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Final Structure Geotechnical Report

BRIDGE REPLACEMENT
FAP 626 (IL 97) OVER LITTLE HAW CREEK
KNOX COUNTY, ILLINOIS
PTB 151-34, WO 2
ROUTE: FAP 626 (IL97)
SECTION: 42-(B,B-1)BR-1
STRUCTURE NO. 048-0015 (EXISTING), 048-0097 (PROPOSED)

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March 25, 2014
Revised June 27, 2014

Prepared for:
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11/30/15

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THOMAS JOSEPH CASEY
062-061853
STATE OF ILLINOIS

6/27/14

SCI No. 2009-3119.51

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

June 27, 2014

Mr. Bruce Schopp, P.E., S.E.
Oates Associates, Inc.
100 Lanter Court, Suite 1
Collinsville, Illinois 62234

RE: Final Structure Geotechnical Report
Bridge Replacement
FAP 626 (IL 97) over Little Haw Creek
Knox County, Illinois
PTB 151-34, WO 2
Route: FAP 626 (IL 97)
Section: 42-(B,B-1)BR-1
Structure No: 048-0015 (Existing), 048-0097 (Proposed)
SCI No.: 2009-3119.51

Dear Mr. Schopp:

Enclosed is our Final *Structure Geotechnical Report (SGR)* dated March 2014, revised June 2014. 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.

Hobson H. Fizette, P.E.
Staff Engineer

Thomas J. Casey, P.E.
Senior Engineer

HHF/TJC/tlw

Enclosure

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Final Structure Geotechnical Report

BRIDGE REPLACEMENT
FAP 626 (IL 97) OVER LITTLE HAW CREEK
KNOX COUNTY, ILLINOIS
PTB 151-34, WO2
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SECTION: 42-(B,B-1)BR-1
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1.0 PROJECT DESCRIPTION

The geotechnical study summarized in this report was performed for the proposed replacement bridge to carry Illinois 97 over the Little Haw Creek near Gilson in rural Knox County, Illinois. The existing structure is a 2-lane, single-span structure (SN 048-0015) with an approximate length of 38 feet (back to back abutment) and an approximate width of 33 feet (out to out deck). The proposed replacement bridge (SN 048-0097) will consist of a 2-lane, single-span bridge, lengthened to approximately 77.3 feet (back to back abutment) and widened to approximately 35.2 feet wide (out to out deck). Based on the *preliminary Type, Size, and Location (TS&L) plan* provided by the Oates Associates, Inc. (Oates), the roadway profile of the new bridge will be raised approximately 1 to 2 feet from the current profile. The existing concrete abutments will be removed and the end-slopes will be cut back to a 2 horizontal to 1 vertical (2H:1V) slope. Based on the provided plans, it appears that staged construction will be required for construction of the new structure. The location of the site is shown on the *Vicinity and Topographic Map*, Figure 1.

2.0 SUBSURFACE EXPLORATION

2.1 Area Geology

Within the project area, the geology is made of unlithified materials consisting of loamy and silty soils that formed in loess (windblown silt deposits) over Illinoisan glacial till deposits (*Soil Survey of Knox County Illinois*, Natural Resources Conservation Service, 2005). These deposits generally overlie Pennsylvanian shale, and coal over Mississippian limestone.

2.2 Exploration Procedures

Two standard penetration test (SPT) borings, designated B-1 and B-2 were drilled near the proposed abutment locations, as shown on the *Site Plan*, Figure 2. Previously, two borings designated as 1 and 2 were drilled in 1979 near the existing abutments, and are included in Appendix A for information purposes. Detailed information regarding the nature and thickness of the soils and rock encountered, and the results of the field sampling and laboratory testing are shown in the appended Boring Logs.

The 2014 boring locations were selected by Oates and IDOT and staked by SCI personnel by measuring from existing site features. The 2014 boring locations were later surveyed by Coombe-Bloxdorf, P.C. and the stations, offsets, and elevations were provided to SCI. The field exploration was performed in general accordance with procedures outlined in the 1999 *IDOT Geotechnical Manual*.

Personnel from SCI were with the drill rig to supervise drilling, log the borings, and perform field unconfined compressive strength tests of the 2014 borings. A Mobile B-57 truck-mounted drill rig equipped with continuous flight augers was used to advance the borings. SPTs were performed with a split-spoon sampler at 2½-foot intervals to 30 feet, and at 5-foot intervals thereafter to the termination depth of the borings. The unconfined compressive strength of the cohesive soils was determined with a Rimac test apparatus. A pocket penetrometer was used to measure the compressive strength if the soils were not conducive to Rimac testing. The SCI borings were drilled to refusal per IDOT specifications to depths of 25 feet to 35 feet below the existing ground surface. While auger refusal did not occur in any of the borings, split spoon sampler refusal did occur within the shale layer in both borings, as detailed further in Table 2.1, and on the appended boring logs. Split-spoon sampler refusal is a designation applied to any material that results in SPT N-values in excess of 100 blows per foot (bpf).

Table 2.1 - Summary of Borings Drilled For Structure SN 048-0097

Boring	Type	Ground Surface Elevation (ft)	Refusal Depth (ft)	Refusal Elevation (ft)	Station	Offset	
B-1	North Abutment	606.6	35.0	571.6	536+66.98	12.0	RT
B-2	South Abutment	606.6	25.0	581.6	537+56.11	12.0	LT

2.3 Subsurface Conditions

Detailed information regarding the nature and thickness of the soils and rock encountered, 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 structure is shown on Figure 2. The generalized soil profiles are included on the subsurface profile, Figure 3.

Below the surficial 3 to 4 inches of asphalt, fill material, extending to depths of approximately 5.5 feet (El. 601.1) was observed in both borings. The fill consisted of silty clay loam (A-6 in accordance with the AASHTO soil classification system, based on our visual classification unless lab tests were noted on the logs) and was most likely associated with the construction of the existing abutments.

Beneath the fill soils, natural cohesive soils, consisting of silty clay loam (A-6), clay loam (A-6) and shaley clay (A-7), were encountered to depths of approximately 14.5 to 15.5 feet (El. 592.1 to 591.1). A relatively thin layer of gravel (A-1) was observed in boring B-1 at a depth of 14.5 to 15.5 feet (El. 592.1 to 591.1) beneath the upper cohesive soils. In general, the natural cohesive soils were soft to medium stiff in consistency with N-values (the sum of the second and third blow count numbers in each sampling interval from the SPT) of 3 to 10 blows per foot (bpf) with an average of 5 bpf, and unconfined compressive strengths obtained from Rimac from 0.2 to 1.5 tons per square foot (tsf) with an average of 0.7 tsf. Moisture contents of these soils ranged from 21 to 31 percent and averaged 26 percent.

Beneath the upper cohesive soils, clayey shale was encountered in both borings until boring termination depths of 25 to 35 feet (El. 581.6 to 571.6). A relatively thin layer of coal was observed in boring B-1 at a depth of 26.0 to 26.5 feet (El. 580.6 to 580.1) within the clayey shale. SPT N-values varied within the shale and ranged from 13 to 100 bpf. Due to the weakness of the shales in the area, a modified standard penetration test (MSPT) was performed within the shale layer in general accordance with the Illinois Center for Transportation report ICT-R27-99 that was performed for IDOT. An MSPT value of 107 blows per foot, and an equivalent unconfined compressive strength of 4.1 tsf were measured at a depth of 23.5 feet (El. 583.1) within the clayey shale layer in boring B-2.

Table 2.2 presents a summary of the depth and elevation that shale was first encountered in each of the SCI borings. We defined intact shale bedrock as the point of the first split spoon sampler refusal.

Table 2.2 – Summary of Shale Elevations

Boring	Depth to Shale (ft)	Top of Shale Elevation (ft)
B-1	22.0	584.6
B-2	22.0	584.6

2.4 Groundwater Conditions

Groundwater levels observed at the time of drilling are summarized in Table 2.3. It should be noted that the groundwater level is subject to seasonal and climatic variations, the water level in Little Haw Creek, and other factors; and may be present at different depths in the future. In addition, without extended periods of observation, measurement of the true groundwater levels may not be possible.

Table 2.3 – Summary of Approximate Groundwater Levels

Boring No.	Groundwater Elevation During Drilling (ft)
B-1	592.6
B-2	594.6

3.0 GEOTECHNICAL EVALUATIONS

In order to provide design recommendations for founding the structures, we performed the following evaluations based on all available data collected and reviewed at the time of this report. This information includes subsurface explorations performed by SCI, preliminary TS&L plans, and communications with Oates personnel familiar with the project. The preliminary TS&L is attached to the SGR in Appendix E.

3.1 Seismic Considerations

3.1.1 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 low plastic silt deposits below the groundwater table. Liquefaction occurs when a rapid development 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 2012 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 7.7 and a Peak Ground Acceleration (PGA) of 0.07g, as determined from data provided by the United States Geological Survey (USGS) National Seismic Hazard Mapping Project and procedures outlined in the All Geotechnical Manual Users (AGMU) 10.1, *Liquefaction Analysis Procedure, dated February 25, 2010*.

3.1.2 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 B and C, which include evaluating the SPT N-values and undrained shear strength, S_u. The following results were calculated:

- Method B using N: 77 bpf (Site Class C)
- Method C using N_{ch}: 95 bpf (Site Class C)
- Method C using S_u: 1,500 psf (Site Class D)

Based on the guidelines in the AGMU, we recommend that Site Class C be used for the project. Based on Table 3.15.2-1, the Seismic Performance Zone is 1. Seismic design parameters for the site are summarized in Table 3.1.

Table 3.1 – Seismic Design Parameters

Seismic Design Parameters	
Site Class	C
F_a	1.20
F_v	1.70
Design Spectral Acceleration at 0.2 sec. (S_{DS})	0.12g
Design Spectral Acceleration at 1.0 sec.(S_{D1})	0.07g
Seismic Performance Zone	Zone 1

3.1.3 Liquefaction Potential Analysis

Based on the techniques outlined in AGMU 10.1, a liquefaction potential analysis is not required for the site. As no liquefaction potential was calculated for the site, the effects of liquefaction on axial pile capacity are neglected.

3.2 Abutment Settlement

Based on the provided TS&L, and discussions with Oates, elevation changes on the order of 1.0 to 2.0 feet are anticipated at the abutments. Due to the minor grade changes, a rigorous settlement analysis was not performed for the abutment soils. Therefore, the effects of down drag on axial pile capacity are neglected.

3.2.1 Embankment Approaches

Based on the provided plans, the embankment approach side slopes will also be widened. Existing slopes steeper than 5H:1V should be benched to provide a level surface prior to placing any new fill material. Benching will provide level surfaces for compaction and reduce the development of inclined planes of potential weakness between the existing soil and the fill material. We recommend the benches be spaced such that the maximum height of cut at the up-slope end of the bench is 5 feet. Should soft or loose soils be encountered during construction, SCI should be retained to review our analyses and recommendations.

3.3 Bridge Approach Slabs

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).

3.4 Slope Stability

SCI conducted slope stability analyses of the end slopes for the new bridge abutments. Based on the proposed plans, the side and end-slopes will be cut to inclinations of approximately 2H:1V. The slope stability analyses for the slopes were conducted using limit equilibrium slope stability methods and the commercially available software program Slope/W (part of the GeoStudio 2012 software package developed by Geo-Slope International). A Morgenstern-Price analysis was used to search for a critical circular failure surface to calculate the factor of safety for the slope. For the analysis, the engineering soil properties from the subsurface exploration data and the given slope geometries were used. The project was evaluated using traditional Allowable Stress Design analyses using Factors of Safety (FS) values presented in the Bridge Manual.

The slopes were evaluated using short-term and long-term loading conditions. A traffic load of 250 pounds per square foot (psf) was used during the analyses. For the static, long-term slope stability analyses, effective stress values were used in a simplified soil profile developed for the bridge embankments and the failure surfaces were limited to the end slopes below the proposed structure.

For the short-term analyses, total stress values were used. In each case, the embankments achieved the minimum factors of safety for the static conditions, as detailed in Table 3.2. The individual output graphics from the analyses are presented in the report Appendix C.

Table 3.2 – Summary of Slope Stability Factors of Safety

Location	End of Construction		Long Term	
	Required Minimum Factor of Safety	Estimated Factor of Safety	Required Minimum Factor of Safety	Estimated Factor of Safety
North Abutment End Slope STA 536+64.33	1.7	2.4	1.7	1.7
South Abutment End Slope STA 537+35.67	1.7	1.9	1.7	1.7

Based on the Seismic Performance Zone 1, and given the design nature of the structure, seismic slope stability analyses were not performed.

3.5 Scour

Abutment foundations are an area of primary concern for damage from scour. Per IDOT Bridge Manual Section 2.3.6.3.2, open abutments protected with class A5, stone dumped riprap, should set the design scour elevation at the bottom of the abutment. Based on the Bridge Manual, and the provided TS&L, the design scour elevations for the 100-year and 500-year events for the abutments are shown in Table 3.3 below.

