Structure Geotechnical Report

F.A.I. Route 74
Section 81-1HB-2
Rock Island County
Job No. P-92-032-01
Contract No. 64C08
PTB No. N/A
I-74 Over 19th Street SB
Structure Nos. 081-0184 (WB) and 081-0185 (EB)
Existing Structure Nos. 081-0103(WB) and 081-0104 (EB)

Submitted June 2014
Revised September 2014

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STATE OF ILLINOIS

9/8/14

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1. Project Description

This report provides geotechnical data and recommendations for the proposed I-74 Over 19th Street SB Bridges, which are part of the South Section of the I-74 over the Mississippi River Project. The project includes reconstruction of I-74 between 29th Street and 14th Avenue in Moline, Illinois. The bridges covered by this structure geotechnical report will be replacements for the existing structures carrying I-74 over 19th Street SB.

Nearby project features that have an impact on the design or construction of the proposed bridges include the I-74 roadway and the 19th Street SB roadway. Geotechnical recommendations for the interstate and street will be contained in a soil survey report prepared by others.

This report supersedes the structure geotechnical reports prepared by CH2M HILL in September 2009 and Hanson Professional Services Inc. in August 2012. This revised report has been prepared to address changes to the project staging and changes to the pile design procedure.

2. Location

The proposed I-74 Over 19th Street SB Bridges are located in the north central portion of Rock Island County, within Section 4 of Township 17 North, Range 1 West of Township 17 North, Range 1 West. They are located at I-74 Sta. 95+07.96. Structure Number 081-0184 carries Westbound (Northbound) I-74 and Ramp AC-D over 19th Street SB, while Structure Number 081-0185 carries Eastbound (Southbound) I-74.

3. Existing Structures

The existing structures, S.N. 081-0103 (Westbound I-74) and S.N. 081-0104 (Eastbound I-74), were constructed in 1971. They are three-span bridges with stub abutments. The piers are multi-column trapezoidal column bents with cap beams and crashwalls. The spill slopes of the existing structures are 2H:1V at right angles to the abutments and are covered with concrete slopewalls. Portions of the existing structure plans are included in the Appendix for reference.

The structures are supported on vertical and batter piles. Concrete piles with a 90 kip allowable capacity were used under the abutments. Timber piles with a 48 kip allowable capacity were used under the piers. Based on the estimated lengths shown on the existing structure plans, the pile tips are located in very stiff to hard clay (glacial till) at Elev. 615 to Elev. 636 for the concrete piles and Elev. 617 to Elev. 629 for the timber piles.

4. Proposed Structures

The general structure type was determined previously by others.. The proposed grade separation structures will be three-span bridges with stub abutments. The new structures will follow the same general profile, but will be approximately 40 ft longer and 30 ft wider than the existing bridges. The proposed abutments will be located behind the current abutments, which will allow the existing spill slopes to be cut back slightly. The proposed spill slopes at the abutments will be 2H:1V with concrete slopewalls. Most of the additional bridge width will be added in the existing median. The outside edges of the proposed structures will be no more than 8 ft beyond the existing bridges.

The structures will be supported on driven pile foundations. Based on information provided by the structure designer, factored vertical loads of approximately 18 kips per foot of abutment and 42 kips per foot of pier will be applied to the piles.

The proposed bridges will be constructed in stages in order to allow traffic on I-74 and 19th Street SB throughout the construction period. The proposed 19th Street SB improvements will be completed in the first stage of construction. The WB bridge and EB bridge will be constructed in the second and third stages, respectively.

5. Site Investigation

The project site is located on the bluffs above the Mississippi River. Steep natural slopes are found to the east and west of the current I-74 alignment. The existing ground near the bridge has been extensively graded for the I-74 mainline and Avenue of the Cities interchange ramps. Existing I-74 is on an embankment that is approximately 21 feet above the surrounding grade. Presently, 19th Street SB slopes down to the northeast at approximately 1% grade, while I-74 slopes down to the south at approximately 1% grade.

The field exploration that was completed for the proposed structure was accomplished in three phases. Test borings were completed in December 2005 and October 2008 by another consultant and in January through March 2008 by IDOT. IDOT provided the data collected by the consultant in 2009. Logs for the borings drilled by IDOT were provided in May 2014.

Eleven borings were drilled at the location of the proposed structures. The borings were advanced using a combination of hollow stem auger and mud rotary using bentonite slurry. The ground surface elevations of the borings vary from 653 ft to 675 ft.

Standard Penetration Test (SPT) samples were collected continuously in the upper 10 to 15 ft of each of the borings drilled by the consultant. The sampling interval varied from 2.5 ft to 20 ft for the deeper portions of these borings. The sampling interval of the IDOT borings was 2.5 ft for the full depth. All SPT samples were collected using an automatic hammer. The borings were advanced to depths between 50 and 102 ft. A limited number of thin-walled Shelby tube samples were also retrieved.

The boring locations are shown on the Boring Location Plan included in the Appendix. Boring logs are included in the Appendix.

6. Laboratory Investigation

Soil samples from the borings were tested by others. The laboratory analysis consisted of sieve and hydrometer analyses, Atterberg limits, CU triaxial, and consolidation testing. The testing of samples collected from the consultant borings does not meet IDOT's current minimum requirements for structure borings. Unconfined strength and moisture contents were available for only a small number of samples. Testing of samples from the IDOT borings does meet the minimum requirements for structure borings.

The locations of the index tests, triaxial tests, and consolidation tests are indicated on the subsurface data profile. The results of laboratory testing are included in the Appendix.

7. Subsurface Profile

A subsurface data profile is presented in the Appendix for use by the structure designer. The data profile includes the consultant-drilled borings that are located near the proposed structures. The data profile was not updated when the IDOT boring logs were received.

The subsurface profile consists of deposits of fill material, loess, gumbotil, and glacial till. All borings were terminated in the glacial till. Bedrock lies more than 100 ft below the ground surface.

Fill was encountered at all boring locations. It extended from the ground surface to the top of the loess, gumbotil or till strata. The fill material generally appeared to be composed of silty clay, loessial soils, which were probably obtained from a local source during construction of the existing highway. Small quantities of sand and rubble were also found within this layer. N-values between 3 and 28 blows per foot penetration were recorded for the fill materials. Unconfined strengths ranged from 0.25 to 4.5 tsf.

A 4 to 12 ft thick layer of silty clay, loessial soil was found in the borings drilled through the current highway embankment. The top of this layer varied from Elev. 666.9 to 648.5 or 8 to 26 ft below existing grade. The consistency of this layer varied significantly, with unconfined strengths from 0.25 to 3.3 tsf.

An approximately 5 ft thick gumbotil transitional layer was encountered in all borings except ILR2001. This sandy, silty clay layer was weathered from the underlying glacial till. SPT N-values of 7 to 17 were recorded for this material. Unconfined strengths were between 1.0 and 2.0 tsf.

The glacial till was encountered in all of the borings with a top between Elev. 635.6 and Elev. 655.1 or 15 to 35 ft below grade. SPT N-values for the till varied between 6 and 43 blows per foot penetration, but were typically between 13 and 30 blows per foot. Unconfined strengths ranged from 1.4 to 6.5 tsf, but were typically between 3.0 and 4.0 tsf. Samples taken from Boring IL030P had a consistently lower strength than those from the other borings.

Groundwater was encountered in Boring IL030P at Elev. 648.5 during drilling. Stabilized ground water readings were not taken in any of the borings. The groundwater, where it was encountered, is located near the top of the till stratum, which could indicate a localized, perched condition. For comparison, the water level in the Mississippi River, approximately 1.4 miles to the north of the site, is usually about Elev. 561.0.

The Illinois State Geological Survey Directory of Coal Mines does not list any mines in the immediate vicinity of the site.

8. Geotechnical Evaluations

Settlement and stability analyses were not completed, since the existing slopes and abutments show no signs of distress and only a small amount of new fill will be required for the proposed structures. Up to 3 ft of fill will be placed in the existing median ditch and along the right shoulder of EB I-74. The finished slopes will satisfy IDOT stability requirements and experience insignificant settlement following construction.

9. Design Recommendations

The IDOT Bureau of Bridges and Structures has requested that a project-specific pile design procedure be used for all bridges in the I-74 over the Mississippi River Project. This pile design procedure is expected to be adopted as official policy prior to construction of this project. Copies of the documents provided by IDOT are included in the Appendix.

The proposed stub abutments should be supported on piles driven into the very stiff to hard glacial till. Pile design capacities and estimated lengths were calculated based on a revised IDOT Static Method spreadsheet dated August 22, 2013. Table 9.1 lists design parameters for several pile types. Resistance factors are dependent upon the soil type but are generally 0.60 for the pile lengths shown. Metal shell piles are expected to provide the most suitable foundation for the subsurface conditions found at this site. There is no settlement expected in the soils between the bottom of the abutments and the glacial till. Drag loads or other geotechnical losses are not expected.

Table 9.1 Pile Design Parameters

Location	Cutoff Elevation (ft)	Pile Type	Factored Resistance Available, R_F (kips)	Geotech Losses, R_{Sdd} (kips)	Nominal Required Bearing, R_N (kips)	Estimated Pile Length (ft)	Soil Setup Pile Length (ft)
081-0184 (WB) North Abutment	667.0	HP 10x42	71 - 208	0	119 - 347	36 - 88	56 - 148
		HP 12x53	89 - 254	0	148 - 423	36 - 88	56 - 149
		HP 12x63	90 - 256	0	150 - 427	36 - 88	56 - 149
		HP 14x73	110 - 305	0	183 - 508	36 - 88	61 - 149
		12″φ x 0.25″ MS	99 - 235	0	165 - 392	36 - 69	56 - 115
		14″φ x 0.25″ MS	118 - 275	0	118 - 459	36 - 69	56 - 114
		14″ precast	159	0	265	39	59
081-0184 (WB) North Pier	650.3	HP 10x42	68 - 230	0	113 - 383	31 - 91	51 - 151
		HP 12x53	84 - 281	0	140 - 468	31 - 91	51 - 152
		HP 12x63	85 - 283	0	141 - 472	31 - 91	51 - 152
		HP 14x73	102 - 338	0	171 - 563	31 - 91	51 - 153
		12″φ x 0.25″ MS	96 - 235	0	160 - 392	31 - 71	51 - 109
		14″φ x 0.25″ MS	114 - 275	0	190 - 459	31 - 71	51 - 109
		14″ precast	159	0	265	33	56
081-0184 (WB) South Pier	651.5	HP 10x42	78 - 242	0	130 - 403	32 - 92	52 - 153
		HP 12x53	97 - 302	0	162 - 504	32 - 92	52 - 157
		HP 12x63	98 - 307	0	163 - 511	32 - 92	52 - 158
		HP 14x73	119 - 365	0	199 - 609	32 - 92	52 - 159
		12″φ x 0.25″ MS	108 - 235	0	180 - 392	32 - 62	52 - 102
		14″φ x 0.25″ MS	128 - 275	0	214 - 459	32 - 62	52 - 102
		14″ precast	159	0	265	32	52
081-0184 (WB) South Abutment	665.1	HP 10x42	88 - 242	0	146 - 403	36 - 96	61 - 156
		HP 12x53	109 - 302	0	181 - 504	36 - 96	64 - 161
		HP 12x63	110 - 324	0	183 - 540	36 - 101	64 - 171
		HP 14x73	133 - 386	0	222 - 643	36 - 101	64 - 172
		12″φ x 0.25″ MS	123 - 235	0	204 - 392	36 - 66	59 - 107
		14″φ x 0.25″ MS	145 - 275	0	242 - 459	36 - 66	61 - 107
		14″ precast	159	0	265	34	54

Note: Where a range of values is shown, pile lengths and capacities may be interpolated between the values given.

Table 9.1 Pile Design Parameters - Continued

Location	Cutoff Elevation (ft)	Pile Type	Factored Resistance Available, R_F (kips)	Geotech Losses, R_{Sdd} (kips)	Nominal Required Bearing, R_N (kips)	Estimated Pile Length (ft)	Soil Setup Pile Length (ft)
081-0185 (EB) North Abutment	669.8	HP 10x42	67 - 242	0	112 - 403	37 - 104	57 - 170
		HP 12x53	83 - 302	0	139 - 504	37 - 104	57 - 176
		HP 12x63	84 - 305	0	140 - 509	37 - 104	57 - 176
		HP 14x73	102 - 363	0	170 - 605	37 - 104	57 - 177
		12″φ x 0.25″ MS	95 - 235	0	158 - 392	37 - 72	57 - 115
		14″φ x 0.25″ MS	112 - 275	0	187 - 459	37 - 72	57 - 115
		14″ precast	159	0	265	39	59
081-0185 (EB) North Pier	650.3	HP 10x42	59 - 192	0	99 - 321	34 - 90	56 - 152
		HP 12x53	71 - 234	0	118 - 391	31 - 90	51 - 153
		HP 12x63	71 - 236	0	119 - 394	31 - 90	51 - 153
		HP 14x73	88 - 282	0	147 - 469	31 - 90	51 - 154
		12″φ x 0.25″ MS	80 - 235	0	133 - 392	31 - 74	46 - 123
		14″φ x 0.25″ MS	96 - 275	0	159 - 459	31 - 74	46 - 123
		14″ precast	159	0	265	39	59
081-0185 (EB) South Pier	651.5	HP 10x42	86 - 229	0	143 - 381	30 - 91	57 - 165
		HP 12x53	108 - 278	0	180 - 463	30 - 91	60 - 165
		HP 12x63	109 - 280	0	182 - 467	30 - 91	60 - 166
		HP 14x73	134 - 333	0	224 - 555	30 - 91	62 - 166
		12″φ x 0.25″ MS	133 - 235	0	221 - 392	32 - 62	62 - 113
		14″φ x 0.25″ MS	159 - 275	0	265 - 459	32 - 62	65 - 113
		14″ precast	159	0	265	30	50
081-0185 (EB) South Abutment	667.2	HP 10x42	87 - 221	0	145 - 368	36 - 90	61 - 168
		HP 12x53	108 - 269	0	181 - 448	39 - 90	61 - 169
		HP 12x63	110 - 271	0	183 - 452	36 - 90	61 - 169
		HP 14x73	134 - 322	0	223 - 537	36 - 90	63 - 170
		12″φ x 0.25″ MS	120 - 235	0	201 - 392	36 - 66	58 - 118
		14″φ x 0.25″ MS	143 - 275	0	239 - 459	36 - 66	58 - 118
		14″ precast	159	0	265	33	53

Note: Where a range of values is shown, pile lengths and capacities may be interpolated between the values given.

One test pile should be driven within the existing I-74 median at each substructure location. A total of 4 test piles should be required during the first stage of construction. No additional test piles are needed during the second stage. Pile shoes are recommended for steel shell piles due to the presence of cobbles and very dense layers in the till. Pile shoes are not necessary for H-Piles. Precoring of the embankment is not necessary.

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 9.2.

Table 9.2 LPILE Parameters

Location	Bottom Elevation (ft)	LPILE Soil Type	Soil Parameters		
North Abutments	658	soft clay	c=600 psf	γ'=115.0 pcf	ε_{50}=0.007
	647	stiff clay w/o water	c=1,800 psf	γ'=62.6 pcf	ε_{50}=0.007
		stiff clay w/o water	c=2,700 psf	γ'=72.6 pcf	ε_{50}=0.005
WB North Pier	646	sand	ϕ=29	γ'=115.0 pcf	k=25 pci
	644	soft clay	c=400 psf	γ'=52.6 pcf	ε_{50}=0.020
		stiff clay w/o water	c=2,700 psf	γ'=72.6 pcf	ε_{50}=0.005
WB South Pier	642	soft clay	c=900 psf	γ'=115.0 pcf	ε_{50}=0.020
	639	soft clay	c=500 psf	γ'=52.6 pcf	ε_{50}=0.020
		stiff clay w/o water	c=2,700 psf	γ'=72.6 pcf	ε_{50}=0.005
EB North Pier	639	sand	ϕ=29	γ'=115.0 pcf	k=25 pci
	636	soft clay	ϕ=29	γ'=52.6 pcf	k=20 pci
		stiff clay w/o water	c=2,700 psf	γ'=72.6 pcf	ε_{50}=0.005
EB South Pier	649	sand	ϕ=29	γ'=115.0 pcf	k=25 pci
		stiff clay w/o water	c=2,700 psf	γ'=72.6 pcf	ε_{50}=0.005
South Abutments	658	stiff clay w/o water	c=2,300 psf	γ'=125.0 pcf	ε_{50}=0.005
	650	soft clay	c=450 psf	γ'=52.6 pcf	ε_{50}=0.020
	642	soft clay	c=800 psf	γ'=52.6 pcf	ε_{50}=0.020
		stiff clay w/o water	c=2,700 psf	γ'=72.6 pcf	ε_{50}=0.005

The bridge is located in a region of relatively low seismic loading. The subsurface profile to a depth of 100 feet consists of less than 15 feet of soft to stiff clay, overlying very stiff clay. This profile is indicative of Site Class C. Seismic design parameters for a 1,000-year return period earthquake are listed in Table 9.2. Based on these seismic parameters, the bridge should be assigned to Seismic Performance Zone 1. The soils found at the site are not liquefaction-susceptible for the design earthquake.

Table 9.3 Seismic Design Parameters

PGA =	0.034	F_{pga} =	1.20	A_S =	0.041
S_S =	0.079	F_a =	1.20	S_{DS} =	0.095
S_1 =	0.036	F_v =	1.70	S_{D1} =	0.061

The approach slab support should be according to the current IDOT standard. The approach footing will bear on compacted embankment material. No special subgrade treatment is required.

10. Construction Considerations

Both stages of construction will require support for near-vertical cuts along the inside shoulders of EB and WB I-74. The existing embankment and native soils will provide sufficient embedment for cantilever sheet piling. The Bridge Manual's Design Guide 3.13.1 – Temporary Sheet Piling Design may be used to design any required temporary sheet piling.

Existing piles will affect the constructability of the new structures. The new piles preferably should be arranged to avoid interferences with the existing piles. If existing piles are removed prior to construction of the new structures, the holes should be backfilled with dry, loose sand.

A piling special provision is required for structures that use the project-specific pile design procedures. A draft copy of this special provision is included in the Appendix.

References

American Association of State Highway and Transportation Officials (2012). *ASHTO LRFD Bridge Design Specifications, 6th Edition.*

CH2M HILL (2009, September). Structure Geotechnical Report I-74 Over 19th Street SB Bridge Structure Nos. 081-0184 (WB) and 081-0185 (EB).

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

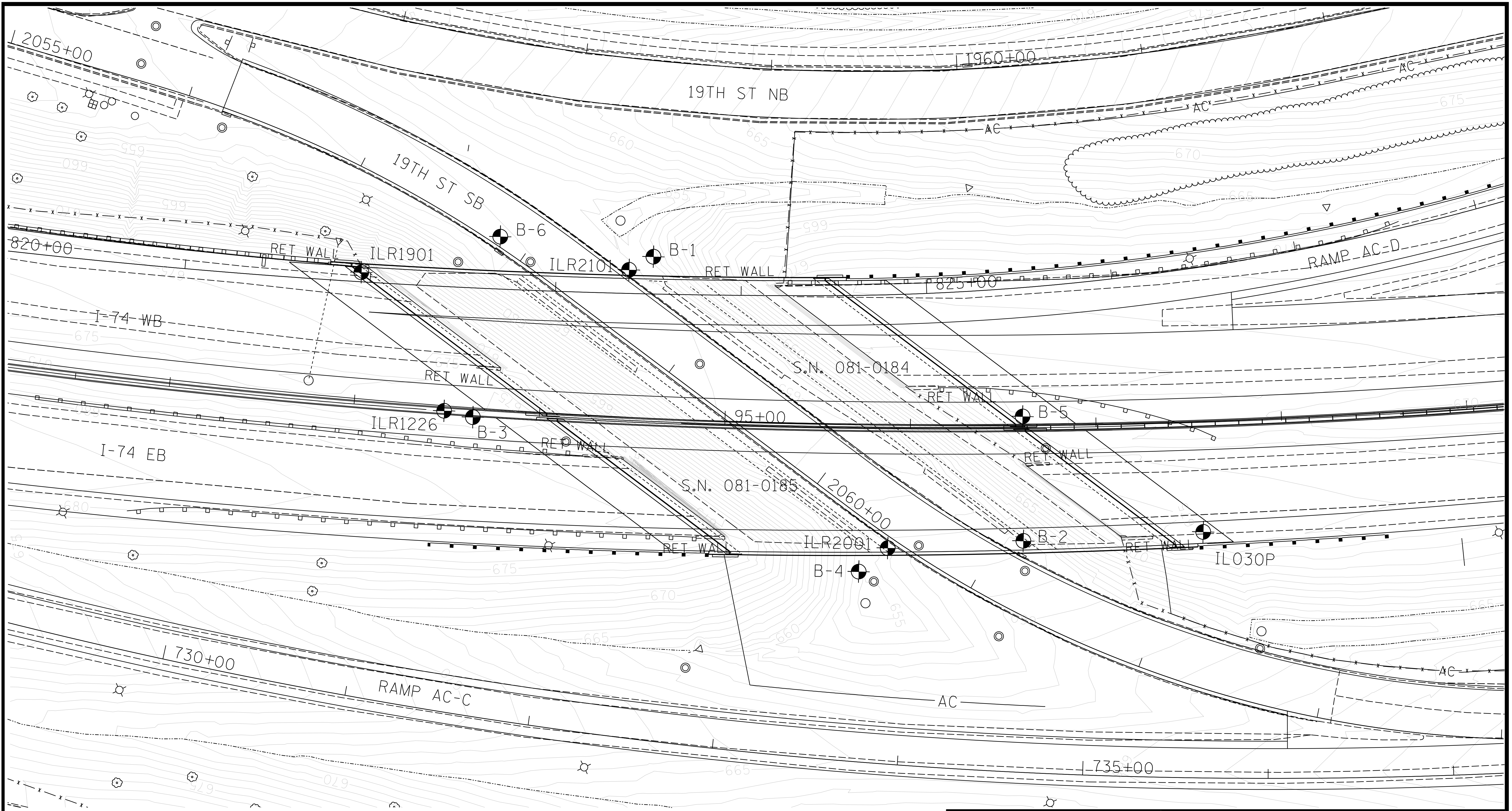
Illinois Department of Transportation (1999). *Geotechnical Manual*.

Illinois Department of Transportation (2012). *Standard Specifications for Road and Bridge Construction*.

Illinois State Geological Survey, Rock Island County coal data, Retrieved July 11, 2012 from http://www.isgs.illinois.edu/maps-data-pub/coal-maps/counties/rockisland.shtml.

Appendix

Boring Location Plan
Subsurface Data Profile
Boring Logs
Soils Laboratory Test Results
Existing Structure Plans
I-74 Pile Design Criteria
Sample Pile Design
Special Provisions



LEGEND

RW600 BORING LOCATION

0 50 100

SCALE IN FEET

N

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HANSON

BORING LOCATION PLAN

I-74 OVER 19TH STREET
S.N. 081-0184 & 081-0185
ROCK ISLAND COUNTY, ILLINOIS

08H0120D

5/30/14

chart00843

05/30/2014

DW_HPS_EJM_SGR_20120716

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

ILR1901
Sta. 92+74, 72′ LT

	N	Qu	w%		
673.62					9" thick asphalt concrete in shoulder followed by subbase
672.82	10	1.0P			Silty Clay, red brown to light grayish brown, moist, friable, possible loess
	8	<0.5P			Gray brown, low plasticity
	6		23	R	Brown with very fine sand throughout, wet, low plasticity, loess (LL=28, PI=4)
		2.5P			Grayish brown, moist
	7	3.3P	25		Dark brown, trace gravel, moist, low plasticity, possible paleosol or old topsoil (LL=49, PI=10, 7.2% organics)
655.12	8	1.5P	18		Sandy Lean Clay Trace Gravel, dark gray, very sandy, very fine gravel embedded throughout, moist, possible gumbotil, clayey part is medium plasticity
		4.0P			Glacial till, unweathered (LL=27, PI=13)
	24	4.5P			Gray, unweathered till, dry
	30	6.5P	11		
635.12	43				No recovery, possibly pounded on piece of cobble
	34				No recovery
625.12	28	3.3P	13		Sandy Lean Clay Trace Gravel, unweathered till
620.00					

620.00			
	19	3.3P	
	22	3.5P	
		3.0P	
	20	3.3P	
573.62		2.8P	Bottom of hole = 100.0 feet

ILR1226
Sta. 93+49, ℄

	N	Qu	w%		
674.93					Grass and Brown Topsoil
673.93	12	4.0P			Clayey Silt (CL-ML) - Gray with orange brown, dry, non plastic, stiff, trace fine sands, occasional root matter, oxidized, possible compacted fill
	28	4.5P			Light brown with orange brown stains, dry, very stiff, non plastic, trace silty sands and medium to fine sub angular to sub rounded gravel, possible compacted fill, brittle, strong cementation slightly oxidized
668.93	10	2.0P			Silt (ML) - Light brown with yellowish orange, moist, dry, non plastic, stiff, trace coarse sands, possible fill, compacted, fill slightly oxidized
666.93	6	2.0P			Silty Clay (CL-ML) - Yellowish orange with gray, moist, medium stiff, low to medium plasticity, oxidized, possible native soil, loess (LL=21, PI=3)
		1.5P			Light gray with orange brown stains, moist, low to medium plasticity, possible native soil
	2	<0.25P			Oxidized, moist, soft, non plastic at top 4" of sample, remainder low to medium plasticity, possible loess
		2.0P	26	R	Dark brown, dry to moist, non plastic, possible loess (LL=28, PI=7)
656.93	5	1.3P			Silt (ML) - Dark brown, dry to moist, non plastic, medium stiff, brittle, some clay, occasional root matter with gray streaks, possible native soil, loess (LL=82, PI=38)
654.93		1.8P			Lean Clay with Sand (CL) - Uniform gray, dry to moist, little coarse to fine sands scattered throughout, possible transition to glacial till
651.93	7	1.2P	19		Sandy Lean Clay (CL) - Uniform gray to greenish gray, dry to moist, low to non plastic, medium stiff to stiff, little to few coarse to fine sands, possible unweathered glacial till (LL=37, PI=23)
649.93	12	3.5P			Sandy Lean Clay Trace Gravel (CL) - gray with light gray, dry to moist, non plastic, stiff, strong cementation, medium to fine sub angular to sub rounded gravels, few coarse to fine sands, possible glacial till
	11	3.0P			Uniform gray, stiff, unweathered, glacial till
	11	1.4P			Uniform gray, unweathered glacial till with occasional sand seams, medium to fine sand seam 1" from top of sample
	15	4.0P			Uniform gray, stiff, dry to moist, unweathered glacial till with scattered sand seams
	14	4.0P			Uniform olive gray, very stiff, dry to moist, unweathered glacial till
624.93					Bottom of hole = 50.0 feet

LEGEND

N	Standard Penetration Test N (blows/ft)
Qu	Unconfined Strength (tsf)
w%	Natural Moisture Content (%)
Q	Unconsolidated Undrained Triaxial Test
R	Consolidated Undrained Triaxial Test
C	Consolidation Test
DD	Water Surface Elevation Encountered in Boring DD = during drilling 24h = 24 hours after completion
558.10 ▽	

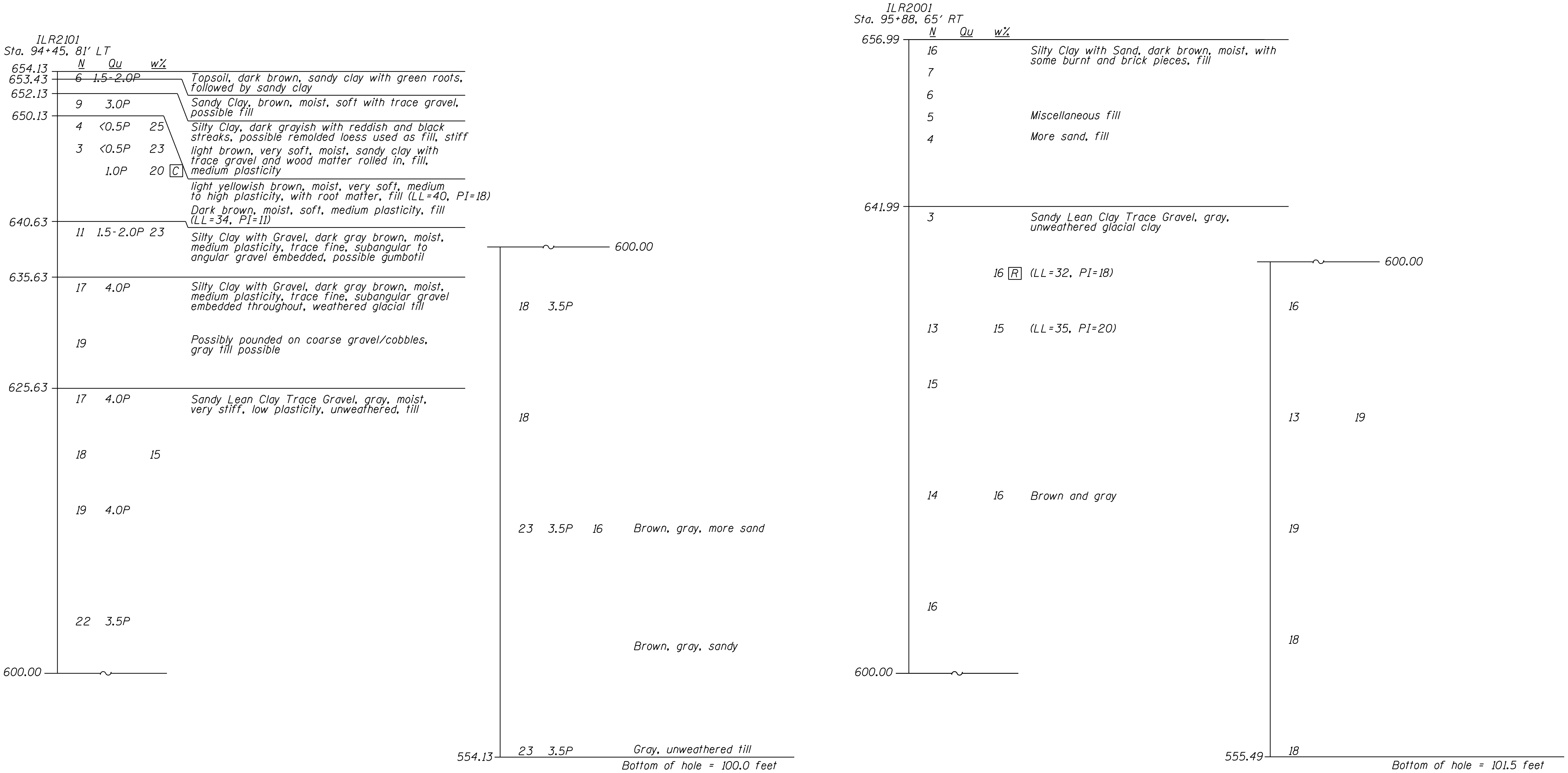
SUBSURFACE DATA PROFILE
STRUCTURE NO. 081-0184 (WB)
STRUCTURE NO. 081-0185 (EB)

PROFESSIONAL DESIGN FIRM LICENSE #184-001084

HANSON
Hanson Professional Services Inc.

