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Final Structure Geotechnical Report

BRIDGE REPLACEMENT
FAP 643 (IL17) OVER INDIAN CREEK
STARK COUNTY, ILLINOIS
PTB 153-42, WO 5
ROUTE: FAP 643 (IL 17)
SECTION: 14-BR-3
STRUCTURE NO. 088-0001 (EXISTING), 088-0032 (PROPOSED)

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September 19, 2014
Revised February 13, 2015

Prepared for:
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11/30/15

LICENSED PROFESSIONAL ENGINEER
THOMAS JOSEPH CASEY
062-061853
STATE OF ILLINOIS

SCI No. 2009-3210.53

2/13/15

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

February 13, 2015

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 643 (IL17) over Indian Creek
Stark County, Illinois
PTB 153-42, WO 5
Route: FAP 643 (IL17)
Section: 14-BR-3
Structure No: 088-0001 (EXISTING), 088-0032 (PROPOSED)
SCI No.: 2009-3210.53

Dear Mr. Schopp:

Enclosed is our Final *Structure Geotechnical Report (SGR)* dated September 19, 2014, revised February 13, 2015. 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, P.E.
Senior Engineer

Thomas J. Casey, P.E.
Senior Geotechnical Engineer

JAM/TJC/tlw

Enclosure

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FAP 643 (IL17) OVER INDIAN CREEK
STARK COUNTY, ILLINOIS
PTB 153-42, WO 5
ROUTE: FAP 643 (IL 17)
SECTION: 14-BR-3
STRUCTURE NO. 088-0001 (EXISTING), 088-0032 (PROPOSED)

1.0 PROJECT DESCRIPTION

The geotechnical study summarized in this report was performed for replacement of the existing bridge which carries Illinois 17 over Indian Creek in Stark County, Illinois. The location of the site is shown on the *Vicinity and Topographic Map*, Figure 1. Based on the project plans for the existing and proposed bridge provided by Oates Associates, Inc. (Oates), the existing structure is a 2-lane, three-span structure (SN 088-0001) with an approximate length of 132.25 feet (back to back abutment) and an approximate width of 46 feet (out to out deck). The proposed replacement bridge (SN 088-0032) will be a single span structure with a length of 119.85-foot and a width of 39.2 feet. The bridge deck will be raised slightly from El. 692.46 and 692.90 to El. 692.68 and 693.10 at the east and west abutments, respectively. The streambed width for the existing bridge was cut to 22 feet with a streambed El. of 668. While the streambed elevation remains at 668, the streambed will be widened to 25.5 feet. The resulting abutment slopes will remain at a two horizontal to one vertical (2H:1V) inclination. The side slopes will be constructed at inclinations of 2H:1V or less. The estimated water surface elevation is around 673.8 feet with the design high water elevation of around 682.3.

2.0 SUBSURFACE EXPLORATION

2.1 Area Geology

Within the project area, the soil geology is made of unlithified materials consisting of loamy and silty soils that formed in loess (windblown silt deposits) over Illinoisan glacial till plains and moraines (*Soil Survey of Stark County Illinois,* Natural Resources Conservation Service, 2005 and USDA Web Soil Survey). Within the project area, these deposits overlie the Carbondale formation which consists of primarily shale with a secondary limestone unit. The remaining minor units consist of claystone, sandstone, coal and black shale.

2.2 Mining Activity

Based on the Illinois Coal Resource Shapefile GIS data provided by the Illinois State Geological Survey, dated April 1, 2014, the site is not undermined. The location of the nearest mines are about 4 miles east/northeast of the bridge location. 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.

2.3 Exploration Procedures

In October 1963, IDOT drilled four standard penetration test (SPT) borings with Shelby tubes, designated B-1 through B-4 near the existing abutment and pier locations, as shown on the *Site Plan*, Figure 2. For purposes of this report, SCI has assumed that the field exploration was performed in general accordance with procedures similar to those outlined in the 1999 *IDOT Geotechnical Manual*.

Borings B-1 was drilled to a depth of 25.5 feet and was advanced to a depth of 33 feet using rock coring techniques. B-2 and B-3 were also drilled to a depth of 33 feet while B-4 was advanced to a depth of 40.5 feet. Each boring terminated in 100 blows per foot rock.

Table 2.1 - Summary of Borings Drilled For Structure SN 088-0001

Boring	Type	Ground Surface Elevation at the time of Drilling (ft)	Boring Depth (ft)	Station	Offset (ft)	
B-1	SPT Boring	677.9	32.0	128+46	11	LT
B-1 ST	Shelby Tube Boring	677.9	19.0	128+46	11	LT
B-2	SPT Boring	677.2	33.0	129+06	22	RT
B-3	SPT Boring	676.4	33.0	129+24	22	LT
B-4	SPT Boring	676.2	40.5	129+84	11	RT
B-4 ST	Shelby Tube Boring	676.2	16.5	129+84	11	RT

2.4 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 existing structure is shown on Figure 2 and the generalized soil profiles are included on the subsurface profile, Figure 3.

While existing fill soils were not present in the 1963 boring, we anticipate up to approximately 17 feet of existing fill was placed to create the present abutments. We have assumed A-6 soils (silty clay loam) were used to create the abutment slopes.

The natural soils consisted of soft to medium stiff silty clays and loams interbedded with loose to dense sands and gravels. These soils were followed by medium to hard shaley clays and shales underlain by coal and limestone. A summary of the subsurface conditions are detailed in Table 2.2.

Table 2.2 – Summary of Subsurface Conditions

Layer	Soil/Rock Description	Elevation (ft)	Average N-Values (bpf)	Average Moisture Content (%)	Average Rimac/Hand Penetrometer Values (tsf)	Average Unconfined Compressive Strength (tsf)
1*	Fill	692.76 – 676.2	--	--	--	1.0
2	Silty to Clay Loam	677.9 - 667.4	5	18	1.6	0.86
3	Sand to Sandy Loam	670.7 – 662.4	10	20	0.3	1.16
4	Clay	665.7 – 655.7	37	17	5.7	--
5	Shale with coal, sandstone, and clay layers	659.9 – 635.7	109	15	4.5	--
5A**	Siltstone / Sandstone	648.2 - 644.9	100	--	12.1	--
6***	Limestone	647.4 – 644.2	100	--	--	--

* Values and thicknesses shown for these layers are estimated from the proposed TS&Ls from 1964 and information detailed in the IDOT Geotechnical Manual.
** Only encountered in B-3 and B-4.
*** Only encountered in B-1 and B-2.

2.5 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 Indian 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)	Groundwater Elevation 24 Hours After Drilling (ft)
B-1	669.4	670.5
B-2	669.5	670.2
B-3	669.4	669.4
B-4	669.2	671.7

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 IDOT, preliminary TS&L plans, and communications with Oates personnel familiar with the project. The preliminary TS&L is included in Appendix C.

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 0.05g, 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: 31 bpf (Site Class D)
- Method C using N_{ch}: 54 bpf (Site Class C)
- Method C using S_u: 3.61 ksf (Site Class C)

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 the bridge are neglected.

3.2 Abutment Settlement

Based on the provided TS&L, and discussions with Oates, elevation changes on the order of less than one foot are anticipated at the abutments. Therefore, a settlement analyses was not completed as settlement of the underlying soil will be negligible. Therefore, the effects of down drag on axial pile capacity are neglected.

3.3 Embankment Slope Stability

SCI conducted a slope stability analysis 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 or less. Since the inclinations of the two abutments are similar and the subsurface conditions at each abutment are similar, SCI ran a stability analyses for the east abutment which can also be applied to the west abutment. 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 D.

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
East Abutment End Slope	1.5	1.7	1.5	1.5

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

3.4 Embankment Approaches

Based on the provided plans, the creek bottom and embankment slopes will also be slightly widened. The end and side slopes will be protected with a layer of rip rap. 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.

3.5 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).

3.6 Scour

Abutment foundations are an area of primary concern for damage from scour. Per IDOT's 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 All Bridge Design Manual Section 14.2, and the provided TS&L, the design scour elevations for the following events for the abutments are shown in Table 3.3.

Table 3.3 – Summary of Design Scour Elevation

Event/Limit State	Design Scour Elevation (ft)		Item 113
	West Abutment	East Abutment	
Q100	683.1	682.7	8
Q200	683.1	682.7	
Design	683.1	682.7	
Check	683.1	682.7	

It should be noted that the above design scour elevations are located at the bottom of the abutments. Therefore, if the bottom elevation of the abutments change, the above design scour elevations will need to be revised.

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.

Table 3.4 – Preliminary Structure Loads

Location	Service I Reaction (kips)	Strength I Reaction (kips)
West Abutment	1,200	1,600
East Abutment	1,200	1,600

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 in the following sections.

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 685.1 and 684.7, while the ground surface elevation during driving was assumed to be 683.1 and 682.7 for the west and east 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. In general, “hard driving” conditions are likely to occur through the very dense sands, hard glacial tills, shale, coal, and limestone; therefore, pile shoes are required.

The pile lengths, as shown in Appendix E, 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 E. 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 will range in height from 5.5 to 10 feet and bear on fill at an approximate elevation of 681. The wingwalls should be designed to withstand lateral earth pressures caused by the weight of the backfill, including slopes behind the walls. An at-rest earth pressure coefficient (K_0) of 0.5 and an equivalent fluid pressure of 60 pcf should be used for design of the wingwalls. The value assumes that positive drainage is provided to prevent the development of hydrostatic pressure.

