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Geotechnical Design Memorandum

F.A.I. Route 74
Section 81-1HB
Rock Island County
Job No. P-92-032-01
Contract No. 64C08
PTB No. N/A
I-74 & Ramp 7th-A Over 19th St. Bridges
Structure Nos. 081-0179, 081-0180, and 081-0181

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Prepared for:
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1. Project Description

This memorandum provides geotechnical data and recommendations for the proposed I-74 Over 19th Street and Ramp 7th-A Over 19th Street Bridges, which are part of the Central Section of the I-74 over the Mississippi River Project. The project includes reconstruction of I-74 between 14th Avenue in Moline, Illinois and Lincoln Road in Bettendorf, Iowa. The bridges covered by this geotechnical design memorandum will be replacements for existing structures carrying I-74 over 19th Avenue.

Nearby project features that have an impact on the design or construction of the proposed retaining wall include the north abutment retaining wall (IL-RW06, S.N. 081-6015), the south abutment retaining wall (IL-RW07, S.N. 081-6016), the I-74 roadway, and the 19th Street roadway. Geotechnical recommendations for Retaining Walls IL-RW06 and IL-RW07 are presented in separate geotechnical design memoranda prepared by Hanson Professional Services Inc. (Hanson). Geotechnical recommendations for the interstate and street are contained in a soil survey report prepared by Hanson.

This memorandum supersedes the structure geotechnical reports prepared by Jacobs Civil Inc. in June 2008 and Hanson Professional Services Inc. in June 2012. This memorandum has been prepared to address significant changes to the structure type and project staging.

2. Location

The proposed I-74 and Ramp 7th-A Over 19th Street Bridges are located in the north central portion of Rock Island County, within Sections 32 and 33 of Township 18 North, Range 1 West. They are located at I-74 Sta. 59+67.00. Structure Number 081-0179 carries Westbound (Northbound) I-74, Structure Number 081-0180 carries Eastbound (Southbound) I-74, and Structure Number 081-0181 carries Ramp 7th-A over 19th Street.

3. Existing Structures

The existing structures, S.N. 081-0099 (EB I-74), S.N. 081-0100 (WB I-74), S.N. 081-0115 (Ramp 7-S onto EB I-74) and S.N. 081-0116 (Ramp S-7 off WB I-74), were constructed in 1975. They are six and seven-span plate girder bridges with total lengths of 455 to 666 feet. Span lengths range from 60 feet to 127 feet. All piers are single cylindrical steel columns with welded box cross-girders that frame into the web plates of the longitudinal girders. All four bridges have separate stub abutments on the north end. The eastbound mainline and ramp bridge and the westbound mainline and ramp bridge each share a stub abutment on the south end. Portions of the existing structure plans are included in the Appendix for reference.

Due to the structures' location at the edge of the bluffs, the profile grades are relatively steep and the clearance above 19th Street is unusually high. Minimum clearance is 14'-7" at the north end of Ramp 7-S. Ramp S-7 and the two mainline structures have at least 27'-1" clearance.

The structures are supported on battered 10BP42 piles. The existing structure plans indicate that the piles were to be driven to refusal but do not indicate a design capacity. Based on the estimated lengths shown on the plans, the pile tips are located on bedrock or in very stiff to hard clay (glacial till).

4. Proposed Structures

The currently proposed structures are significantly different from earlier designs. A study (Modjeski and Masters, 2014) was completed to evaluate several alternative structure types conforming to the revised project staging. After coordination with IDOT, a preferred alternative was selected and developed further. General plan and elevation drawings for the proposed structures were prepared in August 2014.

The proposed grade separation consists of three separate three-span bridges supported on straddle column piers and individual stub abutments. Out to out width of the bridges are 63'-2" for the WB mainline, 63'-5" for the EB mainline, and 45'-5" for Ramp 7th-A. All three bridges have 0° skews but the abutment locations are staggered by one span to accommodate the angled crossing. The crossing has a total of five spans with lengths of 134'-0", 134'-0", 110'-0", 110'-0", and 90'-0".

The bridge and wall geometry are configured for a mixed abutments, where the vertical bridge loads are supported by piles passing through the reinforced soil mass. Based on information provided by the structure designer, the approximate factored superstructure loads on the North Abutment bearings are 1,700 kips and 1,400 kips at each mainline and ramp bridge, respectively. Similarly, the loads on the South Abutments are 1,500 kips and 1,200 kips. The MSE walls will be designed to resist the lateral loads applied to the abutments.

Maximum factored loads on the pier columns are approximately 4,400 kips vertical and 175 kips horizontal for strength load cases. Significantly larger extreme event (vehicle collision) loads are expected at the columns.

Similar to the existing structures, the proposed structures are unusually tall. Due to the steep grade of 19th Street and the angle of the abutments relative to 19th Street, the heights from the top of end slope to the pavement surface are highly variable at each of the six abutments. The total heights of the MSE walls and bridge abutments above finished grade of the end slopes range from approximately 10 to 29 feet along the north abutments and 16 to 33 feet along the south abutments.

The proposed bridges will be constructed in stages in order to allow traffic on I-74 and 19th Street throughout the construction period. The Ramp 7th-A and WB I-74 bridges will be constructed in the first stage while maintaining I-74 traffic on the existing EB bridge. The new EB I-74 Bridge will be constructed during the second stage with I-74 traffic on the new WB bridge. The new substructures will generally be constructed sequentially from north to south with multiple lanes shifts along 19th Street. Traffic will be diverted onto temporary pavement located to the south of the current alignment. This will require partial excavation of the existing bridges' end slopes.

Temporary MSE walls will be required at stage lines located at the north end of the WB North Abutment Wingwall and at the west end of the WB South Abutment. Geotechnical recommendations for these structures are provided in the geotechnical design memoranda for the retaining walls.

5. Site Investigation

The project site is located in the steeply sloping terrain of the bluffs along the Mississippi River. 19th Street is situated in a natural ravine. There was extensive grading of the proposed bridge site during construction of the existing I-74 alignment. Along the current I-74 centerline, the base of the ravine once was between approximately Sta. 58+00 and Sta. 63+50. 19th Street was in the area where the current bridges' north abutment end slopes are located today. The existing bridges' north abutments generally were constructed on the existing hillside at or near the natural grade. The existing bridges' south abutments were constructed on more than 40 feet of fill placed when the highway was constructed.

South of 19th Street, the profile of existing I-74 is split, with the eastbound lanes being approximately 5 feet higher than the westbound lanes. The EB and WB profiles come together just to the north of the existing bridges. The height from the toe of the bridge end slopes to the roadway grade is approximately 25 feet on the north side of 19th Street and 45 feet on the south side. The end slope of the existing EB I-74 and Ramp 7-S bridges' shared south abutment is split into two roughly equal height tiers. Many of the existing bridge piers are located on the end slopes. Presently, 19th Street slopes down to the northwest at approximately 3% grade, while I-74 slopes down to the north at approximately 3% to 6% grade.

Test boring data was shown on the existing structure plans. It is presumed that these borings were drilled in the early 1970's. Fifteen borings were drilled to depths between 30 and 79 feet below grade. Standard penetration tests were generally performed at 2.5-feet intervals until bedrock was encountered. Several of these borings were drilled near the substructure units of the proposed bridges. Although the soil strata logged in the upper part of these borings were disturbed by the original I-74 roadway and bridge construction, the data for the lower strata are useful for design of the new bridges.

The field exploration that was completed specifically for the proposed structures was accomplished in five phases. The first two phases were completed in December 2005 and October 2007 to March 2008 by other consultants. IDOT provided the data collected from those two phases, logs for the borings drilled were provided to Hanson in May 2014. The third phase was completed in June 2010 by Hanson. The primary purpose of the third phase was to collect additional samples of the shallow, softer soils for strength and consolidation testing. The fourth phase was completed by IDOT during February to April 2011. The fifth phase was completed in June 2014 by Hanson. The purpose of the fifth phase was to gather additional data near revised pier and abutment locations. A representative from Hanson logged the borings and performed a general site reconnaissance during the third and fifth phases.

Ten borings were drilled in the first two phases, one boring was drilled in the third phase, nine borings were drilled in the fourth phase and four borings were drilled in the fifth phase. Locations of the borings were selected to avoid the numerous obstructions currently occupying the site. The maximum spacing between borings was approximately 125 feet. Standard Penetration Test (SPT) samples were collected at 2.5 to 5.0 feet intervals in all borings. Several Shelby tube samples were collected at representative locations in cohesive strata. The boring depths ranged from 25.0 to 90.0 feet.

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 first, second and fourth phase borings were tested by others. Unconfined strength and moisture content tests were completed on split-spoon samples from approximately two-thirds of the borings. Index testing was completed on representative samples. Unconfined strength tests were performed on several representative samples collected with Shelby tubes.

The soil samples obtained from the third and fifth phase borings were delivered to Hanson's soils laboratory and subjected to a testing program. Natural moisture content and visual classification tests were competed on all samples. Unconfined compressive strength tests, using a Rimac spring tester, were also completed when possible. Triaxial strength tests and consolidation tests were performed on designated Shelby tube samples.

The locations of the index tests, triaxial tests, and consolidation tests are indicated on the subsurface data profile.

7. Subsurface Profile

A subsurface data profile is presented in the Appendix for use by the structure designer. The data profile includes all of the borings that were recently drilled near the proposed structure. Borings that were drilled prior to the construction of the existing structures are also included in areas where more recent subsurface data is not available.

The subsurface profile consists of deposits of fill material, alluvial soils, and glacial till overlying bedrock. The fill is generally located in the approach embankments on both sides of the existing structures. Alluvial soils are found at shallow depths beneath 19th Street and to the southwest. Glacial till and bedrock are present at depth over the entire site. Strata elevations and depth were quite variable due to the site's location at the base of the bluff and the significant grading completed during construction of the existing structures.

Bedrock was encountered in all of the deeper borings. The bedrock surface is erratic, varying between Elev. 557.8 and Elev. 589.8, but generally sloping down to the northwest. Hard (for soil), greenish gray to black clay shale was encountered in the northwestern portion of the site, while hard (for rock), fractured, gray limestone was encountered to the southeast. In the two borings where both strata were present, the clay shale overlies the limestone. The clay shale has an average unconfined strength of 5.6 tsf with very good rock mass quality. The limestone has an average unconfined strength of 500 tsf with fair to good rock mass quality. In two borings to the southeast, a tan sandstone layer was observed above the limestone. No tests were performed on the sandstone due to poor sample quality.

Glacial till was encountered in all of the borings except ILR0804, which did not penetrate the existing fill. The top of this stratum was encountered between Elev. 617.3 and Elev. 588.8. It is typically brown to gray, very stiff to hard, silty clay with sand and gravel. Unconfined strengths generally were between 2.5 and 3.5 tsf, although softer, weathered zones were occasionally encountered near the top. Standard Penetration Test (SPT) values were typically between 12 and 20 blows per foot. Natural moisture contents ranged from 11 to 22 percent and averaged approximately 14 percent. Thin sand seams were encountered in a few locations within the otherwise clayey till.

Alluvial soils were usually encountered between Elev. 592.0 and Elev. 622.2. These soils were typically brown to gray, medium stiff to stiff, silty clays or loose sands. Unconfined strengths were 0.4 to 1.9 tsf, with an average of 0.8 tsf. SPT values were typically 3 to 5 blows per foot. Natural moisture contents ranged from 12 to 27 percent. The alluvial soils were encountered in the older borings drilled under the current south approach embankment, but these softer soils were not readily apparent in the more recent borings drilled in the same area. It is possible that the alluvial soils were removed during construction of the existing embankments. It is also possible that those softer soils have been compressed by the more than 30 feet of existing fill.

An 8 to 44 feet thick layer of fill was encountered in the borings drilled through the existing embankments. It extended from the ground surface to the top of the till or alluvium. The fill material was typically brown to gray, stiff to very stiff, sandy clay or silty clay with very small quantities of random debris.

The groundwater conditions encountered in the borings were not consistent across the site. The groundwater elevations recorded on the boring logs are summarized in Table 7.1. Stabilized readings were not taken in any of the borings. The groundwater, where it was encountered, was typically located near the top of the till stratum or in a sand layer within the till, which could indicate localized, perched conditions. For comparison, the water level in the Mississippi River, approximately 0.7 miles to the north of the site, is usually about Elev. 561.0.

Table 7.1 Groundwater Elevations

Boring No.	During Drilling	At End of Boring	24-hour Reading
19BR-104	Dry	-	-
19BR-105	580.3	-	-
19BR-106	Dry	-	-
19BR-107	-	-	-
19BR-108	Dry	-	-
19BR-109	595.8	-	-
B-1 (2011)	-	-	-
B-2 (2011)	Dry	Dry	590.8
B-5 (2011)	568.1	-	-
ILR0701	581.3	-	-
ILR0801	-	-	-
ILR0804	-	-	-
RW 06-1	593.8	-	-
RW 06-04	Dry	-	-
RW 06-05	Dry	-	-
RW 07-02	Dry	-	-
RW 07-03	Dry	-	-

The Illinois State Geological Survey Directory of Coal Mines does not list any mines immediately beneath the site; however, the directory does indicate that past mining has occurred in the general vicinity. Shafts for the Zeigler, Poston, and Highland Mines were located approximately 1.5 miles to the southeast of the site. These room and pillar mines were operated in the early 1900's.

8. Geotechnical Evaluations

Slope stability analyses of the abutment end slopes were completed as part of the geotechnical evaluations of Retaining Walls IL-RW06 and IL-RW07. The slopes on the north side of 19th Street meet AASHTO stability requirements without any further treatment. Sections cut through the taller portions of the MSE walls on the south side of 19th Street have factors of safety less than 1.50 and would be considered deficient according to AASHTO requirements. The deficient areas will require treatment of the soft layer underlying the MSE walls. All abutments will meet AASHTO requirements for slope stability if the aggregate column ground improvement (ACGI) recommendations in the retaining wall GDM's are followed.

Estimated settlements vary significantly because of the variable subsurface conditions and the wide range of fill heights across each abutment. The more compressible soils and taller fill heights are found beneath the end of each abutment that is closer to 19th Street. The estimated settlements at each of the three south abutments vary from a maximum of approximately 1 inch located at the west end to less than ½ inch at the east end. The estimated settlements at each of the three north abutments vary from a maximum of approximately 6 inches at the east end to less than 1 inch at the west end. The larger settlements result from the softer alluvial soils that are found near 19th Street. These alluvial soils will consolidate rather quickly, especially when the ACGI that is required in these areas is also considered. Less than 1 inch of the total estimated settlement is expected to occur after the construction period. The retaining wall GDM's include further discussion of the estimated settlements.

Some differential settlement is anticipated near the proposed stage lines. Theoretically, the subgrade soils within approximately 5' of the edge of a stage will consolidate 25% to 33% less than the central portion. When the adjacent stage is placed, the edge of the previous stage will settle to a level approximately equal to the central portion. This would affect pavement constructed on top of the first stage and may be visible in the panel joints on the face of the MSE wall. It could also open some small gaps between the base of the pile-supported abutment cap and the underlying fill. Due to the relatively small settlement magnitudes near the stage lines, this is not expected to be a significant concern for these structures.

9. Design Recommendations

The proposed stub abutments and straddle pier columns should be supported on piles driven to the shale, sandstone, or limestone bedrock. Footings and drilled shafts would not be cost effective for these substructures. Tables 9.1 and 9.2 list design parameters for several pile sizes. The subsurface conditions are variable and borings are not located at each of the current substructure locations. Estimated pile lengths and capacities were calculated from the most conservative nearby boring(s) as indicated in the tables.

Settlement of the softer alluvial soils between the bottom of the retaining wall and the glacial till will result in drag loads. The geotechnical losses shown in Table 9.1 are the result of drag loads and losses within the alluvial soil and any existing soil layers above it. The bottom of the soil layer that causes the drag losses varies widely across the abutments. The lowest elevation at which it is present varies from Elev. 593.5 to 596.0 at the south abutments and from Elev. 595.5 to 598.0 at the north abutments. The drag-inducing layer is not present in some areas, generally located farthest from 19th Street. Drag loads on the portion of the piles embedded in the reinforced soil mass would be substantially larger. To avoid these significant additional losses, the piles should be isolated from the select fill by the use of oversized sleeves. The sleeves should be sized to provide at least 1.5 inches of clearance around the pile and should extend from the bottom of the abutment to the bottom of reinforced soil mass, base of ACGI working platform, or base of fill, whichever is lower.

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 resistance factors used in the project-specific pile design procedure vary depending on the type of bearing strata. H-piles are generally expected to drive to limestone bedrock, which has the highest resistance factor, at the Ramp 7th-A South Abutment, the east column of Pier 4 and the west column of Pier 3. The piles at all other substructures are expected to bear in soft shale or glacial till, or some combination of these strata and limestone.

Table 9.1 Pile Design Parameters

Location	Cutoff Elevation (ft)	Pile Type	Factored Resistance Available, R_F (kips)	Geotechnical Losses, R_{Sdd} (kips)	Nominal Required Bearing, R_N (kips)	Estimated Pile Length (ft)
081-0180 (EB) North Abutment RW07-02	623.1	HP 10x42	52	10	104	51
			112	10	187	62
			252	10	403	68
		HP 12x63	377	12	598	70
		HP 12x74	449	12	709	72
081-0179 (WB) North Abutment S-37 & RW07-03	626.7	HP 12x63	351	38	598	72
		HP 12x74	423	38	709	75
081-0181 (7th-A) North Abutment S-38 & ILR0801	629.3	HP 10x42	55	0	91	24
			110	0	184	51
			188	0	289	69
			262	0	403	72
		HP 12x63	359	30	598	76
081-0180 (EB) Pier 1 East Column 19BR-104	599.8	HP 14x89	552	0	848	45
		HP 14x102	634	0	975	47
		HP 14x117	727	0	1118	49
		HP 16x141	881	0	1355	50
081-0180 (EB) Pier 1 West Column 19BR-104	599.8	HP 14x89	552	0	848	45
		HP 14x102	634	0	975	47
		HP 14x117	727	0	1118	49
		HP 16x141	881	0	1355	50
081-0179 (WB) Pier 2 East Column S-38	602.7	HP 14x89	552	0	848	49
		HP 14x102	634	0	975	51
		HP 14x117	727	0	1118	53
		HP 16x141	881	0	1355	55
081-0179 (WB) Pier 2 Center Column S-39	602.6	HP 14x89	552	0	848	46
		HP 14x102	634	0	975	48
		HP 14x117	727	0	1118	50
		HP 16x141	881	0	1355	51
081-0180 (EB) Pier 2 West Column 19BR-107	603.0	HP 14x89	552	0	848	50
		HP 14x102	634	0	975	52
		HP 14x117	727	0	1118	54
		HP 16x141	881	0	1355	56
081-0181 (7th-A) Pier 3 East Column S-41	605.0	HP 14x89	552	0	848	33
		HP 14x102	634	0	975	35
		HP 14x117	727	0	1118	37
		HP 16x141	881	0	1355	39

Location	Cutoff Elevation (ft)	Pile Type	Factored Resistance Available, R_F (kips)	Geotechnical Losses, R_{Sdd} (kips)	Nominal Required Bearing, R_N (kips)	Estimated Pile Length (ft)
081-0179 (WB) Pier 3 Center Column S-42	605.1	HP 14x89	552	0	848	32
		HP 14x102	634	0	975	34
		HP 14x117	727	0	1118	36
		HP 16x141	881	0	1355	38
081-0179 (WB) Pier 3 West Column 19BR-108	604.1	HP 14x89	594	0	848	35
		HP 14x102	682	0	975	36
		HP 14x117	783	0	1118	37
		HP 16x141	949	0	1355	37
081-0181 (7th-A) Pier 4 East Column B-1 (2011)	607.1	HP 14x89	594	0	848	25
		HP 14x102	682	0	975	26
		HP 14x117	783	0	1118	27
		HP 16x141	949	0	1355	27
081-0181 (7th-A) Pier 4 West Column S-42	606.2	HP 14x89	552	0	848	33
		HP 14x102	634	0	975	35
		HP 14x117	727	0	1118	37
		HP 16x141	881	0	1355	39
081-0180 (EB) South Abutment S-40 & RW06-04	638.6	HP 10x42	49	0	82	24
			160	0	266	65
			205	0	342	75
			262	0	403	80
		HP12x63	328	61	598	82
		HP12x74	400	61	709	83
081-0179 (WB) South Abutment S-43 & RW06-05	639.1	HP12x63	334	55	598	69
		HP12x74	405	56	709	69
081-0181 (7th-A) South Abutment 19BR-109	640.2	HP 10x42	103	63	238	59
			219	63	403	61
		HP12x63	342	77	598	62

Notes: 1. Where a range of values is shown, pile lengths and capacities may be interpolated between the values given.
2. The pile lengths and capacities for HP 10x42 piles have been determined for the mask wall locations only. Values given for the larger pile sizes are representative of the worst case along each abutment.