JOB NO.	SHEET NO. 1	F.A.I RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
08H0120D		74	81-1HB-2	ROCK ISLAND	–	
DATE 8/6/12	3 SHEETS			CONTRACT NO. 64C08		
		FED. ROAD DIST. NO.	ILLINOIS	FED. AID PROJECT		

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION



ILR2101
Sta. 94+45, 81' LT

Elev.	N	Qu	w%	Description
654.13				
653.43	6	1.5-2.0P		Topsoil, dark brown, sandy clay with green roots, followed by sandy clay
652.13	9	3.0P		Sandy Clay, brown, moist, soft with trace gravel, possible fill
650.13	4	<0.5P	25	Silty Clay, dark grayish with reddish and black streaks, possible remolded loess used as fill, stiff
	3	<0.5P	23	light brown, very soft, moist, sandy clay with trace gravel and wood matter rolled in, fill, medium plasticity
		1.0P	20 C	
				light yellowish brown, moist, very soft, medium to high plasticity, with root matter, fill (LL=40, PI=18)
				Dark brown, moist, soft, medium plasticity, fill (LL=34, PI=11)
640.63	11	1.5-2.0P	23	Silty Clay with Gravel, dark gray brown, moist, medium plasticity, trace fine, subangular to angular gravel embedded, possible gumbotil
635.63	17	4.0P		Silty Clay with Gravel, dark gray brown, moist, medium plasticity, trace fine, subangular gravel embedded throughout, weathered glacial till
	19			Possibly pounded on coarse gravel/cobbles, gray till possible
625.63	17	4.0P		Sandy Lean Clay Trace Gravel, gray, moist, very stiff, low plasticity, unweathered, till
	18		15	
	19	4.0P		
	22	3.5P		
600.00				
600.00				
	18	3.5P		
	18			
	23	3.5P	16	Brown, gray, more sand
				Brown, gray, sandy
554.13	23	3.5P		Gray, unweathered till

Bottom of hole = 100.0 feet

ILR2001
Sta. 95+88, 65' RT

Elev.	N	Qu	w%	Description
656.99	16			Silty Clay with Sand, dark brown, moist, with some burnt and brick pieces, fill
	7			
	6			
	5			Miscellaneous fill
	4			More sand, fill
641.99	3			Sandy Lean Clay Trace Gravel, gray, unweathered glacial clay
			16 R	(LL=32, PI=18)
	13		15	(LL=35, PI=20)
	15			
	14		16	Brown and gray
	16			
600.00				
600.00				
	16			
	13		19	
	19			
	18			
555.49	18			

Bottom of hole = 101.5 feet

LEGEND

N	Standard Penetration Test N (blows/ft)
Qu	Unconfined Strength (tsf)
w%	Natural Moisture Content (%)
Q	Unconsolidated Undrained Triaxial Test
R	Consolidated Undrained Triaxial Test
C	Consolidation Test
DD 558.10 ▽	Water Surface Elevation Encountered in Boring DD = during drilling 24h = 24 hours after completion Oh = upon completion

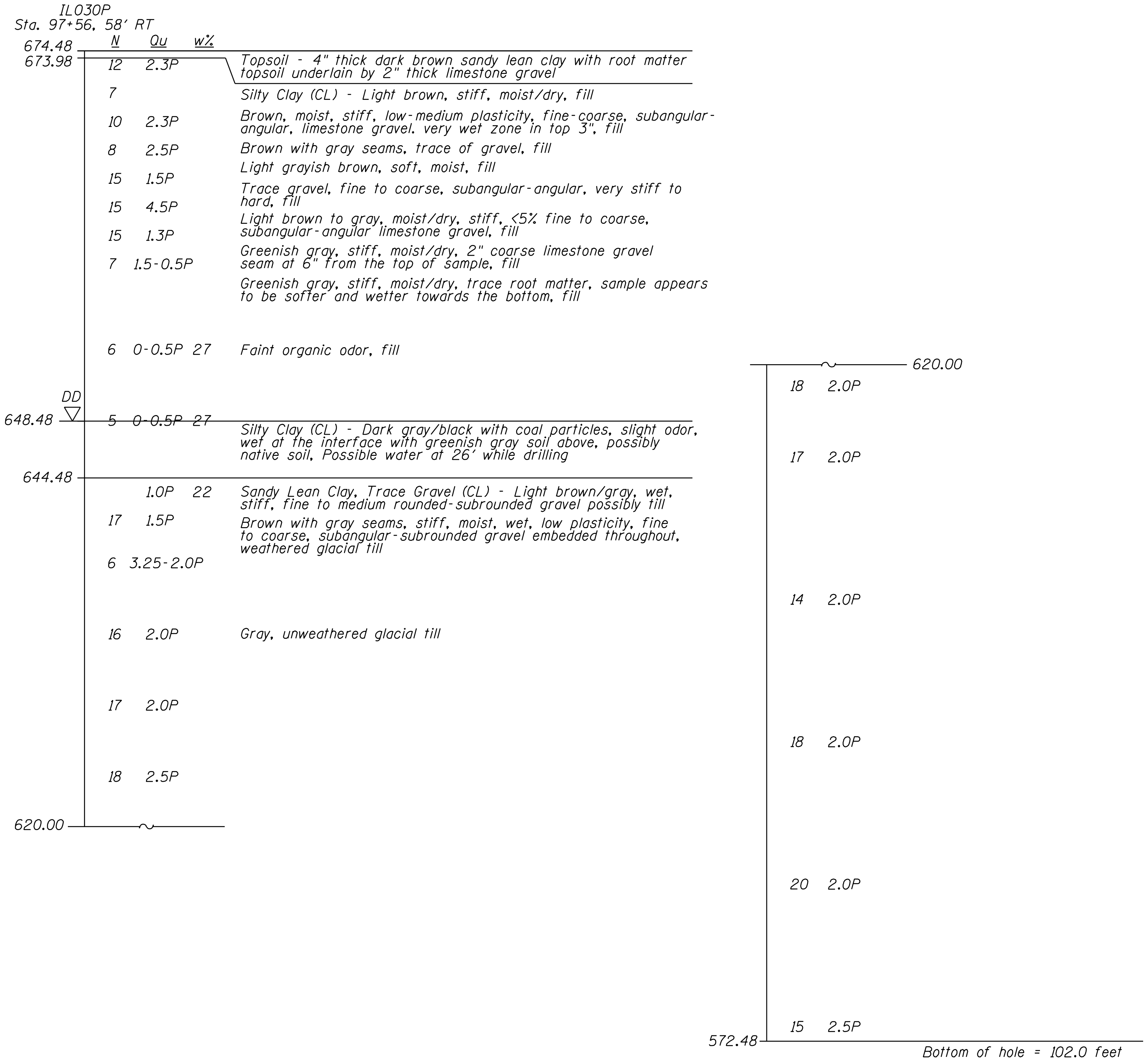
SUBSURFACE DATA PROFILE
STRUCTURE NO. 081-0184 (WB)
STRUCTURE NO. 081-0185 (EB)

PROFESSIONAL DESIGN FIRM LICENSE #184-001084

HANSON
Hanson Professional Services Inc.

JOB NO.	SHEET NO. 2	F.A.I RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
08H0120D		74	81-1HB-2	ROCK ISLAND	-	
DATE 8/6/12	3 SHEETS			CONTRACT NO. 64C08		
		FED. ROAD DIST. NO.	ILLINOIS	FED. AID PROJECT		

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION



IL030P
Sta. 97+56, 58' RT

	N	Qu	w%	
674.48				Topsoil - 4" thick dark brown sandy lean clay with root matter topsoil underlain by 2" thick limestone gravel
673.98	12	2.3P		
	7			Silty Clay (CL) - Light brown, stiff, moist/dry, fill
	10	2.3P		Brown, moist, stiff, low-medium plasticity, fine-coarse, subangular-angular, limestone gravel. very wet zone in top 3", fill
	8	2.5P		Brown with gray seams, trace of gravel, fill
				Light grayish brown, soft, moist, fill
	15	1.5P		Trace gravel, fine to coarse, subangular-angular, very stiff to hard, fill
	15	4.5P		Light brown to gray, moist/dry, stiff, <5% fine to coarse, subangular-angular limestone gravel, fill
	15	1.3P		Greenish gray, stiff, moist/dry, 2" coarse limestone gravel seam at 6" from the top of sample, fill
	7	1.5-0.5P		Greenish gray, stiff, moist/dry, trace root matter, sample appears to be softer and wetter towards the bottom, fill
	6	0-0.5P	27	Faint organic odor, fill
DD ▽ 648.48	5	0-0.5P	27	Silty Clay (CL) - Dark gray/black with coal particles, slight odor, wet at the interface with greenish gray soil above, possibly native soil, Possible water at 26' while drilling
644.48		1.0P	22	Sandy Lean Clay, Trace Gravel (CL) - Light brown/gray, wet, stiff, fine to medium rounded-subrounded gravel possibly till
	17	1.5P		Brown with gray seams, stiff, moist, wet, low plasticity, fine to coarse, subangular-subrounded gravel embedded throughout, weathered glacial till
	6	3.25-2.0P		
	16	2.0P		Gray, unweathered glacial till
	17	2.0P		
	18	2.5P		
620.00				
620.00				
	18	2.0P		
	17	2.0P		
	14	2.0P		
	18	2.0P		
	20	2.0P		
	15	2.5P		
572.48				Bottom of hole = 102.0 feet

LEGEND

N	Standard Penetration Test N (blows/ft)
Qu	Unconfined Strength (tsf)
w%	Natural Moisture Content (%)
Q	Unconsolidated Undrained Triaxial Test
R	Consolidated Undrained Triaxial Test
C	Consolidation Test
DD ▽ 558.10	Water Surface Elevation Encountered in Boring DD = during drilling 24h = 24 hours after completion Oh = upon completion

SUBSURFACE DATA PROFILE
STRUCTURE NO. 081-0184 (WB)
STRUCTURE NO. 081-0185 (EB)

PROFESSIONAL DESIGN FIRM LICENSE #184-001084

Hanson Professional Services Inc.

JOB NO. 08H0120D	SHEET NO. 3	F.A.I RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
DATE 8/6/12	3 SHEETS	74	81-1HB-2	ROCK ISLAND	_	
				CONTRACT NO. 64C08		
		FED. ROAD DIST. NO.	ILLINOIS	FED. AID PROJECT		

Illinois Department of Transportation
Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Page 1 of 2

Date 1/10/08

ROUTE FAI 74 **DESCRIPTION** P92-032-01 Bridge carrying I-74 over SB 19th Street in Moline **LOGGED BY** W. Garza

SECTION 81(1-2,1,2-2)RS-1&M **LOCATION** Moline Twp. - 4 NW, **SEC.** , **TWP.** 17N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** CME-45 Automatic

STRUCT. NO. 081-103, 104
Station

BORING NO. B-1
Station 323+52
Offset 87.00ft Lt CL
Ground Surface Elev. 655.6 ft

Surface Water Elev. 655.1 ft
Stream Bed Elev. 81.5 ft

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

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
MEDIUM brown SILTY CLAY LOAM	653.60			0.8 P	21
STIFF tan SILTY CLAY LOAM	652.10		1 3 4	1.3 P	23
SOFT tan SILTY LOAM	649.60	-5	1 3 3	0.4 P	26
STIFF tan SILTY LOAM	647.10		2 4 6	1.7 S	20
SOFT tan SILTY LOAM	644.60	-10	2 2 2	0.4 P	24
STIFF gray LOAM with GRAVEL	642.10		3 2 3	1.2 B	20
MEDIUM dark gray SANDY LOAM	639.10	-15	1 1 3	0.5 B	20
VERY STIFF brown SILTY CLAY TILL	637.10		1 3 5	3.3 B	15
		-20	4		

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL *(continued)*	634.60		6 8	2.9 B	14
VERY STIFF gray CLAY LOAM TILL	632.10		4 6 11	3.3 B	14
VERY STIFF gray CLAY LOAM TILL	629.60	-25	5 7 11	2.7 B	14
VERY STIFF gray CLAY LOAM TILL	627.10		6 9 11	2.7 B	15
VERY STIFF gray CLAY LOAM TILL	624.60	-30	5 8 12	3.1 B	13
VERY STIFF gray CLAY LOAM TILL	622.10		4 7 10	3.7 B	14
VERY STIFF gray CLAY LOAM TILL	619.60	-35	5 8 11	3.7 B	14
VERY STIFF gray CLAY LOAM TILL	617.10		4 7 10	2.9 B	13
		-40	5		

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)

Illinois Department of Transportation
Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Page 2 of 2

Date 1/10/08

ROUTE FAI 74 **DESCRIPTION** P92-032-01 Bridge carrying I-74 over SB 19th Street in Moline **LOGGED BY** W. Garza

SECTION 81(1-2,1,2-2)RS-1&M **LOCATION** Moline Twp. - 4 NW, **SEC.** , **TWP.** 17N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** CME-45 Automatic

STRUCT. NO. 081-103, 104
Station

BORING NO. B-1
Station 323+52
Offset 87.00ft Lt CL
Ground Surface Elev. 655.6 ft

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL *(continued)*	614.60		9 12	3.1 B	15
VERY STIFF gray CLAY LOAM TILL	612.10		5 9 15	2.9 B	14
VERY STIFF gray CLAY LOAM TILL	609.60	-45	6 8 10	3.3 B	15
VERY STIFF gray CLAY LOAM TILL	607.10		7 9 14	3.9 B	15
VERY STIFF gray CLAY LOAM TILL	604.60	-50	6 8 14	3.5 B	15
VERY STIFF gray CLAY LOAM TILL	602.10		7 9 14	3.5 B	14
VERY STIFF gray CLAY LOAM TILL	599.60	-55	6 8 13	3.7 B	15
HARD gray CLAY LOAM TILL	597.10		5 7 11	4.1 B	16
		-60	6		

Surface Water Elev. 655.1 ft
Stream Bed Elev. 81.5 ft

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

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF olive green CLAY LOAM TILL *(continued)*	594.60		8 12	3.9 B	16
HARD olive green CLAY LOAM TILL	592.10		11 11 13	4.1 B	18
VERY STIFF olive-green CLAY LOAM TILL	589.60	-65	8 13 22	3.7 B	15
HARD gray CLAY LOAM TILL	587.10		7 9 14	4.5 S	14
End of Boring					
		-70			
		-75			
		-80			

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)

Illinois Department of Transportation
Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Page 1 of 2

Date 1/11/08

ROUTE FAI 74 **DESCRIPTION** P92-032-01 Bridge carrying I-74 over SB 19th Street in Moline **LOGGED BY** J. Strating

SECTION 81(1-2,1,2-2)RS-1&M **LOCATION** Moline Twp. - 4 NW, **SEC.** , **TWP.** 17N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** B-53 Diedrich Automatic

STRUCT. NO.
Station

BORING NO. B-2
Station 325+55
Offset 62.00ft Rt CL
Ground Surface Elev. 655.6 ft

Surface Water Elev. 655.1 ft
Stream Bed Elev. 81.5 ft

Groundwater Elev.:
First Encounter 648.1 ft ▼
Upon Completion ft
After **Hrs.** ft

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
MEDIUM brown SILTY CLAY LOAM	653.10			0.5 P	19
LOOSE brown clean medium SAND (fill)	651.60		3 2 2		
VERY LOOSE brown clean medium SAND (fill)	648.60	-5	2 0 1		
VERY STIFF brown/gray CLAY LOAM	646.60	▼	4 5 7	2.4 B	14
VERY STIFF brown CLAY LOAM TILL	644.10	-10	5 8 11	3.3 B	15
VERY STIFF brown/gray CLAY LOAM TILL	641.60		7 9 13	3.5 B	15
VERY STIFF gray CLAY LOAM TILL	639.10	-15	3 5 7	2.6 B	14
VERY STIFF gray CLAY LOAM TILL	636.60		3 5 7	2.4 B	14
		-20			

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL	634.10		3 5 7	2.0 B	14
VERY STIFF gray CLAY LOAM TILL	631.60		3 6 10	3.0 B	14
VERY STIFF gray CLAY LOAM TILL	629.10	-25	2 6 9	3.0 B	14
VERY STIFF gray CLAY LOAM TILL	626.60		3 5 9	2.8 B	13
VERY STIFF gray CLAY LOAM TILL	624.10	-30	2 6 9	2.8 B	14
VERY STIFF gray CLAY LOAM TILL	621.60		3 6 11	2.8 B	14
HARD dark gray CLAY LOAM TILL with ORGANICS	619.10	-35	3 7 13	4.1 B	15
HARD dark gray CLAY LOAM TILL with SAND lens	616.60		3 11 15	5.1 B	15
		-40			

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)

Illinois Department of Transportation
Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Date 1/11/08

ROUTE FAI 74

DESCRIPTION P92-032-01 Bridge carrying I-74 over SB 19th Street in Moline

LOGGED BY J. Strating

SECTION 81(1-2,1,2-2)RS-1&M

LOCATION Moline Twp. - 4 NW, **SEC.** , **TWP.** 17N, **RNG.** 1W

COUNTY Rock Island

DRILLING METHOD Hollow Stem Auger

HAMMER TYPE B-53 Diedrich Automatic

STRUCT. NO.
Station

BORING NO. B-2
Station 325+55
Offset 62.00ft Rt CL
Ground Surface Elev. 655.6 ft

Surface Water Elev. 655.1 ft
Stream Bed Elev. 81.5 ft

Groundwater Elev.:
First Encounter 648.1 ft ▼
Upon Completion ft
After Hrs. ft

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray/green CLAY LOAM TILL 614.10		4 6 9	2.8 B	15
VERY STIFF gray/green CLAY LOAM TILL 611.60		3 8 12	3.0 B	14
	-45			
VERY STIFF gray/green CLAY LOAM TILL 609.10		2 8 12	2.8 B	15
VERY STIFF gray/green CLAY LOAM TILL 606.60		2 6 9	2.8 B	15
	-50			
VERY STIFF gray/green CLAY LOAM TILL 604.10		3 7 9	2.8 B	14
VERY STIFF gray/green CLAY LOAM TILL 601.60		3 7 10	2.8 B	16
	-55			
VERY STIFF gray/green CLAY LOAM TILL 599.10		2 6 11	2.8 B	16
VERY STIFF gray/green CLAY LOAM TILL 596.60		3 7 10	3.5 B	15
	-60			

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray/green SILTY CLAY LOAM 594.10		2 6 10	2.8 B	15
VERY STIFF gray/green CLAY LOAM TILL 591.60		1 6 9	2.8 B	16
	-65			
VERY STIFF gray/green CLAY LOAM TILL 589.10		1 6 10	2.6 B	19
VERY STIFF gray/green CLAY LOAM TILL 586.60		3 5 11	2.6 B	17
End of Boring				
	-70			
	-75			
	-80			

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)

Illinois Department of Transportation
Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Page 1 of 3

Date 1/14/08

ROUTE FAI 74 **DESCRIPTION** P92-032-01 Bridge carrying I-74 over SB 19th Street in Moline **LOGGED BY** W. Garza

SECTION 81(1-2,1,2-2)RS-1&M **LOCATION** Moline Twp. - 4 NW, **SEC.** , **TWP.** 17N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** B-53 Diedrich Automatic

STRUCT. NO.
Station 324+00

BORING NO. B-3
Station 322+59
Offset 5.00ft Lt CL
Ground Surface Elev. 674.6 ft

Surface Water Elev. 655.1 ft
Stream Bed Elev. 81.5 ft

Groundwater Elev.:
First Encounter 654.6 ft ▼
Upon Completion ft
After Hrs. ft

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
MEDIUM light brown SILTY LOAM				0.5 P	20
	672.10				
STIFF dark brown LOAM	670.60		4 6 15	1.5 B	18
		-5			
STIFF tan SILTY LOAM	668.10		9 10 9	1.8 P	19
STIFF tan SILTY LOAM	665.60		3 5 6	1.2 B	23
		-10			
MEDIUM tan SILTY LOAM	663.10		2 3 4	0.5 S	25
MEDIUM rust SILTY LOAM	660.60		1 1 3	0.8 P	30
		-15			
MEDIUM light brown SILTY LOAM	658.10		1 3 4	0.6 S	37
STIFF gray CLAY LOAM	655.60		3 3 5	1.7 B	25
		▼-20			

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
STIFF gray CLAY LOAM	653.10		1 3 4	1.6 B	24
VERY STIFF gray SILTY CLAY	650.60		4 5 7	2.0	23
		-25			
STIFF gray SILTY CLAY LOAM with SAND lens	647.60		1 4 6	1.8 P	14
STIFF gray SILTY CLAY TILL	645.60		5 6 7	1.5 P	28
		-30			
STIFF gray CLAY LOAM TILL	643.10		5 8 11	1.3 P	16
VERY STIFF gray CLAY LOAM TILL	640.60		1 4 7	2.3 B	16
		-35			
VERY STIFF gray CLAY LOAM TILL	638.10		3 5 9	2.3 B	15
VERY STIFF gray CLAY LOAM TILL	635.60		4 6 9	2.3 B	15
		-40			

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)

SOIL BORING LOG

Date 1/14/08

ROUTE FAI 74 **DESCRIPTION** P92-032-01 Bridge carrying I-74 over SB 19th Street in Moline **LOGGED BY** W. Garza

SECTION 81(1-2,1,2-2)RS-1&M **LOCATION** Moline Twp. - 4 NW, **SEC.** , **TWP.** 17N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** B-53 Diedrich Automatic

STRUCT. NO.
Station 324+00

BORING NO. B-3
Station 322+59
Offset 5.00ft Lt CL
Ground Surface Elev. 674.6 ft

Surface Water Elev. 655.1 ft
Stream Bed Elev. 81.5 ft

Groundwater Elev.:
First Encounter 654.6 ft ▼
Upon Completion ft
After Hrs. ft

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL	633.10		4 5 8	2.7 B	15
VERY STIFF gray CLAY LOAM TILL	630.60		3 8 10	2.0 B	16
		-45			
VERY STIFF gray CLAY LOAM TILL	628.10		3 6 9	2.7 B	16
VERY STIFF gray CLAY LOAM TILL	625.60		2 6 10	2.9 B	15
		-50			
VERY STIFF gray CLAY LOAM TILL	623.10		3 7 11	3.3 B	15
VERY STIFF gray CLAY LOAM TILL	620.60		4 8 11	3.5 B	15
		-55			
VERY STIFF gray CLAY LOAM TILL	618.10		4 5 9	2.3 B	14
VERY STIFF gray CLAY LOAM TILL	615.60		6 7 12	3.3 B	16
		-60			

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL	613.10		5 9 12	3.3 B	15
VERY STIFF gray CLAY LOAM TILL	610.60		6 9 12	2.5 B	13
		-65			
HARD gray CLAY LOAM TILL	608.10		6 9 13	4.3 B	15
VERY STIFF gray CLAY LOAM TILL	605.60		5 9 11	3.3 B	16
		-70			
STIFF gray CLAY LOAM TILL with SAND lens	603.10		8 10 15	1.7 B	16
HARD gray CLAY LOAM TILL	600.60		7 10 12	4.3 B	14
		-75			
VERY STIFF gray CLAY LOAM TILL	598.10		3 7 12	3.9 B	14
HARD gray CLAY LOAM TILL	595.60		6 12 16	5.0 B	16
		-80			

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)

Illinois Department of Transportation
Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Page 3 of 3

Date 1/14/08

ROUTE FAI 74 **DESCRIPTION** P92-032-01 Bridge carrying I-74 over SB 19th Street in Moline **LOGGED BY** W. Garza

SECTION 81(1-2,1,2-2)RS-1&M **LOCATION** Moline Twp. - 4 NW, **SEC.** , **TWP.** 17N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** B-53 Diedrich Automatic

STRUCT. NO.
Station 324+00

BORING NO. B-3
Station 322+59
Offset 5.00ft Lt CL
Ground Surface Elev. 674.6 ft

Surface Water Elev. 655.1 ft
Stream Bed Elev. 81.5 ft

Groundwater Elev.:
First Encounter 654.6 ft ▼
Upon Completion ft
After Hrs. ft

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL	593.10		4 9 13	3.9 B	18
HARD gray CLAY LOAM TILL	590.60		9 12 16	4.5 B	17
		-85			
little recovery CLAY LOAM TILL	588.10		12 16 18		
VERY STIFF gray CLAY LOAM TILL	585.60		5 8 11	3.1 B	17
End of Boring					
		-90			
		-95			
		-100			

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)

Illinois Department of Transportation
Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Page 1 of 2

Date 1/15/08

ROUTE FAI 74 **DESCRIPTION** P92-032-01 Bridge carrying I-74 over SB 19th Street in Moline **LOGGED BY** W. Garza

SECTION 81(1-2,1,2-2)RS-1&M **LOCATION** Moline Twp. - 4 NW, **SEC.** , **TWP.** 17N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** B-53 Diedrich Automatic

STRUCT. NO.
Station 324+00

BORING NO. B-4
Station 324+68
Offset 79.00ft Rt CL
Ground Surface Elev. 653.1 ft

Surface Water Elev. 655.1 ft
Stream Bed Elev. 81.5 ft

Groundwater Elev.:
First Encounter 638.1 ft ▼
Upon Completion 631.1 ft ▽
After _____ **Hrs.** _____ ft

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
MEDIUM dark brown LOAM	650.60			0.5 P	16
STIFF brown SILTY CLAY LOAM with SAND lens	648.60	-5	2 3 7	1.5 P	26
LOOSE tan clean medium SAND	646.60		4 3 3		
LOOSE dark brown dirty SAND	644.10	-10	5 2 3		
LOOSE dark brown dirty moist SAND	641.60		1 2 2		
VERY LOOSE gray moist dirty SAND	638.60	▼-15	1 1 2		
SOFT gray SILTY LOAM	636.60		1 1 4	0.3 P	33
No Recovery Assume as above	633.60	-20	4 8 10		

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF tan CLAY LOAM TILL	631.60	▽	5 7 9	3.1 B	14
VERY STIFF gray CLAY LOAM TILL	629.10	-25	3 5 9	2.7 B	15
VERY STIFF gray CLAY LOAM TILL	626.60		3 6 10	2.9 B	15
VERY STIFF gray CLAY LOAM TILL	624.10	-30	5 7 10	2.7 B	16
VERY STIFF gray CLAY LOAM TILL	621.60		5 7 11	3.3 B	15
VERY STIFF gray CLAY LOAM TILL	619.10	-35	5 8 13	3.5 B	15
VERY STIFF gray CLAY LOAM TILL	616.60		4 9 14	2.9 B	14
VERY STIFF gray CLAY LOAM TILL	614.10	-40	5 9 13	2.3 B	15

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)

Illinois Department of Transportation
Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Page 2 of 2

Date 1/15/08

ROUTE FAI 74 **DESCRIPTION** P92-032-01 Bridge carrying I-74 over SB 19th Street in Moline **LOGGED BY** W. Garza

SECTION 81(1-2,1,2-2)RS-1&M **LOCATION** Moline Twp. - 4 NW, **SEC.** , **TWP.** 17N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** B-53 Diedrich Automatic

STRUCT. NO.
Station 324+00

BORING NO. B-4
Station 324+68
Offset 79.00ft Rt CL
Ground Surface Elev. 653.1 ft

Surface Water Elev. 655.1 ft
Stream Bed Elev. 81.5 ft

Groundwater Elev.:
First Encounter 638.1 ft ▼
Upon Completion 631.1 ft ▽
After ______ **Hrs.** ______ ft

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL	611.60		8 11 15	3.3 B	14
VERY STIFF gray CLAY LOAM TILL	609.10		8 12 17	3.5 B	16
		-45			
HARD gray CLAY LOAM TILL	606.60		6 11 15	4.1 B	14
VERY STIFF gray CLAY LOAM TILL	604.10		4 8 13	3.3 B	17
		-50			
VERY STIFF gray CLAY LOAM TILL	601.60		5 11 14	3.3 B	17
HARD gray CLAY LOAM TILL	599.10		8 16 19	4.1 B	15
		-55			
VERY STIFF gray CLAY LOAM TILL	596.60		7 9 15	2.5 B	17
VERY STIFF gray CLAY LOAM TILL with weathered LIMESTONE lens	594.10		5 10 29	3.1 B	14
		-60			

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL	591.60		7 10 15	3.1 B	20
VERY STIFF gray CLAY LOAM TILL	589.10		8 12 16	3.3 B	15
		-65			
VERY STIFF gray CLAY LOAM TILL	586.60		6 8 10	2.1 B	17
VERY STIFF gray CLAY LOAM TILL	584.10		7 10 13	2.9 B	16
		-70			
VERY STIFF gray CLAY LOAM TILL	581.60		8 11 15	3.5 B	19
VERY STIFF gray CLAY LOAM TILL	579.10		7 9 12	3.5 B	18
		-75			
VERY STIFF gray CLAY LOAM TILL	576.60		7 8 12	2.9 B	19
End of Boring					
		-80			

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)

Illinois Department of Transportation
Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Date 3/10/08

ROUTE FAI 74 **DESCRIPTION** P92-032-01 Bridge carrying I-74 over SB 19th Street in Moline **LOGGED BY** W. Garza

SECTION 81(1-2,1,2-2)RS-1&M **LOCATION** Moline Twp. - 4 NW, **SEC.** , **TWP.** 17N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** B-53 Diedrich Automatic

STRUCT. NO.
Station 324+00

BORING NO. B-5
Station 325+55
Offset 5.00ft Lt CL
Ground Surface Elev. 673.1 ft

Surface Water Elev. 655.1 ft
Stream Bed Elev. 81.5 ft

Groundwater Elev.:
First Encounter 653.1 ft ▼
Upon Completion Wash ft
After ____ **Hrs.** ____ ft

