown, Silty Clay to Clay A7-6		WH 1 1	1.2B	17
344.2				
Soft, very moist, grey, Clay to Silty Clay A7-6	10.0	WH 1 WH	0.4B	31
		WH WH WH	0.3B	33
	15.0	WH WH WH	0.3B	31
336.7				
Soft to medium, very moist, grey, Clay to Silty Clay A7-6		WH WH WH	0.5B	29
	20.0	WH WH WH	0.5B	29
331.7				
Soft, very moist, grey, Clay to Silty Clay A7-6		WH WH WH	0.3B	30
329.2				
	25.0	WH		

Description	DEPTH	BLOW	Qu tsf	W%
Medium, very moist, grey, Silty Clay A-6		WH WH	0.6B	28
326.7				
Medium, very moist, grey, Clay A7-6		WH WH WH	0.7B	26
324.2				
Medium, very moist, grey, Clay to Silty Clay A7-6 with Sand layers	30.0	WH 3 2	0.7B	32
321.7				
Very loose, wet, grey, fine Silty Sand with Silty Clay Layers 90% Sand; 7% Silt; 3% Clay		WH WH WH		24
319.2				
Mdium, wey, grey, Sandy Gravel 45% Sand 11% Silt 3% Clay; 41% Gravel	35.0	WH 9 8		14
316.7				
Loose, wet, grey, Sandy Gravel 45% Sand 11% Silt 3% Clay; 41% Gravel		4 4 4		
314.2				
Stiff, very moist, brown, Clay A7-6 with some Gravel	40.0	1 1 2	1.2B	30
309.2				
Medium, very moist, grey, Clay A7-6	45.0	1 2 2	0.8B	34
304.2				
	50.0	WH		

N-Std Pentr Test: 2" OD Sampler,
140# Hammer, 30" Fall (Type Fail, B-Bulge S-Shear E-Estimated P-Penetrometer)

Route: ILL 13 Date: 10/5/2004

Section:

County: Jackson

Boring No: 3-S
Station: 339+94
Offset: 30' LT CL
Ground Surface: 353.7 Ft

Description	Elev.	DEPTH	BLOW	Qu tsf	W%
Soft, wet, brown, Silty Clay A7-6 with Silt Loam layers			4	0.4B	27
			3		
	299.2				
Very loose, wet, grey, fine Silty Sand	298.2	55.0	1		
			1		18
Medium, wet, brown, Clay A7-6			2	0.9B	43
	294.2				
Medium, very moist, brown, Clay A7-6		60.0	2		
			3	0.6B	29
			7		
	289.2				
Hard, dry, black Coal		65.0	100/5"		
Hard, dry, black Coal		70.0	100/3"		
Hard, dry, black Coal	278.7	75.0	100/3"		

Bottom of hole = 75.0 ft

Free water observed at 29.5 ft

Elevation referenced to Bk
East Abut Sta 338+89
Elevation = 383.47 ft.

To convert "N" values to "N60" values multiply by 1.25.

	DEPTH	BLOW	Qu tsf	W%
	80.0			
	85.0			
	90.0			
	95.0			
	100.0			

N-Std Pentr Test: 2" OD Sampler,
140# Hammer, 30" Fall (Type Fail. B-Bulge S-Shear E-Estimated P-Penetrometer)

ILLINOIS DEPARTMENT OF TRANSPORTATION
District Nine Materials

Bridge Foundation
Boring Log

ILL 13 WB Over Big Muddy River

Sheet 1 of 2

Route: ILL 13 **Structure Number:** 039-0013 and 039-0049 **Date:** 9/29/2004

Section

Bored By: Bryan Keller

County: Jackson **Location:** E. Edge of Murphysboro **Checked By:** Rob Graeff

Boring No 4-S
Station 341+61
Offset 31' RT CL
Ground Surface 351.7 **Ft**

Surf Wat Elev: 339.7
Ground Water Elevation
when Drilling 342.2
At Completion
At: 96 **Hrs:** 344.7

Description	DEPTH	BLOW	Qu tsf	W%
Stiff, very moist, brown, Silty Clay A7-6		1		
		2	1.2B	28
		3		
347.2				
Soft, very moist, brown mottled grey, Silty Clay A-6	5.0	1		
		1	0.4B	28
		2		
344.7				
Very soft, wet, grey mottled brown, Silty Clay A-6		WH		
		WH	0.2B	31
		WH		
342.2				
Soft, wet, grey, Clay to Silty Clay A7-6	10.0	WH		
		1	0.3B	31
		WH		
339.7				
Soft to medium, wet, brown mottled grey, Silty Clay to Clay A7-6		WH		
		WH	0.5B	29
		WH		
337.2				
Soft, wet, brown mottled grey, Silty Clay to Clay A7-6	15.0	WH		
		WH	0.4B	28
		WH		
		WH		
		WH	0.4B	28
		WH		
	20.0	WH		
		WH	0.4B	28
		WH		
329.7				
Medium, very moist, grey, Clay to Silty Clay A7-6		WH		
		WH	0.7B	26
		WH		
327.2				
	25.0	WH		

Description	DEPTH	BLOW	Qu tsf	W%
Soft to medium, very moist, grey, Clay A7-6		WH	0.5B	28
		WH		
		WH		
		WH	0.5B	26
		WH		
322.2				
Medium, very moist, brown mottled grey, Clay to Silty Clay A7-6	30.0	WH		
		WH	0.6B	26
		WH		
319.7				
Stiff, very moist, grey, Clay A7-6		1		
		2	1.2B	24
		2		
317.2				
Medium, very moist, grey, Clay A7-6	35.0	WH		
		WH	0.7B	24
		WH		
		WH		
		WH	0.8B	24
		1		
312.2				
Stiff, very moist, brown, Clay A7-6	40.0	WH		
		1	1.2B	27
		1		
307.2				
Stiff, wet, brown, Clay A7-6 with fine Sand layers	45.0	2		
		2	1.2B	43
		WH		
302.2				
	50.0	1		

N-Std Pentr Test: 2" OD Sampler,
140# Hammer, 30" Fall (Type Fail, B-Bulge S-Shear E-Estimated P-Penetrometer)

Route: ILL 13

Date: 9/29/2004

Section:

County: Jackson

Boring No: 4-S
Station: 341+61
Offset: 31' RT CL
Ground Surface: 351.7 Ft

Description	DEPTH	BLOW	Qu tsf	W%		DEPTH	BLOW	Qu tsf	W%
Medium, wet, grey, Silty Clay A-6		4	0.7S	27	Botom of hole = 75.0ft.				
		4			Free water observed at 9.5 ft.				
297.2					Elevation referenced to BK W. Abut Elevation = 383.64 ft.				
Loose to medium, wet, grey, fine Silty Sand with some Gravel 81% Sand 7% Silt 4% Clay 8% Gravel	55.0	1			To convert "N" values to "N60" values multiply by 1.25.	80.0			
		5		21					
		5							
292.2									
Medium, wet, grey, fine Silty Sand with some Gravel 69% Sand 9% Silt 7% Clay 15% Gravel	60.0	3				85.0			
		6		21					
		10							
288.7									
Hard, dry, black Coal									
	65.0	100/3"				90.0			
Hard, moist, black, Coal with Clay Shale Layers	70.0	100/5"				95.0			
Hard, dry, grey, Clay Shale									
276.7	75.0	100/3"				100.0			

N-Std Pentr Test: 2" OD Sampler, 140# Hammer, 30" Fall (Type Fail, B-Bulge S-Shear E-Estimated P-Penetrometer)

ILLINOIS DEPARTMENT OF TRANSPORTATION
District Nine Materials

Bridge Foundation
Boring Log

ILL 13 EB Over Big Muddy River

Route: ILL 13　　Structure Number: 039-0013 and 039-0049　　Date: 10/4/2004

Section　　Bored By: Bryan Keller

County: Jackson　　Location: E. Edge of Murphysboro　　Checked By: Rob Graeff

Boring No 5-S
Station 341+88
Offset 22' RT CL
Ground Surface 351.6 Ft

Surf Wat Elev: 338.5
Ground Water Elevation
when Drilling 312.1
At Completion
At: 24 Hrs: 346.4

Description	Elev.	DEPTH	BLOW	Qu tsf	W%
Stiff, very moist, brown, Silty Clay A-6			1		
			3	1.6S	23
			3		
	347.1				
Stiff, very moist, grey, Clay to Silty Clay A7-6		5.0	1		
			3	1.2B	22
			4		
	344.6				
Very soft, wet, grey, Silty Clay A-6			WH		
			1	0.2B	30
			1		
		10.0	WH		
			1	0.2B	33
			1		
	339.6				
Soft, wet, grey mottled brown, Silty Clay A-6			WH		
			WH	0.3B	29
			WH		
	337.1				
Soft, wet, grey, Clay to Silty Clay A7-6		15.0	WH		
			WH	0.4B	30
			WH		
	334.6				
Medium, very moist, grey, Clay to Silty Clay A7-6			WH		
			WH	0.6B	27
			WH		
	332.1				
Soft, very moist, grey, Silty Clay Loam A-6		20.0	WH		
			WH	0.4B	27
			WH		
	329.6				
Medium, very moist, grey, Silty Clay to Clay A7-6			WH		
			WH	0.6B	27
			WH		
		25.0	WH		

Description	Elev.	DEPTH	BLOW	Qu tsf	W%
Medium, very moist, grey, Silty Clay to Clay A7-6			WH	0.6B	29
			WH		
			WH		
			WH	0.8B	28
			1		
		30.0	WH		
			1	0.9B	25
			1		
	319.6				
Medium to stiff, very moist, grey, Clay to Silty Clay A7-6			WH		
			WH	1.0B	25
			3		
	317.1				
Stiff, very moist, grey, Clay A7-6		35.0	WH		
			1	1.1B	24
			2		
	314.6				
Medium, very moist, brown, Clay A7-6 with Silt Loam seams			1		
			2	0.9S	35
			2		
	312.1				
Stiff, very moist, brown, Clay A7-6 with fine Sand layers		40.0	WH		
			2	1.1S	27
			2		
	307.1				
Very stiff, very moist, grey, Clay Loam A-6 with Sand layers		45.0	3		
			6	2.6P	28
			5		
	302.1				
		50.0	WH		

N-Std Pentr Test: 2" OD Sampler,
140# Hammer, 30" Fall (Type Fail, B-Bulge S-Shear E-Estimated P-Penetrometer)

Route: ILL 13　　Date: 10/4/2004

Section:

County: Jackson

Boring No: 5-S
Station: 341+88
Offset: 22' RT CL
Ground Surface: 351.6 Ft

Description	DEPTH	BLOW	Qu tsf	W%	Description	DEPTH	BLOW	Qu tsf	W%
Very loose, wet, grey, very fine		2		21					
Silty Sand		1							
93% Sand									
5% Silt									
2% Clay									
297.1									
Stiff, wet, brown, Clay	55.0	WH				80.0			
A7-6 with Sand layers		4	1.1B	51					
		10							
292.1									
Medium, wet, grey, fine Sand	60.0	4		25		85.0			
with Gravel and Coal Chips 291.1		18							
Hard, dry, grey, weathered		49							
Clay Shale 289.6									
Hard, dry, grey, Clay Shale		100/5"							
over Coal 288.6									
Bottom of hole = 63.0 ft.									
Free water observed at 39.5 ft.	65.0					90.0			
Elevation referenced to BK of W Abut Sta 343+06 CL Elevation = 383.47 ft.									
To convert "N" values to "N60" values multiply by 1.25.									
	70.0					95.0			
	75.0					100.0			

N-Std Pentr Test: 2" OD Sampler,
140# Hammer 30" Fall (Type Fail. B-Bulge S-Shear E-Estimated P-Penetrometer)

ILLINOIS DEPARTMENT OF TRANSPORTATION
District Nine Materials

Bridge Foundation Boring Log

ILL 13 Over Big Muddy River

Route: ILL 13 | Structure Number: 039-0013 and 039-0049 | Date: 10/8/2004

Section: | Bored By: Bryan Keller

County: Jackson | Location: E. Edge of Murphysboro | Checked By: Rob Graeff

Boring No 6-S
Station 343+17
Offset 22' RT EB CL
Ground Surface 383.0 Ft

Surf Wat Elev: 338.5
Ground Water Elevation
when Drilling 333
At Completion
At: 68 Hrs: 357.5

Description	DEPTH	BLOW	Qu tsf	W%
Very stiff, damp, brown, Silty Clay A-6		1 4 8	2.5B	11
	5.0	2 4 3	2.1B	15
376.0				
Very stiff, moist to very moist, brown, Silty Clay A-6		1 3 3	2.1B	20
373.5				
Stiff, very moist, brown, Silty Clay A7-6	10.0	1 2 2	1.1B	24
371.0				
Stiff, moist to very moist, brown, Silty Clay to Silty Clay Loam A-6		1 3 4	1.3S	20
	15.0	1 2 4	1.3S	23
366.0				
Medium, very moist, brown, Silty Clay Loam A-6		WH 2 2	0.7S	22
363.5				
Medium, very moist, brown, Silty Clay to Silty Clay Loam A-6	20.0	WH 1 1	0.6S	25
361.0				
Medium, moist to very moist, grey, Clay Loam A-6		WH 3 3	0.9B	20
358.5				
	25.0	WH		

Description	DEPTH	BLOW	Qu tsf	W%
Very stiff, moist, brown, Silty Clay A-6		3 4	2.1B	21
356.0				
Soft, very moist, brown, Silty Clay loam A-6		WH 1 2	0.4B	24
353.5				
Stiff, moist to very moist, grey, Clay A7-6	30.0	WH 2 2	1.1B	21
351.0				
Medium, very moist, grey, Clay A7-6		WH 2 2	0.9B	29
	35.0	WH 1 2	0.8B	29
346.0				
Medium, very moist, brown mottled grey, Clay to Silty Clay A7-6		WH 2 3	0.9B	27
343.5				
Medium, very moist, grey, Clay A7-6	40.0	WH 1 3	0.7B	26
338.5				
Stiff, very moist, brown, Clay A7-6	45.0	WH 1 1	1.2S	31
333.5				
	50.0	WH		

N-Std Pentr Test: 2" OD Sampler,
140# Hammer, 30" Fall (Type Fail, B-Bulge S-Shear E-Estimated P-Penetrometer)

Route: ILL 13 Date: 10/8/2004

Section:

County: Jackson

Boring No: 6-S

Station: 343+17

Offset: 22' RT EB CL

Ground Surface: 383.0 Ft

	DEPTH	BLOW	Qu tsf	W%
Medium, very moist, brown, Clay A7-6		2 1	0.8S	28
328.5				
Stiff, very moist, brown, Clay A7-6	55.0	1 1 4	1.4S	30
323.5				
Soft to medium, very moist, grey, Clay to Silty Clay A7-6 with Sand Layers	60.0	WH 2 2	0.5B	24
318.5				
Stiff, very moist, brown, Clay A7-6 with Clay Loam and Sand Layers	65.0	4 7 4	1.3B	32
313.5				
Stiff, very moist, brown, Clay A7-6 with Sand Layers	70.0	WH 3 3	1.1S	35
308.5				
	75.0	1		

	DEPTH	BLOW	Qu tsf	W%
Medium, very moist, brown, Clay A7-6 with Sand Layers		4 2	0.9B	32
303.5				
Loose to medium, wet, grey, fine Sand with Coal Chips and some Gravel 57% Sand 6% Silt 4% Clay 33% Gravel	80.0	4 5 5		23
	85.0			
293.5				
Medium, wet, grey, Sandy Gravel 72% Sand; 15% Silt 6% Clay; 7% Gravel	90.0	4 10 15		17
291.0				
Hard, dry, black Coal		100/3"		
	95.0			
Hard, dry, black Coal		100/5"		
	100.0			

N-Std Pentr Test: 2" OD Sampler,
140# Hammer, 30" Fall (Type Fail. B-Bulge S-Shear E-Estimated P-Penetrometer)

Route: ILL 13

Date: 10/8/2004

Section:

County: Jackson

Boring No: 6-S

Station: 343+17

Offset: 22' RT EB CL

Ground Surface: 383.0 Ft

	DEPTH	BLOW	Qu tsf	W%		DEPTH	BLOW	Qu tsf	W%
Hard, dry, black Clay Shale 280.5		100/3"							
Bottom of hole = 102.5 ft.									
Free water observed at 50.0 ft.	105.0					130.0			
Elevation referenced to Bk W Abut Str # 039-0013 Sta 343+06 Elevation = 383.47									
To convert "N" values to "N60" values multiply by 1.25.									
	110.0					135.0			
	115.0					140.0			
	120.0					145.0			
	125.0					150.0			

N-Std Pentr Test: 2" OD Sampler,
140# Hammer, 30" Fall (Type Fail, B-Bulge S-Shear E-Estimated P-Penetrometer)

Illinois Department of Transportation

Division of Highways
SCI Engineering

SOIL BORING LOG

Date 07/15/09

ROUTE FAP 331 **DESCRIPTION** Structure Replacement crossing Big Muddy River **LOGGED BY** SCI

SECTION 12-2B-2 **LOCATION** East of Murphysboro; SW 1/4, **SEC.** 3, **TWP.** 9S, **RNG.** 2W

COUNTY Jackson **DRILLING METHOD** CME 750 w/HSA **HAMMER TYPE** Automatic

STRUCT. NO. 039-0013
Station 340+71.00

BORING NO. B-101
Station 339+28
Offset 75 ft Lt EB
Ground Surface Elev. 363.0 ft

Surface Water Elev. ____ ft
Stream Bed Elev. ____ ft

Groundwater Elev.:
First Encounter 345.0 ft ▼
Upon Completion - ft
After - **Hrs.** - ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
SILTY CLAY: Brown (A-7) Classification of materials in upper 20 feet based on observation of augered cuttings.					
		-5			
Temporary benchmark - brass disk at southeast corner of east abutment. USGS Topographic Map - El. 384					
		-10			
		-15			
▼					
	343.0	-20			
SILTY CLAY: Gray, some fine sand (A-6) ST pushed 20' to 22'. Recovery 22"; UU - 1.0 tsf; DD - 94.2 pcf; MC - 28% Becomes brownish gray, trace fine sand ST pushed 22' to 24'. Recovery 24"; UU - 1.2 tsf; DD - 92.9 pcf; MC - 29% Becomes gray, some fine sand	338.3	-25			
SAND: Gray, fine (A-3) ST pushed 24' to 26'. Recovery 23". UU - 1.1 tsf; DD - 90.5 pcf; MC - 31%	336.8 336.0				
CLAY: Gray, trace fine sand (A-7) ST pushed from 26' to 28'. Recovery 23"; UU - 1.1 tsf; DD - 87.7 pcf; MC- 33%					
SILTY CLAY: Gray, some fine sand (A-6) Becomes maroon and gray Triaxial shear test performed on ST pushed from 28' to 30'; LL-25, PL-18, PI-7	331.0	-30			
SAND: Maroon and gray, fine (A-3)	329.8				
SILTY CLAY: Maroon, some fine sand (A-6) ST pushed 32' to 34'. Recovery 23"; UU - 3.0 tsf; DD - 99.7 pcf; MC - 25%	329.0 327.0	-35			
SAND: Brown, fine (A-3)					
Boring terminated at 36.0 ft.					
		-40			

Illinois Department of Transportation

Division of Highways
SCI Engineering

SOIL BORING LOG

Date 07/14/09

ROUTE FAP 331 **DESCRIPTION** Structure Replacement crossing Big Muddy River **LOGGED BY** SCI

SECTION 12-2B-2 **LOCATION** East of Murphysboro; SW 1/4, **SEC.** 3, **TWP.** 9S, **RNG.** 2W

COUNTY Jackson **DRILLING METHOD** CME 750 w/HSA **HAMMER TYPE** Automatic

STRUCT. NO. 039-0049
Station 340+71.00

BORING NO. B-103
Station 343+09
Offset 14 ft Rt WB
Ground Surface Elev. 374.0 ft

Surface Water Elev. ___ ft
Stream Bed Elev. ___ ft

Groundwater Elev.:
First Encounter 329.0 ft ▼
Upon Completion - ft
After - **Hrs.** - ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
ASPHALT - 4 inches	373.7				
CONCRETE - 9 inches	372.9				
CRUSHED ROCK	372.5				
FILL: Brown, silty clay, trace to some sand (A-6) Classification of materials based on observation of augered cuttings.	369.0	-5			
FILL: Gray, silty clay (A-7) Becomes brown Temporary benchmark - brass disk at southeast corner of east abutment. USGS Topographic Map - El. 384		-10			
	361.0				
FILL: Grayish brown, silty clay (A-6)		-15			
	358.5				
FILL: Gray, silty clay, trace to some sand (A-7)		-20			