The wingwall foundations can be sized with the following bearing and sliding resistances provided in Table 3.5. Using these design values, total settlement of the wingwalls is estimated to be 1 inch or less.

Table 3.5 – Wingwall Recommended Resistance Factors and Resistance Values

Resistance Type	Service Limit State[A]			Strength Limit State[A]		
	Resistance Factor (ϕ_G)	Nominal Resistance (ksf)[B]	Factored Resistance (ksf)	Resistance Factor (ϕ_G)	Nominal Resistance (ksf)	Factored Resistance (ksf)
Bearing (On fill)	1.00	1.80	1.80	0.45	4.00	1.80
Sliding[C]	1.00	R_N=V*(0.62)	$R_N*\phi_G$	0.85	R_N=V*(0.62)	$R_N*\phi_G$

Notes: [A] Factors obtained from AASHTO LRFD Bridge Design Specifications 2010, Table 10.5.5.2.2-1
[B] Nominal resistance provided to limit total estimated settlement to less than 1 inch.
[C] V = vertical force acting on the footing

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.

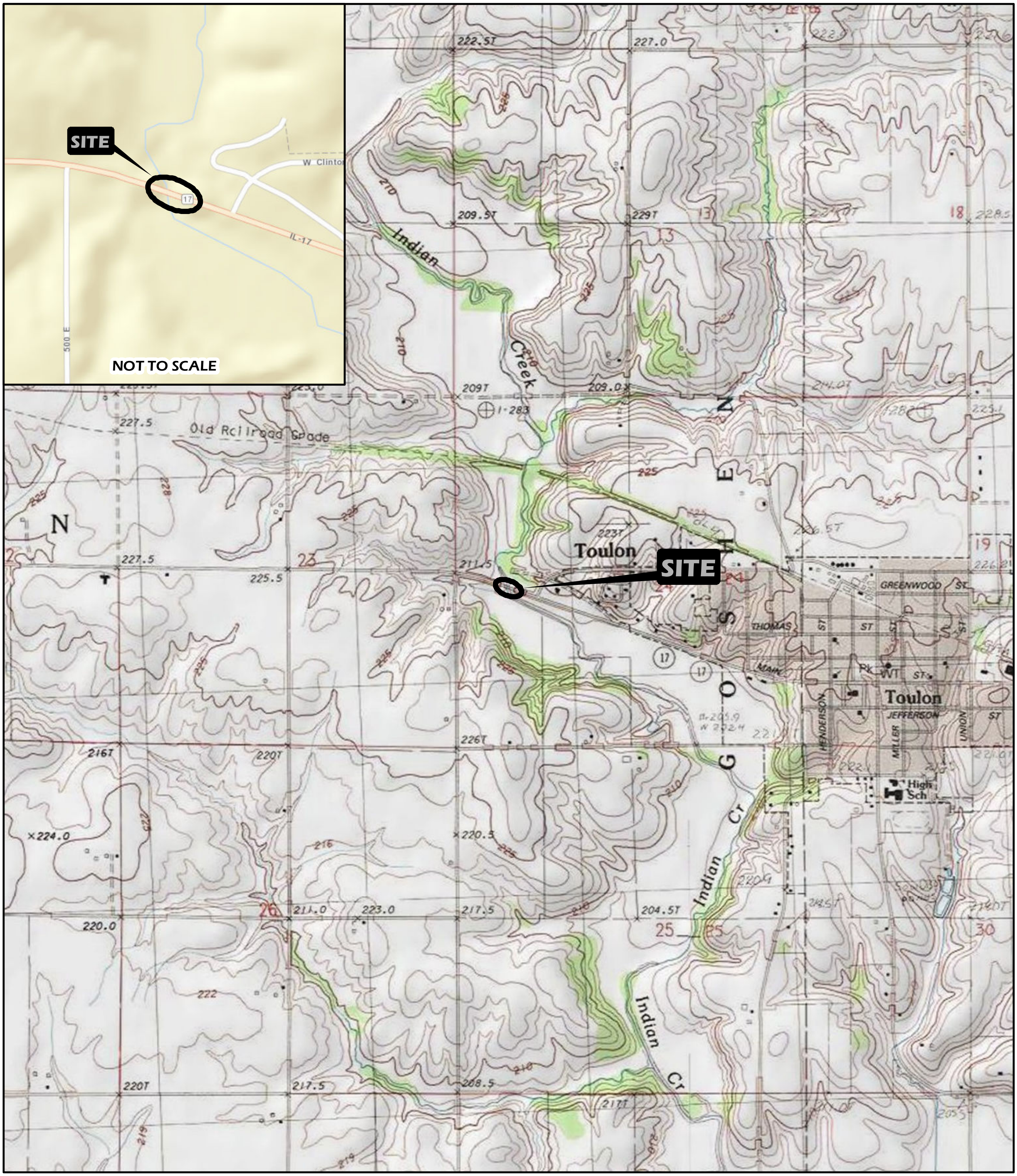
Based on the encountered subsurface conditions, tables for borings B-1 through B-4 summarizing approximate soil modulus parameters (k) for the LPILE analyses are included in Appendix F (Reference: LPILE User's Manual, Ensoft, Inc., July 2004). When pile/shaft design details and load information are refined in the development of the structure plans, LPILE analyses, if warranted, can be performed.

4.0 CONSTRUCTION CONSIDERATIONS

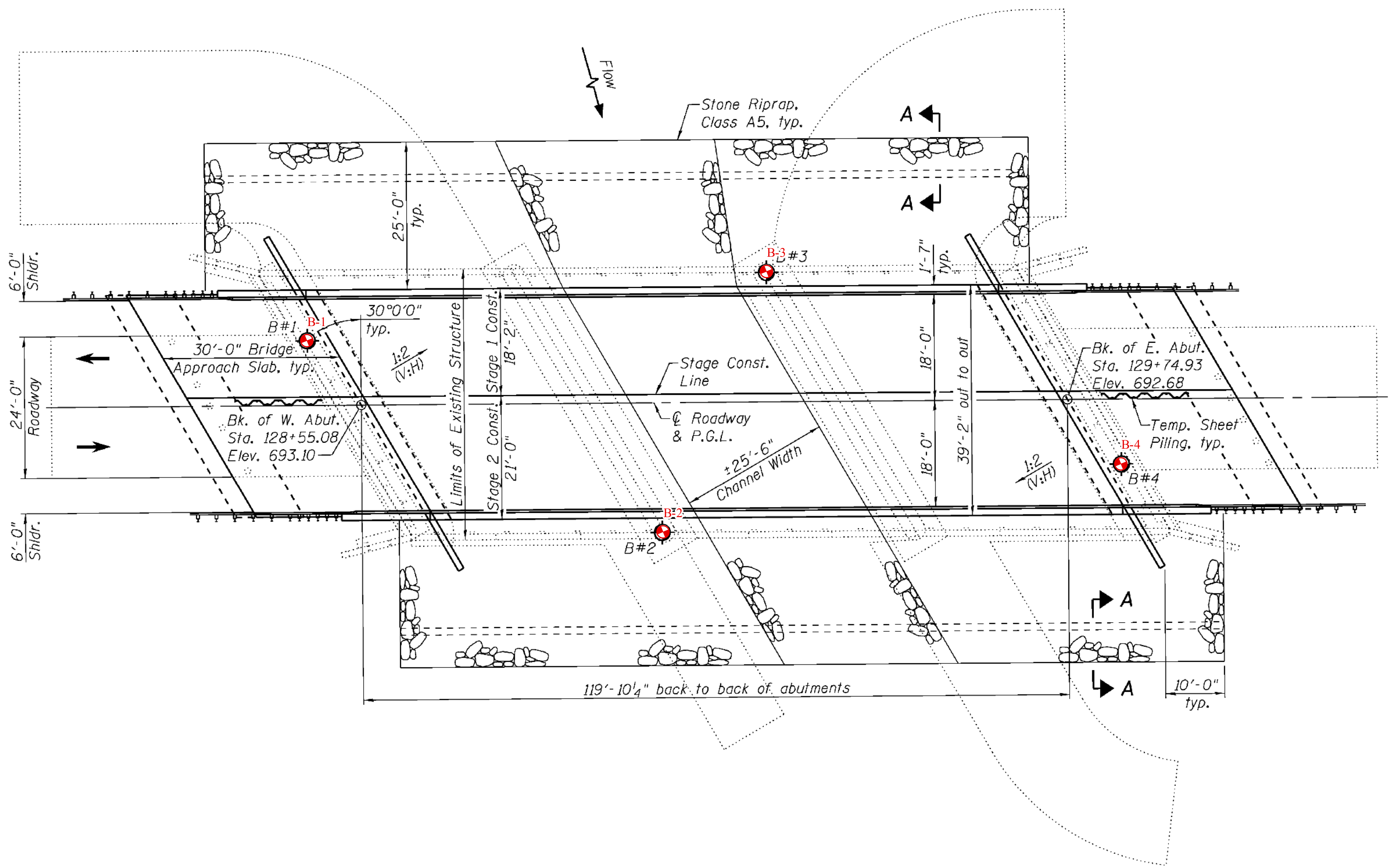
Based on the plans provided, staged construction will be required for the construction of the new structure. It appears that either temporary sheeting, including cantilever temporary sheet piling, or a soil retention system, will be feasible on the both the north and south abutments. Based on the provided 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 10.0 feet, to facilitate pile installation and abutment construction, was used in our analyses. For temporary sheeting, a minimum embedment depth of 10.0 feet with a minimum section modulus of 5.1 cubic inches per foot should be used for planning purposes.

