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August 15 2018
exp Nov. 2019

Prepared for:
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Structure Geotechnical Report

6th St. (FAP 666)
Section (109)VB, (110)VB-5
Sangamon County
Job No. ---
Contract No. 72K43
PTB No. N/A
UPRR & NSRR Over 6th Street
Structure Nos. 084-9962 and 084-9963

December 2017
Revised August 2018

Table of Contents

Tables

1. Project Description

This report provides geotechnical data and recommendations for the proposed Union Pacific Railroad (UPRR) and Norfolk Southern Railroad (NSRR) Bridges at the 6th Street Underpass, which is part of the Springfield Rail Improvements Project. The project includes the relocation of the existing UP tracks from the 3rd Street corridor to the 10th Street corridor and the relocation of the existing NS tracks within the 10th Street corridor. The project includes modifications to four existing grade separations and nine new grade separations. The bridges and retaining walls covered by this structure geotechnical report will replace the existing 6th Street NSRR underpass.

2. Location

The proposed 6th Street Underpass is located in the central portion of Sangamon County, within the Southwest Quarter of Section 3, Township 15 North, Range 5 West. Structure Number 084-9962 carries the UPRR over 6th Street at Sta. 1000+23.59, while Structure Number 084-9963 carries the NSRR over 6th Street at Sta. 999+36.29. They are located at Sta. 47848+71.04 along the UPRR Main 1 alignment and at Sta. 52497+79.01 along the NSRR Main 1 alignment.

3. Existing Structure

The existing NSRR bridge is a three-span through girder structure with a concrete floor. The bridge and the short retaining walls along the curbs of 6th Street were constructed in the 1930's as a replacement for an older bridge. The west abutment of the older bridge was modified and incorporated into the current bridge. The two piers and the east abutment were new construction.

The bridge is founded on spread footings bearing at approximately Elev. 580.0. The available plans do not indicate a design bearing pressure for the footings. Based on the recent borings, the existing footings are bearing on stiff to very stiff glacial till, two to three feet above hard, weathered shale.

4. Proposed Structures

The general structure configuration was determined from an informal type study as discussed later in this report. The proposed structures will be single-span bridges with stub abutments. The superstructures will be steel plate ballast pans on W36 floor beams between 150-inch web through-plate girders. Abutments will be supported by deep foundations independent of the proposed and existing retaining walls. The profile grade of 6th Street will be maintained at existing grade. The low point of the underpass is on the north side of the railroad. Retaining walls will extend from Sta. 998+00.80 to Sta. 1001+66.02.

The bridges will be supported on drilled shaft foundations. Based on information provided by the structure designer, vertical service loads of approximately 4,200 kips per abutment will be applied to the foundations.

Two tiers of retaining walls will be used at the underpass. The existing retaining walls between the elevated sidewalk and curb line will remain. A gap in these walls at Princeton Avenue will be closed with similar new construction. The north end of the east wall will be replaced in kind due to a proposed sewer relocation beneath it. New retaining walls will be constructed between the outside of the sidewalks and the proposed bridges.

Both proposed bridges and the retaining walls will be constructed with the existing rail line active through the construction zone and 6th Street will remain open to traffic. The substructures for the new bridges will be constructed in a top-down sequence. The NSRR Bridge will be built first south of the existing structures along with the south portion of the East and West Retaining Walls. Rail traffic will be diverted onto the newly constructed NSRR Bridge. The remaining retaining walls and the UPRR Bridge can then be constructed after the existing structure is removed.

5. Site Investigation

The project site is located in a highly developed, urban area. At the existing 6th Street railroad crossing, 6th Street is lowered below the existing railroad. Existing grade along the street ranges from approximately Elev. 589.8 to Elev. 585.2 with the lowest point at the railroad and the highest point south of the crossing on 6th Street.

Two (2) test borings designated B-145 and B-146 were completed in September 2013 at the location of the proposed structures using a drill rig operated by Professional Services Industries, Inc. The borings were advanced using hollow stem augers to bedrock. NQ-sized core samples were collected at both boring locations. Standard Penetration Test (SPT) samples were generally collected at 2.5 ft. intervals for top 20 feet and 5.0 ft. intervals thereafter. All SPT samples were collected using an automatic hammer. The borings were advanced to depths between 35.0 and 49.5 ft.

The boring location is shown on the Boring Location Plan included in the Appendix. The boring log and rock core photos are also included in the Appendix.