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
FILL: Gray, silty clay, trace to some sand (A-7) *(continued)*	352.0				
CLAY: Brown and gray (A-7) ST pushed 22' to 24'. Recovery 16". UU - 2.3 tsf; DD - 97.4 pcf; MC - 24%		-25			
	348.5				
SILTY CLAY: Brown and gray (A-7) Triaxial shear test performed on ST pushed from 27' to 29'; LL-47, PL-17, PI-30	344.0	-30			
CLAY: Gray, some sand (A-7)	340.0				
SILTY CLAY: Brown and gray (A-6) ST pushed 34' to 36'. Recovery 19". UU - 1.3 tsf; DD - 96.2 pcf; MC - 27%		-35			
	334.0	-40			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer)
AASHTO Classifications are based on visual classifications unless otherwise noted BBS, form 137 (Rev. 8-99)

Illinois Department of Transportation

Division of Highways
SCI Engineering

SOIL BORING LOG

Date 07/14/09

ROUTE FAP 331 **DESCRIPTION** Structure Replacement crossing Big Muddy River **LOGGED BY** SCI

SECTION 12-2B-2 **LOCATION** East of Murphysboro; SW 1/4, **SEC.** 3, **TWP.** 9S, **RNG.** 2W

COUNTY Jackson **DRILLING METHOD** CME 750 w/HSA **HAMMER TYPE** Automatic

STRUCT. NO. 039-0049
Station 340+71.00

BORING NO. B-103
Station 343+09
Offset 14 ft Rt WB
Ground Surface Elev. 374.0 ft

Surface Water Elev. ft
Stream Bed Elev. ft

Groundwater Elev.:
First Encounter 329.0 ft ▼
Upon Completion - ft
After - **Hrs.** - ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
SILTY CLAY: Brown, trace to some sand (A-7)					
	329.0 ▼	-45			
SAND: Brown and gray, fine to coarse (A-1)	328.5				
CLAY: Gray (A-7) ST pushed 45' to 47'. Recovery 23". UU - 3.1 tsf; DD - 95.6 pcf; MC - 27%	327.5				
SILTY CLAY: Gray (A-6)	327.0				
Boring terminated at 47.0 ft.					
		-50			
		-55			
		-60			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer)
AASHTO Classifications are based on visual classifications unless otherwise noted BBS, form 137 (Rev. 8-99)

Illinois Department of Transportation

Division of Highways
SCI Engineering

SOIL BORING LOG

Date 7/13,14/2009

ROUTE FAP 331 **DESCRIPTION** Structure Replacement crossing Big Muddy River **LOGGED BY** SCI

SECTION 12-2B-2 **LOCATION** East of Murphysboro; SW 1/4, **SEC.** 3, **TWP.** 9S, **RNG.** 2W

COUNTY Jackson **DRILLING METHOD** CME 750 w/HSA **HAMMER TYPE** Automatic

STRUCT. NO. 039-0013
Station 340+71.00

BORING NO. B-104
Station 343+61
Offset 14 ft Rt EB
Ground Surface Elev. 383.0 ft

Surface Water Elev. ____ ft
Stream Bed Elev. ____ ft

Groundwater Elev.:
First Encounter 343.0 ft ▼
Upon Completion - ft
After - **Hrs.** - ft

Description	Elev. (ft)	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
ASPHALT - 9 inches	382.3				
CRUSHED ROCK	382.0				
FILL: Brown, sandy clay, some gravel, cinders (A-4)	380.8		12 9 5	4.5 P	7
FILL: Brown, sandy clay (A-6) Becomes reddish brown		-5	3 3 4	2.0 B	21
FILL: Brown, silty clay, trace to some sand (A-7) Poor recovery	377.5		3 3 3	2.3 P	21
		-10	1 2 4	1.5 P	24
Temporary benchmark - brass disk at southeast corner of east abutment. USGS Topographic Map - El. 384 Poor recovery			3 4 5	2.0 P	22
Becomes gray Becomes brown and gray		-15	1 3 4	1.7 B	29
FILL: Gray, silty clay (A-6)	367.5		2 3 5	1.2 B	23
Becomes brown and gray, trace to some sand	363.0	-20	2 4 7	2.8 B	22

Description	Elev. (ft)	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
FILL: Brown, clay (A-7)					
FILL: Gray, clayey silt (A-4)	361.5		4 6 9	3.0 P	28
FILL: Gray, silty clay, trace to some sand (A-7)	360.0	-25	1 3 5	1.4 B	25
			3 4 8	2.3 P	22
SILTY CLAY: Gray (A-6)	355.0	-30	1 1 3	0.6 B	25
CLAY: Brown, trace sand (A-7)	351.0				
SILTY CLAY: Brown (A-7)	348.8	-35	2 4 4	1.2 B	24
CLAY: Gray and brown, trace shells (A-7)	346.0	▼ -40	3 5 6	3.1 S/10	44

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer)
AASHTO Classifications are based on visual classifications unless otherwise noted BBS, form 137 (Rev. 8-99)

Illinois Department of Transportation

Division of Highways
SCI Engineering

SOIL BORING LOG

Date 7/13,14/2009

ROUTE FAP 331 **DESCRIPTION** Structure Replacement crossing Big Muddy River **LOGGED BY** SCI

SECTION 12-2B-2 **LOCATION** East of Murphysboro; SW 1/4, **SEC.** 3, **TWP.** 9S, **RNG.** 2W

COUNTY Jackson **DRILLING METHOD** CME 750 w/HSA **HAMMER TYPE** Automatic

STRUCT. NO. 039-0013
Station 340+71.00

BORING NO. B-104
Station 343+61
Offset 14 ft Rt EB
Ground Surface Elev. 383.0 ft

Surface Water Elev. ft
Stream Bed Elev. ft

Groundwater Elev.:
First Encounter 343.0 ft ▼
Upon Completion - ft
After - **Hrs.** - ft

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
342.0				
SAND: Greenish gray, fine to medium (A-3)				
339.0		WH		
CLAY: Gray (A-7) Interbedded with of brown, clayey silt		3	1.0	65
	-45	5	B	
Becomes brown and interbedded with brown, silty clay		2		
		3	0.4	33
	-50	3	B	
Interbedded with of brown, clayey silt		3		
		5	1.2	32
	-55	4	B	
Interbedded with brown, clayey silt and gray, fine to medium sand		2		
		4	0.6	40
	-60	7	B	

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
CLAY: Gray (A-7) *(continued)*				
321.0				
SANDY CLAY: Brown (A-6)		3		
318.5		2	0.5	24
CLAY: Brown (A-7)	-65	3	B	
		3		
		3	1.1	29
	-70	3	B	
311.0				
SAND: Gray, fine, some clay and with clay and sandy clay deposits (A-2)		5		
		6	-	
	-75	9		
306.0				
CLAY: Brown (A-7)		13		
303.8		14	3.8	25
	-80	13	P	

Illinois Department of Transportation

Division of Highways
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SOIL BORING LOG

Date 7/13,14/2009

ROUTE FAP 331 **DESCRIPTION** Structure Replacement crossing Big Muddy River **LOGGED BY** SCI

SECTION 12-2B-2 **LOCATION** East of Murphysboro; SW 1/4, **SEC.** 3, **TWP.** 9S, **RNG.** 2W

COUNTY Jackson **DRILLING METHOD** CME 750 w/HSA **HAMMER TYPE** Automatic

STRUCT. NO. 039-0013
Station 340+71.00

BORING NO. B-104
Station 343+61
Offset 14 ft Rt EB
Ground Surface Elev. 383.0 ft

Surface Water Elev. ft
Stream Bed Elev. ft

Groundwater Elev.:
First Encounter 343.0 ft ▼
Upon Completion - ft
After - **Hrs.** - ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
SAND: Bluish gray, greenish gray, and gray, fine to coarse, trace weathered shale fragments and gravel (A-1) *(continued)*	301.0				
SAND: Greenish gray and gray, fine to medium, trace fine gravel (A-3)		-85	11 17 16	-	
Becomes brown	293.8	-90	13 12 12	-	25
CLAY: Brown, with trace limestone fragments (A-7)	291.5				
COAL			50/1.5" 50/1"	-	37
		-95	50/2" 50/0.5"	-	24
	285.0				
CLAYEY SHALE: Grayish brown	284.0		50/5" 50/1.5"	-	13
Boring terminated at 99.0 ft.		-100			

Appendix B

Job Name: **II-13 over Big Muddy**

Job No.: **2008-3043.50**

LIQUEFACTION POTENTIAL ANALYSIS

Prepared by Yong Wu

DATE 08/18/06 ***Performed By:*** SCI Engineering, Inc.

REFERENCE BORING NUMBER(S)== **B-104**
UNITS (METRIC OR ENGLISH)=== 1 (1=ENGLISH OR 2=METRIC)
ELEVATION OF TOP OF BORING== 383 FT.
DEPTH TO GROUND WATER DURING DRILLING=== 40 FT. (Below BORING Ground Surface)
DEPTH TO GROUND WATER DURING EARTH QUAKE====================================== 40 FT. (Below FINISHED Grade Cut or Fill Surface)
MAX. HORZ. GROUND SURFACE ACCELERATION COEFF.================================= 0.33 Coefficient of Gravity
DESIGN EARTH QUAKE MAGNITUDE== 6.93 Moment Magnitude
FINISHED GRADE CUT OR FILL FROM BORING SURFACE================================ 0 FT. (WHICH IS 0.000 KSF. EFFECT. SURCH. FILL PRESS.)
HAMMER TYPE============== == 1 (1=AUTOMATIC HAMMER OR 2=CATHEAD HAMMER)

k alpha > 1 for slope

Settlement Calculation

Sand- or Clay-like (PI<7?) "S" or "C"	Elevation of Sample (Feet)	Boring Sample Depth (Feet)	SPT N Value (Blows)	Sand % Fines <#200 (%)	Clay Pre-consl. Press. (ksf)	Eff. Unit Wt. (kcf)	N1(60)	Eff. Po' While Drilling (ksf)	% Fines Corr SPT N (Ncor)	Corr. Blow Count (N')	Confin. Press. Reduct. (K sig)	Clay OCR	Cyclic Resistance Ratio CRR	Unit Wt. During E.Q. (kcf)	Eff. Po' During E. Q. (ksf)	Total Po During E. Q. (ksf)	Stress Reduc. Factor rd	Earthqk Scaling Factor MSF	Induced Cyclic Stress Ratio CSR	Factor of Safety FOS	FOS	CSR	(N1)60	Layer Thickness (ft.)	Volumetric Strain (%)	Settlement (in.)
C	380.5	2.5	14			0.128	23	0.321	0.0	23	1.00	1.05	0.188	0.122	0.305	0.305	1.000	1.026	0.209	Abv Water	Abv Water	0.21	23			0.00
C	378.0	5	7			0.120	10	0.622	0.0	10	1.00	1.05	0.188	0.114	0.591	0.591	0.992	1.026	0.207	Abv Water	Abv Water	0.21	10			0.00
C	375.5	7.5	6			0.119	8	0.918	0.0	8	1.00	1.05	0.188	0.113	0.872	0.872	0.983	1.026	0.205	Abv Water	Abv Water	0.21	8			0.00
C	373.0	10	6			0.119	7	1.215	0.0	7	1.00	1.05	0.188	0.113	1.154	1.154	0.973	1.026	0.203	Abv Water	Abv Water	0.20	7			0.00
C	370.5	12.5	9			0.123	10	1.523	0.0	10	1.00	1.05	0.188	0.117	1.447	1.447	0.962	1.026	0.201	Abv Water	Abv Water	0.20	10			0.00
C	368.0	15	7			0.120	7	1.824	0.0	7	1.00	1.05	0.188	0.114	1.732	1.732	0.951	1.026	0.199	Abv Water	Abv Water	0.20	7			0.00
C	365.5	17.5	8			0.122	8	2.128	0.0	8	1.00	1.05	0.188	0.116	2.022	2.022	0.939	1.026	0.196	Abv Water	Abv Water	0.20	8			0.00
C	363.0	20	11			0.126	10	2.442	0.0	10	1.00	1.05	0.188	0.119	2.320	2.320	0.927	1.026	0.194	Abv Water	Abv Water	0.19	10			0.00
C	360.5	22.5	15			0.129	13	2.765	0.0	13	1.00	1.05	0.188	0.123	2.627	2.627	0.915	1.026	0.191	Abv Water	Abv Water	0.19	13			0.00
C	358.0	25	8			0.122	7	3.070	0.0	7	1.00	1.05	0.188	0.116	2.917	2.917	0.901	1.026	0.188	Abv Water	Abv Water	0.19	7			0.00
C	355.5	27.5	12			0.127	10	3.387	0.0	10	1.00	1.05	0.188	0.120	3.217	3.217	0.888	1.026	0.186	Abv Water	Abv Water	0.19	10			0.00
C	353.0	30	4			0.114	3	3.672	0.0	3	1.00	1.05	0.188	0.108	3.488	3.488	0.874	1.026	0.183	Abv Water	Abv Water	0.18	3			0.00
C	348.0	35	8			0.122	6	4.281	0.0	6	1.00	1.05	0.188	0.116	4.067	4.067	0.846	1.026	0.177	Abv Water	Abv Water	0.18	6			0.00
S	342.0	41	11	25		0.062	8	4.652	5.1	13	0.93	1.00	0.130	0.062	4.438	4.500	0.812	1.162	0.152	0.85	0.85	0.15	13	6	2.00	1.44
C	339.0	44	8			0.059	6	4.829	0.0	6	1.00	1.05	0.187	0.059	4.615	4.864	0.795	1.026	0.175	1.07	1.07	0.18	6			0.00
C	333.0	50	6			0.057	4	5.169	0.0	4	1.00	1.04	0.186	0.057	4.955	5.579	0.761	1.026	0.179	1.04	1.04	0.18	4			0.00
C	328.0	55	9			0.060	6	5.469	0.0	6	1.00	1.04	0.186	0.060	5.255	6.191	0.733	1.026	0.181	1.03	1.03	0.18	6			0.00
C	323.0	60	11			0.062	7	5.778	0.0	7	1.00	1.04	0.186	0.062	5.564	6.812	0.707	1.026	0.181	1.03	1.03	0.18	7			0.00
C	318.0	65	5			0.055	3	6.054	0.0	3	1.00	1.04	0.185	0.055	5.840	7.400	0.682	1.026	0.181	1.03	1.03	0.18	3			0.00
S	311.0	72	6	25		0.057	4	6.450	5.1	9	0.92	1.00	0.100	0.057	6.236	8.233	0.650	1.162	0.158	0.63	0.63	0.16	9	7	2.50	2.10
C	306.0	77	15			0.065	9	6.773	0.0	9	1.00	1.03	0.185	0.065	6.559	8.867	0.629	1.026	0.178	1.04	1.04	0.18	9			0.00
S	304.0	79	27	20		0.070	16	6.912	4.5	20	0.86	1.00	0.180	0.070	6.698	9.132	0.622	1.162	0.156	1.15	1.15	0.16	20			0.00
S	298.0	85	33	20		0.072	19	7.343	4.5	23	0.83	1.00	0.211	0.072	7.129	9.937	0.601	1.162	0.154	1.36	1.36	0.15	23			0.00
C	294.0	89	24			0.069	13	7.618	0.0	13	1.00	1.03	0.184	0.069	7.404	10.461	0.588	1.026	0.174	1.06	1.06	0.17	13			0.00
Rock	292.0	91	100			0.083	55	7.783	0.0	55	1.00	1.03	0.184	0.083	7.569	10.751	0.582	1.026	0.173	1.06	1.06	0.17	55			0.00
rock	283.0	100	100			0.083	52	8.526	0.0	52	1.00	1.03	0.184	0.083	8.312	12.056	0.562	1.026	0.170	1.08	1.08	0.17	52			0.00
						0.000						1.00			8.312			1.162								0.00
																										3.54

1-S

LIQUEFACTION POTENTIAL ANALYSIS

BUREAU OF BRIDGES AND STRUCTURES FOUNDATIONS UNIT wmk 8/25/01

REFERENCE BORING NUMBER==================================	[redacted]	
STRUCTURE NUMBER=======================================	[redacted]	
ELEVATION OF BORING GROUND SURFACE=======================	[redacted]	Feet
DEPTH TO GROUNDWATER DURING DRILLING=====================	[redacted]	Feet (Below Boring Ground Surface)
DEPTH TO GROUNDWATER DURING EARTHQUAKE===================	[redacted]	Feet (Below Finished Grade Cut or Fill Surface)
MAX. HORZ. GROUND SURFACE ACCELERATION===================	[redacted]	Coefficient of Gravity
DESIGN EARTHQUAKE MEAN MAGNITUDE========================	[redacted]	Moment Magnetude Scale
FINISHED GRADE FILL OR CUT FROM BORING SURFACE============	[redacted]	Ft. Which is 0 ksf Effect.Surch.Fill Press.
ADJUST DIST. #9 N VALUES TO 60% ENERGY TRANSFER===========	[redacted]	(1=Yes OR 2=No)

Sloped Ground Shear Stress Correct. Factor	
(Kα)	1.00
Earthquake Magnitude Scaling Factor	
(MSF)=	1.270

