Structure Geotechnical Report

TR 93A, Bluegrass Lane over
Spring Creek Replacement Structure
Existing S.N. 039-0046
Replacement S.N. 039-0081
Jackson County, Illinois
PTB 168-023

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1.0 Project Description and Proposed Structure Information

1.1 Introduction

This report summarizes the results of a geotechnical investigation performed for the design of a replacement structure for the existing bridge carrying TR 93A, Bluegrass Lane over Spring Creek south of Ava, Jackson County, Illinois. The purpose of this study was to provide a geotechnical assessment of the planned replacement structure, based on subsurface conditions encountered at two borings performed by the Illinois Department of Transportation (IDOT) at the existing structure. The borings were drilled on September 28, 2017. This report describes the exploration procedures used, presents the field and laboratory data, includes an assessment of the subsurface conditions in the area, and provides geotechnical recommendations for the construction.

1.2 Project Description

The project consists of the removal and replacement of the existing structure carrying TR 93A, Bluegrass Lane over Spring Creek in Jackson County, Illinois. The general site area is shown on the attached Vicinity Map, Figure 1 in Appendix A. A plan that shows the approximate locations of the borings performed for this study is presented as the Site and Boring Location Plan, Figure 2 in Appendix A. Spring Creek is oriented north and south beneath the existing Bluegrass Creek bridge and flows southward. The back-to-back abutment length of the existing bridge 30 feet and 0 inches and the out-to-out deck width is 16 feet and 0 inches. It has a skew angle of 0 degrees and a skew direction of north. It was originally built in 1935 as part of S.A. Route 10, Section 13A-NRH, and consists of a single span, steel wide flange beam superstructure with a timber deck. It is supported by closed timber column abutments on timber piles. It is our understanding that the existing structure will be replaced with a new single-span bridge on pile bent abutments. Based on the information provided, it appears the existing road will remain open during construction.

1.3 Proposed Structure Information

The proposed structure will be located approximately 40 feet north of the existing structure, and will consist of a single span with nine 27-inch by 36-inch Polyester Polymer Concrete (PPC) deck beams with a hot mix asphalt (HMA) wearing surface, on spill-thru pile bent abutments. It will have a 27-foot-0-inch out-to-out width and be 56 feet-0 inches back-to-back at the abutments. The bridge side slopes are planned to be 2 horizontal to 1 vertical (2H:1V). We understand that the factored loads for the structure are approximately 700 kips per abutment. The new roadway profile is anticipated to remain essentially unchanged from the existing structure, with less than 5 feet of new fill required for the new approachway embankments.

2.0 Subsurface Exploration and Laboratory Testing

2.1 Subsurface Exploration

The field exploration for this project was conducted by IDOT. The exploration consisted of completing two soil borings within the roadway pavement, with one boring on the east side and one on the west side of the existing bridge. The borings were designated as Borings 1-S and 2-S. The approximate locations of the borings are shown on the Site and Boring Location Plan, Figure 2 in Appendix A.

The two borings for this study were completed on September 28, 2017. Boring 2-S was located 18 feet east of the existing bridge east abutment, offset 10 feet north of the roadway centerline. Boring 1-S was located just 5 feet west of the existing bridge west abutment, offset 3 feet north of the roadway centerline. Each boring was augered through the pavement section and base rock, and then advanced to the top of intact bedrock at depths of 10.5 and 10.0 feet for Borings 1-S and 2-S, respectively, using hollow-stem auger drilling equipment. Split-spoon (SPT) samples were obtained on 2.5-foot centers in the overburden soils. Boring 1-S was advanced into the underlying bedrock using the hollow-stem auger equipment to a termination depth of 18.1 feet; Boring 2-S was extended using this equipment to a termination depth of 17.3 feet. The sampling sequence for each boring is summarized on the Logs of Boring in Appendix B of this report.

Unconfined compression tests were performed in the field on selected split-spoon samples using a Rimac field testing machine. The resulting unconfined compressive strengths are reported on the Logs of Boring.

2.2 Laboratory Testing

A laboratory testing program consisting of natural moisture contents was conducted by IDOT to determine selected engineering properties of the obtained soil samples. The results of the individual tests are presented on the boring logs in Appendix B.

3.0 SUBSURFACE CONDITIONS

Details of the subsurface conditions encountered at the borings are shown on the boring logs. The general subsurface conditions encountered and their pertinent engineering characteristics are described in the following paragraphs. Conditions represented by the borings should be considered applicable only at the boring locations on the date shown; the reported conditions may be different at other locations and at other times.

3.1 GEOLOGY

The site lies in the Shawnee Hills section of the Interior Low Plateaus Province, in a moderately dissected upland area approximately 6 miles north of the Mississippi River bluffs. The area is drained by Spring Creek, a tributary to Kinkaid Creek, and flows north to south into the upper reaches of Kinkaid Lake. The site is underlain by shallow Pennsylvanian bedrock consisting of the Tradewater Formation. This unit is made up of shale, with limestone and sandstone layers and occasional coal seams. Locally, the Murphysville coal seam is the most likely host seam for mining in the area.

3.2 GENERALIZED SUBSURFACE PROFILE

The natural soils at the site are predominantly made up of clay to silty clay and silty clay loam, with AASHTO classifications of A7-6 and A-4. Moisture contents vary from 15 to 23%. The standard penetration test (N) values range from 4 to 7 blows per foot (bpf), for a soft to medium stiff consistency. Rimac unconfined compression test values on samples range from 0.4 to 2.5 tons per square foot (tsf). Although there is a narrow range in N-values, Rimac values change at depths below 8 feet. Between 0 to 8 feet below the ground surface, Rimac values range from 1.6 to 2.5 tsf; from 8 feet to auger refusal at depths of 10 to 10.5 feet, the Rimac values are 0.5 tsf in Boring 1-S and 0.4 tsf in Boring 2-S. A 2.5 foot layer of loose sand was encountered at a depth of 8 feet in Boring 1-S.

Shale was encountered at Elevation 505.5 in Boring 1-S and 505.2 in Boring 2-S. It is described as hard, dry, gray shale with N-values of 100 blows for penetrations of 1.5 to 8 inches. A generalized subsurface profile is shown in Appendix A, Figure 3.