Table 9.2 Pile Uplift Design Parameters

Location	Cutoff Elevation (ft)	Pile Type	Factored Uplift Resistance, R_{FUP} (kips)		Nominal Uplift Capacity, R_S (kips)	Pile Length (ft)
			Strength	Ext. Event		
081-0180 (EB) Pier 1 East Column 19BR-104	599.8	HP 14x89	28	112	140	34
			46	183	228	45
		HP 14x102	28	114	142	34
			49	197	246	47
		HP 14x117	29	115	143	34
			53	211	263	49
		HP 16x141	32	126	158	34
			60	241	302	50
081-0180 (EB) Pier 1 West Column 19BR-104	599.8	HP 14x89	28	112	140	34
			46	183	228	45
		HP 14x102	28	114	142	34
			49	197	246	47
		HP 14x117	29	115	143	34
			53	211	263	49
		HP 16x141	32	126	158	34
			60	241	302	50
081-0179 (WB) Pier 2 East Column S-38	602.7	HP 14x89	31	125	156	39
			47	189	236	49
		HP 14x102	32	126	158	39
			51	203	254	51
		HP 14x117	32	127	159	39
			54	217	271	53
		HP 16x141	35	140	175	39
			64	255	319	55
081-0179 (WB) Pier 2 Center Column S-39	602.6	HP 14x89	17	66	83	34
			36	143	179	46
		HP 14x102	17	67	83	34
			39	156	195	48
		HP 14x117	17	67	84	34
			42	170	212	50
		HP 16x141	19	74	93	34
			49	197	246	51
081-0180 (EB) Pier 2 West Column 19BR-107	603.0	HP 14x89	37	147	183	40
			53	211	263	50
		HP 14x102	37	148	185	40
			56	225	281	52
		HP 14x117	37	150	187	40
			60	239	299	54
		HP 16x141	41	165	206	40
			70	280	350	56

Location	Cutoff Elevation (ft)	Pile Type	Factored Uplift Resistance, R_{FUP} (kips)		Nominal Uplift Capacity, R_S (kips)	Pile Length (ft)
			Strength	Ext. Event		
081-0181 (7th-A) Pier 3 East Column S-41	605.0	HP 14x89	27	108	135	23
			43	172	215	33
		HP 14x102	27	109	136	23
			46	186	232	35
		HP 14x117	28	110	138	23
			50	200	250	37
		HP 16x141	30	121	152	23
			59	237	296	39
081-0179 (WB) Pier 3 Center Column S-42	605.1	HP 14x89	24	94	118	21
			41	165	206	32
		HP 14x102	24	95	119	21
			45	178	223	34
		HP 14x117	24	96	120	21
			48	192	240	36
		HP 16x141	26	106	132	21
			57	228	285	38
081-0179 (WB) Pier 3 West Column 19BR-108	604.1	HP 14x89	25	101	126	30
			33	133	166	35
		HP 14x102	25	102	127	30
			35	140	175	36
		HP 14x117	26	103	129	30
			37	148	185	37
		HP 16x141	28	113	141	30
			41	163	204	37
081-0181 (7th-A) Pier 4 East Column B-1 (2011)	607.1	HP 14x89	17	69	87	20
			25	101	127	25
		HP 14x102	18	70	88	20
			27	108	136	26
		HP 14x117	18	71	88	20
			29	116	144	27
		HP 16x141	19	78	97	20
			32	128	160	27
081-0181 (7th-A) Pier 4 West Column S-42	606.2	HP 14x89	25	99	124	22
			42	170	212	33
		HP 14x102	24	98	122	22
			45	181	226	35
		HP 14x117	25	99	124	22
			49	195	244	37
		HP 16x141	27	109	136	22
			58	231	289	39

A test pile should be driven at the west end of the North Abutment and the east end of the South Abutment of S.N. 081-0179, near the center of the South Abutment of S.N. 081-0180, and at Pier 2 Center Column. Four test piles are recommended. Pile shoes are not recommended for any of the piles, because the shoes would reduce uplift capacity and uplift controls design of the piles supporting the piers.

Table 9.3 Top of Strata Elevations for Foundation Design

Substructure	Existing Fill	Alluvium	Glacial Till	Clay Shale	Limestone
Pier 1 East Column	*	600	594	568	
Pier 1 West Column	*	595	590	568	
Pier 2 East Column	*	604	592	577	
Pier 2 Center Column	*	604	592	582	
Pier 2 West Column	*		598	571	
Pier 3 East Column			*	583	
Pier 3 Center Column	*	605	595	590	
Pier 3 West Column	*	596	589	578	574
Pier 4 East Column	*		608		586
Pier 4 West Column	*	608	600	590	

* Layer extends to existing ground surface

It is anticipated that the lateral resistance for the bridge piers will be provided by lateral loading of the vertical piles. 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 9.4. The top elevations of the existing fill, alluvium, glacial till, clay shale, and limestone strata are provided in Table 9.3. The analyses should consider factored axial and factored lateral loads on the foundations. The P-multipliers in AASHTO Table 10.7.2.4-1 should be used in the analyses.

Table 9.4 LPILE Parameters

Stratum	LPILE Soil Type	Soil Parameters			
Existing Fill	stiff clay w/o water	c=12.5 psi	k=500 pci	γ'=0.072 pci	ε_{50}=0.007
Alluvium	soft clay	c=5.9 psi	k=100 pci	γ'=0.069 pci	ε_{50}=0.010
Glacial Till	stiff clay w/o water	c=19.4 psi	k=500 pci	γ'=0.072 pci	ε_{50}=0.005
Clay Shale	stiff clay w/o water	c=38.9 psi	k=2000 pci	γ'=0.078 pci	ε_{50}=0.004
Limestone	strong rock	q_u=6900 psi		γ'=0.048 pci	

The abutment piles should be assumed to provide no lateral resistance. All lateral loads applied to the abutment should be resisted by soil reinforcement attached to the abutment cap. The estimated lateral forces applied by the superstructure and by the backfill should be shown on the plans so that the MSE supplier can design the reinforcement.

The bridge is located in a region of relatively low seismic loading. The subsurface profile to a depth of 100 feet consists of up to 40 feet of soft to stiff clay, overlying very stiff to hard clay and shale bedrock. This profile is indicative of Site Class C. Seismic design parameters for a 1,000-year return period earthquake are listed in Table 9.5. 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 9.5 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 supports should be according to the current IDOT standard. The approach footings will bear on compacted select fill or embankment material. No special subgrade treatment is required.

10. Construction Considerations

The proposed bridge site is located in an area that was developed prior to the construction of I-74. Remnants of old building or other miscellaneous debris may be present under the existing embankments because typical construction specifications do not require complete removal. The oversized sleeves to be used at the abutment provide little room for adjustment if the piles cannot be driven at their plan locations. For this reason, the pile design parameters provided in Table 9.1 assume that the piles are driven prior to placing the reinforced soil mass. The piles could be driven through the sleeves after the retaining walls are constructed, which would eliminate all geotechnical losses but also increase the construction risk.

All four pier columns to be located in the 19th Street median as well as two of the columns located on the south side of 19th Street are expected to require excavation very close to travel lanes. Temporary sheet piling is feasible at these locations. The Bridge Manual's Design Guide 3.13.1 – Temporary Sheet Piling Design should be used for design. The soil strengths and top of strata elevations listed in Tables 9.3 and 9.4 may be used in lieu of boring data, which do not exist at all of these locations. Guide Bridge Special Provision No. 32, Temporary Sheet Piling (Revised: January 1, 2012), should be included in the construction documents.

Temporary shoring is also anticipated in front of 1:2 existing slopes located on the east side of the East Column of Pier 3 and the north side of existing I-74 EB Pier 41. Design Guide 3.13.1 is not applicable to these locations due to the sloping ground, but temporary sheet piling is still feasible at both locations. If temporary sheet piling is specified, it should be designed using an active earth pressure coefficient of 0.52 and a soil unit weight of 130 pcf. The passive resistance should be based on a soil cohesive strength of 2,900 psf at Pier 3, which is representative of the soils found at the toe of the bluff to the east of the bridge. Average strengths from Boring RW06-04 should be used to calculate passive resistance at Pier 41. These are all nominal values that must be factored for design. If a Temporary Soil Retention System is specified, Guide Bridge Special Provision No. 44, Temporary Soil Retention System (Revised: May 11, 2009) should be included in the construction documents.

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

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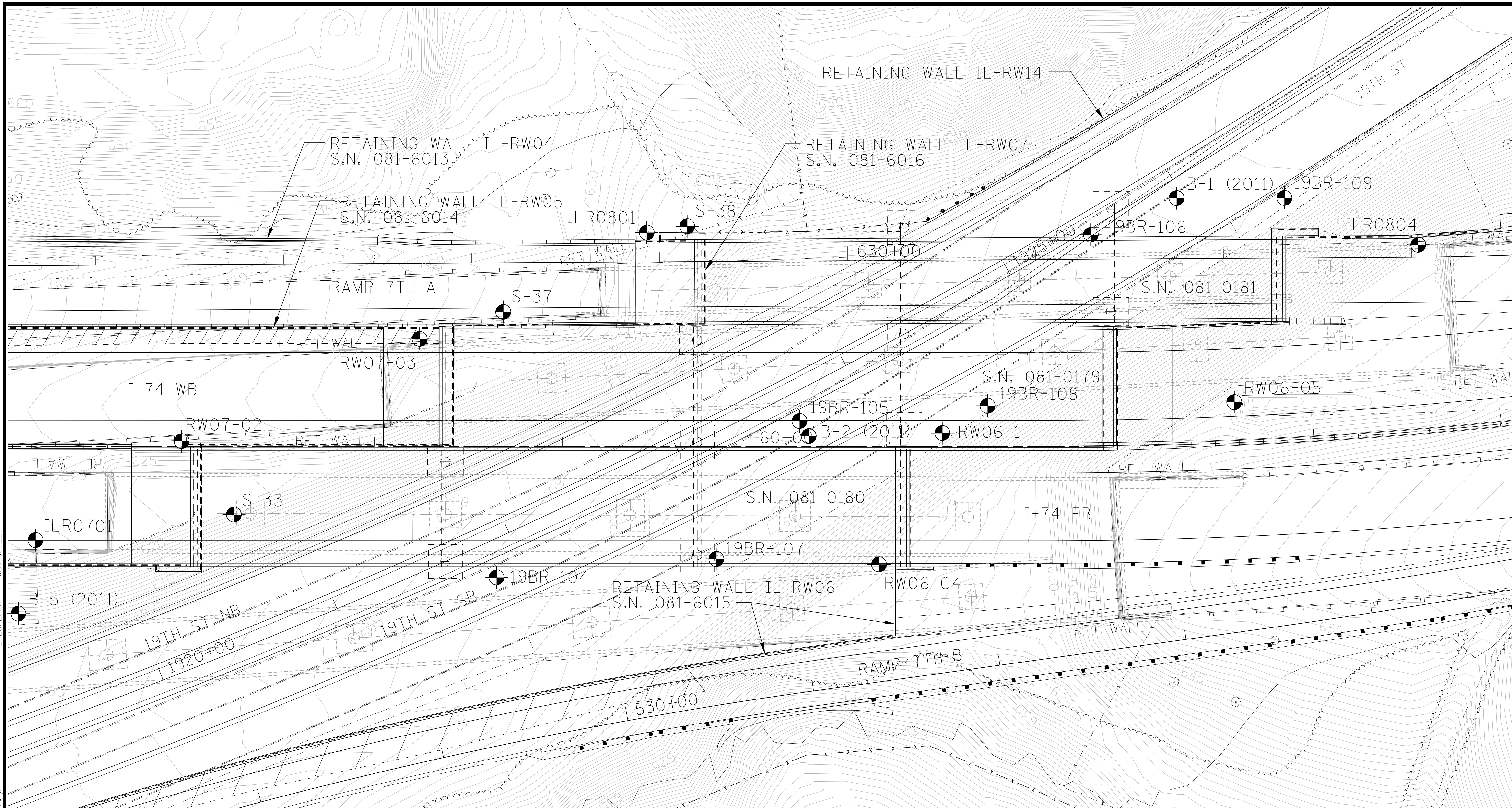
Appendix

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



HANSON

BORING LOCATION PLAN

I-74 & RAMP 7TH-A OVER 19TH STREET
S.N. 081-0179, 0181-0180 & 081-0181
ROCK ISLAND COUNTY, ILLINOIS

08H0120E 2/20/15

081079 A0326 000 Subsurface Data.dgn 2/20/2015 madau00223

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

B-5(2011)
Sta. 56+11, 89' RT

Elev.	N	Qu	w%	Description
613.10		0.5P	13	MEDIUM light brown SILTY CLAY LOAM
	8	0.5P	15	MEDIUM light brown SILTY CLAY LOAM
	7	1.2B	17	STIFF gray/brown SILTY CLAY LOAM
	5	0.6P	20	MEDIUM gray SILTY CLAY LOAM
	13	5.4B	12	HARD tan CLAY LOAM
	9	1.1S	15	STIFF gray SILTY LOAM
	12	2.0B	20	STIFF brown SILTY CLAY LOAM
	8	0.8P	21	MEDIUM gray SILTY CLAY LOAM
593.60	13	2.7B	16	VERY STIFF tan CLAY LOAM TILL
	15	2.5B	15	VERY STIFF tan CLAY LOAM TILL
	14	2.7B	15	VERY STIFF tan/gray CLAY LOAM TILL
	15	2.5B	15	VERY STIFF gray CLAY LOAM TILL
	15	2.5B	15	VERY STIFF gray CLAY LOAM TILL
	16	2.1B	15	VERY STIFF gray CLAY LOAM TILL
	14	2.5B	16	VERY STIFF gray CLAY LOAM TILL
	26	5.4B	18	HARD gray CLAY LOAM TILL
	23	5.7B	18	HARD gray CLAY LOAM TILL
569.10 ▽ DD	21	3.1B	18	VERY STIFF gray CLAY LOAM TILL with SILTY SAND lens
568.10	12			MEDIUM gray clean medium coarse SAND
566.10	16	4.0P	12	MEDIUM gray clean medium coarse SAND with CLAY lens
563.60				VERY DENSE gray weathered SHALE with COAL lens
	100/8" 100/1"			Wash - VERY DENSE olive-green SANDSTONE with DOLOMITE fragments - Auger Refusal @ 52.5'
560.60				Bottom of hole = 52.5 feet

ILR0701
Sta. 56+20, 50' RT

Elev.	N	Qu	w%	Description
629.30				7" Thick ACC followed by gravel subbase to 1.0'
628.70	12			Silty Sandy Clay with Gravel, greenish brown, moist, low plasticity, stiff, with subangular to subrounded gravel embedded throughout, fill/subbase
625.30	9 12	3.0P to 4.0P		Sandy Clay Trace Gravel, dark gray, frozen, stiff, with subangular to subrounded fine to coarse gravel embedded throughout, fill
621.30	6	2.0P	15.5	Silty Clay with gravel, gray, moist, soft to medium stiff, high plasticity, trace gravel, possible fill (LL=38 PI=14)
		1.5P		(LL=32 PI=14)
615.80	5	2.0P	16.0	Sandy Lean Clay Trace Gravel, gray, moist, stiff, medium plasticity, fill or disturbed till (LL=30 PI=14)
610.80	11			Same As Above, turning grayish brown at bottom 3", piece of wood embedded, possible fill
605.80	12	3.0P		Sandy Lean Clay Trace Gravel, brown, moist, stiff, low plasticity, possible weathered till
600.80	15	2.5P to 3.5P	15.0	Same As Above, gray, then brown, split in almost vertical with reddish brown surface, weathered till
595.80	12	2.5P to 3.0P		Sandy Lean Clay Trace Gravel, gray, moist, stiff, low plasticity, unweathered till
	15	2.5P		
DD ▽ 581.30				
580.80	28			Top 3" is same as above; Bottom 12" is Poorly Graded Sand, gray, wet, medium dense, fine to medium sand seam followed by 3" of gray sandy lean clay, trace gravel, till
579.30				Bottom of hole = 50.0 feet

S-33
Sta. 57+25, 36' RT

Elev.	N	Qu	w%	Description
601.3				Silty Clay Loam with Gravel (Till), brown to gray, very stiff
	14	2.9B	14	
	13	2.75B	14	
	14	2.5B	15	
	16	2.4B	14	
	16	2.80B	15	
	15	2.75B	15	
	15	2.7	14	
			14	
	18	2.3B	14	
574.3	23	2.4B 4.5B	15	Clay with Gravel (Till), gray, hard
	31	5.8B	17	
568.8	25	5.0E		Silty Sand
565.8	14			
	100+	7.5E	11	
	100+	7.6S	10	
	100+	7.5S	9	
556.3	100+		8	Bottom of hole = 45.0 feet

LEGEND

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

Note: Borings S-33 and S-37 were drilled prior to construction of the existing bridge. Elevations have been adjusted to current datum.

SUBSURFACE DATA PROFILE
STRUCTURE NO. 081-0179 (WB)
STRUCTURE NO. 081-0180 (EB)
STRUCTURE NO. 081-0181 (7TH-A)

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.
08H0120E		74	81-1-1HB	ROCK ISLAND	-	
DATE 2/20/15	7 SHEETS			CONTRACT NO. 64C08		
		FED. ROAD DIST. NO.	ILLINOIS	FED. AID PROJECT		

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

RW07-02
Sta. 56+98, 3' LT

Elev.	N	Qu	w%	Description
631.20				ASPHALT.
630.70	5	0.50P	15	FILL - Brown to light brown clayey SILT, trace gravel, trace sand.
	14	4.50P	10	
	23		14	- sand seam @ 7.0'.
622.20	12	3.00P	13	Brown and gray silty lean CLAY, trace sand, trace gravel.
		1.75B	14	
		3.88B	13	
		1.84B	14	
	14	2.70P	14	
615.20	19	4.30P	14	Gray moist, very stiff, silty lean CLAY, with trace sand and gravel.
	17	3.30P	15	
	15	2.70P	15	
	15	3.00P	14	
	12	1.70P	16	
	15	2.20P	16	
	19	2.30P	15	
	20	3.30P	14	
	19	2.70P	14	
	28	2.30P	14	
	54	3.30P	19	- coarse sand seam @ 64.3 to 65.0'.
563.70	50/5"	4.50P	14	Gray SHALE.
561.20				Bottom of hole = 70.0 feet

RW07-03
Sta. 58+24, 57' LT

Elev.	N	Qu	w%	Description
629.10				TOPSOIL.
628.85	13	4.50P	12	FILL - Brown silty lean CLAY, trace sand, trace gravel, with limestone fragments.
	11	3.70P	11	
	13			
620.60		1.75B	14	Brown silty lean CLAY, little sand, trace small gravel.
		1.90B		
	5	3.70P	13	
			18	
		1.90B	17	
	18	4.65S	13	
	16	3.69B	12	
	16	3.10B	14	
603.10	16	4.07B	14	Gray, moist, very stiff, silty lean CLAY, with trace sand and trace gravel.
	19	3.88B	13	
594.10				Bottom of hole = 35.0 feet

S-37
Sta. 58+69, 72' LT

Elev.	N	Qu	w%	Description
621.8				Clay Loam, brown
617.3		2.9S	11	Clay Loam, brown, very stiff
	14	2.8B	12	
	16	3.5S	15	
	13	2.7B	11	
	13	2.9B	14	
	13	2.8B	14	
	11	2.6B	14	
	9	2.3B	13	
597.3	23			Sandy Loam, brown, medium
595.8 ▽ DD				
594.8	13	1.8B	11	Clay Loam, gray, stiff
592.3	10	2.3B	14	Clay Till, gray, very stiff
	10	2.6B	14	
	14	2.9B	13	
584.8	14	4.4B	15	Clay Till, gray, hard
	18	4.3B	15	
	18	4.4B	19	
	23	5.9B	11	
574.8	16	3.3B	16	Clay, brown-gray, very stiff
	18	4.6B	12	
569.8	50	6.5B	10	Clayey Shale, gray, hard
	56	6.1S	11	
	66	6.0S	10	
	150	6.5S	9	
560.3				Bottom of hole = 61.5 feet

LEGEND

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

Note: Boring S-38 was drilled prior to construction of the existing bridge. Elevations have been adjusted to current datum.

SUBSURFACE DATA PROFILE
STRUCTURE NO. 081-0179 (WB)
STRUCTURE NO. 081-0180 (EB)
STRUCTURE NO. 081-0181 (7TH-A)

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.
08H0120E		74	81-1-1HB	ROCK ISLAND	_	
DATE 2/20/15	7 SHEETS	CONTRACT NO. 64C08				
		FED. ROAD DIST. NO.	ILLINOIS	FED. AID PROJECT		

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

ILR0801
Sta. 631+07, 16' RT

Elev.	N	Qu	w%	Description
623.02				Grass Matter - followed by silty clay with sands and topsoil
622.02	9			Silty Clay With Sand (CL-ML) - dark brown with brown, dry to moist, non plastic, little to few coarse to fine sands, strong cementation, occasional reddish brick fragements, possible fill
620.02	6			Lean Clay With Sand (CL) - medium brown, dry to moist, low plasticity, medium stiff, little to few coarse to fine sands, dark brown silty pocket at top of sample, possible fill
617.02	8	3.75-4.0P		Sandy Lean Clay (CL) - olive gray with medium brown and gray, dry to moist, medium stiff, few coarse to fine sands, trace fine subangular to subrounded gravels, dark gray with occasional root matter at bottom of sample
615.02	9	1.3		Sandy Lean Clay With Gravel (CL) - medium brown with gray, dry, strongly cemented, stiff, crumbly, few coarse to fine sands, little to trace of medium to fine gravels, occasional medium to fine sand seams scattered throughout, dark gray with heavy _______ matter at top 2" of sample, possible old topsoil followed by native soil; Rimac: Pu = 68 lbs
	8	4.3P		same as above, medium brown, dry to moist, stiff, strongly cemented, glacial till
	9	4.5P		same as above, medium brown to brown, stiff, strongly cemented, dry, glacial till
605.02	12	4.0-4.5P		Sandy lean Clay (CL) - medium brown with orange brown, dry, non plastic, stiff, few coarse to fine sands, frequent sand seams, approximately 1/8"-1/4" thick at center and bottom of sample, sand seams of medium to fine sands, oxidized, possible weathered till with scattered sand seams
	12	1.9B		medium brown with gray, mottled with orange brown, dry, stiff, few coarse to fine sands, very oxidized, small pockets of dark gray to black coal like deposits in middle of sample, possible weathered glacial till; Rimac: Pu = 100 lbs
	11	3.8P		olive gray with light brown, dry to moist, slightly oxidized at top, stiff, possible unweathered glacial till
590.02	12	1.3		Lean Clay With Sand (CL) - uniform gray, dry to moist, stiff, little to few coarse to fine sands, scattered sand pockets, possible unweathered glacial till; Rimac: Pu = 70 lbs
583.52	12			uniform gray, dry to moist, stiff, little to few coarse to fine sands, scattered sand pockets, possible unweathered glacial till
583.02				Clayey Sand With Silt (SC) - gray, moist to wet, medium dense, clay with medium to fine sands, possible residual soil
				Bottom of hole = 40.0 feet

S-38
Sta. 630+85, 19' RT

Elev.	N	Qu	w%	Description
621.8				Silty Clay, black
619.8				Clay Till, brown, soft
	4	0.7B	23	
614.8	4	1.3B	13	Silty Clay, brown, soft
	5	1.0S	18	
609.8	4	0.6B	20	Silty Clay, brown, stiff
606.8	5	1.2B	22	Clay Till, gray, stiff
	4	2.0B	19	
	13	2.3B	16	
	20	1.6B	16	
	16	2.6B	13	
	19	2.7B	15	
590.8	26	3.4B	15	Fine Sand, gray, medium
	17			
	17			
584.8	7	1.5B	22	Clay, gray, stiff
	19	3.9S	20	
	16	3.3S	18	
576.8	29	4.0S	21	Clay Shale, dark gray, hard
	41	4.9S	20	
	62	5.5S	17	
	58	6.0S	18	
	58	4.9S	15	
	58	5.2S	18	
	100+	7.3S	14	
542.8				Bottom of hole = 79.0 feet

19BR-104
Sta. 58+65, 70' RT

Elev.	N	Qu	w%	Description
605.80				CONCRETE - 3" to 4" thick
605.40	9	0.7B	17.2	SILT - reddish brown, little to some clay, crumbly, medium plastic, medium stiff to stiff, moist.
602.30	9	1.7S	22.2	SILT - dark brown to gray with rust color, little to some clay, crumbly, medium plastic, stiff, moist.
599.80	2	0.7B	19.6	SILT - dark brown, and clay to silty CLAY, medium plastic, soft, moist.
597.30	5	0.9B	19.2	CLAY TILL - brown, sandy, little to some fine to coarse sand, trace gravel, crumbly, medium stiff, slightly moist (FILL)
594.80	6	0.5B	17.4	SILT - brown to dark gray, little to some clay, slightly to medium plastc, medium stiff, moist.
592.30	16		2.2	SILT - brown, some fine to coarse sand, and fine gravel, trace clay, moist. [Note: attempted to take Shelby tube at 13.5'; hit gravel; followed up with SPT]
589.80	7		14.8	CLAY TILL - greenish gray to bluish gray, silty, trace to little medium to coarse sand, trace fine gravel, medium plastic, stiff to very stiff, moist (GLACIAL TILL).
		1.3	17.5	-[Dry unit weight = 114.5 pcf]
	13	2.1B	13.5	
	14	3.5B	14.7	(LL=32, PI=17)
	15	3.1B	14.2	
	21	2.8S	16.0	- bluish gray sandy clay till.
	21	4.0B	14.2	- bluish gray sandy clay till.
568.30	55	4.2S	13.6	CLAY SHALE - black to dark gray, no laminations above 48.5 ft, thin laminations and partial rock-like shale chips below 48.5 ft depth, hard (for clay), slightly moist to dry.
	104/9"	>4.5P	10.6	
	50/1"		8.4	- black flaky shale, thinly laminated (start of rock-like shale properties).
	60/1"			
				[Groundwater level not observed in soils or shale during drilling]
547.22	50/1"		6.0	Bottom of hole = 58.58 feet

LEGEND

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

SUBSURFACE DATA PROFILE
STRUCTURE NO. 081-0179 (WB)
STRUCTURE NO. 081-0180 (EB)
STRUCTURE NO. 081-0181 (7TH-A)

PROFESSIONAL DESIGN FIRM LICENSE #184-001084

HANSON
Hanson Professional Services Inc.