Table 3.3 – Summary of Design Scour Elevation

	Event	North Abutment	South Abutment
Design Scour Elevation (ft)	**Q100**	600.8	601.1
	Q500	600.8	601.1

3.6 Mining Activity

Based on the Illinois Coal Resource Shapefile GIS data provided by the Illinois State Geological Survey, dated July 2012, the site is not undermined. In addition, the subject site is approximately 2 miles away from the nearest mapped mine. The listed disclaimer in the Directory states, "Locations of some features on the mine maps may be offset by 500 or more feet due to errors in the original source maps, the compilation process, digitizing, or a combination of these factors." Based on the distance to the nearest mapped underground mine, 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. Preliminary structure loads are provided in Table 3.4 below. Several potential foundation options were considered for supporting the new bridge structure that included driven steel H-Piles, metal shell piles, drilled shafts, and shallow foundations. Metal shell piles are not recommended because the estimated tip elevations are very close to bedrock, which can cause unacceptable risks for pile damage. Shallow foundations are not recommended due to the relatively soft consistency of the shallow subsurface conditions encountered, unless the bottoms of the footings are founded in rock; which would likely result in costly foundation treatment due to the excessive foundation depth. Drilled shaft foundations were determined to be too costly, given the size of the proposed structure, and would also not be compatible with the proposed integral abutments. If the abutments change from an integral abutment to semi-integral abutments, drilled shafts would be a geotechnically feasible foundation option. SCI should be contacted for additional recommendations if drilled shafts will be considered.

For the driven steel H-pile foundation option, we recommend a minimum of two test piles be installed to verify the length of the piles. One test pile should be installed at each abutment to help determine the pile length. Recommendations for all the potential foundation options are provided below.

Table 3.4 – Preliminary Structure Loads

Location	Service I Reaction (kips)	Strength I Reaction (kips)
South Abutment	800	1,100
North Abutment	800	1,100

3.7.1 Driven Steel Piles

The structural capacity of driven piles depends on the allowable stress and cross sectional areas of steel. The pile recommendations in this report assume that Steel H-piles will conform to AASHTO M270 Grade 50 (ASTM 709 Gr 50) or equivalent with a minimum yield stress of 50 kips per square inch (ksi).

Based on the most current IDOT Bridge Manual, All Geotechnical Manual User Memorandums (AGMUs), and Guide Bridge Special Provisions (GBSP), a geotechnical resistance factor (φ_G) of 0.55 was used for the design of the driven pile foundations. As liquefaction and settlement are not concerns at the site, geotechnical losses due to liquefaction and down-drag were not considered necessary in the static or seismic pile design. Geotechnical losses associated with scour were not considered since piers are not being proposed, and it is anticipated that scour will be reduced to above the proposed soil surface by using class A5 riprap at the abutments. During the seismic event the Bridge Manual allows the use of a Geotechnical Resistance Factor (φ_G) of 1.0.

All estimates of capacity were calculated using the "Modified IDOT Static Method" spreadsheet associated with the IDOT Bridge Manual, and appropriate AGMUs and GMSPs, and assume construction verification will follow the "WSDOT" formula outlined in Section 512 of the most current IDOT Standard Specifications for Road and Bridge construction. The top elevations of the piles obtained from the TS&L were 602.8 and 603.1, while the ground surface elevation during driving was assumed to be 600.8 and 601.1 for the north and south abutments, respectively. The tip elevations were calculated from the Modified IDOT Static Method spreadsheets based on the available factored resistance.

We recommend a minimum driven pile center to center spacing of three pile diameters, as recommended by the IDOT Bridge Manual. 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.

A summary of the design capacities, or factored resistance available (R_F), seismic factored resistance (R_{Fseis}), and nominal required bearing (R_N) is presented in Appendix F for each H-pile size. The pile lengths, as shown in Appendix F, were estimated from the embedment depth estimates from the IDOT design spreadsheet and the top elevations estimated from the preliminary TS&L plan. Based on the criteria established in the All Bridge Designers Memorandum (ABD) 12.3, the following H-Pile sizes are suitable for the proposed integral abutments: HP8x36, HP10x42, HP10x57, HP12x53, HP12x63, HP12x74, HP12x84, HP14x73, HP14x89, HP14x102, and HP14x117.

Estimated maximum refusal elevations, based on the IDOT pile capacity analyses, for H-piles are included in Appendix F. It should be noted that H-piles driven into shale may run shorter than the IDOT spreadsheet predicts. The estimated pile lengths should be adjusted based on the test pile results.

3.8 Wingwalls

The wingwalls should be designed to withstand lateral earth pressures caused by the weight of the backfill, including slopes behind the walls. We recommend the equivalent fluid unit weights tabulated below for lateral earth pressures, in pounds per cubic foot, be used in the design of the wingwalls.

The indicated values assume that positive drainage is provided to prevent the development of hydrostatic pressure. Values for granular material should only be used if the granular backfill extends upwards and outwards the full height of the wall at a slope of 45 degrees or flatter from its base. In this case, the granular backfill should be capped with approximately 2 feet of cohesive soil to reduce the potential for surface water infiltration into the granular backfill. With clean granular backfill, filter fabric, such as Mirafi 140N or equivalent, should be placed along the interface between the soil and the granular backfill to reduce the potential for infiltration of the soil into the granular material.

Table 3.5 – Recommended Lateral Earth Pressures – Level Surface

Backfill Type	Equivalent Fluid Unit Weights	
	At-Rest Earth Pressures (pcf)	Active Earth Pressures (pcf)
Cohesive Soil	70	50
Granular Material (1-inch minus)	60	40
Free-Draining Granular Material (1-inch clean)	50	30

The above values are applicable when the surface of the backfill behind the wall is horizontal. In areas where an upward sloped or loaded backfill case occurs, additional pressures will need to be added. If the final design includes upward sloped backfills, SCI should be retained to review our recommendations.

3.9 Lateral Pile Response

A representation of the shaft response under lateral loading exceeding 3 kips per pile is required for design of the bridge superstructure per Section 3.10.1.10 of the 2012 Bridge Manual. 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 borings B-1 and B-2 summarizing approximate soil modulus parameters (k) for the LPILE analyses are included in Appendix D (Reference: LPILE User's Manual, Ensoft, Inc., July 2004). Soils located above the 500-year design scour elevation (Q500) should not be considered during analysis. When pile/shaft design details and load information are refined in the development of the structure plans, LPILE analyses, if warranted, can be performed.

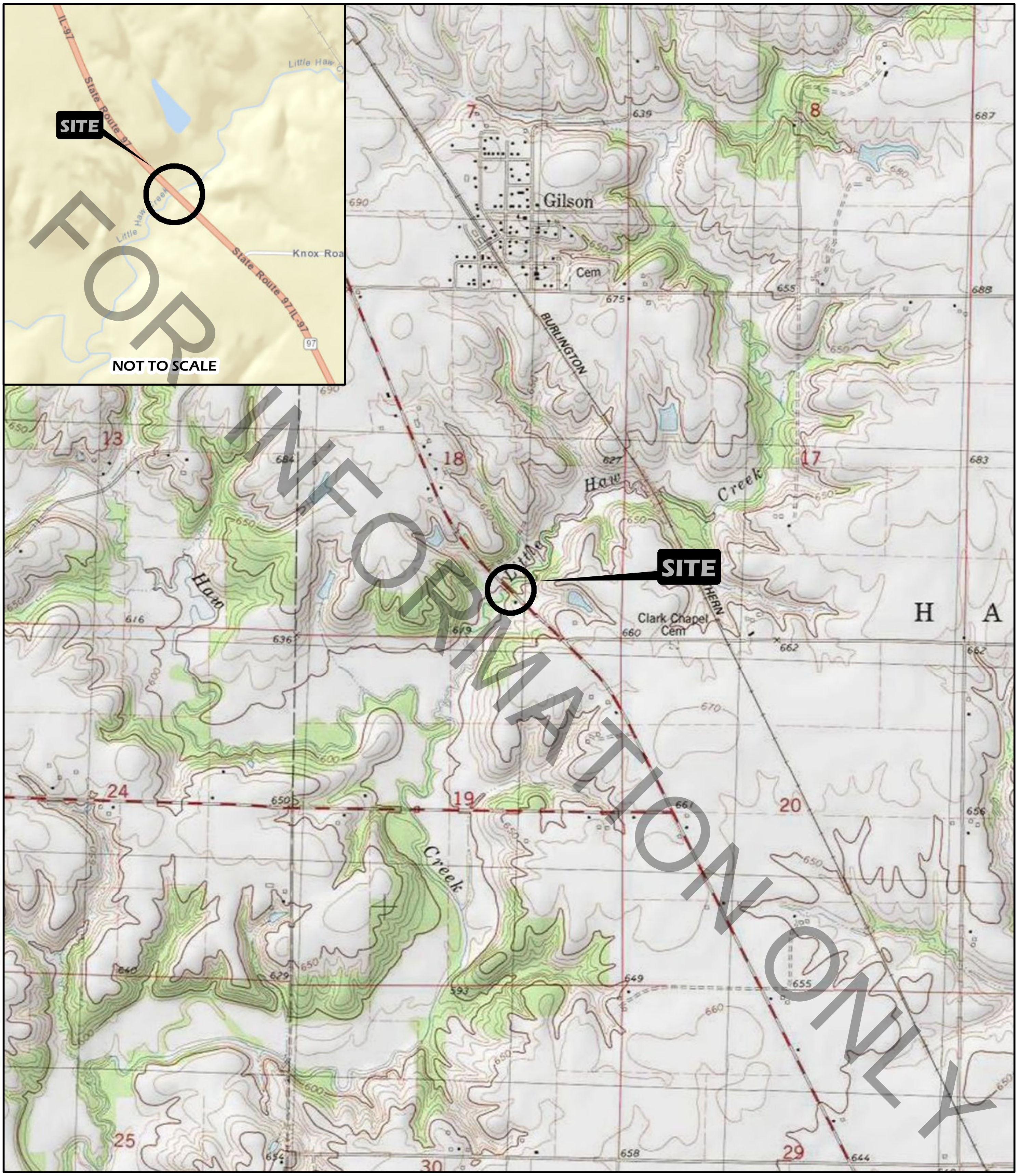
4.0 CONSTRUCTION CONSIDERATIONS

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.

Based on the plans provided, staged construction will be required for the construction of the new structure. It appears that either temporary sheeting, including cantilever temporary sheet piling, or a soil retention system, will be feasible on the both the north and south abutments. Based on the provided plans and discussions with Oates personnel familiar with the project, temporary sheeting will only be required immediately behind the proposed new abutments, and will be embedded into the existing roadway embankment. A maximum retained height of 8.0 feet, to facilitate pile installation and abutment construction, was used in our analyses. For temporary sheeting, a minimum embedment depth of 9.0 feet with a minimum section modulus of 5.1 cubic inches per foot should be used for planning purposes. However, if the soil retention system will be extended from the back of the existing abutment to the back of the new abutments, temporary cantilever sheet piling may not be feasible, and a different type of soil retention system may be required.

5.0 LIMITATIONS

The recommendations provided herein are for the exclusive use of Oates Associates, Inc and IDOT. They are specific only to the project described, and are based on subsurface information obtained at two 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.



SCI

PROJECT NAME
BRIDGE REPLACEMENT
FAP 626 (IL 97) OVER LITTLE HAW CREEK
KNOX COUNTY, ILLINOIS

VICINITY AND TOPOGRAPHIC MAP

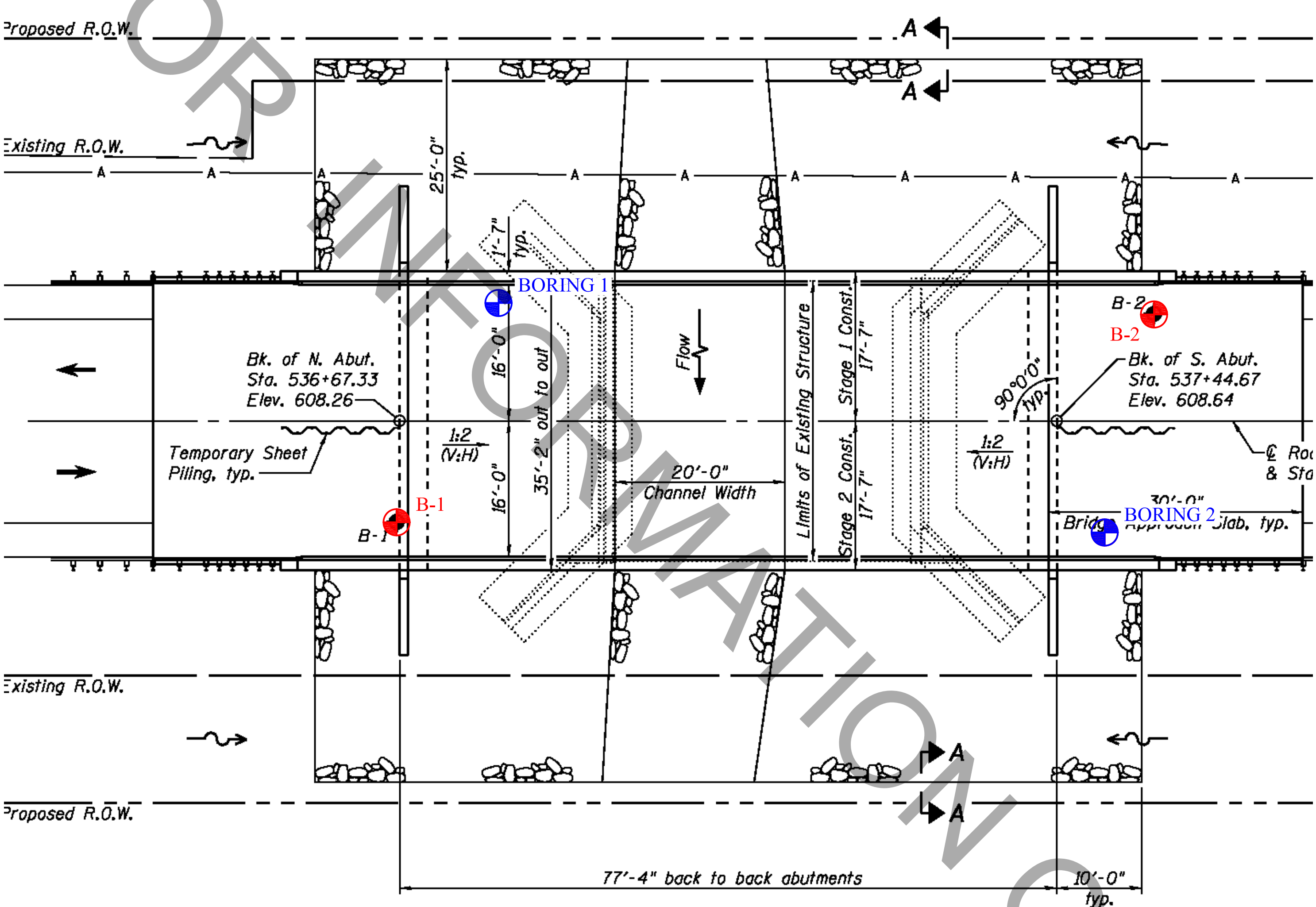
DRAWN BY	RCV	DATE	JOB NUMBER
CHECKED BY	HHF	06/2014	2009-3119.51