3.3 GROUNDWATER

At the time of drilling, groundwater was not observed in the borings. The presence or absence of groundwater at a particular location does not necessarily mean that groundwater will be present or absent at that location at other times. Groundwater levels may vary significantly over time due to the effect of seasonal variations in precipitation, the water level in Spring Creek, or other factors not evident at the time of exploration.

4.0 GEOTECHNICAL EVALUATIONS

4.1 EARTHWORK

Grade changes on the new approach embankments will be similar to those of the existing roadway, requiring on the order of 3 to no more than 5 feet of new fill for the new embankments. The new fill should be placed in horizontal layers rather than attempting to place and compact material on the slope. It is anticipated that the existing road will remain open during construction so that staged construction will not be required.

4.2 SETTLEMENT

Rimac tests in the upper portion of the soil profile indicate the material is relatively stiff. The lower Rimac strength values in the silt loam below could be moderately compressible. However, the N-values in this material are similar to those in the clays above. It is considered likely that the silty samples became disturbed during driving, yielding low Rimac values. Consequently, we do not expect significant settlement occurring from the placement of less than 5 feet of fill for the approachway embankments. In our opinion, downdrag should not be a concern at the abutments.

4.3 MINING ACTIVITY

A review of undermining was made using the Illinois State Geological Survey (ISGS) website for mapped coal mines in Jackson County, Illinois. Based on this information, the project site appears to be close to four underground mines, shown on Figure 4 in Appendix A. Their ISGS Mine Index Numbers are 2506, 7290, 7162, and 7379. Numbers 7290 and 2506 are in the Ava, IL, USGS 7.5 minute quadrangle; number 7162 is in the Oraville, IL, USGS 7.5 minute quadrangle; and number 7379 is in the Raddle, IL, USGS 7.5 minute quadrangle. There is little ISGS information regarding these mines, except that the Murphysboro coal was mined at a depth of about 18 feet, with an average thickness of 3 to 4 feet. There is no information on mined-out acreage. Mine Number 2506 was owned by the Schmidgall Coal Company. In 1936 when the mine was visited, it was filled with water. There were five active mines near Ava, IL whose locations are unknown. They were active from 1879 to 1941, producing a total of 1,191 tons. All known mines are located more than 1,500 feet from the project site, with the nearest being Mine Number 7379. Because of the operational depth of less than 20 feet and the distance from the site, this mine should not have an impact on the site.

4.4 SEISMICITY

Although several significant areas of seismic activity are present in the central United States, the site area is most directly affected by the Wabash Seismic Zone, located in south and east-central Illinois. An assessment of seismic criteria in accord with AASHTO 2009 Guide Specifications for LRFD Seismic Bridge Design has been performed for the site. The IDOT Spreadsheet "Seismic Site Class Determination" was used to determine a Soil Site Class C for the abutments. The United States Geological Survey (USGS) Design Maps Summary Report website was used with the Site Class C classification to provide acceleration coefficient values Sd_s of 0.664 g and

S_{d1} of 0.244 g. The results of the Site Class determination and the Design Maps Summary Report are presented in Appendix C. Based on the conditions encountered in the borings, liquefaction should not occur at this site.

For the purposes of this report, the bridge has been assumed to be classified as "Regular and Essential." In accordance with the IDOT Bridge Manual, 2012 Edition, the structure should be designed for a design earthquake with a 7% probability of exceedance over a 75-year exposure period (1,000-year return period). A Peak Ground Acceleration (PGA) value of 0.290 g was determined from data provided by the United States Geologic Survey (USGS) National Seismic Hazard Mapping Project.

Based on the guidelines in the IDOT All Geotechnical Manual Users (AGMU), including Table 3.15.2-1 in that manual, the Seismic Performance Zone is 3.

4.5 Scour

The scour should be assumed to be taken as 100 percent (%) of the scour predicted in the Hydraulic Report (0% reduction in scour depth). Abutment slope protection should be included to protect against scour potential. Countermeasure options for scour at bridge locations include webwalls to eliminate debris collection between columns, riprap, partially grouted riprap, geotextile sand containers, and sheet piling. Skin friction and lateral load design values for piers and driven piles should be ignored in the scour zone. The design scour elevations should correspond to the elevations of 510.1 feet for the west abutment and 509.6 feet for the east abutment, as determined from the information provided by Oates Associates, Inc. Based on information provided by Oates Associates, Inc., the design scour elevations for the 100-year and 200-year events for the bridges are shown in Table 1.

Table 1.
Summary of Design Scour Elevations

Event/Limit State	Design Scour Elevations (ft.)		Item 113
	W. Abut.	E. Abut.	
Q100	510.1	509.6	8
Q200	510.1	509.6	
Design	510.1	509.6	
Check	510.1	509.6	

4.6 Slope Stability

The new bridge is to be located 40 feet north of the existing bridge. A preliminary plan and profile along the proposed project centerline was provided by Oates Associates. There appears to be very little change between the proposed slope geometry and the existing geometry, suggesting minimal cut to be made. The existing and planned profiles are nearly the same from approximately elevation 512 (where the Spring Creek channel slopes meet the east and west abutments) to the bottom of Spring Creek (approximately elevation 503.5), which is a total vertical height of approximately 8.5 feet and a 1V:2H slope. Above elevation 512, fill will be placed against the abutments and along a flatter existing slope to accommodate the new roadway profile. The proposed profile also indicates top of slope elevations of approximately 517.0 just west of the west abutment and 516.6 just east of the east abutment, creating a total vertical slope height of 13.1 to 13.5 feet. Based on guidance in Chapter 6.5.1, Cut Slopes Stability, of the IDOT Geotechnical Design Manual (December 2015), a slope stability analysis is required for cut slopes greater than 15 feet in depth.

Performance history suggests that the existing slopes have been stable for 84 years, since the bridge was built in 1935. The Bridge Condition Report (BCR) reports that the riprap slope protection is still in place and in good condition, preventing scour from estimated historical flows approaching the calculated 15-year High Water Elevation. The borings indicate that the creek bottom is incised into the underlying shale bedrock. For these reasons, a slope stability analysis was not performed.