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 four 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 FAP 643 (IL40 / IL17) OVER INDIAN CREEK STARK COUNTY, ILLINOIS	**GENERAL NOTES/LEGEND** USGS TOPOGRAPHIC MAP LAFAYETTE, ILLINOIS QUADRANGLE WYOMING, ILLINOIS QUADRANGLE DATED 1983 10' CONTOURS	N W E S
VICINITY AND TOPOGRAPHIC MAP		SCALE 1" = 2000'
DRAWN BY RCV CHECKED BY JAM DATE 02/2015 JOB NUMBER 2009-3210.53		FIGURE 1



GENERAL NOTES/LEGEND

INDICATES APPROXIMATE SOIL BORING LOCATIONS.

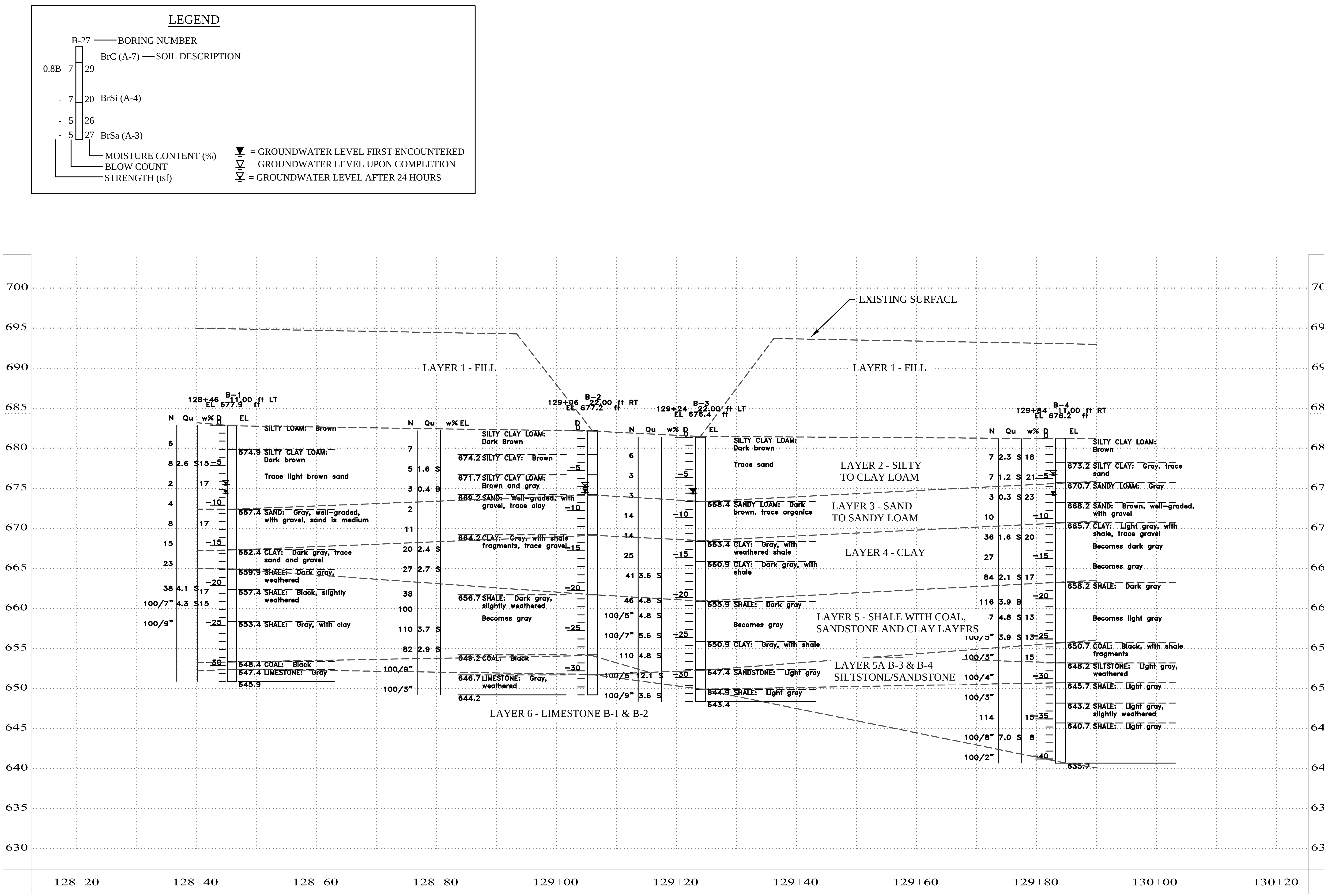
PLAN DATED 8/29/2014 BY OATES ASSOCIATES.
DIMENSIONS AND LOCATIONS ARE APPROXIMATE; ACTUAL MAY VARY. DRAWING SHALL NOT BE USED OUTSIDE THE CONTEXT OF THE REPORT FOR WHICH IT WAS GENERATED.

PROJECT NAME

FAP 643 (IL40 / IL17) OVER INDIAN CREEK
STARK COUNTY, ILLINOIS

SITE PLAN

SCALE	1" = 20'
JOB NUMBER	2009-3210.53
DATE	02/2015
DRAWN BY	RCV
CHECKED BY	JAM
FIGURE	2



General Notes/Legend

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

PROJECT NAME

FAP 643 (IL17) OVER INDIAN CREEK
STARK COUNTY, ILLINOIS

SUBSURFACE PROFILE

SCALE	1" = 7.5' V 1" = 15' H
JOB NUMBER	2009-3210.53
DATE	02/2015
DRAWN BY	RCV
CHECKED BY	JAM
FIGURE	3

Appendix A

FORM NO. B. D. 137 REV. 9-60

BRIDGE FOUNDATION BORING LOG

PROJECT	F-64( )	BRIDGE	SBI 30 over Indian Creek
ROUTE	SBI 30	Date	10-9-63
SEC.	14 BR-2	Bored By	R. A. Willems
COUNTY	Stark	STA.	129+15
		Checked By	W. Barney, Jr.

est but- ent

Boring No.	1
Station	~~128+46~~
Offset	11 Lt ℄

Surface Water El.	- -
Groundwater El. at Completion	669.4
After 24 Hours	670.5

Description	Elevation	Depth	N	Qu t/s.f.	w (%)
Ground Surface	677.9	0			
Loose Brown Silty LOAM					
	674.9		6	-	-
Soft Dark Brown Silty CLAY LOAM		-5	8	S 2.6	15
Trace Lt. Brown Sand 6' to 10'			2	-	17
	667.4	-10	4	-	-
Loose to Medium Gray Well Graded SAND and GRAVEL			8	-	17
Wash Boring 14' to 15 1/2' only	662.4	-15	15	-	-
Stiff Dark Gray CLAY with Traces of SAND and GRAVEL					
	659.9		23	-	-
Soft Dark Grey SHALE					
	657.4	-20	38	S 4.1	17
Medium Black SHALE					
			100 / 7"	S 4.3	15
	653.4				
		-25	100 / 9"	-	-
Dense Gray Shaley CLAY					
	648.4				
Very Dense Black Coal	647.4	-30			
Very Dense Grey Limestone					
	645.9				
End of Boring					
Core 25 1/2' to 33'		-35			
		-40			
		-45			

– Standard Penetration Test – lows per foot to drive 2" .D. Split Spoon Sampler 12" with 40# 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

FORM No. B.D. 137 REV. 9-60

BRIDGE FOUNDATION BORING LOG

PROJECT F-64( )
ROUTE SBI 30
SEC. 14 BR-2
COUNTY Stark

BRIDGE SBI 30 over Indian Creek
STA. 129+15

Date 10-10-63
Bored By R. A. Willems
Checked By W. Barney, Jr.

Pier 1
Boring No. 2
Station 129+06
Offset 22' Rt ℄

Surface Water El. - -
Groundwater El. at Completion 669.5
After 70 Hours 670.2

Description	Elevation	Depth	N	Qu t/s.f.	w (%)
Ground Surface	677.2	0			
Medium Dark Brown Silty CLAY LOAM	674.2		7	-	-
Stiff Brown Silty CLAY	671.7	5	5	S 1.6	-
Very Soft Brown and Gray Silty CLAY LOAM	669.2		3	B 0.4	-
Loose Well Graded SAND and GRAVEL with trace of Clay		10	2	-	-
	664.2		11	-	-
Stiff Gray Shaley CLAY layers of Gray Shale and Trace of Boulders		15	20	S 2.4	-
			27	S 2.7	
	656.7	20	38	-	-
Medium Black SHALE	654.2		100	-	-
Medium Gray SHALE		25	110	S 3.7	-
	649.2		82	S 2.9	-
Very Dense Black COAL	646.7	30	100 / 9"	-	- -
Very Dense, soft gray LIMESTONE	644.2		100 / 3"	-	- -
End of Boring		35			
		40			
		45			

N – Standard Penetration Test – Blows per foot to drive 2" O.D. Split Spoon Sampler 12" with 140# 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

BRIDGE FOUNDATION BORING LOG

PROJECT F-64( )
ROUTE SBI 30
SEC. 14 BR-2
COUNTY Stark

BRIDGE SBI 30 over Indian Creek
STA. 129+15

Date 10-(14,15)-63
Bored By R. A. Willems
Checked By W. Barney, Jr.