GENERAL NOTES/LEGEND
USGS TOPOGRAPHIC MAP
MAQUON, ILLINOIS QUADRANGLE
DATED 1982
10' CONTOURS

N
W E
S

SCALE 1" = 2000'

FIGURE 1



PROJECT NAME
BRIDGE REPLACEMENT
FAP 626 (IL 97) OVER LITTLE HAW CREEK
KNOX COUNTY, ILLINOIS

SITE PLAN

DRAWN BY	RCV	DATE	JOB NUMBER
CHECKED BY	HHF	06/2014	2009-3119.51

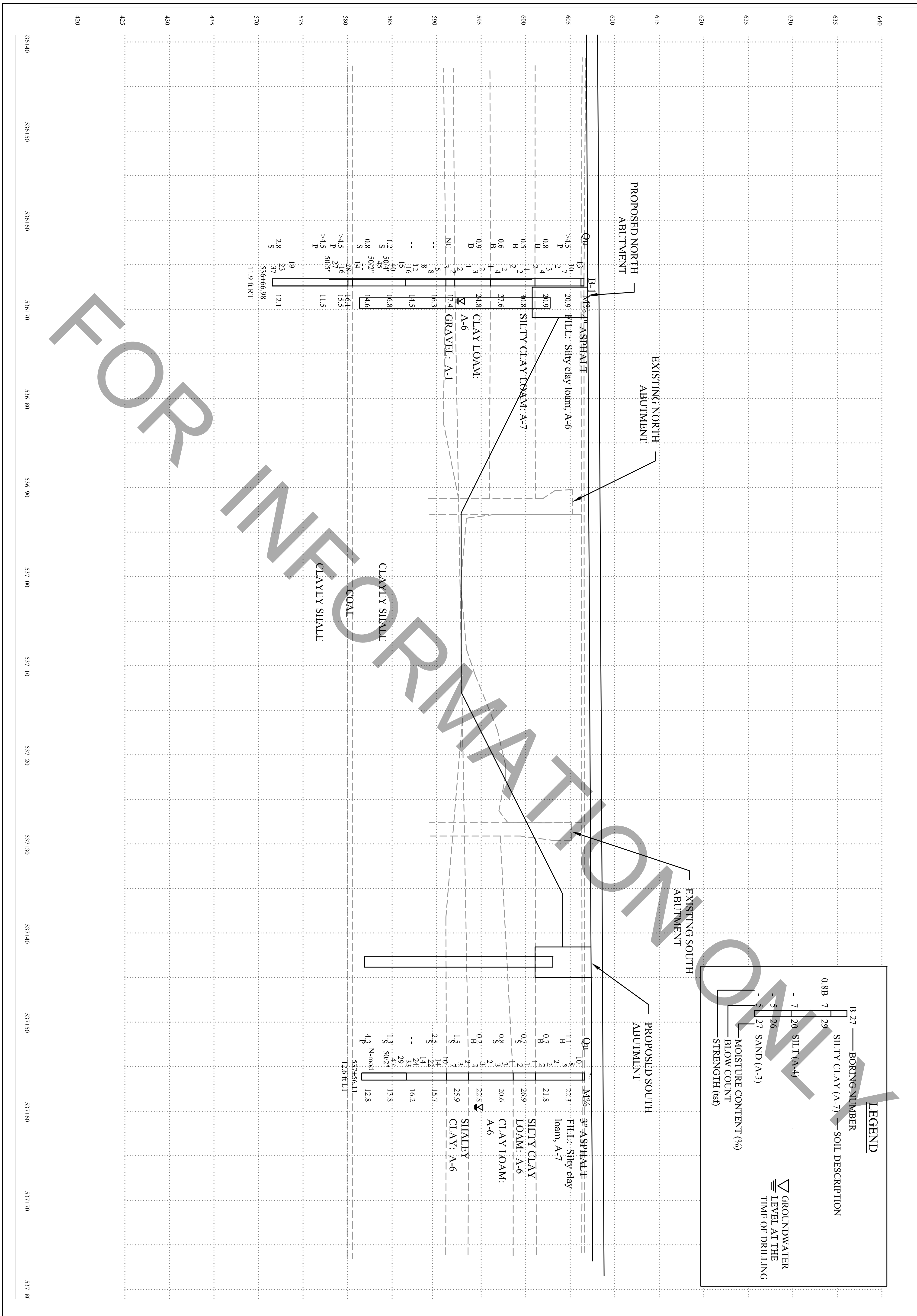
General Notes/Legend

INDICATES APPROXIMATE SOIL BORING LOCATIONS

INDICATES APPROXIMATE SOIL BORING LOCATIONS, DRILLED 1979

BASED ON UNDATED PLAN PROVIDED ELECTRONICALLY ON 03/17/2014 FROM OATES ASSOCIATES.
DIMENSIONS AND LOCATIONS ARE APPROXIMATE; ACTUAL MAY VARY. DRAWING SHALL NOT BE USED OUTSIDE THE CONTEXT OF THE REPORT FOR WHICH IT WAS GENERATED.

SCALE 1" = 20'
FIGURE 2



PROJECT NAME

BRIDGE REPLACEMENT
FAP 626 (IL 97) OVER LITTLE HAW CREEK
KNOX COUNTY, ILLINOIS

SUBSURFACE PROFILE

General Notes/Legend

VARIATIONS IN SUBSURFACE CONDITIONS MAY AND LIKELY EXIST BETWEEN BORINGS. DASHED HORIZONS ARE INTERPRETED AND ARE SHOWN FOR ILLUSTRATION ONLY.

VARIATIONS IN SUBSURFACE CONDITIONS MAY AND LIKELY EXIST BETWEEN BORINGS. DASHED HORIZONS ARE INTERPRETED AND ARE SHOWN FOR ILLUSTRATION ONLY.

SCALE	1" = 5' V 1" = 5' H
JOB NUMBER	2009-3119.51
DATE	06/2014
DRAWN BY	RCV
CHECKED BY	HHF
FIGURE	3

Appendix A

Illinois Department of Transportation
Division of Highways
SCI Engineering inc

SOIL BORING LOG

Date 1/31/14

ROUTE FAP 626 **DESCRIPTION** IL 97 over Little Haw Creek Structure Boring, North Abutment **LOGGED BY** SCI (MGS)

SECTION 42-(B,B-1)BR-1 **LOCATION** SW 1/4 of the SE 1/4, **SEC.** 18, **TWP.** 10N, **RNG.** 3E, 4th **PM**, **Latitude** , **Longitude**

COUNTY Knox **DRILLING METHOD** CFA **HAMMER TYPE** Automatic

STRUCT. NO. 048-0015 (EX) 048-0097 (PR)
Station 537+10

BORING NO. B-1
Station 536+66.98
Offset 11.9 ft RT
Ground Surface Elev. 606.6 ft

Surface Water Elev. N/A ft
Stream Bed Elev. N/A ft

Groundwater Elev.:
First Encounter 592.6 ft ▼
Upon Completion -- ft
After N/A **Hrs.** N/A ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
4" ASPHALT	606.2				
FILL: Brown and gray, silty clay loam, A-6			13 10 7	>4.5 P	21
		-5	2 3 4	0.8 B	21
SILTY CLAY LOAM: Dark gray, with iron stains, A-7	601.1		2 1 2	0.5 B	31
		-10	2 2 4	0.6 B	28
CLAY LOAM: Dark gray, with iron stains, trace coal and roots, A-6	596.1		1 2 3	0.9 B	25
▼		-15	1 2 2	NC	17
GRAVEL: Gray, with clay and fine to coarse sand, A-1	592.1				
CLAYEY SHALE: Light gray, trace fine sand	591.1		3 5 8	- -	16
Trace gravel	586.6	-20	8 12 16	- -	14

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
CLAYEY SHALE: Light gray, trace gravel and fine sand *(continued)*					
With sandy shale layers			15 40 50/4"	1.2 S/10	17
		-25	45 50/2"	0.8 S/10	15
Very hard drilling observed	580.6				
COAL: Black	580.1				
CLAYEY SHALE: Gray, trace gravel and fine sand			14 28 16	- - >4.5 P	16 15
		-30	27 50/5"	>4.5 P	12
	571.6	-35	19 23 37	2.8 S/10	12
Boring terminated at 35.0 ft.					
		-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 inc

SOIL BORING LOG

Date 1/30/14

ROUTE FAP 626 **DESCRIPTION** IL 97 over Little Haw Creek Structure Boring, South Abutment **LOGGED BY** SCI (MGS)

SECTION 42-(B,B-1)BR-1 **LOCATION** SW 1/4 of the SE 1/4, **SEC.** 18, **TWP.** 10N, **RNG.** 3E, 4th **PM**, **Latitude** , **Longitude**

COUNTY Knox **DRILLING METHOD** CFA **HAMMER TYPE** Automatic

STRUCT. NO. 048-0015 (EX) 048-0097 (PR)
Station 537+10

BORING NO. B-2
Station 537+56.11
Offset 12.6 ft LT
Ground Surface Elev. 606.6 ft

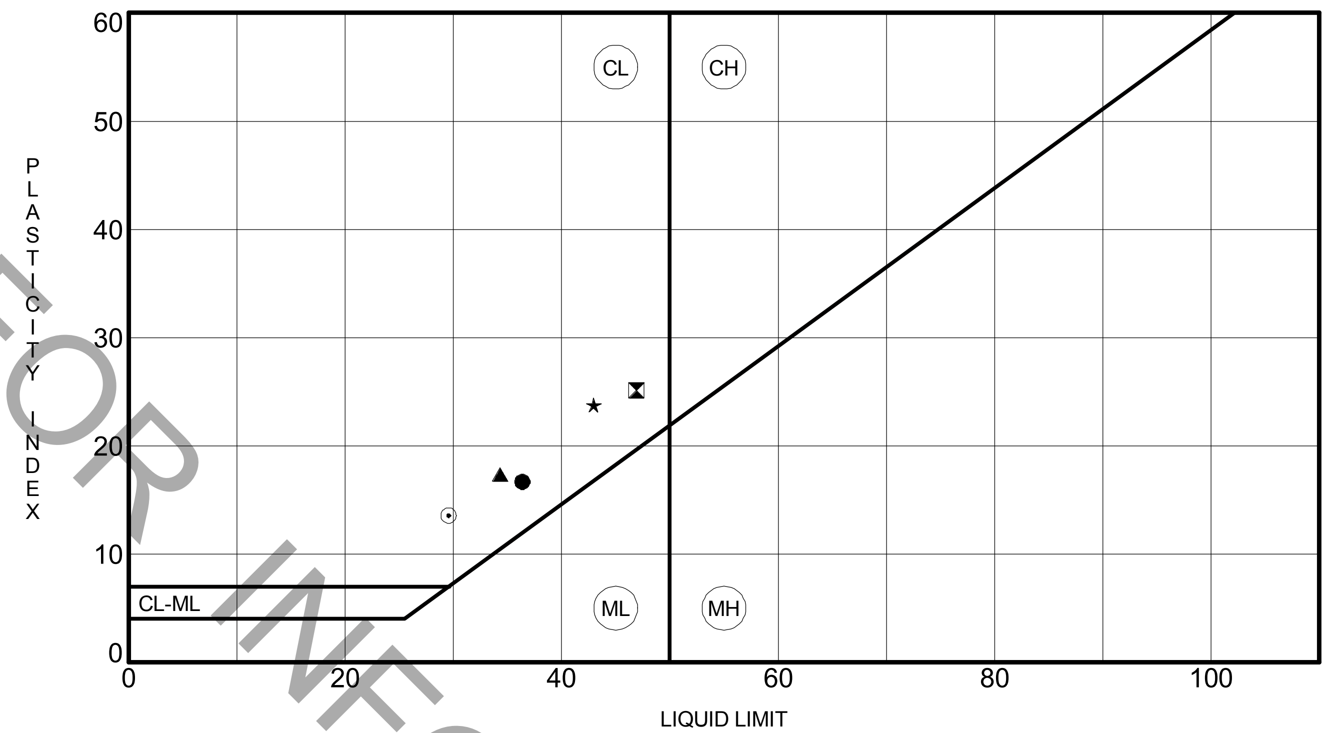
Surface Water Elev. N/A ft
Stream Bed Elev. N/A ft