5.0 FOUNDATION EVALUATIONS AND DESIGN RECOMMENDATIONS

5.1 PILE FOUNDATIONS

The bridge structures may be supported on pile foundations set in drilled sockets penetrating into intact shale bedrock. Steel piles are considered to be feasible for use at this site. Since the piles will not be driven, the recommended axial pile capacities are based on drilled shaft analytical procedures outlined in FHWA-NHI-10-016 (Drilled Shafts Construction Procedures and LRFD Design Methods), Appendix B.1, rather than IDOT driven pile methods. The FHWA appendix outlines the use of N-values obtained in clay shales to estimate unit side and tip resistances. This method was applied conservatively to assess the design values for the shale encountered in the borings at this site. On this basis, TSi recommends a nominal unit side resistance of 7.5 kips per square foot (ksf) and a nominal unit tip resistance of 75.0 ksf for pile sections seated into sockets in intact shale. Geotechnical resistance factors of 0.6 for side resistance and 0.55 for tip resistance are considered appropriate for use with these values. The design is to be based on the dimensions of the rock socket, rather than the pile section. The socket diameters IDOT uses are 18 inches for HP-8 and HP-10 sections, and 24 inches for HP-12 and HP-14 sections. A minimum socket depth of 5 feet is recommended. Capacity reductions for liquefaction and downdrag do not apply at this site. Typical available and suitable pile sections are described in the IDOT Geotechnical Manual, Section 6.13.2, Pile Foundations. Final selection would be based on availability and structural requirements such as pile spacing, installation requirements, etc. The piles should be installed as described in Section 6.3 below.

The two borings for this study encountered intact shale bedrock at a consistent level, Elevation 505. This elevation may be used for design purposes; however, it should be noted that the centerline of the new structure will be located 33 and 40 feet upstream of the two borings, and some variation in that level may exist. Since the base of the rock socket will be several feet below the level of the creek bed, the groundwater will be at least at that level. For design purposes, the maximum anticipated groundwater level may be assumed to be the level of the 100-year High Water Elevation of 510.2.

5.2 LATERAL CAPACITY GEOTECHNICAL PARAMETERS

Lateral load resistance and induced lateral deflection are typically assessed using finite difference computer models based on the lateral modulus-of-subgrade reaction, such as LPILE 2012-06. Based on the conditions encountered in the borings, the parameters are estimated for use in the analysis of the lateral capacity as shown in Tables 2.1 and 2.2, using L-PILE Version 2012-06.

TABLE 2.1.
PARAMETERS FOR USE IN LPILE ANALYSIS AT EAST ABUTMENT

Elevation (ft.)	LPILE Soil Type	Effective Unit Weight (pcf)	Undrained Cohesion (psf)	Bedrock U.C. Strength (psi)	K_rm	Strain at 50% Maximum Stress	Angle of Internal Friction (degrees)	p-y Soil Modulus K (pci)	Rock Modulus, Er (psi)
510 - 508	Loose Sand	58	N/A	N/A	N/A	N/A	30	25	N/A
508 - 505	Soft to MediumClay	63	500	N/A	N/A	0.02	N/A	30	N/A
Below 505	Hard Shale	78	N/A	100	.0005	70*	N/A	N/A	4,000

pcf = pounds per cubic foot
psf = pounds per square foot
pci = pounds per cubic inch
* - Estimated RQD

TABLE 2.2.
PARAMETERS FOR USE IN LPILE ANALYSIS AT WEST ABUTMENT

Elevation (ft.)	LPILE Soil Type	Effective Unit Weight (pcf)	Undrained Cohesion (psf)	Bedrock U.C. Strength (psi)	K_rm	Strain at 50% Maximum Stress	Angle of Internal Friction (degrees)	p-y Soil Modulus K (pci)	Rock Modulus, Er (psi)
510 - 508	Medium Clay	63	1000	N/A	N/A	0.01	N/A	100	N/A
508 - 505	Soft to Medium Clay	63	500	N/A	N/A	0.02	N/A	30	N/A
Below 505	Hard Shale	78	N/A	100	.0005	70*	N/A	N/A	4,000

pcf = pounds per cubic foot
psf = pounds per square foot
pci = pounds per cubic inch
* - Estimated RQD

6.0 Construction Considerations

The construction activities should be performed in accordance with the current IDOT Standard Specifications for Road and Bridge Construction. Trenching, excavating, and bracing should be performed in accordance with Occupational Safety and Health Administration (OSHA) regulations, and other applicable regulatory agencies. In accordance with the OSHA excavation standards, the soil at the site is considered to be Type C, which requires a side slope for excavations no steeper than 1.5H:1.0V. However, worker safety and classification of the excavation soil is the responsibility of the contractor.

6.1 Subgrade Water Protection

Groundwater was not encountered in the borings, but this does not preclude the possibility that groundwater may be present during construction. A 2-foot layer of sand was encountered at Elevation 510 at the existing east abutment boring. Seasonal variations of rainfall could cause groundwater to become perched in this layer. If groundwater seepage occurs, it is anticipated it can be adequately dewatered using sump and pump methods.

6.2 Rock-Socketed Pile Installation

The rock-socketed piles are to be furnished and installed according to the requirements of IDOT Guide Bridge Special Provisions (GBSP) 56, revised April 1, 2016. This requires pre-drilling through the overburden and rock to satisfy the selected pile dimensions and embedment into rock sockets backfilled with concrete. GBSP 56 defines top of rock as "bedded deposits and conglomerate deposits exhibiting the physical characteristics and difficulty of rock removal…which cannot be drilled with earth augers and/or underreaming tools configured to be effective in the soils indicated in the contract documents, and requires the use of rock augers, core barrels, air tools, blasting, or other methods of hand excavation." TSi does not recommend blasting due to the chance of fracturing intact rock. The contractor is responsible for hole stability by using accepted drilling methods and temporary casing where conditions warrant. Permanent casings, side forms, or drilling mud is not allowed. Loose soil, rock, debris, and obstructions are to be removed before placing concrete. Obstructions are unknown, isolated objects not shown on the plans that cause significant decrease in shaft excavation production, requiring coring, breaking, pushing aside, or other means of bypassing. Obstructions should be reported to the Engineer so that a mutual solution of removal with the Contractor can be identified. If water flowing into the hole is excessive, or if pumping operations are likely to cause instability, the water level in the hole should be allowed to stabilize and the concrete placed by tremie methods according to Article 503.08 of the Standard Specifications. Prior to placing the H-pile in the hole, a minimum of 6 inches of Class SI Concrete should be placed. The H-pile must be braced and held in proper positioning until the remainder of the Class SI Concrete is placed to a level of a minimum of 6 inches above the top of rock. Operations that may damage the concrete during curing should be deferred until the concrete reaches a minimum modulus of rupture of 650 pounds per square inch (psi). Coring the concrete can be done between the H-Pile flanges. No additional concrete is to be placed until the required strength is reached. When the required strength is reached, the remainder of the hole to the bottom of the abutment should be filled with Class SI concrete or porous granular material that will not result in bridging.