6. Laboratory Investigation

Soil samples from the borings were tested in Hanson's soils laboratory. The laboratory analysis consisted of moisture content determinations, unconfined strength tests of SPT samples, and unconfined strength tests of rock core samples. The results of the tests are indicated on the subsurface data profile. Data from the rock core tests are included in the appendix.

7. Subsurface Profile

Subsurface data profiles for the proposed bridge and retaining walls are presented in the Appendix for use by the structure designer. The data profile includes the borings that were drilled near the proposed structures. The general subsurface profile consists of deposits of fill material, loess, glacial till, and shale bedrock.

A layer of fill was encountered near the ground surface in B-145. The fill material was composed of sandy clayey silt with brick and rock fragments. The SPT N-values for the fill samples collected were 8 to 12 blows per foot penetration. Unconfined strengths were 4.5 tsf for the fill.

Loessial deposits were encountered in both borings. This stratum has been partially removed at the existing roadway level where B-146 was drilled. The very fine sandy silt to very fine sandy silty clay was encountered below the surficial fill or pavement. The N-value for the loess was 4 to 12 blows per foot penetration. The measured unconfined strength ranged from 0.6 to 3.0 tsf.

A weathered glacial till layer was encountered in both borings. This sandy, silty clay layer was encountered at approximately Elev. 585.0 or about 2 ft. below the current street grade. The N-value was 4 to 6 blows per foot. Measured unconfined strength ranged from 0.7 to 2.5 tsf.

Bedrock was encountered in all borings at approximately Elev. 578.0, or about 9 ft. below the current street grade. The uppermost 5.5 ft. was a shale with various degrees of weathering. A competent, but weak shale layer was encountered from Elev. 572.5 to Elev. 556.0. Unconfined strengths from cores taken in this layer were 11.3 to 21.9 tsf. A coal layer was located beneath the weak shale and extended to the maximum depths of the borings.

Groundwater was not encountered during drilling at any of the boring locations. The borings were drilled during an unusually dry period.

Maps of documented coal mines provided by the Illinois Geological Survey show the proposed site has likely been undermined by the Peabody Coal Company Peabody Mine No. 53 (PCCP 53). This was a room and pillar panel mine that was active between 1887 and 1944. Between 40 and 70 percent of the coal seam is removed in this type of mine. The Springfield coal seam was mined with an average thickness of 5.8 ft. The depth of the mine is 250 ft. If the roof of the mined out area were to collapse, the ground surface could subside and the proposed structures will likely subside with the surrounding area.

8. Geotechnical Evaluations

Several retaining wall and bridge configurations were considered for the proposed grade separation. An underpass requires the use of retaining walls along both sides of the street due to the existing ROW and maximized bridge spans. Non-gravity cantilever walls are the best choice for the conditions at this site, because they can be constructed within the confined span of the proposed bridge spans and would cause the least disruption to rail and roadway traffic and the surrounding properties.

ROW and/or permanent easements for tiebacks are not available. A substantial cantilevered structural member is required to support the temporary grade differences of up to 20 ft. Consequently, sheet pile and driven soldier pile walls are not feasible for the tallest sections of the wall. Drilled soldier pile walls with either wide-flange structural sections or reinforcement bars are feasible and could also directly support the bridge abutments.

Drilled shafts are appropriate for support of the bridge abutments due to the use of drilled foundations for the retaining walls. Spread footings bearing on the relatively shallow bedrock would be feasible, but very costly due the substantial temporary shoring required to excavate near an active rail line.

Slope stability analyses were not necessary, because the 1V:3H slopes beyond the proposed structures will match the existing condition. The retaining wall soldier piles will be socketed into relatively shallow bedrock, preventing a compound slope stability failure. If the retaining walls are designed to satisfy AASHTO external stability and sliding requirements, they will also meet AASHTO and IDOT global stability requirements.

Up to 11 ft. of fill will be placed behind portions of the proposed retaining walls. This fill is located in areas that were excavated for the existing underpass, so the existing subgrade is overconsolidated. Settlement due to the new fill is expected to be less than 0.5 inches.

9. Design Recommendations

The proposed bridge substructures should be supported on drilled shaft foundations with the tips founded in the weak shale. In order to provide a consistent bearing surface on unweathered rock, the estimated tip elevations should be at least 2.0 ft. below the top of weak shale elevations listed in Table 9.1. The shafts should be proportioned to resist the axial loads using the tip resistance and skin resistance of the weak shale given in Table 9.2. Any side resistance contributed by the overlying, much softer layers above should be ignored. Tip resistance within the weak shale decreases with depth due to the presence of the coal layer below. For maximum tip resistance, the drilled shafts should be founded a minimum of two socket diameters above the coal layer. Considering that lateral resistance may control design, reduced tip resistance values are provided in the table for deeper rock sockets.