Groundwater Elev.:
First Encounter 594.6 ft ▼
Upon Completion -- ft
After N/A **Hrs.** N/A ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
3" ASPHALT	606.4				
FILL: Brown and gray, silty clay loam, A-7			10 8 5	1.1 B	22
Trace gravel and iron stains		-5	2 2 2	0.7 B	22
	601.1				
SILTY CLAY LOAM: Gray, with iron stains, trace coal, A-6			1 1 2	0.7 S/10	27
	598.6				
CLAY LOAM: Dark brown, A-6		-10	1 3 3	0.8 S/15	21
Trace coal		▼	2 3 2	0.2 B	23
	593.6				
SHALEY CLAY: Gray, with iron nodules and stains, A-6		-15	2 3 7	1.5 S/10	26
	591.1				
CLAYEY SHALE: Gray			10 14 22	2.5 S/20	16
With sandy shale	586.6	-20	14 24 33	- -	16

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
CLAYEY SHALE: Gray, with sandy shale *(continued)*			29 47 50/2"	1.3 S/15	14
	581.6	-25	N-mod	4.3 P	13
Boring terminated at 25.0 ft.		-30			
		-35			
		-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)



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B-1**	**1.00**	**36.4**	**19.7**	**16.7**		
⧗	**B-1**	**6.00**	**46.9**	**21.8**	**25.1**		
▲	**B-1**	**11.00**	**34.3**	**16.9**	**17.4**		
★	**B-2**	**1.00**	**43.0**	**19.2**	**23.8**		
⊙	**B-2**	**8.50**	**29.5**	**16.0**	**13.5**		

Illinois Department of Transportation
Division of Highways
SCI Engineering, Inc.

ATTERBERG LIMITS' RESULTS

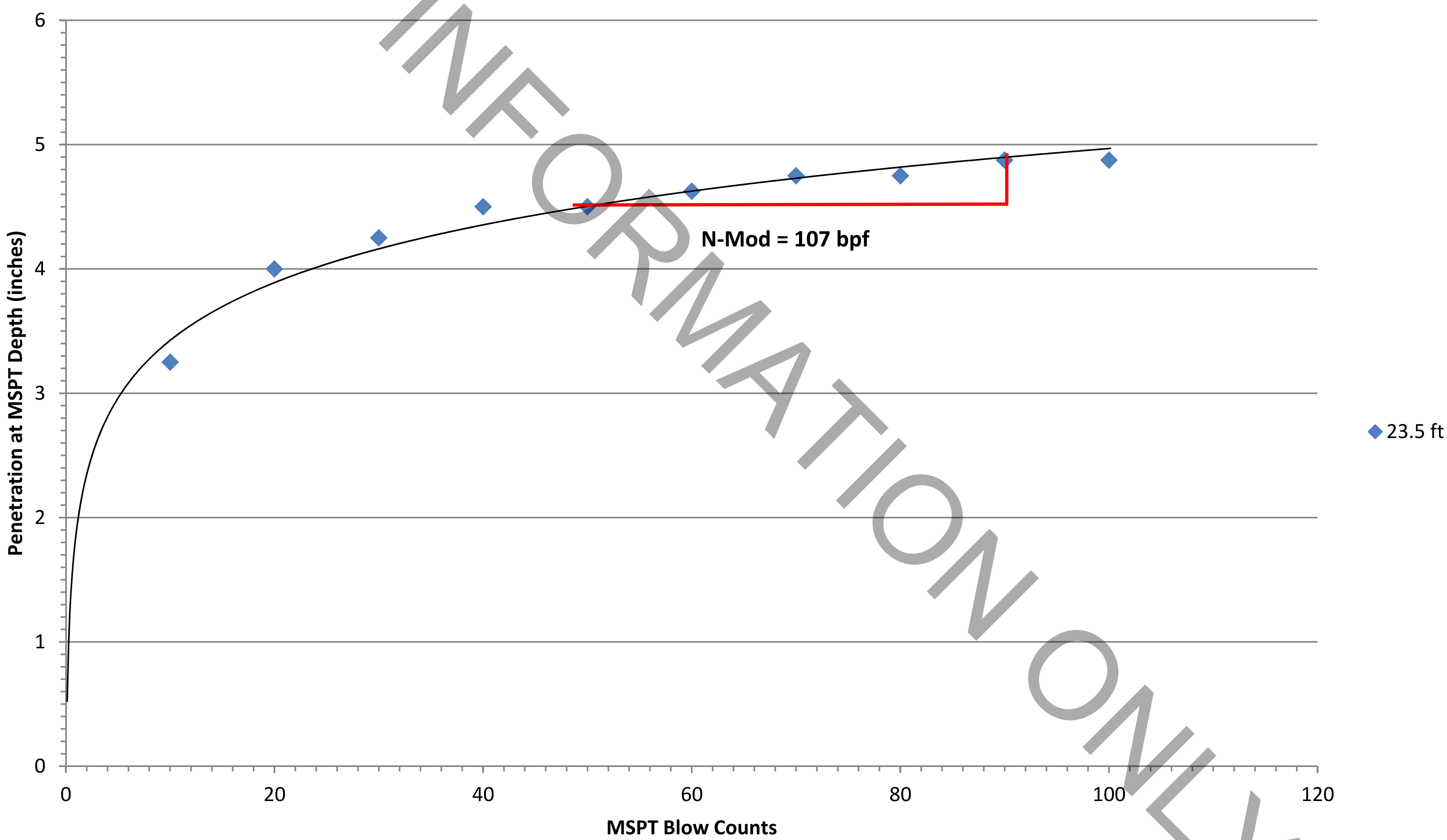
Route: FAP 626

Section: 42-(B,B-1)BR-1

County: Knox

ATTERBERG_LIMITS 2009-3119.51 IL 97 OVER LITTLE HAW CREEK PTB 151, ITEM 34.GPJ IL_DOT.GDT 2/17/14

2009-3119.51
PTB 151-34, WO 2
IL 97 Over Little Haw Creek
Modified Standard Penetration Test Results



Illinois Department of Transportation

Bridge Foundation Boring Log

PROJECT ______ BRIDGE F.A. 626 over LITTLE HAW CREEK Date 11-26-79

ROUTE FA 626 Bored By R. Ward

SEC. (42B-1)BR STA. 537+00 Checked By R. E. Dalton

COUNTY Knox

Boring No. 1
Station 536+79
Offset 14' LT ℄

Surface Water El. ______
Groundwater El. at Completion ~~594.0~~
After - Hours -

Description	Elevation	N	Qu t/s.f.	w (%)
Ground Surface 606.6	0			
BROWN MOIST SILTY CLAY LOAM		11	1.6 S	20
	-5	8	1.0 S	22
600.1				
BROWN MOIST SILTY CLAY		10	1.8 B	22
	-10	11	-	-
595.1				
BLACK WET SILTY CLAY LOAM		12	1.0 B	27
	-15	12	0.8 B	33
590.1				
GRAY DAMP SHALEY CLAY		45	2.5 S	14
	-20	100	-	9
585.1				
CONTINUED NEXT COLUMN				

Description	Elevation	N	Qu t/s.f.	w (%)
GRAY DAMP SANDSTONE		50/5½"	-	11
582.6				
BLACK DAMP SHALE	-25	50/5½"	-	14
579.6				
BLACK DAMP COAL 578.6		75	-	12
GRAY DAMP SHALEY CLAY LOAM	-30	50/2½"	4.1 S	13
		50/4"	3.4 S	16
572.6				
BLACK DAMP COAL 571.1	-35	50/4"	-	25
END OF BORING				
	-40			
	-45			

N-Standard Penetration Test-Blows per foot to drive 2" O.D. Split Spoon Sampler 12" with 140 No. hammer falling 30".

Qu-Unconfined Compressive Strength - t/sf

w - Water Content - percentage of oven dry weight-%.

Type failure:
B - Bulge Failure
S - Shear Failure
E - Estimated Value
P - Penetrometer

Illinois Department of Transportation

Bridge Foundation Boring Log

PROJECT ______ BRIDGE F.A. 626 over Little Haw Creek Date 11-20-79

ROUTE F.A. 626 Bored By R. Ward

SEC. (42B-1)BR STA. 537+00 Checked By R. E. Dalton

COUNTY Knox

Boring No. 2
Station 537+46
Offset 13' RT ℄

Surface Water El. ______
Groundwater El. at Completion 594.7
After - Hours -

	Elevation		N	Qu t/s.f.	w (%)
Ground Surface	606.7	0			
BROWN MOIST SILTY CLAY LOAM			9	1.5 B	20
		-5	7	1.8 B	23
	600.2				
DARK BROWN MOIST SILTY LOAM			5	0.8 B	26
		-10	7	0.9 B	30
	593.7		5	0.5 S	29
BROWN WET SAND AND GRAVEL	592.2				
BROWN AND GRAY DRY CLAY		-15	9	2.3 S	23
	589.7				
GRAY DAMP SANDSTONE			75	-	11
		-20	50/6"	-	12
CONTINUED NEXT COLUMN					

	Elevation		N	Qu t/s.f.	w (%)
			50/4"	-	7
	582.7				
DARK GRAY DAMP SHALE	581.2	-25	100/11½"	-	12
END OF BORING					
		-30			
		-35			
		-40			
		-45			

N-Standard Penetration Test-Blows per foot to drive 2" O.D. Split Spoon Sampler 12" with 140 No. hammer falling 30".

Qu-Unconfined Compressive Strength - t/sf

w - Water Content - percentage of oven dry weight-%.

Type failure:
B - Bulge Failure
S - Shear Failure
E - Estimated Value
P - Penetrometer

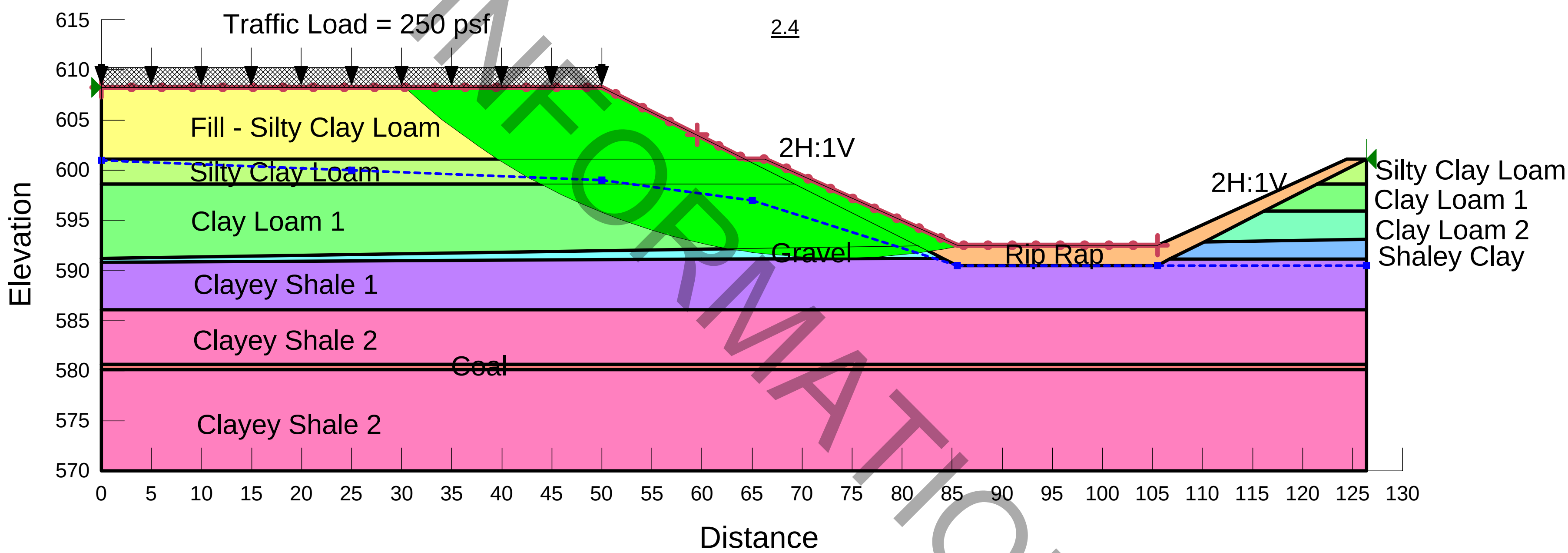
BD 137 (Rev. 4-78)