JOB NO.	SHEET NO. 3	F.A.I RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
08H0120E		74	81-1-1HB	ROCK ISLAND	_	
DATE 2/20/15	7 SHEETS			CONTRACT NO. 64C08		
		FED. ROAD DIST. NO.	ILLINOIS	FED. AID PROJECT		

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

B-2(2011)
Sta. 601+31, 6' LT

Elev.	N	Qu	w%	Description
610.26		1.8P	14	STIFF gray SILTY CLAY LOAM
	16	2.7S	15	VERY STIFF gray/brown SILTY CLAY LOAM
	18	1.2B	15	STIFF brown SILTY CLAY LOAM
	10	2.3P	21	VERY STIFF dark brown SILTY CLAY LOAM
	6	1.0B	16	STIFF dark brown SILTY CLAY LOAM
	13	1.5P		STIFF brown SANDY LOAM with GRAVEL
596.26	22			No recovery, rock blocking sampler
	11			No recovery
591.26 / 590.80 (24h)	15	3.0B	13	VERY STIFF gray CLAY LOAM TILL
	16	2.7B	13	VERY STIFF gray CLAY LOAM TILL
	15	2.7B	13	VERY STIFF gray CLAY LOAM TILL
	16	2.2B	14	VERY STIFF gray CLAY LOAM TILL
	37	2.3S	17	VERY STIFF gray CLAY LOAM TILL
	21	1.3P	27	STIFF gray CLAY TILL with DOLOMITE lenses
	100/6"		52	STIFF gray CLAY TILL
575.26	Rec. = 85% RQD = 15%			Dolomite: gray-buff, alphanitic, dense, pitted and mostly fractured with voids evident. t.s.f.: 572.9 to 572.5
570.26	Rec. = 30% RQD = 0%			Dolomite: as above, pitted, fractured with macro-voiding apparent throughout.
565.26				Bottom of hole = 45.0 feet

19BR-105
Sta. 60+26, 14' LT

Elev.	N	Qu	w%	Description
609.30				CONCRETE - 3" thick concrete plus base course.
608.80	10	1.5P	12.8	SILT - light brown and dark brown, some clay, trace to little gravel, medium plastic, stiff, moist (FILL).
604.80	17	0.8S	12.6	SILT - light brown and gray mottled, little clay, crumbly, slightly to medium plastic, medium stiff, slightly moist to dry.
	4	0.6B	27.4	
600.80	5	0.6S	18.2	SILT - dark brown, little to some clay, crumbly, slightly to medium plastic, medium stiff, moist.
598.30	4	0.4S	16.2	SILT - dark brown, trace to little clay, little fine sand, slight binder, slightly plastic, soft to medium stiff, moist.
595.30	19		4.3	SAND - brown, fine to coarse, clayey, and gravel, loose, moist.
	4		5.5	
590.80	6	1.4B	14.4	CLAY TILL - greenish gray, sandy to silty, trace medium to coarse sand, trace fine gravel, slightly to medium plastic, hard, moist (GLACIAL TILL). -[Dry unit weight = 118 pcf]
		1.9B	14.3	
	12	3.1B	13.8	
	20	3.3B	12.9	- contains thin layers of wet/saturated fine sand.
580.30 (DD)	14	3.3B	15.4	
	50/1"		23.9	- greenish gray to bluish gray with limestone fragments, hard.
574.00	Rec. = 46% RQD = 8%			LIMESTONE - gray, fine grained, hard, dense, very thin to thin bedded, closely to very closely fractured with possible shale and/or clay seams which were not recovered between 35.3' and 40.7', occasional iron-stains at fractures, slightly weathered, poor quality rock but hard where recovered.
	Rec. = 81% RQD = 0%			[Note: driller repeatedly lifted the core barrel while drilling to keep it from jamming. Observation of core pieces suggest numerous near-vertical fractures were encountered, causing core pieces to get stuck in the core catcher and possibly grinding up subsequent rock encountered while drilling.]
	Rec. = 43% RQD = 0%			
	Rec. = 77% RQD = 35%			- 11" thick layer of very soft green-gray, sandy, gravelly clay at 45.8' to 46.7'. - 13" layer of medium gray "birdseye" texture limestone with vertical fractures at 47.5' to 48.6'.
558.50				Bottom of hole = 50.8 feet

19BR-106
Sta. 628+71, 13' RT

Elev.	N	Qu	w%	Description
612.90				Concrete - 4" thick plus base course.
612.40	5	1.2B	15.4	CLAY - yellowish brown, little to some silt, medium plastic, medium stiff, moist
609.40	4	1.0S	12.9	SILT - brown, tan, orange, and dark brown, mottled, some clay, to CLAY, some silt, medium plastic, medium stiff, moist.
	5	1.1B	18.6	
	3	0.8B	18.1	
		0.9B	16.4	- [Dry unit weight = 116.3 pcf] (LL=32, PI=18) - gray and tan silt, little to some clay at 13'.
599.40	13	3.0B	14.8	CLAY TILL - brown to gray and greenish gray, silty to sandy, trace to some fine to coarse sand, trace fine gravel, hard, dry to slightly moist (GLACIAL TILL).
	14	2.7B	15.6	
	29	4.5S	11.2	
	44	6.2B	10.9	
	23		13.1	[Groundwater not noted in soils during drilling operations.]
	50/2"			
585.90	Rec. = 91% RQD = 46%			LIMESTONE - gray, fine grained, hard, dense, thin bedded, horizontal to subhorizontal bedding fractures with several near-vertical to high angle fractures, slightly rough, frequently brown-stained fracture surfaces, slightly to very slightly weathered.
	Rec. = 100% RQD = 63%			- slightly to moderately weathered at 27.0'-27.8'; very weathered below 27.8'.
	Rec. = 100% RQD = 75%			- high angle (60° to 90°) fractures at 27.5'-27.7', 33.8', 35.4' 35.8'-36.0', 36.7', and 37.3'. Mid angle (30° to 60°) at 29.2', 34.0', and 34.5'.
575.60				Bottom of hole = 37.3 feet

LEGEND

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

SUBSURFACE DATA PROFILE
STRUCTURE NO. 081-0179 (WB)
STRUCTURE NO. 081-0180 (EB)
STRUCTURE NO. 081-0181 (7TH-A)

PROFESSIONAL DESIGN FIRM LICENSE #184-001084

HANSON
Hanson Professional Services Inc.

JOB NO.	SHEET NO. 4	F.A.I RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
08H0120E	7 SHEETS	74	81-1-1HB	ROCK ISLAND	_	
DATE 2/20/15				CONTRACT NO. 64C08		
		FED. ROAD DIST. NO.	ILLINOIS	FED. AID PROJECT		

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

B-1 (2011)
Sta. 52+08, ℄

Elev.	N	Qu	w%	Description
614.60		1.3P	10	BROWN stiff SILTY CLAY LOAM
	66			Broken Concrete
	10	3.0P	14	VERY STIFF black SILTY CLAY LOAM
608.10	7	2.0P	11	STIFF tan CLAY LOAM TILL
	8	0.8P	17	MEDIUM tan CLAY LOAM TILL with SAND lens
	11	2.0B	15	STIFF brown CLAY LOAM
	16	2.3B	11	VERY STIFF gray CLAY LOAM TILL
	17	3.3B	13	VERY STIFF gray CLAY LOAM TILL
	32	10.3B	10	HARD gray CLAY LOAM TILL
	66	6.6B	8	HARD gray CLAY LOAM TILL with SANDSTONE at bottom
	22	10.3B	7	HARD gray CLAY LOAM TILL
588.10	100			VERY DENSE weathered SANDSTONE
585.60	Rec. = 60% RQD = 22%			Dolomite: gray-buff, aphanitic, dense, top-half mostly fractured, with clay film and minor pitting. t.s.f. 582.5 to 581.6
580.60	Rec. = 100% RQD = 70%			Dolomite: as above, though mostly solid and thickly bedded. t.s.f. 578.1 to 577.2
575.60				Bottom of hole = 39.0 feet

19BR-107
Sta. 59+82, 60' RT

Elev.	N	Qu	w%	Description
609.10				
608.60				CONCRETE SIDEWALK - concrete (4-1/2" thick) + base course.
	7	1.4B	13.5	CLAY - brown to yellowish brown, some silt, trace gravel, medium plastic, stiff, slightly moist.
605.60	10	1.5B	15.9	SILT - dark brown, little to some clay, trace gravel, crumbly, slight to medium plastic, stiff, moist.
	10	1.3B	15.6	
		1.8P	24.3	- little clay. (LL=28, PI=7)
598.10	5	0.5P	14.4	CLAY TILL - dark brown (to 12.5 ft) to brown, to gray and tan, trace medium to coarse sand, trace fine gravel, stiff, moist (GLACIAL TILL).
	9	2.0B	14.1	- sandy till at 11.0'-12.5'.
		3.3B	14.4	-[Dry unit weight = 119.8 pcf]
590.60	14	2.3B	14.1	CLAY TILL - greenish brown to gray, trace medium to coarse sand, trace fine gravel, hard, moist to dry (GLACIAL TILL).
	20	2.6B	13.8	
	18	2.8B	14.5	
	16	2.7B	13.1	
	14	3.2B	13.9	
	14	3.0P	12.7	
570.60	45	>4.5P	14.9	CLAY SHALE - greenish gray to brown, clayey, hard, slightly to moderately weathered, slightly moist to dry.
565.60	86	>4.5P	13.5	CLAY SHALE - black to dark gray, feint to no laminations, hard, slightly moist to dry.
	113/9"	>4.5P	10.9	
	50/5"	>4.5P	10.3	
				- [Note: driller added water to hole to be able to turn augers below 50' depth]
	50/2"	>4.5P	12.8	- soft, laminated, clayey, sticky; falls apart and readily crumbles when moist; becomes sticky clay when wet.
550.50	50/5"		7.9	- light and dark gray shale cuttings.
				Bottom of hole = 58.6 feet

RW06-04
Sta. 60+68, 63' RT

Elev.	N	Qu	w%	Description
624.70				
624.45				CONCRETE.
	5	4.50P	13	FILL - Dark brown with gray mottles, SILT, little clay, trace sand.
	11	4.50P	19	
	13	3.00P	16	
	22	4.50P	6	
613.70	11	3.30P	18	FILL - Dark gray silty lean CLAY, little sand, trace gravel, with wood fragments.
		2.16B	16	
	9	3.00P	19	
	11	2.50P	22	
602.70				Gray moist, very stiff, silty lean CLAY, with trace sand and gravel.
	15	4.50P	14	
	17	4.30P	15	
589.70	18	4.00P	14	Bottom of hole = 35.0 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
558.10 ▽	DD = during drilling 24h = 24 hours after completion

SUBSURFACE DATA PROFILE
STRUCTURE NO. 081-0179 (WB)
STRUCTURE NO. 081-0180 (EB)
STRUCTURE NO. 081-0181 (7TH-A)

PROFESSIONAL DESIGN FIRM LICENSE #184-001084

HANSON
Hanson Professional Services Inc.

JOB NO.	SHEET NO. 5	F.A.I RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
08H0120E		74	81-1-1HB	ROCK ISLAND	_	
DATE 2/20/15	7 SHEETS			CONTRACT NO. 64C08		
		FED. ROAD DIST. NO.	ILLINOIS	FED. AID PROJECT		

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

RW06-1
Sta. 61+02, 7' LT

Elev.	N	Qu	w%	Description
611.30				CONCRETE
610.80		2.50P	14 R / 14 / 17 C	FILL - Light gray, slightly moist, SILT
608.30		1.80P	13	FILL - Very dark brown, moist, clayey SILT with trace gravel
605.30	17	2.00P	15	FILL - Gray, moist, medium dense, silty, medium-grained SAND with trace gravel, wood, brick and rock fragments
	50/4"		12	
600.30		1.65S	20 / 17	Dark brown, moist, stiff, sandy SILT with trace gravel
	8		16	
596.30				Dark brown, moist, sandy, clayey SILT with trace gravel
595.30		0.50P	12	Dark brown, wet, dense, silty SAND with trace gravel
593.80 ▽ DD				
593.30	8	0.54B	18	Gray and brown, moist, medium stiff, silty CLAY with sand and trace gravel
588.80	21	2.61B	14	Gray and brown, moist, very stiff, silty CLAY with sand and gravel
586.30				Bottom of hole = 25.0 feet

19BR-108
Sta. 61+26, 22' LT

Elev.	N	Qu	w%	Description
611.60				CONCRETE SIDEWALK - 4.5" thick concrete plus base course.
611.00	6	1.6B	13.8	CLAY - olive brown and gray, some to and silt, trace to little medium to coarse sand, trace fine gravel, very stiff, moist (GLACIAL TILL-FILL).
	12	3.0B	18.2	
605.60	10	0.8B	18.4	SILT - dark brown, little to some clay, trace gravel, trace organics, slightly to medium plastic, medium stiff to stiff, moist
	5	0.9B	24.2	
600.60	5	0.7B	24.1	CLAY - brown, little silt, trace sand, with gravel, to SILT and clay, with gravel or cobble, slightly to medium plastic, medium stiff, moist (LL=21, PI=5)
	17		13.9	- cobble at 14.5'-15.0'.
595.60		2.5B	14.2	CLAY TILL - greenish brown to gray, trace to little medium to coarse sand, trace fine gravel, hard, moist to dry (GLACIAL TILL). -[Dry unit weight = 116.7 pcf]
	13	3.4B	13.9	
	16	3.1B	14.4	
		2.8P		
	14	2.9B	14.8	
	50/3"	2.5P	17.3	- greenish gray and red silty clay till, crumbly, moist.
581.80				CLAY - red, silty, shaly, crumbly, dry to slightly moist (TILL or CLAY SHALE).
578.10	91	3.5P	14.8	CLAY SHALE - greenish gray, clayey, hard, laminated, slightly to moderately weathered, slightly moist to dry. -[Groundwater not observed in soils and shale during drilling operations]
573.90	Rec. = 77% RQD = 0%			LIMESTONE - gray, fine grained, dense, hard, very thin to thin bedded, horizontal to subhorizontal slightly rough fractures with some high angle (60° to 90°) fractures, slightly weathered with faint iron stains on some fractures, occasional stylolites.
	Rec. = 93% RQD = 23%			
	Rec. = 100% RQD = 45%			
563.70				Bottom of hole = 47.9 feet

RW06-05
Sta. 62+58, 22' RT

Elev.	N	Qu	w%	Description
644.60				TOPSOIL.
644.35	14	3.50P	15	FILL - Brown lean CLAY, trace silt, trace sand, with organics.
	7	1.75B	23	
	18	3.50P	17	
636.10	11	3.10B	16	FILL - Brown and gray silty lean CLAY, trace sand, trace gravel, with wood debris and brick fragments.
		1.55S	19	
		1.60S	18	
	15	3.30S	16	
	16	4.46S	16	
	16	4.50P	16	
	20	2.25S	16	
	20	3.30S	18	
	21	4.50P	16	
607.60	15	2.50P	22	FILL - Gray clayey SILT, little sand, trace gravel, with red brick fragments.
	16		10	
597.60	17	3.30S	15	Gray moisy, very stiff, silty lean CLAY, with trace sand and trace gravel.
	26	6.01B	12	
	26	3.69B	15	
584.60				Bottom of hole = 60.0 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

SUBSURFACE DATA PROFILE
STRUCTURE NO. 081-0179 (WB)
STRUCTURE NO. 081-0180 (EB)
STRUCTURE NO. 081-0181 (7TH-A)

PROFESSIONAL DESIGN FIRM LICENSE #184-001084

HANSON
Hanson Professional Services Inc.

JOB NO.		F.A.I RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
08H0120E	SHEET NO. 6	74	81-1-1HB	ROCK ISLAND	_	
DATE 2/20/15	7 SHEETS			CONTRACT NO. 64C08		
		FED. ROAD DIST. NO.	ILLINOIS	FED. AID PROJECT		

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

ILR0804
Sta. 626+96, 7' RT

Elev.	N	Qu	w%	Description
641.39				Clay (CL) - gray, moist, very stiff to hard, trace sand and fine angular gravel
	7	4.1S		
	16	2.5P		
635.39	4			Clay (CL) - gray to greenish gray, moist, trace fine to coarse grained sand, stiff to hard
	6	1.9B		RIMAC: Pu = 31 lbs (Bulging)
	10	4.1B		RIMAC: Pu = 68 lbs (Bulging)
	13	3.0P		
	14	4.5P		
	36	4.5P		shale in tip
606.39				Bottom of hole = 35.0 feet

19BR-109
Sta. 627+68, 32' RT

Elev.	N	Qu	w%	Description
614.30				TOPSOIL - (roots) 1" to 2" thick.
614.10	9	2.3S	12.8	SILT - brown, tan and orange mottled, little clay, slightly to medium plastic, stiff to crumbly, moist
610.80	11	1.9B	20.4	CLAY - greenish gray and brown, little silt, waxy, medium plastic, stiff, moist.
608.30	4	0.8B	16.0	CLAY - brown and tan, some to and silt, trace sand, medium plastic, medium stiff, moist.
605.80	7	0.8S	16.7	SILT - dark brown to brown, little to some clay, trace fine sand, slightly to medium plastic, medium stiff to stiff, moist.
602.30	6	1.0B	16.6	CLAY - gray and brown mottled, some silt, medium plastic, stiff, moist.
600.80	10	0.7B	14.2	CLAY - brown and red brown, sandy, grading from clayey silt with fine to coarse sand, trace gravel to very soft wet sandy clay.
DD	4	0.5B	18.4	
595.80	8		13.9	GRAVEL - brown to reddish brown, clayey, angular, saturated.
593.30	11	3.2B	9.7	CLAY - greenish gray, little to some silt, medium to highly plastic, stiff to very stiff, moist.
		2.9B	14.9	- [Dry unit weight = 120.7 pcf]
				- trace sand at bottom of shelby tube.
587.80	98/10"		15.4	CLAY SHALE - bluish to greenish gray, clayey, hard, no laminations, slightly weathered, slightly moist to dry.
	55/3"	>4.5P	10.7	
583.80	Rec. = 86% RQD = 60%			CLAY SHALE - bluish to greenish gray, clayey, hard, no laminations, slightly weathered. Intermixed sandy shale and limestone at 30.5'-32.2'.
582.10	Rec. = 91% RQD = 74%			LIMESTONE - gray with yellowish brown and iron-staining along fractures in the upper 6 ft, fine grained, occasional stylolites, dense, hard, sound, thin bedded, primarily uneven horizontal to subhorizontal fractures with occasional high angle fractures, slightly weathered to fresh. - iron stained fractures at 32.8', 36.0', 36.2', 36.5', 36.8', 38.2'. - vertical fracture at 35.4'-35.6', 80° to 60° curvilinear fracture at 36.6'-36.8', 60° jagged brown-stained fracture at 36.4'. - fresh rock below 38.2'. - [Note: RQD shown for Run 1 is based on length of recovered rock, not on length of run. RQD=40% for entire length of run (including material washed away from augers and ground up during the drilling operations).]
572.00				Bottom of hole = 42.3 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
558.10 DD = during drilling
24h = 24 hours after completion

SUBSURFACE DATA PROFILE
STRUCTURE NO. 081-0179 (WB)
STRUCTURE NO. 081-0180 (EB)
STRUCTURE NO. 081-0181 (7TH-A)

PROFESSIONAL DESIGN FIRM LICENSE #184-001084

HANSON
Hanson Professional Services Inc.