	Boring Data			Conditions During Drilling					Conditions During Earthquake							
Elev. of Sample (Feet)	Boring Sample Depth (Feet)	S.P.T. N Value (Blows)	% Fines < #200 (%)	Effect. Unit Weight (KCF.)	Effect. Vertical Stress (KSF.)	Overburd. & Drillrod Corrected (Ni)60	Fines Content Corrected (Ni)60cs	CSR Resisting Mag 7.5 CRR(7.5)	Effect. Unit Weight (KCF.)	Effect. Vertical Stress (KSF.)	Total Vertical Stress (KSF.)	Confining, Sloping & Mag. Correct. (Kσ)(Kα)(MSF)	Corrected CRR(7.5) Resisting CRR	Stress Reduct. Factor (rd)	Earth Quake Induced CSR	FACTOR OF SAFETY CRR/CSR
380.2	[redacted]	[redacted]	[redacted]	0.128	0.385	36.893	49.272	1.000	0.128	0.385	0.385	1.417	1.417	0.993	0.126	ABO. WAT.
377.7	[redacted]	[redacted]	[redacted]	0.123	0.699	20.281	29.338	0.391	0.123	0.691	0.691	1.400	0.547	0.986	0.125	ABO. WAT.
375.2	[redacted]	[redacted]	[redacted]	0.123	1.006	17.376	25.851	0.297	0.123	0.983	0.983	1.390	0.413	0.980	0.124	ABO. WAT.
372.7	[redacted]	[redacted]	[redacted]	0.121	1.311	13.030	20.636	0.224	0.121	1.279	1.279	1.386	0.310	0.974	0.123	ABO. WAT.
370.2	[redacted]	[redacted]	[redacted]	0.122	1.614	13.258	20.909	0.227	0.122	1.577	1.577	1.388	0.315	0.968	0.123	ABO. WAT.
367.7	[redacted]	[redacted]	[redacted]	0.120	1.916	10.212	17.254	0.186	0.120	1.871	1.871	1.397	0.260	0.961	0.122	ABO. WAT.
365.2	[redacted]	[redacted]	[redacted]	0.115	2.209	6.499	12.799	0.139	0.115	2.111	2.111	1.408	0.195	0.955	0.121	ABO. WAT.
362.7	[redacted]	[redacted]	[redacted]	0.120	2.502	9.383	16.260	0.176	0.120	2.405	2.405	1.428	0.251	0.949	0.120	ABO. WAT.
360.2	[redacted]	[redacted]	[redacted]	0.115	2.796	6.060	12.272	0.133	0.115	2.698	2.698	1.453	0.193	0.943	0.119	ABO. WAT.
357.7	[redacted]	[redacted]	[redacted]	0.118	3.087	7.868	14.441	0.156	0.118	2.974	2.974	1.483	0.232	0.936	0.119	ABO. WAT.
355.2	[redacted]	[redacted]	[redacted]	0.113	3.376	4.809	10.771	0.117	0.057	2.453	2.453	1.431	0.167	0.930	0.118	NL
352.7	[redacted]	[redacted]	[redacted]	0.113	3.659	4.723	10.667	0.116	0.057	1.738	1.894	1.392	0.161	0.924	0.128	NL
350.2	[redacted]	[redacted]	[redacted]	0.117	3.946	6.505	12.806	0.139	0.060	1.927	2.239	1.399	0.194	0.903	0.133	NL
347.7	[redacted]	[redacted]	[redacted]	0.115	4.236	5.496	11.596	0.126	0.059	2.100	2.568	1.408	0.177	0.883	0.137	NL
345.2	[redacted]	[redacted]	[redacted]	0.113	4.521	4.526	10.431	0.113	0.057	2.194	2.818	1.413	0.160	0.862	0.140	NL
342.7	[redacted]	[redacted]	[redacted]	0.050	4.724	1.807	7.169	0.081	0.050	2.157	2.937	1.411	0.115	0.842	0.145	NL
340.2	[redacted]	[redacted]	[redacted]	0.050	4.848	1.820	7.184	0.081	0.050	2.131	3.067	1.409	0.115	0.821	0.150	NL
337.7	[redacted]	[redacted]	[redacted]	0.044	4.966	0.917	6.100	0.073	0.044	2.134	3.226	1.409	0.103	0.801	0.153	NL
335.2	[redacted]	[redacted]	[redacted]	0.044	5.076	0.924	6.109	0.073	0.044	2.124	3.372	1.409	0.103	0.780	0.157	NL
332.7	[redacted]	[redacted]	[redacted]	0.053	5.197	2.792	8.350	0.092	0.053	2.450	3.854	1.431	0.132	0.760	0.152	NL
330.2	[redacted]	[redacted]	[redacted]	0.053	5.329	2.808	8.370	0.092	0.053	2.797	4.357	1.463	0.135	0.739	0.146	NL
327.7	[redacted]	[redacted]	[redacted]	0.065	5.476	11.283	18.539	0.200	0.065	3.255	4.971	1.519	0.304	0.719	0.139	NL
325.2	[redacted]	[redacted]	[redacted]	0.065	5.637	11.319	18.583	0.200	0.065	3.742	5.614	1.594	0.320	0.698	0.133	NL
322.7	[redacted]	[redacted]	[redacted]	0.061	5.794	7.574	14.089	0.152	0.061	3.796	5.824	1.604	0.245	0.678	0.132	NL
320.2	[redacted]	[redacted]	[redacted]	0.061	5.947	7.605	14.127	0.153	0.061	3.841	6.025	1.612	0.246	0.657	0.131	NL
317.7	[redacted]	[redacted]	[redacted]	0.061	6.099	7.637	14.165	0.153	0.061	3.993	6.333	1.640	0.251	0.637	0.128	NL
315.2	[redacted]	[redacted]	[redacted]	0.061	6.251	7.670	14.203	0.154	0.061	4.146	6.642	1.670	0.257	0.616	0.125	NL
312.7	[redacted]	[redacted]	[redacted]	0.065	6.409	12.511	20.013	0.216	0.065	4.448	7.100	1.734	0.375	0.596	0.121	NL
310.2	[redacted]	[redacted]	[redacted]	0.065	6.572	12.554	20.065	0.217	0.065	4.762	7.570	1.807	0.392	0.575	0.116	NL
307.7	[redacted]	[redacted]	[redacted]	0.055	6.723	3.880	9.656	0.105	0.055	4.543	7.507	1.755	0.184	0.555	0.116	NL
305.2	[redacted]	[redacted]	[redacted]	0.055	6.860	3.901	9.681	0.105	0.055	4.299	7.419	1.701	0.179	0.534	0.117	NL
297.7	[redacted]	[redacted]	[redacted]	0.061	7.296	7.915	14.498	0.157	0.061	4.963	8.551	1.858	0.291	0.473	0.103	NL
292.7	[redacted]	[redacted]	[redacted]	0.073	7.630	28.885	29.251	0.386	0.073	6.044	9.944	2.179	0.841	0.432	0.090	9.341
290.2	[redacted]	[redacted]	[redacted]	0.073	7.811	28.955	29.322	0.390	0.073	6.752	10.808	2.435	0.949	0.411	0.083	11.369

2-S

LIQUEFACTION POTENTIAL ANALYSIS

BUREAU OF BRIDGES AND STRUCTURES FOUNDATIONS UNIT wmk 8/25/01

REFERENCE BORING NUMBER================================	[redacted]	
STRUCTURE NUMBER=======================================	[redacted]	
ELEVATION OF BORING GROUND SURFACE=====================	[redacted]	Feet
DEPTH TO GROUNDWATER DURING DRILLING===================	[redacted]	Feet (Below Boring Ground Surface)
DEPTH TO GROUNDWATER DURING EARTHQUAKE=================	[redacted]	Feet (Below Finished Grade Cut or Fill Surface)
MAX. HORZ. GROUND SURFACE ACCELERATION=================	[redacted]	Coefficient of Gravity
DESIGN EARTHQUAKE MEAN MAGNITUDE=======================	[redacted]	Moment Magnetude Scale
FINISHED GRADE FILL OR CUT FROM BORING SURFACE=========	[redacted]	Ft. Which is 0 ksf Effect.Surch.Fill Press.
ADJUST DIST. #9 N VALUES TO 60% ENERGY TRANSFER========	[redacted]	(1=Yes OR 2=No)

Sloped Ground Shear Stress Correct. Factor (Kα)	1.00
Earthquake Magnitude Scaling Factor (MSF)=	1.270

	Boring Data			Conditions During Drilling					Conditions During Earthquake							
Elev. of Sample (Feet)	Boring Sample Depth (Feet)	S.P.T. N Value (Blows)	% Fines < #200 (%)	Effect. Unit Weight (KCF.)	Effect. Vertical Stress (KSF.)	Overburd. & Drillrod Corrected (Ni)60	Fines Content Corrected (Ni)60cs	CSR Resisting Mag 7.5 CRR(7.5)	Effect. Unit Weight (KCF.)	Effect. Vertical Stress (KSF.)	Total Vertical Stress (KSF.)	Confining, Sloping & Mag. Correct. (Kσ)(Kα)(MSF)	Corrected CRR(7.5) Resisting CRR	Stress Reduct. Factor (rd)	Earth Quake Induced CSR	FACTOR OF SAFETY CRR/CSR
351.4	[redacted]	[redacted]	[redacted]	0.124	0.371	25.243	35.291	1.000	0.124	0.371	0.371	1.418	1.418	0.993	0.126	ABO. WAT.
348.9	[redacted]	[redacted]	[redacted]	0.120	0.676	15.473	23.567	0.261	0.062	0.511	0.542	1.409	0.368	0.986	0.133	NL
346.4	[redacted]	[redacted]	[redacted]	0.111	0.963	5.919	12.103	0.131	0.055	0.468	0.656	1.412	0.185	0.980	0.174	NL
343.9	[redacted]	[redacted]	[redacted]	0.104	1.231	2.689	8.227	0.091	0.050	0.550	0.893	1.407	0.128	0.974	0.201	NL
341.4	[redacted]	[redacted]	[redacted]	0.097	1.482	1.258	6.509	0.076	0.044	0.610	1.109	1.404	0.107	0.968	0.223	NL
338.9	[redacted]	[redacted]	[redacted]	0.097	1.725	1.196	6.435	0.075	0.044	0.686	1.341	1.400	0.105	0.961	0.238	NL
336.4	[redacted]	[redacted]	[redacted]	0.097	1.967	1.148	6.377	0.075	0.044	0.797	1.608	1.396	0.104	0.955	0.244	NL
333.9	[redacted]	[redacted]	[redacted]	0.097	2.210	1.109	6.331	0.074	0.044	0.907	1.874	1.392	0.104	0.949	0.248	NL
331.4	[redacted]	[redacted]	[redacted]	0.097	2.453	1.078	6.294	0.074	0.044	1.018	2.141	1.389	0.103	0.943	0.251	NL
328.9	[redacted]	[redacted]	[redacted]	0.097	2.695	1.053	6.263	0.074	0.044	1.128	2.408	1.387	0.103	0.936	0.253	NL
326.4	[redacted]	[redacted]	[redacted]	0.097	2.938	1.031	6.237	0.074	0.044	1.239	2.674	1.386	0.102	0.930	0.254	NL
323.9	[redacted]	[redacted]	[redacted]	0.111	3.197	4.042	9.850	0.107	0.055	1.515	3.107	1.387	0.148	0.924	0.240	NL
321.4	[redacted]	[redacted]	[redacted]	0.050	3.398	2.003	7.404	0.083	0.050	1.727	3.474	1.392	0.116	0.903	0.230	NL
318.9	[redacted]	[redacted]	[redacted]	0.061	3.536	8.021	14.626	0.158	0.061	1.962	3.865	1.401	0.222	0.883	0.220	NL
316.4	[redacted]	[redacted]	[redacted]	0.055	3.681	4.013	9.815	0.107	0.055	2.206	4.265	1.414	0.151	0.862	0.211	NL
313.9	[redacted]	[redacted]	[redacted]	0.076	3.845	41.069	49.247	1.000	0.076	2.653	4.869	1.449	1.449	0.842	0.196	7.401
311.4	[redacted]	[redacted]	[redacted]	0.076	4.035	40.895	49.055	1.000	0.076	3.264	5.636	1.520	1.520	0.821	0.180	8.458
308.9	[redacted]	[redacted]	[redacted]	0.053	4.195	2.992	8.590	0.094	0.053	2.928	5.455	1.478	0.139	0.801	0.189	NL
306.4	[redacted]	[redacted]	[redacted]	0.053	4.327	3.003	8.603	0.094	0.053	2.533	5.217	1.438	0.136	0.780	0.204	NL
303.9	[redacted]	[redacted]	[redacted]	0.053	4.459	3.014	8.617	0.095	0.053	2.665	5.505	1.450	0.137	0.760	0.199	NL
301.4	[redacted]	[redacted]	[redacted]	0.053	4.591	3.025	8.631	0.095	0.053	2.797	5.792	1.463	0.139	0.739	0.194	NL
298.9	[redacted]	[redacted]	[redacted]	0.044	4.713	1.014	1.325	0.005	0.044	2.693	5.844	1.453	0.007	0.719	0.198	0.037
296.4	[redacted]	[redacted]	[redacted]	0.044	4.823	1.020	1.331	0.005	0.044	2.567	5.874	1.441	0.007	0.698	0.203	0.036
293.9	[redacted]	[redacted]	[redacted]	0.065	4.959	12.280	14.222	0.154	0.065	3.290	6.753	1.524	0.234	0.678	0.176	1.330
291.4	[redacted]	[redacted]	[redacted]	0.065	5.120	12.294	14.236	0.154	0.065	4.064	7.684	1.654	0.255	0.657	0.158	1.617

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LIQUEFACTION POTENTIAL ANALYSIS

BUREAU OF BRIDGES AND STRUCTURES FOUNDATIONS UNIT wmk 8/25/01

REFERENCE BORING NUMBER================================	[redacted]	
STRUCTURE NUMBER=======================================	[redacted]	
ELEVATION OF BORING GROUND SURFACE=====================	[redacted]	Feet
DEPTH TO GROUNDWATER DURING DRILLING===================	[redacted]	Feet (Below Boring Ground Surface)
DEPTH TO GROUNDWATER DURING EARTHQUAKE=================	[redacted]	Feet (Below Finished Grade Cut or Fill Surface)
MAX. HORZ. GROUND SURFACE ACCELERATION=================	[redacted]	Coefficient of Gravity
DESIGN EARTHQUAKE MEAN MAGNITUDE=======================	[redacted]	Moment Magnetude Scale
FINISHED GRADE FILL OR CUT FROM BORING SURFACE=========	[redacted]	Ft. Which is 0 ksf Effect.Surch.Fill Press.
ADJUST DIST. #9 N VALUES TO 60% ENERGY TRANSFER========	[redacted]	(1=Yes OR 2=No)

Sloped Ground Shear Stress Correct. Factor (Kα)	1.00
Earthquake Magnitude Scaling Factor (MSF)=	1.270

	Boring Data			Conditions During Drilling					Conditions During Earthquake							
Elev. of Sample (Feet)	Boring Sample Depth (Feet)	S.P.T. N Value (Blows)	% Fines < #200 (%)	Effect. Unit Weight (KCF.)	Effect. Vertical Stress (KSF.)	Overburd. & Drillrod Corrected (Ni)60	Fines Content Corrected (Ni)60cs	CSR Resisting Mag 7.5 CRR(7.5)	Effect. Unit Weight (KCF.)	Effect. Vertical Stress (KSF.)	Total Vertical Stress (KSF.)	Confining, Sloping & Mag. Correct. (Kσ)(Kα)(MSF)	Corrected CRR(7.5) Resisting CRR	Stress Reduct. Factor (rd)	Earth Quake Induced CSR	FACTOR OF SAFETY CRR/CSR
350.7	[redacted]	[redacted]	[redacted]	0.131	0.392	44.660	58.592	1.000	0.131	0.392	0.392	1.417	1.417	0.993	0.126	ABO. WAT.
348.2	[redacted]	[redacted]	[redacted]	0.115	0.699	10.139	17.167	0.185	0.059	0.520	0.658	1.409	0.261	0.986	0.158	NL
345.7	[redacted]	[redacted]	[redacted]	0.104	0.973	2.946	8.535	0.094	0.050	0.432	0.726	1.414	0.133	0.980	0.208	NL
343.2	[redacted]	[redacted]	[redacted]	0.097	1.223	1.349	6.618	0.077	0.044	0.492	0.942	1.410	0.108	0.974	0.236	NL
340.7	[redacted]	[redacted]	[redacted]	0.097	1.466	1.265	6.518	0.076	0.044	0.575	1.181	1.406	0.107	0.968	0.252	NL
338.2	[redacted]	[redacted]	[redacted]	0.097	1.709	1.202	6.442	0.075	0.044	0.686	1.447	1.400	0.105	0.961	0.257	NL
335.7	[redacted]	[redacted]	[redacted]	0.097	1.951	1.153	6.383	0.075	0.044	0.797	1.714	1.396	0.105	0.955	0.260	NL
333.2	[redacted]	[redacted]	[redacted]	0.097	2.194	1.113	6.336	0.075	0.044	0.907	1.980	1.392	0.104	0.949	0.263	NL
330.7	[redacted]	[redacted]	[redacted]	0.097	2.436	1.082	6.298	0.074	0.044	1.018	2.247	1.389	0.103	0.943	0.264	NL
328.2	[redacted]	[redacted]	[redacted]	0.097	2.679	1.056	6.267	0.074	0.044	1.128	2.514	1.387	0.103	0.936	0.264	NL
325.7	[redacted]	[redacted]	[redacted]	0.097	2.922	1.034	6.241	0.074	0.044	1.239	2.780	1.386	0.102	0.930	0.265	NL
323.2	[redacted]	[redacted]	[redacted]	0.057	3.114	5.119	11.143	0.121	0.057	1.544	3.241	1.388	0.167	0.924	0.246	NL
320.7	[redacted]	[redacted]	[redacted]	0.044	3.241	1.025	1.917	0.005	0.044	1.670	3.523	1.390	0.007	0.903	0.242	0.029
318.2	[redacted]	[redacted]	[redacted]	0.068	3.381	17.433	20.376	0.221	0.068	1.986	3.996	1.402	0.309	0.883	0.225	1.373
315.7	[redacted]	[redacted]	[redacted]	0.061	3.541	8.182	10.734	0.116	0.061	2.444	4.609	1.431	0.166	0.862	0.206	0.807
313.2	[redacted]	[redacted]	[redacted]	0.053	3.683	3.070	8.684	0.095	0.053	2.303	4.625	1.420	0.135	0.842	0.214	NL
310.7	[redacted]	[redacted]	[redacted]	0.053	3.815	3.077	8.692	0.095	0.053	2.269	4.747	1.418	0.135	0.821	0.218	NL
308.2	[redacted]	[redacted]	[redacted]	0.055	3.950	4.111	9.934	0.108	0.055	2.455	5.088	1.432	0.154	0.801	0.210	NL
305.7	[redacted]	[redacted]	[redacted]	0.055	4.088	4.119	9.943	0.108	0.055	2.646	5.435	1.448	0.156	0.780	0.203	NL
303.2	[redacted]	[redacted]	[redacted]	0.060	4.232	7.219	13.663	0.148	0.060	2.902	5.847	1.475	0.218	0.760	0.194	NL
300.7	[redacted]	[redacted]	[redacted]	0.060	4.381	7.226	13.672	0.148	0.060	3.170	6.272	1.508	0.223	0.739	0.185	NL
298.2	[redacted]	[redacted]	[redacted]	0.053	4.522	3.104	8.725	0.096	0.053	3.125	6.382	1.502	0.144	0.719	0.186	NL
295.7	[redacted]	[redacted]	[redacted]	0.053	4.654	3.114	8.737	0.096	0.053	3.061	6.474	1.494	0.143	0.698	0.187	NL
293.2	[redacted]	[redacted]	[redacted]	0.063	4.799	10.403	17.484	0.189	0.063	3.499	7.069	1.555	0.293	0.678	0.174	NL
290.7	[redacted]	[redacted]	[redacted]	0.063	4.956	10.414	17.497	0.189	0.063	3.963	7.688	1.634	0.308	0.657	0.162	NL

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LIQUEFACTION POTENTIAL ANALYSIS

BUREAU OF BRIDGES AND STRUCTURES FOUNDATIONS UNIT wmk 8/25/01

REFERENCE BORING NUMBER================================	[redacted]	
STRUCTURE NUMBER=======================================	[redacted]	
ELEVATION OF BORING GROUND SURFACE=====================	[redacted]	Feet
DEPTH TO GROUNDWATER DURING DRILLING===================	[redacted]	Feet (Below Boring Ground Surface)
DEPTH TO GROUNDWATER DURING EARTHQUAKE=================	[redacted]	Feet (Below Finished Grade Cut or Fill Surface)
MAX. HORZ. GROUND SURFACE ACCELERATION=================	[redacted]	Coefficient of Gravity
DESIGN EARTHQUAKE MEAN MAGNITUDE=======================	[redacted]	Moment Magnetude Scale
FINISHED GRADE FILL OR CUT FROM BORING SURFACE=========	[redacted]	Ft. Which is 0 ksf Effect.Surch.Fill Press.
ADJUST DIST. #9 N VALUES TO 60% ENERGY TRANSFER========	[redacted]	(1=Yes OR 2=No)

Sloped Ground Shear Stress Correct. Factor (Kα)	1.00
Earthquake Magnitude Scaling Factor (MSF)=	1.270