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
MEDIUM light brown SILTY LOAM	670.60			0.5 P	17
VERY STIFF tan SILTY LOAM	669.10		6 7 7	2.8 P	21
		-5			
STIFF gray SILTY CLAY LOAM	666.60		4 5 8	1.4 B	23
VERY STIFF gray SILTY LOAM	664.10		5 5 7	2.1 S	19
		-10			
VERY STIFF gray SILTY LOAM with GRAVEL	661.60		12 6 7	2.5 P	19
No Recovery	659.10		3 4 6		
		-15			
SOFT gray SILTY LOAM	656.60		1 1 2	0.3 B	30
SOFT gray SILTY LOAM	654.10		0 1 3	0.3 P	27
		▼-20			

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
MEDIUM gray/brown LOAM with 11% ORGANICS	651.10		0 2 3	0.7 B	37
VERY STIFF light brown CLAY LOAM TILL	649.10		2 8 10	3.1 B	15
		-25			
VERY STIFF light brown CLAY LOAM TILL	646.60		8 10 15	2.9 B	14
VERY STIFF gray CLAY LOAM TILL	644.10		8 10 13	3.7 S	13
		-30			
VERY STIFF gray CLAY LOAM TILL	641.60		3 5 8	2.7 B	14
VERY STIFF gray CLAY LOAM TILL	639.10		8 10 14	2.9 B	14
		-35			
VERY STIFF gray CLAY LOAM TILL	636.60		7 9 12	2.9 B	14
VERY STIFF gray CLAY LOAM TILL	634.10		6 8 12	3.1 B	14
		-40			

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)

Illinois Department of Transportation
Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Date 3/10/08

ROUTE FAI 74 **DESCRIPTION** P92-032-01 Bridge carrying I-74 over SB 19th Street in Moline **LOGGED BY** W. Garza

SECTION 81(1-2,1,2-2)RS-1&M **LOCATION** Moline Twp. - 4 NW, **SEC.** , **TWP.** 17N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** B-53 Diedrich Automatic

STRUCT. NO.
Station 324+00

BORING NO. B-5
Station 325+55
Offset 5.00ft Lt CL
Ground Surface Elev. 673.1 ft

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL	631.60		6 8 11	3.3 B	14
Wash VERY STIFF gray CLAY LOAM TILL	629.10		3 7 10	3.3 B	15
		-45			
VERY STIFF gray CLAY LOAM TILL	626.60		6 9 11	2.9 B	15
VERY STIFF gray CLAY LOAM TILL	624.10		7 12 17	3.5 B	15
		-50			
VERY STIFF gray CLAY LOAM TILL	621.60		9 11 17	3.3 B	15
Wash VERY STIFF gray CLAY LOAM TILL	619.10		5 6 10	2.7 B	14
		-55			
VERY STIFF gray CLAY LOAM TILL	616.60		10 11 15	3.3 B	15
VERY STIFF gray CLAY LOAM TILL	614.10		5 8 12	2.7 B	17
		-60			

Surface Water Elev. 655.1 ft
Stream Bed Elev. 81.5 ft

Groundwater Elev.:
First Encounter 653.1 ft ▼
Upon Completion Wash ft
After ____ **Hrs.** ____ ft

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL	611.60		9 10 11	2.9 B	14
HARD olive-green CLAY LOAM TILL	609.10		7 11 15	4.1 B	16
		-65			
VERY STIFF gray CLAY LOAM TILL	606.60		6 9 12	3.7 B	16
VERY STIFF gray CLAY LOAM TILL	604.10		5 8 11	3.3 B	17
		-70			
VERY STIFF gray CLAY LOAM TILL	601.60		6 11 15	2.7 B	15
VERY STIFF gray CLAY LOAM TILL	599.10		6 8 11	3.1 B	20
		-75			
VERY STIFF gray CLAY LOAM TILL	596.60		7 10 15	3.5 B	16
VERY STIFF gray CLAY LOAM TILL	594.10		6 8 10	3.7 B	30
		-80			

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)

Illinois Department of Transportation
Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Page 3 of 3

Date 3/10/08

ROUTE FAI 74 **DESCRIPTION** P92-032-01 Bridge carrying I-74 over SB 19th Street in Moline **LOGGED BY** W. Garza

SECTION 81(1-2,1,2-2)RS-1&M **LOCATION** Moline Twp. - 4 NW, **SEC.** , **TWP.** 17N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** B-53 Diedrich Automatic

STRUCT. NO.
Station 324+00

BORING NO. B-5
Station 325+55
Offset 5.00ft Lt CL
Ground Surface Elev. 673.1 ft

Surface Water Elev. 655.1 ft
Stream Bed Elev. 81.5 ft

Groundwater Elev.:
First Encounter 653.1 ft ▼
Upon Completion Wash ft
After ____ **Hrs.** ____ ft

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL	591.60		6 10 12	2.7 B	21
VERY STIFF gray CLAY LOAM TILL	589.10		4 9 12	2.9 B	18
		-85			
VERY STIFF gray CLAY LOAM TILL	586.60		6 8 11	2.7 B	18
VERY STIFF gray CLAY LOAM TILL	584.10		5 6 10	2.9 B	18
End of Boring					
		-90			
		-95			
		-100			

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)

Illinois Department of Transportation

Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Page 1 of 2

Date 3/11/08

ROUTE FAI 74 **DESCRIPTION** P92-032-01 Bridge carrying I-74 over SB 19th Street in Moline **LOGGED BY** W. Garza

SECTION 81(1-2,1,2-2)RS-1&M **LOCATION** Moline Twp. - 4 NW, **SEC.** , **TWP.** 17N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** B-53 Diedrich Automatic

STRUCT. NO.
Station 324+00

BORING NO. B-6
Station 322+67
Offset 93.00ft Lt CL
Ground Surface Elev. 655.9 ft

Surface Water Elev. 655.1 ft
Stream Bed Elev. 81.5 ft

Groundwater Elev.:
First Encounter None ft
Upon Completion Dry ft
After ___ **Hrs.** ft

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
DRY brown fine SAND	653.40				
LOOSE brown fine SAND	651.90		2 1 3		
LOOSE brown fine SAND	649.40	-5	2 3 4		
LOOSE brown fine SAND	646.40		2 1 4		
SOFT gray LOAM with SAND	643.90	-10	2 2 2	0.4 P	18
STIFF gray CLAY LOAM TILL	641.90		0 1 3	1.9 B	15
VERY STIFF gray CLAY LOAM TILL	639.40	-15	3 6 9	2.5 B	14
STIFF gray CLAY LOAM TILL	636.90		4 7 9	1.9 B	14
		-20			

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL	634.40		4 8 8	2.3 B	15
VERY STIFF gray CLAY LOAM TILL	631.90		4 6 9	2.7 B	15
VERY STIFF gray CLAY LOAM TILL	629.40	-25	5 6 11	2.9 B	16
VERY STIFF gray CLAY LOAM TILL	626.90		3 5 9	2.1 B	16
VERY STIFF gray CLAY LOAM TILL	624.40	-30	3 7 10	3.1 B	14
VERY STIFF gray CLAY LOAM TILL	621.90		3 6 9	2.9 B	14
VERY STIFF gray CLAY LOAM TILL with COAL	619.40	-35	4 7 10	2.7 B	14
VERY STIFF gray CLAY LOAM TILL	616.90		3 5 8	2.1 B	16
		-40			

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)

Illinois Department of Transportation
Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Page 2 of 2

Date 3/11/08

ROUTE FAI 74 **DESCRIPTION** P92-032-01 Bridge carrying I-74 over SB 19th Street in Moline **LOGGED BY** W. Garza

SECTION 81(1-2,1,2-2)RS-1&M **LOCATION** Moline Twp. - 4 NW, **SEC.** , **TWP.** 17N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** B-53 Diedrich Automatic

STRUCT. NO.
Station 324+00

BORING NO. B-6
Station 322+67
Offset 93.00ft Lt CL
Ground Surface Elev. 655.9 ft

Surface Water Elev. 655.1 ft
Stream Bed Elev. 81.5 ft

Groundwater Elev.:
First Encounter None ft
Upon Completion Dry ft
After ____ **Hrs.** ____ ft

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL	614.40		4 8 9	2.7 B	15
VERY STIFF gray CLAY LOAM TILL	611.90		2 7 11	2.9 B	14
		-45			
VERY STIFF gray CLAY LOAM TILL	609.40		2 8 10	3.3 B	19
VERY STIFF gray CLAY LOAM TILL	606.90		2 7 10	2.9 B	18
		-50			
VERY STIFF gray CLAY LOAM TILL	604.40		2 6 10	2.1 B	15
VERY STIFF gray CLAY LOAM TILL	601.90		3 8 11	2.9 B	18
		-55			
VERY STIFF gray CLAY LOAM TILL	599.40		3 6 10	3.0 B	17
VERY STIFF gray CLAY LOAM TILL	596.90		2 6 10	3.1 B	15
		-60			

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL	594.40		4 8 10	3.3 B	16
	591.90				
		-65			
VERY STIFF gray CLAY LOAM TILL	589.40		2 6 9	2.5 B	14
	586.90				
		-70			
VERY STIFF gray CLAY LOAM TILL	584.40		4 9 14	3.7 B	13
	581.90				
		-75			
VERY STIFF gray CLAY LOAM TILL	579.40		2 6 8	2.5 B	16
End of Boring					
		-80			

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)

Illinois Department of Transportation
Division of Highways
CH2M HILL

SOIL BORING LOG

Page 1 of 3

Date 12/12/05

ROUTE I-74 **DESCRIPTION** New I-74 Bridge Over Mississippi River - Illinois Approach **LOGGED BY** B. Karnik

SECTION I-74 Bridge over Mississippi River **LOCATION** (N=558179.679, E=2460420.083), **SEC.** 32, **TWP.** 18N, **RNG.** 1W, 4th **PM**

COUNTY Rock Island **DRILLING METHOD** HSA, CME 55 **HAMMER TYPE** CME AUTOMATIC

STRUCT. NO.
Station

BORING NO. IL030P
Station 97+55.72
Offset 58.20ft
Ground Surface Elev. 674.48 ft

Surface Water Elev. ft
Stream Bed Elev. ft

Groundwater Elev.:
First Encounter 648.5 ft ▼
Upon Completion ft
After Hrs. ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Topsoil 4" thick dark brown sandy lean clay with root matter topsoil underlain by 2" thick limestone gravel□Hole offset to south abutment gps #47	673.98		5 7 5 5	2.3 P	
Silty Clay(CL) Light brown, stiff, moist/dry, fill Brown, moist, stiff, low-medium plasticity, fine-coarse, subangular-angular, limestone gravel, very wet zone in top 3", fill			2 3 4 5		
Brown with gray seams, trace of gravel, fill		-5	3 5 5 6	2.3 P	
Light grayish brown, soft, moist, fill			3 3 5 5	2.5 P	
Trace gravel, fine to coarse, subangular-angular, very stiff to hard, fill		-10	5 7 8 9	1.5 P	
Light brown to gray, moist/dry, stiff, <5% fine to coarse, subangular-angular limestone gravel, fill			6 7 8 10	4.5 P	
Greenish gray, stiff, moist/dry, 2" coarse limestone gravel seam at 6" from the top of the sample, fill			5 7 8 6	1.3 P	
Greenish gray, stiff, moist/dry, trace root matter, sample appears to be softer and wetter towards the bottom, fill		-15	2 3 4 5	2.0, 1.5-0.5 P	
		-20			

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Silty Clay(CL) Light brown, stiff, moist/dry, fill *(continued)* Faint organic odor, fill			2 3 3 4	0-0.5 P	27.0
		-25	2 2 3 2	0-0.5 P	27.0
Silty Clay(CL) Dark gray/black with coal particles, slight odor, wet at the interface with the greenish gray soil above, possibly native soil□Possible water at 26' while drilling	648.48 ▼				
Sandy Lean Clay, Trace Gravel (CL) Light brown/gray, wet, stiff, fine to medium rounded-subrounded gravel possibly till	644.48	-30		1.0 P	22.0
Brown with gray seams, stiff, moist, wet, low plasticity, fine to coarse, subangular-subrounded gravel embedded throughout, weathered glacial till			6 7 10 10	1.5 P	
		-35	3 4 2 9	3.25-2.0 P	
		-40			

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)

SOIL BORING LOG

Page 2 **of** 3

Date 12/12/05

ROUTE I-74 **DESCRIPTION** New I-74 Bridge Over Mississippi River - Illinois Approach **LOGGED BY** B. Karnik

SECTION I-74 Bridge over Mississippi River **LOCATION** (N=558179.679, E=2460420.083), **SEC.** 32, **TWP.** 18N, **RNG.** 1W, 4th **PM**

COUNTY Rock Island **DRILLING METHOD** HSA, CME 55 **HAMMER TYPE** CME AUTOMATIC

STRUCT. NO. ______
Station ______

BORING NO. IL030P
Station 97+55.72
Offset 58.20ft
Ground Surface Elev. 674.48 ft

Surface Water Elev. ______ ft
Stream Bed Elev. ______ ft

Groundwater Elev.:
First Encounter 648.5 ft ▼
Upon Completion ______ ft
After ______ **Hrs.** ______ ft

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Sandy Lean Clay, Trace Gravel (CL) Light brown/gray, wet, stiff, fine to medium rounded-subrounded gravel possibly till *(continued)* Gray, unweathered glacial till		4		
		6	2.0	
		10	P	
		11		
	-45	4		
		7	2.0	
		10	P	
		11		
	-50	6		
		7	2.5	
		11	P	
		12		
	-55	4		
		7	2.0	
		11	P	
		11		
	-60			

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Sandy Lean Clay, Trace Gravel (CL) Light brown/gray, wet, stiff, fine to medium rounded-subrounded gravel possibly till *(continued)*		4		
		7	2.0	
		10	P	
		11		
	-65			
	-70	4		
		6	2.0	
		8	P	
		11		
	-75			
	-80			

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)

Illinois Department of Transportation
Division of Highways
CH2M HILL

SOIL BORING LOG

Page 3 of 3

Date 12/12/05

ROUTE I-74 **DESCRIPTION** New I-74 Bridge Over Mississippi River - Illinois Approach **LOGGED BY** B. Karnik

SECTION I-74 Bridge over Mississippi River **LOCATION** (N=558179.679, E=2460420.083), **SEC.** 32, **TWP.** 18N, **RNG.** 1W, 4th **PM**

COUNTY Rock Island **DRILLING METHOD** HSA, CME 55 **HAMMER TYPE** CME AUTOMATIC

STRUCT. NO.
Station

BORING NO. IL030P
Station 97+55.72
Offset 58.20ft
Ground Surface Elev. 674.48 ft

Surface Water Elev. ft
Stream Bed Elev. ft

Groundwater Elev.:
First Encounter 648.5 ft ▼
Upon Completion ft
After **Hrs.** ft

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Sandy Lean Clay, Trace Gravel (CL) Light brown/gray, wet, stiff, fine to medium rounded-subrounded gravel possibly till *(continued)*		4 8 10 11	2.0 P	
	-85			
	-90	6 8 12 12	2.0 P	
	-95			
	-100			

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
572.48		5 6 9 11	2.5 P	
End of Boring				
	-105			
	-110			
	-115			
	-120			

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)

BBS, from 137 (Rev. 8-99)

Illinois Department of Transportation
Division of Highways
CH2M HILL

SOIL BORING LOG

Page 1 of 5

Date 3/26/08

ROUTE I-74 **DESCRIPTION** North Abutment at I-74 NB over 19th SB **LOGGED BY** B. Karnik

SECTION I-74 Bridge over Mississippi River **LOCATION** (N=558653.8025, E=2460400.4911), **SEC.** 32, **TWP.** 18N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** HSA, CME 55 **HAMMER TYPE** CME AUTOMATIC

STRUCT. NO. ____
Station ____

BORING NO. ILR1901
Station 92+74.15
Offset 71.87ft
Ground Surface Elev. 673.62 ft

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

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

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
9" thick asphalt concrete in shoulder followed by subbase 672.82				
Silty Clay, red brown to light grayish brown, moist, friable, possible loess		6		
		4	1.0	
		6	P	
		6		
Gray brown, low plasticity		2		
	-5	3	<0.5	
		5	P	
		6		
Brown with very fine sand throughout, wet, low plasticity, loess		2		
		3		23.0
		3		
		5		
Grayish brown, moist				
			2.5	
			P	
	-10			
Dark brown, trace gravel, moist, low plasticity, possible paleosol or old topsoil		2		
		3	3.3	25.0
	-15	4	P	
655.12				
		2		
		3	1.5	18.0
	-20	5	P	

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Sandy Lean Clay Trace Gravel, dark gray, very sandy, very fine gravel embedded throughout, moist, possible gumbotil, clayey part is medium plasticity *(continued)*			4.0	
			P	
Glacial till, unweathered				
Gray, unweathered till, dry		8		
		11	4.5	
		13	P	
	-25			
		5		
		15	6.5	11.0
	-30	15	P	
	-35			
635.12				
No Recovery, possibly pounded on piece of cobble		18		
		20		
	-40	23		

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)

BBS, from 137 (Rev. 8-99)

SOIL BORING LOG

Date 3/26/08

ROUTE I-74 **DESCRIPTION** North Abutment at I-74 NB over 19th SB **LOGGED BY** B. Karnik

SECTION I-74 Bridge over Mississippi River **LOCATION** (N=558653.8025, E=2460400.4911), **SEC.** 32, **TWP.** 18N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** HSA, CME 55 **HAMMER TYPE** CME AUTOMATIC

STRUCT. NO.
Station

BORING NO. ILR1901
Station 92+74.15
Offset 71.87ft
Ground Surface Elev. 673.62 **ft**

Surface Water Elev. **ft**
Stream Bed Elev. **ft**

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

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
No Recovery, possibly pounded on piece of cobble *(continued)*				
No Recovery		12		
		14		
	-45	20		
625.12				
Sandy Lean Clay Trace Gravel, unweathered till		10		
		10	3.3	13.0
	-50	18	P	
	-55			
		4		
		7	3.3	
	-60	12	P	

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Sandy Lean Clay Trace Gravel, unweathered till *(continued)*				
	-65			
		9		
		10	3.5	
	-70	12	P	
	-75			
			3.0	
	-80		P	

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)

Illinois Department of Transportation
Division of Highways
CH2M HILL

SOIL BORING LOG

Page 3 of 5

Date 3/26/08

ROUTE I-74 **DESCRIPTION** North Abutment at I-74 NB over 19th SB **LOGGED BY** B. Karnik

SECTION I-74 Bridge over Mississippi River **LOCATION** (N=558653.8025, E=2460400.4911), **SEC.** 32, **TWP.** 18N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** HSA, CME 55 **HAMMER TYPE** CME AUTOMATIC

STRUCT. NO. ____
Station ____

BORING NO. ILR1901
Station 92+74.15
Offset 71.87ft
Ground Surface Elev. 673.62 **ft**

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

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

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Sandy Lean Clay Trace Gravel, unweathered till *(continued)*				
	-85			
		6		
		8	3.3	
	-90	12	P	
	-95			
575.12				
			2.8	
	-100		P	

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
End of Boring				
	-105			
	-110			
	-115			
	-120			

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)

Illinois Department of Transportation
Division of Highways
CH2M HILL

SOIL BORING LOG

Page 1 **of** 6

Date 3/27/08

ROUTE I-74 **DESCRIPTION** I-74 over 19th Street SB, Pier 2 **LOGGED BY** B. Karnik

SECTION I-74 Bridge over Mississippi River **LOCATION** (N=558336.9838, E=2460355.1544), **SEC.** 32, **TWP.** 18N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** HSA, CME 55 **HAMMER TYPE** CME AUTOMATIC

STRUCT. NO.
Station

BORING NO. ILR2001
Station 95+88.28
Offset 64.63ft
Ground Surface Elev. 656.99 **ft**

Surface Water Elev. **ft**
Stream Bed Elev. **ft**

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

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Silty Clay with Sand, dark brown, moist, with some burnt and brick pieces, fill		4		
		12		
		4		
		5		
		3		
		3		
		4		
		4		
		3		
	-5	3		
		3		
		5		
Miscellaneous fill		3		
		3		
		2		
More sand, fill		3		
		3		
		1		
		3		
	-10	2		
641.99	-15			
Sandy Lean Clay Trace Gravel, gray, unweathered glacial clay		1		
		1		
		2		
	-20			

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Sandy Lean Clay Trace Gravel, gray, unweathered glacial clay *(continued)*				16.0
	-25	4		
		5		15.0
		8		
	-30	4		
		6		
		9		
	-35			
	-40			

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)

Illinois Department of Transportation
Division of Highways
CH2M HILL

SOIL BORING LOG

Page 2 of 6

Date 3/27/08

ROUTE I-74 **DESCRIPTION** I-74 over 19th Street SB, Pier 2 **LOGGED BY** B. Karnik

SECTION I-74 Bridge over Mississippi River **LOCATION** (N=558336.9838, E=2460355.1544), **SEC.** 32, **TWP.** 18N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** HSA, CME 55 **HAMMER TYPE** CME AUTOMATIC

STRUCT. NO.
Station

BORING NO. ILR2001
Station 95+88.28
Offset 64.63ft
Ground Surface Elev. 656.99 **ft**

Surface Water Elev. ______ **ft**
Stream Bed Elev. ______ **ft**

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

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Sandy Lean Clay Trace Gravel, gray, unweathered glacial clay *(continued)* Brown and gray		4 7 7		16.0
	-45			
	-50	4 7 9		
	-55			
	-60			

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Sandy Lean Clay Trace Gravel, gray, unweathered glacial clay *(continued)*		7 7 9		
	-65			
	-70	3 6 7		19.0
	-75			
	-80			

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)

Illinois Department of Transportation
Division of Highways
CH2M HILL

SOIL BORING LOG

Page 3 of 6

Date 3/27/08

ROUTE I-74 **DESCRIPTION** I-74 over 19th Street SB, Pier 2 **LOGGED BY** B. Karnik

SECTION I-74 Bridge over Mississippi River **LOCATION** (N=558336.9838, E=2460355.1544), **SEC.** 32, **TWP.** 18N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** HSA, CME 55 **HAMMER TYPE** CME AUTOMATIC

STRUCT. NO.
Station

BORING NO. ILR2001
Station 95+88.28
Offset 64.63ft
Ground Surface Elev. 656.99 ft

Surface Water Elev. ft
Stream Bed Elev. ft

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

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Sandy Lean Clay Trace Gravel, gray, unweathered glacial clay *(continued)*		6 8 11			Sandy Lean Clay Trace Gravel, gray, unweathered glacial clay *(continued)* 555.49		6 7 11		
					End of Boring				
	-85					-105			
	-90	7 9 9				-110			
	-95					-115			
	-100					-120			

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)

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CH2M HILL

SOIL BORING LOG

Page 1 of 5

Date 3/25/08

ROUTE I-74 **DESCRIPTION** I-74 Over 19th Street NB Pier 1 **LOGGED BY** B. Karnik

SECTION I-74 Bridge over Mississippi River **LOCATION** (N=558518.3241, E=2460449.8461), **SEC.** 32, **TWP.** 18N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** HSA, CME 55 **HAMMER TYPE** CME AUTOMATIC

STRUCT. NO.
Station

BORING NO. ILR2101
Station 94+44.72
Offset 81.45ft
Ground Surface Elev. 654.13 ft

Surface Water Elev. ft
Stream Bed Elev. ft

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

	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Topsoil, dark brown, sandy clay with green roots, followed by sandy clay	653.43		2	1.5	
			2	to	
			4	2.0	
Sandy Clay, brown, moist, soft with trace gravel, possible fill	652.13		4	P	
			4		
Silty Clay, dark grayish with reddish and black streaks, possible remolded loess used as fill, stiff			4	3.0	
			5	P	
	650.13		5		
light brown, very soft, moist, sandy clay with trace gravel and wood matter rolled in, fill, medium plasticity			2		
		-5	2	<0.5	25.0
			2	P	
			3		
light yellowish brown, moist, very soft, medium to high plasticity, with root matter, fill			1		
			1	<0.5	23.0
			2	P	
			2		
Dark brown, moist, soft, medium plasticity, fill				1.0	
				P	
		-10			
	640.63				
Silty Clay with Gravel, dark gray brown, moist, medium plasticity, trace fine, subangular to angular gravel embedded, possible gumbotil			4		
			5	1.5	23.0
		-15	6	to	
				2.0	
				P	
	635.63				
			4		
			8	4.0	
		-20	9	P	

	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Silty Clay with Gravel, dark gray brown, moist, medium plasticity, trace fine, subangular gravel embedded throughout, weathered glacial till *(continued)*					
Possibly pounded on coarse gravel/cobbles, gray till possible			5		
			9		
		-25	10		
	625.63				
Sandy Lean Clay Trace Gravel, gray, moist, very stiff, low plasticity, unweathered, till			5		
			8	4.0	
		-30	9	P	
			5		
			8		15.0
		-35	10		
			6		
			8	4.0	
		-40	11	P	

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)

Illinois Department of Transportation
Division of Highways
CH2M HILL

SOIL BORING LOG

Page 2 of 5

Date 3/25/08

ROUTE I-74 **DESCRIPTION** I-74 Over 19th Street NB Pier 1 **LOGGED BY** B. Karnik

SECTION I-74 Bridge over Mississippi River **LOCATION** (N=558518.3241, E=2460449.8461), **SEC.** 32, **TWP.** 18N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** HSA, CME 55 **HAMMER TYPE** CME AUTOMATIC

STRUCT. NO. ____
Station ____

BORING NO. ILR2101
Station 94+44.72
Offset 81.45ft
Ground Surface Elev. 654.13 ft

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Sandy Lean Clay Trace Gravel, gray, moist, very stiff, low plasticity, unweathered, till *(continued)*				
	-45			
Dry/moist		6		
		10	3.5	
	-50	12	P	
	-55			
Dry		5		
		8	3.5	
	-60	10	P	

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

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

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Sandy Lean Clay Trace Gravel, gray, moist, very stiff, low plasticity, unweathered, till *(continued)*				
	-65			
		5		
		8		
	-70	10		
	-75			
Brown, gray, more sand		8		
		10	3.5	16.0
	-80	13	P	

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)

Illinois Department of Transportation
Division of Highways
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SOIL BORING LOG

Page 3 **of** 5

Date 3/25/08

ROUTE I-74 **DESCRIPTION** I-74 Over 19th Street NB Pier 1 **LOGGED BY** B. Karnik

SECTION I-74 Bridge over Mississippi River **LOCATION** (N=558518.3241, E=2460449.8461), **SEC.** 32, **TWP.** 18N, **RNG.** 1W

COUNTY Rock Island **DRILLING METHOD** HSA, CME 55 **HAMMER TYPE** CME AUTOMATIC

STRUCT. NO.
Station

BORING NO. ILR2101
Station 94+44.72
Offset 81.45ft
Ground Surface Elev. 654.13 **ft**

Surface Water Elev. ______ **ft**
Stream Bed Elev. ______ **ft**

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

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Sandy Lean Clay Trace Gravel, gray, moist, very stiff, low plasticity, unweathered, till *(continued)*				
	-85			
Brown, gray, sandy				
	-90			
	-95			
Gray, unweathered till		7		
		9	3.5	
554.13	-100	14	P	

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
-105			
-110			
-115			
-120			

End of Boring

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)

BBS, from 137 (Rev. 8-99)

Illinois Department of Transportation

Division of Highways
CH2M HILL

SOIL BORING LOG

Page 1 of 2

Date 9/29/07

ROUTE I-74 **DESCRIPTION** New I-74 Bridge Over Mississippi River - Illinois Approach **LOGGED BY** F. Abreu

SECTION I-74 Bridge over Mississippi River **LOCATION** (N=558586.894, E=2460345.056), **SEC.** 32, **TWP.** 18N, **RNG.** 1W, 4th **PM**

COUNTY Rock Island **DRILLING METHOD** HSA, CME 55 **HAMMER TYPE** CME AUTOMATIC

STRUCT. NO.
Station

BORING NO. ILR1226
Station
Offset
Ground Surface Elev. 674.93 ft

Surface Water Elev. ft
Stream Bed Elev. ft

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

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Grass And Brown Topsoil	673.93				
Clayey Silt(CL-ML) Gray with orange brown, dry, non plastic, stiff, trace fine sands, occasional root matter, oxidized, possible compacted fill			4 5 7 8	4.0 P	
Light brown with orange brown stains, dry, very stif, non plastic, trace silt sands and medium to fine sub angular to sub rounded gravel, possible compacted fill, brittle, strong cementation slightly oxidized	668.93	-5	3 10 18 21	4.5 P	
Silt (ML) Light rown with yellowish orange, moist, dry, non plastic, stiff, trace coarse sands, possible fill compacted, fill slightly oxidized	666.93		5 4 6 6	2.0 P	
Silty Clay(CL-ML) Yellowish orange with gray, moist, medium stiff, low to medium plasticity, oxidized, possible native soil, loess		-10	3 4 2 2	2.0 P	
Light gray with orange brown stains, moist, low to medium plasticity, possible native soil				1.5 P	
Oxidized, moist, soft, non plastic at top 4" of sample, remainder low to medium plasticity, possible loess		-15	1 1 1 2	<0.25 P	
Dark brown, dry to moist, non plastic, possible loess	656.93			2.0 P	
Silt (ML) Dark brown, dry to moist, non plastic, medium stiff, brittle, some clay, occasional root matter with	654.93	-20	0 2 3 4	1.3 P	

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
gray streaks, possible native soil, loess					
Lean Clay With Sand(CL) Uniform gray, dry to moist, little coarse to fine sands scattered throughout, possible transition to glacial till	651.93			1.8 P	
Sandy Lean Clay(CL) Uniform gray to greenish gray, dry to moist, low to non plastic, medium stiff to stiff, little to few coarse to fine sands, possible unweathered clacial till	649.93	-25	2 3 4 5	1.2	
Sandy Lean Clay Trace Gravel (CL) Gray with light gray, dry to moist, non plastic, stiff, strong cementation, medium to fine sub angular to sub rounded gravels, few coarse to fine sands, possible glacial till		-30	2 5 7 10	3.5 P	
Uniform gray, stiff, unweathered, glacial till		-35	1 5 6 10	3.0 P	
Uniform gray, unweathered glacial till with occasional sand seams, medium to fine sand seam 1" from top of sample		-40	3 5 6 8	1.4	

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)

Illinois Department of Transportation
Division of Highways
CH2M HILL

SOIL BORING LOG

Page 2 of 2

Date 9/29/07

ROUTE I-74 **DESCRIPTION** New I-74 Bridge Over Mississippi River - Illinois Approach **LOGGED BY** F. Abreu

SECTION I-74 Bridge over Mississippi River **LOCATION** (N=558586.894, E=2460345.056), **SEC.** 32, **TWP.** 18N, **RNG.** 1W, 4th **PM**