Pier 2
Boring No. 3
Station 129+24
Offset 22' Lt ℄

Surface Water El. -
Groundwater El. at Completion 669.4
After 1 Hours 669.4

Description	Elevation	Depth	N	Qu t/s.f.	w (%)
Ground Surface	676.4	0			
Loose Dark Brown Silty CLAY LOAM	673.4		6	-	-
Loose Dark Brown Silty CLAY LOAM with Traces of Sand		-5	3	-	-
	668.4		3	-	-
Very Soft Black SANDY LOAM Trace of Organic Material		-10	14	-	-
	663.4		14	-	-
Medium Gray Shaley CLAY	660.9	-15	25	-	-
Hard Black Shaley CLAY			41	S 3.6	-
	655.9	-20	46	S 4.8	-
Hard Black SHALE	653.4		100/5"	S 4.8	-
Hard Gray SHALE	650.9	-25	100/7"	S 5.6	-
Hard Gray Shaley CLAY			110	S 4.8	-
	647.4				
Very Dense, hard Lt. Gray SANDSTONE		-30	100/5"	S 12.1+	-
	644.9				
Very Dense Lt. Gray SHALE	643.4		100/9"	S 3.6	-
End of Boring					

N – Standard Penetration Test – Blows per foot to drive 2" O.D. Split Spoon Sampler 12" with 140# 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

BRIDGE FOUNDATION BORING LOG

PROJECT F-64( )
ROUTE SBI 30
SEC. 14 BR-2
COUNTY Stark

BRIDGE SBI 30 over Indian Creek
STA. 129+15

Date 10-(11,14)63
Bored By R. A. Willems
Checked By W. Barney, Jr.

ast
ut-
ent

Boring No. 4
Station 129+84
Offset 11' Rt ℄

Surface Water El. - -
Groundwater El. at Completion 669.2
After 24 Hours 671.7

Description	Elevation	Depth	N	Qu t/s.f.	w (%)
Ground Surface	676.2	0			
Medium Brown Silty CLAY LOAM	673.2		7	S 2.3	18
Medium Gray Silty CLAY Trace of SAND	670.7	-5	7	S 1.2	21
Soft Gray Sandy LOAM	668.2		3	B 0.3	23
Loose Brown, Well Graded SAND and GRAVEL	665.7	-10	10	-	-
Stiff Light Gray Shaley CLAY Trace Boulders	663.2		36	S 1.6	20
Stiff Dark Gray Shaley CLAY	660.7	-15	27	-	-
Firm Gray Shaley CLAY	658.2		84	S 2.1	17
Medium Black SHALE		-20	116	E 3.9	-
	654.2				

Description	Elevation	Depth	N	Qu t/s.f.	w (%)
	654.2		87	S 4.8	13
Medium Lt. Gray SHALE	650.7	-25	100/5"	S 3.9	13
Soft Black Shaley COAL	648.2		100/3"	-	15
Soft Lt. Gray SILTSTONE	645.7	-30	100/4"	-	-
Hard Lt. Gray SHALE	643.2		100/3"	-	-
Medium Lt. Gray SHALE	640.7	-35	114	-	15
Hard Lt. Gray SHALE			100/8"	S 7.0	8
	635.7	-40	100/2"		
End of Boring					
		-45			

N – Standard Penetration Test – Blows per foot to drive 2" O.D. Split Spoon Sampler 12" with 140# 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

SHELBY TUBE TEST DATA
Route SBI-30 Section 14BR-2 Stark County
Boring No. 1, Station 128 + 46, 11' Lt. S.L.
Ground Surface Elevation 677.9

Specimen	Depth Ft.	* Compr. Strength	Water %	Wet Wt. Lbs./Cu. Ft.	Description
1-1	.5	----	23.7	-----	Brown SiCL - Friable
1-2	1.0	----	16.8	-----	" " "
1-3	1.5	----	19.0	-----	" " "
2-1	2.0	----	19.2	84.0	" " "
2-2	2.5	----	20.6	-----	" " "
2-3	3.0	----	20.5	-----	" " "
2-4	3.5	----	20.0	-----	" " "
2-5	4.0	----	16.1	96.7	" " "
3-1	4.5	----	17.6	-----	Dark gray SiCL to CL
3-2	5.0	----	18.4	-----	" " CL
3-3	5.5	----	13.9	111.0	Gray SiCL
3-4	6.0	1.20	10.6	108.3	Light gray SaL
3-5	6.5	1.12	9.6	109.1	" " "
4-1	9.0	----	14.7	-----	" " " to sand
5-1	10.5	----	15.5	-----	Gray Sandy Clay Loam and fine gravel
5-2	11.0	----	21.5	-----	" " " " " " "
5-3	11.5	----	19.6	-----	" " " " " " "
6-1	13.0	----	15.4	-----	" -brown sand and fine gravel
6-2	13.5	----	15.2	141.7	" SiC with sand, gravel, and coal particles
6-3	14.0	----	26.6	-----	" " " " " " " "
7-1	15.5	----	16.3	-----	Dark gray organic SiC with sand, gravel, & coal particles
7-2	16.0	----	20.2	-----	" " " " " " " " " "
7-3	16.5	----	19.9	-----	" " -black clay - shaley
8-1	17.0	----	21.9	-----	" " " " "
8-2	17.5	----	24.6	-----	" " " " "
8-3	18.0	----	24.0	-----	" " " " "
8-4	18.5	----	25.3	-----	" " " " "
8-5	19.0	----	25.1	-----	" " " " "

* Unconfined compressive strength in tons per sq. ft.

SHELBY TUBE TEST DATA
Route SBI-30 Section 14 BR-2 Stark County
Boring No. 4, Station 129 + 84, 11' Rt. S.L.
Ground Surface Elevation 676.2

Specimen	Depth Ft.	* Compr. Strength	Water %	Wet Wt. Lbs./Cu. Ft.	Description
1-1	.5	----	16.8	-----	Brown SiCL with roots
1-2	1.0	----	14.2	102.3	" " " "
1-3	1.5	.89	14.7	99.5	" " " "
2-1	2.5	1.34	16.4	98.7	Dark brown SiCL with roots
2-2	3.0	----	20.0	97.9	" " " " "
2-3	3.5	----	16.9	109.8	" " " " "
2-4	4.0	.89	17.3	105.5	Mottled" " " "
3-1	6.0	----	22.0	119.4	" " " " "
3-2	6.5	.76	20.9	113.0	" " " " trace of sand
4-1	7.0	.42	29.9	124.6	" " " changing to SaL
4-2	7.5	----	24.6	129.0	Gray SaL
4-3	8.0	----	25.7	124.6	" " to sand
4-4	8.5	----	21.6	130.1	" sand with traces of SiCL
4-5	9.0	----	18.4	-----	" " " gravel
5-1	10.0	----	19.0	-----	Brown "
5-2	10.5	----	18.5	127.8	" " with large rocks
5-3	11.0	----	12.7	141.7	" " " " "
5-4	11.5	----	10.2	150.8	" SaL " " "
6-1	13.5	----	10.7	144.1	" " " " "
6-2	14.0	----	12.9	150.0	" " " " "
7-1	16.5	----	17.0	-----	Black shale - like material

* Unconfined compressive strength in tons per sq. ft.

Appendix B

Not Performed Per AGMU 10.1

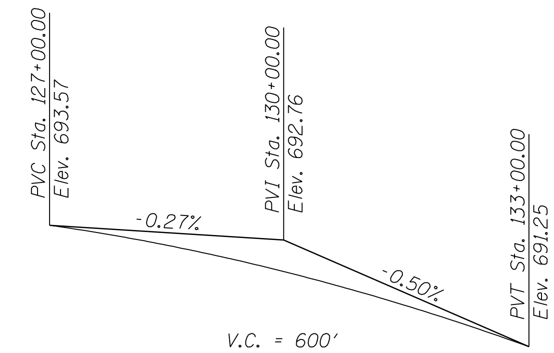
Appendix C

Benchmark: Chiseled square on top of southeast wingwall, Sta. 129+97.80, 23.85' RT., Elev. 694.57.

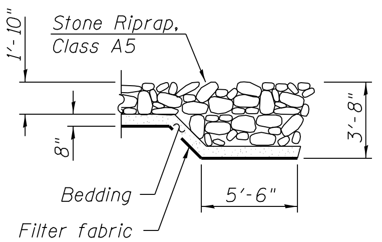
Existing Structure: S.N. 088-0001 was originally built in 1965 as S.B.I. Route 30, Section 14-BR-2. The structure consists of three spans of 36" PPC I-beams with a cast in place concrete deck supported by open abutments founded on steel H-piles and solid wall piers founded on spread footings. The back to back abutment length is 132'-3" and the out to out width is 46'-0". Structure to be removed and replaced.