	Boring Data			Conditions During Drilling					Conditions During Earthquake							
Elev. of Sample (Feet)	Boring Sample Depth (Feet)	S.P.T. N Value (Blows)	% Fines < #200 (%)	Effect. Unit Weight (KCF.)	Effect. Vertical Stress (KSF.)	Overburd. & Drillrod Corrected (Ni)60	Fines Content Corrected (Ni)60cs	CSR Resisting Mag 7.5 CRR(7.5)	Effect. Unit Weight (KCF.)	Effect. Vertical Stress (KSF.)	Total Vertical Stress (KSF.)	Confining, Sloping & Mag. Correct. (Kσ)(Kα)(MSF)	Corrected CRR(7.5) Resisting CRR	Stress Reduct. Factor (rd)	Earth Quake Induced CSR	FACTOR OF SAFETY CRR/CSR
348.7	[redacted]	[redacted]	[redacted]	0.113	0.339	9.709	16.651	0.180	0.113	0.339	0.339	1.420	0.255	0.993	0.126	ABO. WAT.
346.2	[redacted]	[redacted]	[redacted]	0.108	0.615	5.405	11.486	0.124	0.108	0.607	0.607	1.404	0.175	0.986	0.125	ABO. WAT.
343.7	[redacted]	[redacted]	[redacted]	0.097	0.871	1.556	6.868	0.079	0.044	0.608	0.670	1.404	0.111	0.980	0.137	NL
341.2	[redacted]	[redacted]	[redacted]	0.044	1.048	1.457	6.749	0.078	0.044	0.465	0.683	1.412	0.110	0.974	0.181	NL
338.7	[redacted]	[redacted]	[redacted]	0.044	1.158	1.423	6.707	0.077	0.044	0.575	0.950	1.406	0.109	0.968	0.202	NL
336.2	[redacted]	[redacted]	[redacted]	0.044	1.269	1.394	6.673	0.077	0.044	0.686	1.216	1.400	0.108	0.961	0.216	NL
333.7	[redacted]	[redacted]	[redacted]	0.044	1.380	1.371	6.645	0.077	0.044	0.797	1.483	1.396	0.107	0.955	0.225	NL
331.2	[redacted]	[redacted]	[redacted]	0.044	1.490	1.351	6.621	0.077	0.044	0.907	1.750	1.392	0.107	0.949	0.232	NL
328.7	[redacted]	[redacted]	[redacted]	0.044	1.601	1.335	6.602	0.077	0.044	1.018	2.016	1.389	0.106	0.943	0.237	NL
326.2	[redacted]	[redacted]	[redacted]	0.044	1.712	1.321	6.585	0.076	0.044	1.128	2.283	1.387	0.106	0.936	0.240	NL
323.7	[redacted]	[redacted]	[redacted]	0.044	1.822	1.309	6.571	0.076	0.044	1.239	2.549	1.386	0.106	0.930	0.243	NL
321.2	[redacted]	[redacted]	[redacted]	0.044	1.933	1.300	6.559	0.076	0.044	1.350	2.816	1.386	0.106	0.924	0.244	NL
318.7	[redacted]	[redacted]	[redacted]	0.055	2.057	5.149	11.178	0.121	0.055	1.640	3.262	1.390	0.168	0.903	0.228	NL
316.2	[redacted]	[redacted]	[redacted]	0.044	2.181	1.277	6.532	0.076	0.044	1.764	3.542	1.393	0.106	0.883	0.225	NL
313.7	[redacted]	[redacted]	[redacted]	0.044	2.292	1.271	6.526	0.076	0.044	1.682	3.616	1.391	0.106	0.862	0.235	NL
311.2	[redacted]	[redacted]	[redacted]	0.050	2.409	2.531	8.037	0.089	0.050	1.900	3.990	1.398	0.124	0.842	0.224	NL
308.7	[redacted]	[redacted]	[redacted]	0.050	2.533	2.518	8.021	0.089	0.050	2.131	4.377	1.409	0.125	0.821	0.214	NL
306.2	[redacted]	[redacted]	[redacted]	0.050	2.657	2.507	8.008	0.089	0.050	2.255	4.657	1.417	0.126	0.801	0.210	NL
303.7	[redacted]	[redacted]	[redacted]	0.050	2.781	2.497	7.997	0.089	0.050	2.379	4.937	1.426	0.126	0.780	0.205	NL
301.2	[redacted]	[redacted]	[redacted]	0.061	2.919	9.934	16.921	0.183	0.061	2.791	5.505	1.463	0.267	0.760	0.190	NL
298.7	[redacted]	[redacted]	[redacted]	0.061	3.071	9.864	16.837	0.182	0.061	3.231	6.102	1.516	0.275	0.739	0.177	NL
296.2	[redacted]	[redacted]	[redacted]	0.063	3.226	12.250	13.783	0.149	0.063	3.437	6.464	1.545	0.231	0.719	0.171	1.345
293.7	[redacted]	[redacted]	[redacted]	0.063	3.383	12.176	13.707	0.148	0.063	3.649	6.831	1.579	0.234	0.698	0.166	1.413
291.2	[redacted]	[redacted]	[redacted]	0.067	3.546	19.364	23.176	0.256	0.067	3.933	7.271	1.628	0.416	0.678	0.159	2.619
288.7	[redacted]	[redacted]	[redacted]	0.067	3.714	19.248	23.055	0.254	0.067	4.227	7.722	1.686	0.428	0.657	0.152	2.813

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LIQUEFACTION POTENTIAL ANALYSIS

BUREAU OF BRIDGES AND STRUCTURES FOUNDATIONS UNIT wmk 8/25/01

REFERENCE BORING NUMBER================================	[redacted]	
STRUCTURE NUMBER=======================================	[redacted]	
ELEVATION OF BORING GROUND SURFACE=====================	[redacted]	Feet
DEPTH TO GROUNDWATER DURING DRILLING===================	[redacted]	Feet (Below Boring Ground Surface)
DEPTH TO GROUNDWATER DURING EARTHQUAKE=================	[redacted]	Feet (Below Finished Grade Cut or Fill Surface)
MAX. HORZ. GROUND SURFACE ACCELERATION=================	[redacted]	Coefficient of Gravity
DESIGN EARTHQUAKE MEAN MAGNITUDE=======================	[redacted]	Moment Magnetude Scale
FINISHED GRADE FILL OR CUT FROM BORING SURFACE=========	[redacted]	Ft. Which is 0 ksf Effect.Surch.Fill Press.
ADJUST DIST. #9 N VALUES TO 60% ENERGY TRANSFER========	[redacted]	(1=Yes OR 2=No)

Sloped Ground Shear Stress Correct. Factor	
(Kα)	1.00
Earthquake Magnitude Scaling Factor	
(MSF)=	1.270

	Boring Data			Conditions During Drilling					Conditions During Earthquake							
Elev. of Sample (Feet)	Boring Sample Depth (Feet)	S.P.T. N Value (Blows)	% Fines < #200 (%)	Effect. Unit Weight (KCF.)	Effect. Vertical Stress (KSF.)	Overburd. & Drillrod Corrected (Ni)60	Fines Content Corrected (Ni)60cs	CSR Resisting Mag 7.5 CRR(7.5)	Effect. Unit Weight (KCF.)	Effect. Vertical Stress (KSF.)	Total Vertical Stress (KSF.)	Confining, Sloping & Mag. Correct. (Kσ)(Kα)(MSF)	**Corrected CRR(7.5) Resisting CRR**	Stress Reduct. Factor (rd)	**Earth Quake Induced CSR**	FACTOR OF SAFETY CRR/CSR
348.1	[redacted]	[redacted]	[redacted]	0.115	0.403	11.717	19.061	0.206	0.115	0.403	0.403	1.416	**0.291**	0.991	**0.126**	**ABO. WAT.**
345.6	[redacted]	[redacted]	[redacted]	0.117	0.692	11.954	19.345	0.209	0.060	0.525	0.587	1.409	**0.294**	0.985	**0.140**	**NL**
343.1	[redacted]	[redacted]	[redacted]	0.104	0.968	2.969	8.563	0.094	0.050	0.465	0.683	1.412	**0.133**	0.979	**0.182**	**NL**
340.6	[redacted]	[redacted]	[redacted]	0.104	1.227	2.708	8.249	0.091	0.050	0.545	0.920	1.407	**0.128**	0.973	**0.208**	**NL**
338.1	[redacted]	[redacted]	[redacted]	0.097	1.478	1.266	6.519	0.076	0.044	0.633	1.164	1.403	**0.107**	0.966	**0.225**	**NL**
335.6	[redacted]	[redacted]	[redacted]	0.097	1.720	1.203	6.444	0.075	0.044	0.708	1.394	1.399	**0.105**	0.960	**0.240**	**NL**
333.1	[redacted]	[redacted]	[redacted]	0.097	1.963	1.155	6.386	0.075	0.044	0.819	1.661	1.395	**0.104**	0.954	**0.245**	**NL**
330.6	[redacted]	[redacted]	[redacted]	0.097	2.206	1.116	6.339	0.075	0.044	0.929	1.928	1.391	**0.104**	0.948	**0.249**	**NL**
328.1	[redacted]	[redacted]	[redacted]	0.097	2.448	1.084	6.301	0.074	0.044	1.040	2.194	1.389	**0.103**	0.941	**0.252**	**NL**
325.6	[redacted]	[redacted]	[redacted]	0.097	2.691	1.058	6.270	0.074	0.044	1.151	2.461	1.387	**0.103**	0.935	**0.253**	**NL**
323.1	[redacted]	[redacted]	[redacted]	0.097	2.933	1.036	6.244	0.074	0.044	1.261	2.728	1.386	**0.102**	0.929	**0.255**	**NL**
320.6	[redacted]	[redacted]	[redacted]	0.104	3.184	2.034	7.441	0.084	0.050	1.454	3.076	1.387	**0.116**	0.920	**0.247**	**NL**
318.1	[redacted]	[redacted]	[redacted]	0.108	3.448	2.995	8.594	0.094	0.053	1.714	3.492	1.391	**0.131**	0.899	**0.232**	**NL**
315.6	[redacted]	[redacted]	[redacted]	0.108	3.718	2.946	8.535	0.094	0.053	1.900	3.834	1.398	**0.131**	0.879	**0.225**	**NL**
313.1	[redacted]	[redacted]	[redacted]	0.111	3.991	3.870	9.644	0.105	0.055	2.077	4.167	1.406	**0.147**	0.858	**0.218**	**NL**
310.6	[redacted]	[redacted]	[redacted]	0.055	4.198	3.850	9.620	0.105	0.055	2.260	4.506	1.417	**0.148**	0.838	**0.212**	**NL**
308.1	[redacted]	[redacted]	[redacted]	0.055	4.336	3.864	9.637	0.105	0.055	2.398	4.800	1.427	**0.150**	0.817	**0.207**	**NL**
305.6	[redacted]	[redacted]	[redacted]	0.064	4.484	10.652	17.783	0.192	0.064	2.734	5.292	1.457	**0.279**	0.797	**0.196**	**NL**
303.1	[redacted]	[redacted]	[redacted]	0.064	4.644	10.669	17.803	0.192	0.064	3.092	5.806	1.497	**0.288**	0.776	**0.185**	**NL**
300.6	[redacted]	[redacted]	[redacted]	0.053	4.789	2.919	3.064	0.056	0.053	2.971	5.842	1.483	**0.082**	0.756	**0.188**	**0.438**
298.1	[redacted]	[redacted]	[redacted]	0.053	4.921	2.933	3.078	0.056	0.053	2.824	5.850	1.466	**0.082**	0.735	**0.193**	**0.422**
295.6	[redacted]	[redacted]	[redacted]	0.066	5.070	13.730	21.476	0.234	0.066	3.323	6.505	1.528	**0.357**	0.715	**0.177**	**NL**
293.1	[redacted]	[redacted]	[redacted]	0.066	5.234	13.753	21.503	0.234	0.066	3.855	7.193	1.614	**0.378**	0.694	**0.164**	**NL**
290.6	[redacted]	[redacted]	[redacted]	0.081	5.418	65.827	68.779	1.000	0.081	4.473	7.967	1.739	**1.739**	0.674	**0.152**	**11.432**

6-S

LIQUEFACTION POTENTIAL ANALYSIS

BUREAU OF BRIDGES AND STRUCTURES FOUNDATIONS UNIT wmk 8/25/01

REFERENCE BORING NUMBER================================	[redacted]	
STRUCTURE NUMBER=======================================	[redacted]	
ELEVATION OF BORING GROUND SURFACE=====================	[redacted]	Feet
DEPTH TO GROUNDWATER DURING DRILLING===================	[redacted]	Feet (Below Boring Ground Surface)
DEPTH TO GROUNDWATER DURING EARTHQUAKE=================	[redacted]	Feet (Below Finished Grade Cut or Fill Surface)
MAX. HORZ. GROUND SURFACE ACCELERATION=================	[redacted]	Coefficient of Gravity
DESIGN EARTHQUAKE MEAN MAGNITUDE=======================	[redacted]	Moment Magnetude Scale
FINISHED GRADE FILL OR CUT FROM BORING SURFACE==========	[redacted]	Ft. Which is 0 ksf Effect.Surch.Fill Press.
ADJUST DIST. #9 N VALUES TO 60% ENERGY TRANSFER=========	[redacted]	(1=Yes OR 2=No)

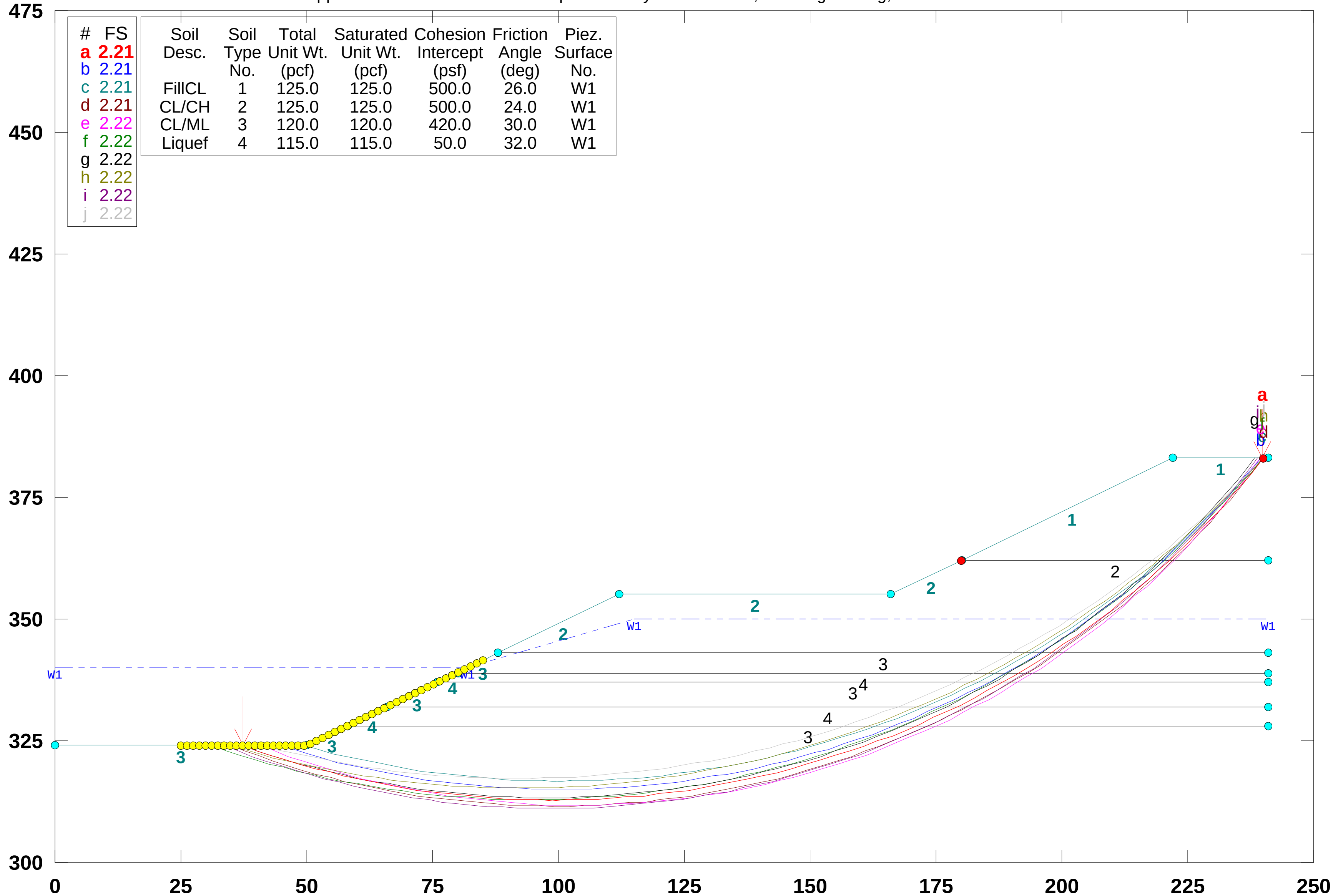
Sloped Ground Shear Stress Correct. Factor (Kα)	1.00
Earthquake Magnitude Scaling Factor (MSF)=	1.270

	Boring Data			Conditions During Drilling					Conditions During Earthquake							
Elev. of Sample (Feet)	Boring Sample Depth (Feet)	S.P.T. N Value (Blows)	% Fines < #200 (%)	Effect. Unit Weight (KCF.)	Effect. Vertical Stress (KSF.)	Overburd. & Drillrod Corrected (Ni)60	Fines Content Corrected (Ni)60cs	CSR Resisting Mag 7.5 CRR(7.5)	Effect. Unit Weight (KCF.)	Effect. Vertical Stress (KSF.)	Total Vertical Stress (KSF.)	Confining, Sloping & Mag. Correct. (Kσ)(Kα)(MSF)	**Corrected CRR(7.5) Resisting CRR**	Stress Reduct. Factor (rd)	**Earth Quake Induced CSR**	FACTOR OF SAFETY CRR/CSR
380	[redacted]	[redacted]	[redacted]	0.123	0.369	23.301	32.961	1.000	0.123	0.369	0.369	1.418	**1.418**	0.993	**0.126**	**ABO. WAT.**
377.5	[redacted]	[redacted]	[redacted]	0.117	0.668	12.102	19.522	0.211	0.117	0.659	0.659	1.402	**0.296**	0.986	**0.125**	**ABO. WAT.**
375	[redacted]	[redacted]	[redacted]	0.115	0.958	8.905	15.686	0.170	0.115	0.927	0.927	1.391	**0.236**	0.980	**0.124**	**ABO. WAT.**
372.5	[redacted]	[redacted]	[redacted]	0.111	1.240	5.359	11.431	0.124	0.111	1.185	1.185	1.387	**0.172**	0.974	**0.123**	**ABO. WAT.**
370	[redacted]	[redacted]	[redacted]	0.117	1.524	8.681	15.418	0.167	0.117	1.478	1.478	1.387	**0.231**	0.968	**0.123**	**ABO. WAT.**
367.5	[redacted]	[redacted]	[redacted]	0.115	1.814	6.997	13.396	0.145	0.115	1.796	1.796	1.394	**0.202**	0.961	**0.122**	**ABO. WAT.**
365	[redacted]	[redacted]	[redacted]	0.111	2.096	4.448	10.337	0.112	0.111	2.032	2.032	1.404	**0.157**	0.955	**0.121**	**ABO. WAT.**
362.5	[redacted]	[redacted]	[redacted]	0.104	2.364	2.145	7.574	0.085	0.104	2.197	2.197	1.413	**0.120**	0.949	**0.120**	**ABO. WAT.**
360	[redacted]	[redacted]	[redacted]	0.115	2.637	6.239	12.486	0.135	0.115	2.515	2.515	1.437	**0.194**	0.943	**0.119**	**ABO. WAT.**
357.5	[redacted]	[redacted]	[redacted]	0.117	2.927	7.070	13.484	0.146	0.060	2.229	2.229	1.415	**0.207**	0.936	**0.119**	**NL**
355	[redacted]	[redacted]	[redacted]	0.108	3.208	2.960	8.552	0.094	0.053	1.576	1.732	1.388	**0.130**	0.930	**0.130**	**NL**
352.5	[redacted]	[redacted]	[redacted]	0.111	3.481	3.874	9.648	0.105	0.055	1.645	1.957	1.390	**0.146**	0.924	**0.139**	**NL**
350	[redacted]	[redacted]	[redacted]	0.111	3.758	3.809	9.571	0.104	0.055	1.819	2.287	1.395	**0.145**	0.903	**0.144**	**NL**
347.5	[redacted]	[redacted]	[redacted]	0.108	4.031	2.817	8.381	0.092	0.053	1.915	2.539	1.398	**0.129**	0.883	**0.148**	**NL**
345	[redacted]	[redacted]	[redacted]	0.113	4.306	4.637	10.565	0.114	0.057	2.085	2.865	1.407	**0.161**	0.862	**0.150**	**NL**
342.5	[redacted]	[redacted]	[redacted]	0.111	4.586	3.669	9.402	0.102	0.055	2.270	3.206	1.418	**0.145**	0.842	**0.151**	**NL**
340	[redacted]	[redacted]	[redacted]	0.111	4.863	3.634	9.361	0.102	0.055	2.370	3.462	1.425	**0.145**	0.821	**0.152**	**NL**
337.5	[redacted]	[redacted]	[redacted]	0.104	5.131	1.804	7.164	0.081	0.050	2.381	3.629	1.426	**0.116**	0.801	**0.155**	**NL**
335	[redacted]	[redacted]	[redacted]	0.104	5.390	1.794	7.152	0.081	0.050	2.379	3.783	1.426	**0.116**	0.780	**0.157**	**NL**
332.5	[redacted]	[redacted]	[redacted]	0.053	5.585	2.693	8.232	0.091	0.053	2.584	4.144	1.443	**0.131**	0.760	**0.154**	**NL**
330	[redacted]	[redacted]	[redacted]	0.053	5.717	2.711	8.253	0.091	0.053	2.797	4.513	1.463	**0.133**	0.739	**0.151**	**NL**
327.5	[redacted]	[redacted]	[redacted]	0.057	5.855	4.547	10.456	0.113	0.057	3.046	4.918	1.492	**0.169**	0.719	**0.147**	**NL**
325	[redacted]	[redacted]	[redacted]	0.057	5.997	4.573	10.487	0.114	0.057	3.304	5.332	1.526	**0.173**	0.698	**0.143**	**NL**
322.5	[redacted]	[redacted]	[redacted]	0.055	6.137	3.680	9.416	0.103	0.055	3.391	5.575	1.538	**0.158**	0.678	**0.141**	**NL**
320	[redacted]	[redacted]	[redacted]	0.055	6.275	3.702	9.442	0.103	0.055	3.473	5.813	1.551	**0.159**	0.657	**0.139**	**NL**
317.5	[redacted]	[redacted]	[redacted]	0.064	6.424	10.233	17.279	0.186	0.064	3.893	6.389	1.621	**0.302**	0.637	**0.132**	**NL**
315	[redacted]	[redacted]	[redacted]	0.064	6.583	10.277	17.332	0.187	0.064	4.335	6.987	1.709	**0.320**	0.616	**0.126**	**NL**
312.5	[redacted]	[redacted]	[redacted]	0.059	6.736	5.633	11.759	0.127	0.059	4.309	7.117	1.703	**0.217**	0.596	**0.125**	**NL**
310	[redacted]	[redacted]	[redacted]	0.059	6.882	5.662	11.795	0.128	0.059	4.271	7.235	1.695	**0.216**	0.575	**0.124**	**NL**
307.5	[redacted]	[redacted]	[redacted]	0.059	7.028	5.692	11.831	0.128	0.059	4.417	7.537	1.727	**0.221**	0.555	**0.120**	**NL**
305	[redacted]	[redacted]	[redacted]	0.059	7.174	5.722	11.866	0.128	0.059	4.563	7.839	1.760	**0.226**	0.534	**0.116**	**NL**
297.5	[redacted]	[redacted]	[redacted]	0.063	7.630	9.675	10.754	0.117	0.063	5.190	8.934	1.918	**0.224**	0.473	**0.103**	**2.166**
292.5	[redacted]	[redacted]	[redacted]	0.071	7.965	24.371	30.250	1.000	0.071	6.069	10.125	2.187	**2.187**	0.432	**0.091**	**23.951**