COUNTY Rock Island **DRILLING METHOD** HSA, CME 55 **HAMMER TYPE** CME AUTOMATIC

STRUCT. NO. ____________
Station ____________

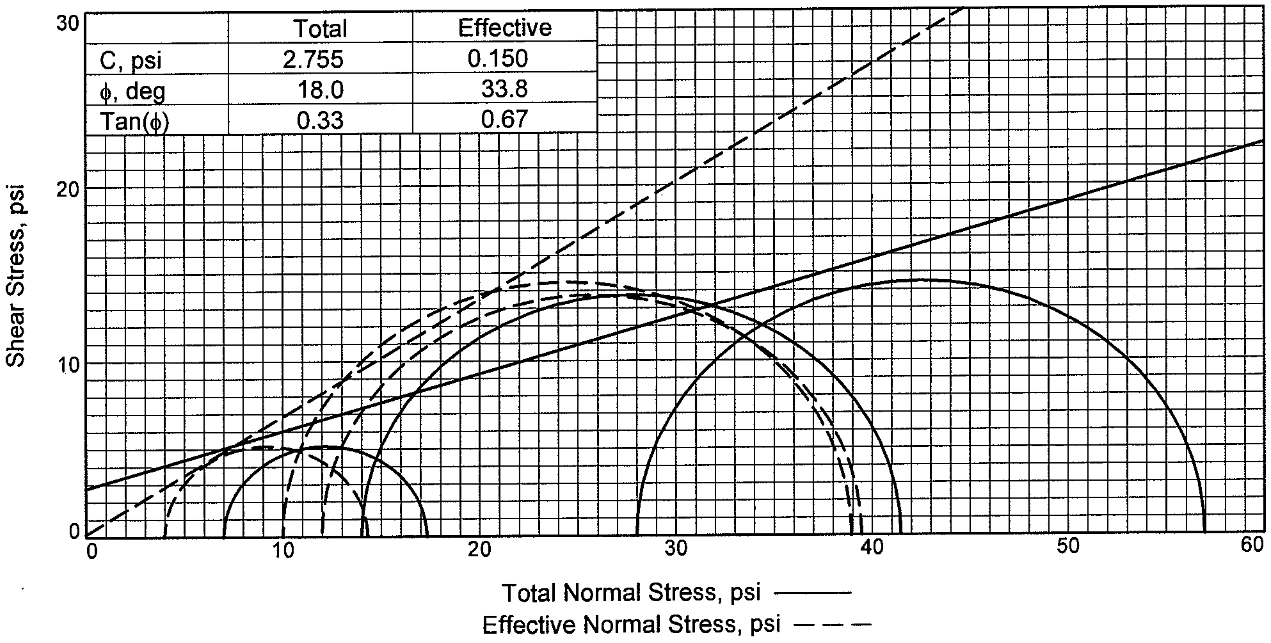
BORING NO. ILR1226
Station ____________
Offset ____________
Ground Surface Elev. 674.93 **ft**

Surface Water Elev. ________ **ft**
Stream Bed Elev. ________ **ft**

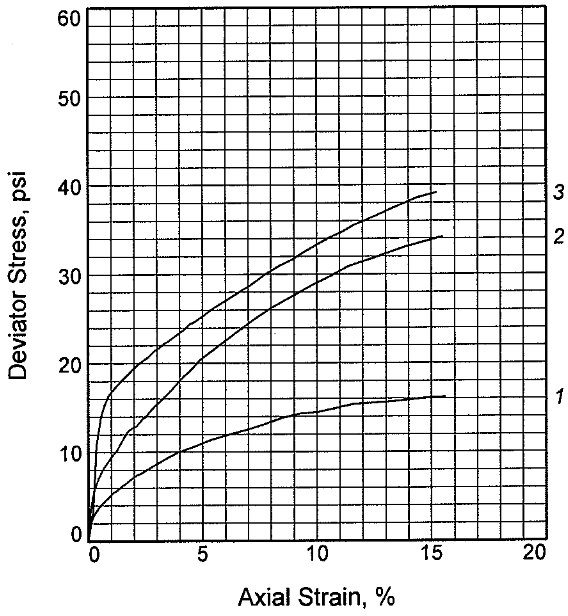
Groundwater Elev.:
First Encounter ________ **ft**
Upon Completion ________ **ft**
After _____ **Hrs.** ________ **ft**

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Sandy Lean Clay Trace Gravel (CL) Gray with light gray, dry to moist, non plastic, stiff, strong cementation, medium to fine sub angular to sub rounded gravels, few coarse to fine sands, possible glacial till *(continued)*					
Uniform gray, stif, dry to moist, unweathered glacial till with scattered sand seams			4		
			6	4.0	
			9	P	
		-45	10		
Uniform olive gray, very stiff, dry to moist, unweathered glacial till			2		
			6	4.0	
			8	P	
BOB 50' bgs at 10:00 A.M.	624.93	-50	10		
End of Boring					
		-55			
		-60			

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)



	Total	Effective
C, psi	2.755	0.150
ϕ, deg	18.0	33.8
Tan(ϕ)	0.33	0.67



	Sample No.	1	2	3
Initial	Water Content,	30.2	25.6	22.1
	Dry Density, pcf	88.4	95.4	99.0
	Saturation,	89.9	90.0	85.2
	Void Ratio	0.9072	0.7676	0.7019
	Diameter, in.	2.840	2.850	2.830
	Height, in.	5.380	5.360	5.580
At Test	Water Content,	31.6	26.2	24.0
	Dry Density, pcf	90.9	98.6	102.3
	Saturation,	100.0	100.0	100.0
	Void Ratio	0.8543	0.7087	0.6474
	Diameter, in.	2.813	2.818	2.799
	Height, in.	5.330	5.300	5.520
	Strain rate, in./min.	0.001	0.001	0.001
	Back Pressure, psi	70.00	70.00	70.00
	Cell Pressure, psi	77.00	84.00	98.00
	Fail. Stress, psi	10.31	27.47	28.95
	Total Pore Pr., psi	73.00	72.00	88.00
	Ult. Stress, psi	16.16	34.16	39.12
	Total Pore Pr., psi	70.00	68.00	83.00
	$\bar{\sigma}_1$ Failure, psi	14.31	39.47	38.95
	$\bar{\sigma}_3$ Failure, psi	4.00	12.00	10.00

Type of Test:
CU with Pore Pressures

Sample Type: ST

Description: DARK BROWN SANDY LEAN CLAY

Specific Gravity= 2.70

Remarks: Lab No. 2139

Figure ________

Client: TERRACON (#07045052)

Project: I-74 CROSSING-BETTENDORF-MOLINE

Source of Sample: ILR1226 **Depth:** 16-18'

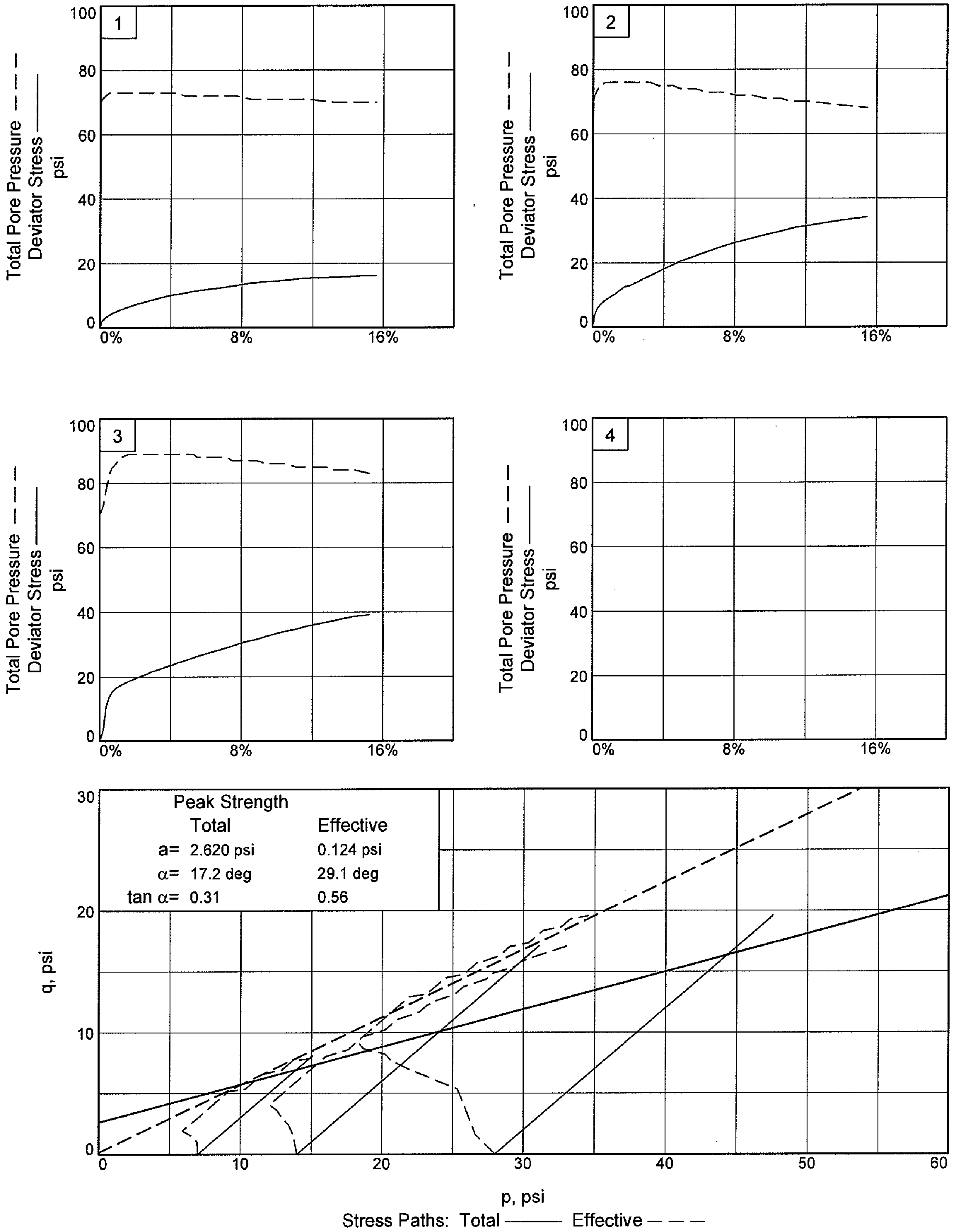
Sample Number: T2

Proj. No.: 19636.040 **Date:** 3-24-08

TRIAXIAL SHEAR TEST REPORT

H. C. NUTTING COMPANY

Tested By: FCE **Checked By:** GS



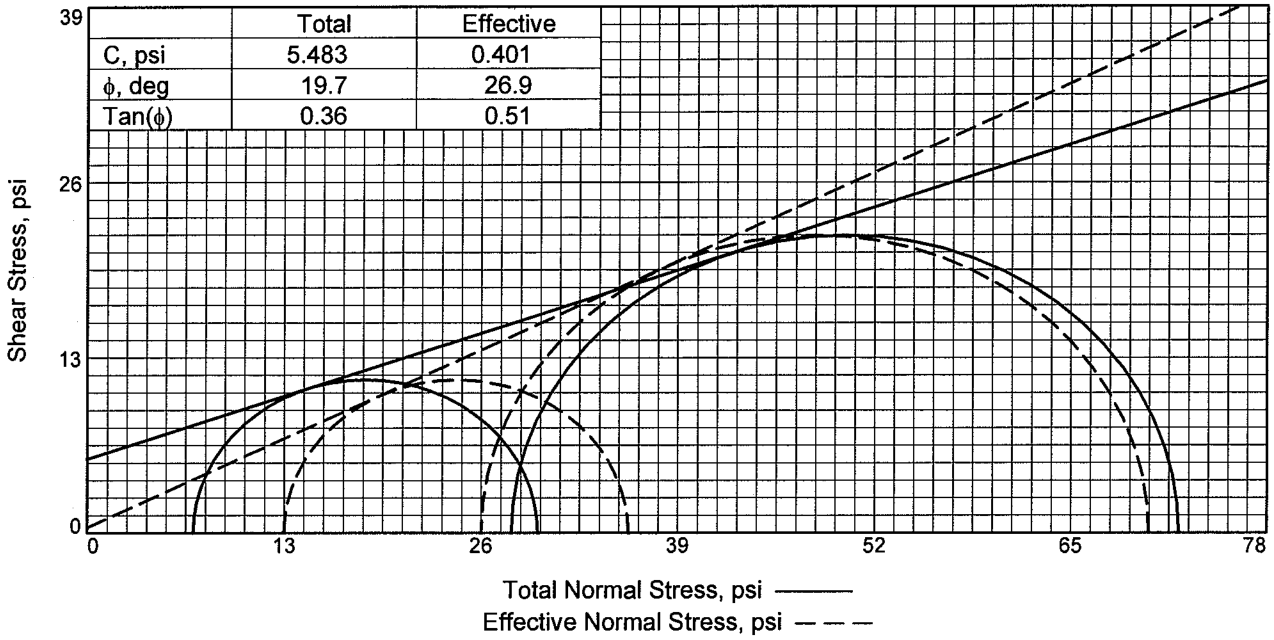
Peak Strength

	Total	Effective
a=	2.620 psi	0.124 psi
α=	17.2 deg	29.1 deg
tan α=	0.31	0.56

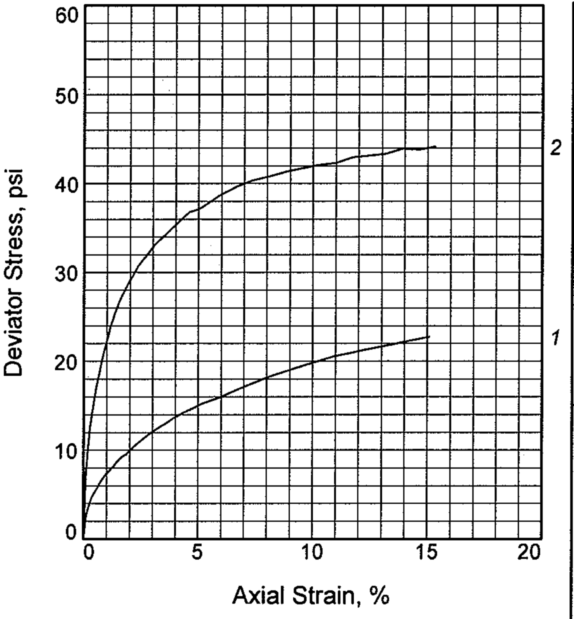
Client: TERRACON (#07045052)
Project: I-74 CROSSING-BETTENDORF-MOLINE
Source of Sample: ILR1226 **Depth:** 16-18' **Sample Number:** T2
Project No.: 19636.040 **Figure** ______

H. C. NUTTING COMPANY

Tested By: FCE **Checked By:** GS



	Total	Effective
C, psi	5.483	0.401
ϕ, deg	19.7	26.9
Tan(ϕ)	0.36	0.51



	Sample No.	1	2
Initial	Water Content,	16.4	14.9
	Dry Density, pcf	117.7	121.0
	Saturation,	99.3	98.6
	Void Ratio	0.4534	0.4137
	Diameter, in.	2.880	2.860
	Height, in.	5.280	5.430
At Test	Water Content,	15.9	14.0
	Dry Density, pcf	119.0	123.7
	Saturation,	100.0	100.0
	Void Ratio	0.4369	0.3826
	Diameter, in.	2.869	2.839
	Height, in.	5.260	5.390
	Strain rate, in./min.	0.000	0.000
	Back Pressure, psi	70.00	70.00
	Cell Pressure, psi	77.00	98.00
	Fail. Stress, psi	22.73	44.15
	Total Pore Pr., psi	64.00	72.00
	Ult. Stress, psi		
	Total Pore Pr., psi		
	$\bar{\sigma}_1$ Failure, psi	35.73	70.15
	$\bar{\sigma}_3$ Failure, psi	13.00	26.00

Type of Test:
CU with Pore Pressures

Sample Type: ST

Description: BROWN GRAY SANDY LEAN CLAY W/GRAVEL

LL= 32 **PL=** 14 **PI=** 18

Specific Gravity= 2.74

Remarks: Lab No.3315

Figure ________

Client: TERRACON

Project: I-74 EXPANSION CENTRAL

Source of Sample: ILR2001 **Depth:** 18'

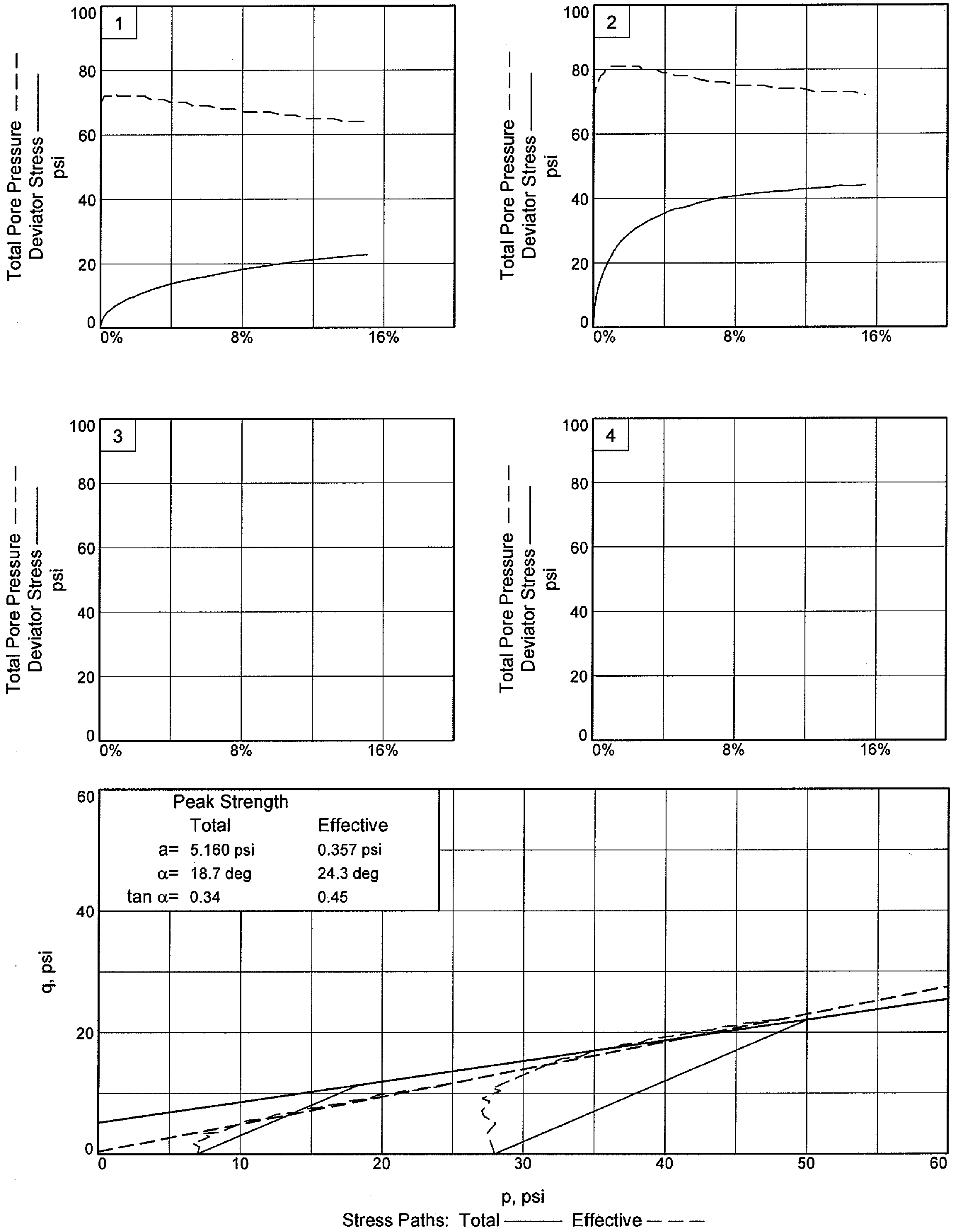
Sample Number: T-1

Proj. No.: 07045052 **Date:** 4-22-08

TRIAXIAL SHEAR TEST REPORT

H. C. NUTTING COMPANY

Tested By: FCE **Checked By:** GS



Peak Strength

	Total	Effective
a=	5.160 psi	0.357 psi
α=	18.7 deg	24.3 deg
tan α=	0.34	0.45

Client: TERRACON

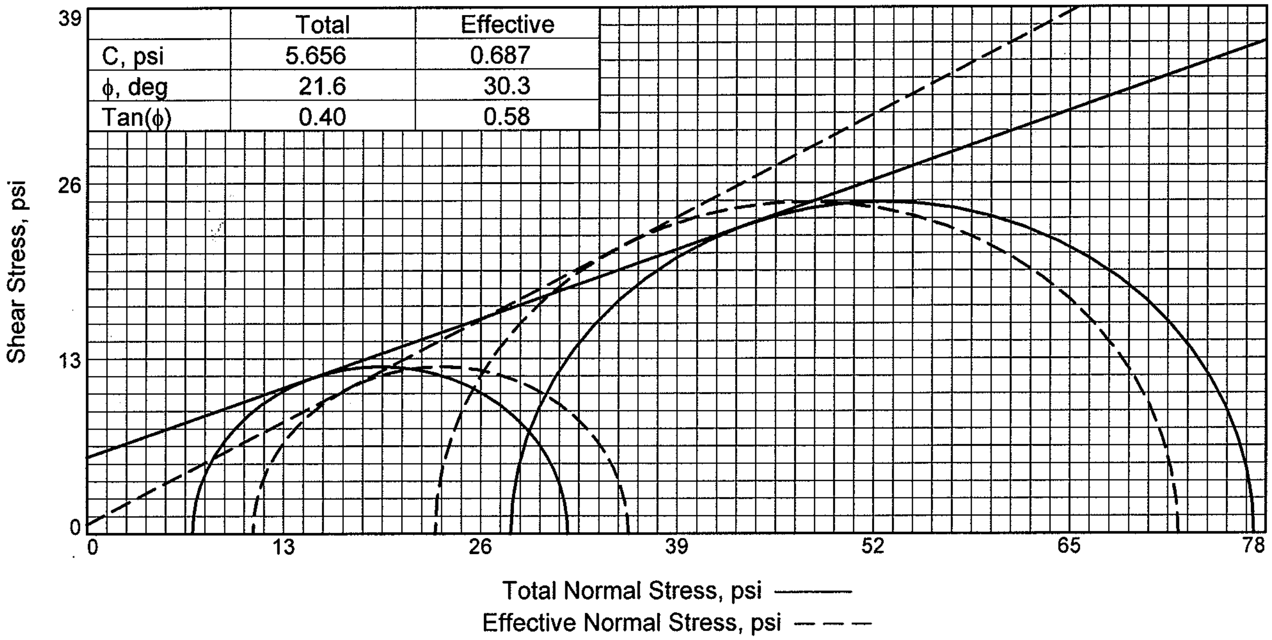
Project: I-74 EXPANSION CENTRAL

Source of Sample: ILR2001 **Depth:** 18' **Sample Number:** T-1

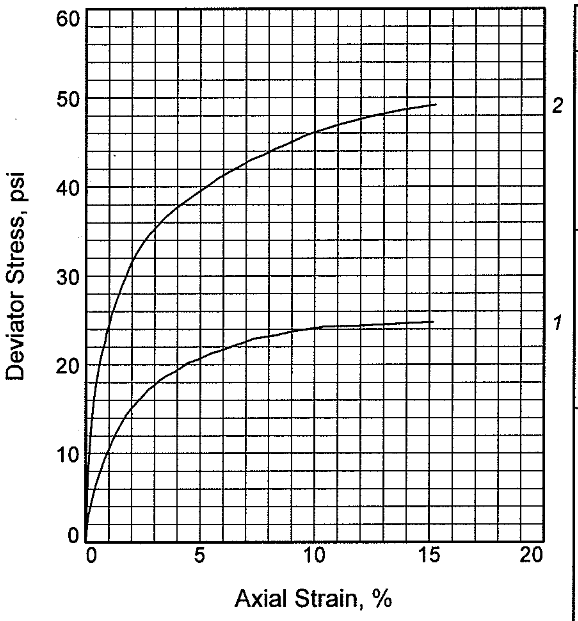
Project No.: 07045052 **Figure** ________

H. C. NUTTING COMPANY

Tested By: FCE **Checked By:** GS



	Total	Effective
C, psi	5.656	0.687
ϕ, deg	21.6	30.3
Tan(ϕ)	0.40	0.58



	Sample No.	1	2
Initial	Water Content,	23.0	23.9
	Dry Density, pcf	102.5	101.7
	Saturation,	96.5	98.1
	Void Ratio	0.6440	0.6575
	Diameter, in.	2.830	2.840
	Height, in.	5.520	5.180
At Test	Water Content,	23.5	22.9
	Dry Density, pcf	103.1	104.1
	Saturation,	100.0	100.0
	Void Ratio	0.6350	0.6193
	Diameter, in.	2.825	2.818
	Height, in.	5.510	5.140
	Strain rate, in./min.	0.002	0.002
	Back Pressure, psi	70.00	70.00
	Cell Pressure, psi	77.00	98.00
	Fail. Stress, psi	24.77	49.19
	Total Pore Pr., psi	66.00	75.00
	Ult. Stress, psi		
	Total Pore Pr., psi		
	$\bar{\sigma}_1$ Failure, psi	35.77	72.19
	$\bar{\sigma}_3$ Failure, psi	11.00	23.00

Type of Test:
CU with Pore Pressures

Sample Type: ST

Description: BROWN GRAY SILTY LEAN CLAY

Specific Gravity= 2.70

Remarks: Lab No. 3314

Figure ________

Client: TERRACON

Project: I-74 EXPANSION CENTRAL

Source of Sample: ILR1901 **Depth:** 8'