Table 9.1 Top of Strata Elevations for Foundation Design

Location	Existing Fill	Loess	Glacial Till	Weathered Shale	Weak Shale	Coal
Abutments and Walls	*	595.0	585.0	578.0	572.5	556.0

* Existing ground surface or assumed bottom of excavation for existing structure.

Table 9.2 Drilled Shaft Axial Load Design Parameters – West Abutment

Stratum		Nominal Side Resistance (ksf)	Resistance Factor φ_{stat}	Nominal Tip Resistance (ksf)	Resistance Factor φ_{stat}
Fill		-	-	-	-
Loess		-	-	-	-
Glacial Till		-	-	-	-
Weathered Shale		2.2	0.45	40	0.40
Weak Shale	2.0D above coal	7.0	0.50[1]	75	0.50[1]
	1.5D above coal			65	0.50[1]
	1.0D above coal			50	0.50[1]
	0.5D above coal			35	0.50[1]
Coal		-	-	-	-

[1] Use FS=2.5 for AREMA allowable stress design

The structure designer should evaluate lateral resistance of the drilled shafts based on both soil and structure properties. Soil parameters for generating P-y curves with the LPILE computer program are given in Table 9.3. Parameters not provided in the table should use the default values assigned by the LPILE program. Factored axial and factored lateral loads should be used for structural design of the soldier piles. The P-multipliers in AASHTO Table 10.7.2.4-1 should be used in the analyses

Soldier pile walls retaining level ground should be designed for an active earth pressure of 40 pcf if drainage is provided along the face of the wall. For soldier piles retaining slopes, the earth pressure should be calculated using a 32° friction angle and a 120 pcf unit weight. Surcharges due to the weight of soil behind the abutments and railroad live loads should also be applied as applicable. Drilled soldier piles for the underpass retaining walls will not have significant vertical load and may be supported in either rock or soil as required by the wall heights. Table 9.1 provides design strata elevations for the various soil layers found along the walls. The structure designer should evaluate lateral resistance based on both soil and structure properties. Soil parameters for generating P-y curves with the LPILE computer program are given in Table 9.3. Factored axial and factored lateral loads should be used for structural design of the soldier piles. The P-multipliers in AASHTO Table 10.7.2.4-1 should be used in the analyses.

Table 9.2 provides geotechnical design parameters for axial resistance of drilled soldier piles. When soldier piles are tipped in the weak shale, only the side and tip resistance of that layer should be included in the axial strength. If soldier piles are tipped above the weak shale, the side resistance should be neglected in the upper 5 ft. and bottom 2D of the shaft, but all layers may be included in the axial strength.

Table 9.3 LPILE Parameters

Stratum	LPILE Soil Type	Soil Parameters				
Proposed Fill	sand	φ=32°	γ=125 pcf	k=90 pci		
Existing Fill	sand	φ=28°	γ'=58 pcf	k=20 pci		
Loess	stiff clay w/o water	c=1,000 psf	γ'=58 pcf			
Glacial Till	stiff clay w/o water	c=800 psf	γ'=66 pcf			
Weathered Shale	stiff clay w/o water	c=4,500 psf	γ'=72 pcf			
Weak Shale	weak rock	q_u=167 psi	γ'=81 pcf	E_i=1,000 ksi	RQD=37	$k_{rm}=5x10^{-4}$

* Existing ground surface or assumed bottom of excavation for existing structure.

Soldier pile retaining walls should be detailed to include geocomposite wall drain and an underdrain collector as shown in Figures 3.11.3.2.1-2 and 3.11.3.2.1-3 of the IDOT Bridge Manual. Any fill placed behind soldier piles should be porous granular embankment placed in thin lifts and lightly compacted with hand-held or walk-behind compactors.

Semi-gravity walls, which will be used as the final wall facing in front of the East Abutment of the NSRR Bridge, should be designed for an active earth pressure of 40 pcf if drainage is provided behind the wall. Surcharges due to the weight of soil behind the abutments and railroad live loads should either be applied to the semi-gravity wall or resisted by the temporary shoring left in place. The semi-gravity wall will bear on the stem of the existing bridge abutment and on granular backfill. The wall should be designed for a factored bearing resistance of 6.0 ksf and a factored sliding resistance of 0.62 times the vertical load.