Appendix C

2008-3043.50 IL 13 over the Big Muddy East Abutment - Long Term

r:\emtapps\stedwin files\083043\elterm.pl2 Run By: Sarah Stock, SCI Engineering, Inc. 11/20/2009 10:41AM

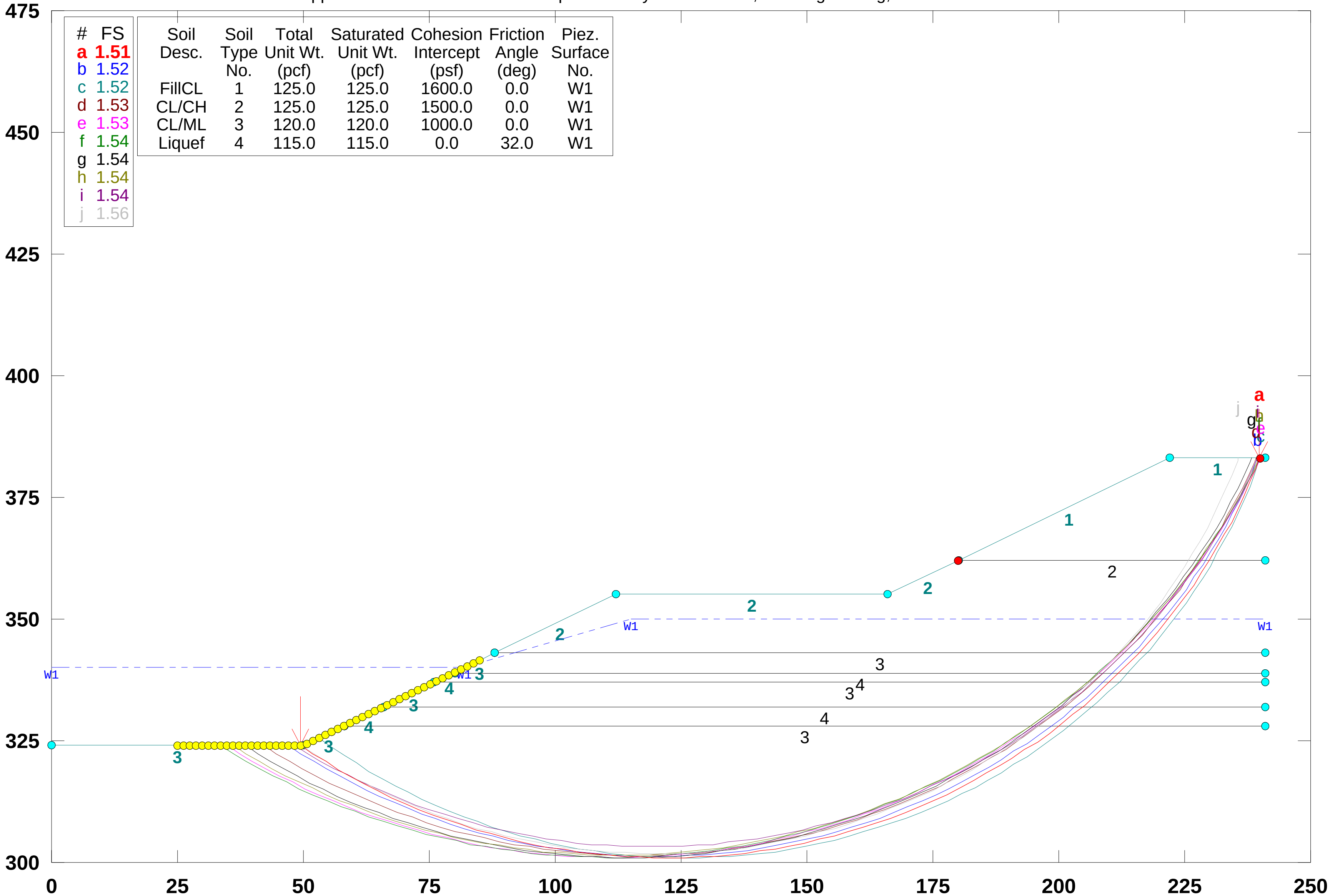


STABL6H FSmin=2.21

Safety Factors Are Calculated By The Modified Bishop Method

2008-3043.50 IL 13 over the Big Muddy East Abutment - Short Term

r:\emtapps\stedwin files\083043\esterm.pl2 Run By: Sarah Stock, SCI Engineering, Inc. 11/20/2009 02:20PM

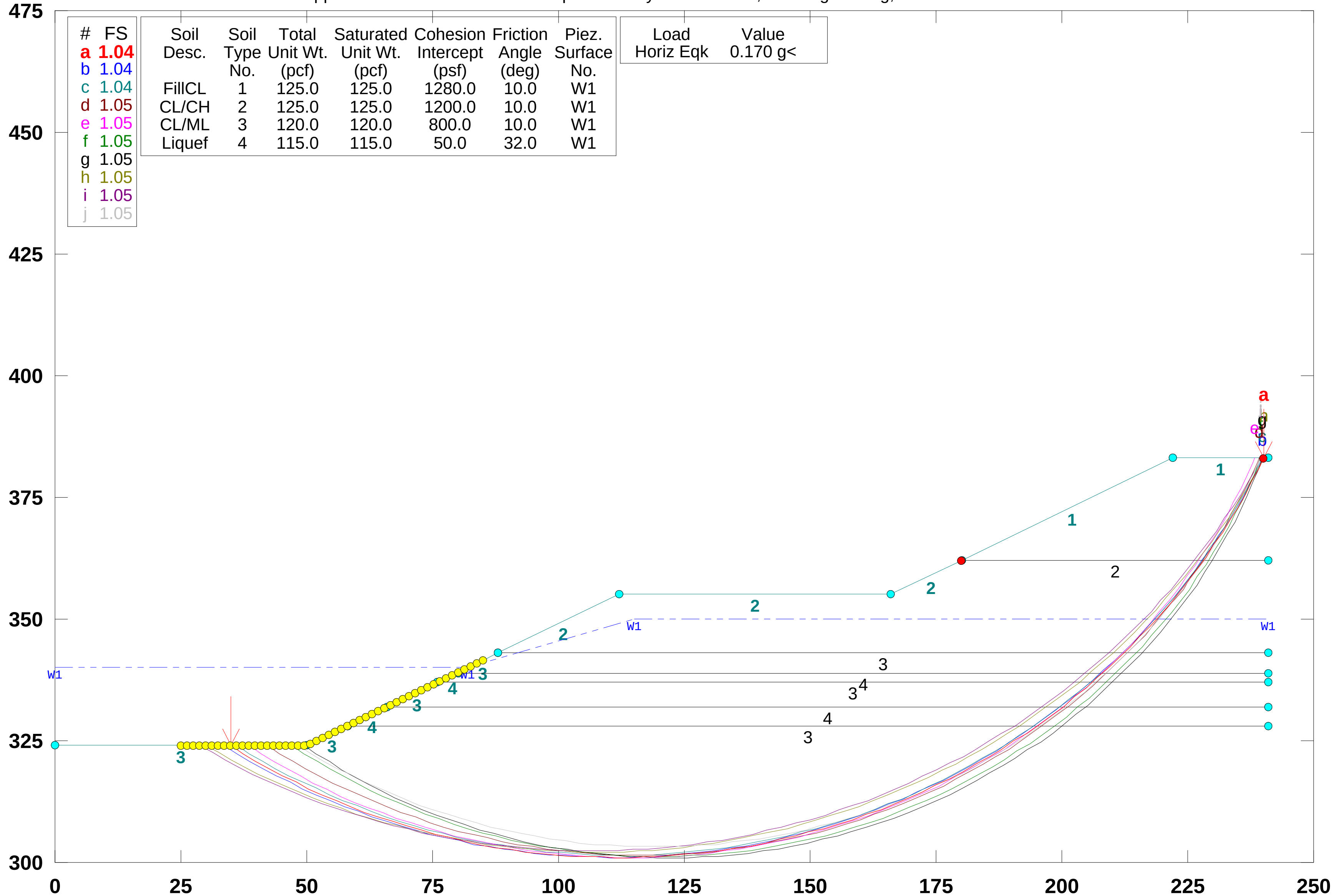


STABL6H FSmin=1.51

Safety Factors Are Calculated By The Modified Bishop Method

2008-3043.50 IL 13 over the Big Muddy East Abutment - Long Term, Seismic

r:\emtapps\stedwin files\083043\estseis.pl2 Run By: Sarah Stock, SCI Engineering, Inc. 11/20/2009 02:27PM



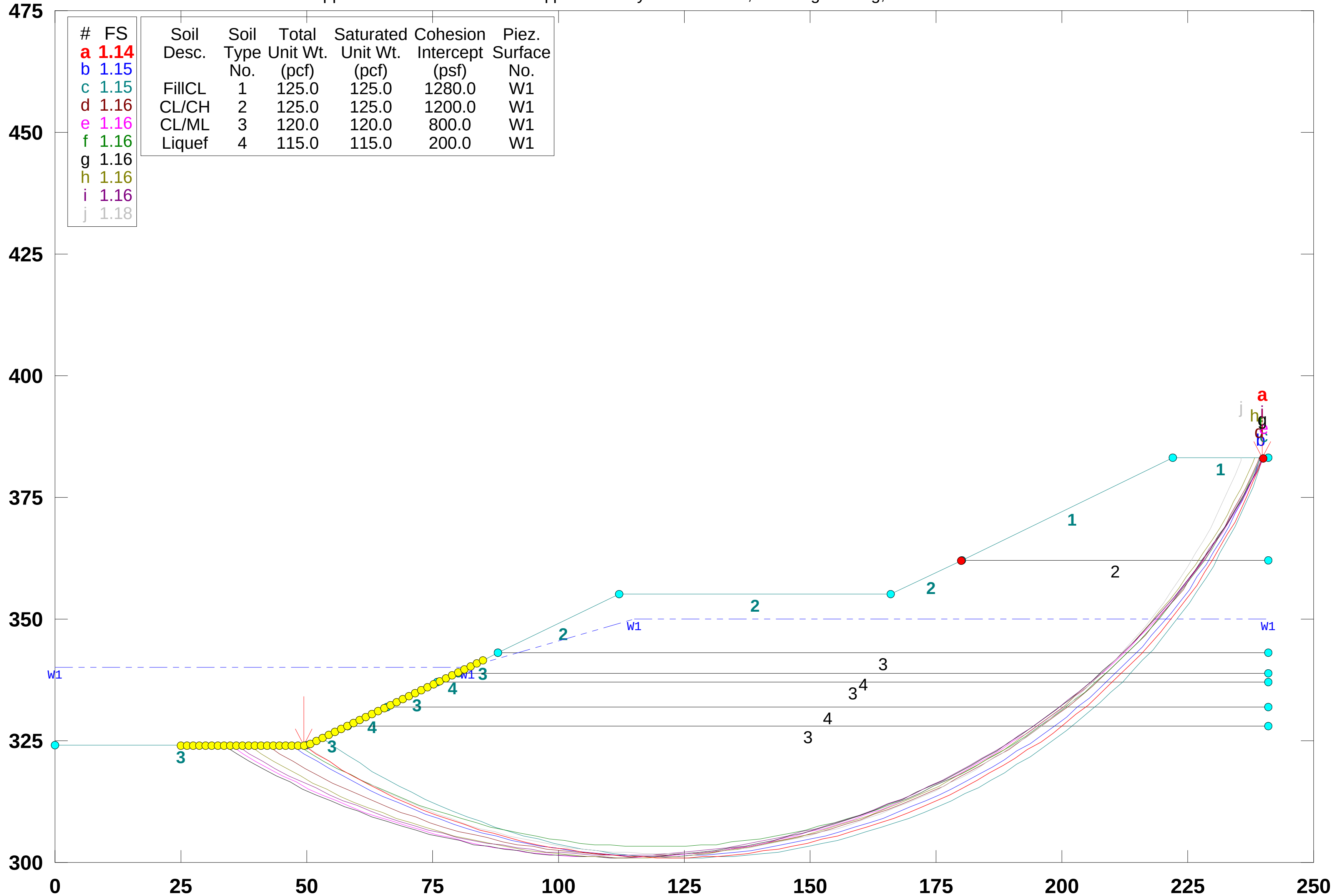
STABL6H FSmin=1.04

Safety Factors Are Calculated By The Modified Bishop Method

STED

2008-3043.50 IL 13 over the Big Muddy East Abutment - Long Term

r:\emtapps\stedwin files\083043\eltliq.pl2   Run By: Sarah Stock, SCI Engineering, Inc.   11/20/2009   02:30PM



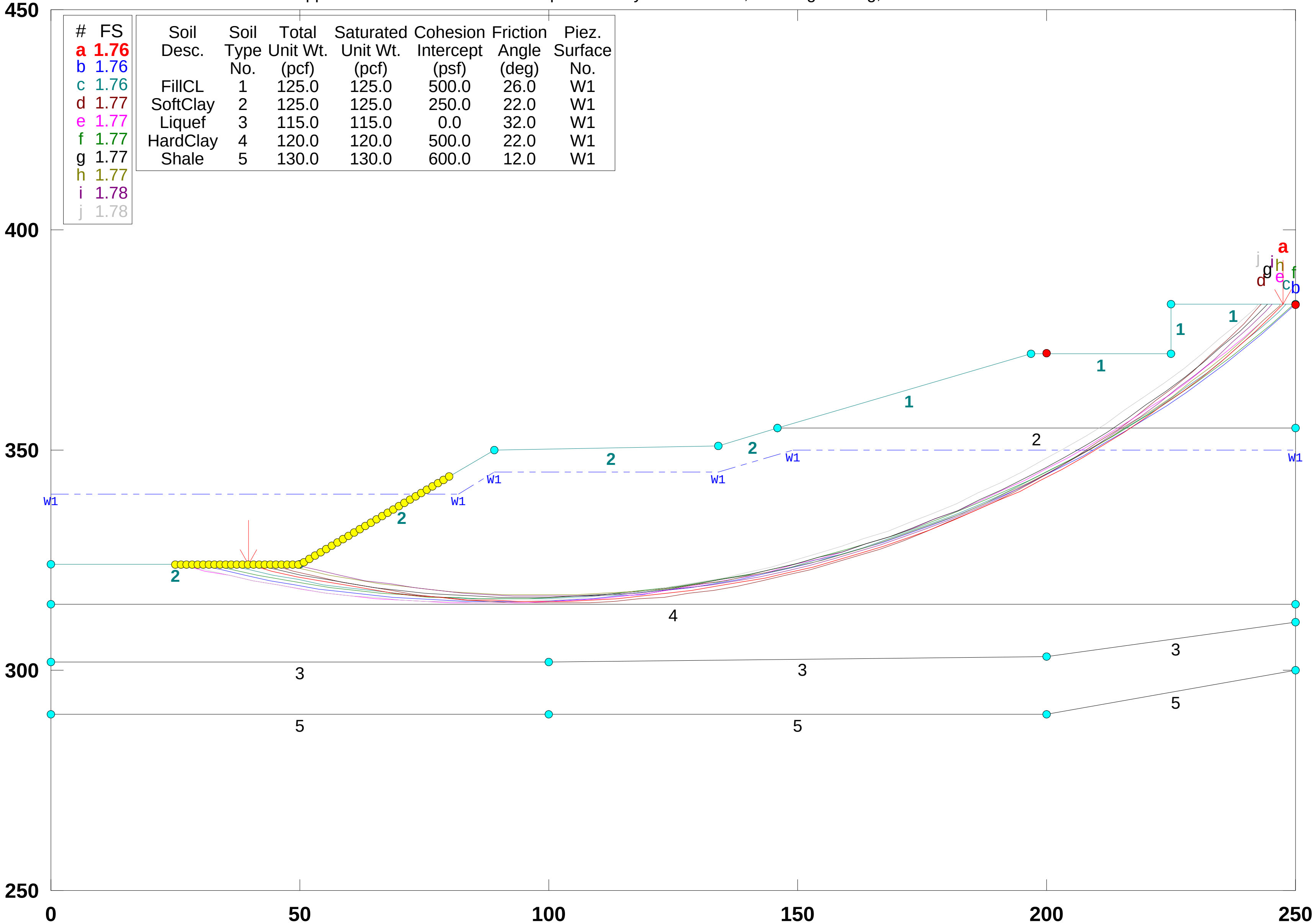
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Piez. Surface No.
FillCL	1	125.0	125.0	1280.0	W1
CL/CH	2	125.0	125.0	1200.0	W1
CL/ML	3	120.0	120.0	800.0	W1
Liquef	4	115.0	115.0	200.0	W1

STABL6H FSmin=1.14

Safety Factors Are Calculated By The Modified Bishop Method

2008-3043.50 IL 13 over the Big Muddy West Abutment - Long Term

r:\emtapps\stedwin files\083043\wlterm.pl2 Run By: Sarah Stock, SCI Engineering, Inc. 11/25/2009 12:44PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
FillCL	1	125.0	125.0	500.0	26.0	W1
SoftClay	2	125.0	125.0	250.0	22.0	W1
Liquef	3	115.0	115.0	0.0	32.0	W1
HardClay	4	120.0	120.0	500.0	22.0	W1
Shale	5	130.0	130.0	600.0	12.0	W1