Because the shale was not cored or tested, its rock quality is unknown. TSi recommends that at least one of the H-pile locations be pre-drilled to obtain and test core samples of the shale in the interval extending at least 5 feet below the planned bottom of the pile. As detailed in Section 5.1, each pile should be embedded a minimum of 5 feet into unweathered intact shale.

6.3 Subgrade, Fill and Backfill

Earthwork activities including backfill and fill should be performed in accordance with Section 205 of the Standard Specifications.

7.0 Report Limitations

This geotechnical report has been prepared for the exclusive use of **Oates Associates, Inc.** and the **Illinois Department of Transportation** for the specific application to the subject project. The information and recommendations contained in this report have been made in accordance with generally accepted geotechnical and foundation engineering practices; no other warranties are implied or expressed.

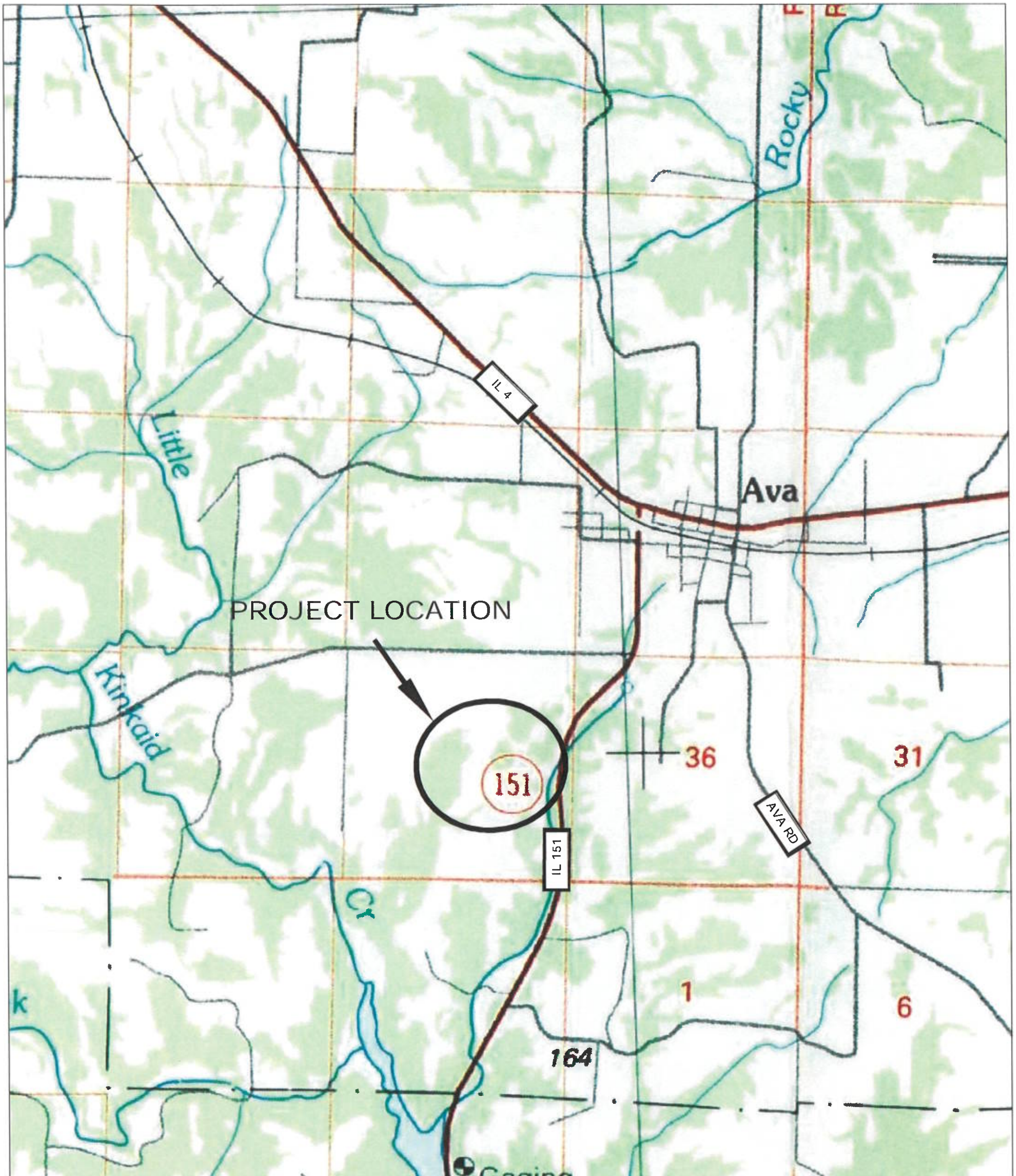
The assessments and recommendations submitted in this report are based in part upon the data obtained from the borings. The nature and extent of variations between the borings may not be evident at this time. If variations appear evident at a later date, it may be necessary to re-evaluate the recommendations of this report.

We emphasize that this report was prepared for design purposes only and may not be sufficient to prepare an accurate construction bid. Contractors reviewing this report should acknowledge that the information and recommendations contained herein are for design purposes.

If conditions at the site have changed due to natural causes or other operations, this report should be reviewed by TSi to determine the applicability of the analyses and recommendations considering the changed conditions. The report should also be reviewed by TSi if changes occur in the structure locations, sizes, and types, in the planned loads, elevations, grading and site development plans or the project concepts.

TSi requests the opportunity to review the final plans and specifications for the project prior to construction to verify that the recommendations in this report are properly interpreted and incorporated in the design and construction documents. If TSi is not accorded the opportunity to make this recommended review, we can assume no responsibility for the misinterpretation of our recommendations.