Appendix B

Not Performed Per AGMU 10.1

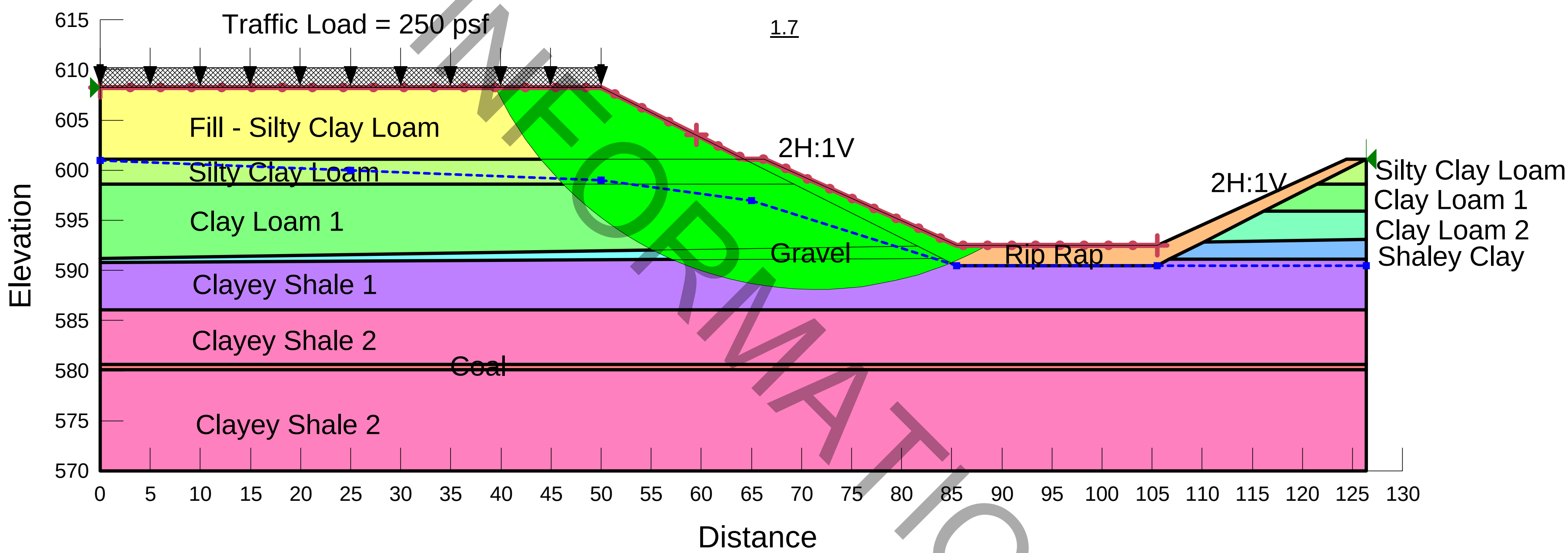
Appendix C

2009-3119.51
IL 97 Over Little Haw Creek
North Abutment
Short Term Condition



Name: Fill - Silty Clay Loam Unit Weight: 120 pcf Cohesion': 1,000 psf Phi': 0 °
Name: Silty Clay Loam Unit Weight: 120 pcf Cohesion': 500 psf Phi': 0 °
Name: Clay Loam 1 Unit Weight: 120 pcf Cohesion': 800 psf Phi': 0 °
Name: Clay Loam 2 Unit Weight: 120 pcf Cohesion': 200 psf Phi': 0 °
Name: Gravel Unit Weight: 120 pcf Cohesion': 0 psf Phi': 35 °
Name: Shaley Clay Unit Weight: 120 pcf Cohesion': 1,500 psf Phi': 0 °
Name: Clayey Shale 1 Unit Weight: 120 pcf Cohesion': 1,200 psf Phi': 0 °
Name: Clayey Shale 2 Unit Weight: 120 pcf Cohesion': 3,500 psf Phi': 0 °
Name: Coal Unit Weight: 120 pcf Cohesion': 1,000 psf Phi': 0 °
Name: Rip Rap Unit Weight: 125 pcf Cohesion': 0 psf Phi': 38 °

2009-3119.51
IL 97 Over Little Haw Creek
North Abutment
Long Term Condition



Name: Fill - Silty Clay Loam Unit Weight: 120 pcf Cohesion': 250 psf Phi': 24 °

Name: Silty Clay Loam Unit Weight: 120 pcf Cohesion': 125 psf Phi': 24 °

Name: Clay Loam 1 Unit Weight: 120 pcf Cohesion': 200 psf Phi': 24 °

Name: Clay Loam 2 Unit Weight: 120 pcf Cohesion': 75 psf Phi': 24 °

Name: Gravel Unit Weight: 120 pcf Cohesion': 0 psf Phi': 35 °

Name: Shaley Clay Unit Weight: 120 pcf Cohesion': 375 psf Phi': 16 °

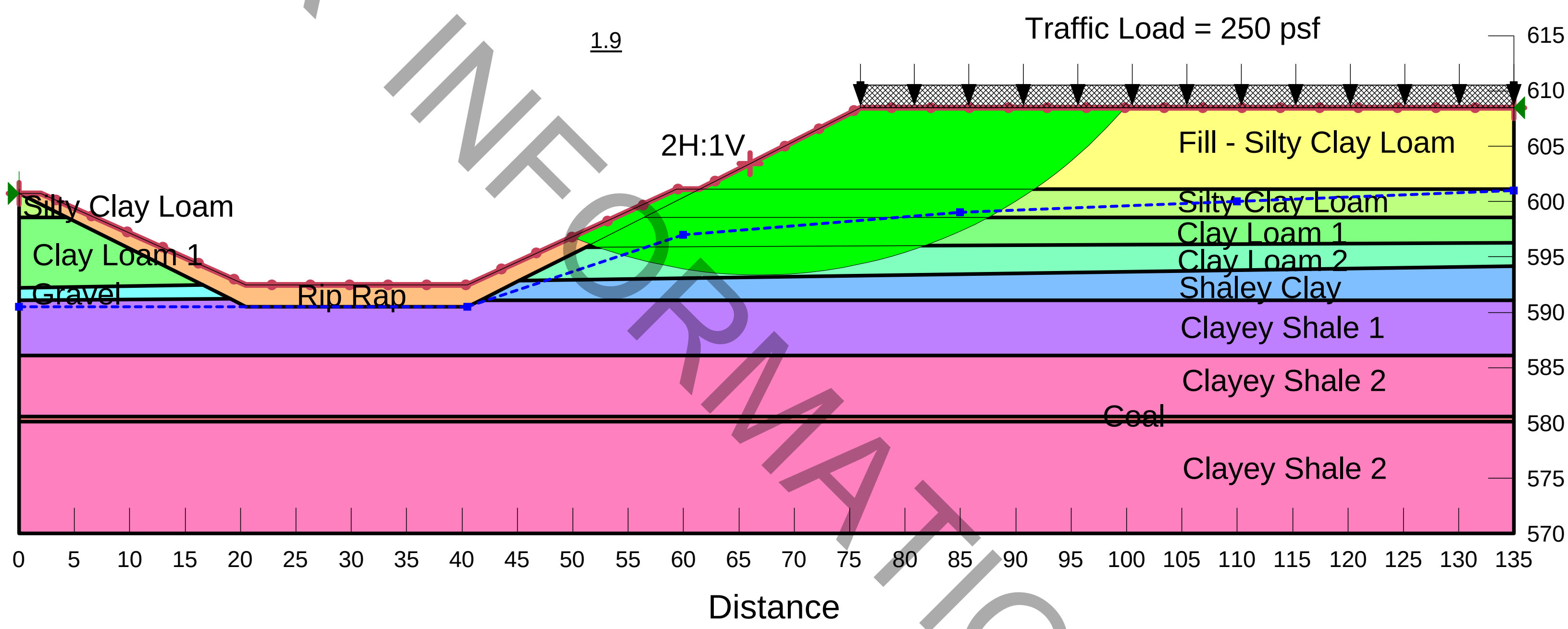
Name: Clayey Shale 1 Unit Weight: 120 pcf Cohesion': 300 psf Phi': 16 °

Name: Clayey Shale 2 Unit Weight: 120 pcf Cohesion': 875 psf Phi': 16 °

Name: Coal Unit Weight: 120 pcf Cohesion': 250 psf Phi': 10 °

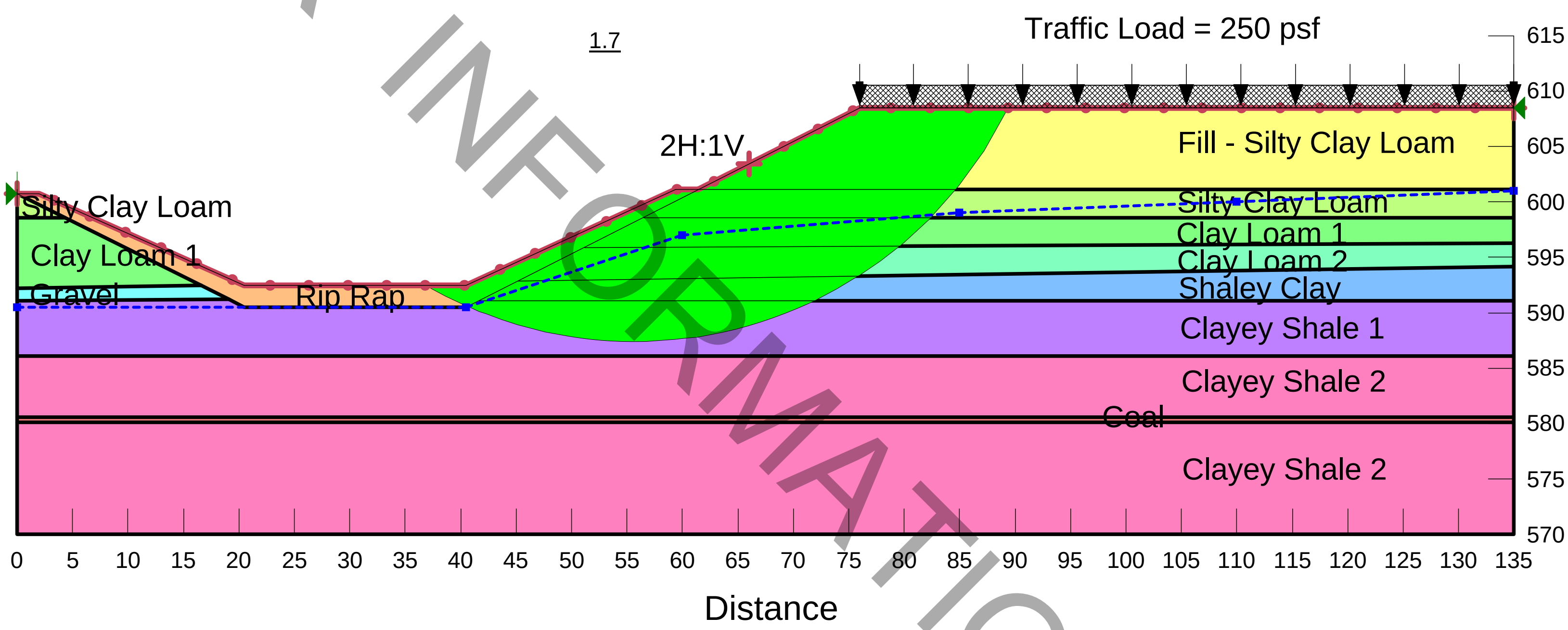
Name: Rip Rap Unit Weight: 125 pcf Cohesion': 0 psf Phi': 38 °

2009-3119.51
IL 97 Over Little Haw Creek
South Abutment
Short Term Condition



Name: FILL - Silty Clay Loam Unit Weight: 120 pcf Cohesion': 1,000 psf Phi': 0 °
Name: Silty Clay Loam Unit Weight: 120 pcf Cohesion': 500 psf Phi': 0 °
Name: Clay Loam 1 Unit Weight: 120 pcf Cohesion': 800 psf Phi': 0 °
Name: Clay Loam 2 Unit Weight: 120 pcf Cohesion': 200 psf Phi': 0 °
Name: Gravel Unit Weight: 120 pcf Cohesion': 0 psf Phi': 35 °
Name: Shaley Clay Unit Weight: 120 pcf Cohesion': 1,500 psf Phi': 0 °
Name: Clayey Shale 1 Unit Weight: 120 pcf Cohesion': 1,200 psf Phi': 0 °
Name: Clayey Shale 2 Unit Weight: 120 pcf Cohesion': 3,500 psf Phi': 0 °
Name: Coal Unit Weight: 120 pcf Cohesion': 1,000 psf Phi': 0 °
Name: Rip Rap Unit Weight: 125 pcf Cohesion': 0 psf Phi': 38 °