Semi-gravity walls to be constructed behind the curb of 6th Street should be designed for an active earth pressure of 40 pcf if drainage is provided behind the wall. Pedestrian surcharge, using an active earth pressure coefficient of 0.33, should also be applied. These walls will bear on medium stiff to stiff clayey soils. The walls should be designed for a factored bearing resistance of 2.0 ksf and a factored sliding resistance of 0.7 ksf.

The project is located in a region of low seismic activity, which is caused primarily by earthquakes in the New Madrid Fault Zone, 225 miles south of the site. The subsurface profile to a depth of 100 ft. below the assumed point of drilled shaft fixity consists of weak shale bedrock. This profile is indicative of Soil Type C. Seismic design parameters obtained from the 2017 AREMA Seismic Design for Railway Structures Specifications are listed in Table 9.4. The soils found at the site are not liquefaction-susceptible for the design earthquakes.

Table 9.4 Seismic Design Parameters

Ground Motion Level	PGA	F_{pga}	S_s	F_a	S_1	F_v
Level 1 (100 year)	0.010	1.2	0.025	1.2	0.005	1.7
Level 2 (475 year)	0.040	1.2	0.090	1.2	0.035	1.7
Level 3 (2475 year)	0.10	1.2	0.22	1.2	0.10	1.7

10. Construction Considerations

The “top of rock” as shown on the plans should be the top of the weathered shale as defined in this report. This elevation should be used to estimate quantities for drilled shaft and drilled soldier pile rock excavation. The weathered shale is expected to require additional drilling effort as compared to the soil layers above.

It is anticipated that the drilled shafts and soldier pile shaft excavations will be constructed using either the dry method or temporary casing method. Shafts that extend into the highly weathered shale stratum should be detailed with the 6-inch size reduction as described in Section 3.10.2.4 of the Bridge Manual. This allows the contractor to seat an over-sized casing into the bedrock to remediate water-bearing or sloughing soils that are sometimes encountered. At this site, the problem soils are most likely to be encountered immediately above the bedrock and in areas that have been backfilled during previous construction.

Drilled shafts supporting the bridges should be installed with access ducts for crosshole sonic logging in accordance with railroad requirements. Guide Bridge Special Provision #91, Crosshole Sonic Logging Testing of Drilled Shafts (April 20, 2016) should be included with the contract documents.

Temporary shoring will be required to remove conflicting portions of the existing bridge abutments while maintaining rail traffic. Cantilever sheet piling is not feasible due to the substantial railroad surcharge loads. It is anticipated that the NSRR will require that any temporary soil retention system supporting active tracks be fully designed and included in the contract plans. The temporary soil retention system for this structure is expected to utilize some of the drilled shafts for the proposed abutments as a temporary tangent pile retaining wall. At these locations, secant lagging shafts should be installed to prevent the loss of soil between the drilled shafts. In locations where secant lagging is used, horizontal drains that penetrate the secant lagging should be installed at not more than 12 ft. horizontal and 6 ft. vertical spacing over the full height of the secant lagging. The horizontal drains should have not less than 2.5 ft. of 3 in. diameter slotted PVC well casing extending behind the secant lagging and should be plumbed to drain to a suitable outlet.

When determining the assumed construction sequence, access for a drill rig should be considered. The existing retaining walls along 6th Street will restrict access to many of the drilled shafts and drilled soldier piles. Most conventional drilling equipment would not be capable of reaching over the existing walls or any completed construction that projects above the ground surface.

References

American Railway Engineering and Maintenance-of-Way Association (2017). *AREMA Design Specifications.*

American Association of State Highway and Transportation Officials (2014-2016). *ASHTO LRFD Bridge Design Specifications, Seventh Edition with Interim Revisions.*

Chenoweth, C.A., Bargh, M.H., & Treworgy, C.G. (2009). *Directory of Coal Mines in Illinois, 7.5-Minute Quadrangle Series, Springfield East & West Quadrangles, Sangamon County.* Champaign, Illinois: Illinois State Geological Survey

Illinois Department of Transportation (2012). *Bridge Manual*.

Illinois Department of Transportation (2015). *Geotechnical Manual*.

Illinois Department of Transportation (2016). *Standard Specifications for Road and Bridge Construction*.