JOB NO.	SHEET NO. 7	F.A.I RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
08H0120E	7 SHEETS	74	81-1-1HB	ROCK ISLAND	_	
DATE 2/20/15				CONTRACT NO. 64C08		
		FED. ROAD DIST. NO.	ILLINOIS	FED. AID PROJECT		

Illinois Department of Transportation
Division of Highways
JCI

SOIL BORING LOG

Date 9/14/07

ROUTE I-74 **DESCRIPTION** New I-74 Bridge Over Mississippi River - Illinois Approach **LOGGED BY** KJB

SECTION ______ **LOCATION** (N=561990.925, E=2459643.925), **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. 19BR-104
Station ______
Offset ______
Ground Surface Elev. 605.80 ft

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

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

Description	Elev. (ft)	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
CONCRETE - 3" to 4" thick	605.40				
SILT - reddish brown, little to some clay, crumbly, medium plastic, medium stiff to stiff, moist.			2		
			4	0.7	17.2
			5	B	
	602.30				
SILT - dark brown to gray with rust color, little to some clay, crumbly, medium plastic, stiff, moist.			4		
			5	1.7	22.2
		-5	4	S	
	599.80				
SILT - dark brown, and clay to silty CLAY, medium plastic, soft, moist.			2		
			1	0.7	19.6
			1	B	
	597.30				
CLAY TILL - brown, sandy, little to some fine to coarse sand, trace gravel, crumbly, medium stiff, slightly moist (FILL?)			1		
			2	0.9	19.2
		-10	3	B	
	594.80				
SILT - brown to dark gray, little to some clay, slightly to medium plastic, medium stiff, moist.			WOH		
			2	0.5	17.4
			4	B	
	592.30				
SILT - brown, some fine to coarse sand, and fine gravel, trace clay, moist.			7		
			8		2.2
		-15	8		
[Note: attempted to take Shelby tube at 13.5'; hit gravel; followed up with SPT]	589.80				
CLAY TILL - greenish gray to bluish gray, silty, trace to little medium to coarse sand, trace fine gravel, medium plastic, stiff to very stiff, moist (GLACIAL TILL).			6		
			5		14.8
			2		
-[Dry unit weight = 114.5 pcf]					
				1.3	
		-20		P	

Description	Elev. (ft)	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
CLAY TILL - greenish gray to bluish gray, silty, trace to little medium to coarse sand, trace fine gravel, medium plastic, stiff to very stiff, moist (GLACIAL TILL). *(continued)*			3		
			5	2.1	13.5
			8	B	
			3		
			6	3.5	
		-25	8	B	
			4		
			7	3.1	14.2
			8	B	
- bluish gray sandy clay till.			5		
			9	2.8	16.0
		-30	12	S	
- bluish gray sandy clay till.			7		
			9	4.0	14.2
		-35	12	B	
	568.30				
CLAY SHALE - black to dark gray, no laminations above 48.5 ft, thin laminations and partial rock-like shale chips below 48.5 ft depth, hard (for clay), slightly moist to dry.			14		
			22	4.2	13.6
		-40	33	S	

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
JCI

SOIL BORING LOG

Date 9/14/07

ROUTE I-74 **DESCRIPTION** New I-74 Bridge Over Mississippi River - Illinois Approach **LOGGED BY** KJB

SECTION **LOCATION** (N=561990.925, E=2459643.925), **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. 19BR-104
Station
Offset
Ground Surface Elev. 605.80 **ft**

Surface Water Elev. ft
Stream Bed Elev. ft

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

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
CLAY SHALE - black to dark gray, no laminations above 48.5 ft, thin laminations and partial rock-like shale chips below 48.5 ft depth, hard (for clay), slightly moist to dry. *(continued)*				
		24		
		54	>4.5	10.6
	-45	50/3"	P	
- black flaky shale, thinly laminated (start of rock-like shale properties).		48		
		50/1"		8.4
	-50			
		60/1"		
	-55			
[Groundwater level not observed in soils or shale during drilling]				
547.22		48		
End of Boring		50/1"		6.0
	-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)

Illinois Department of Transportation
Division of Highways
JCI

SOIL BORING LOG

Date 9/14/07

ROUTE I-74 **DESCRIPTION** New I-74 Bridge Over Mississippi River - Illinois Approach **LOGGED BY** KJB

SECTION ______ **LOCATION** (N=561828.313, E=2459724.286), **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. 19BR-105
Station ______
Offset ______
Ground Surface Elev. 609.30 ft

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

Groundwater Elev.:
First Encounter 580.3 ft ▼
Upon Completion ______ ft
After ___ **Hrs.** ______ ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
CONCRETE - 3" thick concrete plus base course.	608.80				
SILT - light brown and dark brown, some clay, trace to little gravel, medium plastic, stiff, moist (FILL?).			2		
			5	1.5	12.8
			5	P	
			6		
	604.80		10	0.8	12.6
SILT - light brown and gray mottled, little clay, crumbly, slightly to medium plastic, medium stiff, slightly moist to dry.		-5	7	S	
			3		
			2	0.6	27.4
			2	B	
	600.80				
SILT - dark brown, little to some clay, crumbly, slight to medium plastic, medium stiff, moist.			2		
			2	0.6	18.2
		-10	3	S	
	598.30				
SILT - dark brown, trace to little clay, little fine sand, slight binder, slightly plastic, soft to medium stiff, moist.			2		
			2	0.4	16.2
			2	S	
	595.30		3		
SAND - brown, fine to coarse, clayey, and gravel, loose, moist.			7		4.3
		-15	12		
			5		
			2		5.5
			2		
	590.80				
			1		
			3	1.4	14.4
		-20	3	B	

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
CLAY TILL - greenish gray, sandy to silty, trace medium to coarse sand, trace fine gravel, slightly to medium plastic, hard, moist (GLACIAL TILL). *(continued)* **-[Dry unit weight = 118 pcf]**					
				1.9	14.3
				B	
			4		
			5	3.1	13.8
		-25	7	B	
			6		
			10	3.3	12.9
			10	B	
- contains thin layers of wet/saturated fine sand.	▼		4		
			7	3.3	15.4
		-30	7	B	
			21		
			50/1"		23.9
- greenish gray to bluish gray with limestone fragments, hard.	574.00	-35			
Borehole continued with rock coring.					
		-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
JCI

ROCK CORE LOG

Date 9/14/07

ROUTE I-74 **DESCRIPTION** New I-74 Bridge Over Mississippi River - Illinois Approach **LOGGED BY** KJB

SECTION ____ **LOCATION** (N=561828.313, E=2459724.286), **SEC.** 32, **TWP.** 18N, **RNG.** 1W, 4th **PM**

COUNTY Rock Island **CORING METHOD** NQ Core

STRUCT. NO. ____
Station ____

CORING BARREL TYPE & SIZE NQ Wireline

Core Diameter 1.8 in
Top of Rock Elev. 574.80 ft
Begin Core Elev. 574.00 ft

BORING NO. 19BR-105
Station ____
Offset ____
Ground Surface Elev. 609.30 ft

Description	Elev.	DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
LIMESTONE - gray, fine grained, hard, dense, very thin to thin bedded, closely to very closely fractured with possible shale and/or clay seams which were not recovered between 35.3' and 40.7', occasional iron-stains at fractures, slightly weathered, poor quality rock but hard where recovered.	574.00		Run 1	46	8	2.8	
		-40					
[Note: driller repeatedly lifted the core barrel while drilling to keep it from jamming. Observation of core pieces suggest numerous near-vertical fractures were encountered, causing core pieces to get stuck in the core catcher and possibly grinding up subsequent rock encountered while drilling.]			Run 2	81	0		
			Run 3	43	0	1.7	488.6
		-45					
- 11" thick layer of very soft green-gray, sandy, gravelly clay at 45.8' to 46.7'.			Run 4	77	35	4.4	
- 13" layer of medium gray "birdseye" texture limestone with vertical fractures at 47.5' to 48.6'.							
		-50					
	558.50						
End of Boring							
		-55					

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

Illinois Department of Transportation
Division of Highways
JCI

SOIL BORING LOG

Date 9/13/07

ROUTE I-74 **DESCRIPTION** New I-74 Bridge Over Mississippi River - Illinois Approach **LOGGED BY** KJB

SECTION ______ **LOCATION** (N=561671.671, E=2459820.632), **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. 19BR-106
Station ______
Offset ______
Ground Surface Elev. 612.90 ft

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

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

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
CONCRETE - 4" thick plus base course.	612.40				
CLAY - yellowish brown, little to some silt, medium plastic, medium stiff, moist.			1		
			2	1.2	15.4
			3	B	
	609.40				
SILT - brown, tan, orange, and dark brown, mottled, some clay, to CLAY, some silt, medium plastic, medium stiff, moist.			WOH		
			1	1.0	12.9
		-5	3	S	
			WOH		
			2	1.1	18.6
			3	B	
			WOH		
			WOH	0.8	18.1
		-10	3	B	
				0.9	16.4
- **[Dry unit weight = 116.3 pcf]**				B	
- gray and tan silt, little to some clay at 13'.	599.40				
CLAY TILL - brown to gray and greenish gray, silty to sandy, trace to some fine to coarse sand, trace fine gravel, hard, dry to slightly moist (GLACIAL TILL).			5		
			6	3.0	14.8
		-15	7	B	
			3		
			6	2.7	15.6
			8	B	
			6		
			11	4.5	11.2
		-20	18	S	

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
CLAY TILL - brown to gray and greenish gray, silty to sandy, trace to some fine to coarse sand, trace fine gravel, hard, dry to slightly moist (GLACIAL TILL). *(continued)*			9		
			25	6.2	10.9
			19	B	
			7		
			9		13.1
		-25	14		
[Groundwater not noted in soils during drilling operations.]					
			5		
	585.90		50/2"		
Borehole continued with rock coring.					
		-30			
		-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
JCI

ROCK CORE LOG

Date 9/13/07

ROUTE I-74 **DESCRIPTION** New I-74 Bridge Over Mississippi River - Illinois Approach **LOGGED BY** KJB

SECTION ______ **LOCATION** (N=561671.671, E=2459820.632), **SEC.** 32, **TWP.** 18N, **RNG.** 1W, 4th **PM**

COUNTY Rock Island **CORING METHOD** NQ Core

STRUCT. NO. ______
Station ______

BORING NO. 19BR-106
Station ______
Offset ______
Ground Surface Elev. 612.90 ft

CORING BARREL TYPE & SIZE NQ Wireline

Core Diameter 1.8 in
Top of Rock Elev. 586.20 ft
Begin Core Elev. 585.90 ft

Description	Elev.	DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
LIMESTONE - gray, fine grained, hard, dense, thin bedded, horizontal to subhorizontal bedding fractures with several near-vertical to high angle fractures, slightly rough, frequently brown-stained fracture surfaces, slightly to very slightly weathered.	585.90		Run 1	91	46	3.2	309.9
		-30					
- slightly to moderately weathered at 27.0'-27.8'; very slightly weathered below 27.8'. - high angle (60° to 90°) fractures at 27.5'-27.7', 33.8', 35.4', 35.8'-36.0', 36.7', and 37.3'. Mid angle (30° to 60°) at 29.2', 34.0', and 34.5'.			Run 2	100	63	3.2	
		-35					
			Run 3	100	75	4	
End of Boring	575.60						
		-40					
		-45					

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

Illinois Department of Transportation
Division of Highways
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SOIL BORING LOG

Date 9/10/07

ROUTE I-74 **DESCRIPTION** New I-74 Bridge Over Mississippi River - Illinois Approach **LOGGED BY** KJB

SECTION ______ **LOCATION** (N=561873.84, E=2459651.753), **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. 19BR-107
Station ______
Offset ______
Ground Surface Elev. 609.10 ft

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

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

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
CONCRETE SIDEWALK - concrete (4-1/2" thick) + base course.	608.60		3		
CLAY - brown to yellowish brown, some silt, trace gravel, medium plastic, stiff, slightly moist.			4	1.4	13.5
			3	B	
	605.60				
SILT - dark brown, little to some clay, trace gravel, crumbly, slight to medium plastic, stiff, moist.			4		
			5	1.5	15.9
		-5	5	B	
			2		
			4	1.3	15.6
			6	B	
- little clay.				1.8	24.3
		-10		P	
	598.10				
CLAY TILL - dark brown (to 12.5 ft) to brown, to gray and tan, trace medium to coarse sand, trace fine gravel, stiff, moist (GLACIAL TILL).			2		
- sandy till at 11.0'-12.5'.			2	0.5	14.4
			3	P	
			3		
			4	2.0	14.1
		-15	5	B	
				3.3	14.4
				B	
-[Dry unit weight = 119.8 pcf]	590.60				
			4		
			6	2.3	14.1
		-20	8	B	

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
CLAY TILL - greenish brown to gray, trace medium to coarse sand, trace fine gravel, hard, moist to dry (GLACIAL TILL). *(continued)*			5		
			9	2.6	13.8
			11	B	
			5		
			8	2.8	14.5
		-25	10	B	
			6		
			7	2.7	13.1
			9	B	
			5		
			5	3.2	13.9
		-30	9	B	
			4		
			5	3.0	12.7
		-35	9	P	
	570.60				
CLAY SHALE - greenish gray to brown, clayey, hard, slightly to moderately weathered, slightly			6		
			17	>4.5	14.9
		-40	28	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)

BBS, from 137 (Rev. 8-99)

Illinois Department of Transportation
Division of Highways
JCI

SOIL BORING LOG

Date 9/10/07

ROUTE I-74 **DESCRIPTION** New I-74 Bridge Over Mississippi River - Illinois Approach **LOGGED BY** KJB

SECTION ______ **LOCATION** (N=561873.84, E=2459651.753), **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. 19BR-107
Station ______
Offset ______
Ground Surface Elev. 609.10 ft

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

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

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
moist to dry. CLAY SHALE - greenish gray to brown, clayey, hard, slightly to moderately weathered, slightly moist to dry. *(continued)*	565.60				
CLAY SHALE - black to dark gray, feint to no laminations, hard, slightly moist to dry.			16		
			29	>4.5	13.5
		-45	57	P	
			19		
			58	>4.5	10.9
			55/3"	P	
			20		
			50/5"	>4.5	10.3
		-50		P	
- [Note: driller added water to hole to be able to turn augers below 50' depth]					
			33		
- soft, laminated, clayey, sticky; falls apart and readily crumbles when moist; becomes sticky clay when wet.			50/2"	>4.5	12.8
		-55		P	
- light and dark gray shale cuttings.	550.50		50/5"		7.9
End of Boring		-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)

Illinois Department of Transportation
Division of Highways
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SOIL BORING LOG

Date 9/11/07

ROUTE I-74 **DESCRIPTION** New I-74 Bridge Over Mississippi River - Illinois Approach **LOGGED BY** KJB

SECTION ______ **LOCATION** (N=561728.148, E=2459730.629), **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.	19BR-108
Station	
Offset	
Ground Surface Elev.	611.60 ft
Surface Water Elev.	ft
Stream Bed Elev.	ft
Groundwater Elev.:	
First Encounter	ft
Upon Completion	ft
After ___ Hrs.	ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
CONCRETE SIDEWALK - 4.5" thick concrete plus base course.	611.00				
CLAY - olive brown and gray, some to and silt, trace to little medium to coarse sand, trace fine gravel, very stiff, moist (GLACIAL TILL - FILL?).			4 2 4	1.6 B	13.8
		-5	2 5 7	3.0 B	18.2
	605.60				
SILT - dark brown, little to some clay, trace gravel, trace organics, slightly to medium plastic, medium stiff to stiff, moist			4 5 5	0.8 B	18.4
		-10	2 2 3	0.9 B	24.2
	600.60				
CLAY - brown, little silt, trace sand, with gravel, to SILT and clay, with gravel or cobble, slightly to medium plastic, medium stiff, moist.			WOH 2 3	0.7 B	24.1
- cobble at 14.5'-15.0'.		-15	3 5 12		13.9
	595.60				
CLAY TILL - greenish brown to gray, trace to little medium to coarse sand, trace fine gravel, hard, moist to dry (GLACIAL TILL). **-[Dry unit weight = 116.7 pcf]**				2.5 B	14.2
		-20	5 5 8	3.4 B	13.9
CLAY TILL - greenish brown to gray, trace to little medium to coarse sand, trace fine gravel, hard, moist to dry (GLACIAL TILL). *(continued)*			5 7 9	3.1 B	14.4
		-25		2.8 P	
			4 6 8	2.9 B	14.8
- greenish gray and red silty clay till, crumbly, moist.	581.80	-30	30 50/3"	2.5 P	17.3
CLAY - red, silty, shaly, crumbly, dry to slightly moist (TILL or CLAY SHALE?).	578.10				
CLAY SHALE - greenish gray, clayey, hard, laminated, slightly to moderately weathered, slightly moist to dry.		-35	18 31 60	3.5 P	14.8
- [Groundwater not observed in soils and shale during drilling operations]	573.90				
Borehole continued with rock coring.		-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
JCI

ROCK CORE LOG

Date 9/11/07

ROUTE I-74 **DESCRIPTION** New I-74 Bridge Over Mississippi River - Illinois Approach **LOGGED BY** KJB

SECTION ______ **LOCATION** (N=561728.148, E=2459730.629), **SEC.** 32, **TWP.** 18N, **RNG.** 1W, 4th **PM**

COUNTY Rock Island **CORING METHOD** NQ Core

STRUCT. NO. ______
Station ______

CORING BARREL TYPE & SIZE NQ Wireline

Core Diameter 1.8 **in**
Top of Rock Elev. 573.90 **ft**
Begin Core Elev. 573.90 **ft**

BORING NO. 19BR-108
Station ______
Offset ______
Ground Surface Elev. 611.60 **ft**

Description	Elev.	DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
LIMESTONE - gray, fine grained, dense, hard, very thin to thin bedded, horizontal to subhorizontal slightly rough fractures with some high angle (60° to 90°) fractures, slightly weathered with faint iron stains on some fractures, occasional stylolites.	573.90		Run 1	77	0	3.4	
		-40					
			Run 2	93	23	4	503.4
		-45					
			Run 3	100	45	3.5	
	563.70						
End of Boring							
		-50					
		-55					

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

Illinois Department of Transportation
Division of Highways
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SOIL BORING LOG

Date 9/12/07

ROUTE I-74 **DESCRIPTION** New I-74 Bridge Over Mississippi River - Illinois Approach **LOGGED BY** KJB

SECTION ______ **LOCATION** (N=561568.395, E=2459838.396), **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. 19BR-109
Station ______
Offset ______
Ground Surface Elev. 614.30 ft

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

Groundwater Elev.:
First Encounter 595.8 ft ▼
Upon Completion ______ ft
After ___ **Hrs.** ______ ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
TOPSOIL - (roots) 1" to 2" thick.	614.10				
SILT - brown, tan and orange mottled, little clay, slightly to medium plastic, stiff to crumbly, moist.			2		
			4	2.3	12.8
			5	S	
	610.80				
CLAY - greenish gray and brown, little silt, waxy, medium plastic, stiff, moist.			3		
			5	1.9	20.4
		-5	6	B	
	608.30				
CLAY - brown and tan, some to and silt, trace sand, medium plastic, medium stiff, moist.			2		
			2	0.8	16.0
			2	B	
	605.80				
SILT - dark brown to brown, little to some clay, trace fine sand, slightly to medium plastic, medium stiff to stiff, moist.			2		
			3	0.8	16.7
		-10	4	S	
			1		
	602.30		3	1.0	16.6
CLAY - gray and brown mottled, some silt, medium plastic, stiff, moist.			3	B	
	600.80				
CLAY - brown and red brown, sandy, grading from clayey silt with fine to coarse sand, trace gravel to very soft wet sandy clay.			3		
			3	0.7	14.2
		-15	7	B	
			WOH		
			2	0.5	18.4
			2	B	
	595.80 ▼				
GRAVEL - brown to reddish brown, clayey, angular, saturated.			2		
			4		13.9
		-20	4		

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
GRAVEL - brown to reddish brown, clayey, angular, saturated. *(continued)*	593.30				
CLAY - greenish gray, little to some silt, medium to highly plastic, stiff to very stiff, moist.			2		
			4	3.2	9.7
			7	B	
-[Dry unit weight = 120.7 pcf]				2.9	14.9
		-25		B	
- trace sand at bottom of shelby tube.					
	587.80		7		
CLAY SHALE - bluish to greenish gray, clayey, hard, no laminations, slightly weathered, slightly moist to dry.			48		15.4
			50/4"		
			48		
			55/3"	>4.5	10.7
		-30		P	
	583.80				
Borehole continued with rock coring.					
		-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)

BBS, from 137 (Rev. 8-99)

Illinois Department of Transportation
Division of Highways
JCI

ROCK CORE LOG

Date 9/12/07

ROUTE I-74 **DESCRIPTION** New I-74 Bridge Over Mississippi River - Illinois Approach **LOGGED BY** KJB

SECTION ______ **LOCATION** (N=561568.395, E=2459838.396), **SEC.** 32, **TWP.** 18N, **RNG.** 1W, 4th **PM**

COUNTY Rock Island **CORING METHOD** NQ Core

STRUCT. NO. ______
Station ______

CORING BARREL TYPE & SIZE NQ Wireline

Core Diameter 1.8 in
Top of Rock Elev. 583.80 ft
Begin Core Elev. 583.80 ft

BORING NO. 19BR-109
Station ______
Offset ______
Ground Surface Elev. 614.30 ft

Description	Elev.	DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
CLAY SHALE - bluish to greenish gray, clayey, hard, no laminations, slightly weathered.	583.80		Run 1	86	60	3.3	690.7
- intermixed sandy shale and limestone at 30.5'-32.2'.	582.10						
LIMESTONE - gray with yellowish brown and iron-staining along fractures in the upper 6 ft, fine grained, occasional stylolites, dense, hard, sound, thin bedded, primarily uneven horizontal to subhorizontal fractures with occasional high angle fractures, slightly weathered to fresh.							
- iron stained fractures at 32.8', 36.0', 36.2', 36.5', 36.8', 38.2'.		-35					
- vertical fracture at 35.4'-35.6'; 80° to 60° curvilinear fracture at 36.6'-36.8'; 60° jagged brown-stained fracture at 36.4'.			Run 2	91	74	2.8	
- fresh rock below 38.2'.							
- [Note: RQD shown for Run 1 is based on length of recovered rock, not on length of run. RQD= 40% for entire length of run (including material washed away from augers and ground up during the drilling operations).]		-40					
	572.00						
End of Boring		-45					
		-50					

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

Illinois Department of Transportation
Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Date 2/16/11

ROUTE FAI 74 **DESCRIPTION** 081-0099, 0100 P92-032-01 I-74 over 19th Street, north of 12th Avenue **LOGGED BY** J. Wenzel