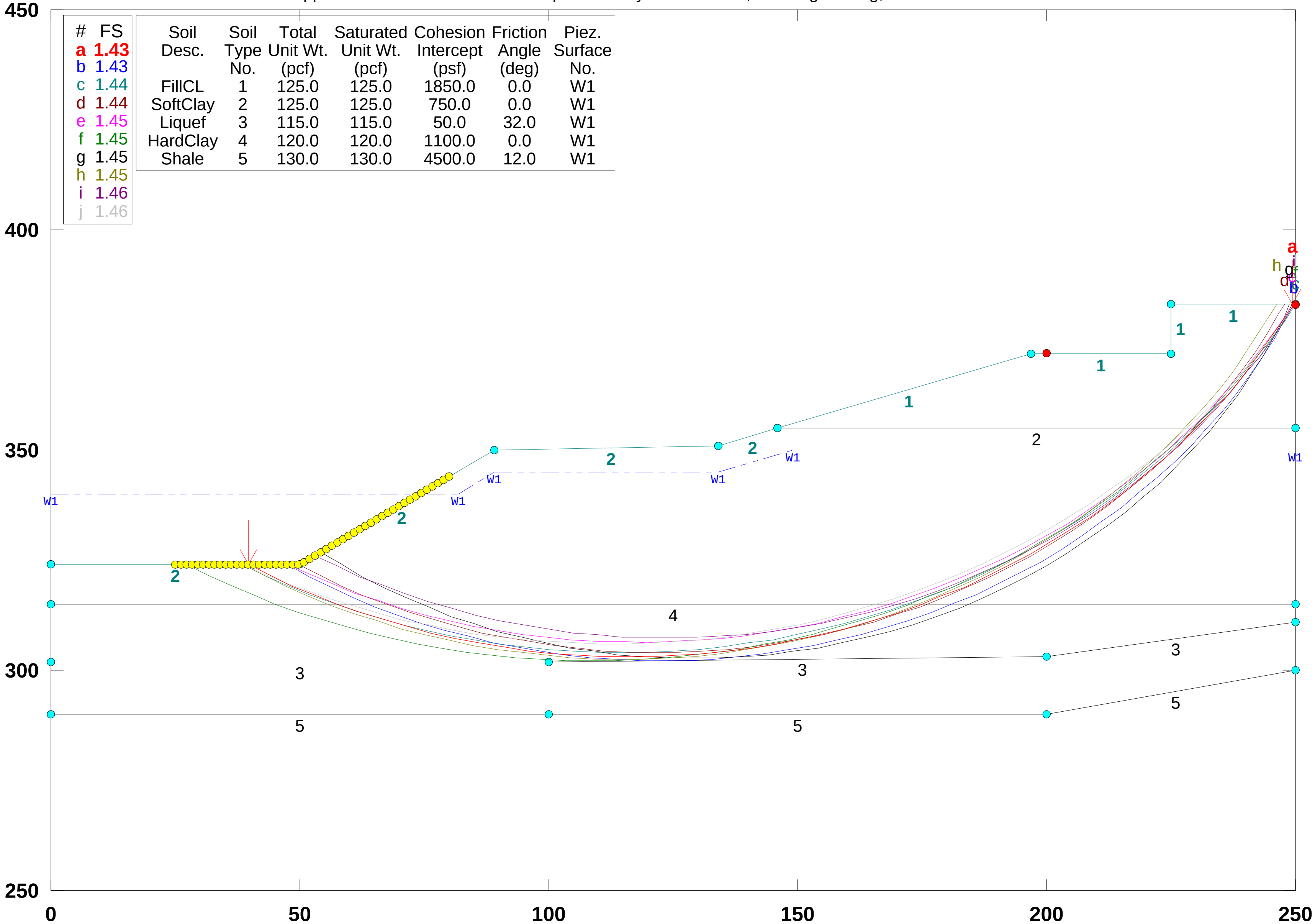
STABL6H FSmin=1.76

Safety Factors Are Calculated By The Modified Bishop Method

STED

2008-3043.50 IL 13 over the Big Muddy West Abutment - Short Term

r:\emtapps\stedwin files\083043\wsterm.pl2 Run By: Sarah Stock, SCI Engineering, Inc. 11/25/2009 12:45PM



#	FS
a	1.43
b	1.43
c	1.44
d	1.44
e	1.45
f	1.45
g	1.45
h	1.45
i	1.46
j	1.46

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
FillCL	1	125.0	125.0	1850.0	0.0	W1
SoftClay	2	125.0	125.0	750.0	0.0	W1
Liquef	3	115.0	115.0	50.0	32.0	W1
HardClay	4	120.0	120.0	1100.0	0.0	W1
Shale	5	130.0	130.0	4500.0	12.0	W1

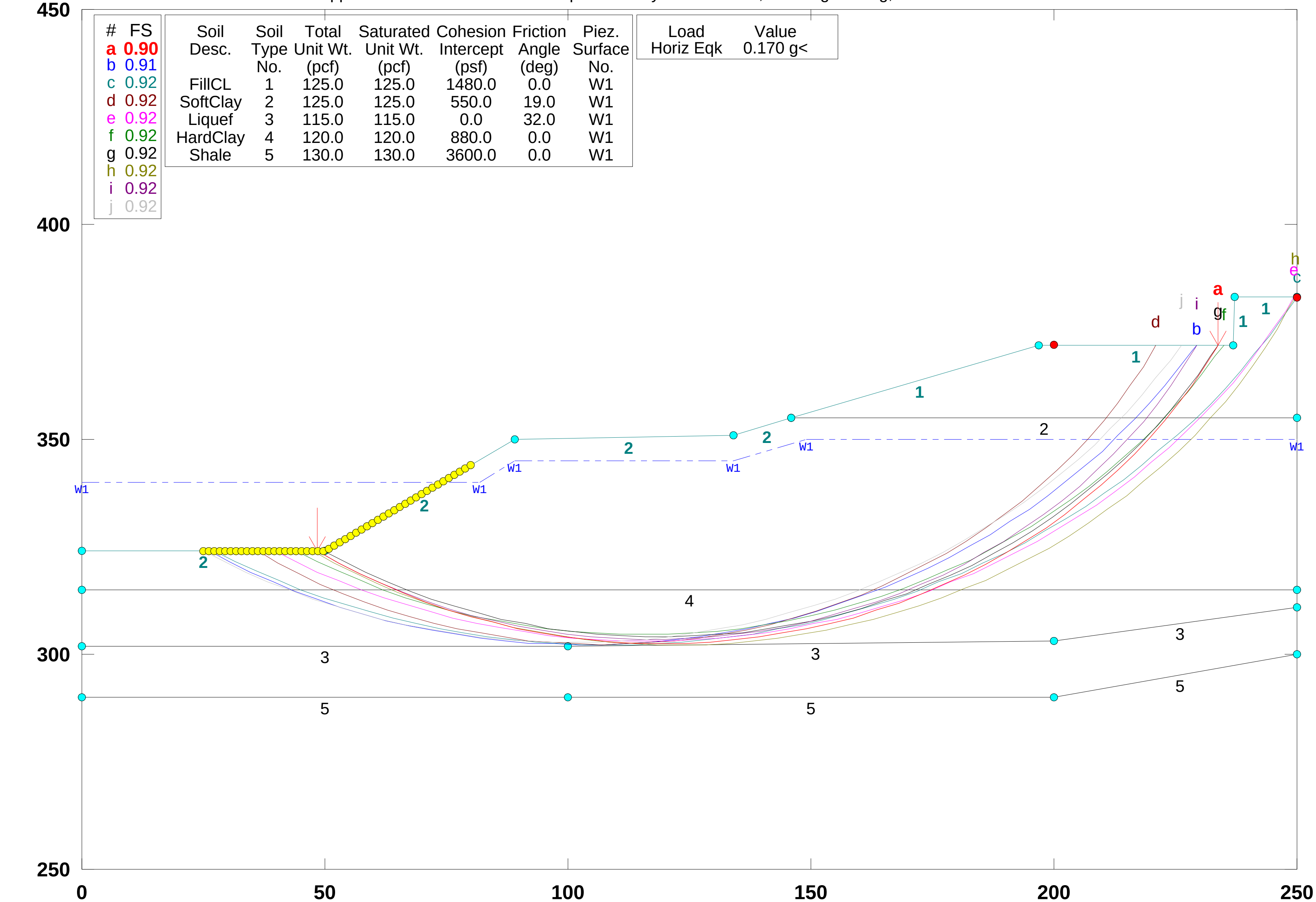
STABL6H FSmin=1.43

Safety Factors Are Calculated By The Modified Bishop Method

STED

2008-3043.50 IL 13 over the Big Muddy West Abutment - Seismic

r:\emtapps\stedwin files\083043\wltseis.pl2   Run By: Sarah Stock, SCI Engineering, Inc.   11/23/2009   03:59PM



#	FS
a	0.90
b	0.91
c	0.92
d	0.92
e	0.92
f	0.92
g	0.92
h	0.92
i	0.92
j	0.92

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
FillCL	1	125.0	125.0	1480.0	0.0	W1
SoftClay	2	125.0	125.0	550.0	19.0	W1
Liquef	3	115.0	115.0	0.0	32.0	W1
HardClay	4	120.0	120.0	880.0	0.0	W1
Shale	5	130.0	130.0	3600.0	0.0	W1

Load	Value
Horiz Eqk	0.170 g<

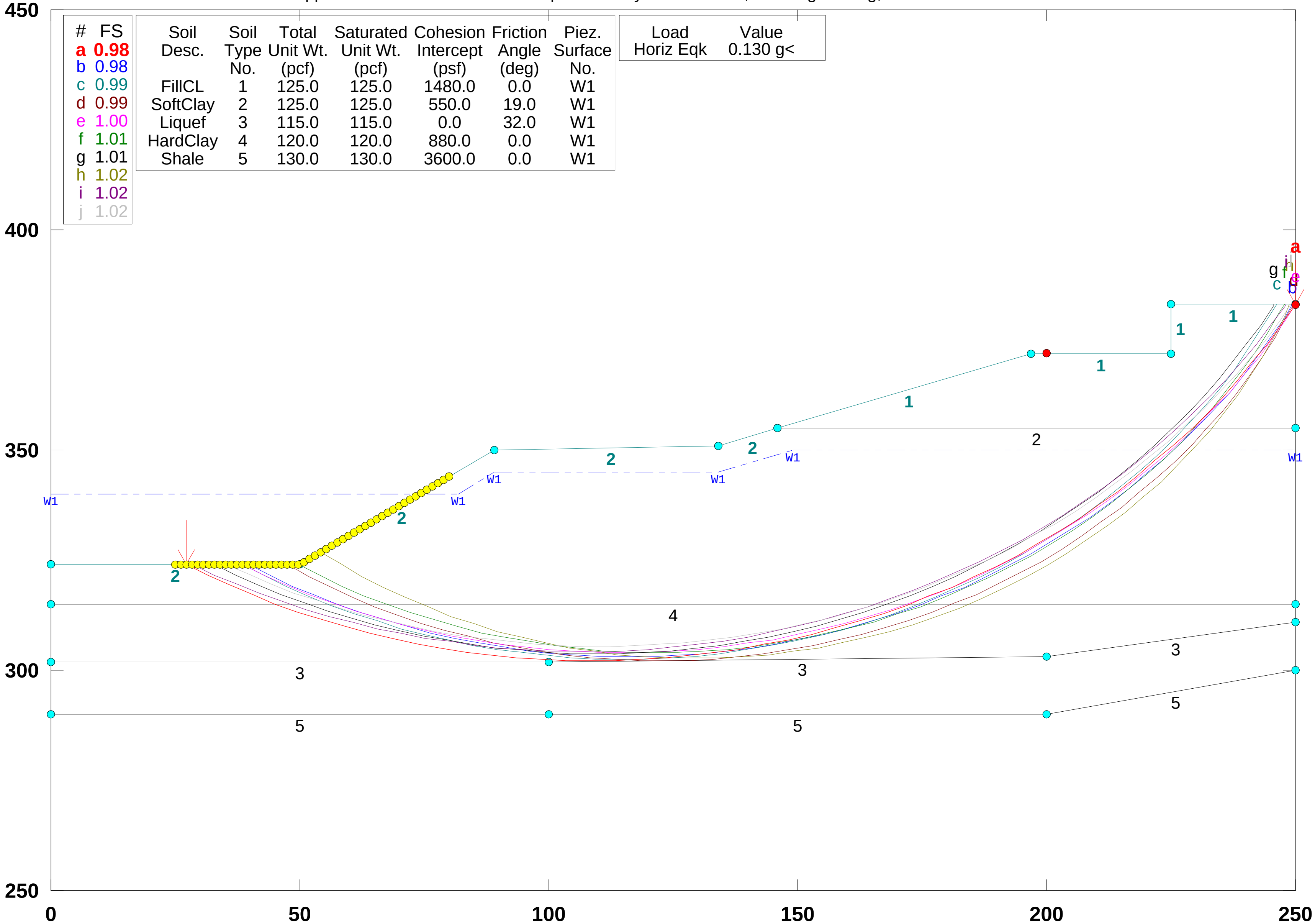
STABL6H FSmin=0.90

Safety Factors Are Calculated By The Modified Bishop Method

STED

2008-3043.50 IL 13 over the Big Muddy West Abutment - Seismic

r:\emtapps\stedwin files\083043\wltseis.pl2 Run By: Sarah Stock, SCI Engineering, Inc. 11/25/2009 12:45PM

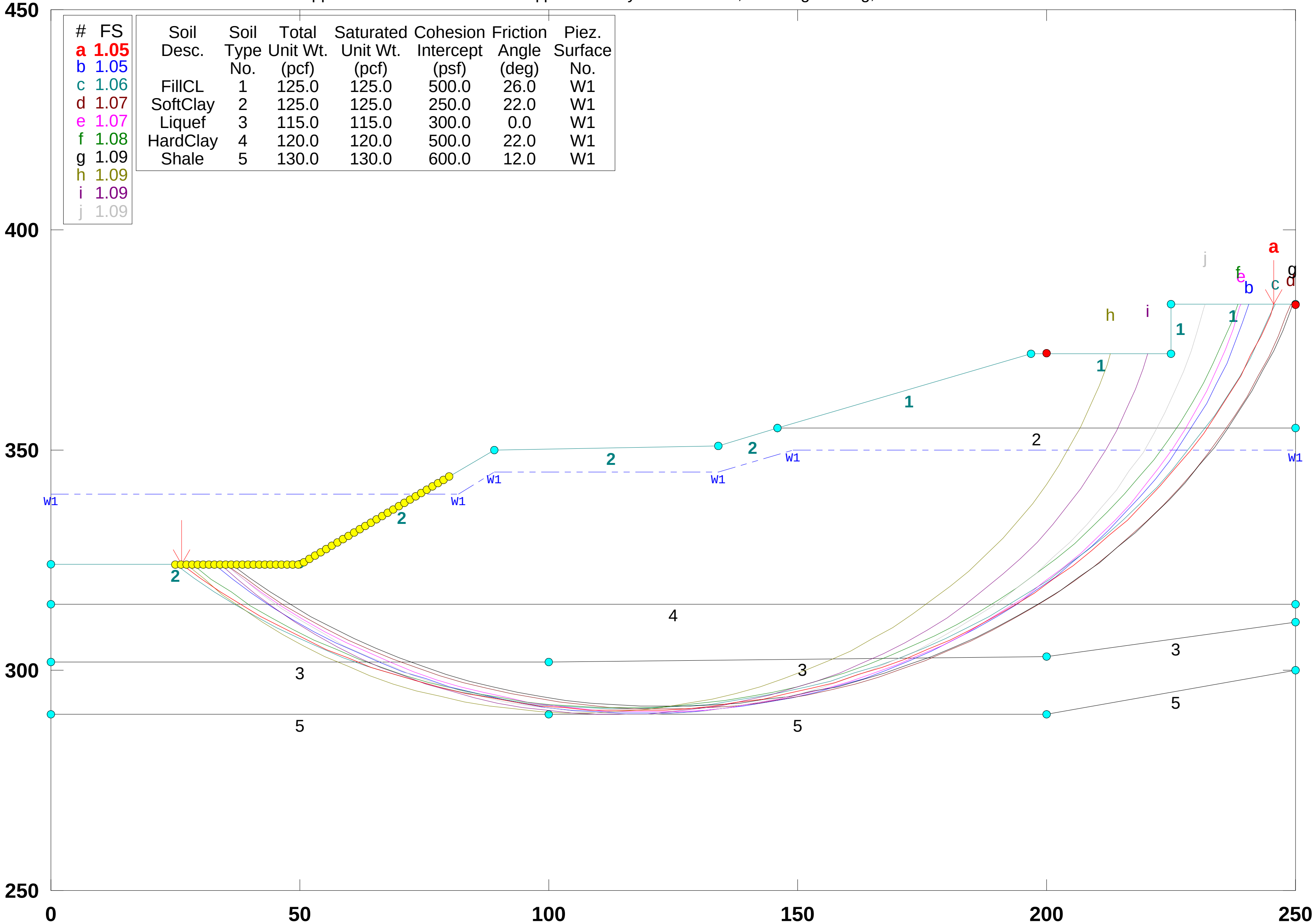


STABL6H FSmin=0.98

Safety Factors Are Calculated By The Modified Bishop Method

2008-3043.50 IL 13 over the Big Muddy West Abutment - LT After Liquefaction

r:\emtapps\stedwin files\083043\wltliq.pl2 Run By: Sarah Stock, SCI Engineering, Inc. 11/25/2009 12:46PM



#	FS
a	1.05
b	1.05
c	1.06
d	1.07
e	1.07
f	1.08
g	1.09
h	1.09
i	1.09
j	1.09

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
FillCL	1	125.0	125.0	500.0	26.0	W1
SoftClay	2	125.0	125.0	250.0	22.0	W1
Liquef	3	115.0	115.0	300.0	0.0	W1
HardClay	4	120.0	120.0	500.0	22.0	W1
Shale	5	130.0	130.0	600.0	12.0	W1