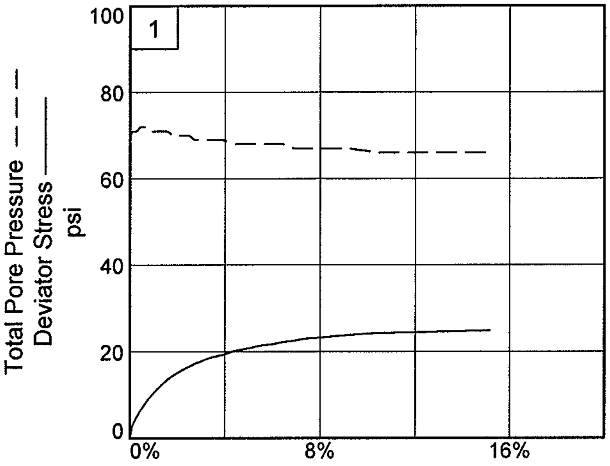
Sample Number: T-1

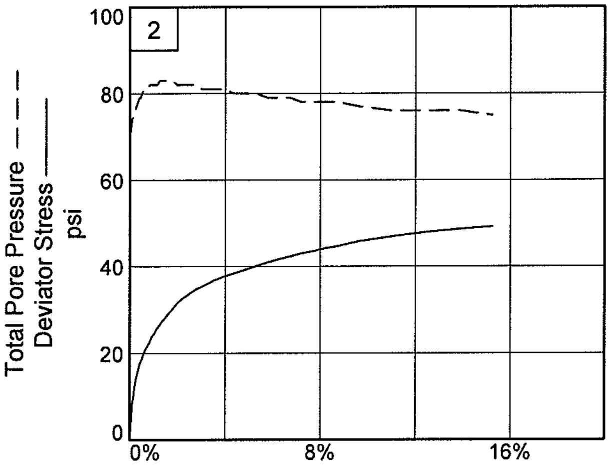
Proj. No.: 07045052 **Date:** 4-14-08

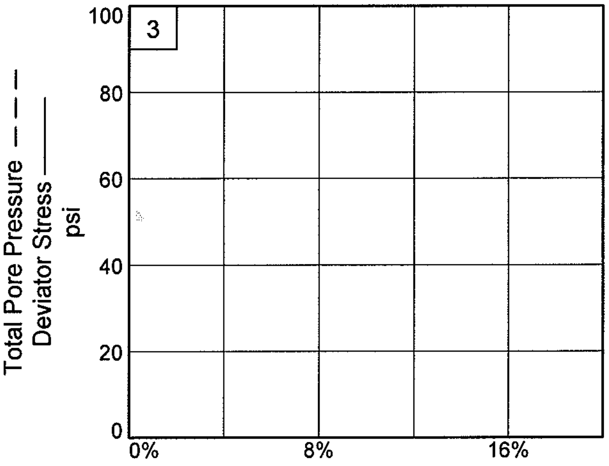
TRIAXIAL SHEAR TEST REPORT

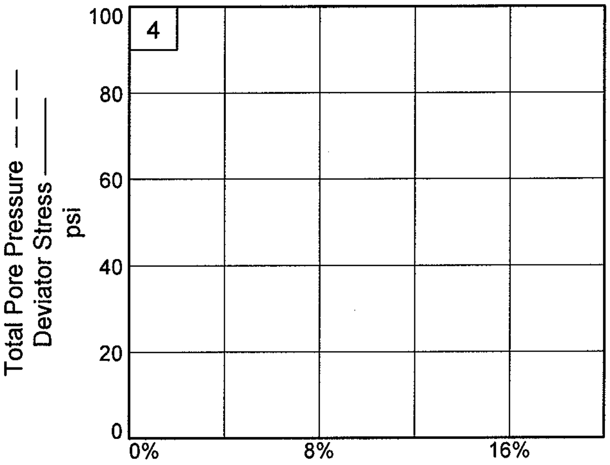
H. C. NUTTING COMPANY

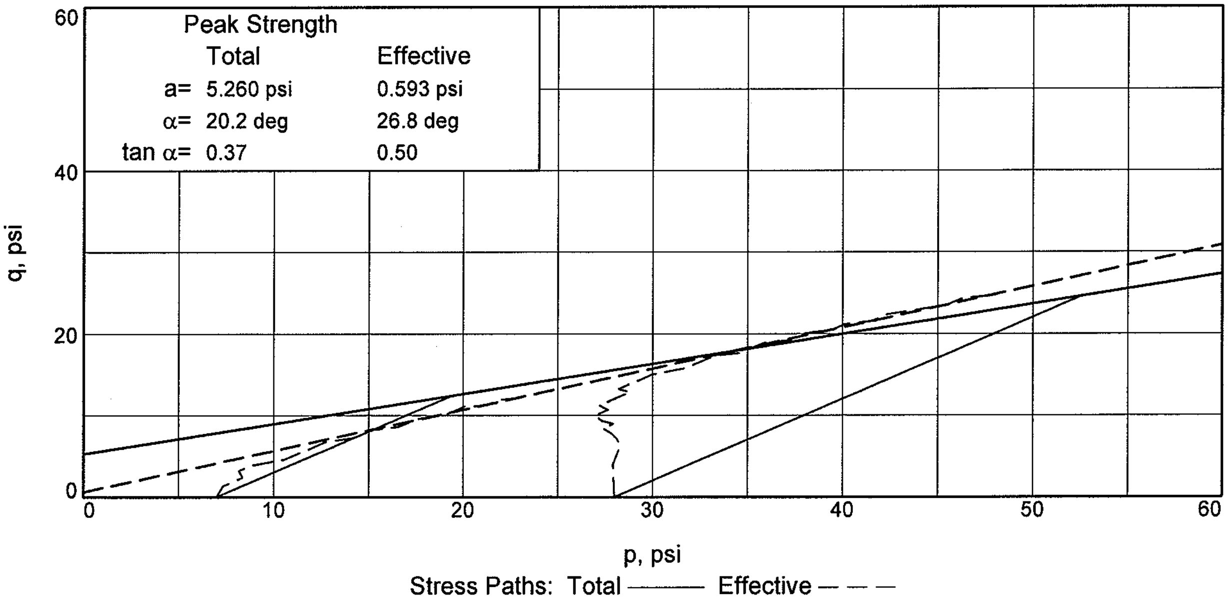
Tested By: FCE **Checked By:** GS











Peak Strength

	Total	Effective
a=	5.260 psi	0.593 psi
α=	20.2 deg	26.8 deg
tan α=	0.37	0.50

Client: TERRACON

Project: I-74 EXPANSION CENTRAL

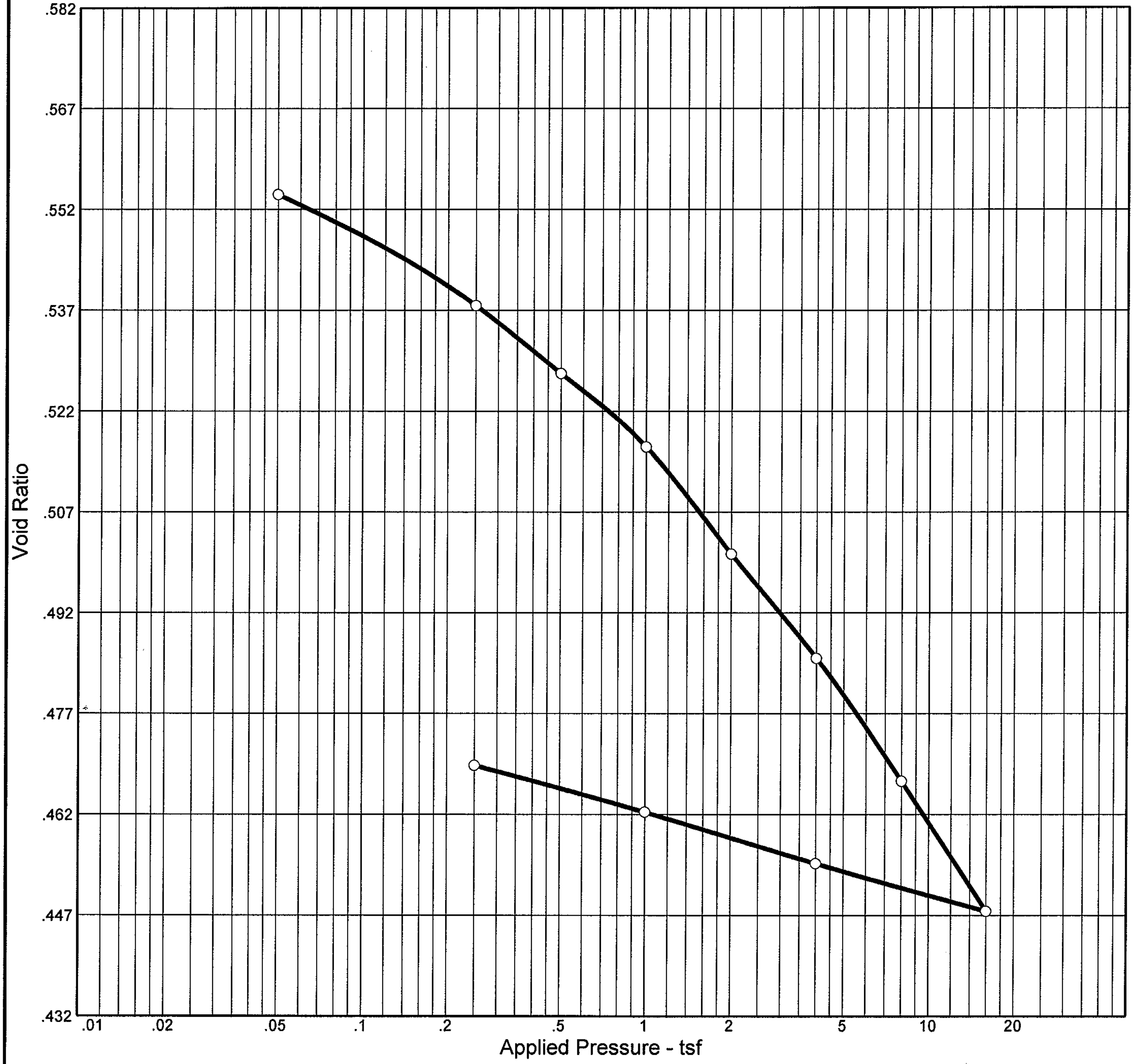
Source of Sample: ILR1901 **Depth:** 8' **Sample Number:** T-1

Project No.: 07045052 **Figure** ________

H. C. NUTTING COMPANY

Tested By: FCE **Checked By:** GS

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Swell Press. (tsf)	Swell %	e_o
Sat.	Moist.											
96.3 %	19.9 %	108.7			2.72		1.58	0.06	0.01			0.562

MATERIAL DESCRIPTION	USCS	AASHTO
LT. BROWN GRAY SILT, MOIST - STIFF		

Project No. 07045052 **Client:** TERRACON

Project: I-74 EXPANSION CENTRAL

Source: ILR2101 **Sample No.:** T-1 **Elev./Depth:** 8'

CONSOLIDATION TEST REPORT

H. C. NUTTING COMPANY

Remarks:

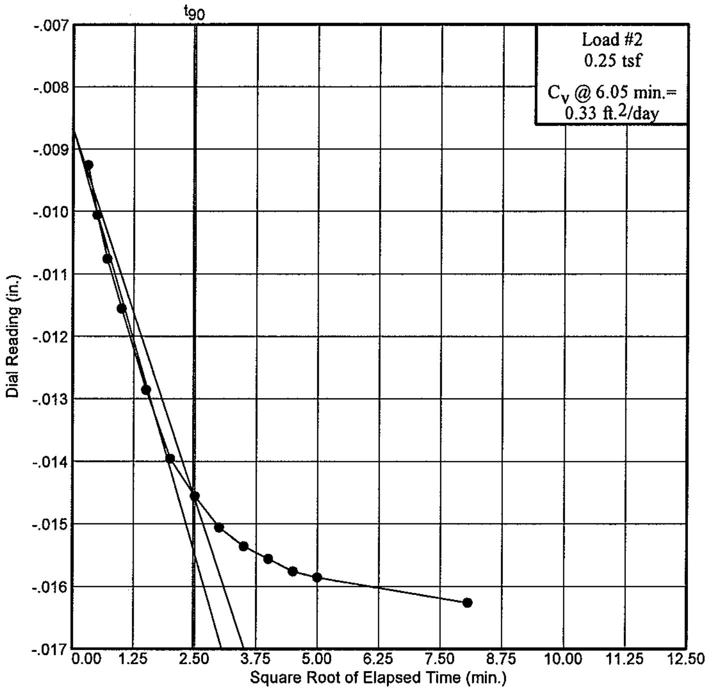
Lab No. 3313

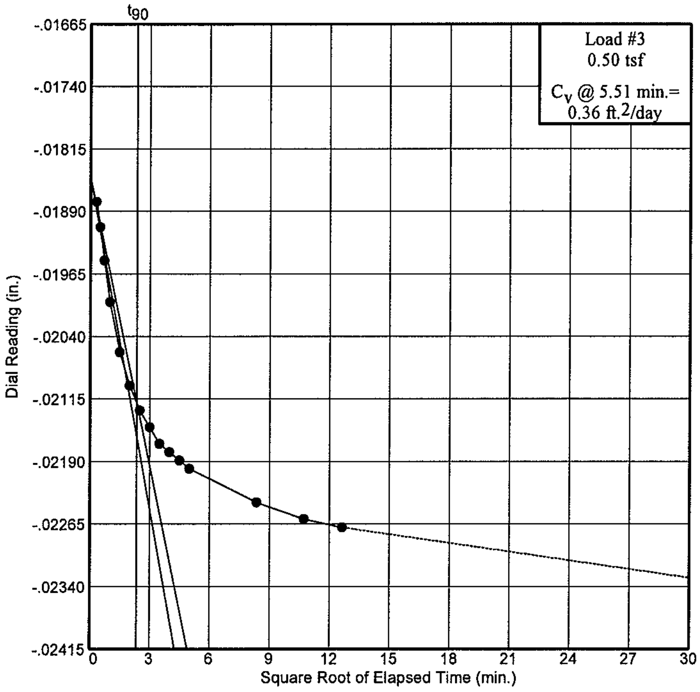
Figure

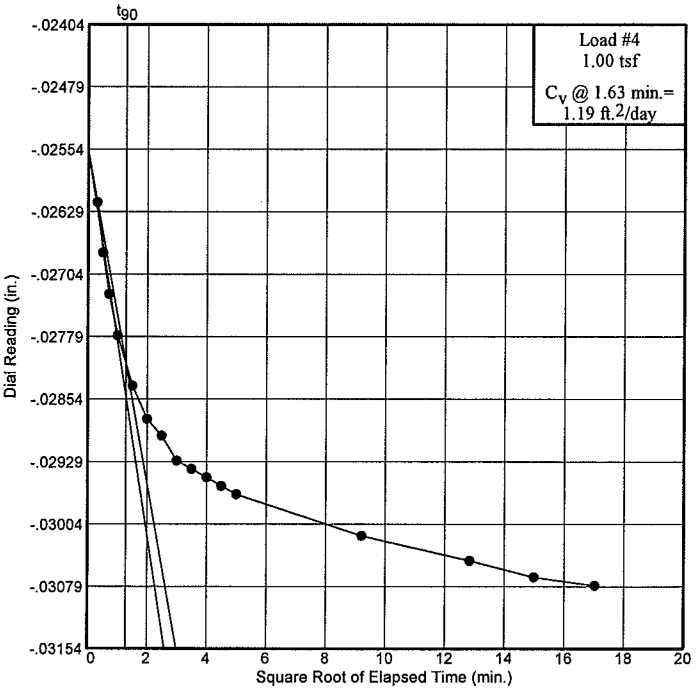
Dial Reading vs. Time

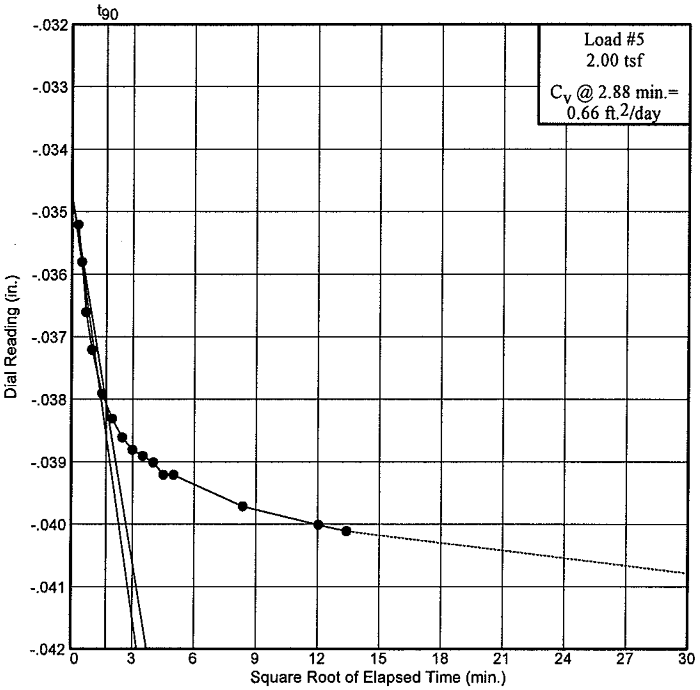
Project No.: 07045052
Project: I-74 EXPANSION CENTRAL

Source: ILR2101 Sample No.: T-1 Elev./Depth: 8'









Dial Reading vs. Time

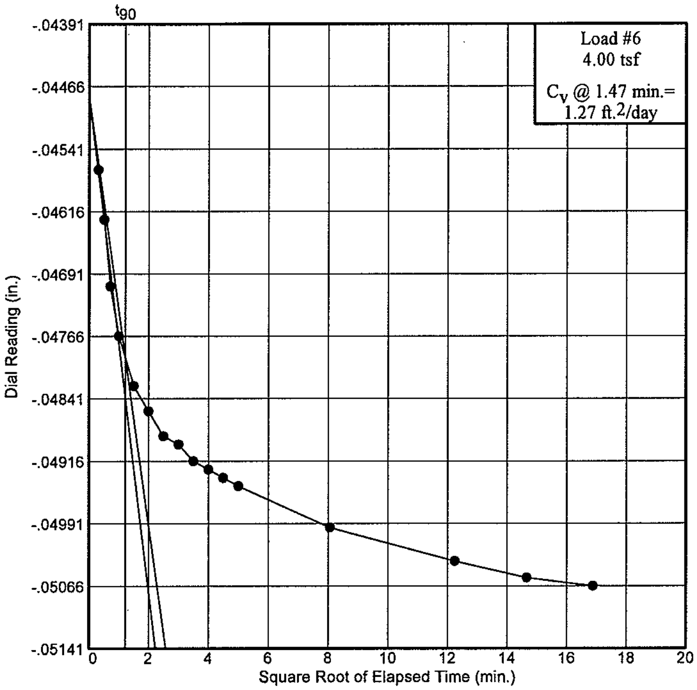
H. C. NUTTING COMPANY

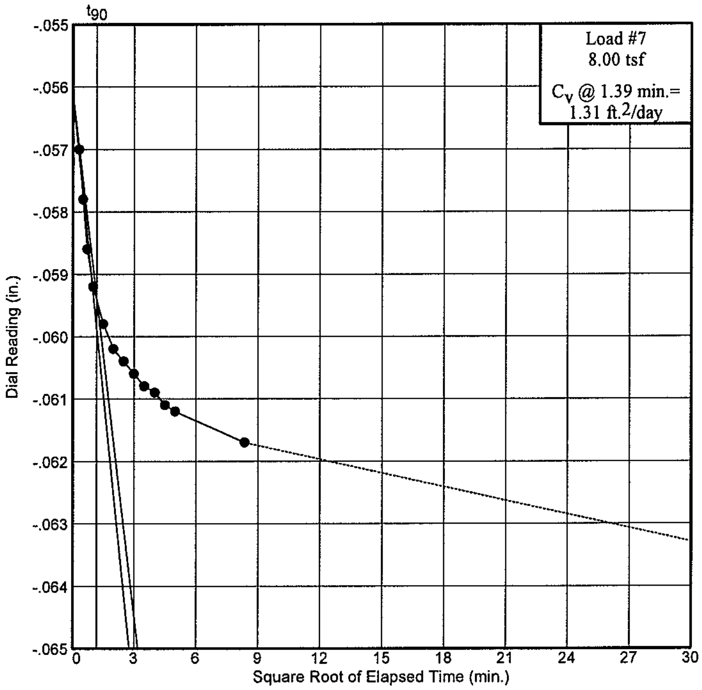
Figure

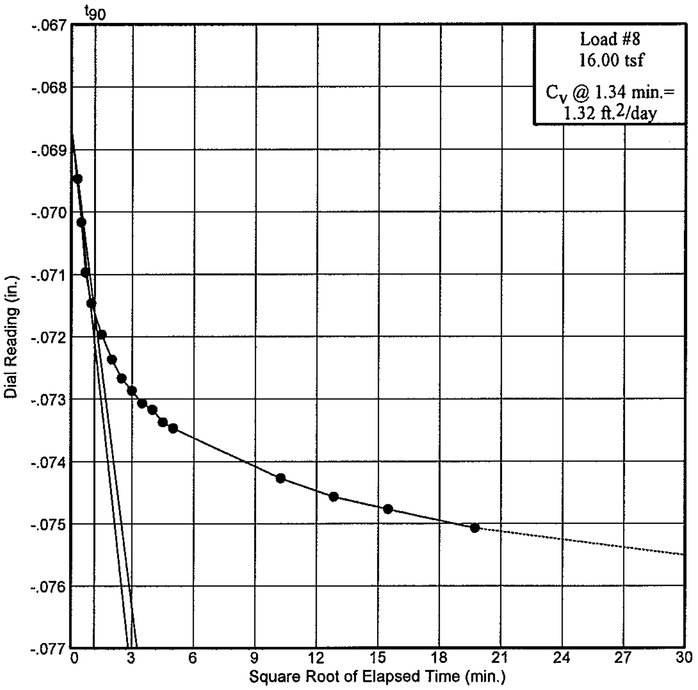
Dial Reading vs. Time

Project No.: 07045052
Project: I-74 EXPANSION CENTRAL

Source: ILR2101 Sample No.: T-1 Elev./Depth: 8'







Dial Reading vs. Time

H. C. NUTTING COMPANY

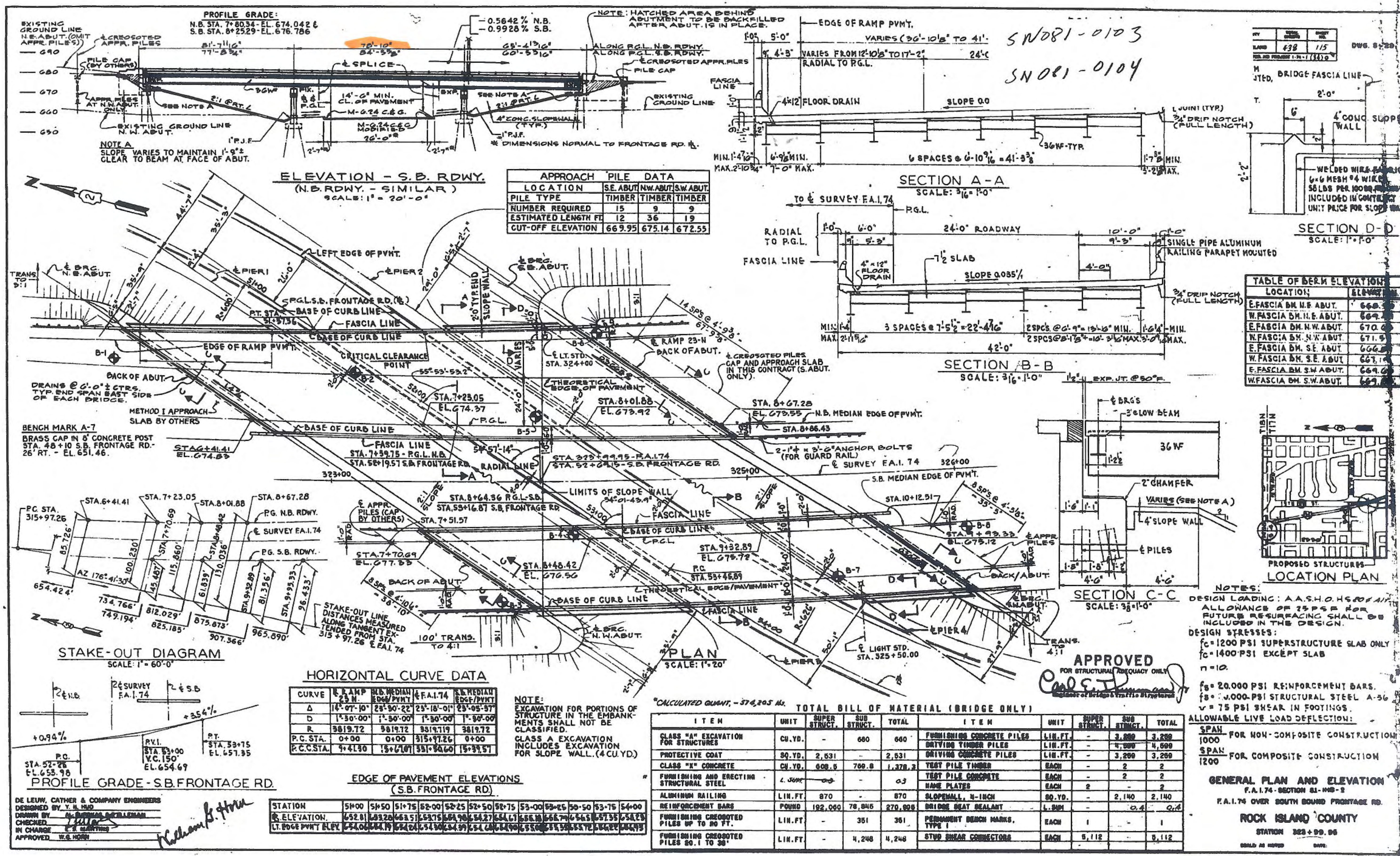
Figure

SN081-0103

SN081-0104

	438	115

DWG. 8-20



APPROACH PILE DATA

LOCATION	S.E. ABUT.	N.W. ABUT.	S.W. ABUT.
PILE TYPE	TIMBER	TIMBER	TIMBER
NUMBER REQUIRED	15	9	9
ESTIMATED LENGTH FT.	12	36	19
CUT-OFF ELEVATION	669.95	675.14	672.55

TABLE OF BERM ELEVATIONS

LOCATION	ELEVATION
E. FASCIA BM. N.E. ABUT.	668.[illegible]
W. FASCIA BM. N.E. ABUT.	669.[illegible]
E. FASCIA BM. N.W. ABUT.	670.[illegible]
W. FASCIA BM. N.W. ABUT.	671.[illegible]
E. FASCIA BM. S.E. ABUT.	666.[illegible]
W. FASCIA BM. S.E. ABUT.	667.1[illegible]
E. FASCIA BM. S.W. ABUT.	669.6[illegible]
W. FASCIA BM. S.W. ABUT.	669.[illegible]

HORIZONTAL CURVE DATA

CURVE	℄ RAMP 23 N.	N.B. MEDIAN EDGE/PVM'T	℄ F.A.I. 74	S.B. MEDIAN EDGE/PVM'T
Δ	14°-07'-10"	23°-30'-22"	23°-18'-01"	23°-05'-57"
D	1°-30'-00"	1°-30'-00"	1°-30'-00"	1°-30'-00"
R	3819.72	3819.72	3819.719	3819.72
P.C. STA.	0+00	0+00	315+97.26	0+00
P.C.C. STA.	9+41.30	15+67.07	331+50.60	15+39.57

NOTE:
EXCAVATION FOR PORTIONS OF STRUCTURE IN THE EMBANKMENTS SHALL NOT BE CLASSIFIED.
CLASS A EXCAVATION INCLUDES EXCAVATION FOR SLOPE WALL. (4 CU. YD.)

EDGE OF PAVEMENT ELEVATIONS
(S.B. FRONTAGE RD.)

STATION	51+00	51+50	51+75	52+00	52+25	52+50	52+75	53+00	53+25	53+50	53+75	54+00
℄ ELEVATION.	652.81	653.28	653.51	653.75	653.98	654.27	654.67	655.1[illegible]	655.7[illegible]	656.5	657.35	658.25
LT. EDGE PVM'T ELEV.	654.06	654.19	654.24	654.30	654.39	654.6[illegible]	654.90	655.0[illegible]	655.3[illegible]	655.72	656.22	656.93

"CALCULATED QUANT. - 374,205 Ms.

TOTAL BILL OF MATERIAL (BRIDGE ONLY)

ITEM	UNIT	SUPER STRUCT.	SUB STRUCT.	TOTAL
CLASS "A" EXCAVATION FOR STRUCTURES	CU. YD.	-	660	660
PROTECTIVE COAT	SQ. YD.	2,531	-	2,531
CLASS "X" CONCRETE	CU. YD.	608.5	769.8	1,378.3
" FURNISHING AND ERECTING STRUCTURAL STEEL	L. SUM	~~0.3~~	-	0.3
ALUMINUM RAILING	LIN. FT.	870	-	870
REINFORCEMENT BARS	POUND	192,060	78,846	270,906
FURNISHING CREOSOTED PILES UP TO 20 FT.	LIN. FT.	-	361	361
FURNISHING CREOSOTED PILES 20.1 TO 30'	LIN. FT.	-	4,248	4,248
FURNISHING CONCRETE PILES	LIN. FT.	-	3,269	3,269
DRIVING TIMBER PILES	LIN. FT.	-	~~4,609~~	4,609
DRIVING CONCRETE PILES	LIN. FT.	-	3,269	3,269
TEST PILE TIMBER	EACH	-	2	2
TEST PILE CONCRETE	EACH	-	2	2
NAME PLATES	EACH	2	-	2
SLOPEWALL, 4-INCH	SQ. YD.	-	2,140	2,140
BRIDGE SEAT SEALANT	L. SUM	-	0.4	0.4
PERMANENT BENCH MARKS, TYPE I	EACH	1	-	1
STUD SHEAR CONNECTORS	EACH	5,112	-	5,112

APPROVED
FOR STRUCTURAL ADEQUACY ONLY
Carl E. Thunman Jr
Engineer of Bridge & Traffic Structures

NOTES:
DESIGN LOADING: A.A.S.H.O. HS20-44 ALT.
ALLOWANCE OF 25 PSF FOR FUTURE RESURFACING SHALL BE INCLUDED IN THE DESIGN.
DESIGN STRESSES:
fc = 1200 PSI SUPERSTRUCTURE SLAB ONLY
fc = 1400 PSI EXCEPT SLAB
n = 10.
fs = 20,000 PSI REINFORCEMENT BARS.
fs = [illegible],000 PSI STRUCTURAL STEEL A-36
v = 75 PSI SHEAR IN FOOTINGS.
ALLOWABLE LIVE LOAD DEFLECTION:
$\frac{\text{SPAN}}{1000}$ FOR NON-COMPOSITE CONSTRUCTION
$\frac{\text{SPAN}}{1200}$ FOR COMPOSITE CONSTRUCTION

GENERAL PLAN AND ELEVATION

F.A.I. 74 - SECTION 81-1HB-2
F.A.I. 74 OVER SOUTH BOUND FRONTAGE RD.

ROCK ISLAND COUNTY

STATION 323+99.96

SCALE: AS NOTED   DATE:

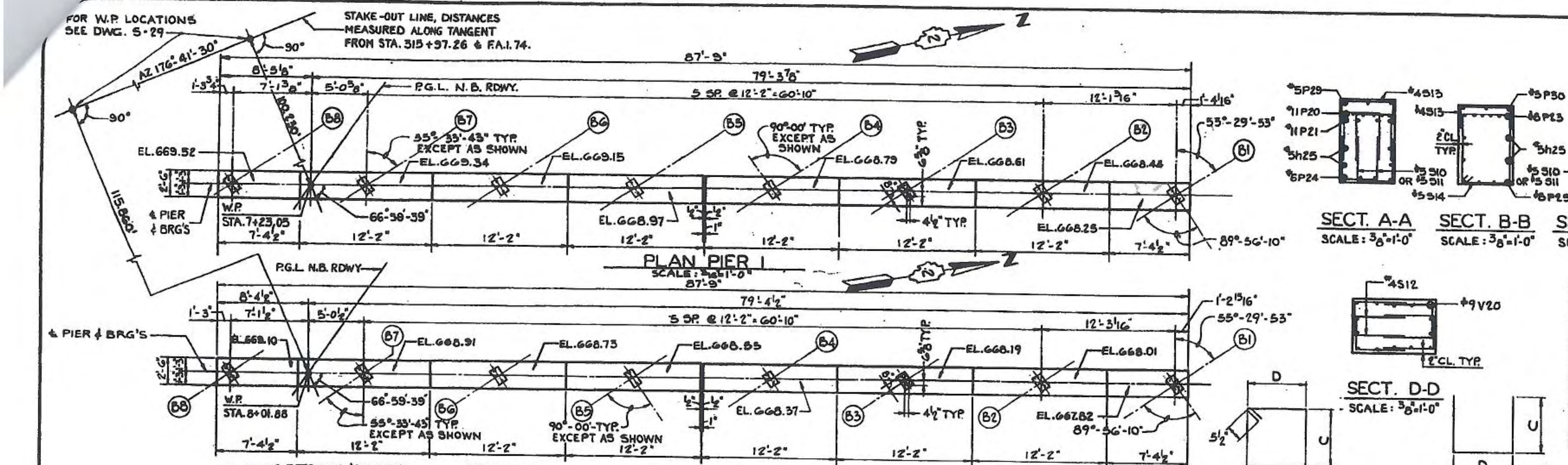
DE LEUW, CATHER & COMPANY ENGINEERS
DESIGNED BY Y. H. HUO
DRAWN BY [illegible]
CHECKED [illegible]
IN CHARGE E. S. MARTINS
APPROVED W.G. HORN

William G. Horn

Rev. 11-00-69 w.L.