Appendix

Boring Location Plan
Subsurface Data Profile
Boring Logs
Rock Core Photographs

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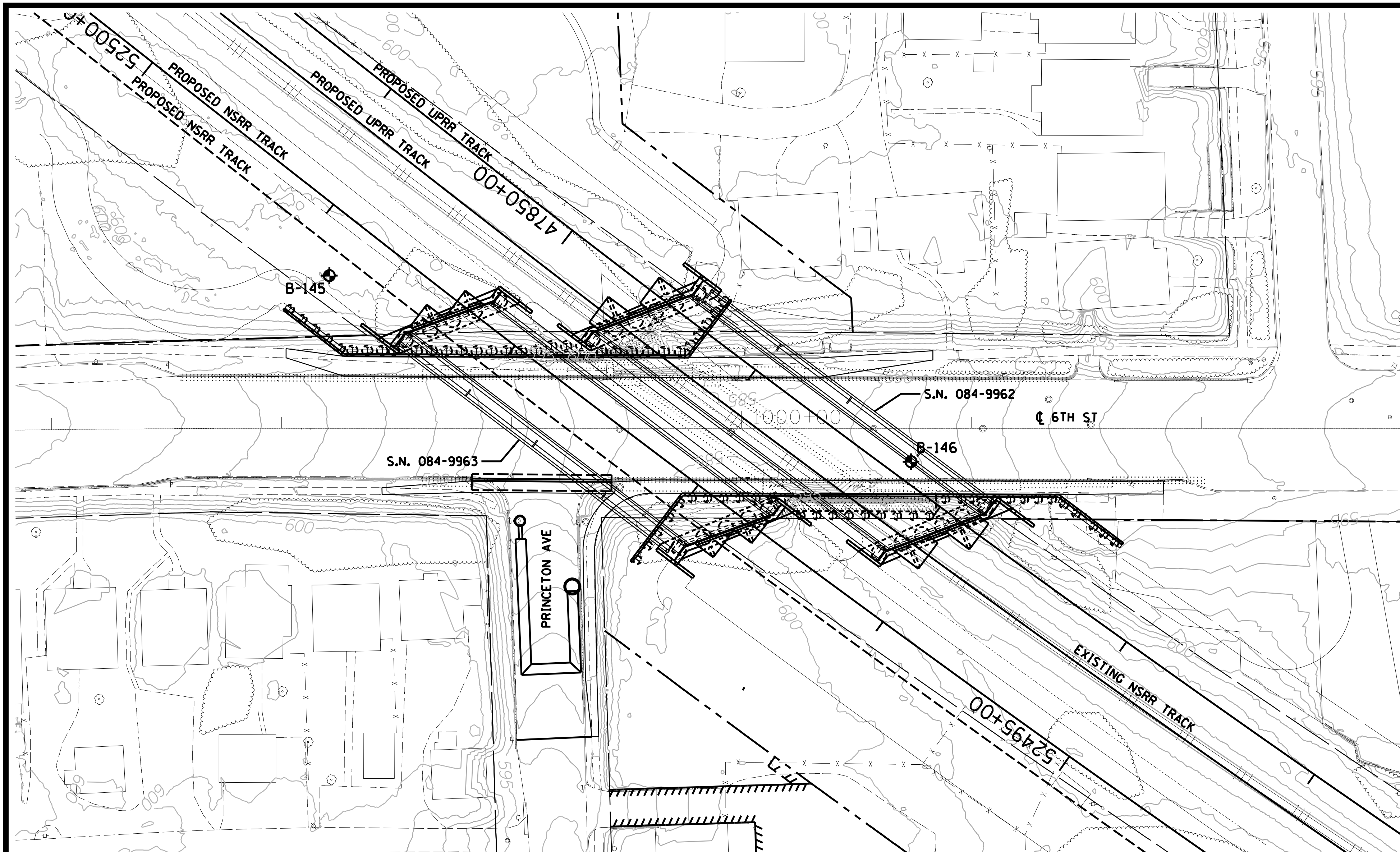