SECTION 81-1HB **LOCATION** Moline Twp. - 32SE, **SEC.** , **TWP.** 18N, **RNG.** 1W

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

STRUCT. NO. 081-0099, 0100
Station

BORING NO. B-1
Station 52+08
Offset 0.00ft off BL - 19th St.
Ground Surface Elev. 614.60 ft

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

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

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
BROWN stiff SILTY CLAY LOAM	612.60			1.3 P	10
Broken Concrete	611.10		5 15 51		
VERY STIFF black SILTY CLAY LOAM	608.10	-5	4 4 6	3.0 P	14
STIFF tan CLAY LOAM TILL	606.10		2 3 4	2.0 P	11
MEDIUM tan CLAY LOAM TILL with SAND lens	603.60	-10	2 4 4	0.8 P	17
STIFF brown CLAY LOAM	601.10		1 3 8	2.0 B	15
VERY STIFF gray CLAY LOAM TILL	598.60	-15	2 6 10	2.3 B	11
VERY STIFF gray CLAY LOAM TILL	596.10		3 7 10	3.3 B	13
HARD gray CLAY LOAM TILL		-20	7		
HARD gray CLAY LOAM TILL *(continued)*	593.60		11 21	10.3 B	10
HARD gray CLAY LOAM TILL with SANDSTONE at bottom	591.10		6 14 52	6.6 B	8
HARD gray CLAY LOAM TILL	588.10	-25	5 14 8	10.3 B	7
VERY DENSE weathered SANDSTONE	585.60		4 43 57		
Borehole continued with rock coring.		-30 -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
Illinois Department of Transportation

ROCK CORE LOG

Date 2/16/11

ROUTE FAI 74 **DESCRIPTION** 081-0099, 0100 P92-032-01 I-74 over 19th Street, north of 12th Avenue **LOGGED BY** J. Wenzel

SECTION 81-1HB **LOCATION** Moline Twp. - 32SE, **SEC.** , **TWP.** 18N, **RNG.** 1W

COUNTY Rock Island **CORING METHOD**

STRUCT. NO. 081-0099, 0100 **CORING BARREL TYPE & SIZE**
Station

Core Diameter 2 in
Top of Rock Elev. 588.10 ft
Begin Core Elev. 585.60 ft

BORING NO. B-1
Station 52+08
Offset 0.00ft off BL - 19th St.
Ground Surface Elev. 614.60 ft

Description	Elev.	DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
Dolomite: gray-buff, aphanitic, dense, top-half mostly fractured, with clay film and minor pitting. t.s.f.: 582.5 to 581.6	585.60 580.60	-30	1	60	22	4.4	795
Dolomite: as above, though mostly solid and thickly bedded. t.s.f.: 578.1 to 577.2	580.60 575.60	-35	2	100	70	4.2	900
End of Boring	575.60	-40 -45					

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

Illinois Department of Transportation
Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Date 2/19/11

ROUTE FAI 74 **DESCRIPTION** 081-0099, 0100 P92-032-01 I-74 over 19th Street, north of 12th Avenue **LOGGED BY** M. Jacoby

SECTION 81-1HB **LOCATION** Moline Twp. - 32SE, **SEC.** , **TWP.** 18N, **RNG.** 1W

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

STRUCT. NO. 081-0099, 0100
Station

BORING NO. B-2
Station 49+75
Offset 0.00ft off BL - 19th St.
Ground Surface Elev. 610.26 ft

Surface Water Elev. ft
Stream Bed Elev. ft

Groundwater Elev.:
First Encounter Dry ft
Upon Completion Dry ft
After 24 **Hrs.** 590.8 ft ▽

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
STIFF gray SILTY CLAY LOAM	608.26			1.8 P	14
VERY STIFF gray/brown SILTY CLAY LOAM	606.76		5 7 9	2.7 S	15
STIFF brown SILTY CLAY LOAM	604.26	-5	2 4 14	1.2 B	15
VERY STIFF dark brown SILTY CLAY LOAM	601.76		4 5 5	2.3 P	21
STIFF dark brown SILTY CLAY LOAM	599.26	-10	0 3 3	1.0 B	16
STIFF brown SANDY LOAM with GRAVEL	596.26		4 6 7	1.5 P	
No recovery, rock blocking sampler	594.26	-15	12 15 7		
No recovery	591.26		2 4 7		
▽		-20	4		

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF gray CLAY LOAM TILL *(continued)*	589.26		7 8	3.0 B	13
VERY STIFF gray CLAY LOAM TILL	586.76		3 6 10	2.7 B	13
VERY STIFF gray CLAY LOAM TILL	584.26	-25	2 6 9	2.7 B	13
VERY STIFF gray CLAY LOAM TILL	581.76		2 6 10	2.2 B	14
VERY STIFF gray CLAY LOAM TILL	579.26	-30	4 12 25	2.3 S	17
STIFF gray CLAY TILL with DOLOMITE lenses	576.76		15 15 6	1.3 P	27
STIFF gray CLAY TILL	575.26	-35	100/6"		52
Borehole continued with rock coring.					
		-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

ROCK CORE LOG

Date 2/19/11

ROUTE FAI 74 DESCRIPTION 081-0099, 0100 P92-032-01 I-74 over 19th Street, north of 12th Avenue LOGGED BY M. Jacoby

SECTION 81-1HB LOCATION Moline Twp. - 32SE, **SEC.** , **TWP.** 18N, **RNG.** 1W

COUNTY Rock Island CORING METHOD

STRUCT. NO. 081-0099, 0100 CORING BARREL TYPE & SIZE
Station

Core Diameter 2 in
Top of Rock Elev. 575.26 ft
Begin Core Elev. 575.26 ft

BORING NO. B-2
Station 49+75
Offset 0.00ft off BL - 19th St.
Ground Surface Elev. 610.26 ft

Description	Elev.	DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
Dolomite: gray-buff, aphanitic, dense, pitted and mostly fractured with voids evident. t.s.f.: 572.9 to 572.5	575.26		1	85	15	2.2	228
	570.26	-40					
Dolomite: as above, pitted, fractured with macro-voiding apparent throughout.			2	30	0	2	
	565.26	-45					
End of Boring							
		-50					
		-55					

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

Illinois Department of Transportation
Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Date 3/22/11

ROUTE FAI 74 **DESCRIPTION** 081-0099, 0100 P92-032-01 I-74 over 19th Street, north of 12th Avenue **LOGGED BY** W. Garza

SECTION 81-1HB **LOCATION** Moline Twp. - 32SE, **SEC.** , **TWP.** 18N, **RNG.** 1W

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

STRUCT. NO. 081-0099, 0100
Station

BORING NO. B-5
Station 79+98
Offset 37.00ft Lt BL - SB Ramp
Ground Surface Elev. 613.1 ft

Surface Water Elev. ft
Stream Bed Elev. ft

Groundwater Elev.:
First Encounter 568.1 ft ▼
Upon Completion Wash ft
After Hrs. ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
MEDIUM light brown SILTY CLAY LOAM	610.60			0.5 P	13
MEDIUM light brown SILTY CLAY LOAM	609.10		2 4 4	0.5 P	15
		-5			
STIFF gray/brown SILTY CLAY LOAM	606.60		2 3 4	1.2 B	17
MEDIUM gray SILTY CLAY LOAM	604.10		2 2 3	0.6 P	20
		-10			
HARD tan CLAY LOAM	601.60		7 6 7	5.4 B	12
STIFF gray SILTY LOAM	599.10		2 4 5	1.1 S	15
		-15			
STIFF brown SILTY CLAY LOAM	596.60		3 5 7	2.0 B	20
MEDIUM gray SILTY CLAY LOAM	593.60		3 3 5	0.8 P	21
		-20			

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
VERY STIFF tan CLAY LOAM TILL	591.60		3 6 7	2.7 B	16
VERY STIFF tan CLAY LOAM TILL	589.10		5 6 9	2.5 B	15
		-25			
VERY STIFF tan/gray CLAY LOAM TILL	586.60		3 5 9	2.7 B	15
VERY STIFF gray CLAY LOAM TILL	584.10		4 6 9	2.5 B	15
		-30			
VERY STIFF gray CLAY LOAM TILL	581.60		4 6 9	2.5 B	15
VERY STIFF gray CLAY LOAM TILL	579.10		4 6 10	2.1 B	15
		-35			
VERY STIFF gray CLAY LOAM TILL	576.60		4 5 9	2.5 B	16
HARD gray CLAY LOAM TILL	574.10		5 11 15	5.4 B	18
		-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 3/22/11

ROUTE FAI 74 DESCRIPTION 081-0099, 0100 P92-032-01 I-74 over 19th Street, north of 12th Avenue LOGGED BY W. Garza

SECTION 81-1HB LOCATION Moline Twp. - 32SE, SEC. , TWP. 18N, RNG. 1W

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

STRUCT. NO. 081-0099, 0100
Station

BORING NO. B-5
Station 79+98
Offset 37.00ft Lt BL - SB Ramp
Ground Surface Elev. 613.1 ft

Surface Water Elev. ft
Stream Bed Elev. ft

Groundwater Elev.:
First Encounter 568.1 ft ▼
Upon Completion Wash ft
After Hrs. ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
HARD gray CLAY LOAM TILL			6		
			9	5.7	18
	571.60		14	B	
VERY STIFF gray CLAY LOAM TILL with SILTY SAND lens			4		
			8	3.1	18
	569.10		13	B	
		▼-45			
MEDIUM gray clean medium coarse SAND			0		
			5		
	566.10		7		
MEDIUM gray clean medium coarse SAND with CLAY lens			3		
			5	4.0	12
	563.60		11	P	
		-50			
VERY DENSE gray weathered SHALE with COAL lens			40		
	561.60		100/8"		
	560.60				
Wash VERY DENSE olive-green SANDSTONE with DOLOMITE fragments Auger Refusal @ 52.5' End of Boring			100/1"		
		-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)

Illinois Department of Transportation
Division of Highways
CH2M HILL

SOIL BORING LOG

Date 3/28/08

ROUTE I-74 **DESCRIPTION** I-74 SB Near 7th Avenue **LOGGED BY** B. Karnik

SECTION I-74 Bridge over Mississippi River **LOCATION** (N=562235.7741, E=2459668.0033), **SEC.** 32, **TWP.** 18N, **RNG.** 1W

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

STRUCT. NO. ____
Station ____

BORING NO. ILR0701
Station ____
Offset ____
Ground Surface Elev. 629.30 ft

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

Groundwater Elev.:
First Encounter 581.3 ft ▼
Upon Completion ____ ft
After ____ **Hrs.** ____ ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
7" Thick ACC followed by gravel subbase to 1.0'	628.70				
Silty Sandy Clay with Gravel,			2		
greenish brown, moist, low			2		
plasticity, stiff, with subangular to subrounded gravel embedded			10		
throughout, fill/subbase	625.30				
Sandy Clay Trace Gravel, dark			4		
gray, frozen, stiff, with subangular		-5	5		
to subrounded fine to coarse			4		
gravel embedded throughout, fill			5		
			3		
			5	3.0	
			6	to	
	621.30		6	4.0	
Silty Clay with Gravel, gray, moist,			2	P	
soft to medium stiff, high plasticity,			2	2.0	15.5
trace gravel, possible fill			3	P	
		-10	3		
				1.5	
				P	
	615.80				
Sandy Lean Clay Trace Gravel,			3		
gray, moist, stiff, medium			2	2.0	16.0
plasticity, fill or disturbed till		-15	3	P	
	610.80				
Same As Above, turning grayish			3		
brown at bottom 3", piece of wood			4		
embedded, possible fill		-20	7		

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Same As Above, turning grayish brown at bottom 3", piece of wood embedded, possible fill *(continued)*					
	605.80				
Sandy Lean Clay Trave Gravel,			5		
brown, moist, stiff, low plasticity,			6	3.0	
possible weathered till		-25	6	P	
	600.80				
Same as Above, gray, then brown,			6		
split in almost vertical with reddish			7	2.5	15.0
brown surface, weathered till		-30	8	to	
				3.5	
				P	
	595.80				
Sandy Lean Clay Trace Gravel,			4		
gray, moist, stiff, low plasticity,			6	2.5	
unweathered till		-35	6	to	
				3.0	
				P	
			5		
			6	2.5	
		-40	9	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

Date 3/28/08

ROUTE I-74 **DESCRIPTION** I-74 SB Near 7th Avenue **LOGGED BY** B. Karnik

SECTION I-74 Bridge over Mississippi River **LOCATION** (N=562235.7741, E=2459668.0033), **SEC.** 32, **TWP.** 18N, **RNG.** 1W

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

STRUCT. NO.
Station

BORING NO. ILR0701
Station
Offset
Ground Surface Elev. 629.30 **ft**

Surface Water Elev. ft
Stream Bed Elev. ft

Groundwater Elev.:
First Encounter 581.3 ft ▼
Upon Completion ft
After Hrs. ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Sandy Lean Clay Trace Gravel, gray, moist, stiff, low plasticity, unweathered till *(continued)*					
		-45			
	580.80	▼			
Top 3" is same as above Bottom 12" is Poorly Graded Sand, gray, wet, medium dense, fine to medium sand seam followed by 3" of gray sandy lean clay, trace gravel, till			12		
			16		
	579.30	-50	12		
End of Boring					
		-55			
		-60			

DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
-65			
-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
CH2M HILL

SOIL BORING LOG

Date 10/5/07

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=561907.847, E=2459825.874), **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. ILR0801
Station ______
Offset ______
Ground Surface Elev. 623.02 ft

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

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

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Grass Matter followed by silty clay with sands and topsoil	622.02				
Silty Clay With Sand(CL-ML) dark brown with brown, dry to moist, non plastic, little to few coarse to fine sands, strong cementation, occasional reddish brick fragments, possible fill	620.02		4 4 5 5		
Lean Clay With Sand(CL) medium brown, dry to moist, low plasticity, medium stiff, little to few coarse to fine sands, dark brown silty pocket at top of sample, possible fill	617.02	-5	3 3 3 3		
Sandy Lean Clay(CL) olive gray with medium brown and gray, dry to moist, medium stiff, few coarse to fine sands, trace fine subangular to subrounded gravels, dark gray with occasional root matter at bottom of sample	615.02		1 3 5 6	3.75-4.0 P	
Sandy Lean Clay With Gravel (CL) medium brown with gray, dry, strongly cemented, stiff, crumbly, few coarse to fine sands, little to trace of medium to fine gravels, occasional medium to fine sand seams scattered throughout, dark gray with heavy ________ matter at top 2" of sample, possible old topsoil followed by native soil Rimac: Pu = 68 lbs		-10	3 4 5 6	1.3	
same as above, medium brown, dry to moist, stiff, strongly cemented, glacial till			2 3 5 7	4.3 P	
same as above, medium brown to brown, stiff, strongly cemented, dry, glacial till	605.02	-15	3 4 5 6	4.5 P	
		-20	3 5 7 10	4.0-4.5 P	

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Sandy Lean Clay(CL) medium brown with orange brown, dry, non plastic, stiff, few coarse to fine sands, frequent sand seams, approximately 1/8"-1/4" thick at center and bottom of sample, sand seams of medium to fine sands, oxidized, possible weathered till with scattered sand seams *(continued)* medium brown with gray, mottled with orange brown, dry, stiff, few coarse to fine sands, very oxidized, small pockets of dark gray to black coal like deposits in middle of sample, possible weathered glacial till Rimac: Pu = 100 lbs		-25	3 5 7 10	1.9 B	
olive gray with light brown, dry to moist, slightly oxidized at top, stiff, possible unweathered glacial till	590.02	-30	3 5 6 9	3.8 P	
Lean Clay With Sand(CL) uniform gray, dry to moist, stiff, little to few coarse to fine sands, scattered sand pockets, possible unweathered glacial till Rimac: Pu = 70 lbs		-35	3 5 7 9	1.3	
uniform gray, dry to moist, stiff, little to few coarse to fine sands, scattered sand pockets, possible unweathered glacial till	583.52 583.02	-40	2 4 8 12		

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

Date 10/5/07

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=561907.847, E=2459825.874), **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. ILR0801
Station
Offset
Ground Surface Elev. 623.02 **ft**

Surface Water Elev. ft
Stream Bed Elev. ft

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

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Clayey Sand With Silt(SC) gray, moist to wet, medium dense, clay with medium to fine sands, possible residual soil				
End of Boring				
	-45			
	-50			
	-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)

Illinois Department of Transportation
Division of Highways
CH2M HILL

SOIL BORING LOG

Date 10/9/07

ROUTE I-74 **DESCRIPTION** New I-74 Bridge Over Mississippi River - Illinois Approach **LOGGED BY** Bose, Kaustav/SCO

SECTION I-74 Bridge over Mississippi River **LOCATION** (N=561497.653, E=2459812.286), **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. ILR0804
Station
Offset
Ground Surface Elev. 641.39 ft

Surface Water Elev. ft
Stream Bed Elev. ft

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

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Clay (CL) gray, moist, very stiff to hard, trace sand and fine angular gravel				
		1		
		2	4.1	
		5	S	
		2		
	-5	7	2.5	
		9	P	
635.39				
Clay (CL) gray to greenish gray, moist, trace fine to coarse grained sand, stiff to hard		1		
		2		
		2		
	-10			
RIMAC: Pu = 31lbs (Bulging)		2		
		2	1.9	
		4	B	
	-15			
RIMAC: Pu = 68lbs (bulging)		2		
		4	4.1	
		6	B	
	-20			

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Clay (CL) gray to greenish gray, moist, trace fine to coarse grained sand, stiff to hard *(continued)*				
		3		
		4	3.0	
		9	P	
	-25			
		3		
		6	4.5	
		8	P	
	-30			
shale in tip		4		
		10	4.5	
		26	P	
606.39	-35			
End of Boring				
	-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

HANSON

Date 6/22/10

ROUTE F.A.I. 74 **DESCRIPTION** I-74 Over Mississippi River **LOGGED BY** JMB

SECTION 81-1-2 **LOCATION** SE¼ of SEC. 32, TWP. 18N, RNG. 1W, 4th P.M.

COUNTY Rock Island **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** Auto

STRUCT. NO. 081-6015
Station
BORING NO. RW 06-1
Station 61+02
Offset 7' Lt.
Ground Surface Elev. 611.3 ft

Surface Water Elev.
Stream Bed Elev.
Groundwater Elev.:
First Encounter 593.8 ft ▼
Upon Completion ft
After Hrs. ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
CONCRETE	610.80				
FILL - Light gray, slightly moist, SILT				2.50P	14
		2			14
					17
	608.30				
FILL - Very dark brown, moist, clayey SILT with trace gravel		4		1.80P	13
	605.30	6			
FILL - Gray, moist, medium dense, silty, medium-grained SAND with trace gravel, wood, brick and rock fragments			5 6 11	2.00P	15
		8			
			11 23 50/4"		12
		10			
	600.30				
Dark brown, moist, stiff, sandy SILT with trace gravel					20
		12		1.65S	17
		14	11 4 4		16
	596.30				
Dark brown, moist, sandy, clayey SILT with trace gravel	595.30	16			
Dark brown, wet, dense, silty SAND with trace gravel				0.50P	12
▼	593.30	18			
Gray and brown, moist, medium stiff, silty CLAY with sand and trace gravel			3 3 5	0.54B	18
		20			
Gray and brown, moist, medium stiff, silty CLAY with sand and trace gravel *(continued from previous page)*		22			
	588.80				
Gray and brown, moist, very stiff, silty CLAY with sand and gravel		24	8 10 11	2.61B	14
	586.30				
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)

HANSON **SOIL BORING LOG**

Date 6/25/14

ROUTE F.A.I. 74 DESCRIPTION I-74 Over Mississippi River LOGGED BY RPD

SECTION 81-1-2 LOCATION SE¼ of SEC. 32, TWP. 18N, RNG. 1W, 4th P.M.

COUNTY Rock Island DRILLING METHOD Continuous Flight Auger HAMMER TYPE Auto

STRUCT. NO. 081-6015
Station
BORING NO. RW 06-04
Station 60+68
Offset 63' Rt.
Ground Surface Elev. 624.7 ft

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

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
3.0" CONCRETE.	624.45				
FILL - Dark brown with gray mottles, SILT, little clay, trace sand.		2	2 3 2	4.50P	13
		4	3 4 7	4.50P	19
		6–8	4 5 8	3.00P	16
		10	6 11 11	4.50P	16
	613.70				
FILL - Dark gray silty lean CLAY, little sand, trace gravel, with wood fragments.		12	3 5 6	3.30P	18
		14		2.16B	16
		16–18	3 4 5	3.00P	19
		20	2 5	2.50P	22
FILL - Dark gray silty lean CLAY, little sand, trace gravel, with wood fragments. *(continued from previous page)*			6		
	602.70	22			
Gray moist, very stiff, silty lean CLAY, with trace sand and gravel.		24	3 5 10	4.50P	14
		28–30	5 7 10	4.30P	15
		34	5 8 10	4.00P	14
	589.70				
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)

HANSON SOIL BORING LOG

Date 6/25/14

ROUTE F.A.I. 74 **DESCRIPTION** I-74 Over Mississippi River **LOGGED BY** RPD

SECTION 81-1-2 **LOCATION** SW¼ of SEC. 33, TWP. 18N, RNG. 1W, 4th P.M.

COUNTY Rock Island **DRILLING METHOD** Continuous Flight Auger **HAMMER TYPE** Auto

STRUCT. NO. 081-6015
Station
BORING NO. RW 06-05
Station 62+58
Offset 22' Rt.
Ground Surface Elev. 644.6 ft

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

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
3.0" TOPSOIL.	644.35				
FILL - Brown lean CLAY, trace silt, trace sand, with organics.		2	3 7 7	3.50P	15
		4	1 2 5	1.75B	23
		6	5 8 10	3.50P	17
	636.10	8			
FILL - Brown and gray silty lean CLAY, trace sand, trace gravel, with wood debris and brick fragments.		10	3 5 6	3.10B	16
		12		1.60S	18
				1.55S	19
		14	3 6 9	3.30S	16
		16	4 7 9	4.46S	16
		18	5 7	4.50P	16
		20			

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
FILL - Brown and gray silty lean CLAY, trace sand, trace gravel, with wood debris and brick fragments. *(continued from previous page)*			9		
		22			
		24	5 8 12	2.52S	16
		26			
		28	7 9 11	3.30S	18
		30			
		32			
		34	7 8 13	4.50P	16
		36			
	607.60				
FILL - Gray clayey SILT, little sand, trace gravel, with red brick fragments.		38	3 6 9	2.50P	22
		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)

BBS, from 137 (Rev. 8-99)

HANSON SOIL BORING LOG

Date 6/25/14

ROUTE F.A.I. 74 DESCRIPTION I-74 Over Mississippi River LOGGED BY RPD

SECTION 81-1-2 LOCATION SW¼ of SEC. 33, TWP. 18N, RNG. 1W, 4th P.M.