APPENDIX A



NOT TO SCALE

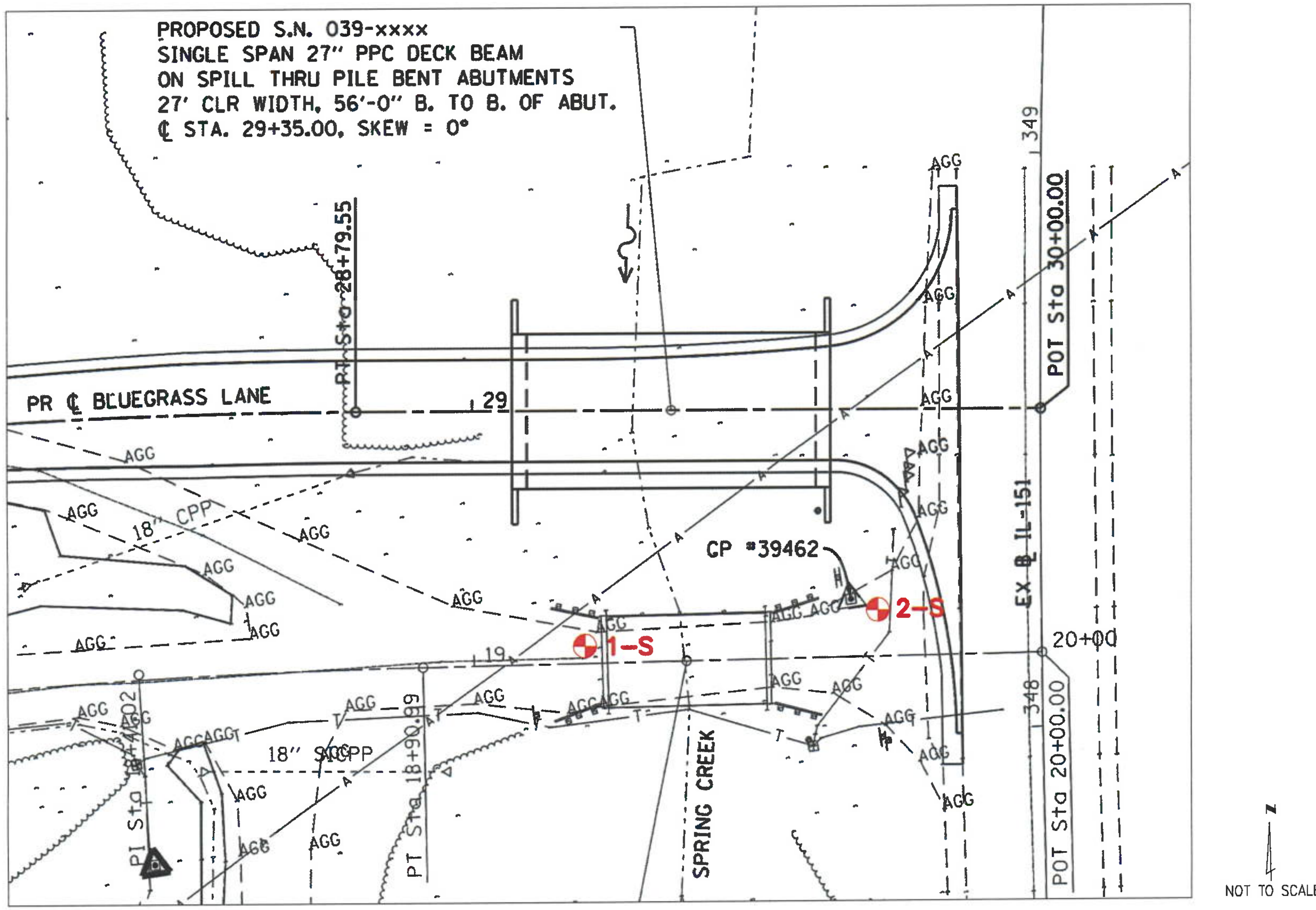
NOTE:
DRAWING PREPARED FROM AN IMAGE OBTAINED FROM USGS NATIONAL MAP VIEWER ON 07/09/18

TSi geotechnical, inc.
1340 NORTH PRICE ROAD
ST. LOUIS, MISSOURI 63132

VICINITY MAP

TR 93A (BLUEGRASS DR.) OVER SPRING CREEK
JACKSON COUNTY, ILLINOIS

Drawn By: SLY	Checked By: ACE
Project No. 20185023.00	Date: 07/11/18 Figure 1



LEGEND

1-S ⊕ APPROXIMATE BORING LOCATION AND NUMBER

NOTE: DRAWING PREPARED FROM IMAGE OBTAINED FROM OATES ASSOCIATES ON 12/18/17

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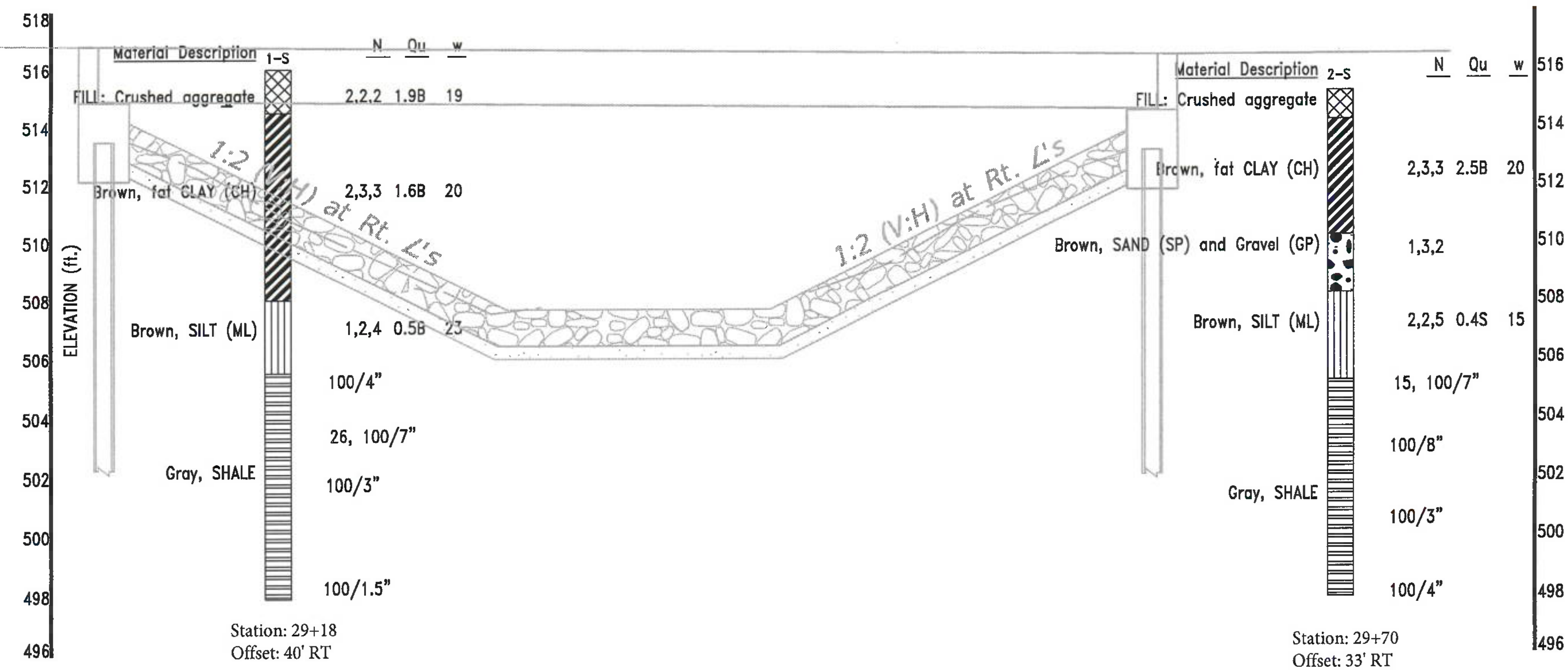
1340 NORTH PRICE ROAD
ST. LOUIS, MISSOURI 63132