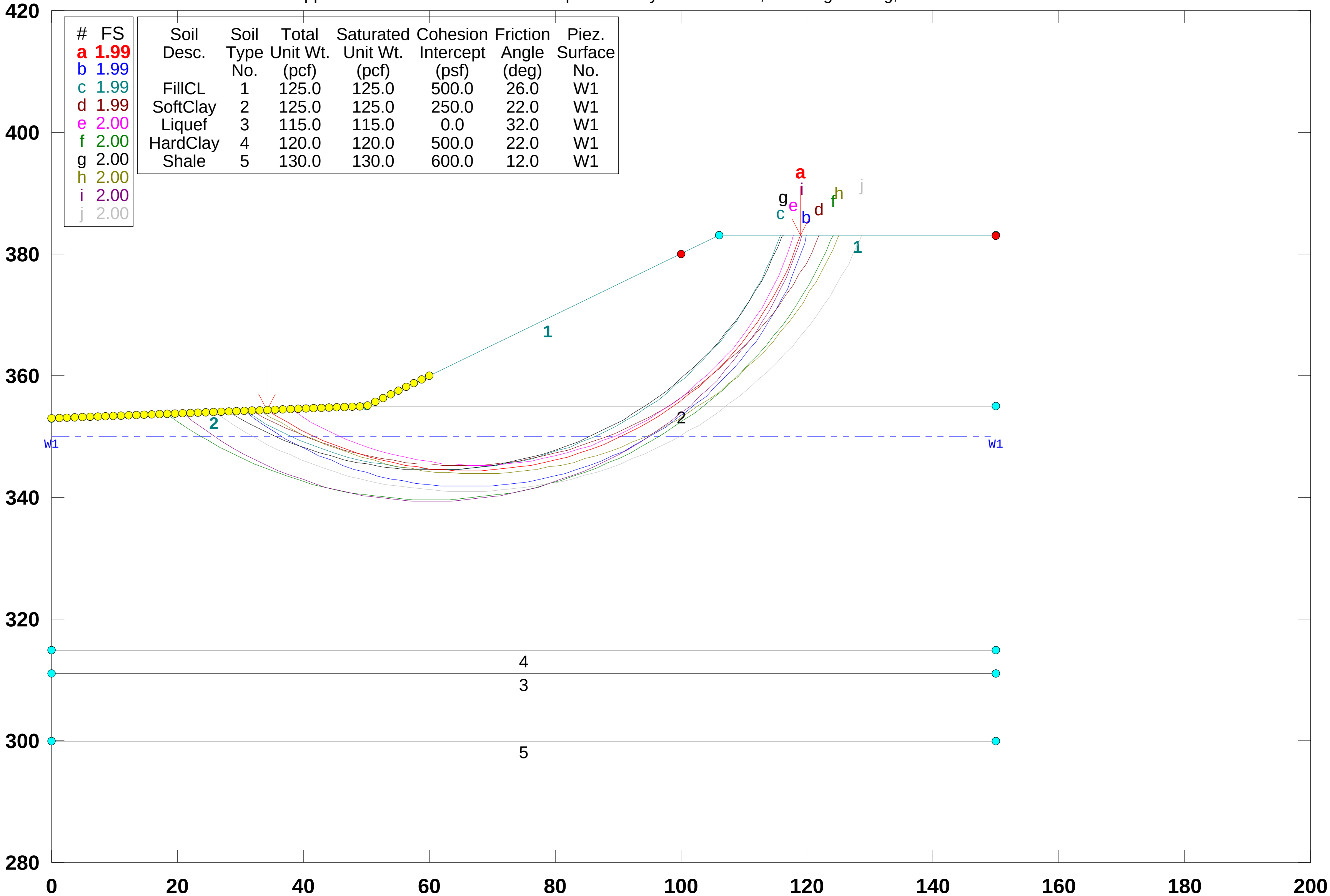
STABL6H FSmin=1.05

Safety Factors Are Calculated By The Modified Bishop Method

STED

2008-3043.50 IL 13 over the Big Muddy West Abutment - Side Slope

r:\emtapps\stedwin files\083043\westside.pl2 Run By: Sarah Stock, SCI Engineering, Inc. 11/23/2009 11:59AM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
FillCL	1	125.0	125.0	500.0	26.0	W1
SoftClay	2	125.0	125.0	250.0	22.0	W1
Liquef	3	115.0	115.0	0.0	32.0	W1
HardClay	4	120.0	120.0	500.0	22.0	W1
Shale	5	130.0	130.0	600.0	12.0	W1

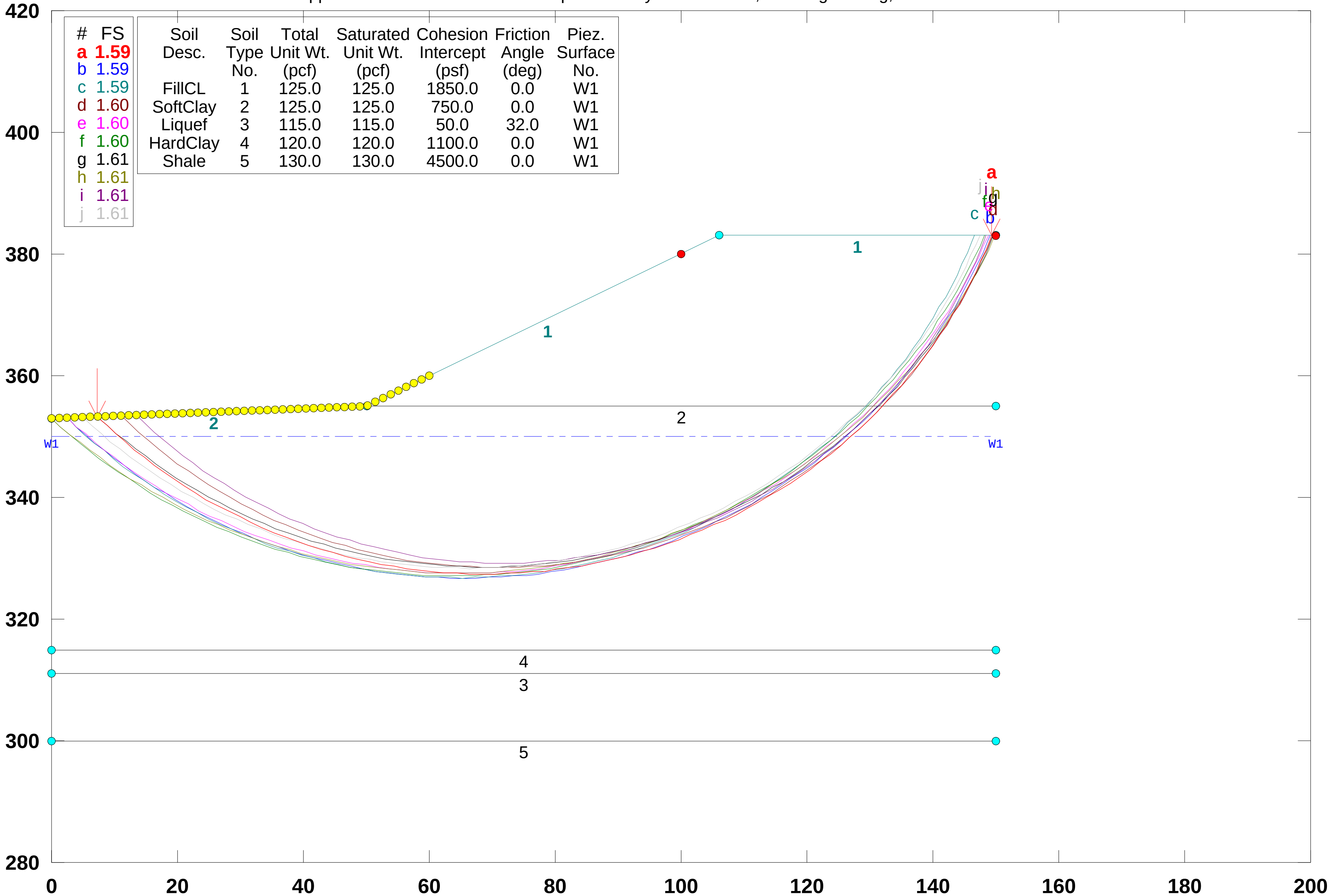
STABL6H FSmin=1.99

Safety Factors Are Calculated By The Modified Bishop Method

STED

2008-3043.50 IL 13 over the Big Muddy West Abutment - Side Slope, Short Term

r:\emtapps\stedwin files\083043\wsidest.pl2 Run By: Sarah Stock, SCI Engineering, Inc. 11/23/2009 11:59AM

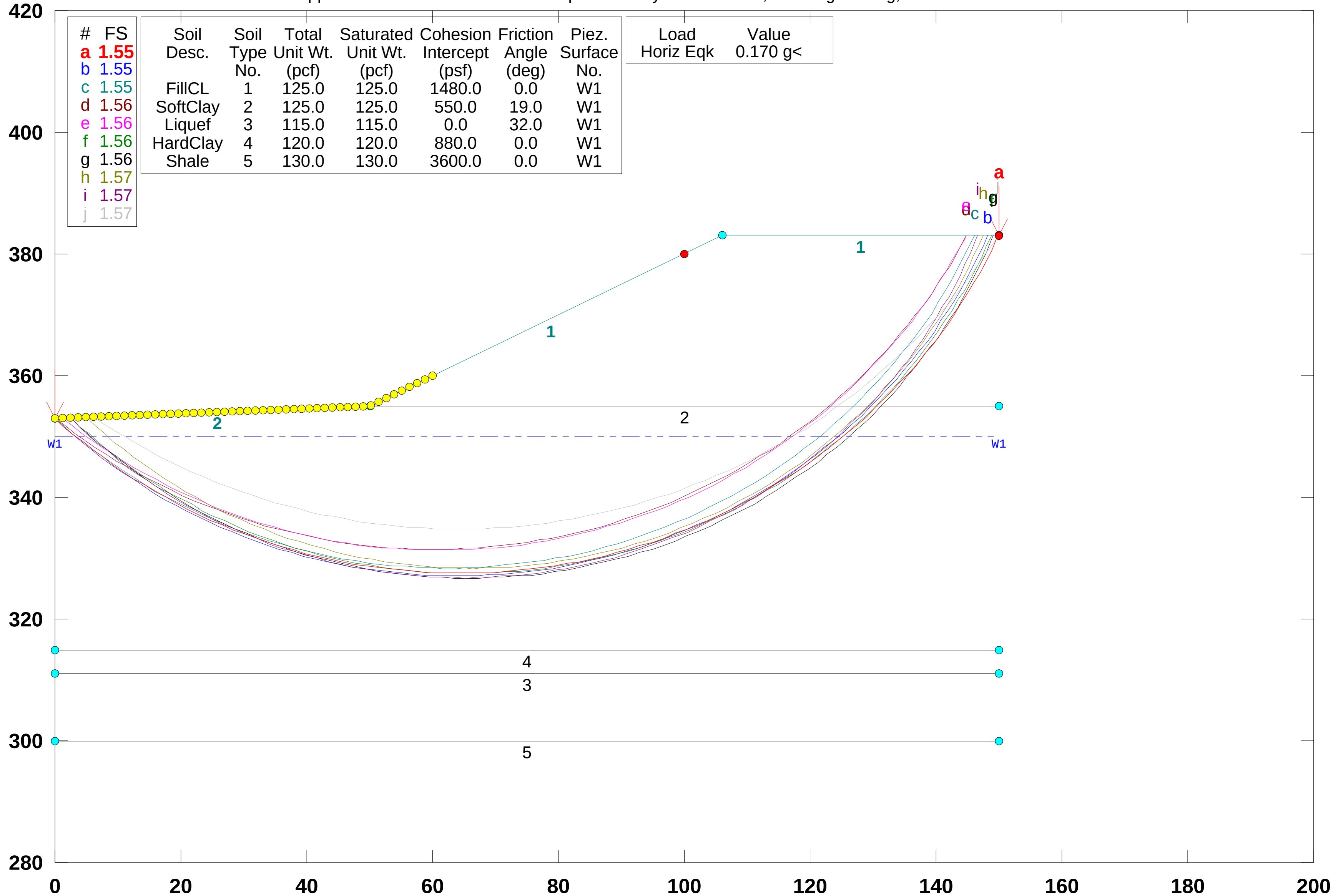


STABL6H FSmin=1.59

Safety Factors Are Calculated By The Modified Bishop Method

2008-3043.50 IL 13 over the Big Muddy West Abutment - Side Slope, Seismic

r:\emtapps\stedwin files\083043\wsidesei.pl2 Run By: Sarah Stock, SCI Engineering, Inc. 11/23/2009 03:27PM



#	FS
a	1.55
b	1.55
c	1.55
d	1.56
e	1.56
f	1.56
g	1.56
h	1.57
i	1.57
j	1.57

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
FillCL	1	125.0	125.0	1480.0	0.0	W1
SoftClay	2	125.0	125.0	550.0	19.0	W1
Liquef	3	115.0	115.0	0.0	32.0	W1
HardClay	4	120.0	120.0	880.0	0.0	W1
Shale	5	130.0	130.0	3600.0	0.0	W1

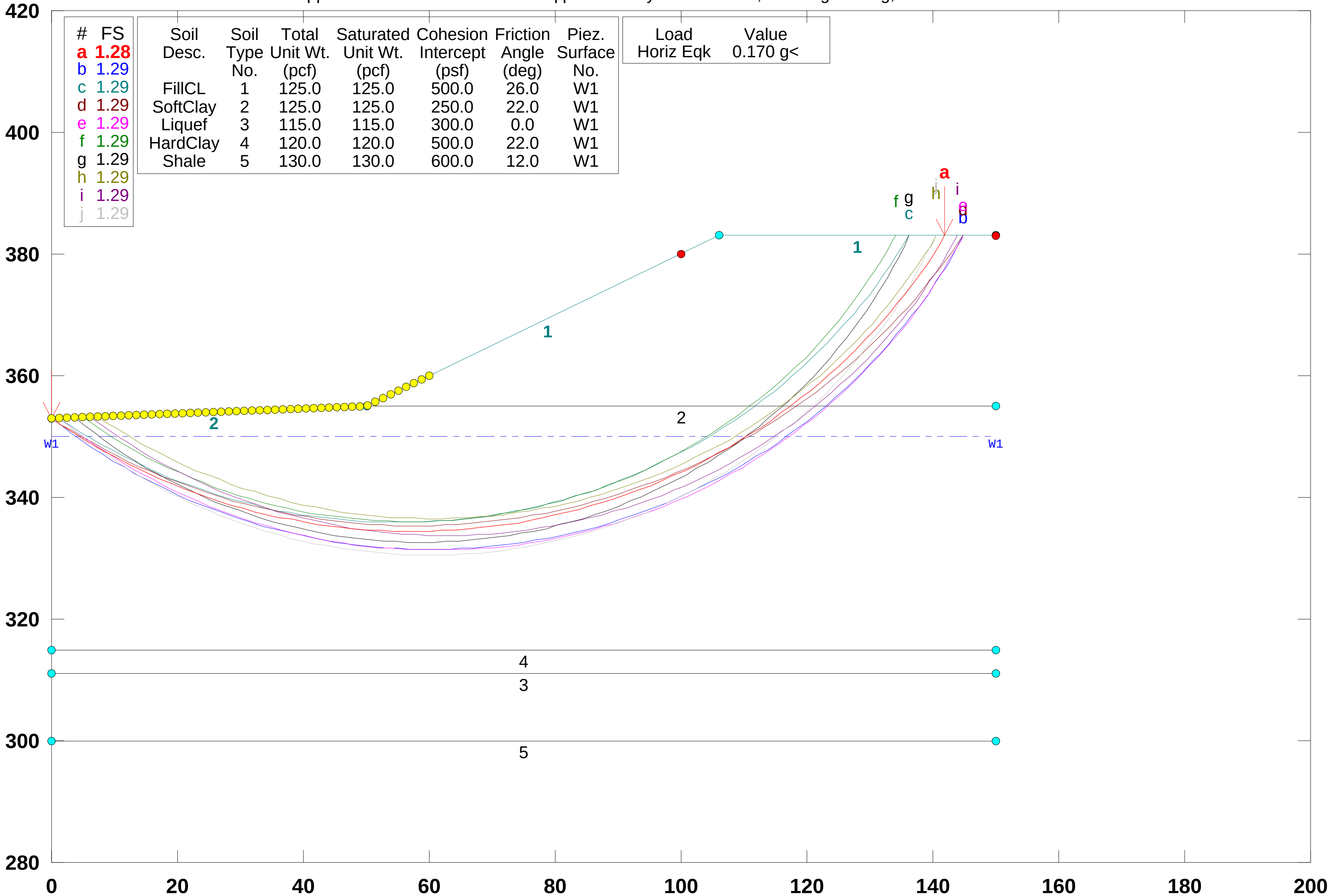
Load	Value
Horiz Eqk	0.170 g<

STABL6H FSmin=1.55

Safety Factors Are Calculated By The Modified Bishop Method

2008-3043.50 IL 13 over the Big Muddy West Abutment - Side Slope, Liquefaction

r:\emtapps\stedwin files\083043\wsideliq.pl2 Run By: Sarah Stock, SCI Engineering, Inc. 11/23/2009 12:02PM



STABL6H FSmin=1.28

Safety Factors Are Calculated By The Modified Bishop Method

STED

Appendix D

APPENDIX D

PROJECT: Bridge Replacement – IL 13 over Big Muddy **SCI No.** 2008-3043.50
LOCATION: Jackson County, Illinois
CLIENT: Oates Associates, Inc.
STRUCTURE: 039-0013 (EB), 039-0049 (WB)

Table D.1 – Soil Modulus Parameters (k) for 1-S

Depth (ft)	Elevation (ft)	Abbreviated Soil Description	Soil Modulus Parameter (pci)	Effective Unit Weight (pcf)	Cohesion (psf)	Phi (degrees)	E_{50}/k_{rm}
0 - 17	383.2 – 366.2	Silty Clay Loam	500	110	2,000	--	0.007
17 – 28.2	366.2 – 355.0	Silty Clay to Clay	500	115	1,475	--	0.007
28.2 – 39.5	355.0 – 343.7	Silty Clay to Clay	500	55	1,500	--	0.007
39.5 – 54.5	343.7 – 328.7	Silty Clay to Clay	500	60	1,730	--	0.007
54.5 – 74.5	328.7 – 308.7	Clay with Silt and Sand Layers	500	60	1,700	--	0.007
74.5 – 79.5	308.7 – 303.7	Clay	100	75	900	--	0.020
79.5 – 89.5	303.7 – 293.7	Clay with Sand Layers	100	55	600	--	0.020
89.5 – 94.5	293.7 – 288.7	Sand	60	50	--	35	--
Below 94.5	Below 288.7	Clayey Shale over Coal	--	130	5,000	--	0.0005

Table D.2 – Soil Modulus Parameters (k) for 2-S

Depth (ft)	Elevation (ft)	Abbreviated Soil Description	Soil Modulus Parameter (pci)	Effective Unit Weight (pcf)	Cohesion (psf)	Phi (degrees)	E_{50}/k_{rm}
0 – 4.5	354.4 – 349.9	Silty Clay	1,000	115	2600	--	0.005
4.5 - 7	349.9 – 347.4	Clay	500	60	1700	--	0.007
7 – 34.5	347.4 – 319.9	Silty Clay to Clay	30	55	470	--	0.020
34.5 – 39.5	319.9 – 314.9	Clay	100	65	1,050	--	0.007
39.5 – 44.5	314.9 – 309.9	Sandy Gravel	125	45	--	37	--
44.5 – 54.5	309.5 – 299.9	Clay	100	75	1,100	--	0.007
54.5 – 59.5	299.9 – 294.9	Silty Sand	20	50	--	32	--
59.5 – 64.5	294.9 – 289.9	Sand with trace gravel	60	50	--	35	--
Below 64.5	Below 289.9	Clayey Shale over Coal	--	130	5,000	--	0.0005

APPENDIX D

PROJECT: Bridge Replacement – IL 13 over Big Muddy **SCI No.** 2008-3043.50
LOCATION: Jackson County, Illinois
CLIENT: Oates Associates, Inc.
STRUCTURE: 039-0013 (EB), 039-0049 (WB)

Table D.3 – Soil Modulus Parameters (k) for 3-S

Depth (ft)	Elevation (ft)	Abbreviated Soil Description	Soil Modulus Parameter (pci)	Effective Unit Weight (pcf)	Cohesion (psf)	Phi (degrees)	E_{50}/k_{rm}
0 – 3	353.7 – 350.7	Silty Clay	2,000	115	4,500	--	0.005
3 - 7	350.7 – 346.7	Silty Clay	1,000	60	2,500	--	0.010
7 – 32	346.7 – 321.7	Clay to Silty Clay	100	60	550	--	0.020
32 – 34.5	321.7 – 319.2	Silty Sand	20	50	--	32	--
34.5 – 37	319.2 – 316.7	Sandy Gravel	60	45	--	37	--
37 – 39.5	316.7 – 314.2	Sandy Gravel	20	45	--	37	--
39.5 – 59.5	314.2 – 294.2	Clay to Silty Clay	100	65	780	--	0.020
59.5 – 64.5	294.2 – 289.2	Clay	100	60	600	--	0.020
Below 64.5	Below 289.2	Coal	--	130	5,000	--	0.0005

Table D.4 – Soil Modulus Parameters (k) for 4-S

Depth (ft)	Elevation (ft)	Abbreviated Soil Description	Soil Modulus Parameter (pci)	Effective Unit Weight (pcf)	Cohesion (psf)	Phi (degrees)	E_{50}/k_{rm}
0 – 4.5	351.7 – 347.2	Silty Clay	500	115	1,200	--	0.007
4.5 - 7	347.2 – 344.7	Silty Clay	30	115	400	--	0.020
7 – 32	344.7 – 319.7	Silty Clay to Clay	30	60	450	--	0.020
32 – 54.5	319.7 – 297.2	Silty Clay and Clay	100	65	970	--	0.010
54.5 – 63	297.2 –288.7	Silty Sand	60	50	--	35	--
Below 63	Below 288.7	Coal	--	130	5,000	--	0.0005

APPENDIX D

PROJECT: Bridge Replacement – IL 13 over Big Muddy **SCI No.** 2008-3043.50
LOCATION: Jackson County, Illinois
CLIENT: Oates Associates, Inc.
STRUCTURE: 039-0013 (EB), 039-0049 (WB)