COUNTY Rock Island DRILLING METHOD Continuous Flight Auger HAMMER TYPE Auto

STRUCT. NO. 081-6015
Station
BORING NO. RW 06-05
Station 62+58
Offset 22' Rt.
Ground Surface Elev. 644.6 ft

Surface Water Elev.
Stream Bed Elev.

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

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
FILL - Gray clayey SILT, little sand, trace gravel, with red brick fragments. *(continued from previous page)*		42			
		44	5 7 9		10
	597.60	46			
Gray moist, very stiff, silty lean CLAY, with trace sand and trace gravel.		48			
		50	5 7 10	3.30S	15
		52			
		54	6 11 15	6.01B	12
		56			
		58	7 11 15	3.69B	15
	584.60	60			

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)

HANSON SOIL BORING LOG

Date 6/23/14

ROUTE	F.A.I. 74	**DESCRIPTION**	I-74 Over Mississippi River	**LOGGED BY**	RPD
SECTION	81-1-2	**LOCATION**	SE¼ of SEC. 32, TWP. 18N, RNG. 1W, 4th P.M.		
COUNTY	Rock Island	**DRILLING METHOD**	Continuous Flight Auger	**HAMMER TYPE**	Auto

STRUCT. NO. 081-6016
Station
BORING NO. RW 07-02
Station 57+08
Offset 14' Lt.
Ground Surface Elev. 631.2 ft

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

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
6" ASPHALT.	630.70				
FILL - Brown to light brown clayey SILT, trace gravel, trace sand.		2	1 2 3	0.50P	15
		4	5 6 8	4.50P	10
		6			
- sand seam @ 7.0'.		8	6 8 15		14
	622.20				
Brown and gray silty lean CLAY, trace sand, trace gravel.		10	4 5 7	3.00P	13
				1.75B	14
		12		3.88B	13
				1.84B	14
		14	3 5 9	2.70P	14
	615.20	16			
Gray moist, very stiff, silty lean CLAY, with trace sand and gravel.		18	5 7 12	4.30P	14
		20	3 5	3.30P	15

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Gray moist, very stiff, silty lean CLAY, with trace sand and gravel. *(continued from previous page)*		12		
	22			
	24	3 6 9	2.70P	15
	26			
	28			
	30	4 6 9	3.00P	14
	32			
	34	3 5 7	1.70P	16
	36			
	38			
	40	3 6 9	2.20P	16

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

HANSON **SOIL BORING LOG**

Date 6/23/14

ROUTE F.A.I. 74 **DESCRIPTION** I-74 Over Mississippi River **LOGGED BY** RPD

SECTION 81-1-2 **LOCATION** SE¼ of SEC. 32, TWP. 18N, RNG. 1W, 4th P.M.

COUNTY Rock Island **DRILLING METHOD** Continuous Flight Auger **HAMMER TYPE** Auto

STRUCT. NO. 081-6016
Station
BORING NO. RW 07-02
Station 57+08
Offset 14' Lt.
Ground Surface Elev. 631.2 ft

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

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Gray moist, very stiff, silty lean CLAY, with trace sand and gravel. *(continued from previous page)*				
	44	6 7 12	2.30P	15
	49	5 8 12	3.30P	14
	54	6 8 11	2.70P	14
	59	6 11 17	2.30P	14

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Gray moist, very stiff, silty lean CLAY, with trace sand and gravel. *(continued from previous page)*					
- coarse sand seam @ 64.3 to 65.0'.		64	14 22 32	3.30P	19
Gray SHALE.	563.70	67.5			
		69	24 50/5"	4.50P	14
End of Boring	561.20	70			

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

HANSON SOIL BORING LOG

Date 6/23/14

ROUTE F.A.I. 74 DESCRIPTION I-74 Over Mississippi River LOGGED BY RPD

SECTION 81-1-2 LOCATION SW¼ of SEC. 33, TWP. 18N, RNG. 1W, 4th P.M.

COUNTY Rock Island DRILLING METHOD Continuous Flight Auger HAMMER TYPE Auto

STRUCT. NO. 081-6016
Station
BORING NO. RW 07-03
Station 58+25
Offset 60' Lt.
Ground Surface Elev. 629.1 ft

Surface Water Elev.
Stream Bed Elev.

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

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
3.0" TOPSOIL. 628.85				
FILL - Brown silty lean CLAY, trace sand, trace gravel, with limestone fragments.	2	4 6 7	4.50P	12
	4	4 4 7	3.70P	11
	6	3 5 8		
620.60	8			
Brown silty lean CLAY, little sand, trace small gravel.			1.75B	14
			1.90B	14
	10			
	12	2 2 3	3.70P	13
	14			18
			1.90B	17
	16	4 7 11	4.65S	13
	18	4 6	3.69B	12
	20			

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Brown silty lean CLAY, little sand, trace small gravel. *(continued from previous page)*		10		
	22			
	24	4 6 10	3.10B	14
603.10	26			
Gray, moist, very stiff, silty lean CLAY, with trace sand and trace gravel.	28	4 6 10	4.07B	14
	30			
	32			
	34	6 8 11	3.88B	13
594.10				
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)

Boring 19BR-105

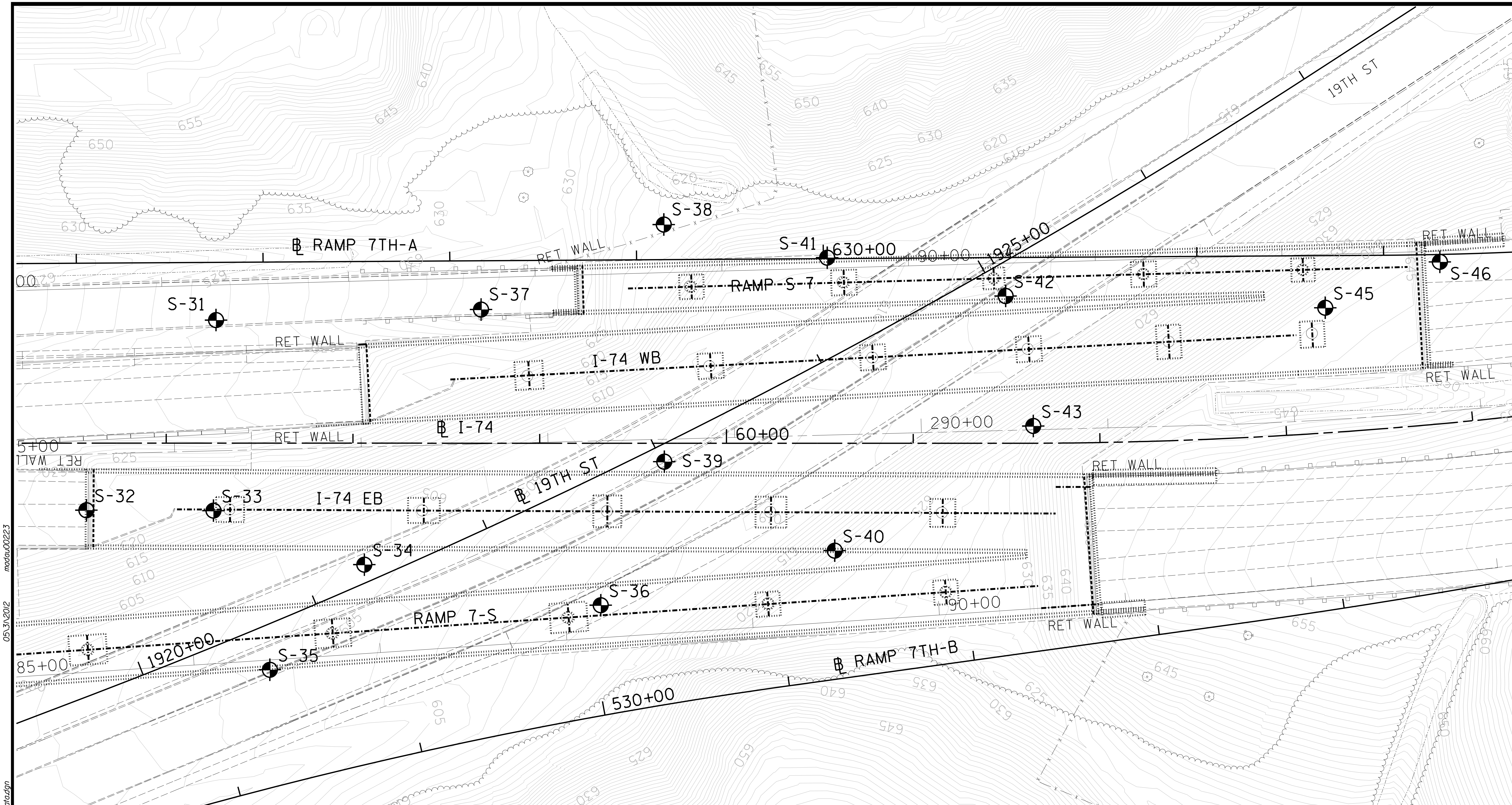
Run	Depth (ft)	REC (%)	RQD (%)
1	35.3 – 40.7	46	8
2	40.7 – 42.9	81	0
3	42.9 – 45.8	43	0
4	45.8 – 50.8	77	35

Boring 19BR-106			
Run	Depth (ft)	REC (%)	RQD (%)
1	27.0 – 30.8	91	46
2	30.8 – 35.8	100	63
3	35.8 – 37.3	100	75

Boring 19BR-108			
Run	Depth (ft)	REC (%)	RQD (%)
1	37.7 – 40.9	77	0
2	40.9 – 45.9	93	23
3	45.9 – 47.9	100	45

Boring 19BR-109

Run	Depth (ft)	REC (%)	RQD (%)
1	30.5 – 35.8	86	60
2	35.8 – 42.3	91	74



LEGEND

RW600 BORING LOCATION

0 50 100

SCALE IN FEET

HANSON

BORING LOCATION PLAN

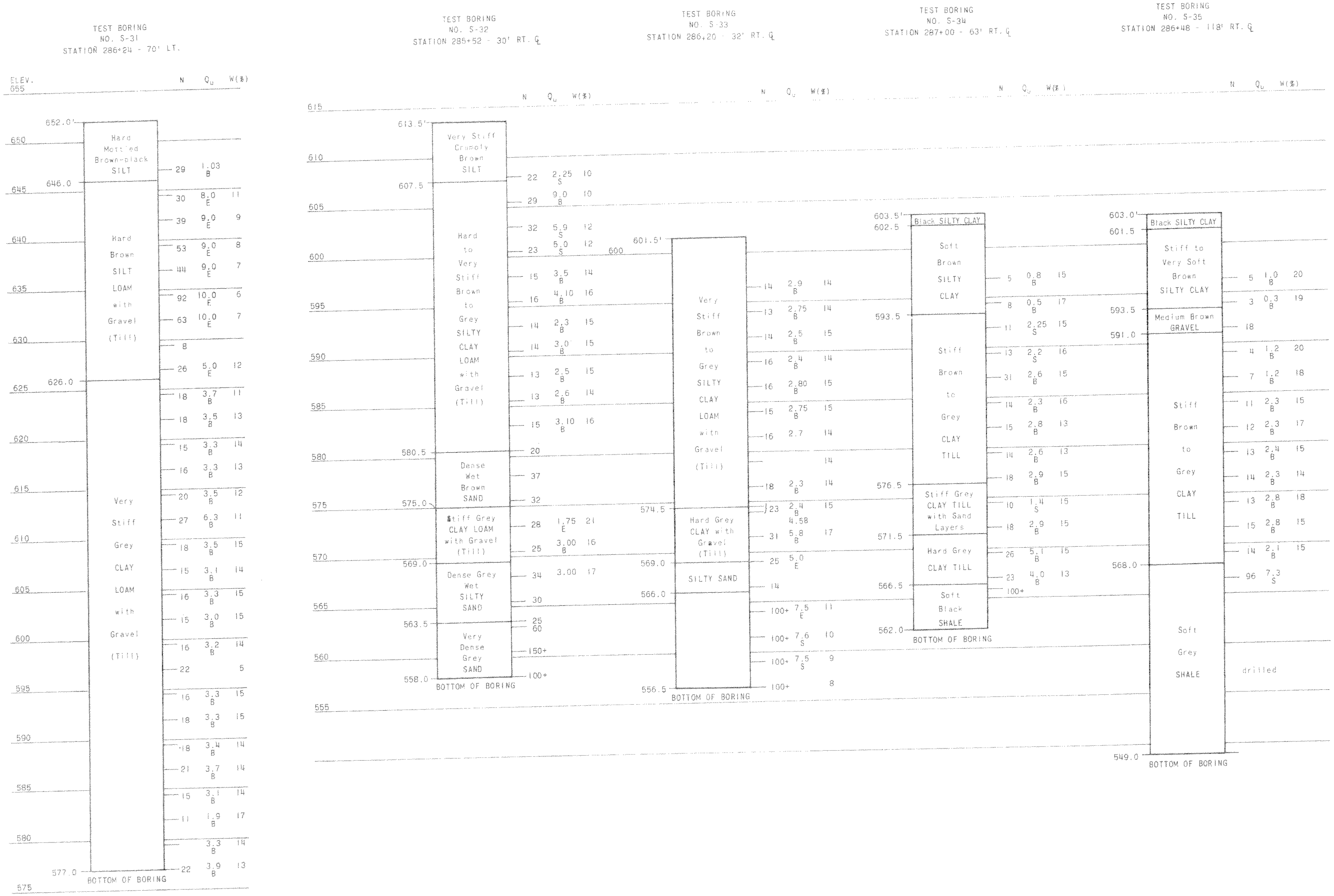
EXISTING I-74, RAMP 7-S, & RAMP S-7
OVER 19TH STREET
ROCK ISLAND COUNTY, ILLINOIS

08H0120E 5/30/12

0810179-A0326-000-Subsurface Data.dgn 05/31/2012 madau00223

ROUTE NO.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
F.A.I. 74	81-1HB	ROCK ISLAND	389	252
FED. ROAD DIST. NO. 7	ILLINOIS	FED. AID PROJECT I-74		

DWG. NO. B-4



TEST BORINGS

F.A.I. 74 - SECTION 81-1HB

F.A.I. 74 & RAMPS OVER RELOC. 19TH ST.

ROCK ISLAND COUNTY

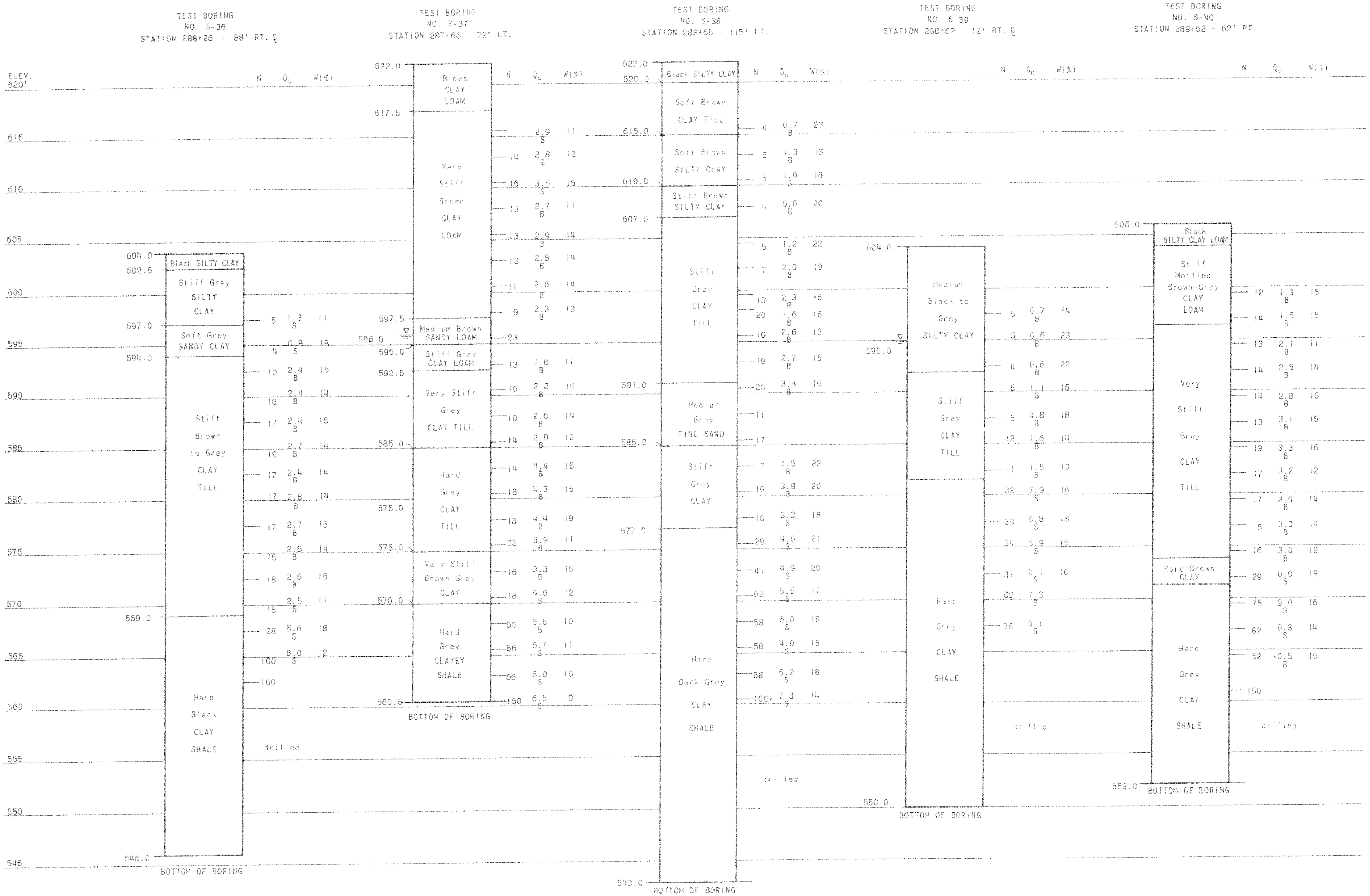
STATION 289 + 23.09

SCALE: AS NOTED DATE:

DE LEUW, CATHER & COMPANY ENGINEERS
DESIGNED BY M. VADKERTY
DRAWN BY H. DE PERCZEL
CHECKED G. C. WAY
IN CHARGE E. S. MARTINS
APPROVED W.G. HORN

ROUTE NO.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
F.A.I. 74	81-IHB	ROCK ISLAND	389	253
FED. ROAD DIST. NO. 7	ILLINOIS	FED. AID PROJECT I-74		

DWG. NO. B-5



TEST BORINGS

F.A.I. 74-SECTION 81-IHB

F.A.I. 74 & RAMPS OVER RELOC. 19TH ST.

ROCK ISLAND COUNTY

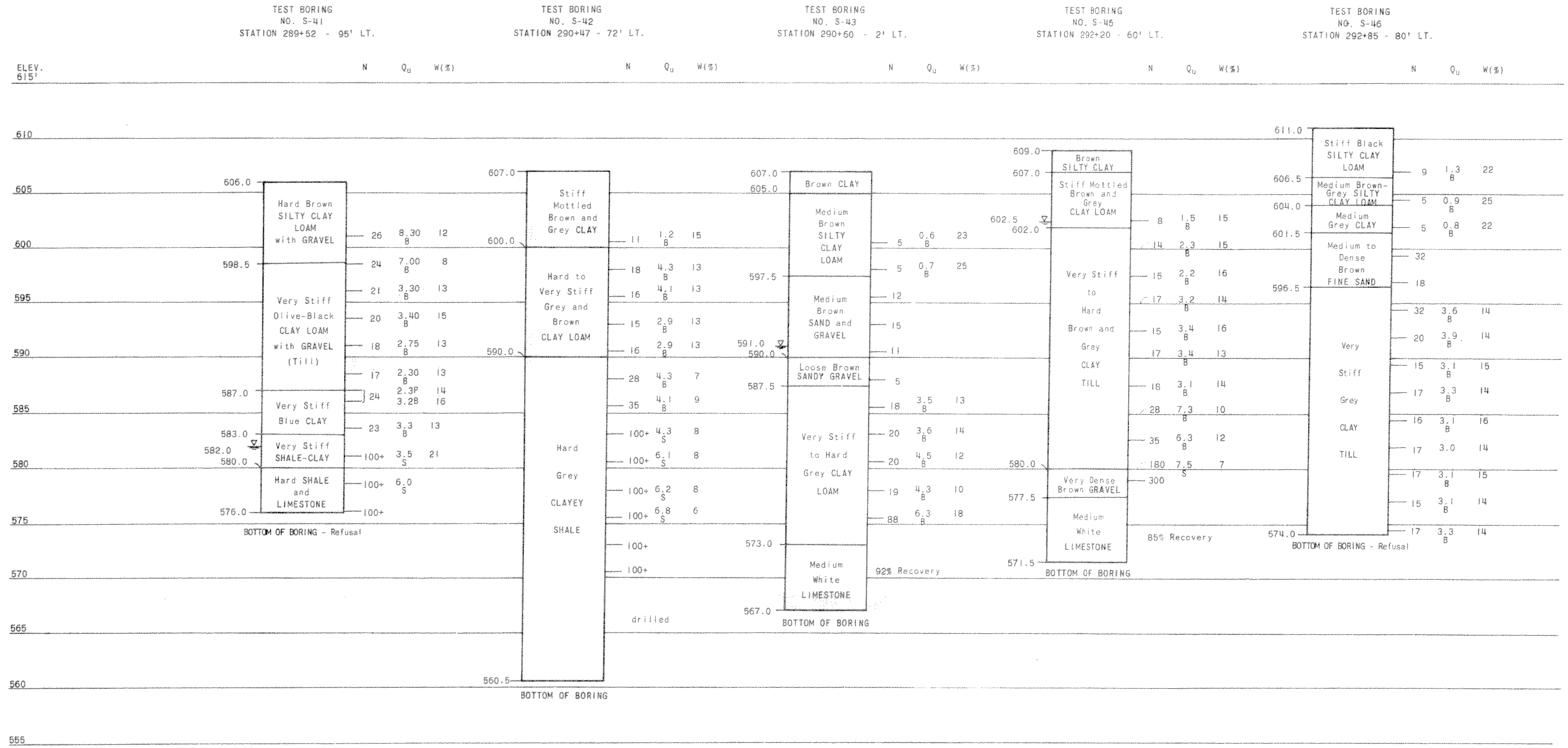
STATION 289+23.09

SCALE: AS NOTED DATE:

DE LEUW, CATHER & COMPANY ENGINEERS
DESIGNED BY M. VADKERTY
DRAWN BY H. DE PERCZEL
CHECKED G. C. WAY
IN CHARGE E. S. MARTINS
APPROVED W.G. HORN

ROUTE NO.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
F.A.I. 74	81-1HB	ROCK ISLAND	389	256
FED. ROAD DIST. NO. 7	ILLINOIS	FED. AID PROJECT I-74		

DWG. NO. B-6



DE LEUW, CATHER & COMPANY ENGINEERS
DESIGNED BY M. VADKERTY
DRAWN BY H. DE PERCZEL
CHECKED G. C. WAY
IN CHARGE E. S. MARTINS
APPROVED W.G. HORN

TEST BORINGS

F.A.I. 74 - SECTION 81-1HB

F.A.I. 74 & RAMPS OVER RELOC. 19TH ST.