SITE AND BORING LOCATION PLAN		
TR 93A (BLUEGRASS DR.) OVER SPRING CREEK JACKSON COUNTY, ILLINOIS		
Drawn By: SLY	Checked By: ACE	
Pro'ect No. 20185023.00	Date: 07/12/18	Figure 2



PROFILE KEY:

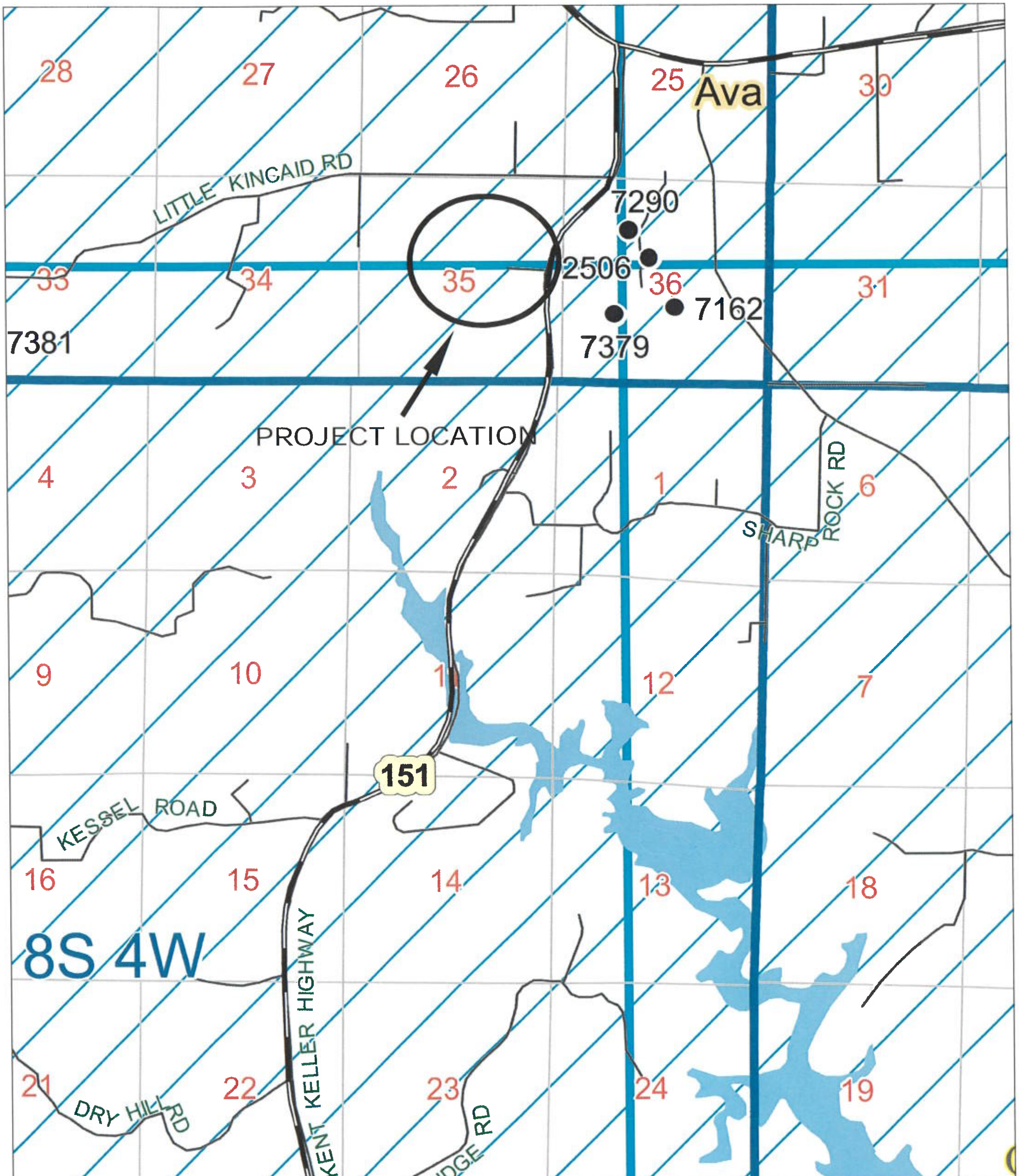
BORING — N: SPT BLOW-COUNT — Qu: RIMAC/PP VALUE (TSF) — w%: MOISTURE CONTENT

VERTICAL SCALE: 1" = 2'

NOTE: THIS PLAN WAS PREPARED FROM DRAWINGS RECEIVED FROM OATES ASSOCIATES on 07/23/2018. BORING LOCATIONS SHOWN ON PROFILE ARE APPROXIMATE.