2009-3119.51
IL 97 Over Little Haw Creek
South Abutment
Long Term Condition



Name: FILL - Silty Clay Loam Unit Weight: 120 pcf Cohesion': 250 psf Phi': 24 °
Name: Silty Clay Loam Unit Weight: 120 pcf Cohesion': 125 psf Phi': 24 °
Name: Clay Loam 1 Unit Weight: 120 pcf Cohesion': 200 psf Phi': 24 °
Name: Clay Loam 2 Unit Weight: 120 pcf Cohesion': 75 psf Phi': 24 °
Name: Gravel Unit Weight: 120 pcf Cohesion': 0 psf Phi': 35 °
Name: Shaley Clay Unit Weight: 120 pcf Cohesion': 375 psf Phi': 16 °
Name: Clayey Shale 1 Unit Weight: 120 pcf Cohesion': 300 psf Phi': 16 °
Name: Clayey Shale 2 Unit Weight: 120 pcf Cohesion': 875 psf Phi': 16 °
Name: Coal Unit Weight: 120 pcf Cohesion': 250 psf Phi': 10 °
Name: Rip Rap Unit Weight: 125 pcf Cohesion': 0 psf Phi': 38 °

Appendix D

APPENDIX D

PROJECT: IL 97 Over Little Haw Creek
LOCATION: Knox County, Illinois
CLIENT: Oates Associates, Inc.
STRUCTURE: 048-0015 (EXISTING), 048-0097 (PROPOSED)
SCI NO.: 2009-3119.51

Table D.1 – Soil Modulus Parameters (k) for North Abutment (B-1)

Depth (ft)	Elevation (ft)	Abbreviated Soil Description	Effective Unit Weight (pcf)	Cohesion (psf)	Phi (degrees)	Soil Modulus Parameter (pci)	E_{50}
0.0 to 4.7	600.8 to 596.1	Silty Clay Loam	117	500	--	30	0.009
4.7 to 8.7	596.1 to 592.1	Clay Loam	117	900	--	90	0.008
8.7 to 9.7	592.1 to 591.1	Gravel	48	--	35	25	--
9.7 to 20.2	591.1 to 580.6	Clayey Shale	58	1,200	--	200	0.007
20.2 to 20.7	580.6 to 580.1	Coal	32	1,000		100	0.007
20.7 +	Below 580.1	Clayey Shale	58	3,500	--	850	0.005

Table D.2 – Soil Modulus Parameters (k) for South Abutment (B-2)

Depth (ft)	Elevation (ft)	Abbreviated Soil Description	Effective Unit Weight (pcf)	Cohesion (psf)	Phi (degrees)	Soil Modulus Parameter (pci)	E_{50}
0.0 to 2.5	601.1 to 598.6	Silty Clay Loam	117	700	--	60	0.009
2.5 to 5.0	598.6 to 596.1	Clay Loam	117	800	--	75	0.008
5.0 to 7.5	596.1 to 593.6	Clay Loam	117	200	--	5	0.02
7.5 to 10.0	593.6 to 591.1	Shaley Clay	58	1,500	--	300	0.007
10.0 to 20.5	591.1 to 580.6	Clayey Shale	58	2,500	--	600	0.006
20.5 to 21.0	580.6 to 580.1	Coal[1]	32	1,000	--	100	0.007
21.0 +	Below 580.1	Clayey Shale[1]	58	3,500	--	850	0.005

[1]Estimated from B-1

Appendix E

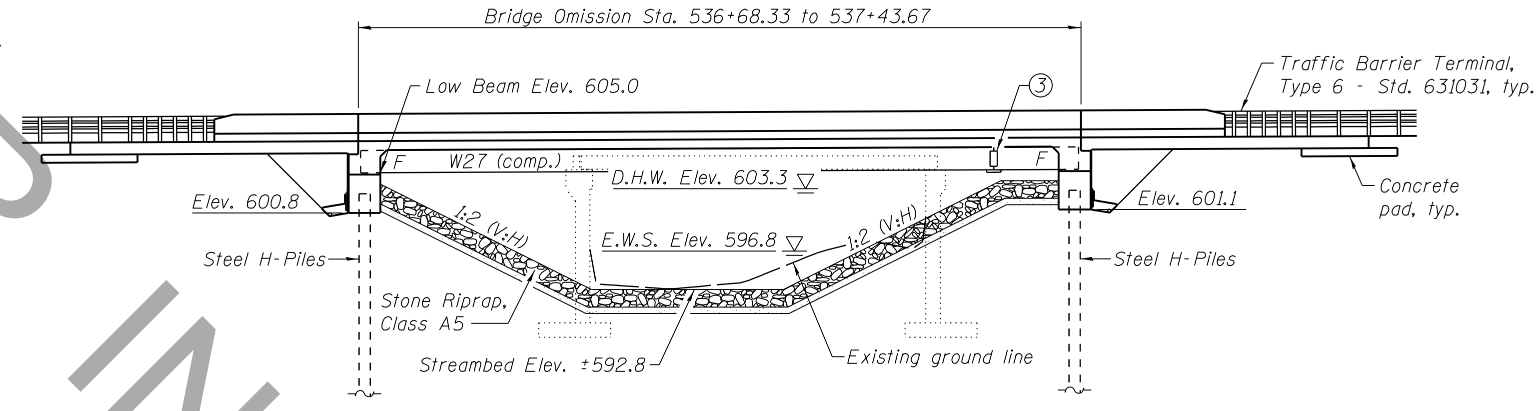
Bench Mark: BM 11 - Chiseled square on northeast wingwall, Sta. 536+87.00, 18.40' LT. Elev. 604.71.

Existing Structure: S.N. 048-0015 was originally built in 1926 as S.B.I. Route 8, Section 42B. In 1980, the superstructure and portions of the substructure were removed and replaced under Section (42B-1)BR. The structure consists of a single-span PPC deck beam superstructure supported by closed concrete abutments. The back to back abutment length is 38'-0" and the out to out width is 33'-0". Structure to be removed and replaced.

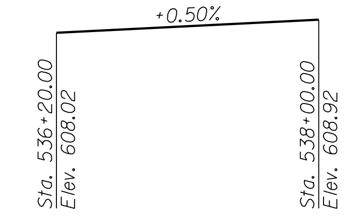
Traffic Control: One lane of traffic will be maintained utilizing stage construction.

Salvage: None

Notes:
1. Stage 1 substructure shall only be removed to a height that allows adequate clearance for construction of the Stage 1 superstructure. Remaining substructure to be removed during Stage 2 removal.
2. Structural steel shall be galvanized.
3. Optional field splice to accommodate galvanizing.
4. Pile fixity shall be investigated during final design.



ELEVATION

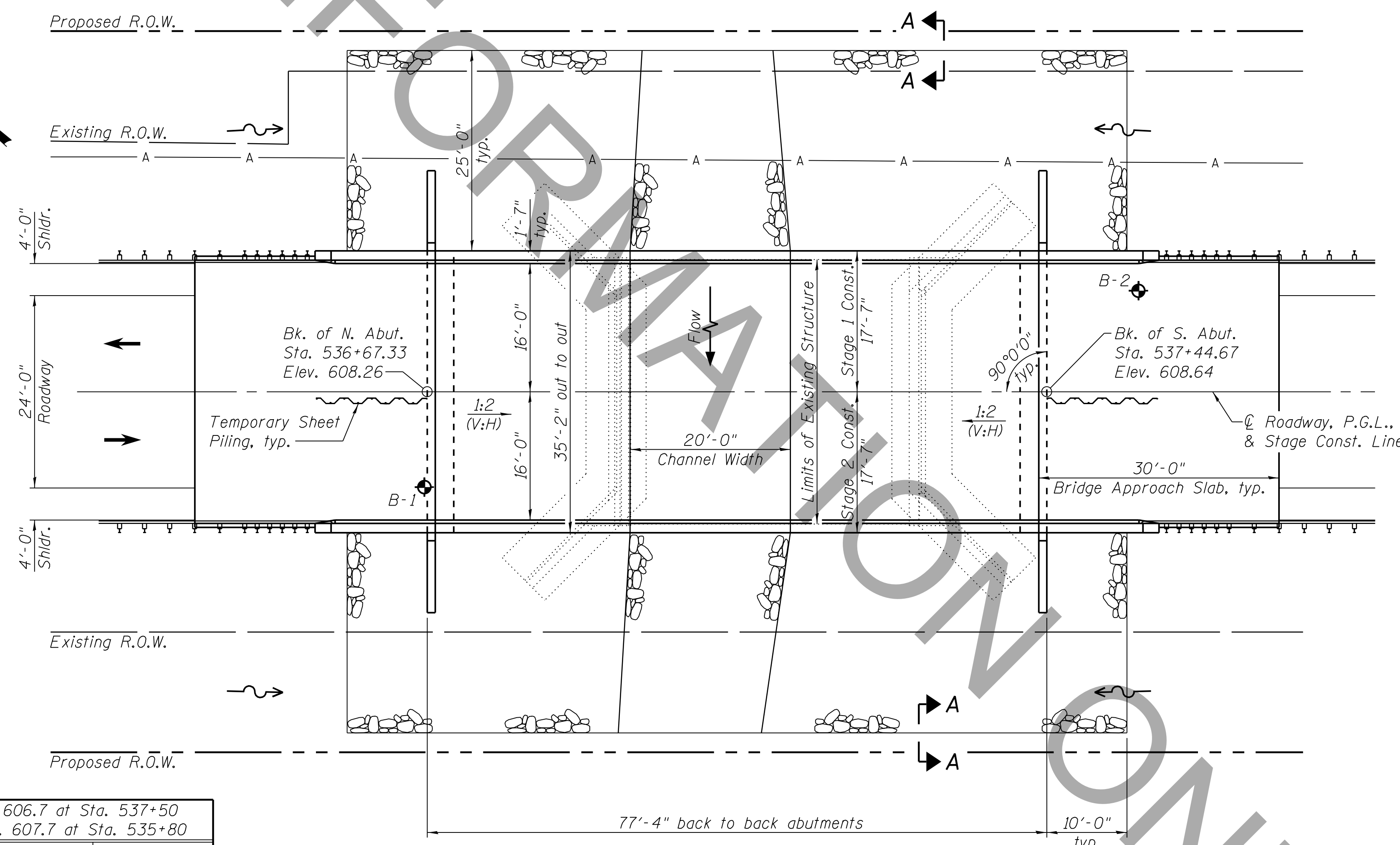


PROFILE GRADE
(Along ℄ Roadway)

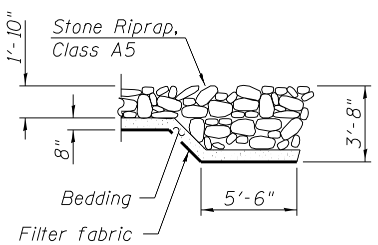
DESIGN SPECIFICATIONS
2012 AASHTO LRFD Bridge Design Specifications, 6th Edition with 2013 Interims

DESIGN STRESSES
FIELD UNITS
f'c = 3,500 psi
fy = 60,000 psi (Reinforcement)
fy = 50,000 psi (M270 Grade 50) 2

HIGHWAY CLASSIFICATION
F.A.P. 626 - IL Rte. 97
Functional Class: Minor Arterial (Rural)
ADT: 1,650 (2011); 2,013 (2031)
ADTT: 225 (2011); 274 (2031)
DHV: 201
Design Speed: 55 m.p.h.
Posted Speed: 55 m.p.h.
Two-Way Traffic Directional Distribution: 50:50



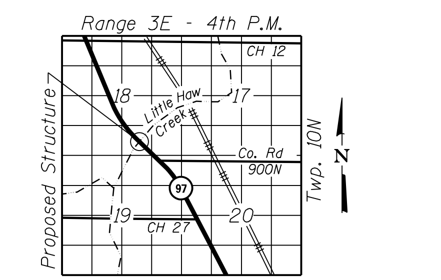
PLAN



SECTION A-A

LOADING HL-93
Allow 50#/sq. ft. for future wearing surface.