LEGEND

B-146 BORING LOCATION

N

60 0 60 120

HANSON

BORING LOCATION PLAN

UPRR & NSRR OVER 6TH STREET
S.N. 084-9962 & 084-9963
SPRINGFIELD, ILLINOIS

09L0179B 12/19/17

B-145
Sta. 998+21, 66' LT
9/5/13

Elev.	N	Qu	w%	Description
601.0				TOPSOIL
600.04	8	4.50P	15	Brown very fine sandy clayey SILT, some brick and rock fragments - FILL.
	12	4.50P	16	
595.04	12	3.00P	21	Brown and gray very fine sandy SILT.
	8	1.44B	23	
590.04	7	3.00P	24	Brown very fine sandy SILT, some clay.
587.54	5	0.58B	26	Dark gray very fine sandy silty CLAY.
585.04	5	1.03B	24	Gray very fine sandy silty CLAY, trace small gravel.
	5	0.70B	22	
577.54	63	4.50P	16	Brown and gray SHALE. (HIGHLY WEATHERED SHALE)
572.54	50/4"		9	Gray SHALE.
	50/5"		8	
566.04	Rec. = 77% RQD = 73% Rec. = 90% RQD = 56%			Gray sandy SHALE, micaceous.
562.54	11.3 Rec. = 90% RQD = 48% Rec. = 99% RQD = 68%			Gray clayey SHALE.
558.04				Gray sandy SHALE, micaceous.
556.04	Rec. = 100% RQD = 46% Rec. = 67% RQD = 0%			COAL.
551.54				Bottom of Hole = 49.5 feet

B-146
Sta. 1000+74, 15' RT
9/11/13

Elev.	N	Qu	w%	Description
587.0				
586.61				ASPHALT.
585.86	4		24	CONCRETE. Dark gray very fine sandy silty CLAY.
583.53	4	0.66B	25	Blue-gray very fine to fine sandy silty CLAY.
	6	2.47S	19	
578.53	57	4.50P	14	Brown and gray SHALE. (HIGHLY WEATHERED SHALE)
576.03	50	4.50P	11	Gray SHALE.
	50/5"		11	
572.03	Rec. = 81% RQD = 19% Rec. = 88% RQD = 71% 12.7 Rec. = 75% RQD = 44% Rec. = 85% RQD = 51% 21.9 Rec. = 91% RQD = 78%			Gray clayey SHALE, micaceous.
556.5	Rec. = 100% RQD = 78%			Stiff to very stiff gray shaley CLAY.
553.5				Gray sandy SHALE, micaceous.
553.0				COAL.
552.0				Bottom of Hole = 35.0 feet

LEGEND

N Standard Penetration Test N (blows/ft)
Qu Unconfined Strength (tsf)
w% Natural Moisture Content (%)
558.10 DD Water Surface Elevation Encountered in Boring
DD = during drilling
0h = at completion
24h = 24 hours after completion

DESIGNED	RGC	11/13/17
DRAWN	EJM	11/13/17
REVIEWED	RGC	11/13/17

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HANSON

USER NAME = madau00223	DESIGNED - $DESIGN$	REVISED - $REVDATE1$
	CHECKED - $CHECKED$	REVISED - $REVDATE2$
PLOT SCALE = $SCALE1$	DRAWN - $DRAWN$	REVISED - $REVDATE3$
PLOT DATE = $DATE1$	CHECKED - $CHECKED$	REVISED - $REVDATE4$

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

SUBSURFACE DATA PROFILE
S.N. 084-9962 & 084-9963
SHEET NO. 1 OF 1 SHEETS

F.A.P. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
666	(109)VB, (110)VB-5	SANGAMON	-	-
		CONTRACT NO. 72K43		
	ILLINOIS FED. AID PROJECT			

HANSON **SOIL BORING LOG**

Date 9/5/13

ROUTE ____ **DESCRIPTION** Springfield Rail Improvements Project **LOGGED BY** ARP

SECTION ____ **LOCATION** SW ¼ of SEC. 3, TWP. 15N, RNG. 5W, 3rd P.M.

COUNTY Sangamon **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** Auto

STRUCT. NO. ____
Station ____
BORING NO. B-145
Station 998+21
Offset 66' LT
Ground Surface Elev. 601.0 ft

Surface Water Elev. ____
Stream Bed Elev. ____
Groundwater Elev.:
First Encounter ____ ft
Upon Completion ____ ft
After ____ **Hrs.** ____ ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
TOPSOIL	600.04				
Brown very fine sandy clayey SILT, some brick and rock fragments - FILL.		2	4 3 5	4.50P	15
		4	5 5 7	4.50P	16
	595.04	6			
Brown and gray very fine sandy SILT.			4 5 7	3.00P	21
		8			
		10	3 3 5	1.44B	23
	590.04				
Brown very fine sandy SILT, some clay.		12	3 3 4	3.00P	24
	587.54				
Dark gray very fine sandy silty CLAY.		14	1 2 3	0.58B	26
	585.04	16			
Gray very fine sandy silty CLAY, trace small gravel.			1 2 3	1.03B	24
		18			
		20	1 2 3	0.70B	22

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Gray very fine sandy silty CLAY, trace small gravel. *(continued from previous page)*		22			
	577.54				
Brown and gray SHALE. (HIGHLY WEATHERED SHALE)		24	11 25 38	4.50P	16
		26			
	572.54	28			
Gray SHALE.			43 50/4"		9
		30			
		32			
		34	50/5"		8
	566.04				
see Rock Core log.					