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

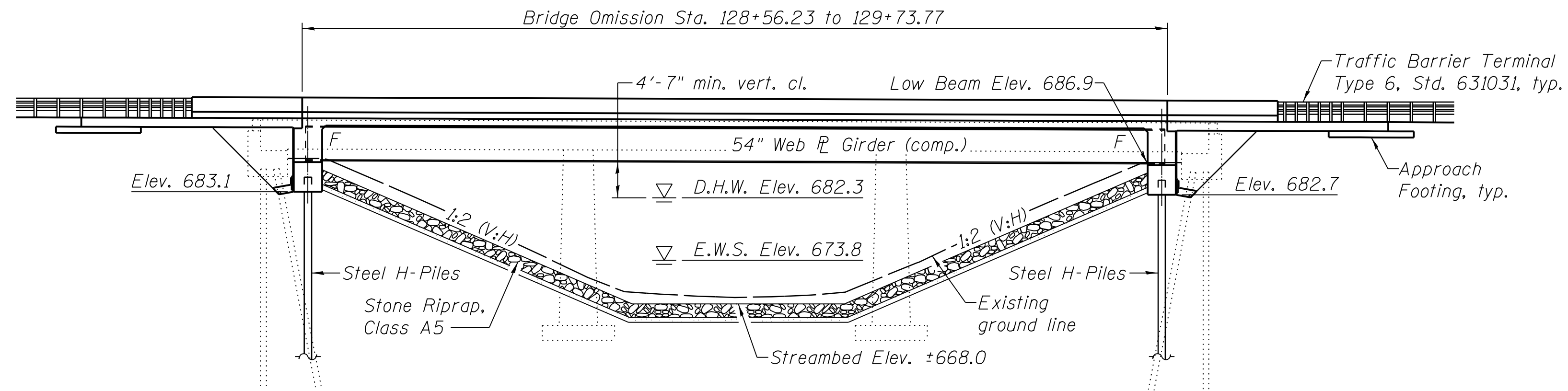
Salvage: None



PROFILE GRADE
(Along ℄ Roadway)



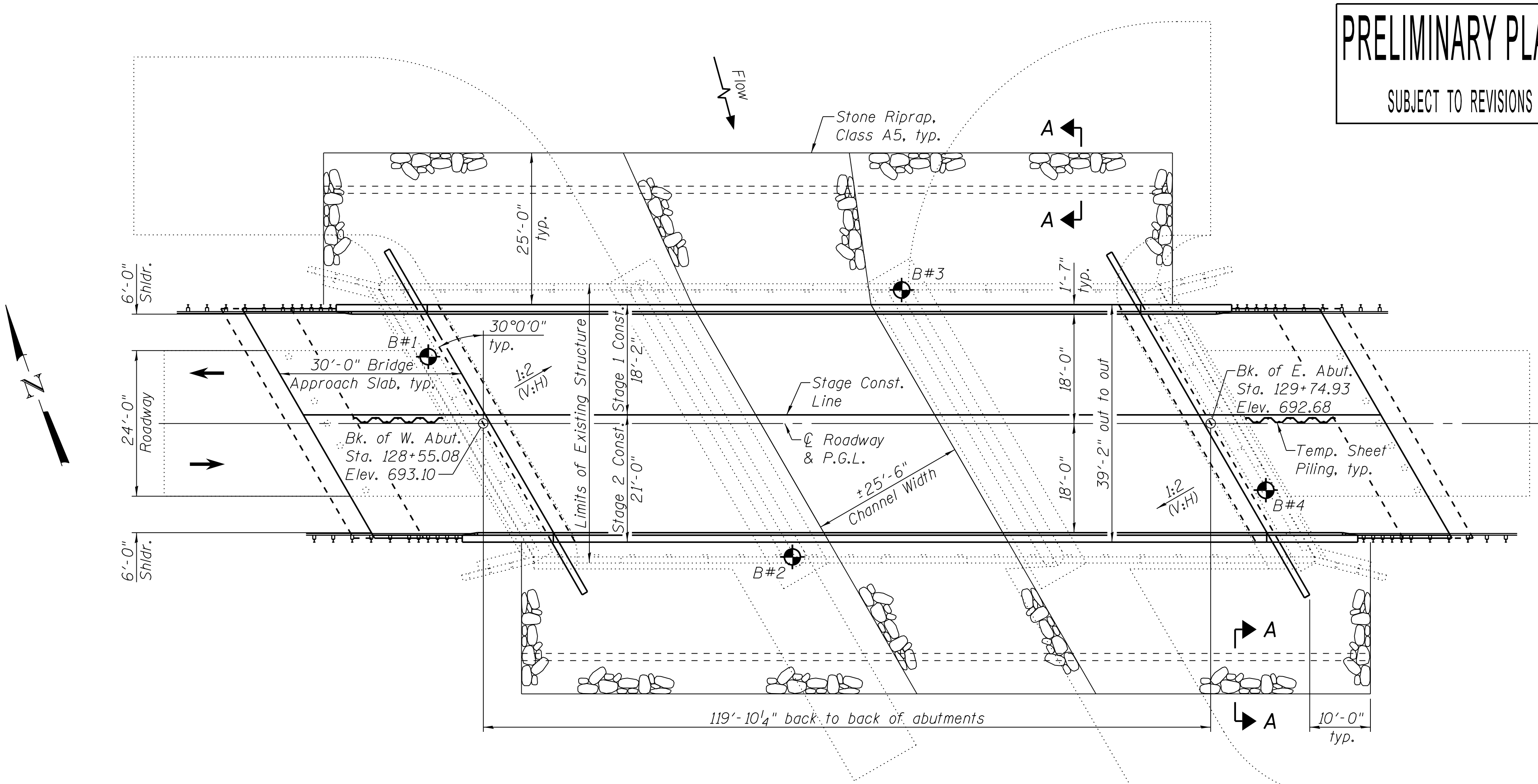
SECTION A-A



ELEVATION

PRELIMINARY PLANS

SUBJECT TO REVISIONS



PLAN

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

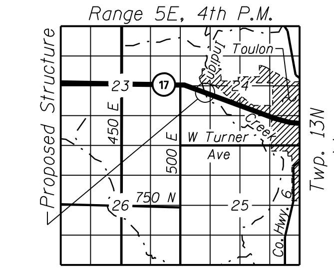
DESIGN STRESSES
FIELD UNITS

f'c = 3,500 psi
fy = 60,000 psi (Reinforcement)
fy = 50,000 psi (AASHTO M270 Grade 50)

HIGHWAY CLASSIFICATION
F.A.P. Rte. 643 - IL Rte. 17
Functional Class: Minor Arterial (Rural)
ADT: 1,650 (2013); 2000 (2033)
ADTT: 165 (2013); 200 (2033)
DHV: 165
Design Speed: 55 m.p.h.
Posted Speed: 55 m.p.h.
Two-Way Traffic
Directional Distribution: 50:50

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

SEISMIC DATA
Seismic Performance Zone (SPZ) = X
Design Spectral Acceleration at 1.0 sec. (SD1) = X.XXg
Design Spectral Acceleration at 0.2 sec. (SDS) = X.XXg
Soil Site Class = X



LOCATION SKETCH

WATERWAY INFORMATION

Drainage Area = 41.25 sq. mi.		Low Grade Elev. 691.6 at Sta. 132+30							
Flood	Freq.	Q	Opening Sq. Ft.		Nat.	Head - Ft.		Headwater El.	
	Yr.	C.F.S.	Exist.	Prop.	H.W.E.	Exist.	Prop.	Exist.	Prop.
	10	4,170	561	770	681.0	0.9	0.0	681.9	681.0
	25	5,530	620	835	681.7	1.6	0.0	683.3	681.7
Design	50	6,560	658	890	682.3	2.1	0.1	684.4	682.4
Base	100	7,660	690	940	682.8	2.6	0.2	685.4	683.0
Max. Calc.	500	10,300	756	1,010	683.5	5.0	1.7	688.5	685.2

DESIGN SCOUR ELEVATION TABLE

Design Scour Elevations (ft.)		
	W. Abut.	E. Abut.
Q100	683.1	682.7
Q500	683.1	682.7

GENERAL PLAN & ELEVATION
IL RTE. 17 OVER INDIAN CREEK
F.A.P. RTE. 643 - SEC. 14-BR-3
STARK COUNTY
STATION 129+15.00
STRUCTURE NO. 088-XXXX

FILE NAME = H:\P\29048\WO 5 SN088-0001 IL 17 over Indian Creek Phase 1\Structural\TSL\Microstation\088xxxx-68895-001-General Plan & Elevation.dgn

OATES ASSOCIATES Engineering + Architecture
ILLINOIS DESIGN FIRM LICENSE NO: 184.001115

USER NAME =	DESIGNED - JAD	REVISED -
	CHECKED - SJN	REVISED -
PLOT SCALE =	DRAWN - JAD	REVISED -
PLOT DATE = 8/29/2014	CHECKED - SJN	REVISED -