ROCK ISLAND COUNTY

STATION 289+23.09

SCALE: AS NOTED DATE:

HORIZONTAL CURVE DATA

ITEM	CURVE 1	CURVE 2	CURVE 3	CURVE 4	CURVE 5
Δ	0°-49'-25"	19°-12'-00"	1°-44'-56"	2°-27'-13"	10°-34'-11"
D	2°-30'-00"	2°-30'-00"	2°-27'-56"	1°-00'-00"	4°-00'-00"
R	291.83'	2291.83'	2323.83'	5729.58'	1432.39'
T	16.47'	387.63'	35.47'	122.70'	133.55'
L	32.94'	768.00'	70.94'	245.37'	266.33'
L.C.	32.94'	764.41'	70.94'	245.35'	265.94'
P.C.STA.	92+67.06	291+41.68	8+29.05	90+54.63	47+62.50
P.I.STA.	92+83.53	295+29.31	8+64.53	91+77.33	48+96.04
P.T.STA.	93+00.00	299+09.68	8+99.99	93+00.00	50+28.82

WORKPOINTS STATION EQUATIONS

W.P.1 - STA. 86+67.80 RAMP 7-S = STA. 46+98.72 ℄ 19TH ST.
W.P.2 - STA. 6+34.89 ℄ S.B. = STA. 49+18.57 ℄ 19TH ST.
W.P.3 - STA. 7+77.71 ℄ N.B. = STA. 50+23.78 ℄ 19TH ST.
W.P.4 - STA. 90+75.66 RAMP S-7 = STA. 51+43.95 ℄ 19TH ST.

NOTES:

ALL DIMENSIONS SHOWN ARE BETWEEN POINTS ON A HORIZONTAL PLANE AT A TEMPERATURE OF 50°F.

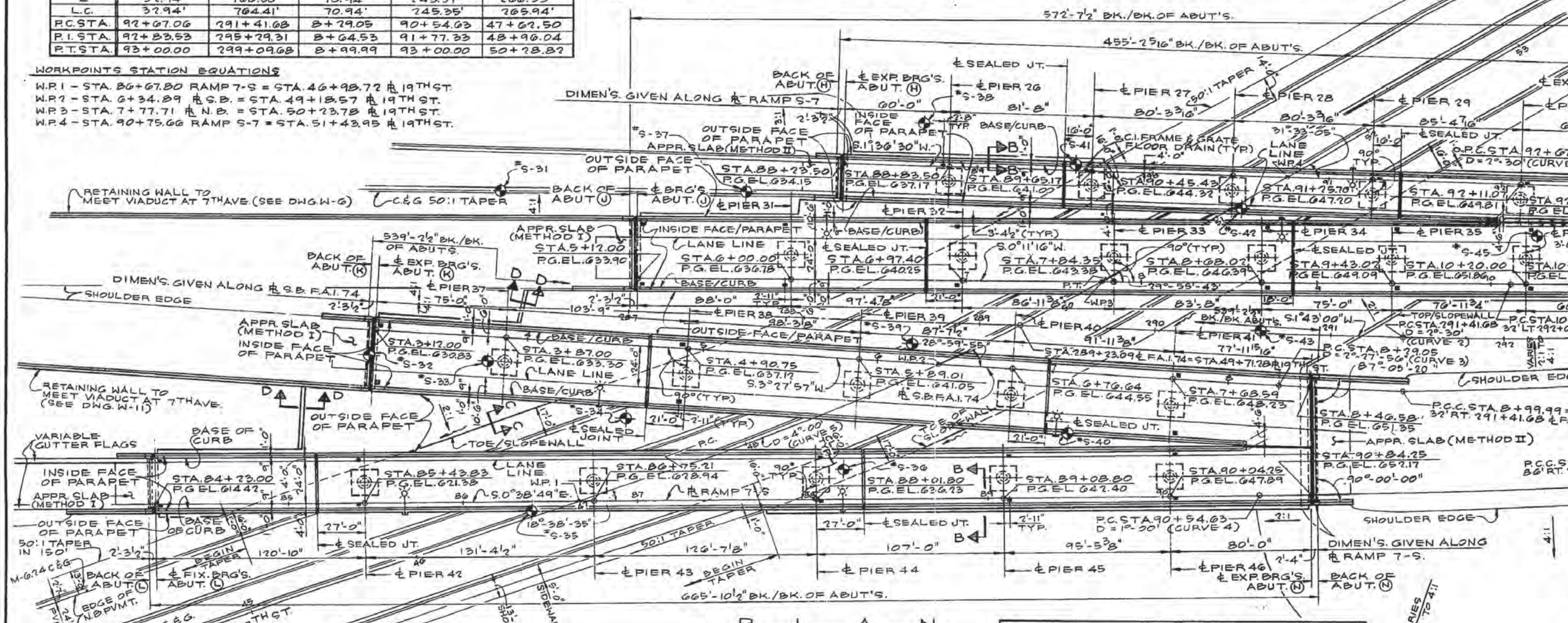
- INDICATES FLOOR DRAIN.
- INDICATES LOCATION OF BORING HOLES.
- INDICATES LIGHT STANDARD (BY OTHERS)

BENCH MARKS

A-4B - CONC. MONUMENT 77' RT. ℄ F.A.I. 74 EL. 600.660
A-4C - CONC. MONUMENT 110' LT. ℄ F.A.I. 74 EL. 605.080
NO EXISTING STRUCTURE.

ROUTE NO.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
F.A.I. 74	81-1HB	ROCK ISLAND	389	213
FED. ROAD DIST. NO. 7	ILLINOIS	FED. AID PROJECT I-74		

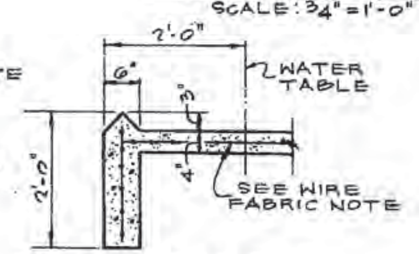
DWG. NO. S-173



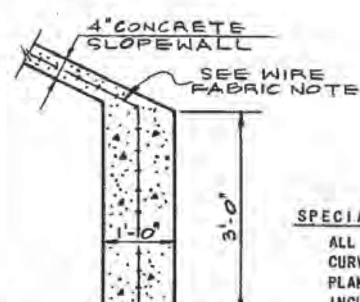
PLAN
SCALE: 1" = 30'-0"

SLOPEWALL SECTION AT ABUTMENTS
SCALE: 3/4" = 1'-0"

SECTION D-D
SCALE: 3/4" = 1'-0"



SECTION B-B
SCALE: 3/4" = 1'-0"



SECTION C-C
SCALE: 3/4" = 1'-0"

WIRE FABRIC NOTE

WELD WIRE FABRIC 6×6 MESH #4 WIRES WEIGHTING 58 POUNDS PER 100 SQUARE FEET INCLUDED IN CONTRACT UNIT PRICE FOR SLOPEWALL

SLOPEWALL LAYOUT AND QUANTITIES ARE INCLUDED IN HIGHWAY PLANS FOR SECTION 81-1-2.

* APPROACH SLAB PILES NOT INCLUDED. SEE APPROACH SLAB DRAWINGS.

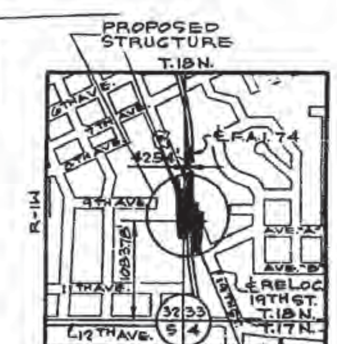
CALCULATED PLAN WEIGHT OF STRUCTURAL STEEL--3,258,390 LBS.

SPECIAL NOTE:

ALL PROFILE GRADE ELEVATIONS AND VERTICAL CURVE DATA GIVEN THROUGHOUT THE STRUCTURAL PLANS REFER TO TOP OF CONCRETE AND DO NOT INCLUDE THE 1 1/2" BITUMINOUS CONCRETE SURFACE COURSE.

TOTAL BILL OF MATERIALS - SEC. 81-1HB

ITEM	UNIT	SUPER-STRUCTURE	SUB-STRUCTURE	TOTAL
CLASS A EXCAVATION FOR STRUCTURES	CU. YD.	-	1,987	1,987
CLASS X CONCRETE	CU. YD.	2,484.9	1,298.6	3,783.5
CLASS A CONCRETE	CU. YD.	-	59.1	59.1
PROTECTIVE COAT	SQ. YD.	1,846	-	1,846
ALUMINUM RAILING	LIN. FT.	4,501	-	4,501
REINFORCEMENT BARS	POUND	600,840	119,497	720,337
BITUMINOUS CONCRETE SURFACE COURSE, CLASS I	TON	691	-	691
COAL TAR INTERLAYER PROTECTIVE COAT	SQ. YD.	8,190	-	8,190
FURNISHING CREOSOTED PILES UP TO 20 FEET*	LIN. FT.	-	820	820
FURNISHING CREOSOTED PILES 20.1 TO 30 FEET*	LIN. FT.	-	450	450
FURNISH AND ERECT STRUCTURAL STEEL	LUMP SUM	1	-	1
FURNISHING STEEL PILES (10BP42)	LIN. FT.	-	17,025	17,025
DRIVING TIMBER PILES*	LIN. FT.	-	1,070	1,070
DRIVING STEEL PILES	LIN. FT.	-	17,025	17,025
TEST PILE (TIMBER)	EACH	-	3	3
BRIDGE DRAINAGE SYSTEM, 19TH STREET	LUMP SUM	1	-	1
TEST PILE STEEL (10BP42)	EACH	-	29	29
NAME PLATES	EACH	3	-	3
STUD SHEAR CONNECTORS, 4"	EACH	15,183	-	15,183
PREFORMED JOINT SEALER 2½	LIN. FT.	251	-	251
MODULAR PREFORMED EXPANSION JOINT, 3"	LIN. FT.	85	-	85
MODULAR PREFORMED EXPANSION JOINT, 2"	LIN. FT.	217	-	217
IMPACT ATTENUATION DEVICE, 8 BAY, NARROW WIDTH	EACH	1	-	1



LOCATION PLAN

DESIGN LOADING:
HS 20-44 & ALT.

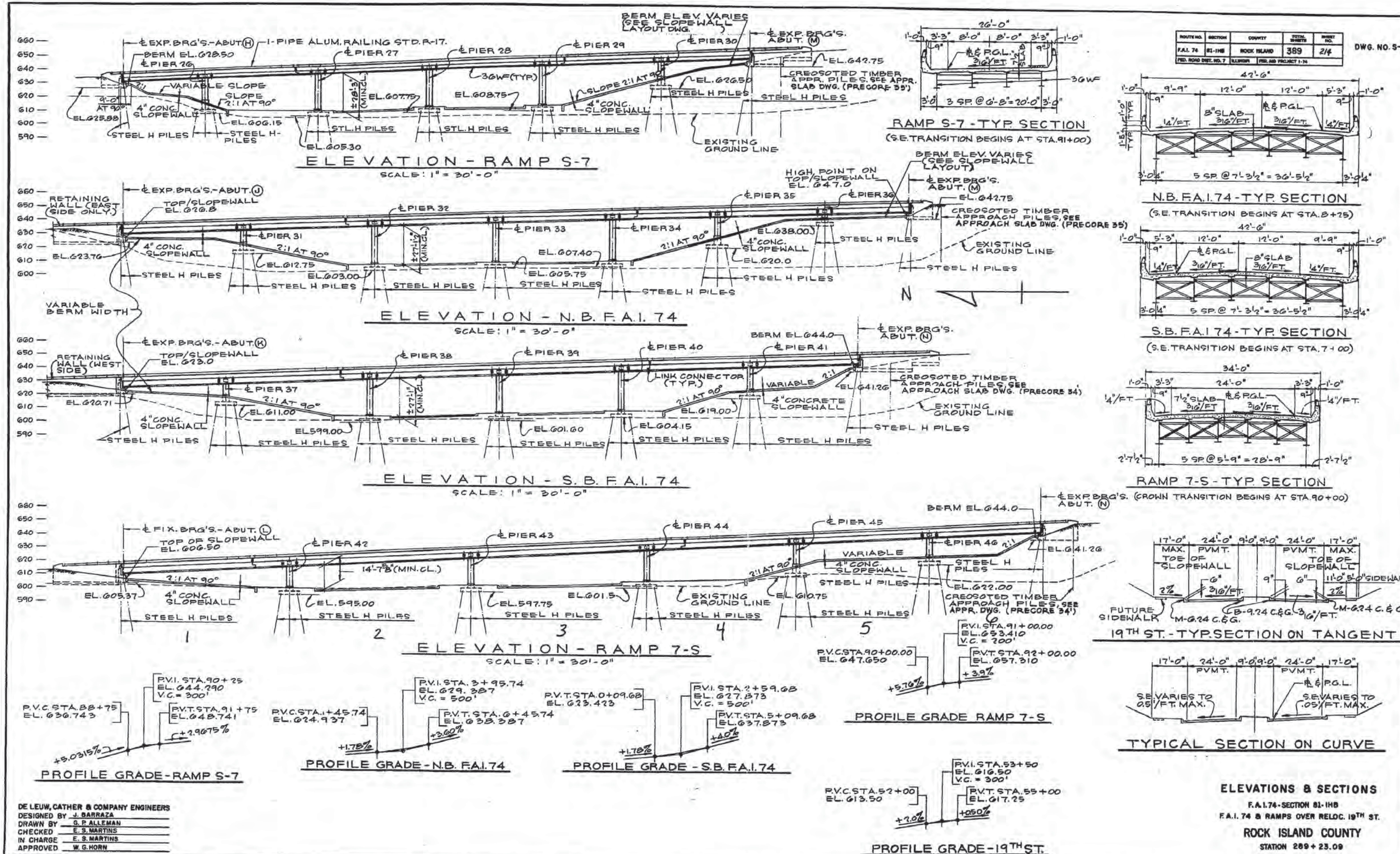
DESIGN STRESSES:

fc = 1200 P.S.I. SUPERSTRUCTURE DECK SLABS.
fc = 1400 P.S.I. CURBS, PARAPETS AND SUBSTRUCTURE.
fc = 1000 P.S.I. RETAINING WALLS.
fs = 20,000 P.S.I. REINF. BARS & STRUCT. STEEL (A-36).
v = 75 P.S.I. MAX. ALLOW. SHEAR IN FOOTINGS.
n = 10

ALLOWABLE L.L. DEFLECTION - L/1000 (NON-COMPOSITE), L/1200 (COMPOSITE)

DE LEUW, CATHER & COMPANY ENGINEERS
DESIGNED BY J. BARRAZA
DRAWN BY G. P. ALLEMAN
CHECKED E. S. MARTINS
IN CHARGE E. S. MARTINS
APPROVED W.G. HORN

GENERAL PLAN
F.A.I. 74 - SECTION 81-1HB
F.A.I. 74 & RAMPS OVER RELOC. 19TH ST.
ROCK ISLAND COUNTY
STATION 289 + 23.09
SCALE: AS NOTED DATE:



DE LEUW, CATHER & COMPANY ENGINEERS

DESIGNED BY J. BARRAZA
DRAWN BY G. P. ALLEMAN
CHECKED E. S. MARTINS
IN CHARGE E. S. MARTINS
APPROVED W. G. HORN

ELEVATIONS & SECTIONS

F.A.I. 74 - SECTION 81-1HB
F.A.I. 74 & RAMPS OVER RELOC. 19TH ST.

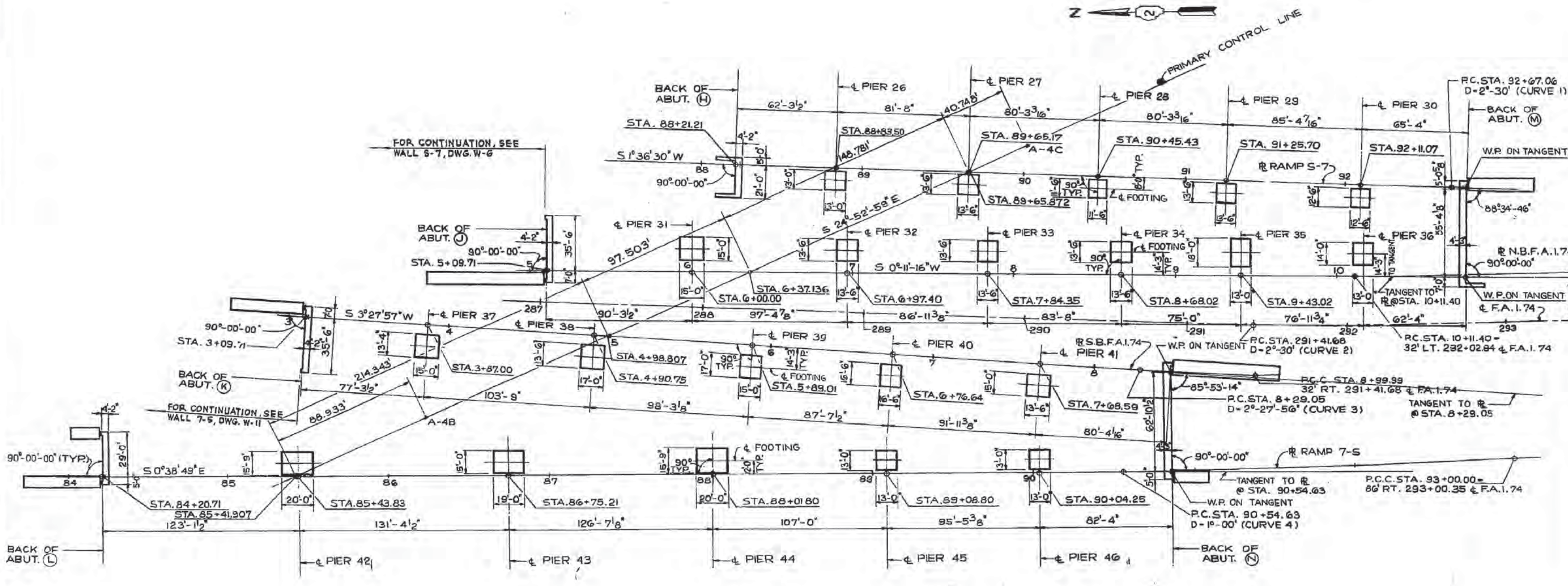
ROCK ISLAND COUNTY

STATION 289 + 23.09

SCALE: AS NOTED DATE:

DWG. NO. S-175

ROUTE NO.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
F.A.I. 74	81-1HB	ROCK ISLAND	389	215
FED. ROAD DIST. NO. 7	ILLINOIS	FED. AID PROJECT I-74		



FOUNDATION PLAN

SCALE: 1" = 30'-0"

DE LEUW, CATHER & COMPANY ENGINEERS
DESIGNED BY J.A. BARRAZA
DRAWN BY L. TROUSIL
CHECKED [illegible]
IN CHARGE E.S. MARTINS
APPROVED W.G. HORN

SUBSTRUCTURE LOCATION

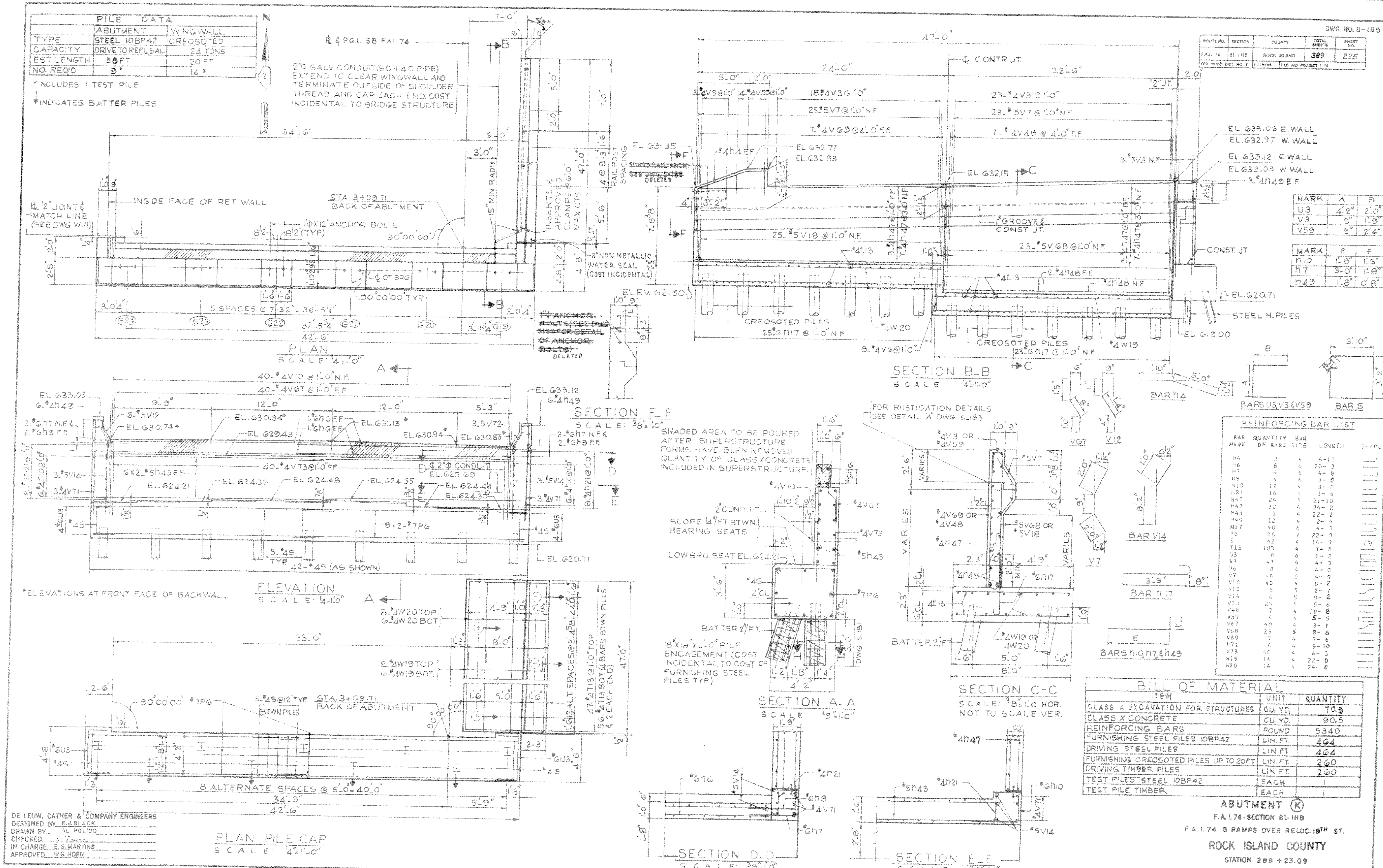
F.A.I. 74 - SECTION 81-1HB

F.A.I. 74 & RAMPS OVER RELOC. 19TH ST.