SEISMIC DATA
Seismic Performance Zone (SPZ) = 1
Design Spectral Acceleration at 1.0 sec. (SD1) = 0.07g
Design Spectral Acceleration at 0.2 sec. (SDS) = 0.12g
Soil Site Class = C



LOCATION SKETCH

DESIGN SCOUR ELEVATION TABLE

Design Scour Elevations (ft.)		
	N. Abut.	S. Abut.
Q100	600.8	601.1
Q500	600.8	601.1

WATERWAY INFORMATION

Drainage Area = 8.75 sq. mi.
Existing Low Grade Elev. 606.7 at Sta. 537+50
Proposed Low Grade Elev. 607.7 at Sta. 535+80

Flood	Freq. Yr.	Q C.F.S.	Opening Sq. Ft. Exist.	Opening Sq. Ft. Prop.	Nat. H.W.E.	Head - Ft. Exist.	Head - Ft. Prop.	Headwater El. Exist.	Headwater El. Prop.
	10	1,690	258	348	602.3	0.9	0.8	603.2	603.1
Design	50	2,710	292	408	603.3	3.0	2.3	606.3	605.6
Base	100	3,180	306	433	603.7	3.2	2.3	606.9	606.0
Scour Design Check	200	3,647	320	458	604.1	3.1	2.3	607.2	606.4
Exist. Overtopping	80	3,075	303	-	603.6	3.2	-	606.8	-
Max. Calc.	500	4,320	-	489	604.6	-	2.6	-	607.2

GENERAL PLAN & ELEVATION
IL RTE. 97 OVER LITTLE HAW CREEK
F.A.P. RTE. 626 - SEC. 42-(B,B-1)BR-1
KNOX COUNTY
STATION 537+06.00
STRUCTURE NO. 048-0097

Appendix F

APPENDIX F

PROJECT: IL 97 Over Little Haw Creek
LOCATION: Knox County, Illinois
CLIENT: Oates Associates, Inc.
STRUCTURE: 048-0015 (EXISTING), 048-0097 (PROPOSED)
SCI NO.: 2009-3119.51

Table F.1 – Estimated Maximum Driving Elevations for North Abutment (B-1)

Pile Type and Size	Estimated Refusal Elevation (ft)
HP 8 X 36	579.1
HP 10 X 42	580.1
HP 10 X 57	577.1
HP 12 X 53	580.1
HP 12 X 63	579.1
HP 12 X 74	577.1
HP 12 X 84	576.1
HP 14 X 73	579.1
HP 14 X 89	577.1
HP 14 X 102	576.1
HP 14 X 117	574.1

Table F.2 – Estimated Maximum Driving Elevations for South Abutment (B-2)

Pile Type and Size	Estimated Refusal Elevation (ft)
HP 8 X 36	580.1
HP 10 X 42	580.1
HP 10 X 57	578.1
HP 12 X 53	580.1
HP 12 X 63	579.1
HP 12 X 74	577.1
HP 12 X 84	576.1
HP 14 X 73	579.1
HP 14 X 89	577.1
HP 14 X 102	576.1
HP 14 X 117	574.1

I D O T S T A T I C M E T H O D O F E S T I M A T I N G P I L E L E N G T H

I.D.O.T. BBS FOUNDATIONS AND GEOTECHNICAL UNIT

Modified 10/18/2011

SUBSTRUCTURE============================== North Abutment
REFERENCE BORING ========================== B-1
LRFD or ASD or SEISMIC ======================== LRFD
PILE CUTOFF ELEV. =========================== 602.80 ft
GROUND SURFACE ELEV. AGAINST PILE DURING DR 600.80 ft
GEOTECHNICAL LOSS TYPE (None, Scour, Liquef., DD None
BOTTOM ELEV. OF SCOUR, LIQUEF., or DD ================= ft
TOP ELEV. OF LIQUEF. (so layers above apply DD) ============ ft

TOTAL FACTORED SUBSTRUCTURE LOAD ================ kips
TOTAL LENGTH OF SUBSTRUCTURE (along skew)=== 32.00 ft
NUMBER OF ROWS OF PILES PER SUBSTRUCTURE = 1

Approx. Factored Loading Applied per pile at 8 ft. Cts ===== 0.00 KIPS
Approx. Factored Loading Applied per pile at 3 ft. Cts ===== 0.00 KIPS

PILE TYPE AND SIZE ========== Steel HP 12 X 53
Plugged Pile Perimeter==================== 3.967 FT. Unplugged Pile Perimeter========== 5.800 FT.
Plugged Pile End Bearing Area============== 0.983 SQFT. Unplugged Pile End Bearing Area===== 0.108 SQFT.

MAX. REQUIRED BEARING & RESISTANCE for Selected Pile, Soil Profile, & Losses

Maximum Nominal Req'd Bearing of Pile	Maximum Nominal Req.d Bearing of Boring	Maximum Factored Resistance Available in Boring	Maximum Pile Driveable Length in Boring
418 KIPS	**418** KIPS	**230** KIPS	**23** FT.

BOT. OF LAYER ELEV. (FT.)	LAYER THICK. (FT.)	UNCONF. COMPR. STRENGTH (TSF.)	S.P.T. N VALUE (BLOWS)	GRANULAR OR ROCK LAYER DESCRIPTION	NOMINAL PLUGGED SIDE RESIST. (KIPS)	NOMINAL PLUGGED END BRG. RESIST. (KIPS)	NOMINAL PLUGGED TOTAL RESIST. (KIPS)	NOMINAL UNPLUG'D SIDE RESIST. (KIPS)	NOMINAL UNPLUG'D END BRG. RESIST. (KIPS)	NOMINAL UNPLUG'D TOTAL RESIST. (KIPS)	NOMINAL REQ'D BEARING (KIPS)	FACTORED GEOTECH. LOSS FROM SCOUR or DD (KIPS)	FACTORED GEOTECH. LOSS LOAD FROM DD (KIPS)	FACTORED RESISTANCE AVAILABLE (KIPS)	ESTIMATED PILE LENGTH (FT.)
598.60	2.20	0.50	3		3.4		11.7	5.0		***5.9***	6	0	0	3	4
596.10	2.50	0.60	6		4.6	8.3	20.4	6.7	0.9	***13.0***	13	0	0	7	7
592.10	4.00	0.90	5		10.3	12.4	27.8	15.1	1.4	***27.8***	28	0	0	15	11
591.10	1.00		4	Sandy Gravel	0.4	9.5	53.1	0.5	1.0	***31.1***	31	0	0	17	12
588.60	2.50	2.50	13		13.4	34.5	83.5	19.5	3.8	***52.5***	52	0	0	29	14
586.10	2.50		28	Hard Till	3.0	51.4	157.6	4.4	5.6	***64.7***	65	0	0	36	17
585.10	1.00			Shale	49.4	122.5	207.0	72.3	13.4	***136.9***	137	0	0	75	17.7
584.10	1.00			Shale	49.4	122.5	256.4	72.3	13.4	***209.2***	209	0	0	115	18.7
583.10	1.00			Shale	49.4	122.5	305.8	72.3	13.4	***281.4***	281	0	0	155	19.7
582.10	1.00			Shale	49.4	122.5	355.2	72.3	13.4	***353.7***	354	0	0	195	20.7
581.10	1.00			Shale	49.4	122.5	***404.6***	72.3	13.4	425.9	405	0	0	223	21.7
580.10	1.00			Shale	49.4	122.5	***454.0***	72.3	13.4	498.2	~~454~~	~~0~~	~~0~~	~~250~~	~~22.7~~
579.10	1.00			Shale	49.4	122.5	***503.4***	72.3	13.4	570.4	~~503~~	~~0~~	~~0~~	~~277~~	~~23.7~~
578.10	1.00			Shale	49.4	122.5	***552.9***	72.3	13.4	642.7	~~553~~	~~0~~	~~0~~	~~304~~	~~24.7~~
577.10	1.00			Shale	49.4	122.5	***602.3***	72.3	13.4	714.9	~~602~~	~~0~~	~~0~~	~~331~~	~~25.7~~
576.10	1.00			Shale	49.4	122.5	***651.7***	72.3	13.4	787.2	~~652~~	~~0~~	~~0~~	~~358~~	~~26.7~~
575.10	1.00			Shale	49.4	122.5	***701.1***	72.3	13.4	859.4	~~701~~	~~0~~	~~0~~	~~386~~	~~27.7~~
574.10	1.00			Shale	49.4	122.5	***750.5***	72.3	13.4	931.7	~~751~~	~~0~~	~~0~~	~~413~~	~~28.7~~
573.10	1.00			Shale	49.4	122.5	***799.9***	72.3	13.4	1003.9	~~800~~	~~0~~	~~0~~	~~440~~	~~29.7~~
572.10	1.00			Shale	49.4	122.5	***849.3***	72.3	13.4	1076.2	~~849~~	~~0~~	~~0~~	~~467~~	~~30.7~~
571.10	1.00			Shale	49.4	122.5	***898.8***	72.3	13.4	1148.4	~~899~~	~~0~~	~~0~~	~~494~~	~~31.7~~
570.10	1.00			Shale	49.4	122.5	***948.2***	72.3	13.4	1220.7	~~948~~	~~0~~	~~0~~	~~521~~	~~32.7~~
569.60	0.50			Shale	24.7	122.5	***972.9***	36.1	13.4	1256.8	~~973~~	~~0~~	~~0~~	~~535~~	~~33.2~~
569.10	0.50			Shale	24.7	122.5	***997.6***	36.1	13.4	1292.9	~~998~~	~~0~~	~~0~~	~~549~~	~~33.7~~
568.60	0.50			Shale	24.7	122.5	***1022.3***	36.1	13.4	1329.1	~~1022~~	~~0~~	~~0~~	~~562~~	~~34.2~~
568.10	0.50			Shale	24.7	122.5	***1047.0***	36.1	13.4	1365.2	~~1047~~	~~0~~	~~0~~	~~576~~	~~34.7~~
567.60	0.50			Shale	24.7	122.5	***1071.7***	36.1	13.4	1401.3	~~1072~~	~~0~~	~~0~~	~~589~~	~~35.2~~
567.10	0.50			Shale	24.7	122.5	***1096.4***	36.1	13.4	1437.4	~~1096~~	~~0~~	~~0~~	~~603~~	~~35.7~~
566.60	0.50			Shale	24.7	122.5	***1121.1***	36.1	13.4	1473.6	~~1121~~	~~0~~	~~0~~	~~617~~	~~36.2~~
566.10	0.50			Shale	24.7	122.5	***1145.8***	36.1	13.4	1509.7	~~1146~~	~~0~~	~~0~~	~~630~~	~~36.7~~
565.60	0.50			Shale	24.7	122.5	***1170.5***	36.1	13.4	1545.8	~~1171~~	~~0~~	~~0~~	~~644~~	~~37.2~~
565.10	0.50			Shale	24.7	122.5	***1195.2***	36.1	13.4	1581.9	~~1195~~	~~0~~	~~0~~	~~657~~	~~37.7~~
564.60	0.50			Shale	24.7	122.5	***1219.9***	36.1	13.4	1618.1	~~1220~~	~~0~~	~~0~~	~~671~~	~~38.2~~
564.10	0.50			Shale	24.7	122.5	***1244.6***	36.1	13.4	1654.2	~~1245~~	~~0~~	~~0~~	~~685~~	~~38.7~~
563.60	0.50			Shale	24.7	122.5	***1269.4***	36.1	13.4	1690.3	~~1269~~	~~0~~	~~0~~	~~698~~	~~39.2~~
563.10	0.50			Shale	24.7	122.5	***1294.1***	36.1	13.4	1726.4	~~1294~~	~~0~~	~~0~~	~~712~~	~~39.7~~
562.60	0.50			Shale	24.7	122.5	***1318.8***	36.1	13.4	1762.6	~~1319~~	~~0~~	~~0~~	~~725~~	~~40.2~~
562.10	0.50			Shale		122.5			13.4						

Pile Design Table for North Abutment utilizing Boring #B-1

	Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 8 X 36			
	4	2	4
	9	5	7
	17	9	11
	21	11	12
	35	19	14
	44	24	17
	286	157	24
Steel HP 10 X 42			
	5	3	4
	11	6	7
	22	12	11
	26	14	12
	44	24	14
	54	30	17
	335	184	23
Steel HP 10 X 57			
	5	3	4
	11	6	7
	23	12	11
	27	15	12
	46	25	14
	58	32	17
	454	250	26

	Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 12 X 53			
	6	3	4
	13	7	7
	28	15	11
	31	17	12
	52	29	14
	65	36	17
	418	230	23
Steel HP 12 X 63			
	6	3	4
	13	7	7
	28	15	11
	32	18	12
	54	30	14
	68	37	17
	497	273	24
Steel HP 12 X 74			
	6	3	4
	14	8	7
	29	16	11
	33	18	12
	56	31	14
	71	39	17
	589	324	26
Steel HP 12 X 84			
	7	4	4
	14	8	7
	29	16	11
	34	19	12
	57	31	14
	74	41	17
	664	365	27

	Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 14 X 73			
	7	4	4
	16	9	7
	33	18	11
	38	21	12
	64	35	14
	80	44	17
	578	318	24
Steel HP 14 X 89			
	8	4	4
	16	9	7
	34	18	11
	39	22	12
	66	36	14
	85	47	17
	705	388	26
Steel HP 14 X 102			
	8	4	4
	17	9	7
	34	19	11
	41	22	12
	68	37	14
	88	49	17
	810	445	27
Steel HP 14 X 117			
	8	4	4
	17	10	7
	35	19	11
	42	23	12
	70	38	14
	93	51	17
	929	511	29

Pile Design Table for North Abutment utilizing Boring #B-1

	Nominal Required Bearing (Kips)	Seismic Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 8 X 36			
	4	4	4
	9	9	7
	17	17	11
	21	21	12
	35	35	14
	44	44	17
	286	286	24
Steel HP 10 X 42			
	5	5	4
	11	11	7
	22	22	11
	26	26	12
	44	44	14
	54	54	17
	335	335	23
Steel HP 10 X 57			
	5	5	4
	11	11	7
	23	23	11
	27	27	12
	46	46	14
	58	58	17
	454	454	26

	Nominal Required Bearing (Kips)	Seismic Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 12 X 53			
	6	6	4
	13	13	7
	28	28	11
	31	31	12
	52	52	14
	65	65	17
	418	418	23
Steel HP 12 X 63			
	6	6	4
	13	13	7
	28	28	11
	32	32	12
	54	54	14
	68	68	17
	497	497	24
Steel HP 12 X 74			
	6	6	4
	14	14	7
	29	29	11
	33	33	12
	56	56	14
	71	71	17
	589	589	26
Steel HP 12 X 84			
	7	7	4
	14	14	7
	29	29	11
	34	34	12
	57	57	14
	74	74	17
	664	664	27