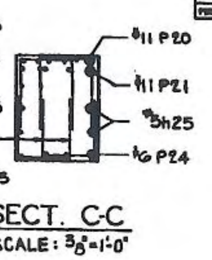
ROUTE NO.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
F.A.I. 74	81-IHB-2	ROCK ISLAND	438	122
FED. ROAD DIST. NO. 7	ILLINOIS	FED. AID PROJECT I-74		

DWG. S-36



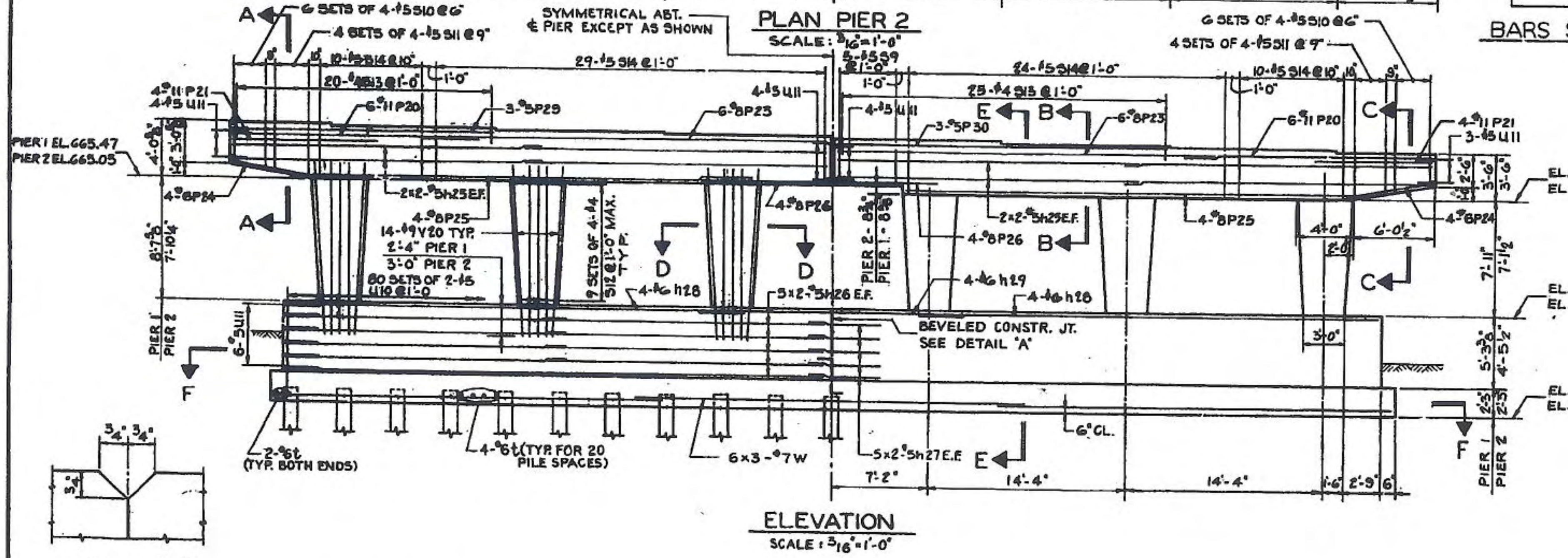
PLAN PIER 1
SCALE: 3/16"=1'-0"

PLAN PIER 2
SCALE: 3/16"=1'-0"

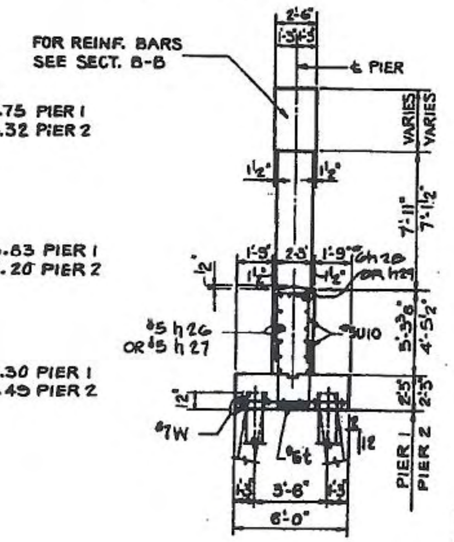


SECT. C-C
SCALE: 3/8"=1'-0"

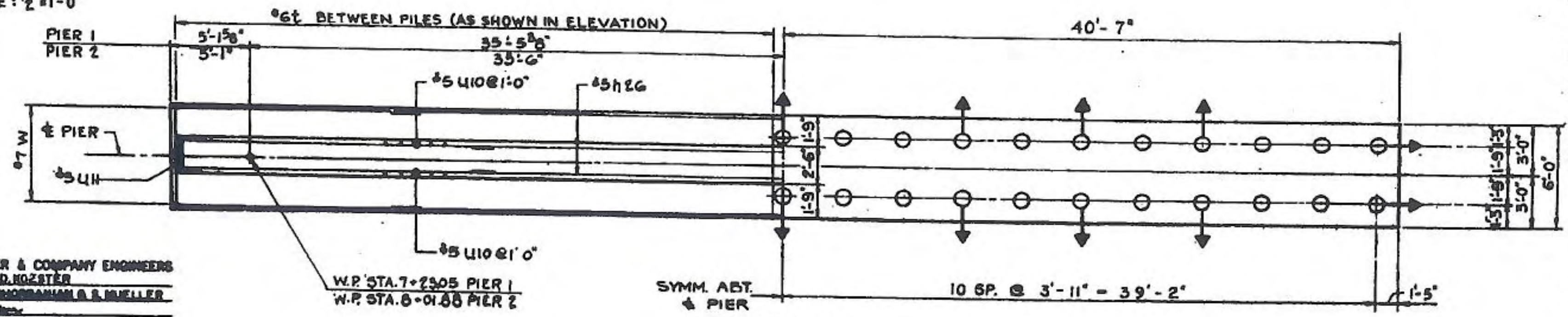
BAR	C	D
S9	2'-6"	2'-2"
S10	2'-0"	1'-6"
S11	2'-3"	1'-6"
S12	2'-4"	1'-4"
S13	1'-8"	2'-2"
S14	3'-2"	2'-2"
U10	4'-1"	2'-2"
U11	1'-3"	2'-2"



ELEVATION
SCALE: 3/16"=1'-0"



SECTION E-E
SCALE: 3/16"=1'-0"



SECTION F-F
SCALE: 3/16"=1'-0"

BAR LIST

BAR	NO. PIER 1	NO. PIER 2	TOTAL	SIZE	LENGTH	SHAPE
h25	16	16	32	#5	22'-6"	—
h26	20	20	40	#5	20'-6"	—
h27	20	20	40	#5	21'-3"	—
h28	8	8	16	#6	33'-6"	—
h29	4	4	8	#6	15'-10"	—
P20	12	12	24	#11	18'-0"	—
P21	8	8	16	#11	12'-3"	—
P23	12	12	24	#8	27'-6"	—
P24	8	8	16	#6	6'-0"	—
P25	8	8	16	#8	32'-6"	—
P26	8	8	16	#8	9'-3"	—
P29	3	3	6	#5	19'-0"	—
P30	3	3	6	#5	24'-0"	—
S9	5	5	10	#5	10'-3"	☐
S10	48	48	96	#5	5'-6"	☐
S11	32	32	64	#5	6'-0"	☐
S12	216	216	432	#4	6'-0"	⊔
S13	45	45	90	#4	5'-6"	⊔
S14	73	73	146	#8	11'-7"	☐
t	84	84	168	#6	5'-6"	—
U10	160	160	320	#5	10'-4"	⊔
U11	19	19	38	#5	4'-8"	⊔
V20	84	84	168	#9	18'-3"	—
W	18	18	36	#7	28'-0"	—

BILL OF MATERIAL

ITEM	UNIT	QUANTITY PIER 1	QUANTITY PIER 2
CLASS A EXCAVATION FOR STRUCT.	CU. YD.	136	119
CLASS X CONCRETE	CU. YD.	122.2	114.7
REINFORCEMENT BARS	LBS.	15,160	15,160
FURNISHING CREOSOTED PILES 20.1 TO 38 FT.	LIN. FT.	861	1386
DRIVING TIMBER PILES	LIN. FT.	861	1386
TEST PILES	EACH	1	—

NOTES:

ANCHOR BOLTS TO PROJECT 3½" ABOVE CONCRETE ON PIER 1 AND 4" ABOVE CONCRETE ON PIER 2.
SPACE REINFORCEMENT IN CAP TO MISS ANCHOR BOLTS.
ALL EDGES TO HAVE STD. ¾" CHAMFER EXCEPT AS NOTED.
POUR STEPS MONOLITHICALLY WITH PIER CAP.
ALL BAR DIMENSIONS ARE OUT TO OUT.
MINIMUM BAR LAP = 24 DIA.
KEY TO BAR INDICATION: 5×2-#6 ETC. INDICATES.
5 LINES OF #6 BARS WITH 2 LENGTHS PER LINE.

PILE DATA

LOCATION	PIER 1	PIER 2
PILE TYPE	TIMBER	TIMBER
MINIMUM CAPACITY TONS	24	24
NUMBER REQUIRED	42 #	42
ESTIMATED LENGTH FT.	21.	33
CUT OFF ELEVATION	650.30	651.49

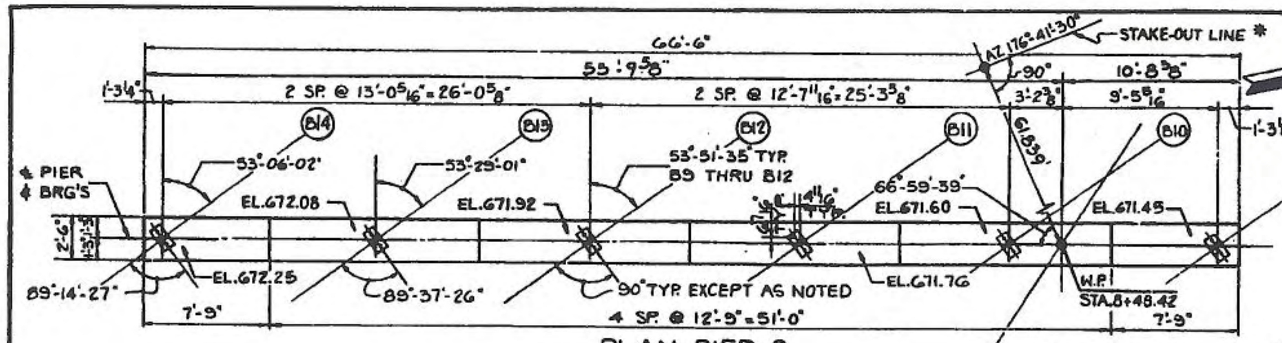
INCLUDES 1 TEST PILE

DE LEUW, CATHER & COMPANY ENGINEERS
DESIGNED BY R.D. KOZSTER
DRAWN BY [illegible] & S. MUELLER
CHECKED [illegible]
IN CHARGE E. S. MARTINS
APPROVED W.S. HORN

PIERS 1 & 2
F.A.I. 74 - SECTION 81-IHB-2
F.A.I. 74 OVER SOUTH BOUND FRONTAGE RD.
ROCK ISLAND COUNTY
STATION 323+90.96
SCALE: AS NOTED DATE:

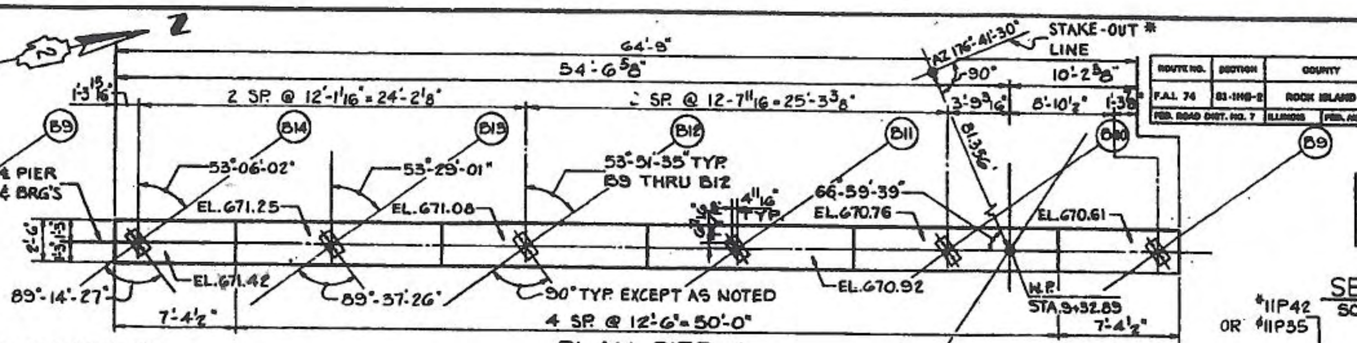
ROUTE NO.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
F.A.I. 74	81-1HB-2	ROCK ISLAND	498	123
FED. ROAD DIST. NO. 7	ILLINOIS	FED. AID PROJECT I-74		

DWG. S-37

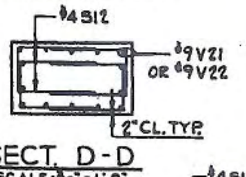


PLAN PIER 3
SCALE: ³⁄₁₆"=1'-0"

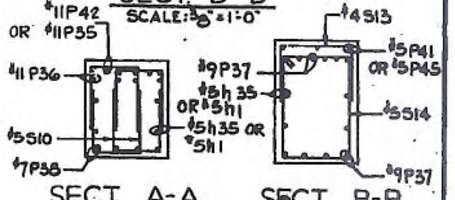
* FOR LOCATION OF W.P. SEE DWG. S-29.



PLAN PIER 4
SCALE: ³⁄₁₆"=1'-0"

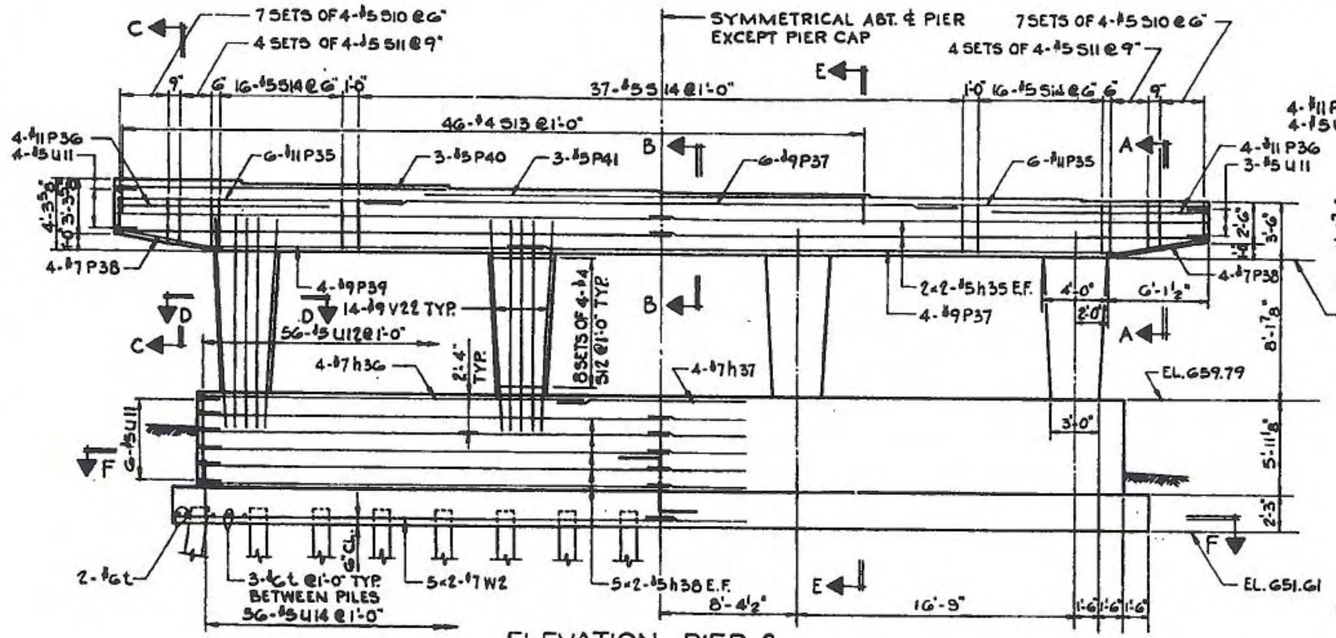


SECT. D-D
SCALE: ⅜"=1'-0"

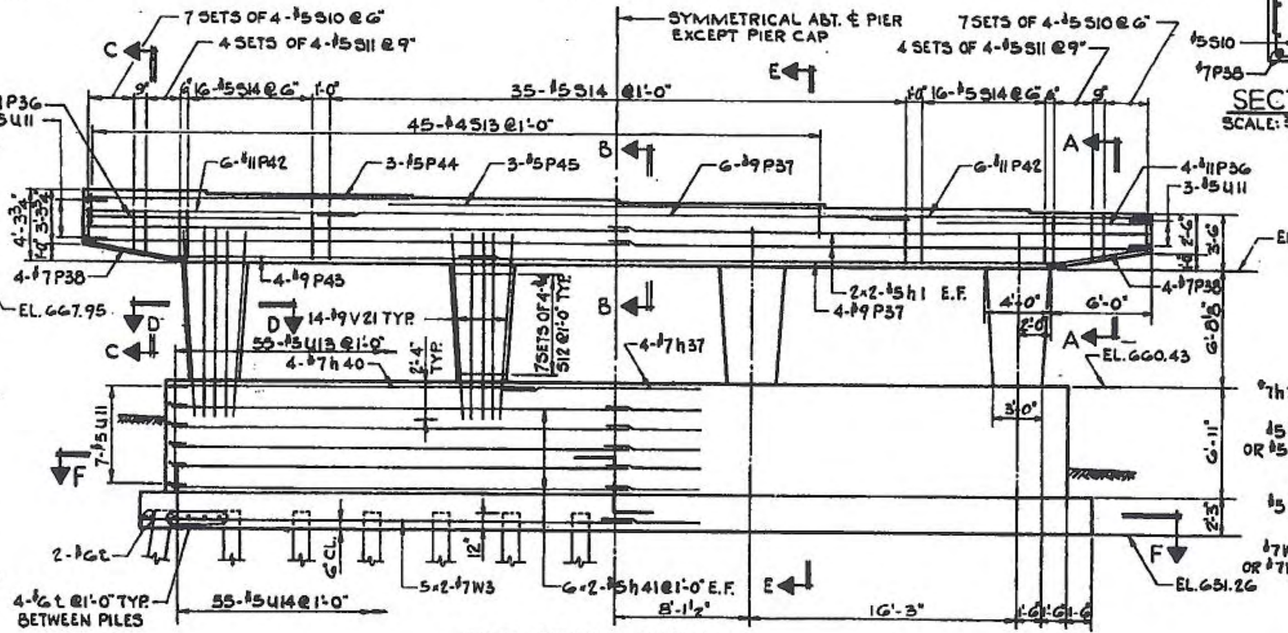


SECT. A-A
SCALE: ⅜"=1'-0"

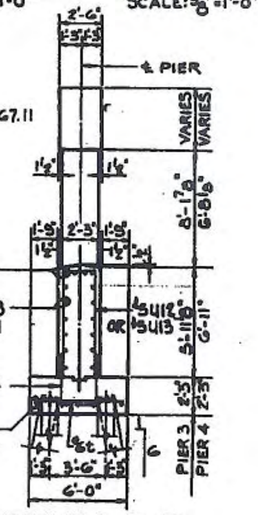
SECT. B-B
SCALE: ⅜"=1'-0"



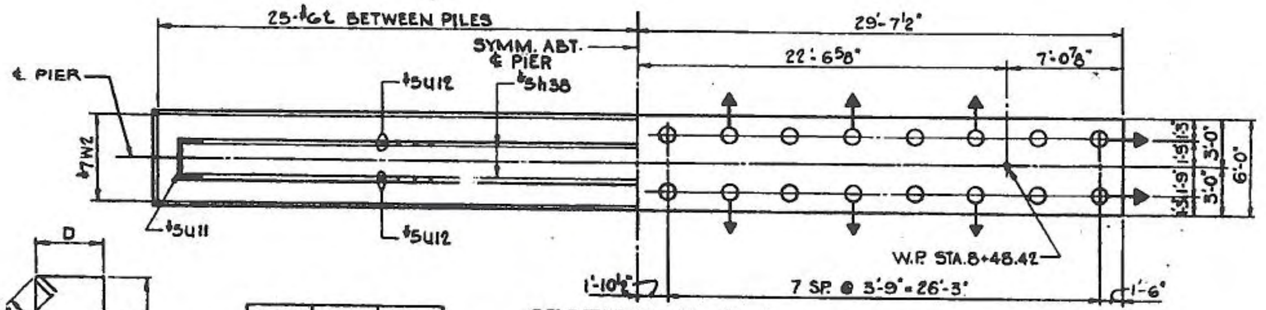
ELEVATION PIER 3
SCALE: ³⁄₁₆"=1'-0"



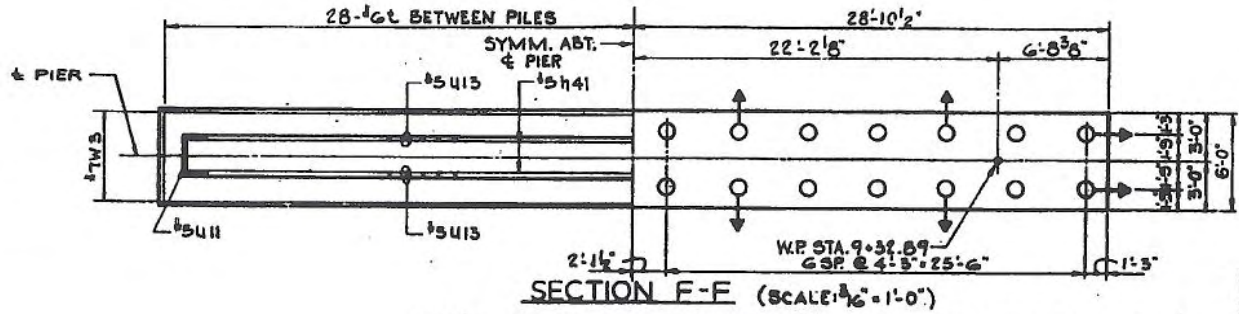
ELEVATION PIER 4
SCALE: ³⁄₁₆"=1'-0"



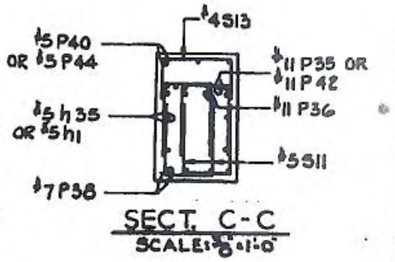
SECTION E-E
SCALE: ³⁄₁₆"=1'-0"



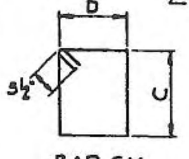
SECTION F-F
SCALE: ³⁄₁₆"=1'-0"



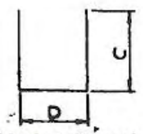
SECTION F-F (SCALE: ³⁄₁₆"=1'-0")



SECT. C-C
SCALE: ⅜"=1'-0"



BAR S14



BARS S10, S11, S12, S13
U11, U12, U13 & U14

BAR	C	D
S10	2'-0"	1'-6"
S11	2'-3"	1'-6"
S12	2'-4"	1'-4"
S13	1'-8"	2'-2"
S14	3'-2"	2'-2"
U11	1'-3"	2'-2"
U12	5'-9"	2'-2"
U13	6'-9"	2'-2"
U14	3'-7"	2'-2"

DE LEUW, CATHER & COMPANY ENGINEERS
DESIGNED BY R.D. KOESTER
DRAWN BY R.S. [illegible] & S. MUELLER
CHECKED
IN CHARGE E.S. MARTINIS
APPROVED W.G. HORN

BAR LIST

BAR	NO.		TOTAL	SIZE	LENGTH	SHAPE	BAR	NO.		TOTAL	SIZE	LENGTH	SHAPE	BAR	NO.		TOTAL	SIZE	LENGTH	SHAPE
	PIER 3	PIER 4						PIER 3	PIER 4						PIER 3	PIER 4				
h1	–	8	8	#5	33'-0"	—	P40	3	–	3	#5	20'-3"	—	t	49	56	105	#6	5'-6"	—
h35	8	–	8	#5	33'-9"	—	P41	3	–	3	#5	26'-9"	—							
h36	4	–	4	#7	23'-6"	—	P42	–	12	12	#11	16'-9"	—	U11	19	21	40	#5	4'-6"	⊔
h37	4	4	8	#7	34'-0"	—	P43	–	4	4	#9	19'-0"	—	U12	56	–	56	#5	13'-8"	⊔
h38	20	–	20	#5	28'-6"	—	P44	–	3	3	#5	19'-6"	—	U13	–	55	55	#5	18'-8"	⊔
h40	–	4	4	#7	22'-3	—	P45	–	3	3	#5	26'-3"	—	U14	56	55	111	#5	9'-4"	⊔
h41	–	24	24	#5	27'-9"	—														
							S10	56	56	112	#5	5'-6"	⊔	V21	–	56	56	#9	11'-3"	—
P35	12	–	12	#11	17'-6"	—	S11	32	32	64	#5	6'-0"	⊔	V22	56	–	56	#9	12'-9"	—
P36	8	8	16	#11	13'-0"	—	S12	128	112	240	#4	6'-0"	⊔							
P37	10	10	20	#9	36'-0"	—	S13	46	45	91	#4	5'-6"	⊔	W2	10	–	10	#7	30'-5"	—
P38	8	8	16	#7	6'-6"	—	S14	69	67	136	#5	11'-7"	□	W3	–	10	10	#7	29'-6"	—
P39	4	–	4	#9	20'-6"	—														

NOTES: ANCHOR BOLTS TO PROJECT 5½" ABOVE CONCRETE ON PIER 3 AND 4" ABOVE CONCRETE ON PIER 4. SPACE REINFORCEMENT IN CAP TO MISS ANCHOR BOLTS. ALL EDGES TO HAVE STD. ¾" CHAMFER EXCEPT AS NOTED. POUR STEPS MONOLITHICALLY WITH PIER CAP. ALL BAR DIMENSIONS ARE OUT TO OUT. MINIMUM BAR LAP = 24 DIA. KEY TO BAR INDICATION: 5×2-#6 ETC. INDICATES. 5 LINES OF #6 BARS WITH 2 LENGTHS PER LINE.

BILL OF MATERIAL

ITEM	UNIT	QUANTITY	
		PIER 3	PIER 4
CLASS A EXCAVATION FOR STRUCT.	CU. YD.	108	117
CLASS X CONCRETE	CU. YD.	97.6	93.7
REINFORCEMENT BARS	LBS.	11,705	11,470
FURNISHING CREOSOTED PILES 20.1 TO 35 FT.	LIN. FT.	1056	621
DRIVING TIMBER PILES	LIN. FT.	1056	621
TEST PILE (TIMBER)	EACH	—	1

PILE DATA

LOCATION	PIER 3	PIER 4
PILE TYPE	TIMBER	TIMBER
MINIMUM CAPACITY TONS	24	24
NUMBER REQUIRED	32	28#
ESTIMATED LENGTH FT.	33	23
CUT - OFF ELEVATION	652.61	652.26

INCLUDES 1 TEST PILE

PIERS 3 & 4
F.A.I.74 - SECTION 81-1HB-2
F.A.I.74 OVER SOUTH BOUND FRONTAGE RD.
ROCK ISLAND COUNTY
STATION 323+99.95
SCALE: AS NOTED DATE:

I-74 Final Design Project Team Site > Tasks > Task 868: Re-evaluation of the Illinois Viaduct Pile Design

Tasks: Task 868: Re-evaluation of the Illinois Viaduct Pile Design

! The content of this item will be sent as an e-mail message to the person or group assigned to the item.

New Item | Edit Item | Delete Item | Workflows | Alert Me | Version History

Title	Task 868: Re-evaluation of the Illinois Viaduct Pile Design
Priority	(2) Normal
Status	Completed
% Complete	100%
Assigned To	David Morrill
Description	Following the FHWA Geotechnical Review Meeting conducted on September 11, 2013, Bill Kramer provided David an email containing additional discussions regarding the FHWA comment on the pile design and construction for the structures in Illinois to be built using the Illinois IDOT spec book and BBS Bridge Manual. Bill suggested that the Benesch Team recheck the piles using an increased resistance factor of 0.60 for piles in soil, 0.65 for H-piles on shale, and 0.70 for H-piles on rock, rather than using 0.55 for all conditions. In addition, the maximum nominal bearing that can be specified for H-piles would increase from 54% to 65% of the H-pile yield strength times its cross-sectional area. To use these increased design values, Bill provided a Guide Bridge Special provision (GBSP) that would be added to the contract plans to assure the piles are not overdriven. Bill also suggested the Benesch Team run some design phase wave equation analysis to verify the pile can be driven to the rock with the hammer size limitations in the GBSP and not overstress the pile in the process. This task is assigned to Andrew to review Bill Kramer's September 11 e-mail and prepare a disposition of Bill's comments and outline the appropriate steps to be taken for the Illinois viaduct pile design.
Start Date	9/23/2013
Due Date	10/7/2013
Carbon Copy	Hossam A. Abdou; Ahmad Abu-Hawash; Robert Chantome; Chris Cromwell; Timothy Dunlay; Andrew J. Keaschall; John M. Kulicki; Rebecca A. Marruffo; Norm McDonald; Todd B. McMeans; Ron Meyer; David Morrill; Thomas P. Murphy; Kevin Placzek; Andrew Wilson; Bob Stanley; Philip A. Ritchie; Robert J. Tipton; Mark Thomson; Sheila Moynihan; Jerilyn M. Hassard; David W. Petermeier
Comments	10/23/13 David Morrill - per Mark Thomson's post below, the new pile design criteria outlined above will be reflected in the calculations and drawings for the I-74 Illinois Viaduct and associated Ramps C and D and for the I-74 and Ramp 7th A structures over 19th street and the I-74 structures over 12th Avenue. The intent of this task has been fulfilled and it is hereby closed. 10/23/13: Mark Thomson: IDOT BB&S is moving forward with plans to revise

IDOT's piling policies in the near future and these changes should be in place well before construction of this project takes place. BB&S agrees that the design team should incorporate the piling changes on the IL structures for this project. As noted, it is anticipated that the structure plans will be revised along with the work to incorporate revisions for changing to the 3B staging option. Revised plans will require BB&S review and approval. If there are any questions, please contact this office. This task is assigned back to Benesch to incorporate the changes.

10/15/2013: Andrew J. Keaschall: The piers and abutments for the proposed Illinois Viaduct and ramp structures are supported on piles driven to rock. The piles at the abutments range in length from 35 to 45 feet and most of them are driven prior to placement of embankment. The piles at the piers range in length from 10 feet to 25 feet. The strata overlying the bedrock varies over the length of the viaduct from soft clayey silts to loose sandy gravels. Pile installation in this area is likely to be very simple in the early stages and will likely be controlled by the special provision phrase "For piles driven to rock, pile driving shall be stopped, independent of the nominal driven bearing predicted by the formula in Article 512.14, when the minimum penetration rate is ¼ in. over 5 blows (or equivalently a maximum penetration rate of 20 blows per 1 in. for no more than 5 blows)." Based on these parameters, the design phase WEAP analysis is likely not required for this particular situation.

We would like to take advantage of the additional capacity available with the proposed modifications to Illinois DOT's pile capacity and GBSP (documents attached). Typically we have found the most efficient pile configuration is one that reduces the overall number of piles based on geometric constraints and then selects a pile that has adequate capacity for that configuration. The design team followed this methodology (even using HP 14x117 in a few places) and maximized the pile spacing while minimizing the number of different pile sections used. Therefore, potential savings associated with pile reconfiguration are likely to be minimal, however, across the board, the pile size can be reduced (in many cases by two sizes).