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer)
The SPT (N value) is the sum of the last two blow values in each sampling zone (AASHTO T206)

HANSON

ROCK CORE LOG

Date 9/5/13

ROUTE ______ **DESCRIPTION** Springfield Rail Improvements Project **LOGGED BY** ARP

SECTION ______ **LOCATION** SW ¼ of SEC. 3, TWP. 15N, RNG. 5W, 3rd P.M.

COUNTY Sangamon **CORING METHOD** NQ Core

STRUCT. NO. ______
Station ______

CORING BARREL TYPE & SIZE NQ

Core Diameter 1.874 in
Top of Rock Elev. 566.04 ft
Begin Core Elev. 566.04 ft

BORING NO. B-145
Station 998+21
Offset 66' LT
Ground Surface Elev. 601.04

Description	Elev.	DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
Gray sandy SHALE, micaceous.	566.04	36	Run 1	77	73		
		38	Run 2	90	56		
Gray clayey SHALE.	562.54	40	Run 3	90	48		11.3
		42	Run 4	99	68		
Gray sandy SHALE, micaceous.	558.04	44					
COAL.	556.04	46	Run 5	100	46		
		48	Run 6	67	0		
End of Boring	551.54						

Color pictures of the cores ______
Cores will be stored for examination until ______
The "Strength" column represents the uniaxial compressive strength of the core sample (ASTM D-2938)

HANSON SOIL BORING LOG

Date 9/11/13

ROUTE ____ **DESCRIPTION** Springfield Rail Improvements Project **LOGGED BY** ARP

SECTION ____ **LOCATION** SW ¼ of SEC. 3, TWP. 15N, RNG. 5W, 3rd P.M.

COUNTY Sangamon **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** Auto

STRUCT. NO. ____
Station ____
BORING NO. B-146
Station 1000+74
Offset 15' RT
Ground Surface Elev. 587.0 ft

Surface Water Elev. ____
Stream Bed Elev. ____

Groundwater Elev.:
First Encounter ____ ft
Upon Completion ____ ft
After ____ **Hrs.** ____ ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
ASPHALT.	586.61				
CONCRETE.	585.86				
Dark gray very fine sandy silty CLAY.		2	3 2 2		24
	583.53				
Blue-gray very fine to fine sandy silty CLAY.		4	2 2 2	0.66B	25
		6	2 2 4	2.47S	19
	578.53	8			
Brown and gray SHALE. (HIGHLY WEATHERED SHALE)			9 19 38	4.50P	14
	576.03	10			
Gray SHALE.			22 50	4.50P	11
		12			
		14	26 50/5"		11
	572.03				
see Rock Core log.					

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer)
The SPT (N value) is the sum of the last two blow values in each sampling zone (AASHTO T206)

HANSON

ROCK CORE LOG

Date 9/11/13

ROUTE ______ **DESCRIPTION** Springfield Rail Improvements Project **LOGGED BY** ARP

SECTION ______ **LOCATION** SW ¼ of SEC. 3, TWP. 15N, RNG. 5W, 3rd P.M.