	Nominal Required Bearing (Kips)	Seismic Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 14 X 73			
	7	7	4
	16	16	7
	33	33	11
	38	38	12
	64	64	14
	80	80	17
	578	578	24
Steel HP 14 X 89			
	8	8	4
	16	16	7
	34	34	11
	39	39	12
	66	66	14
	85	85	17
	705	705	26
Steel HP 14 X 102			
	8	8	4
	17	17	7
	34	34	11
	41	41	12
	68	68	14
	88	88	17
	810	810	27
Steel HP 14 X 117			
	8	8	4
	17	17	7
	35	35	11
	42	42	12
	70	70	14
	93	93	17
	929	929	29

IDOT STATIC METHOD OF ESTIMATING PILE LENGTH

I.D.O.T. BBS FOUNDATIONS AND GEOTECHNICAL UNIT

Modified 10/18/2011

SUBSTRUCTURE============================== South Abutment
REFERENCE BORING ========================== B-2
LRFD or ASD or SEISMIC ======================== LRFD
PILE CUTOFF ELEV. =========================== 603.10 ft
GROUND SURFACE ELEV. AGAINST PILE DURING DR 601.10 ft
GEOTECHNICAL LOSS TYPE (None, Scour, Liquef., DD None
BOTTOM ELEV. OF SCOUR, LIQUEF., or DD ================= ft
TOP ELEV. OF LIQUEF. (so layers above apply DD) ============ ft

TOTAL FACTORED SUBSTRUCTURE LOAD ================= kips
TOTAL LENGTH OF SUBSTRUCTURE (along skew)=== 32.00 ft
NUMBER OF ROWS OF PILES PER SUBSTRUCTURE = 1
Approx. Factored Loading Applied per pile at 8 ft. Cts ===== 0.00 KIPS
Approx. Factored Loading Applied per pile at 3 ft. Cts ===== 0.00 KIPS

PILE TYPE AND SIZE =========== Steel HP 12 X 53
Plugged Pile Perimeter==================== 3.967 FT. Unplugged Pile Perimeter=========== 5.800 FT.
Plugged Pile End Bearing Area============== 0.983 SQFT. Unplugged Pile End Bearing Area===== 0.108 SQFT.

MAX. REQUIRED BEARING & RESISTANCE for Selected Pile, Soil Profile, & Losses

Maximum Nominal Req'd Bearing of Pile	Maximum Nominal Req.d Bearing of Boring	Maximum Factored Resistance Available in Boring	Maximum Pile Driveable Length in Boring
418 KIPS	**418** KIPS	**230** KIPS	**23** FT.

BOT. OF LAYER ELEV. (FT.)	LAYER THICK. (FT.)	UNCONF. COMPR. STRENGTH (TSF.)	S.P.T. N VALUE (BLOWS)	GRANULAR OR ROCK LAYER DESCRIPTION	NOMINAL PLUGGED SIDE RESIST. (KIPS)	NOMINAL PLUGGED END BRG. RESIST. (KIPS)	NOMINAL PLUGGED TOTAL RESIST. (KIPS)	NOMINAL UNPLUG'D SIDE RESIST. (KIPS)	NOMINAL UNPLUG'D END BRG. RESIST. (KIPS)	NOMINAL UNPLUG'D TOTAL RESIST. (KIPS)	NOMINAL REQ'D BEARING (KIPS)	FACTORED GEOTECH. LOSS FROM SCOUR or DD (KIPS)	FACTORED GEOTECH. LOSS LOAD FROM DD (KIPS)	FACTORED RESISTANCE AVAILABLE (KIPS)	ESTIMATED PILE LENGTH (FT.)
598.60	2.50	0.70	3		5.2		16.2	7.6		**8.8**	9	0	0	5	5
596.10	2.50	0.80	6		5.9	11.0	**13.8**	8.6	1.2	16.5	14	0	0	8	7
593.60	2.50	0.20	5		1.6	2.8	33.4	2.4	0.3	**20.8**	21	0	0	11	10
591.10	2.50	1.50	10		9.5	20.7	56.7	13.9	2.3	**36.3**	36	0	0	20	12
588.60	2.50	2.50	36		13.4	34.5	140.3	19.5	3.8	**63.5**	64	0	0	35	15
586.10	2.50		57	Hard Till	7.8	104.7	165.9	11.4	11.5	**76.8**	77	0	0	42	17
585.10	1.00			Shale	49.4	122.5	215.3	72.3	13.4	**149.1**	149	0	0	82	18
584.10	1.00			Shale	49.4	122.5	264.7	72.3	13.4	**221.3**	221	0	0	122	19
583.10	1.00			Shale	49.4	122.5	314.1	72.3	13.4	**293.6**	294	0	0	161	20
582.10	1.00			Shale	49.4	122.5	**363.5**	72.3	13.4	365.8	364	0	0	200	21
581.10	1.00			Shale	49.4	122.5	**412.9**	72.3	13.4	438.1	413	0	0	227	22
580.10	1.00			Shale	49.4	122.5	**462.4**	72.3	13.4	510.4	~~462~~	~~0~~	~~0~~	~~254~~	~~23~~
579.10	1.00			Shale	49.4	122.5	**511.8**	72.3	13.4	582.6	~~512~~	~~0~~	~~0~~	~~281~~	~~24~~
578.10	1.00			Shale	49.4	122.5	**561.2**	72.3	13.4	654.9	~~561~~	~~0~~	~~0~~	~~309~~	~~25~~
577.10	1.00			Shale	49.4	122.5	**610.6**	72.3	13.4	727.1	~~611~~	~~0~~	~~0~~	~~336~~	~~26~~
576.10	1.00			Shale	49.4	122.5	**660.0**	72.3	13.4	799.4	~~660~~	~~0~~	~~0~~	~~363~~	~~27~~
575.10	1.00			Shale	49.4	122.5	**709.4**	72.3	13.4	871.6	~~709~~	~~0~~	~~0~~	~~390~~	~~28~~
574.10	1.00			Shale	49.4	122.5	**758.8**	72.3	13.4	943.9	~~759~~	~~0~~	~~0~~	~~417~~	~~29~~
573.10	1.00			Shale	49.4	122.5	**808.3**	72.3	13.4	1016.1	~~808~~	~~0~~	~~0~~	~~445~~	~~30~~
572.10	1.00			Shale	49.4	122.5	**857.7**	72.3	13.4	1088.4	~~858~~	~~0~~	~~0~~	~~472~~	~~31~~
571.10	1.00			Shale	49.4	122.5	**907.1**	72.3	13.4	1160.6	~~907~~	~~0~~	~~0~~	~~499~~	~~32~~
570.10	1.00			Shale	49.4	122.5	**956.5**	72.3	13.4	1232.9	~~956~~	~~0~~	~~0~~	~~526~~	~~33~~
569.60	0.50			Shale	24.7	122.5	**981.2**	36.1	13.4	1269.0	~~981~~	~~0~~	~~0~~	~~540~~	~~33.5~~
569.10	0.50			Shale	24.7	122.5	**1005.9**	36.1	13.4	1305.1	~~1006~~	~~0~~	~~0~~	~~553~~	~~34~~
568.60	0.50			Shale	24.7	122.5	**1030.6**	36.1	13.4	1341.2	~~1031~~	~~0~~	~~0~~	~~567~~	~~34.5~~
568.10	0.50			Shale	24.7	122.5	**1055.3**	36.1	13.4	1377.4	~~1055~~	~~0~~	~~0~~	~~580~~	~~35~~
567.60	0.50			Shale	24.7	122.5	**1080.0**	36.1	13.4	1413.5	~~1080~~	~~0~~	~~0~~	~~594~~	~~35.5~~
567.10	0.50			Shale	24.7	122.5	**1104.7**	36.1	13.4	1449.6	~~1105~~	~~0~~	~~0~~	~~608~~	~~36~~
566.60	0.50			Shale	24.7	122.5	**1129.4**	36.1	13.4	1485.7	~~1129~~	~~0~~	~~0~~	~~621~~	~~36.5~~
566.10	0.50			Shale	24.7	122.5	**1154.1**	36.1	13.4	1521.9	~~1154~~	~~0~~	~~0~~	~~635~~	~~37~~
565.60	0.50			Shale	24.7	122.5	**1178.9**	36.1	13.4	1558.0	~~1179~~	~~0~~	~~0~~	~~648~~	~~37.5~~
565.10	0.50			Shale	24.7	122.5	**1203.6**	36.1	13.4	1594.1	~~1204~~	~~0~~	~~0~~	~~662~~	~~38~~
564.60	0.50			Shale	24.7	122.5	**1228.3**	36.1	13.4	1630.2	~~1228~~	~~0~~	~~0~~	~~676~~	~~38.5~~
564.10	0.50			Shale	24.7	122.5	**1253.0**	36.1	13.4	1666.4	~~1253~~	~~0~~	~~0~~	~~689~~	~~39~~
563.60	0.50			Shale	24.7	122.5	**1277.7**	36.1	13.4	1702.5	~~1278~~	~~0~~	~~0~~	~~703~~	~~39.5~~
563.10	0.50			Shale	24.7	122.5	**1302.4**	36.1	13.4	1738.6	~~1302~~	~~0~~	~~0~~	~~716~~	~~40~~
562.60	0.50			Shale	24.7	122.5	**1327.1**	36.1	13.4	1774.8	~~1327~~	~~0~~	~~0~~	~~730~~	~~40.5~~
562.10	0.50			Shale		122.5			13.4						

Pile Design Table for South Abutment utilizing Boring #B-2

	Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 8 X 36			
	6	3	5
	9	5	7
	14	8	10
	24	13	12
	43	24	15
	52	28	17
	286	157	23
Steel HP 10 X 42			
	7	4	5
	11	6	7
	17	10	10
	30	17	12
	53	29	15
	64	35	17
	335	184	23
Steel HP 10 X 57			
	8	4	5
	11	6	7
	18	10	10
	31	17	12
	56	31	15
	68	37	17
	454	250	25

	Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 12 X 53			
	9	5	5
	14	8	7
	21	11	10
	36	20	12
	64	35	15
	77	42	17
	418	230	23
Steel HP 12 X 63			
	9	5	5
	14	8	7
	22	12	10
	37	21	12
	66	37	15
	80	44	17
	497	273	24
Steel HP 12 X 74			
	9	5	5
	14	8	7
	22	12	10
	38	21	12
	69	38	15
	83	46	17
	589	324	26
Steel HP 12 X 84			
	10	5	5
	14	8	7
	23	12	10
	39	22	12
	72	39	15
	86	47	17
	664	365	27

	Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 14 X 73			
	11	6	5
	17	9	7
	25	14	10
	44	24	12
	78	43	15
	95	52	17
	578	318	24
Steel HP 14 X 89			
	11	6	5
	17	9	7
	26	14	10
	46	25	12
	82	45	15
	100	55	17
	705	388	26
Steel HP 14 X 102			
	12	6	5
	17	10	7
	27	15	10
	47	26	12
	86	47	15
	103	57	17
	810	445	27
Steel HP 14 X 117			
	12	7	5
	18	10	7
	28	15	10
	48	27	12
	89	49	15
	108	59	17
	929	511	29

Pile Design Table for South Abutment utilizing Boring #B-2

	Nominal Required Bearing (Kips)	Seismic Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 8 X 36			
	6	6	5
	9	9	7
	14	14	10
	24	24	12
	43	43	15
	52	52	17
	286	286	23
Steel HP 10 X 42			
	7	7	5
	11	11	7
	17	17	10
	30	30	12
	53	53	15
	64	64	17
	335	335	23
Steel HP 10 X 57			
	8	8	5
	11	11	7
	18	18	10
	31	31	12
	56	56	15
	68	68	17
	454	454	25

	Nominal Required Bearing (Kips)	Seismic Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 12 X 53			
	9	9	5
	14	14	7
	21	21	10
	36	36	12
	64	64	15
	77	77	17
	418	418	23
Steel HP 12 X 63			
	9	9	5
	14	14	7
	22	22	10
	37	37	12
	66	66	15
	80	80	17
	497	497	24
Steel HP 12 X 74			
	9	9	5
	14	14	7
	22	22	10
	38	38	12
	69	69	15
	83	83	17
	589	589	26
Steel HP 12 X 84			
	10	10	5
	14	14	7
	23	23	10
	39	39	12
	72	72	15
	86	86	17
	664	664	27

	Nominal Required Bearing (Kips)	Seismic Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 14 X 73			
	11	11	5
	17	17	7
	25	25	10
	44	44	12
	78	78	15
	95	95	17
	578	578	24
Steel HP 14 X 89			
	11	11	5
	17	17	7
	26	26	10
	46	46	12
	82	82	15
	100	100	17
	705	705	26
Steel HP 14 X 102			
	12	12	5
	17	17	7
	27	27	10
	47	47	12
	86	86	15
	103	103	17
	810	810	27
Steel HP 14 X 117			
	12	12	5
	18	18	7
	28	28	10
	48	48	12
	89	89	15
	108	108	17
	929	929	29

Important Information about Your

Geotechnical Engineering 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 information is provided 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 engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering 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 engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering 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 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 engineering 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 engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight 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 engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering 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 method 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 Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering 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 engineering 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 engineering 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 engineering 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 the risk of such outcomes, 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 engineering 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.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth 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.

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