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

SHEET NO. 1 OF 2 SHEETS

F.A.P. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
643	14-BR-3	STARK		

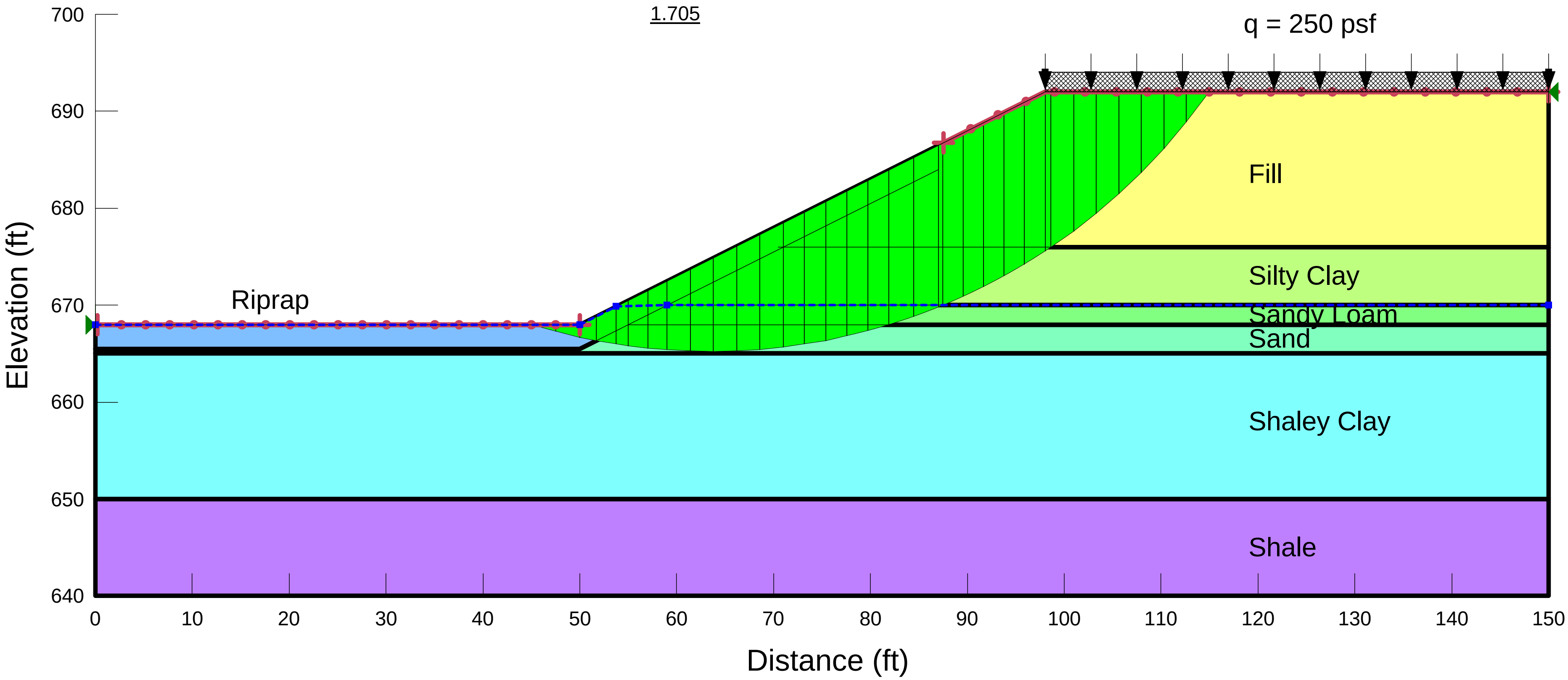
CONTRACT NO. 68895

ILLINOIS FED. AID PROJECT

Appendix D

2009-3210.53: PTB 153, WO 5 IL-17 over Indian Creek

East Abutment - Short Term



Name: Fill Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 1,000 psf Phi': 0 °

Name: Silty Clay Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 1,000 psf Phi': 0 °

Name: Sandy Loam Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 500 psf Phi': 0 °

Name: Sand Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 0 psf Phi': 30 °

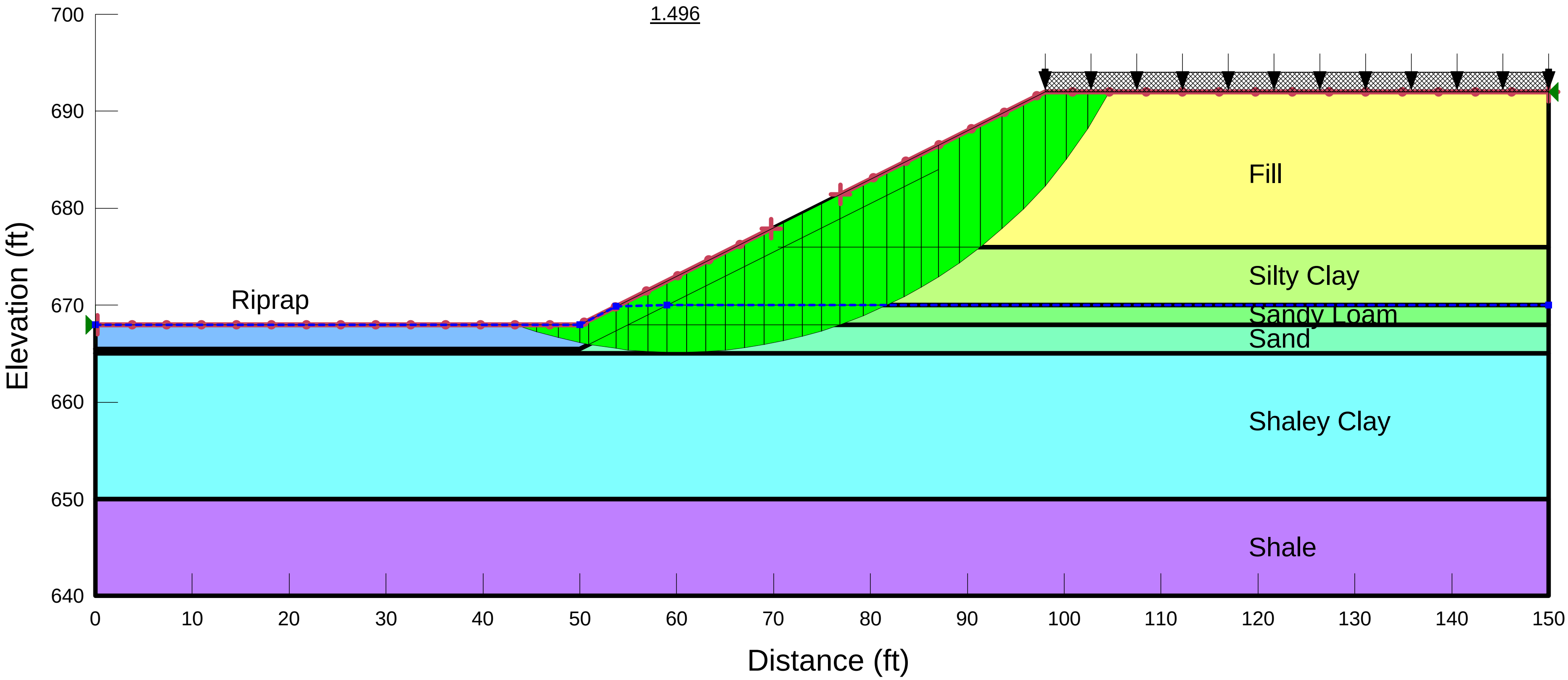
Name: Shaley Clay Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 2,000 psf Phi': 0 °

Name: Rip Rap Model: Mohr-Coulomb Unit Weight: 135 pcf Cohesion': 0 psf Phi': 38 °

Name: Shale Model: Bedrock (Impenetrable)

2009-3210.53: PTB 153, WO 5 IL-17 over Indian Creek

East Abutment - Long Term



Name: Fill Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 250 psf Phi': 26 °
Name: Silty Clay Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 250 psf Phi': 26 °
Name: Sandy Loam Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 100 psf Phi': 24 °
Name: Sand Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion': 0 psf Phi': 30 °
Name: Shaley Clay Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 500 psf Phi': 15 °
Name: Rip Rap Model: Mohr-Coulomb Unit Weight: 135 pcf Cohesion': 0 psf Phi': 38 °
Name: Shale Model: Bedrock (Impenetrable)

Appendix E

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============================== West Abut
REFERENCE BORING ========================== B-1
LRFD or ASD or SEISMIC ======================== LRFD
PILE CUTOFF ELEV. =========================== 685.10 ft
GROUND SURFACE ELEV. AGAINST PILE DURING DR 683.10 ft
GEOTECHNICAL LOSS TYPE (None, Scour, Liquef., DD None
BOTTOM ELEV. OF SCOUR, LIQUEF., or DD ======== 683.10 ft
TOP ELEV. OF LIQUEF. (so layers above apply DD) ============ ft

TOTAL FACTORED SUBSTRUCTURE LOAD ======== 1600 kips
TOTAL LENGTH OF SUBSTRUCTURE (along skew)=== 119.85 ft
NUMBER OF ROWS OF PILES PER SUBSTRUCTURE = 10
Approx. Factored Loading Applied per pile at 8 ft. Cts ===== 10.68 KIPS
Approx. Factored Loading Applied per pile at 3 ft. Cts ===== 4.01 KIPS