TSi engineering, Inc.
5850 ARSENAL STREET
ST. LOUIS, MISSOURI 63139

SOIL PROFILE	
TR 93A (BLUEGRASS LANE) OVER SPRING CREEK JACKSON COUNTY, ILLINOIS	
Drawn By: ST	Checked By: FHH
Project No. 20185023.00	Date: 07/27/2018 — Figure 3



NOT TO SCALE

N

NOTE:
DRAWING PREPARED FROM AN IMAGE OBTAINED FROM ISGS JACKSON COUNTY COAL MAP ON 07/09/18

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1340 NORTH PRICE ROAD
ST. LOUIS, MISSOURI 63132

VICINITY MAP

TR 93A (BLUEGRASS DR.) OVER SPRING CREEK
JACKSON COUNTY, ILLINOIS

Drawn By: SLY	Checked By: ACE
Project No. 20185023.00	Date: 07/09/18 — Figure 3

APPENDIX B

ILLINOIS DEPARTMENT OF TRANSPORTATION
District Nine Materials

Bridge Foundation
Boring Log

TR 93A off IL 151 (Bluegrass Lane) Over Spring Creek

Sheet 1 of 1

Route: Bluegrass Lane   Structure Number: 039-0046   Date: 9/28/2017

Section ______   Bored By: R Moberly

County: Jackson   Location: 6.7 mi North of IL Rte 3   Checked By: A Hayes

Boring No 1-S
Station 5' W of W Abut
Offset 3' N CL
Ground Surface 516.0 Ft

Surf Wat Elev: 503.3
Ground Water Elevation
when Drilling ______
At Completion ______
At: Hrs:

Description	Elevation	DEPTH	BLOWS	Qu tsf	W%
Crushed aggregate					
	514.5				
Stiff, moist, brown, Clay to Silty Clay A7-6					
			2		
			2	1.9B	19
			2		
		5.0			
			2		
			3	1.6B	20
			3		
	508.0				
Soft to Medium, very moist, Brown, Silty Clay Loam A-4			1		
			2	0.5B	23
			4		
		10.0			
	505.5				
Hard, dry, grey, Clay Shale			100/4"		
			26		
			100/7"		
		15.0			
			100/3"		
	497.5		100/1.5"		
		20.0			
Bottom of hole = 18.1 feet					
No free water observed					
Elevation referenced to BM 39462 = 515.2 feet					
		25.0			

Notes	DEPTH	BLOWS	Qu tsf	W%
Borehole advanced with hollow stem auger (8" O.D, 3.25" I.D.)				
To convert "N" values to "N60" multiply by 1.25				
	30.0			
	35.0			
	40.0			
	45.0			
	50.0			

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

ILLINOIS DEPARTMENT OF TRANSPORTATION
District Nine Materials

Bridge Foundation Boring Log

TR 93A off IL 151 (Bluegrass Lane) Over Spring Creek

Route: Bluegrass Lane Structure Number: 039-0046 Date: 9/28/2017

Section

Bored By: R Moberly

County: Jackson Location: 6.7 mi North IL Rte 3 Checked By: A Hayes

Boring No 2-S
Station 18' E of E Abut
Offset 10' N CL
Ground Surface 515.2 Ft

Surf Wat Elev: 503.3
Ground Water Elevation
when Drilling
At Completion
At: Hrs:

	DEPTH	BLOWS	Qu tsf	W%
Crushed aggregate				
514.2				
Very stiff, moist, brown, Silty Clay to Clay A7-6				
		2		
		3	2.5B	20
		3		
510.2	5.0	1		
Loose, moist, brown, Sand and Gravel		3		
		2		
508.2				
Soft, very moist, brown, Clay Loam to Silty Clay Loam A-4		2		
		2	0.4S	15
		5		
505.2	10.0	15		
Hard, dry, grey and brown, Clay Shale		100/7"		
		100/8"		
	15.0	100/3"		
497.7		100/4"		
Bottom of hole = 17.3 feet	20.0			
No free water observed				
Elevation referenced to BM# 39462 = 515.2 feet				
	25.0			

Borehole advanced with hollow stem auger (8" O.D, 3.25" I.D.)

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

DEPTH	BLOWS	Qu tsf	W%
30.0			
35.0			
40.0			
45.0			
50.0			

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

APPENDIX C

SEISMIC SITE CLASS DETERMINATION

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

Modified on 12/10/10

PROJECT TITLE==== SGR 3 Bluegrass Lane Over Spring Creek

Substructure 1

Base of Substruct. Elev. (or ground surf for bents)	510.1	ft.
Pile or Shaft Dia.	12	inches
Boring Number	1 - S	
Top of Boring Elev.	516	ft.

Approximate Fixity Elev. 512.1 ft.

Individual Site Class Definition:

N (bar):	58	(Blows/ft.)	Soil Site Class C
N_{ch} (bar):	NA	(Blows/ft.)	NA
s_u (bar):	3.93	(ksf)	Soil Site Class C <----Controls

Seismic Soil Column Depth (ft)	Bot. Of Sample Elevation	Sample Thick. (ft.)	N	Qu (tsf)	Layer Description Boundary
4.1	508.0	2.00	6	1.60	B
6.6	505.5	2.50	6	0.50	B
7.1	505.0	0.50	100	####	B
100.0	412.1	92.90	100	####	R

Substructure 2

Base of Substruct. Elev. (or ground surf for bents)	509.6	ft.
Pile or Shaft Dia.	12	inches
Boring Number	2 - S	
Top of Boring Elev.	515.2	ft.

Approximate Fixity Elev. 511.6 ft.

Individual Site Class Definition:

N (bar):	59	(Blows/ft.)	Soil Site Class C
N_{ch} (bar):	77	(Blows/ft.)	Soil Site Class C <----Controls
s_u (bar):	3.68	(ksf)	Soil Site Class C

Seismic Soil Column Depth (ft)	Bot. Of Sample Elevation	Sample Thick. (ft.)	N	Qu (tsf)	Layer Description Boundary
3.4	508.2	1.50	5		B
6.4	505.2	3.00	7	0.40	B
7.4	504.2	1.00	100	####	B
100.0	411.6	92.60	100	####	R

Substructure 3

Base of Substruct. Elev. (or ground surf for bents)		ft.
Pile or Shaft Dia.		inches
Boring Number		
Top of Boring Elev.		ft.

Approximate Fixity Elev. ft.