COUNTY Sangamon **CORING METHOD** NQ Core

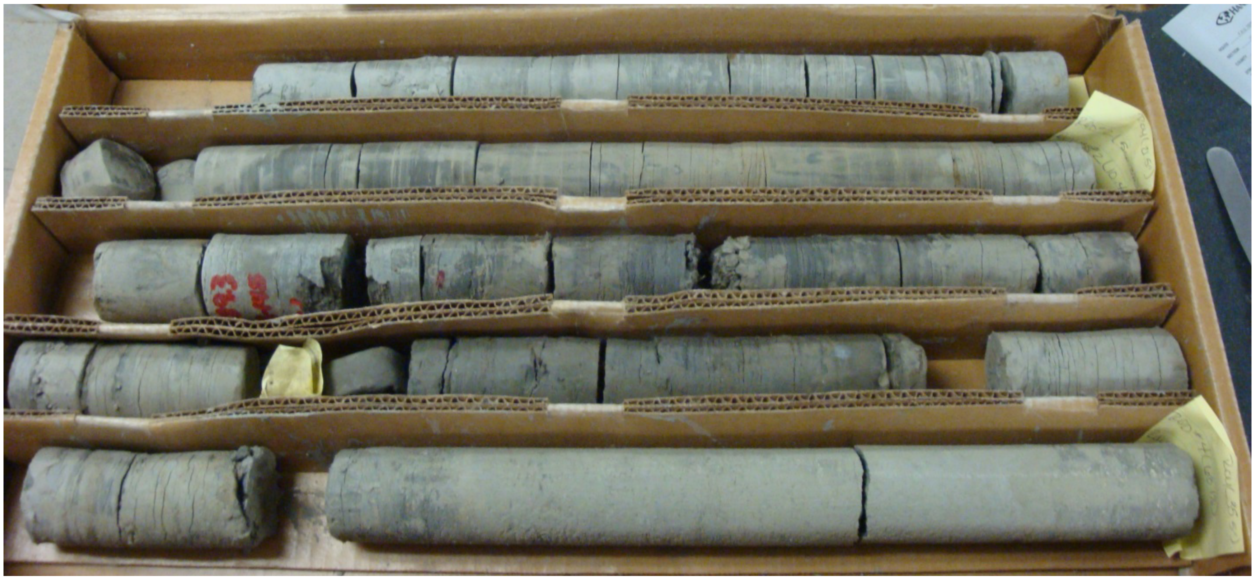
STRUCT. NO. ______ **CORING BARREL TYPE & SIZE** NQ
Station ______

Core Diameter 1.874 in
Top of Rock Elev. 572.03 ft
Begin Core Elev. 572.03 ft

BORING NO. B-146
Station 1000+74
Offset 15' RT
Ground Surface Elev. 587.03

Description	Elev.	DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
Gray clayey SHALE, micaceous.	572.03	16	Run 1	81	19		
		18	Run 2	88	71		
		20	Run 3	75	44		12.7
		22, 24	Run 4	85	51		
		26, 28, 30	Run 5	91	78		21.9
Stiff to very stiff gray shaley CLAY.	556.53	32	Run 6	100	78		
Gray sandy SHALE, micaceous.	553.53						
COAL.	553.03	34					
End of Boring	552.03						

Color pictures of the cores ______
Cores will be stored for examination until ______
The "Strength" column represents the uniaxial compressive strength of the core sample (ASTM D-2938)



Boring B-145 35.0 - 44.5 ft			
Run	Depth (ft)	REC (%)	RQD (%)
1	35.0 - 37.0	77	73
2	37.0 - 39.0	90	56
3	39.0 - 41.5	90	48
4	41.5 - 44.5	99	68

Boring B-145 44.5 - 49.5 ft			
Run	Depth (ft)	REC (%)	RQD (%)
5	44.5 - 46.5	100	46
6	46.5 - 49.5	67	0

Boring B-146			
15.0 - 25.0 ft			
Run	Depth (ft)	REC (%)	RQD (%)
1	15.0 - 17.0	81	19
2	17.0 - 19.0	88	71
3	19.0 - 21.0	75	44
4	21.0 - 25.0	85	51

Boring B-146			
25.0 - 34.0 ft			
Run	Depth (ft)	REC (%)	RQD (%)
5	25.0 - 30.0	91	78
6	30.0 - 34.0	100	78

Boring B-146
34.0 - 35.0 ft

Run	Depth (ft)	REC (%)	RQD (%)
6	34.0 - 35.0	100	78

ROCK CORE COMPRESSIVE STRENGTH TESTING
Data and Photograph Sheet

HANSON

PROJECT DESCRIPTION:	Springfield Rail Improvement
PROJECT LOCATION:	Springfield IL
PROJECT NUMBER:	09L0179B

Input By:	*RIN*	Date: *09/18/13*
Checked By:	*JDM*	Date: *09/18/13*
Balance #:	*G09745*	
Caliper #:	*7142658*	

ROCK CORE TESTING DATA

Boring Name	Sample Number	Run Number	Depth Range (ft)	Elevation Range (ft)	Moisture Content (%)	Unit Weight (pcf)	**Unconfined Compressive Strength (psi)**	**Unconfined Compressive Strength (tsf)**
B-145	1	3	39.3 - 39.6	561.7 - 561.4	N/A	142.2	**156.8**	**11.3**
B-146	1	3	19.9 - 20.2	567.1 - 566.8	N/A	143.1	**175.8**	**12.7**
B-146	2	5	25.5 - 25.8	561.5 - 561.2	N/A	144.8	**303.6**	**21.9**

ROCK CORE TESTING PHOTOGRAPHS

B-145 - 1

B-146 - 1

B-146 - 2