PILE TYPE AND SIZE ========== Steel HP 8 X 36
Plugged Pile Perimeter==================== 2.695 FT. Unplugged Pile Perimeter=========== 3.892 FT.
Plugged Pile End Bearing Area============== 0.454 SQFT. Unplugged Pile End Bearing Area===== 0.074 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
286 KIPS	**286** KIPS	**157** KIPS	**32** 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.)
680.10	3.00	2.00			9.4		22.1	13.6		***15.7***	16	0	0	9	5
677.90	2.20	2.00			6.9	12.7	***21.4***	10.0	2.1	24.4	21	0	0	12	7
674.90	3.00		6	Very Fine Silty Sand	0.8	5.1	33.6	1.1	0.8	***27.3***	27	0	0	15	10
672.40	2.50	2.60	8		9.3	16.5	***28.1***	13.5	2.7	38.4	28	0	0	15	13
669.90	2.50		2	Very Fine Silty Sand	0.2	1.7	***30.0***	0.3	0.3	39.0	30	0	0	16	15
667.40	2.50		4	Very Fine Silty Sand	0.4	3.4	***36.1***	0.6	0.6	40.5	36	0	0	20	18
664.90	2.50		8	Medium Sand	1.0	9.0	45.0	1.4	1.5	***43.2***	43	0	0	24	20
662.40	2.50		15	Medium Sand	1.8	17.0	49.3	2.7	2.8	***46.3***	46	0	0	25	23
660.90	1.50		23	Hard Till	1.0	19.5	50.4	1.5	3.2	***47.7***	48	0	0	26	24
659.90	1.00		23	Hard Till	0.7	19.5	88.1	1.0	3.2	***54.7***	55	0	0	30	25
658.90	1.00		38	Shale	33.6	56.5	121.6	48.5	9.2	***103.2***	103	0	0	57	26.2
657.90	1.00		38	Shale	33.6	56.5	155.2	48.5	9.2	***151.7***	152	0	0	83	27.2
657.40	0.50		38	Shale	16.8	56.5	***172.0***	24.2	9.2	175.9	172	0	0	95	27.7
656.40	1.00		100	Shale	33.6	56.5	***205.6***	48.5	9.2	224.4	206	0	0	113	28.7
655.40	1.00		100	Shale	33.6	56.5	***239.1***	48.5	9.2	272.9	239	0	0	132	29.7
654.40	1.00		100	Shale	33.6	56.5	***272.7***	48.5	9.2	321.3	273	0	0	150	30.7
653.40	1.00		100	Shale	33.6	56.5	***334.6***	48.5	9.2	374.4	~~335~~	~~0~~	~~0~~	~~184~~	~~31.7~~
652.40	1.00		100	Hard Till	5.8	84.8	***340.4***	8.4	13.8	382.8	~~340~~	~~0~~	~~0~~	~~187~~	~~33~~
651.40	1.00		100	Hard Till	5.8	84.8	***346.2***	8.4	13.8	391.2	~~346~~	~~0~~	~~0~~	~~190~~	~~34~~
650.40	1.00		100	Hard Till	5.8	84.8	***352.0***	8.4	13.8	399.6	~~352~~	~~0~~	~~0~~	~~194~~	~~35~~
649.40	1.00		100	Hard Till	5.8	84.8	***357.8***	8.4	13.8	407.9	~~358~~	~~0~~	~~0~~	~~197~~	~~36~~
648.40	1.00		100	Hard Till	5.8	84.8	***335.3***	8.4	13.8	411.8	~~335~~	~~0~~	~~0~~	~~184~~	~~37~~
647.40	1.00		100	Shale	33.6	56.5	***425.5***	48.5	9.2	469.4	~~425~~	~~0~~	~~0~~	~~234~~	~~37.7~~
646.40	1.00		100	Limestone	67.1	113.1	***492.6***	97.0	18.3	566.4	~~493~~	~~0~~	~~0~~	~~271~~	~~38.7~~
645.90	0.50		100	Limestone	33.6	113.1	***526.2***	48.5	18.3	614.8	~~526~~	~~0~~	~~0~~	~~289~~	~~39.2~~
644.90	1.00		100	Limestone	67.1	113.1	***593.3***	97.0	18.3	711.8	~~593~~	~~0~~	~~0~~	~~326~~	~~40.2~~
644.40	0.50		100	Limestone		113.1			18.3						

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============================== East Abut
REFERENCE BORING ========================== B-4
LRFD or ASD or SEISMIC ======================== LRFD
PILE CUTOFF ELEV. =========================== 684.70 ft
GROUND SURFACE ELEV. AGAINST PILE DURING DR 682.70 ft
GEOTECHNICAL LOSS TYPE (None, Scour, Liquef., DD None
BOTTOM ELEV. OF SCOUR, LIQUEF., or DD ======== 682.70 ft
TOP ELEV. OF LIQUEF. (so layers above apply DD) ============ ft

TOTAL FACTORED SUBSTRUCTURE LOAD ======== 1600 kips
TOTAL LENGTH OF SUBSTRUCTURE (along skew)=== 119.85 ft
NUMBER OF ROWS OF PILES PER SUBSTRUCTURE = 10
Approx. Factored Loading Applied per pile at 8 ft. Cts ===== 10.68 KIPS
Approx. Factored Loading Applied per pile at 3 ft. Cts ===== 4.01 KIPS

PILE TYPE AND SIZE ========== Steel HP 8 X 36
Plugged Pile Perimeter==================== 2.695 FT. Unplugged Pile Perimeter========== 3.892 FT.
Plugged Pile End Bearing Area============= 0.454 SQFT. Unplugged Pile End Bearing Area===== 0.074 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
286 KIPS	**286** KIPS	**157** KIPS	**33** 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.)
679.70	3.00	2.00			9.4		22.1	13.6		15.7	16	0	0	9	5
676.20	3.50	2.00			11.0	12.7	35.0	15.9	2.1	31.8	32	0	0	17	9
673.20	3.00	2.30	7		10.3	14.6	38.3	14.9	2.4	45.6	38	0	0	21	12
670.70	2.50	1.20	7		5.5	7.6	38.1	8.0	1.2	52.6	38	0	0	21	14
668.20	2.50	0.30	3		1.6	1.9	49.1	2.4	0.3	56.5	49	0	0	27	17
665.70	2.50		10	Medium Sand	1.2	11.3	49.2	1.8	1.8	58.1	49	0	0	27	19
663.20	2.50	1.60	36		6.8	10.2	68.7	9.8	1.7	69.9	69	0	0	38	22
660.70	2.50		27	Hard Till	2.0	22.9	61.2	2.9	3.7	71.2	61	0	0	34	24
658.20	2.50	2.10	84		8.1	13.4	112.5	11.7	2.2	89.9	90	0	0	49	27
657.20	1.00		116	Shale	33.6	56.5	146.0	48.5	9.2	138.4	138	0	0	76	27.5
656.20	1.00		116	Shale	33.6	56.5	179.6	48.5	9.2	186.9	180	0	0	99	28.5
655.20	1.00		116	Shale	33.6	56.5	213.2	48.5	9.2	235.3	213	0	0	117	29.5
654.20	1.00		116	Shale	33.6	56.5	246.7	48.5	9.2	283.8	247	0	0	136	30.5
653.20	1.00		87	Shale	33.6	56.5	280.3	48.5	9.2	332.3	280	0	0	154	31.5
652.20	1.00		87	Shale	33.6	56.5	313.9	48.5	9.2	380.8	~~314~~	~~0~~	~~0~~	~~173~~	~~32.5~~
651.20	1.00		87	Shale	33.6	56.5	347.5	48.5	9.2	429.3	~~347~~	~~0~~	~~0~~	~~191~~	~~33.5~~
650.20	1.00		87	Shale	33.6	56.5	381.0	48.5	9.2	477.7	~~381~~	~~0~~	~~0~~	~~210~~	~~34.5~~
649.20	1.00		100	Shale	33.6	56.5	414.6	48.5	9.2	526.2	~~415~~	~~0~~	~~0~~	~~228~~	~~35.5~~
648.20	1.00		100	Shale	33.6	56.5	448.2	48.5	9.2	574.7	~~448~~	~~0~~	~~0~~	~~246~~	~~36.5~~
647.20	1.00		100	Shale	33.6	56.5	481.7	48.5	9.2	623.2	~~482~~	~~0~~	~~0~~	~~265~~	~~37.5~~
646.20	1.00		100	Shale	33.6	56.5	515.3	48.5	9.2	671.7	~~515~~	~~0~~	~~0~~	~~283~~	~~38.5~~
645.20	1.00		100	Shale	33.6	56.5	548.9	48.5	9.2	720.1	~~549~~	~~0~~	~~0~~	~~302~~	~~39.5~~
644.20	1.00		100	Shale	33.6	56.5	582.5	48.5	9.2	768.6	~~582~~	~~0~~	~~0~~	~~320~~	~~40.5~~
643.20	1.00		114	Shale	33.6	56.5	616.0	48.5	9.2	817.1	~~616~~	~~0~~	~~0~~	~~339~~	~~41.5~~
642.20	1.00		114	Shale	33.6	56.5	649.6	48.5	9.2	865.6	~~650~~	~~0~~	~~0~~	~~357~~	~~42.5~~
641.20	1.00		114	Shale	33.6	56.5	683.2	48.5	9.2	914.1	~~683~~	~~0~~	~~0~~	~~376~~	~~43.5~~
640.20	1.00		114	Shale	33.6	56.5	716.8	48.5	9.2	962.5	~~717~~	~~0~~	~~0~~	~~394~~	~~44.5~~
639.20	1.00		100	Shale	33.6	56.5	750.3	48.5	9.2	1011.0	~~750~~	~~0~~	~~0~~	~~413~~	~~45.5~~
638.20	1.00		100	Shale	33.6	56.5	783.9	48.5	9.2	1059.5	~~784~~	~~0~~	~~0~~	~~431~~	~~46.5~~
637.20	1.00		100	Shale	33.6	56.5	817.5	48.5	9.2	1108.0	~~817~~	~~0~~	~~0~~	~~450~~	~~47.5~~
636.20	1.00		100	Shale	33.6	56.5	851.0	48.5	9.2	1156.4	~~851~~	~~0~~	~~0~~	~~468~~	~~48.5~~
635.20	1.00		100	Shale		56.5			9.2						