There are approximately 12,000 linear feet of pile on the Illinois viaduct and associated Ramp C and D. Incorporating the new pile methodology would result in a savings of about 25 pounds per foot of pile (on average) for a total weight savings of 300,000 lbs. This reduction in weight would result in a cost savings of approximately $150,000 for these structures.

With the Illinois DOT's approval, this change will be incorporated for the viaduct. Final plans will be re-submitted as a result of incorporating the Option 3B construction schedule revision and will reflect the updated pile sections with their associated NRB and FRA values.

The structures over 19th street, 12th Avenue and Ramp 7th A over 19th Street have to be re-designed as a result of the Option 3B MOT modifications. Again, with IDOT approval, the updated pile design procedure will be incorporated into the re-design.

This task is re-assigned to Mark Thomson of IDOT for review and discussion with Bill Kramer to provide direction on implementing the new pile criteria.

Attachments	IDOT Pile Design and Construction changes.docx Piling GBSP (WHKS Rev 9-4-13).docx

Version: 5.0
Created at 9/23/2013 11:34 AM by Diane M. Campione
Last modified at 10/23/2013 7:20 PM by David Morrill

IDOT pile design and construction changes proposed for implementation in 2013

1. **New larger H-pile and Metal Shell (MS)** pile sizes will be allowed to be used in design and specified on plans. The following is a list of our current and new pile sizes which will be available.

New piles to be added:

- **Metal Shell 16"Φ w/.312" walls**
- **Metal Shell 16"Φ w/.375" walls**
- **Steel HP 16 X 88**
- **Steel HP 16 X 101**
- **Steel HP 16 X 121**
- **Steel HP 16 X 141**
- **Steel HP 16 X 162**
- **Steel HP 16 X 183**
- **Steel HP 18 X 135**
- **Steel HP 18 X 157**
- **Steel HP 18 X 181**
- **Steel HP 18 X 204**

Current piles to remain available:

- Metal Shell 12"Φ w/.179" walls
- Metal Shell 12"Φ w/.25" walls
- Metal Shell 14"Φ w/.25" walls
- Metal Shell 14"Φ w/.312" walls
- Steel HP 8 X 36
- Steel HP 10 X 42
- Steel HP 10 X 57
- Steel HP 12 X 53
- Steel HP 12 X 63
- Steel HP 12 X 74
- Steel HP 12 X 84
- Steel HP 14 X 73
- Steel HP 14 X 89
- Steel HP 14 X 102
- Steel HP 14 X 117

2. The yield strength **(fy) of Metal Shell piles will be increased from 45ksi to 50ksi** (ASTM A-252 Grade 3 Modified). This will result in a 10% increase in the maximum nominal bearing that can be specified since it is currently computed by taking 85% of the shell yield strength times its steel cross-sectional area)

3. Piles designed using the WSDOT driving formula as construction bearing acceptance will use an **increased resistance factor of 0.60 for piles in soil, 0.65 for H-piles on shale, and 0.7 for H-piles on other rock, rather than 0.55** for all conditions.

4. The **maximum nominal bearing that can be specified for H-piles will increase from 54% to 65%** of the H-pile yield strength times its cross-sectional area. This will result in a 20% increase in the maximum nominal bearing that can be specified.

5. **A new "Soil Setup Pile Length" will be shown on the plans, in addition to the "Estimated Pile Length" currently shown.** While the Estimated pile length is determined using the IDOT Static Method of estimating pile length with the resistance factor for the WSDOT field verification formula (0.6), the setup length is determined using the resistance factor for the IDOT Static Method (0.3). This longer setup length provides theoretically the depth at which pile driving can be stopped and the pile accepted as having capacity without further verification, even though the WSDOT formula does not show bearing. However, accepting the soil setup length pile capacities independent of field bearing verification requires that quality soils boring data is available within 75' of the substructure. Therefore, until we become confident this length consistently provides capacity, piles within 85% of plan bearing will be allowed to setup for at least 24 hours while others must be left for a minimum of 48 hours and re-tapped to verify bearing. A table with longer recommended waiting times based on soil type has been included in the specification so it is understood that the capacities at minimum 24 or 48 hours do not reflect the full setup possible.

6. The **WSDOT dynamic formula will include a new Cs factor which will equal 0.8 when re-tapping** a pile to check for setup capacity gain after a waiting period and 1.0 at all other times. The WSDOT formula was developed to predict long term pile capacity at the end of initial driving and thus includes the average setup expected. When using this formula to check for the actual setup at a specific site, the average setup must be removed from the formula which is done by reducing its capacity by 20% (multiplying by 0.8).

7. **Reduced hammers energy requirements will be added to the specification for piles driven to rock.** This new range of acceptable hammer sizes is based on the WSDOT formula, plan bearing and penetration rates between 4 and 20 blows/inch. Driving can be stopped when the formula shows bearing or when the penetration rate is < ¼ in. over 5 blows for no more than 5 blows, whichever occurs first. Test piles driven to rock will only be required to be driven to plan bearing, not 110% of plan bearing. The current hammer energy criteria (based on the WSDOT formula, plan bearing and penetration rates between 1 and 10 blows/inch) will be retained but only used for piles driven in soil.

Soil (current)

$$E \geq \frac{32.9\ R_N}{F_{eff}}$$

$$E \leq \frac{65.8\ R_N}{F_{eff}}$$

Rock (new)

$$E \geq \frac{28.6\ R_N}{F_{eff}}$$

$$E \leq \frac{41.1\ R_N}{F_{eff}}$$

8. **A new Simplified Stress Formula (SSF) has been developed to estimate pile stresses during driving.** Designers will now be able to estimate pile stress, considering the specific soil conditions, and avoid the use of those which indicate possible damage during driving. The SSF can also be used by contractors or inspectors to evaluate various hammers being considered and avoid the use of those which indicate possible pile damage. The SSF has been added to our static method of estimating pile length and the WSDOT Pile Bearing Verification spreadsheets. Unacceptable risk of pile damage is defined as SSF estimated stress levels > 90 % of the pile yield strength.

		SOIL		ROCK	
Pile Type & Size	**Max. Nominal Required Bearing (kips)**	**Maximum Hammer Size (Kip-ft)**	**Minimum Hammer Size (Kip-ft)**	**Maximum Hammer Size (Kip-ft)**	**Minimum Hammer Size (Kip-ft)**
Metal Shell 12"Φ w/.179" walls	283	39568	19784		
Metal Shell 12"Φ w/.25" walls	392	54919	27460		
Metal Shell 14"Φ w/.25" walls	459	64260	32130		
Metal Shell 14"Φ w/.312" walls	570	79849	39925		
Metal Shell 16"Φ w/.312" walls	654	91493	45746		
Metal Shell 16"Φ w/.375" walls	782	109526	54763		
Steel HP 8 X 36	344	48223	24112	30101	20960
Steel HP 10 X 42	403	56413	28207	35214	24520
Steel HP 10 X 57	546	76462	38231	47729	33234
Steel HP 12 X 53	504	70500	35250	44007	30643
Steel HP 12 X 63	598	83734	41867	52269	36395
Steel HP 12 X 74	709	99197	49599	61921	43116
Steel HP 12 X 84	799	111908	55954	69855	48641
Steel HP 14 X 73	695	97363	48682	60775	42319
Steel HP 14 X 89	848	118787	59394	74149	51631
Steel HP 14 X 102	975	136478	68239	85192	59320
Steel HP 14 X 117	1118	156527	78264	97707	68035
Steel HP 16 X 88	839	117390	58695	73277	51024
Steel HP 16 X 101	972	136045	68023	84922	59132
Steel HP 16 X 121	1164	162890	81445	101679	70800
Steel HP 16 X 141	1355	189735	94868	118436	82468
Steel HP 16 X 162	1550	217035	108518	135477	94334
Steel HP 16 X 183	1758	246155	123078	153654	106991
Steel HP 18 X 135	1297	181545	90773	113324	78909
Steel HP 18 X 157	1502	210210	105105	131217	91368
Steel HP 18 X 181	1729	242060	121030	151098	105211
Steel HP 18 X 204	1957	273910	136955	170979	119055

Axial Geotechnical Resistance Design of Driven Piles

This Design Guide has been developed to provide geotechnical and structural engineers with the most recent methods and procedures required by the Department to determine the nominal and factored axial geotechnical resistance of a pile to help ensure cost effective foundation design and construction.

The Geotechnical Engineer must evaluate the subsurface soil/rock profile, develop pile design table(s) for each substructure, and provide them to the structure designer in the Structure Geotechnical Report (SGR). Each table shall contain a series of Nominal Required Bearing (R_N) values, the corresponding Factored Resistances Available (R_F) for design, the Estimated Pile Lengths, and the Soil Setup Pile Lengths, for all feasible pile types. The number of pile types and sizes covered as well as the range of R_N values provided must be large enough to allow the designer sufficient selection to determine the most economical pile type, size and layout such that the factored loading from the LRFD Strength Limit State and Extreme Event Load Combinations is $\leq R_F$. The corresponding R_N provided on the plans will typically be obtained during driving as indicated by dynamic formula or other nominal pile resistance field verification method. To develop the pile design tables, the geotechnical engineer shall use the IDOT Static Method of estimating this nominal pile resistance during driving and provide these values in the SGR as feasible R_N values which can be specified by the designer.

The original IDOT Static Method was developed over 40 years ago to correspond to the allowable pile resistance indicated during driving by the ENR dynamic formula. With the change to LRFD and FHWA Gates formula in 2007, the Department initiated an extensive research study with Dr. James Long of the University of Illinois at Urbana-Champaign to evaluate several static methods and dynamic formulas to determine the most accurate method for estimating pile lengths and resistances for the soils, piles, and hammers common to Illinois. The results of Phase 1 of the research, completed in 2009, indicated that an updated IDOT Static Method (with the new Pile Type Correction Factors) was more accurate than all other static estimating methods studied, including the program “DRIVEN”. It was also found to correspond closest to the most accurate dynamic formula studied which was the WSDOT formula, developed by Tony Allen of the Washington State DOT in 2005. Based on this research, the WSDOT formula was chosen to replace the FHWA Gates formula as the standard method of construction verification with the IDOT Static Method, described below, chosen for use in developing the SGR pile design tables. Phase 2 of the U of I research was completed in 2012 and included the acquisition of additional pile driving analyzer data to further improve correlation of the static and dynamic methods, increase pile capacity, identify potential for pile damage, and provide procedures to prevent piles from running excessively long. The design guide has been subsequently updated to reflect these improvements.

Nominal Required Bearing (R_N) represents the nominal pile resistance expected at any specific length during driving that can be specified by the Designer. It must be calculated at various estimated lengths and is the first step in developing the pile design table.

In the case of displacement piles (such as metal shell, precast, and timber piles), R_N shall be calculated as the sum of the side and tip resistance as follows:

$$R_N = (F_S q_S A_{SA} + F_P q_P A_P)*(I_G)$$

Where the nominal side resistance ($F_S q_S A_{SA}$) is the product of the following:

F_S = The pile type correction factor for side resistance (0.758 for displacement piles in cohesionless soils & 1.174 for displacement piles in cohesive soils)
q_S = The nominal unit side resistance
A_{SA} = The surface area of the pile

And the nominal tip resistance ($F_P q_P A_P$) is the product of the following:

F_P = The pile type correction factor for tip resistance (0.758 for displacement piles in cohesionless soils & 1.174 for displacement piles in cohesive soils)
q_P = The nominal unit tip resistance
A_P = The tip area of the pile

In the case of non-displacement piles (such as steel H-piles), the R_N shall be taken as the lesser of the following:

The fully “plugged” side and tip resistance defined as:

$$R_N = (F_S q_S A_{SAp} + F_P q_P A_{Pp})*(I_G)$$

And the fully “unplugged” side and tip resistance defined as:

$$R_N = (F_S q_S A_{SAu} + F_P q_P A_{Pu})*(I_G)$$

Where:

F_S = The pile type correction factor for side resistance (0.15 for non-displacement piles in cohesionless soils, 0.75 for non-displacement piles in cohesive soils & 1.0 for non-displacement piles in rock)

F_P = The pile type correction factor for tip resistance (0.3 for non-displacement piles in cohesionless soils, 1.5 for non-displacement piles in cohesive soils & 1.0 for non-displacement piles in rock)

A_{SAu} = The unplugged surface area = (4 x flange width + 2 x member depth) x pile length

A_{SAp} = The plugged surface area = (2 x flange width + 2 x member depth) x pile length

A_{Pu} = The cross-sectional area of steel member

A_{Pp} = The flange width x member depth

In the above equations, the term I_G is the bias factor ratio (equal to 0.87 for soil and 1.0 for rock) and is discussed in further detail later in the design guide. The Nominal Unit Side Resistance (q_S) and Nominal Unit Tip Resistance (q_P) shall be calculated as follows:

- *Nominal Unit Side Resistance* (q_S) of **granular soils** is computed using the equations below:

For Hard Till, the equations below are used for the range of N values indicated:

$q_S = 0.07N$ for $N < 30$

$q_S = 0.00136N^2 - 0.00888N + 1.13$ for $N \geq 30$

Very Fine Silty Sand, the equations below are used for the range of N values indicated:

$q_S = 0.1N$ for $N < 30$

$q_S = 42.58e^{\left[\frac{(N-175.05)^2}{-7944}\right]}$ for $30 \leq N < 74$

$q_S = 0.297N - 10.2$ for $N \geq 74$

Fine Sand, the equations below are used for the range of N values indicated:

$q_S = 0.11N$ for $N < 30$

$q_S = 0.3256N + \frac{182}{N} - 12.51$ for $30 \leq N < 66$

$q_S = 0.329N - 9.91$ for $N \geq 66$

Medium Sand, the equations below are used for the range of N values indicated:

$q_S = 0.117N$ for $N < 26$
$q_S = 0.00404N^2 - 0.0697N + 2.13$ for $26 \leq N < 55$
$q_S = 0.356N - 9.1$ for $N \geq 55$

Clean Coarse Sand, the equations below are used for the range of N values indicated:

$q_S = 0.128N$ for $N < 24$
$q_S = 0.00468N^2 - 0.0693N + 2.05$ for $24 \leq N < 50$
$q_S = 0.394N - 9.42$ for $N \geq 50$

Sandy Gravel, the equations below are used for the range of N values indicated:

$q_S = 0.129N$ for $N < 20$
$q_S = 0.0074N^2 - 0.187N + 3.36$ for $20 \leq N < 40$
$q_S = 0.52N - 12.9$ for $N \geq 40$

Where N = Field measured SPT blow count (blows/ft)

- *Nominal Unit Side Resistance* (q_S) of **cohesive soils**, shall be calculated using the equations below for the range of Q_U values indicated:

$q_S = \frac{-1}{2500}Q_u^3 - 0.177Q_u^2 + 1.09Q_u$ for $Q_u \leq 1.5$ tsf
$q_S = 0.0495Q_u^3 - 0.347Q_u^2 + 1.278Q_u - 0.068$ for $1.5 \text{ tsf} < Q_u < 2$ tsf
$q_S = 0.47Q_u + 0.555$ for $2 \text{ tsf} \leq Q_u < 4.5$ tsf
$q_S = 2.67$ ksf for $4.5 \text{ tsf} \leq Q_u$

Where Q_u = Unconfined compression strength of the soil in tsf.
Note that Q_u is input in tsf and q_S is output in ksf.
If $Q_u > 3$ tsf and $N > 30$, treat as granular and use Hard Till equations.

- *Nominal Unit Side Resistance* (q_S) of **rock**, shall be calculated using the equations below for the type of rock encountered:

$q_S = 12.0$ ksf for Shale
$q_S = 20.0$ ksf for Sandstone
$q_S = 24.0$ ksf for Limestone/Dolomite

- *Nominal Unit Tip Resistance* (q_P) of **granular soils,** shall be calculated as follows:

$$q_P = \frac{0.8\ N\ D_b}{D} \leq q_\ell$$

Where:

q_ℓ = 8N for sands and gravel

q_ℓ = 6N for fine silty sand and hard till

D = Pile diameter or width (ft)

D_b = Depth of penetration into soil (ft)

N = Field measured SPT blow count (blows/ft)

- *Nominal Unit Tip Resistance* (q_P) of **cohesive soils,** shall be calculated as follows:

$$q_P = 9Q_u$$

Note that Q_u is input in tsf and q_P is output in ksf.

- *Nominal Unit Tip Resistance* (q_P) of **rock,** shall be calculated using the equations below for the type of rock encountered:

q_P = 120.0 ksf	for Shale
q_P = 200.0 ksf	for Sandstone
q_P = 240.0 ksf	for Limestone/Dolomite

Note that actual pile penetration into rock is related to several factors including rock type and strength, degree of weathering, hammer energy, and nominal required bearing. The above empirical side and tip resistance values for rock, when used with the soil side resistance, should provide a conservative, yet practical, estimate of pile penetration into rock and thus total estimated pile length.

Maximum Nominal Required Bearing ($R_{N\ MAX}$) is the maximum R_N value that can typically be specified on the plans to avoid dynamic stresses during driving which would cause damage to the pile. The value may be determined by use of the Simplified Stress Formula (SSF), discussed below, or a wave equation analysis considering the site specific soils and driving equipment to permit more cost effective designs. In the absence of a site specific wave equation drivability analysis or unless SSF indicates a lesser value should be used, the $R_{N\ MAX}$ may be calculated using the following empirical relationships:

- Metal Shell Piles: $R_{N\ MAX} = 0.85xF_YA_S$

  Where: F_Y = yield strength of the steel shell (50 ksi)
  A_S = the steel shell cross-sectional area (in.^{2})

- Steel H-Piles: $R_{N\ MAX} = 0.65xF_YA_S$

  Where: F_Y = yield strength of the steel (50 ksi)
  A_S = the steel cross-sectional area (in.^{2})

- Precast Piles: $R_{N\ MAX} = 0.3xf'_cxA_g$

  Where: f'_c = compressive strength of concrete (4.5 or 5 ksi)
  A_g = gross concrete cross sectional area of pile (in.^{2})

- Timber Piles: $R_{N\ MAX} = 0.5xF_{co}xA_P$

  Where: F_{co} = resistance in compression parallel to grain (2.7 ksi)
  A_P = cross-sectional timber area at top of pile (in.^{2})

The SSF is a method developed by the U of I to provide a relatively simple and reasonably accurate estimation of the maximum pile stresses during the driving process. The method consists of numerous equations presented near the end of the design guide and has been integrated into the IDOT Static Method of Estimating Pile Length spreadsheet to predict an estimated driving stress for metal shell and steel H-piles.

Use of the SSF requires knowledge of the pile driving system (hammer weight, hammer cushion data, etc.) that is typically unknown during the design phase. To facilitate use of the SSF, a database of open-ended diesel hammers have been incorporated into the IDOT Static Method of Estimating Pile Length spreadsheet to allow driving stresses to be calculated for an array of hammers satisfying the hammer energy requirements for the WSDOT formula. The stresses from the array of hammers have been averaged to indicate an “Average Estimated Driving Stress” as the pile enters each soil or rock layer.

Empirical relationships based solely upon F_Y and cross-sectional pile area can result in poor protection against pile damage during driving. While the $R_{N\ MAX}$ values listed above are generally anticipated to result in acceptable driving stresses, scenarios may be encountered that prevent piles from reaching $R_{N\ MAX}$ prior to exceeding the maximum acceptable driving stress of $0.9*F_Y$. For instance, steel H-piles being driven to shallow rock may become overstressed prior to reaching $R_{N\ MAX}$ and R_N values less than $R_{N\ MAX}$ may need to be chosen to ensure acceptable driving stresses. The SSF is particularly useful during design in identifying soil layers that are considered hard driving conditions for metal shell piles and may result in large driving stresses and potential pile damage. Alternate pile types should be selected when driving stresses are anticipated to exceed $0.9*F_Y$ before an acceptable penetration depth or bearing is achieved. In addition, the SSF has also been incorporated into the WSDOT Pile Bearing Verification spreadsheet to allow Contractors and field inspectors the opportunity to evaluate the estimated driving stresses for the various hammer configurations being considered by the Contractor.

<u>Factored Resistance Available</u> (R_F) represents the net long term axial factored geotechnical resistance available at the top of the pile to support factored structure loadings. It accounts for losses in geotechnical resistance that occurs after driving due to scour, downdrag (DD_R), or liquefaction (Liq.), resistance required to support downdrag loads (DD_L) and reflects the resistance factor used to verify R_N. R_F shall be calculated using the following equation:

$$R_F = R_N(\phi_G) - (DD_R\text{+Scour+Liq.})x(\phi_G)\ x(I_G)\ - DD_L x(\gamma_p)$$

Where:

Scour = nominal side resistance (loss) of soil above the design scour elevation.
Liq. = nominal side resistance (loss) of soil within liquefiable layers.
DD_R = nominal side resistance (loss) of soil expected to settle > 0.4 in.
DD_L = nominal side resistance (load) of soil expected to settle > 0.4 in.
ϕ_G = the Geotechnical Resistance Factor for the construction verification of R_N
I_G = the Bias Factor Ratio relating the IDOT Static Method to the construction verification method used.
γ_p = the DD_L Load Factor for the downdrag soil loading on the pile

Applying the geotechnical resistance factor (ϕ_G) to the geotechnical losses may appear unconservative. However, AASHTO LRFD Article 10.7.3.7 requires the factored loads (R_F + $\gamma_p DD_L$) be ≤ the factored resistance below the downdrag layers. Thus, the pile must be driven to a R_N equal to the nominal downdrag resistance (DD_R) to install the pile through the downdrag layer plus (R_F + γ_p DD_L)/ϕ_G which results in both the geotechnical losses and R_N being multiplied by ϕ_G.

The nominal values of the downdrag (DD_R and DD_L), Scour, and Liquefaction (Liq.) shall be calculated using the IDOT Static Method side resistance equations provided above and as described below.

- *Downdrag* is considered twice to represent the loss in side resistance (DD_R) and again to account for the added loading (DD_L) applied to the pile. The LRFD load groups specify that the portion of downdrag which applies a loading to the pile be included with loadings from other applicable sources. However, it is IDOT's policy to require that the downdrag loading (DD_L) and downdrag reduction in resistance (DD_R) for a pile be taken into account by the geotechnical engineer so it can be incorporated in the SGR pile design tables. Thus they should not be included by the structural engineer in calculating the factored loadings.

- *Scour* protection is provided by accounting for the loss in side resistance of soil layers above the design scour elevation in determining the R_F available to designers. The Scour term shall be taken as zero when calculating the R_F to resist Extreme Event I seismic loadings.

- *Liquefaction* is the loss of side resistance in layers expected to liquefy (Liq.) due to the design seismic event. Since liquefied soil of sufficient thickness consolidates, any non-liquefiable layers above such soils will settle and produce downdrag effects which must also be taken into account. Thus, in addition to Liq., losses from DD_R and DD_L for the layers above the liquefied soils shall be calculated and included in the R_F equation. However Liq. and downdrag caused by liquefaction shall only be considered when calculating the R_F to resist Extreme Event I seismic loadings.

The values of geotechnical losses (Scour, DD_R, DD_L, and Liq.) for non-displacement steel H-piles shall be calculated using the surface area assumption, A_{SAp} (representing "plugged" conditions), regardless of whether the controlling value of R_N used "plugged" or "unplugged" side resistance.

Values for the Geotechnical Resistance Factor, Bias Factor Ratio, and DD_L Load Factor, shall be selected as follows:

- *The Geotechnical Resistance Factor* (ϕ_G) shall be selected to represent the reliability of the method used during construction to verify that the R_N has been developed. Statistical calibration from ongoing U of I research using local dynamic pile driving analyzer testing indicates that a ϕ_G of 0.60 should be used to compute R_F for friction piles when the WSDOT formula is specified for construction verification. When more accurate construction verification methods are proposed, such as with static load test or a Pile Driving Analyzer (PDA), the resistance factor used may be increased to the values provided in the AASHTO specifications.

  Research and statistical calibration by U of I has also determined that ϕ_G for the IDOT Static Method for friction piles, without the use of any construction verification methods, should be taken to be 0.3. Comparison of the resistance factors for the WSDOT formula and IDOT Static Method indicates that there is typically a significant advantage to measuring the driven bearing of a pile in the field using a construction verification method. In order to rely on the IDOT Static Method to provide a reliable design pile length without R_N verification, it is critical that the subsurface conditions are adequately characterized at the substructure unit under consideration. To ensure reliable subsurface data, it is recommended that borings be located such that no foundation element is more than 75 ft from a boring location. At such locations, a second pile length will also be provided using the IDOT Static Method ϕ_G of 0.3, in addition to the standard estimated length provided for WSDOT formula. This length should provide the maximum depth the pile should need to be driven to when the formula does not indicate bearing. However, until sufficient confidence is developed, piles reaching this depth will be allowed to setup and re-tapped to verify adequate bearing. This length may be much deeper than the estimated pile length and will be referred to as the Soil Setup Length.

  For end bearing piles being driven to rock, ϕ_G shall equal 0.70 except for piles driven to shale in which case ϕ_G shall equal 0.65. A reduced ϕ_G is specified for shale to account for relaxation that has been reported by some DOT's and continues to be studied by ongoing research with the U of I.

- *The Bias Factor Ratio* (I_G), shall be included in the calculation for the nominal pile resistance (R_N) and also be applied to the geotechnical losses (Scour, DD_R, and Liq.) to account for differences in bias between the method used to estimate these values (using the IDOT static method) and the construction method used to verify the R_N (typically the WSDOT formula). Research by the U of I indicates that I_G should equal 0.87 in soil layers and 1.0 in rock layers when correlating the IDOT Static Method to the WSDOT formula. Since determining the pile

Soil Setup Length at each R_N using the IDOT Static Method is independent of the construction verification method, I_G shall equal 1.0.

- *The DD$_L$ Load Factor* (γ_p) shall be equal to 1.0 for DD_L caused by cohesive or granular soil layers for piles in compression. This load factor has been determined using statistical calibration data for the IDOT Static Method as outlined near the end of the design guide.

  γ_p shall be equal to 0.30 for DD_L caused by cohesive or granular soil layers when the pile is required to provide pullout or uplift resistance.

If it becomes clear during the planning process that earthquake forces may govern the pile design, the SGR pile tables should include both the R_F to support Extreme Event I Limit State loadings by setting the ϕ_G to 1.0, as well as the R_F to support Strength Limit State loadings by setting ϕ_G to the value corresponding to the construction verification method being used (typically 0.6 for the WSDOT formula for friction piles and 0.65 or 0.7 for end bearing piles driven to rock).

In load cases requiring piles to provide uplift resistance, the factored tension or pullout resistance of the pile shall be determined using the nominal side resistance equations provided above and applying a geotechnical resistance factor (ϕ_G) of 0.20 for uplift under Strength Limit State loadings and 0.8 for uplift under Extreme Event I Limit State loadings. For non-displacement steel H-piles, pullout resistance shall be computed using the surface area assumption (A_{SAp}) for a "plugged" condition only. This calculation will provide the minimum tip elevation which must be specified on the plans ensure pullout resistance.

Estimated Pile Lengths shall be provided in the pile design tables corresponding to the R_N and R_F values computed using the equations above. Since calculating these values requires assumption of the pile length, the procedures and guidance provided below shall be used in determining how these lengths should be selected and which should be provided in the pile design tables in the SGR:

- The geotechnical engineer should contact the structural engineer to obtain preliminary substructure locations and their total factored vertical loading as well as the ground surface, pile cutoff, and bottom of footing/substructure excavation elevations.

- The geotechnical engineer shall evaluate the subsurface soil and rock boring data to develop the profile of pile design parameters (N and Qu) at each substructure.

- Compute the relationship between R_N and pile penetration expected as the pile is driven from the footing/substructure excavation elevation through the various soil design profile for each possible pile type at every substructure. This is typically done by breaking up the soil profile into smaller (≈ 2.5' thick) layers and selecting pile lengths corresponding to the bottom of each layer. This provides the R_N consisting of the cumulative side resistance of all layers above the bottom of the layer in question and the tip resistance of the layer just below the bottom of the layer in question.

- Determine the maximum nominal required bearing feasible to specify without causing damage to the pile. This is most often done using the empirical relationships provided above for approximating $R_{N\ MAX}$, but lesser values may need to be considered depending upon the estimated driving stresses determined using the SSF. Wave equations analysis may also be used to determine if higher values of R_N can be provided in the pile design tables.

- Use the total vertical factored substructure loadings divided by the maximum and minimum pile spacing to provide an initial estimate of the range of R_F and determine the corresponding estimated pile lengths to provide in the tables.

- Discuss this initial range of R_F and the corresponding estimated lengths with the structural engineer to help finalize the range to be included in the SGR. It is preferred that the tables contain too many, rather than too few values to allow the designer the most data upon which to determine the most economical pile type and foundation design layout.

- It is important to again verify the preliminary information and adjust the pile design tables if any elevations or loads have changed. The estimated pile length contained in the design tables (and shown on the plans) must include the portions of the pile which will be incorporated in the substructure and footing. Thus, the ground surface adjacent to the pile during driving and proposed pile cutoff elevations must be accurately determined and documented in the SGR.

- In addition, the pile Soil Setup Length (L_{SETUP}) should also be provided for the range of R_F being reported in the SGR. L_{SETUP} is the pile length using the IDOT Static Method ϕ_G of 0.3 which does not require construction verification. L_{SETUP} should be provided in the contract plans to indicate the maximum length that the piles should be driven to in the event that the construction verification method is indicating insufficient R_N and the piles drive significantly longer than the estimated pile length shown on the plans. In this instance, a waiting period shall be endured and the piles re-tapped to check gain in nominal driven bearing due to soil setup.

Construction Verification Methods are typically used in the field to measure the nominal driven bearing (R_{NDB}) of a pile as it is installed, and in some cases afterwards. The benefit of using such methods is that it allows the use of larger design capacities due to the uncertainty in R_N being limited only to the reliability of the construction verification method being used. They also offer the advantage of providing the resistance at each pile which addresses concerns over the soil strength variability across a site and the accuracy of the soils testing. The alternative to relying on construction verification methods is to use a theoretical method (such as the IDOT static pile design procedure), using a bias ratio factor of 1.0 and the methods geotechnical resistance factor (0.3 in the case of the IDOT Static Method). However, since this method is dependent on the soils data and subsequently the assumed soil properties, the quality of soils investigation is critical when not using a construction verification method.