Individual Site Class Definition:

N (bar):		(Blows/ft.)	NA
N_{ch} (bar):		(Blows/ft.)	NA
s_u (bar):		(ksf)	NA

Seismic Soil Column Depth (ft)	Bot. Of Sample Elevation	Sample Thick. (ft.)	N	Qu (tsf)	Layer Description Boundary

Substructure 4

Base of Substruct. Elev. (or ground surf for bents)		ft.
Pile or Shaft Dia.		inches
Boring Number		
Top of Boring Elev.		ft.

Approximate Fixity Elev. ft.

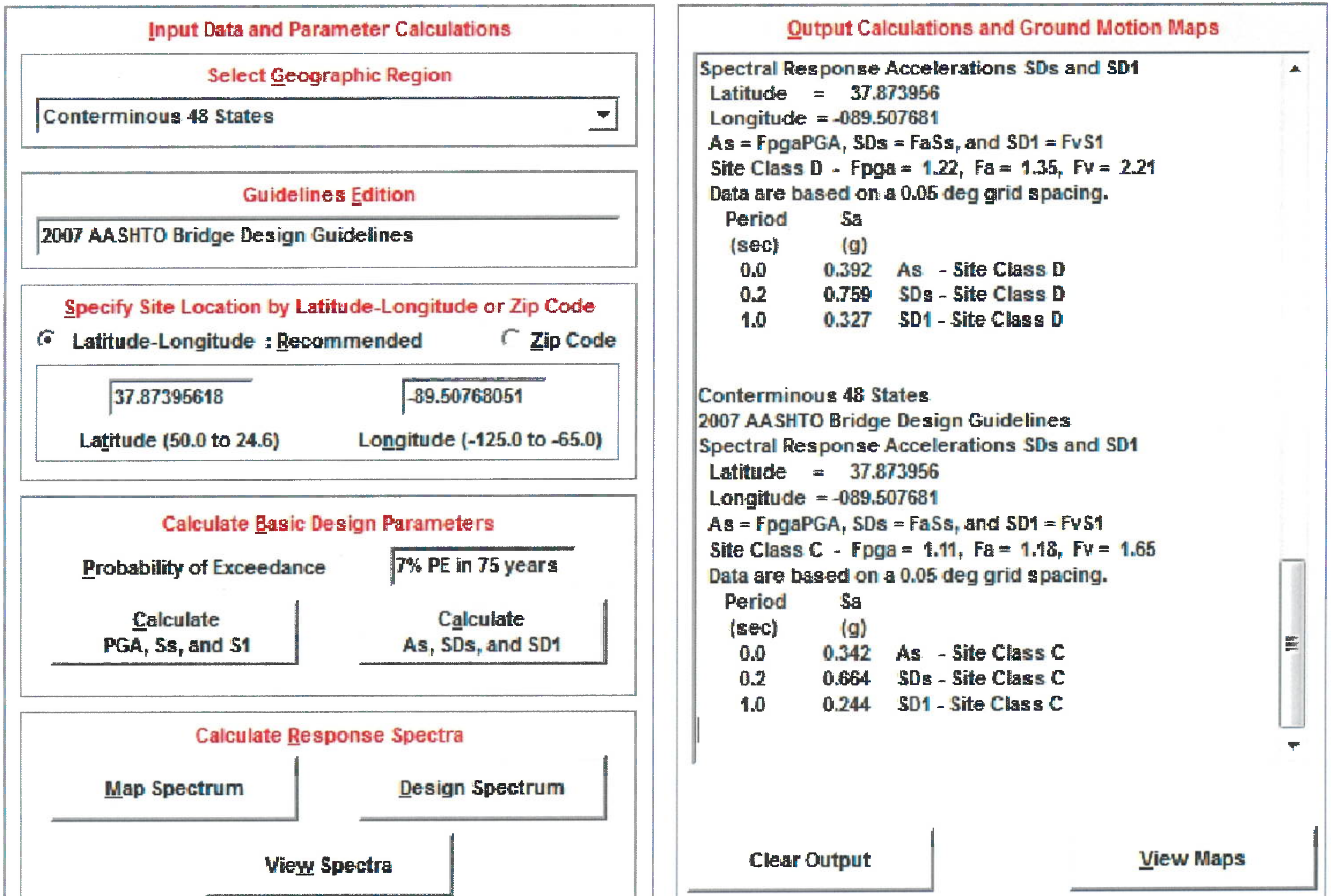
Individual Site Class Definition:

N (bar):		(Blows/ft.)	NA
N_{ch} (bar):		(Blows/ft.)	NA
s_u (bar):		(ksf)	NA

Seismic Soil Column Depth (ft)	Bot. Of Sample Elevation	Sample Thick. (ft.)	N	Qu (tsf)	Layer Description Boundary

Global Site Class Definition: Substructures 1 through 2

N (bar):	59	(Blows/ft.)	Soil Site Class C
N_{ch} (bar):	88	(Blows/ft.)	Soil Site Class C
s_u (bar):	3.81	(ksf)	Soil Site Class C <----Controls



Input Data and Parameter Calculations
Select Geographic Region
Conterminous 48 States
Guidelines Edition
2007 AASHTO Bridge Design Guidelines
Specify Site Location by Latitude-Longitude or Zip Code
Latitude-Longitude : Recommended
Zip Code
37.87395618
-89.50768051
Latitude (50.0 to 24.6)
Longitude (-125.0 to -65.0)
Calculate Basic Design Parameters
Probability of Exceedance
7% PE in 75 years
Calculate PGA, Ss, and S1
Calculate As, SDs, and SD1
Calculate Response Spectra
Map Spectrum
Design Spectrum
View Spectra
Output Calculations and Ground Motion Maps
Spectral Response Accelerations SDs and SD1
Latitude = 37.873956
Longitude = -089.507681
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.22, Fa = 1.35, Fv = 2.21
Data are based on a 0.05 deg grid spacing.
Period Sa
(sec) (g)
0.0 0.392 As - Site Class D
0.2 0.759 SDs - Site Class D
1.0 0.327 SD1 - Site Class D
Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 37.873956
Longitude = -089.507681
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class C - Fpga = 1.11, Fa = 1.18, Fv = 1.65
Data are based on a 0.05 deg grid spacing.
Period Sa
(sec) (g)
0.0 0.342 As - Site Class C
0.2 0.664 SDs - Site Class C
1.0 0.244 SD1 - Site Class C
Clear Output
View Maps