Pile Design Table for West Abut utilizing Boring #B-1

	Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 8 X 36			
	16	9	5
	21	12	7
	27	15	10
	28	15	13
	30	16	15
	36	20	18
	43	24	20
	46	25	23
	48	26	24
	55	30	25
	286	157	32
Steel HP 10 X 42			
	19	11	5
	28	15	7
	34	19	10
	35	19	13
	38	21	15
	47	26	18
	54	30	20
	57	32	23
	59	33	24
	68	37	25
	335	184	31
Steel HP 10 X 57			
	20	11	5
	28	16	7
	35	19	10
	36	20	13
	39	21	15
	48	26	18
	55	30	20
	59	32	23
	61	34	24
	72	39	25
	454	250	32
	454	250	38
Steel HP 12 X 53			
	23	13	5
	35	19	7
	41	22	10
	43	23	13
	46	26	15
	59	33	18
	64	35	20
	69	38	23
	71	39	24
	81	45	25
	418	230	31

	Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 12 X 63			
	24	13	5
	35	19	7
	42	23	10
	43	24	13
	47	26	15
	60	33	18
	66	36	20
	71	39	23
	73	40	24
	85	47	25
	497	273	32
Steel HP 12 X 74			
	25	14	5
	36	20	7
	43	24	10
	43	24	13
	48	26	15
	61	34	18
	67	37	20
	72	40	23
	74	41	24
	88	48	25
	589	324	32
	589	324	38
Steel HP 12 X 84			
	26	14	5
	37	20	7
	44	24	10
	44	24	13
	48	27	15
	62	34	18
	68	37	20
	73	40	23
	75	41	24
	91	50	25
	555	305	37
	664	365	38
Steel HP 14 X 73			
	29	16	5
	43	24	7
	50	27	10
	51	28	13
	57	31	15
	73	40	18
	78	43	20
	84	46	23
	86	47	24
	100	55	25
	578	318	32

	Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 14 X 89			
	30	16	5
	44	24	7
	51	28	10
	52	28	13
	57	32	15
	74	41	18
	80	44	20
	86	47	23
	88	49	24
	105	58	25
	705	388	32
	705	388	38
Steel HP 14 X 102			
	30	17	5
	44	24	7
	52	29	10
	52	29	13
	58	32	15
	75	41	18
	81	45	20
	87	48	23
	90	49	24
	109	60	25
	676	372	37
	810	445	38
Steel HP 14 X 117			
	32	17	5
	45	25	7
	53	29	13
	59	32	15
	76	42	18
	83	46	20
	89	49	23
	92	50	24
	113	62	25
	685	377	37
	929	511	39

Pile Design Table for East Abut utilizing Boring #B-4

	Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 8 X 36			
	16	9	5
	32	17	9
	38	21	14
	49	27	17
	49	27	19
	61	34	24
	90	49	27
	286	157	33
Steel HP 10 X 42			
	19	11	5
	40	22	9
	47	26	14
	63	35	19
	79	43	24
	112	61	27
	335	184	32
Steel HP 10 X 57			
	20	11	5
	41	22	9
	48	27	14
	65	36	19
	81	44	24
	116	64	27
	454	250	35
Steel HP 12 X 53			
	23	13	5
	47	26	9
	57	32	14
	80	44	19
	99	55	24
	134	74	27
	418	230	32

	Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 12 X 63			
	24	13	5
	49	27	9
	58	32	14
	80	44	19
	100	55	24
	138	76	27
	497	273	33
Steel HP 12 X 74			
	25	14	5
	50	27	9
	59	32	14
	82	45	19
	102	56	24
	141	78	27
	589	324	35
Steel HP 12 X 84			
	26	14	5
	50	28	9
	60	33	14
	83	46	19
	104	57	24
	145	80	27
	664	365	36
Steel HP 14 X 73			
	29	16	5
	58	32	9
	69	38	14
	99	54	19
	124	68	24
	163	90	27
	578	318	33

	Nominal Required Bearing (Kips)	Factored Resistance Available (Kips)	Estimated Pile Length (Ft.)
Steel HP 14 X 89			
	30	16	5
	59	32	9
	70	38	14
	100	55	19
	126	69	24
	169	93	27
	705	388	35
Steel HP 14 X 102			
	30	17	5
	60	33	9
	71	39	14
	102	56	19
	128	70	24
	172	95	27
	810	445	36
Steel HP 14 X 117			
	32	17	5
	62	34	9
	71	39	14
	103	57	19
	129	71	24
	177	98	27
	929	511	38

APPENDIX E

PROJECT: FAP 646 (IL40 / IL17) over Indian Creek
LOCATION: Stark County, Illinois
CLIENT: Oates Associates, Inc.
STRUCTURE: 088-0001 (EXISTING); 088-0032 (PROPOSED)
SCI NO.: 2009-3210.53

Table E.1 – Estimated Maximum Driving Elevations for West Abutment (B-1)

Pile Type and Size	Estimated Pile Length (ft)	Estimated Refusal Elevation (ft)
HP 8 X 36	32	653.1
HP 10 X 42	31	654.1
HP 10 X 57	38	647.1
HP 12 X 53	31	654.1
HP 12 X 63	32	653.1
HP 12 X 74	38	647.1
HP 12 X 84	38	647.1
HP 14 X 73	32	653.1
HP 14 X 89	38	647.1
HP 14 X 102	38	647.1
HP 14 X 117	39	646.1

Table E.2 – Estimated Maximum Driving Elevations for East Abutment (B-4)

Pile Type and Size	Estimated Pile Length (ft)	Estimated Refusal Elevation (ft)
HP 8 X 36	33	651.7
HP 10 X 42	32	652.7
HP 10 X 57	35	649.7
HP 12 X 53	32	652.7
HP 12 X 63	33	651.7
HP 12 X 74	35	649.7
HP 12 X 84	36	648.7
HP 14 X 73	33	651.7
HP 14 X 89	35	649.7
HP 14 X 102	36	648.7
HP 14 X 117	38	646.7

Appendix F

APPENDIX F

PROJECT: FAP 646 (IL40 / IL17) over Indian Creek
LOCATION: Stark County, Illinois
CLIENT: Oates Associates, Inc.
STRUCTURE: 088-0001 (EXISTING); 088-0032 (PROPOSED)
SCI NO.: 2009-3210.53

Table F.1 – Soil Modulus Parameters (k) for B-1 (West Abutment)

Depth (ft)	Elevation (ft)	Abbreviated Soil Description	Effective Unit Weight (pcf)	Cohesion (tsf)	Phi (degrees)	Soil Modulus Parameter (pci)	E_{50}	k_{rm}
0 – 15.2	693.1 – 677.9	Fill	120	1.0	0	500	0.005	--
15.2 – 28.7	677.9 – 667.4	Silty Loam to Silty Clay Loam	121	1.2	0	500	0.005	--
28.7 – 25.7	670.4 – 667.4	Silty Loam to Silty Clay Loam	58.6	1.2	0	500	0.005	--
25.7 – 30.7	667.4 – 662.4	Sand and Gravel	57.6	--	35	40	--	--
30.7 – 33.2	662.4 – 659.9	Clay with sand and gravel	77.6	2.5	0	1000	0.005	--
33.2 – 45.7	659.9 – 647.4	Shale / Shaley Clay	87.6	4.1	0	2000	0.004	--
45.7 +	647.4 +	Limestone	150	50.0	0	--	--	0.00005

Table F.2 – Soil Modulus Parameters (k) for B-4 (East Abutment)

Depth (ft)	Elevation (ft)	Abbreviated Soil Description	Effective Unit Weight (pcf)	Cohesion (tsf)	Phi (degrees)	Soil Modulus Parameter (pci)	E_{50}	k_{rm}
0 – 16.5	692.7 – 676.2	Fill	120	1.0	0	500	0.005	--
16.5 – 19.5	676.2 – 673.2	Silty Clay Loam	120	1.5	0	500	0.007	--
19.5 – 22.0	673.2 – 670.7	Silty Clay	58.6	1.0	0	100	0.007	--
22.0 – 24.5	670.7 – 668.2	Sandy Loam	58.6	0.49	0	30	0.007	--
24.5 – 27.0	668.2 – 665.7	Sand and Gravel	57.6	--	35	20	--	--
27.0 – 34.5	665.7 – 658.2	Shaley Clay	77.6	1.9	0	200	0.007	--
34.5 – 48.0	658.2 – 645.7	Shale / Siltstone	87.6	3.9	0	1000	0.005	--
48.0 +	645.7 +	Shale	87.6	7.0	0	2000	0.004	--

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.

ASFE
THE BEST PEOPLE ON EARTH

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