Although there are a number of construction methods available, IDOT has chosen to use the WSDOT formula as the primary means of determining the R_{NDB} of piles considering research completed by the U of I. The WSDOT formula was initially developed to provide a R_{NDB} of a pile, using hammer energy and pile penetration rate at end-of-driving (EOD), that corresponds to the nominal bearing determined using a static load test. The U of I has further studied the correlation between the capacity predicted by the WSDOT formula using EOD data and the capacity measured using dynamic testing at beginning-of-redrive (BOR) conducted days later. Elapsed time between EOD measurements and static load tests or BOR data allows for dissipation of increased pore water pressure that often occurs during pile driving typically resulting in an increase in capacity. This increase in capacity is referred to as soil setup.

The WSDOT formula, in its original form, has been developed to predict a certain amount of setup based upon EOD data. This was also taken into consideration by the U of I in the statistical calibration resulting in the previously discussed 0.60 ϕ_G. As such, using the original form of the WSDOT formula with BOR data to verify soil setup will likely result in an over prediction of pile capacity. As such, IDOT has introduced a soil setup correction factor, C_s, into the WSDOT formula to account for the average assumed setup. Thus, the C_s value shall equal 1.0 during and at the end-of-driving (EOD), but shall be taken as 0.8 after any beginning-of-redrive (BOR) procedure. The modified WSDOT formula including the C_s is shown below and the remaining variables are defined in the IDOT construction specifications.

$$R_{NDB} = \frac{6.6\ C_S\ F_{eff}\ E\ \ln(10N_b)}{1000}$$

Reliable prediction of the R_{NDB} of a pile bearing in soil, using the WSDOT formula, is partially dependent upon the hammer chosen by the Contractor to drive the pile. An overly robust hammer can suggest very low pile penetration resistance while an undersized hammer may not generate a pile penetration that is sufficient to mobilize the full pile capacity. To address this, IDOT construction specifications requires that pile driving hammers be capable of operating at an energy that results in a pile penetration rate (N_b) between 1 and 10 blows per inch according to the WSDOT formula for EOD and the R_N indicated in the plans. When R_{NDB} is required to be verified using BOR data, an N_b greater than 10 may be experienced depending upon the magnitude of the gain in R_{NDB} due to soil setup. U of I research data suggests that the R_{NDB} predicted by the WSDOT formula remains reliable when compared to R_{NDB} predicted by dynamic testing for a N_b up to approximately 20 when using BOR data and the above mentioned C_s factor. As such, the IDOT construction specifications require that R_N be achieved at an N_b between 1 and 10 for EOD but permits an expanded N_b range of 1 to 20 for BOR.

As an alternative to the WSDOT formula, the field inspector may analyze BOR data using the Wave Equation Analysis of Piles (WEAP) software program. When performing WEAP using the nominal side and tip resistances estimated by the IDOT Static Method, piles will only be required to achieve a R_{NDB} equal to 85% of R_N indicated in the pile data in the contract plans. The reduction in R_{NDB} is a reflection of the statistical bias of the WEAP method compared to dynamic testing and BOR data.

Simplified Stress Formula (SSF) is a method developed by the U of I for estimating stresses during metal shell and steel H-pile driving and is derived from WEAP stress predictions. Equations for estimating driving stresses using the SSF are provided below. Reference is made to research report FHWA-ICT-12-011, "Improved Design for Driven Piles on a Pile Load Test Program in Illinois", for further information regarding development of the SSF method. It is noted that the SSF was developed according to driving data for open-ended diesel hammers as this is the dominant hammer type used on IDOT projects. The Department has extrapolated beyond the research data to include other hammer types, as indicated in some of the formulas found below, and checked the SSF predictions against a limited number of WEAP results.

σ_C = corrected peak compressive stress (ksi)

$$= \frac{\sigma_P\ C_O}{C_S\ C_W\ C_L\ C_R}$$

σ_P = peak compressive stress (ksi)

$$= \frac{F_P}{A_P}$$

F_P = peak force (kips)

$$= C_F\ V_H\ I_H$$

C_F = peak force coefficient

$$= \frac{1}{W_D}\ e^{(-\xi\ T_X)}\ \sin\ (W_D\ T_X) \quad \text{for } I_R > 0.5$$

$$= \frac{1}{e} \quad \text{for } I_R = 0.5$$

$$= \frac{1}{W_D}\ e^{(-\xi\ T_X)}\ \sinh\ (W_D\ T_X) \quad \text{for } I_R < 0.5$$

ξ = damping ratio

$$= \frac{1}{2\ I_R}$$

$$W_D = \sqrt{\xi^2 - 1} \quad \text{for } \xi > 1$$

$$= \sqrt{1 - \xi^2} \quad \text{for } \xi < 1$$

C_O = overall correction factor

= 0.9 for diesel hammers

= 1.25 for air/steam hammers

A_P = pile cross-sectional area (in.^{2})

I_R = impedance ratio

$$= \frac{I_P}{I_H}$$

I_P = pile impedance (k*s/ft)

$$= \frac{E\ A_P}{c}$$

c = wave speed of pile material (ft/s)

$$= \sqrt{\frac{144\ E\ g}{\rho}}$$

E = modulus of elasticity of pile material (ksi)

g = acceleration of gravity (ft/s^2)

ρ = density of pile material (kcf)

$T_X = \frac{1}{W_D} \text{ atan} \left(\frac{W_D}{\xi}\right)$ for $I_R > 0.5$

$= 1$ for $I_R = 0.5$

$= \frac{1}{W_D} \text{ atanh} \left(\frac{W_D}{\xi}\right)$ for $I_R < 0.5$

C_S = pile set correction factor

$= 0.6281\, s^2 - 0.0058\, s + 0.6956$

s = pile set (in.)

$= \frac{1}{N_b}$

N_b = hammer blows per inch of pile penetration

C_L = pile length correction factor

$= 0.0046\, L + 0.7265$ (for metal shell piles)

$= 0.0011\, L + 0.8953$ (for steel H-piles)

I_H = hammer impedance (k*s/ft)

$= \sqrt{\frac{12\, k_c\, W_H}{g}}$

k_c = hammer cushion axial stiffness (k/in.)

$= \frac{A_C\, E_C}{t}$

V_H = ram impact velocity

$= \sqrt{2\, g \text{ eff } S_T}$

eff = hammer efficiency

= 0.80 for diesel hammers

= 0.67 for single acting air/steam hammers

= 0.50 for double acting air/steam hammers

S_T = hammer stroke (ft)

C_W = hammer ram weight correction factor

$= 1.395 \left(\frac{W_H}{A_P}\right)^2 - 2.869 \left(\frac{W_H}{A_P}\right) + 2.106$

W_H = weight of hammer ram (kips)

L = embedded length of pile in the ground (ft)

A_C = area of hammer cushion (in.^{2})

E_C = composite modulus of elasticity for 2-material hammer cushion (ksi)

$= \frac{E_1 E_2\, t}{(E_1 t_2) + (E_2 t_1)}$

E_1 = modulus of elasticity for hammer cushion material #1 (ksi)

E_2 = modulus of elasticity for hammer cushion material #2 (ksi)

t_1 = thickness of hammer cushion material #1 (in.)

t_2 = thickness of hammer cushion material #2 (in.)

t = total composite thickness for 2-material hammer cushion (in.)

C_R = pile side resistance proportion correction factor

$= -0.5006\, P_S^2 + 0.8226\, P_S + 0.8105$ (for metal shell piles)

$= -0.9767\, P_S^2 + 1.233\, P_S + 0.7044$ (for steel H-piles)

P_S = ratio of cumulative side resistance to total pile resistance

The Downdrag (DD_L) Load Factor (γ_p) has been statistically calibrated for the IDOT Static Method used to estimate the DD_L demand for the Strength Limit State and the WSDOT formula typically used for construction verification of the geotechnical resistance of the pile. An adjusted version of the corrected First Order Second Moment calibration method (used by the U of I in the report FHWA-ICT-12-011, "Improved Design for Driven Piles on a Pile Load Test Program in Illinois") that includes DD_L in addition to dead and live load has been used to generate a load factor consistent with the target reliability index. The adjusted version of the calibration method is indicated below.

ϕ = WSDOT construction verification method geotechnical resistance factor

$$= \frac{\lambda_R Q \sqrt{\frac{1+COV(Q)^2}{1+COV(R)^2}}}{E(Q)e^{\left[\beta\sqrt{\ln\left[\left(1+COV(R)^2\right)\left(1+COV(Q)^2\right)\right]}\right]}}$$

= 0.6

λ_R = WSDOT construction verification method bias factor

= 0.910

COV(R) = WSDOT construction verification method coefficient of variation

= 0.252

Q = random variable for load

$= \gamma_D Q_D + \gamma_{DD} Q_{DD} + \gamma_L Q_L$

Q_D, Q_{DD}, and Q_L = dead, downdrag, and live loads

γ_D, γ_{DD}, and γ_L = dead, downdrag, and live load factors

$\gamma_D = 1.25$ and $\gamma_L = 1.75$

COV(Q) = load coefficients of variation

$$COV(Q)^2 = \frac{\frac{Q_D^2}{Q_L^2}\lambda_{Q_D}^2\ COV(Q_D)^2 + \ \lambda_{Q_L}^2\ COV(Q_L)^2 + \frac{Q_{DD}^2}{Q_L^2}\lambda_{Q_{DD}}^2\ COV(Q_{DD})^2}{\frac{Q_D^2}{Q_L^2}\lambda_{Q_D}^2 + 2\frac{Q_D}{Q_L}\lambda_{Q_D}\lambda_{Q_L} + 2\frac{Q_D Q_{DD}}{Q_L^2}\lambda_{Q_D}\lambda_{Q_{DD}} + \lambda_{Q_L}^2 + 2\frac{Q_{DD}}{Q_L}\lambda_{Q_{DD}}\lambda_{Q_L} + \frac{Q_{DD}^2}{Q_L^2}\lambda_{Q_{DD}}^2}$$

λ_{Q_D}, $\lambda_{Q_{DD}}$, and λ_{Q_L} = bias factors for dead, downdrag and live loads

$\lambda_{Q_D}=1.05$ and $\lambda_{Q_L}=1.15$

$COV(Q_D)$, $COV(Q_{DD})$, and $COV(Q_L)$ = dead, downdrag, and live load coefficients of variation

$COV(Q_D) = 0.1$, $COV(Q_{DD}) = COV(KIDOT)$, and $COV(Q_L) = 0.2$

COV(KIDOT) = IDOT Static Method coefficient of variation

= 0.492

μ_{KIDOT} = mean $\frac{\text{Predicted (IDOT Static Method) Resistance}}{\text{Measured (CAPWAP(BOR)) Resistance}}$

= 1.45

$\lambda_{Q_{DD}}$ = bias for the median 50^{th} percentile of the IDOT Static Method

$$= \frac{\sqrt{1+COV(KIDOT)^2}}{\mu_{KIDOT}}$$

$$= \frac{\sqrt{1+(0.492)^2}}{1.45} = 0.77$$

β = target reliability index

= 2.33

E(Q) = expected load

$= \lambda_{Q_D} Q_D + \lambda_{Q_{DD}} Q_{DD} + \lambda_{Q_L} Q_L$

$\frac{Q_D}{Q_L}$ = ratio of dead load to live load

= 2.0 (assumed); $Q_L = 0.5\ Q_D$

$\frac{Q_{DD}}{Q_D}$ = ratio of downdrag load to dead load

= 0.5 (assumed); $Q_{DD} = 0.5\ Q_D$

Substituting all of the above variables into the equation shown for ϕ, trial and error calculations indicate that the downdrag load factor, γ_{DD}, ≈ 1.0.

IDOT STATIC METHOD OF ESTIMATING PILE LENGTH-WSDOT VERIFICATION

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

Modified 8/22/2013

SUBSTRUCTURE & REFERENCE BORING========== WB South Abutment, Boring B-5

LRFD, ASD, or EXTREME EVENT ================== LRFD
PILE CUTOFF ELEV. ============================ 665.10 FT
GROUND SURFACE ELEV. AGAINST PILE ========== 664.10 FT (DURING DRIVING)
GEOTECH. LOSS TYPE (None, Scour, Liquef., DD) ==== None
BOTTOM ELEV. OF SCOUR, LIQUEF., or DD ======== FT
TOP ELEV. OF LIQUEF. (so layers above apply DD) === FT

TOTAL FACTORED SUBSTRUCTURE LOAD ========= 2394 KIPS
TOTAL LENGTH OF SUBSTRUCTURE (along skew)==== 133.00 FT
NUMBER OF ROWS OF PILES PER SUBSTRUCTURE = 2
Approx. Factored Loading Applied per pile at 8 ft. Cts ============ 72.00 KIPS
Approx. Factored Loading Applied per pile at 3 ft. Cts ============ 27.00 KIPS

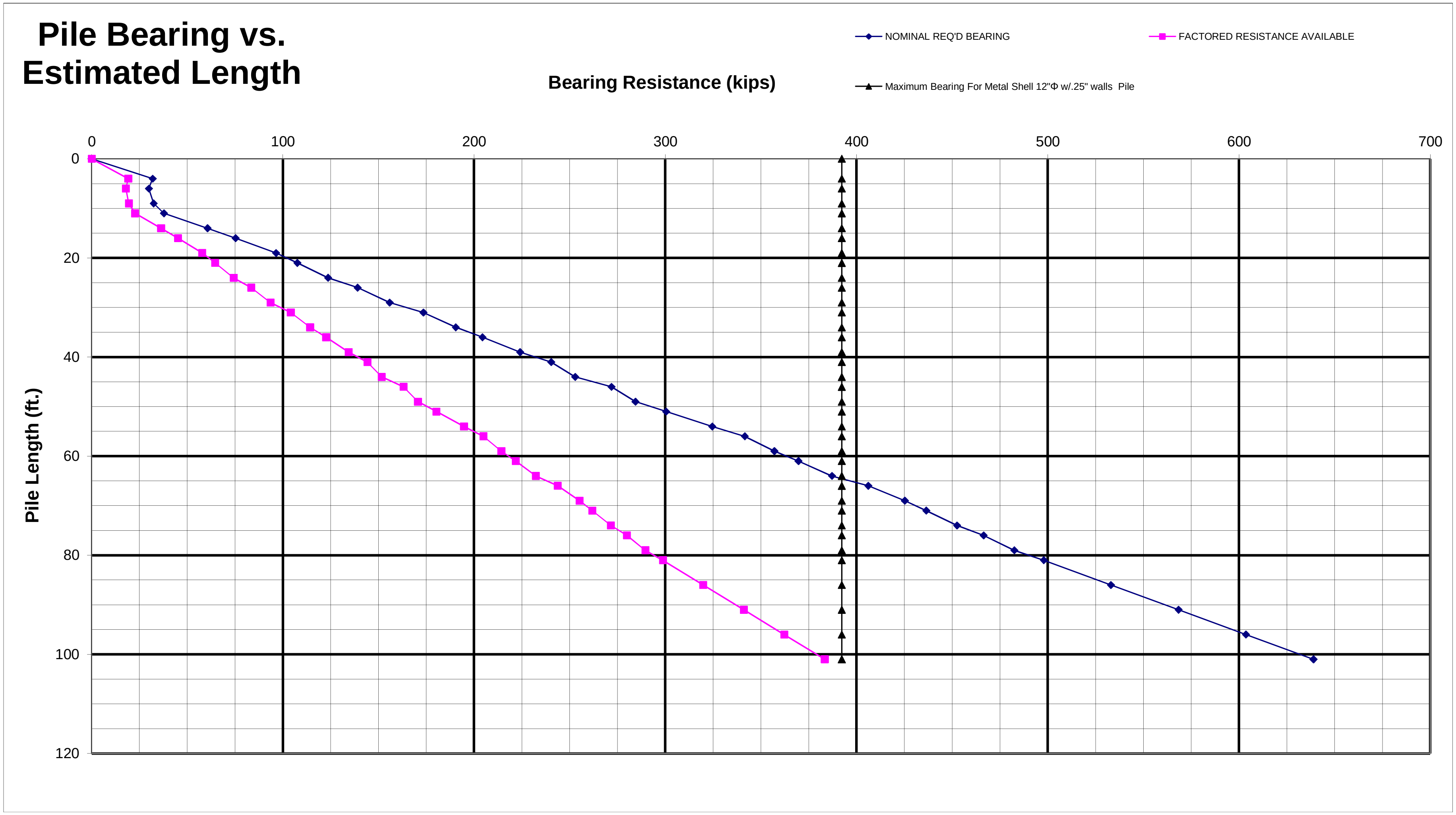
PILE TYPE AND SIZE ========== Metal Shell 12"Φ w/.25" walls
Pile Perimeter=========================== 3.142 FT
Pile End Bearing Area===================== 0.785 SQFT

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 Resist. Available in Boring	Maximum Pile Driveable Length in Boring
392 KIPS	**392** KIPS	**235** KIPS	**66** FT
	Avg. Est.'d Driving Stress		Soil Setup Pile Length
	33.7 KSI		**107** 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***						***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)***	***SOIL SETUP PILE LENGTH (FT)***	***AVERAGE ESTIMATED DRIVING STRESS (KSI)***
					SIDE RESIST. (KIPS)	***END BRG. RESIST. (KIPS)***	***TOTAL RESIST. (KIPS)***										
661.60	2.50	2.50	13		13.9		***31.9***				32	0	0	19	4	14	-
659.10	2.50	2.50	10		13.9	18.0	***29.9***				30	0	0	18	6	14	-
656.60	2.50	0.30	3		2.5	2.2	***32.4***				32	0	0	19	9	14	-
654.10	2.50	0.30	4		2.5	2.2	***37.8***				38	0	0	23	11	16	-
651.60	2.50	0.70	5		5.4	5.1	***60.6***				61	0	0	36	14	21	-
649.10	2.50	3.10	18		16.1	22.4	***75.2***				75	0	0	45	16	26	-
646.60	2.50	2.90	25		15.4	20.9	***96.4***				96	0	0	58	19	31	21.0
644.10	2.50	3.70	23		18.4	26.7	***107.6***				108	0	0	65	21	34	20.8
641.60	2.50	2.70	13		14.6	19.5	***123.7***				124	0	0	74	24	39	22.3
639.10	2.50	2.90	24		15.4	20.9	***139.1***				139	0	0	83	26	44	22.6
636.60	2.50	2.90	21		15.4	20.9	***155.9***				156	0	0	94	29	46	23.4
634.10	2.50	3.10	20		16.1	22.4	***173.5***				173	0	0	104	31	54	24.1
631.60	2.50	3.30	19		16.9	23.8	***190.4***				190	0	0	114	34	56	25.0
629.10	2.50	3.30	17		16.9	23.8	***204.4***				204	0	0	123	36	59	25.4
626.60	2.50	2.90	20		15.4	20.9	***224.1***				224	0	0	134	39	66	26.6
624.10	2.50	3.50	29		17.6	25.3	***240.3***				240	0	0	144	41	69	27.3
621.60	2.50	3.30	28		16.9	23.8	***252.9***				253	0	0	152	44	74	27.8
619.10	2.50	2.70	16		14.6	19.5	***271.8***				272	0	0	163	46	79	28.6
616.60	2.50	3.30	26		16.9	23.8	***284.4***				284	0	0	171	49	81	29.2
614.10	2.50	2.70	20		14.6	19.5	***300.5***				300	0	0	180	51	86	29.7
611.60	2.50	2.90	21		15.4	20.9	***324.5***				325	0	0	195	54	91	31.0
609.10	2.50	4.10	26		19.9	29.6	***341.5***				342	0	0	205	56	96	31.8
606.60	2.50	3.70	21		18.4	26.7	***357.0***				357	0	0	214	59	101	32.5
604.10	2.50	3.30	19		16.9	23.8	***369.6***				370	0	0	222	61	102	33.0
601.60	2.50	2.70	26		14.6	19.5	***387.1***				387	0	0	232	64	106	33.6
599.10	2.50	3.10	19		16.1	22.4	***406.2***				~~406~~	~~0~~	~~0~~	~~244~~	~~66~~	~~111~~	~~34.4~~
596.60	2.50	3.50	25		17.6	25.3	***425.2***				~~425~~	~~0~~	~~0~~	~~255~~	~~69~~	~~115~~	~~34.9~~
594.10	2.50	3.70	18		18.4	26.7	***436.4***				~~436~~	~~0~~	~~0~~	~~262~~	~~71~~	~~118~~	~~35.3~~
591.60	2.50	2.70	22		14.6	19.5	***452.5***				~~453~~	~~0~~	~~0~~	~~272~~	~~74~~	~~122~~	~~36.0~~
589.10	2.50	2.90	21		15.4	20.9	***466.4***				~~466~~	~~0~~	~~0~~	~~280~~	~~76~~	~~125~~	~~36.7~~
586.60	2.50	2.70	19		14.6	19.5	***482.5***				~~483~~	~~0~~	~~0~~	~~290~~	~~79~~	~~129~~	~~36.9~~
584.10	2.50	2.90	16		15.4	20.9	***497.9***				~~498~~	~~0~~	~~0~~	~~299~~	~~81~~	~~133~~	~~37.5~~
579.10	5.00	2.90	16		30.8	20.9	***533.0***				~~533~~	~~0~~	~~0~~	~~320~~	~~86~~	~~142~~	~~37.7~~
574.10	5.00	3.50	23		35.3	25.3	***568.3***				~~568~~	~~0~~	~~0~~	~~341~~	~~91~~	~~151~~	~~37.8~~
569.10	5.00	3.50	23		35.3	25.3	***603.6***				~~604~~	~~0~~	~~0~~	~~362~~	~~96~~	~~159~~	~~38.1~~
564.10	5.00	3.50	23		35.3	25.3	***638.9***				~~639~~	~~0~~	~~0~~	~~383~~	~~101~~	~~168~~	~~38.0~~
554.10	10.00	3.50	23			25.3											

Pile Bearing vs. Estimated Length



PILING
Effective: March __, 2013

This Special Provision amends the following provisions of the Standard Specifications for Road and Bridge Construction.

512.10 Driving Equipment. Revise the first, second and third paragraphs of Article 512.10(a) to read as follows:

(a) Hammers. Piles shall be driven with an impact hammer such as a drop, steam/air, hydraulic, or diesel. The driving system selected by the Contractor shall not result in damage to the pile. The impact hammer shall be capable of being operated at an energy which will maintain a pile penetration rate between 1 and 10 blows per 1 in. (25 mm) when the nominal driven bearing of the pile approaches the nominal required bearing in soil for the end-of-driving condition described in Article 512.14. To avoid potential damage to steel piles driven to rock, the impact hammer shall operate at an energy corresponding to a pile penetration rate between 4 and 20 blows per 1 in. (25 mm) as the pile nears and develops the nominal required bearing in rock.

For hammer selection purposes, the minimum and maximum hammer energy necessary to achieve these penetrations may be estimated as follows.

Soil	Rock
$E \geq \frac{32.9\ R_N}{F_{eff}}$ (English)	$E \geq \frac{28.6\ R_N}{F_{eff}}$ (English)
$E \leq \frac{65.8\ R_N}{F_{eff}}$ (English)	$E \leq \frac{41.1\ R_N}{F_{eff}}$ (English)
$E \geq \frac{10.0\ R_N}{F_{eff}}$ (metric)	$E \geq \frac{8.7\ R_N}{F_{eff}}$ (metric)
$E \leq \frac{20.0\ R_N}{F_{eff}}$ (metric)	$E \leq \frac{12.5\ R_N}{F_{eff}}$ (metric)

Where:

R_N = Nominal required bearing in kips (kN)
E = Energy developed by the hammer per blow in ft-lb (J)
F_{eff} = Hammer efficiency factor according to Article 512.14.

The above hammer options, hammer energy range, and pile penetration rates shall be applicable unless noted otherwise in the construction documents.

512.11 Penetration of Piles. Revise Article 512.11 to read as follows:

Piles shall be installed to a penetration that satisfies all of the following.

(a) The nominal driven bearing, as determined by the formula in Article 512.14, is not less than the nominal required bearing shown on the plans except as permitted below for piles driven to rock.

(b) The pile tip elevation is at or below the minimum tip elevation shown on the plans. In cases where no minimum tip elevation is provided, the piles shall be driven to a penetration of at least 10 ft (3 m) below the bottom of footing or below undisturbed earth, whichever is greater.

Except as required to satisfy minimum tip elevations required in 512.11(b) above, piles not bearing on rock are not required to be driven more than one additional foot (300 mm) after the nominal driven bearing equals or exceeds the nominal required bearing; more than three additional inches (75 mm) after the nominal driven bearing exceeds 110 percent of the nominal required bearing; or more than one additional inch (25 mm) after the nominal driven bearing exceeds 150 percent of the nominal required bearing. For piles driven to rock, pile driving shall be stopped, independent of the nominal driven bearing predicted by the formula in Article 512.14, when the minimum penetration rate is ¼ in. over 5 blows (or equivalently a maximum penetration rate of 20 blows per 1 in. for no more than 5 blows). When piles not bearing or rock fail to achieve nominal driven bearings in excess of the nominal required bearing after driving the full furnished lengths, but are within 85 percent of nominal required bearing, these piles shall be left for a minimum of 24 hours to allow for soil setup and retesting before splicing and driving additional length. After the waiting period has passed, the pile shall be redriven to check the gain in nominal driven bearing upon soil setup. The soil setup nominal driven bearing shall be based on the number of redriving blows necessary to drive the pile an additional 2 in. (75 mm) using a hammer that has been warmed up by applying at least 20 blows to another pile. Within the additional 2 in., the redriving data should be carefully observed and the bearing determined for each ½ in. of pile penetration. In addition to the pile penetration rate, field inspectors are encouraged to carefully monitor the hammer energy during the redrive as increased driving resistance from soil setup may result in greater rebound of the hammer ram and developed hammer energy than experienced during the initial pile driving procedure. The soil setup nominal driven bearing may be taken as the largest value recorded at the ½ in. increments. These piles will be accepted if they exhibit a nominal driven bearing larger than nominal required bearing. In addition, piles within a group, and adjacent to a retested pile that has achieved the nominal required bearing within the additional 2 in. of pile penetration, may be accepted provided the piles exhibited driving behavior similar to the retested pile prior to the setup period. Acceptance of such piles shall be subject to approval of the Engineer and shall require that a minimum of 20 percent of the piles within the group, and no fewer than 2, be retested and achieve the nominal required bearing within the additional 2 in. of pile penetration. Locations of the retested piles should be uniformly scattered across the pile group.

When piles have been driven in excess of the indicated estimated pile length and are not within 85 percent of the nominal required bearing, piles should not be driven longer than the soil setup pile length indicated in the plans. When piles have been driven to this length, they shall be left for a minimum of 48 hours and redriven to check the gain in nominal driven bearing due to soil setup using the above procedure. The Bureau of Bridges and Structures should be contacted for further disposition when piles have not achieved the nominal required bearing upon redrive.

The above mentioned waiting periods for redriving piles to check for gain in nominal driven bearing due to soil setup are minimums and some soil types may exhibit greater soil setup with increased waiting period. When feasible, longer waiting periods that are a function of the soil type at the pile location are encouraged. The following waiting periods are recommended prior to redriving piles to try and maximize the gain in nominal driven bearing due to soil setup:

Recommended Waiting Periods for Redrive Based on Soil Type

Clean Sands = 1 day
Silty Sands = 2 days
Sandy Silts = 4 days
Silts and Clays = 8 days

512.14 Determination of Nominal Driven Bearing. Revise the first paragraph of Article 512.14 to read as follows:

The nominal driven bearing of each pile shall be determined by the WSDOT formula as follows.

$$R_{NDB} = \frac{6.6\ C_s\ F_{eff}\ E\ Ln\ (10N_b)}{1000} \quad \text{(English)}$$

$$R_{NDB} = \frac{21.7\ C_s\ F_{eff}\ E\ Ln\ (10N_b)}{1000} \quad \text{(metric)}$$

Where:

R_{NDB} = Nominal driven bearing of the pile in kips (kN)
C_s = Soil setup correction factor
  1.0 for EOD data
  0.8 for BOR data
N_b = Number of hammer blows per inch (25 mm) of pile penetration
E = Energy developed by the hammer per blow in ft lb (J)
F_{eff} = Hammer efficiency factor taken as:
  0.55 for air/steam hammers
  0.47 for open-ended diesel hammers and steel piles or metal shell piles
  0.37 for open-ended diesel hammers and concrete or timber piles
  0.35 for closed-ended diesel hammers
  0.28 for drop hammers

End-of-driving (EOD) data refers to the information that is collected and analyzed during the initial pile installation procedure. Beginning-of-redrive (BOR) data refers to the redriving information that is collected and analyzed when the pile is driven less than 2 in. following a waiting period to check the gain in nominal driven bearing due to soil setup. When redriving piles, a significant reduction in R_{NDB} is often observed as the pile penetration exceeds 2 in. If the pile does not achieve the required nominal driven bearing within the 2 in. of additional penetration during the redrive, the nominal driven bearing of the pile shall continue to be determined using the WSDOT formula and soil setup correction factor for EOD data after the pile has been driven 4 additional inches.

Per Article 512.10, the hammer chosen by the contractor is required to be capable of developing the nominal required bearing capacity of piles bearing in soil at EOD at an N_b between 1 and 10. When evaluating R_{NDB} of piles bearing in soil for the same hammer using the WSDOT formula and BOR data, the permissible range of N_b is between 1 and 20.

As an alternative to the WSDOT formula, qualified personnel may analyze BOR data using the Wave Equation Analysis of Piles (WEAP) software program. When performing WEAP of BOR data using the Department's geotechnical pile design procedure, piles will only be required to achieve a nominal driven bearing equal to 85% of nominal required bearing indicated in the contract plans.

512.15 Test Piles. Revise the third paragraph of Article 512.15 to read as follows:

Test piles not bearing on rock shall be driven to a nominal driven bearing ten percent greater than the nominal required bearing shown on the plans. The Engineer may stop the driving of any test pile not bearing on rock at tip penetrations exceeding 10 ft (3 m) beyond the estimated length to check for pile setup according to Article 512.11. After any retesting, the Contractor shall recommence test pile driving, providing piling, splices, and any retests until the nominal driven bearing during driving reaches ten percent more than the nominal required bearing or the Engineer stops the driving due to having sufficient data to provide the itemized list of furnished lengths. Test piles bearing on rock shall be driven to the nominal required bearing shown on the plans except pile driving shall be stopped when the pile penetration rate satisfies the criteria indicated in Article 512.11.

1006.05 Metal Piling and Steel Casing. Replace 1006.05(a) and (b) with the following:

(a) Metal Shell Piling. Metal shell piling shall be according to ASTM A 252, Grade 3 except the minimum yield strength shall be 50,000 psi (345,000 kPa).

(b) Steel Piling. Steel piling shall be according to AASHTO M 270, Grade 50 (M 270M, Grade 345).