Table D.5 – Soil Modulus Parameters (k) for 5-S

Depth (ft)	Elevation (ft)	Abbreviated Soil Description	Soil Modulus Parameter (pci)	Effective Unit Weight (pcf)	Cohesion (psf)	Phi (degrees)	E_{50}/k_{rm}
0 – 4.5	351.6 – 347.1	Silty Clay	500	115	1,600	--	0.007
4.5 - 7	347.1 – 344.6	Silty Clay	500	60	1,200	--	0.007
7 - 32	344.6 – 319.6	Silty Clay to Clay	30	65	500	--	0.020
32 – 44.5	319.6 – 307.1	Clay	500	65	1,025	--	0.010
44.5 – 49.5	307.1 – 302.1	Clay Loam	1000	65	2,600	--	0.005
49.5 – 54.5	302.1 – 297.1	Silty Sand	20	55	--	32	--
54.5 – 59.5	297.1 – 292.1	Clay	500	75	1,100	--	0.007
59.5 - 62	292.1 – 289.6	Sand with Gravel	125	55	--	35	--
Below 62	Below 289.6	Clayey Shale over Coal	--	130	5,000	--	0.0005

Table D.6 – Soil Modulus Parameters (k) for 6-S

Depth (ft)	Elevation (ft)	Abbreviated Soil Description	Soil Modulus Parameter (pci)	Effective Unit Weight (pcf)	Cohesion (psf)	Phi (degrees)	E_{50}/k_{rm}
0 – 9.5	383 – 373.5	Silty Clay	1,000	115	2,230	--	0.005
9.5 - 17	373.5 – 366	Silty Clay to Silty Clay Loam	500	115	1,230	--	0.007
17 – 25.5	366 – 357.5	Clay and Silty Clay	500	115	1,075	--	0.007
25.5 – 79.5	357.5 – 303.5	Clay and Silty Clay	100	65	920	--	0.010
79.5 – 92	303.5 – 291	Sand	60	55	--	35	--
Below 92	Below 291	Coal over Clayey Shale	--	130	5,000	--	0.0005

APPENDIX D

PROJECT: Bridge Replacement – IL 13 over Big Muddy **SCI No.** 2008-3043.50
LOCATION: Jackson County, Illinois
CLIENT: Oates Associates, Inc.
STRUCTURE: 039-0013 (EB), 039-0049 (WB)

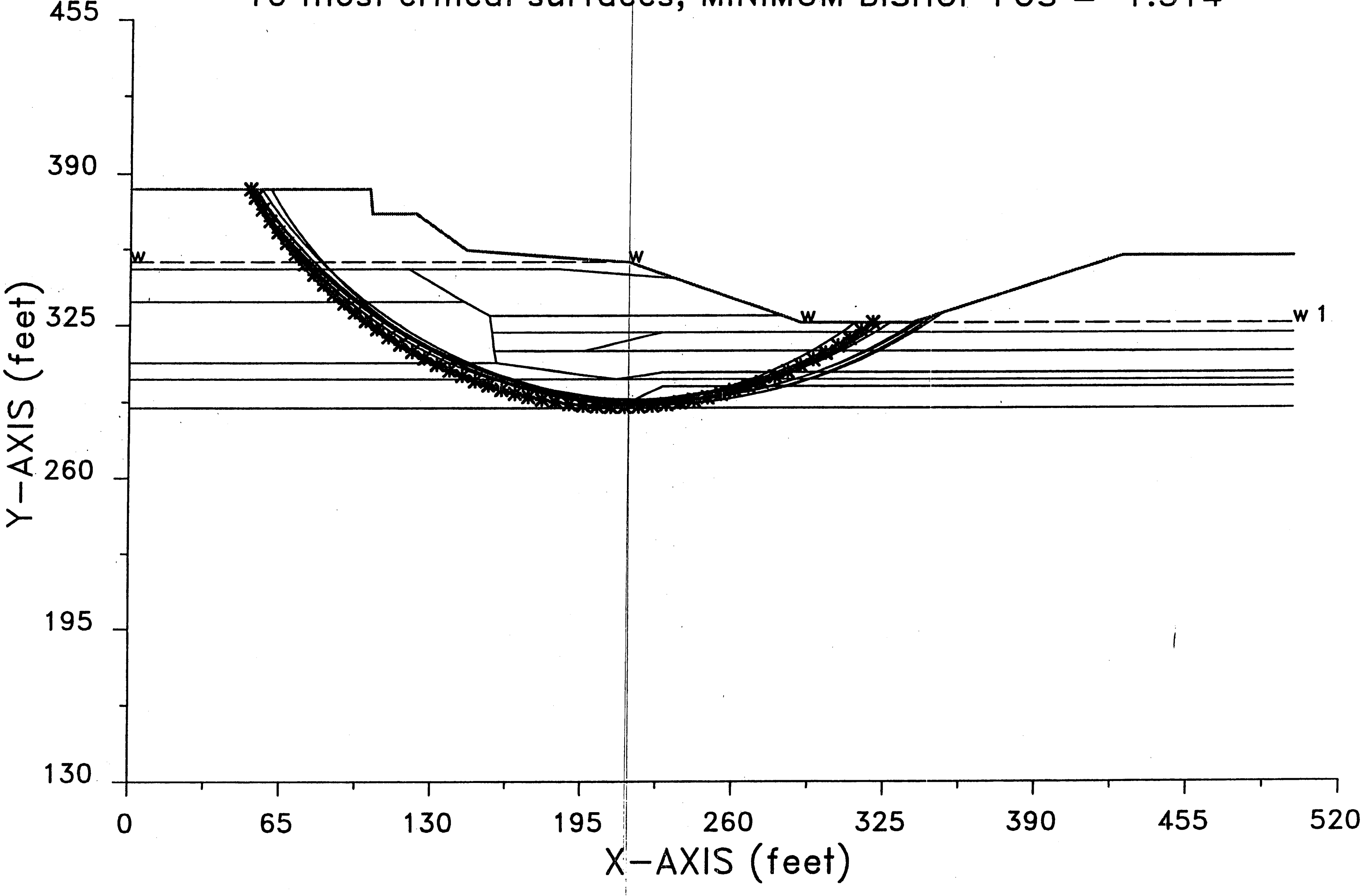
Table D.7 – Soil Modulus Parameters (k) for B-104

Depth (ft)	Elevation (ft)	Abbreviated Soil Description	Soil Modulus Parameter (pci)	Effective Unit Weight (pcf)	Cohesion (psf)	Phi (degrees)	E_{50}/k_{rm}
0 - 28	383 – 355	Fill - Silty Clay, Clay, Clayey Silt, Sandy Clay	1,000	125	2,250	--	0.005
28 - 40	355 – 343	Silty Clay and Clay	500	115	1,600	--	0.007
40 – 72	343 – 311	Silty Clay and Clay	100	65	800	--	0.010
72 – 91.5	311 – 291.5	Sand	125	55	--	35	--
Below 91.5	Below 291.5	Coal over Clayey Shale	--	130	5,000	--	0.0005

Appendix E

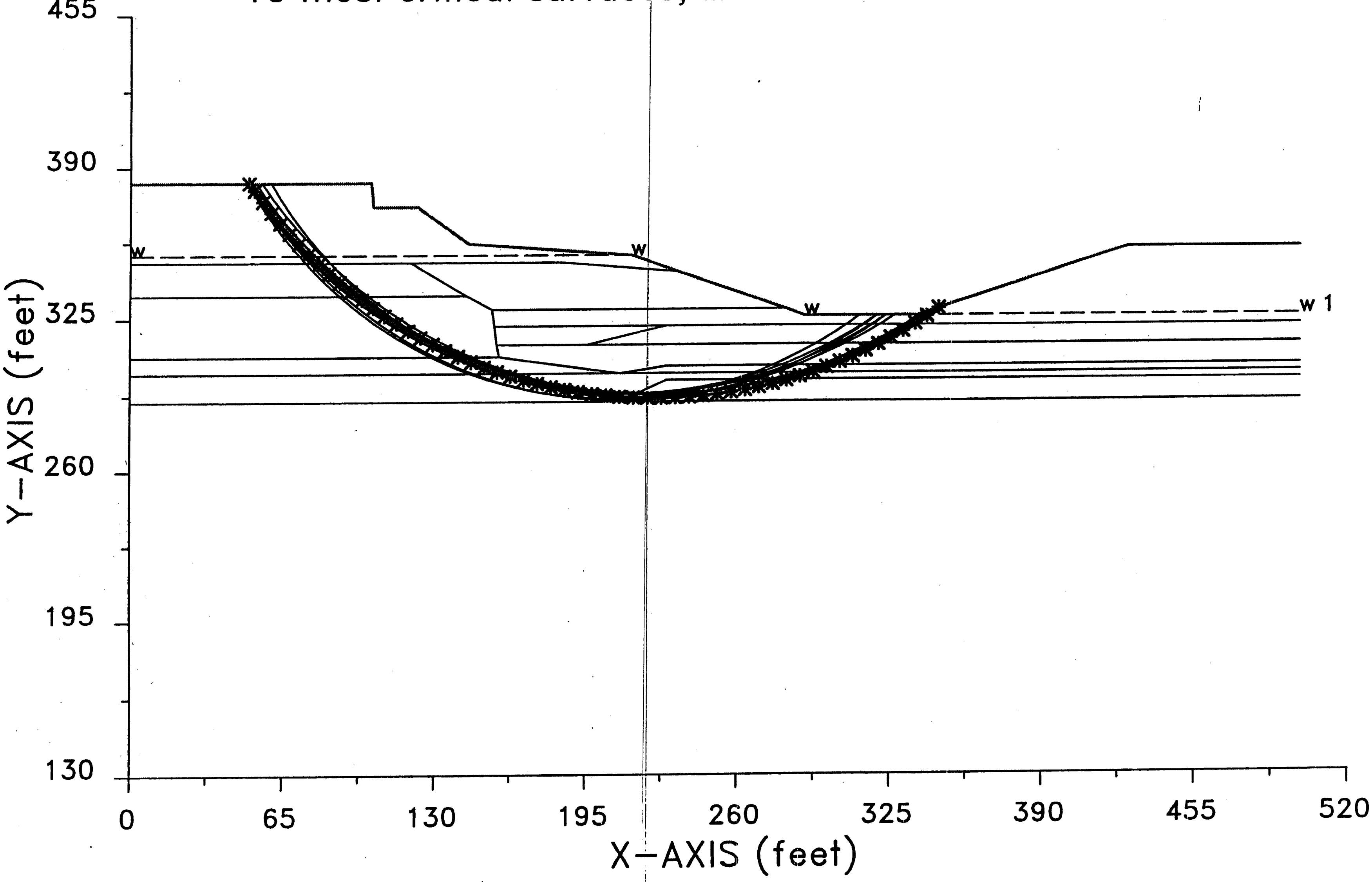
MUDDYEA1 4-22-** 7:45

II 13 over Big Muddy, E Abut, Tr #1
10 most critical surfaces, MINIMUM BISHOP FOS = 1.514



MUDDYE1Q 4-22-** 8:04

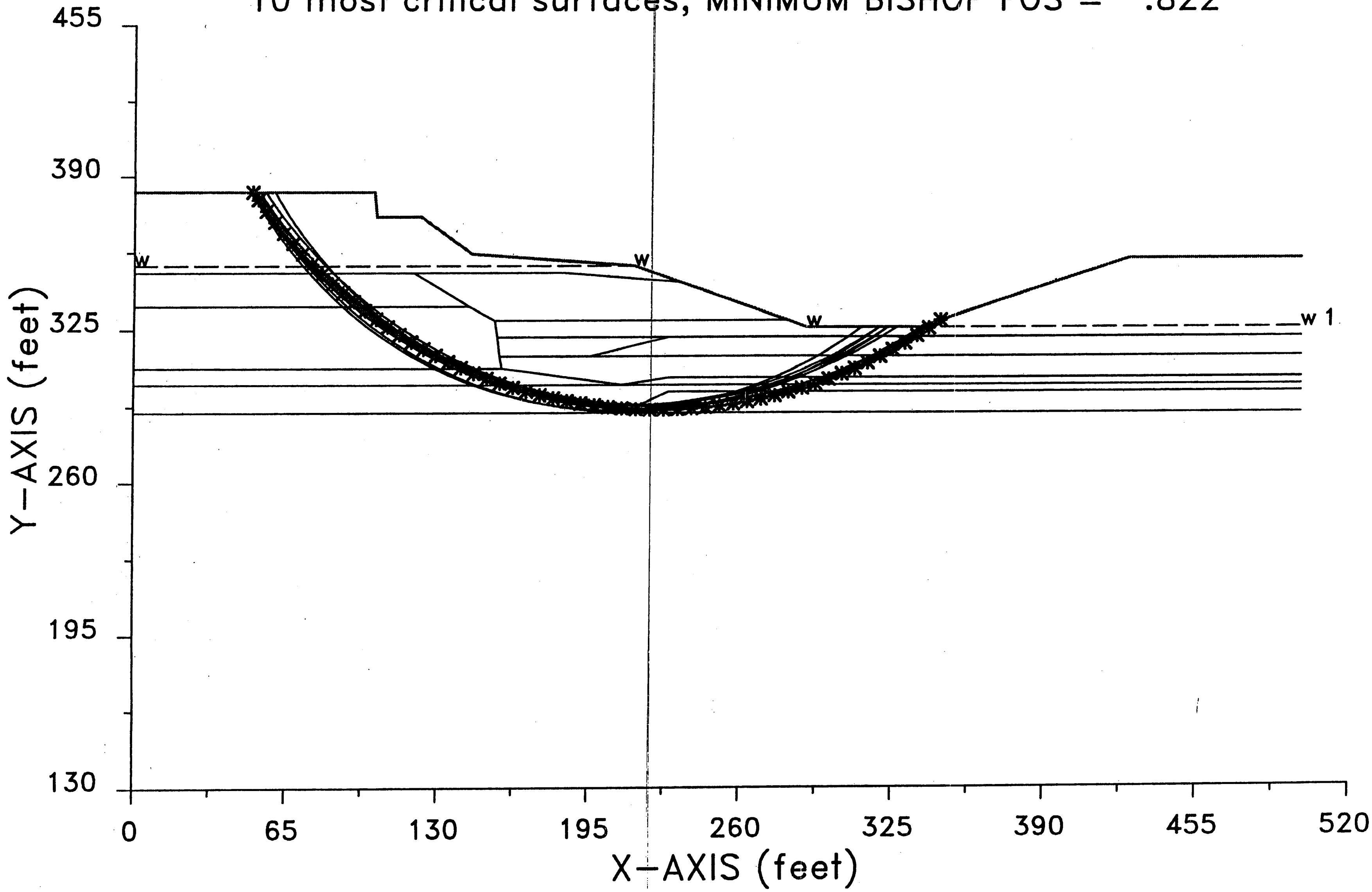
II 13 over Big Muddy, E Abut, w/1.3e
10 most critical surfaces, MINIMUM BISHOP FOS = .972



MUDDYEQ2 4-26-** 8:20

II 13 over Big Muddy, E Abut,.195eq

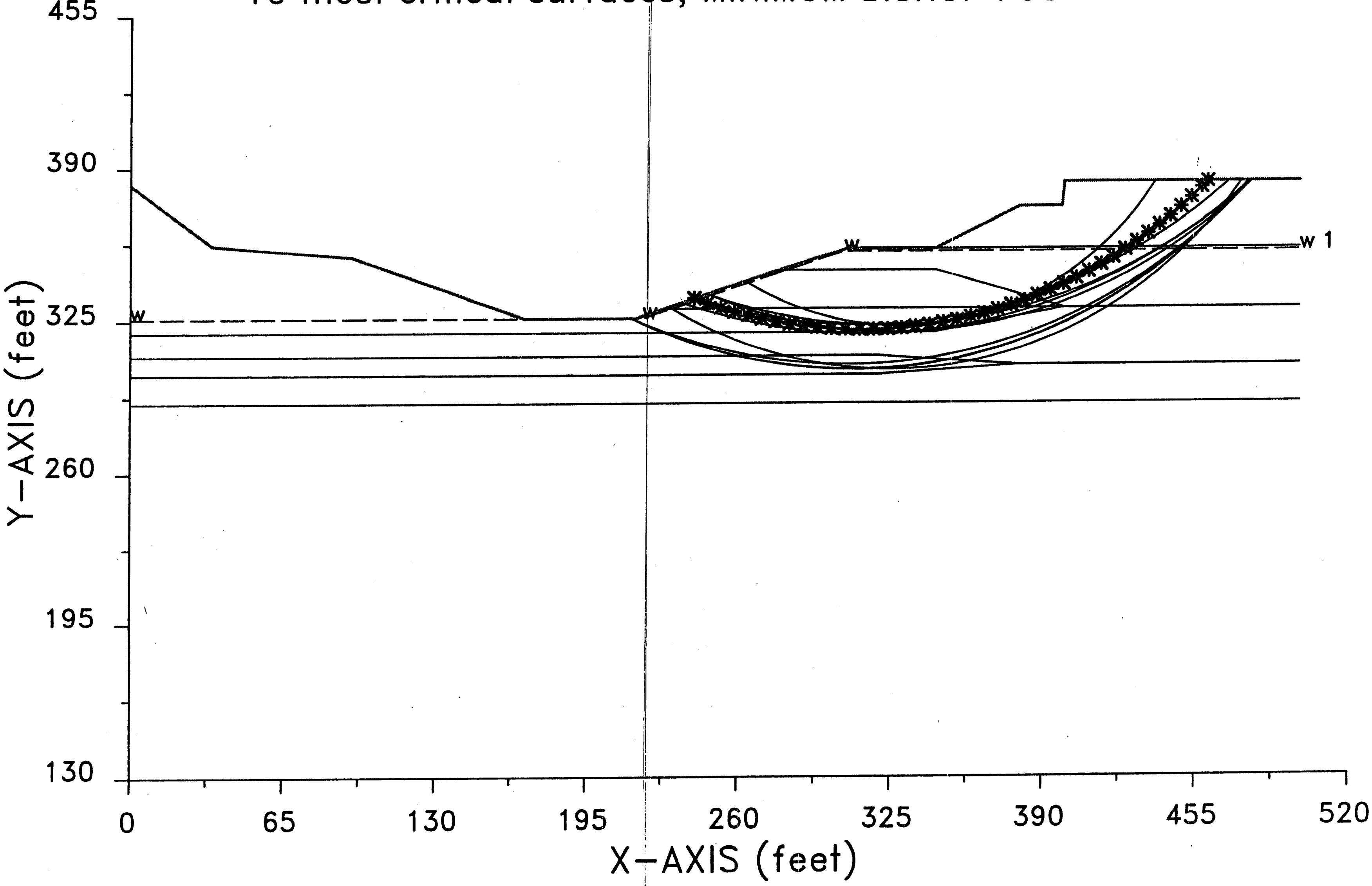
10 most critical surfaces, MINIMUM BISHOP FOS = .822



MUDDYWAN 4-22-** 10:31

IL 13 over Muddy River, W Abut, no E

10 most critical surfaces, MINIMUM BISHOP FOS = 1.268



IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes. While you cannot eliminate all such risks, you can manage them. The following suggestions and observations are offered to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical study is unique, each geotechnical report is unique, prepared *solely* for the client. *No one except you* should rely on your geotechnical report without first conferring with the geotechnical engineer who prepared it. *And no one—not even you*—should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Report Is Based on a Unique Set of Project-specific Factors

Geotechnical engineers consider a number of unique project-specific factors when establishing the scope of a study. Typical factors include the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and its configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical report include those that affect the:

- function and character of the proposed structure,
- elevation, configuration, location, orientation, or loading of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical report is based on conditions that existed at the time the study was performed. Do not rely on a geotechnical report whose adequacy may have been affected by the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions *only* at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an *opinion* about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective way of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Report is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk*.

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce such risks, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a geotechnical study. For that reason, a geotechnical report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Rely on Your Geotechnical Engineer for Additional Assistance

Membership in ASFE exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

The preceding paragraphs are based on information provided by ASFE.

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