ROCK ISLAND COUNTY

STATION 289 + 23.09

SCALE: AS NOTED DATE:



DWG. NO. S-185

ROUTE NO.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
F.A.I. 74	81-1HB	ROCK ISLAND	389	225
FED. ROAD DIST. NO. 7	ILLINOIS	FED. AID PROJECT I-74		

PILE DATA		
	ABUTMENT	WINGWALL
TYPE	STEEL 10BP42	CREOSOTED
CAPACITY	DRIVE TO REFUSAL	24 TONS
EST. LENGTH	58 FT	20 FT.
NO. REQ'D	9*	14

*INCLUDES 1 TEST PILE

↓ INDICATES BATTER PILES

MARK	A	B
U3	4'-2"	2'-0"
V3	9"	1'-9"
V59	9"	2'-4"

MARK	E	F
n10	1'-8"	1'-6"
h7	3'-0"	1'-8"
h49	1'-8"	0'-8"

REINFORCING BAR LIST

BAR MARK	QUANTITY OF BARS	BAR SIZE	LENGTH	SHAPE
H4	2	4	6-10	
H6	6	6	20- 3	
H7	4	6	4- 8	
H9	4	6	3- 0	
H10	12	4	3- 2	
H21	16	4	1- 8	
H43	24	5	21-10	
H47	32	4	24- 2	
H48	3	4	22- 2	
H49	12	4	2- 4	
N17	48	6	4- 5	
P6	16	7	22- 0	
S	42	4	14- 9	
T13	103	4	7- 8	
U3	8	6	8- 2	
V3	47	4	4- 3	
V6	8	4	4- 0	
V7	48	5	4- 9	
V10	40	4	8- 2	
V12	6	5	2- 7	
V14	6	5	9- 2	
V1[illegible]	25	5	5- 6	
V48	7	4	10- 8	
V59	4	4	5- 5	
V67	40	4	3- 1	
V68	23	5	8- 8	
V69	7	4	7- 6	
V71	6	4	9- 10	
V73	40	4	6- 3	
W19	14	4	22- 0	
W20	14	4	24- 0	

BILL OF MATERIAL

ITEM	UNIT	QUANTITY
CLASS A EXCAVATION FOR STRUCTURES	CU. YD.	70.3
CLASS X CONCRETE	CU. YD.	90.5
REINFORCING BARS	POUND	5340
FURNISHING STEEL PILES 10BP42	LIN. FT.	464
DRIVING STEEL PILES	LIN. FT.	464
FURNISHING CREOSOTED PILES UP TO 20FT	LIN. FT.	260
DRIVING TIMBER PILES	LIN. FT.	260
TEST PILES STEEL 10BP42	EACH	1
TEST PILE TIMBER	EACH	1

ABUTMENT (K)

F.A.I. 74 - SECTION 81-1HB

F.A.I. 74 & RAMPS OVER RELOC. 19TH ST.

ROCK ISLAND COUNTY

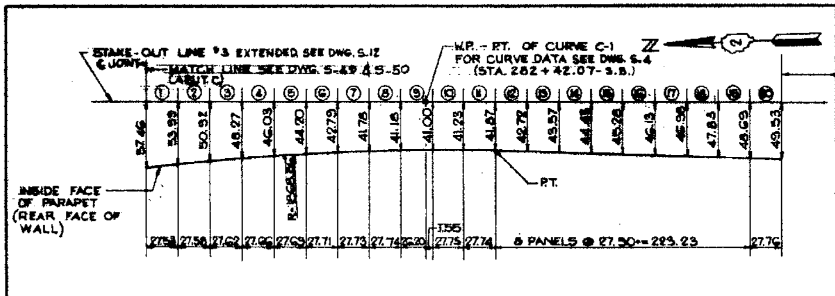
STATION 289 + 23.09

SCALE: AS NOTED DATE:

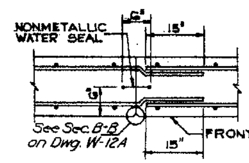
Rev. 5-22-73

DWG. NO. W-11

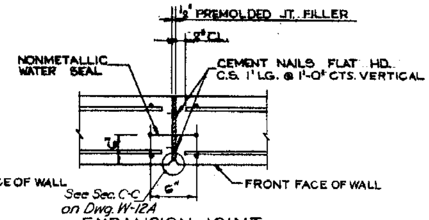
ROUTE NO.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
F.A.I. 74	81-1-2	ROCK ISLAND	389	290
FED. ROAD DIST. NO. 7	ILLINOIS	FED. AID PROJECT I-74		



STAKE - OUT DIAGRAM
SCALE: 1" = 50'-0"

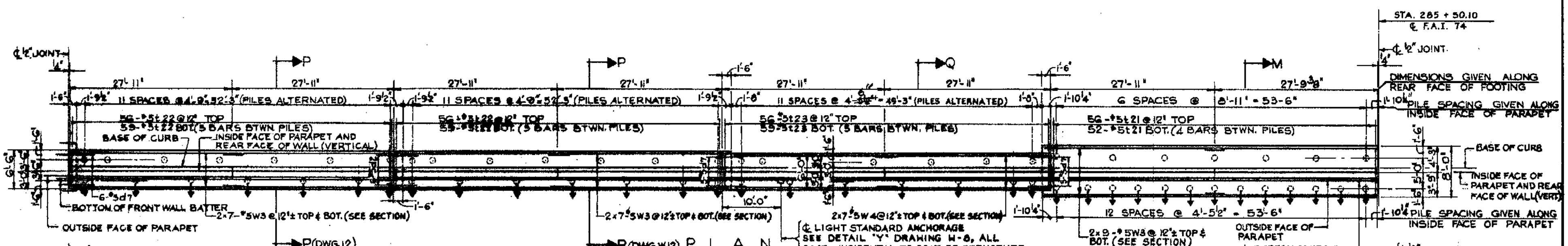


CONTRACTION JOINT
SCALE: 1" = 1'-0"

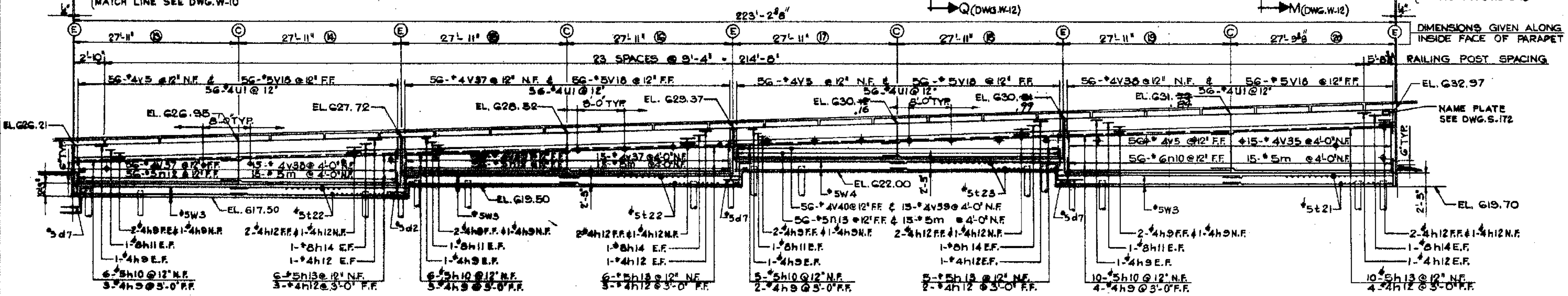


EXPANSION JOINT
SCALE: 1" = 1'-0"

Note: For Rustication Details see Dwg W-12A.



PLAN
SCALE: 1/8" = 1'-0"



ELEVATION (FRONT FACE)
SCALE: 1/8" = 1'-0"

PILE DATA				
PANEL NO.	13 & 14	15 & 16	17 & 18	19 & 20
PILE TYPE	CREOSOTED	CREOSOTED	CREOSOTED	CREOSOTED
CAPACITY	23 TONS	22 TONS	18 TONS	18 TONS
NUMBER REQUIRED	14 *	14	14 *	20
ESTIMATED LENGTH	19 FT.	23 FT.	27 FT.	25 FT.
CUT-OFF ELEVATION	618.50	620.50	623.00	620.70

* INCLUDES 1 TEST PILE

RETAINING WALL 7-S

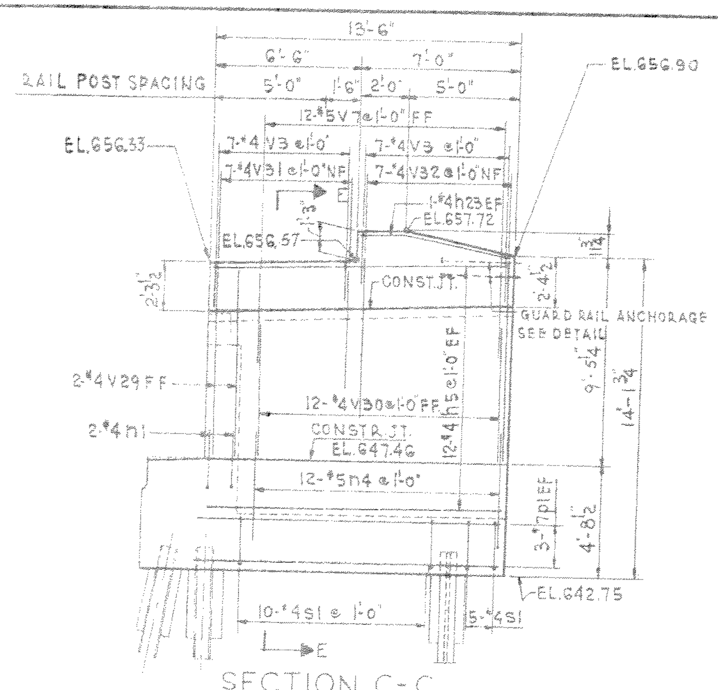
F.A.I. 74 - SECTION 81-1-2

PANELS 13 THRU 20

ROCK ISLAND COUNTY

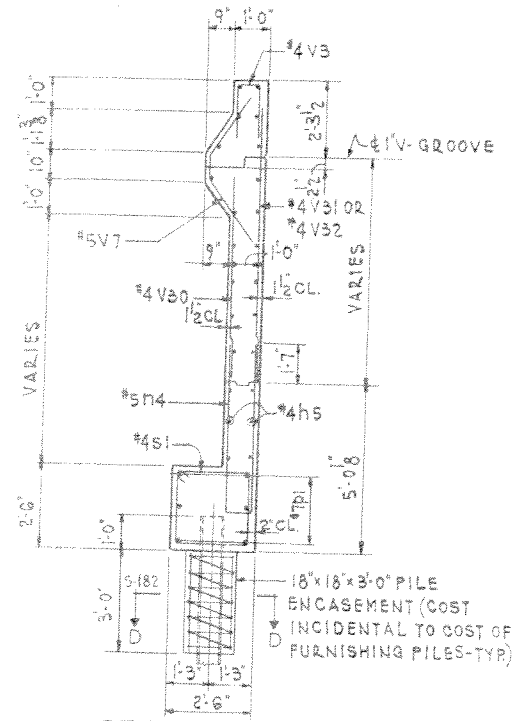
SCALE: AS NOTED DATE:

DE LEUW, CATHER & COMPANY ENGINEERS
DESIGNED BY R. KENNEDY
DRAWN BY AL FOLLIDO
CHECKED
IN CHARGE V. S. MARTINS
APPROVED W.S. HEIN



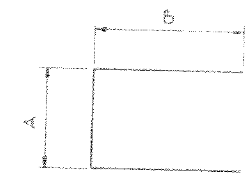
SECTION C-C

SCALE: 1/4"=1'-0"

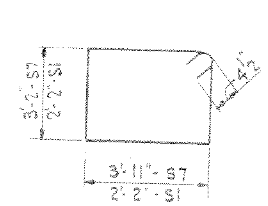


SECTION E-E

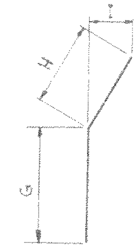
SCALE: 3/8"=1'-0"



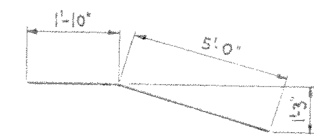
MARK	A	B
h1	1'-6"	2'-6"
h4	9"	5'-3"
U	3'-9"	1'-9"
U1	4'-4"	2'-2"
V22	3'-9"	2'-3"
V23	3'-9"	2'-9"
V24	3'-9"	3'-6"
V3	9"	1'-9"



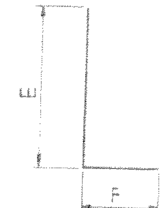
BARS: S7 & S1



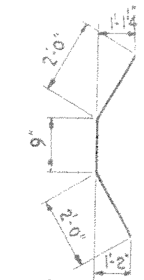
MARK	G	H	I
V25	6'-5"	1'-0"	6 1/2
V29	6'-8"	2'-0"	1'-1 1/4"



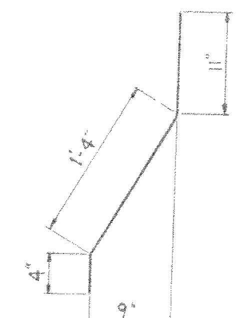
BAR h23



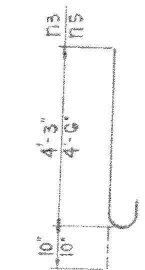
MARK	E	F
h1	3'-3"	1'-6"
h2	1'-6"	1'-6"
h10	1'-8"	1'-6"
h11	1'-8"	10"
V9	6'-4"	1'-3"
V28	4'-6"	1'-3"



BAR V7



BAR V12



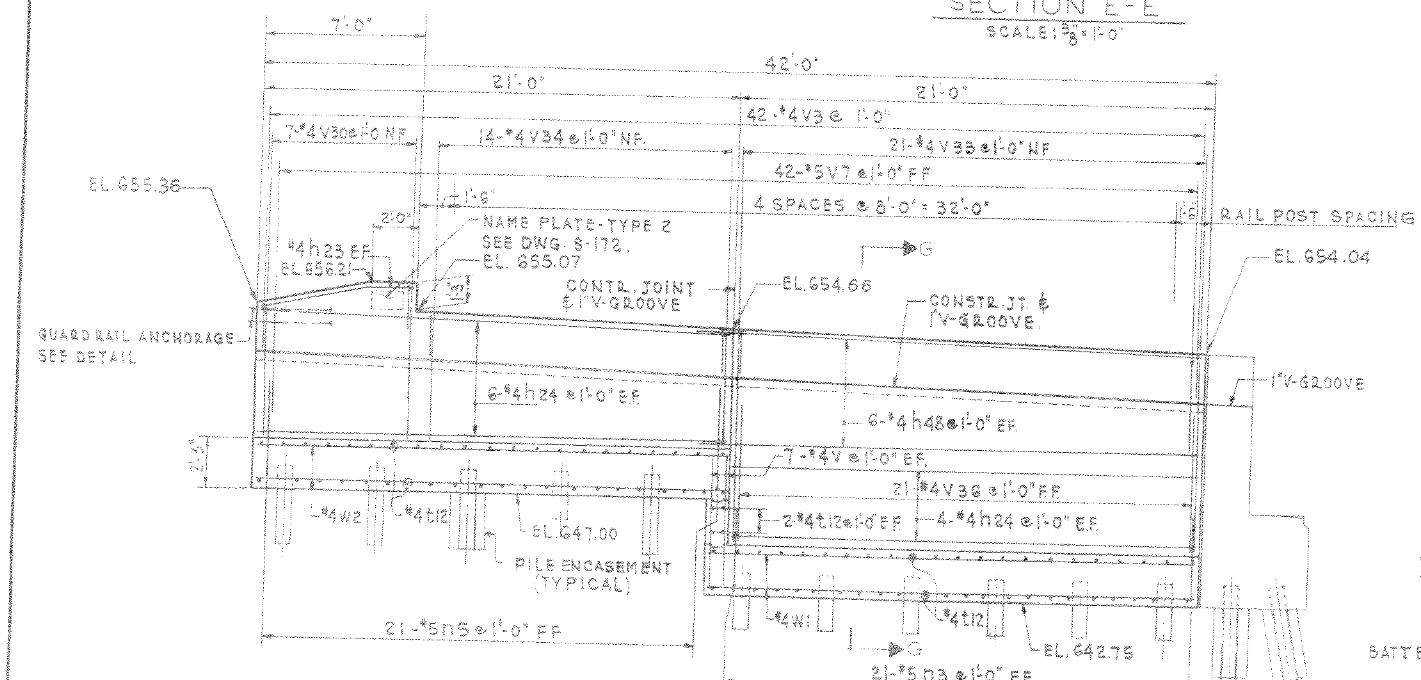
BARS: n3 & n5

DWG. NO. S-183

ROUTE NO.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
F.A.I. 74	81-IHB	ROCK ISLAND	389	223
FED. ROAD DIST. NO. 7	ILLINOIS	FED. AID PROJECT I-74		

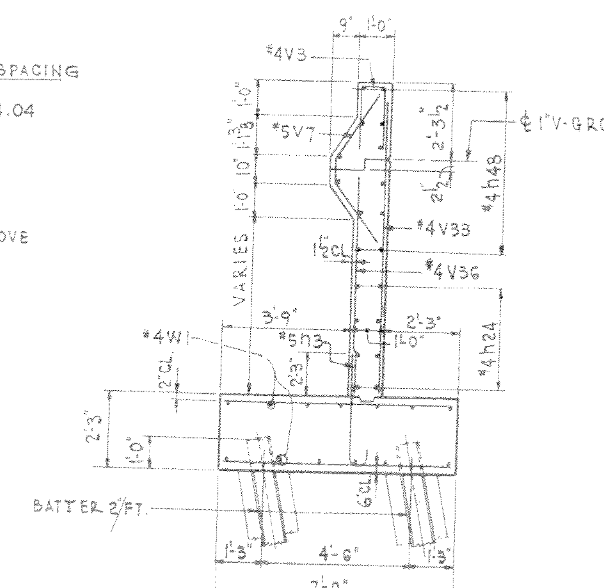
REINFORCING BAR LIST

BAR MARK	QUANTITY OF BARS	BAR SIZE	LENGTH	SHAPE
H1	6	6	4- 9	
H2	10	4	3- 0	
H5	24	4	13- 2	
H10	4	4	3- 2	
H11	6	4	2- 6	
H14	34	4	23- 3	
H15	1	4	14- 0	
H16	15	5	6- 3	
H17	10	5	8- 0	
H18	10	5	8- 6	
H19	5	5	6-10	
H20	5	5	10- 3	
H21	6	4	1- 8	
H22	12	6	22- 6	
H23	4	4	6-10	
H24	20	4	20- 8	
H45	2	6	3- 4	
H48	12	4	22- 2	
N1	2	4	6- 6	
N3	21	5	5- 1	
N4	12	5	11- 3	
N5	21	5	5- 4	
P1	6	7	14- 3	
P3	24	7	24- 0	
S1	15	4	9- 5	
S7	64	4	14-11	
T12	90	4	6- 8	
U	4	6	7- 3	
U1	4	6	8- 8	
V	37	4	6- 2	
V3	56	4	4- 3	
V7	54	5	4- 9	
V9	41	5	7- 7	
V10	41	4	8- 2	
V12	3	5	2- 7	
V22	18	5	8- 3	
V23	22	5	9- 3	
V24	22	5	10- 9	
V25	3	5	7- 5	
V26	3	5	8- 5	
V27	64	4	2- 11	
V28	23	5	5- 9	
V29	2	4	8- 8	
V30	19	4	6- 0	
V31	7	4	8- 5	
V32	7	4	9- 2	
V33	21	4	8-10	
V34	14	4	5- 4	
V36	21	4	7- 2	
W1	13	4	21- 8	
W2	13	4	20- 8	



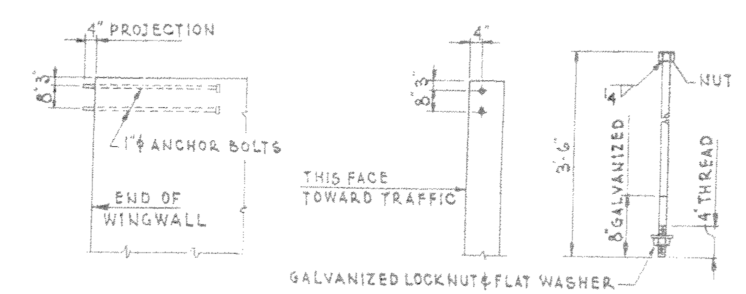
SECTION F-F

SCALE: 1/4"=1'-0"



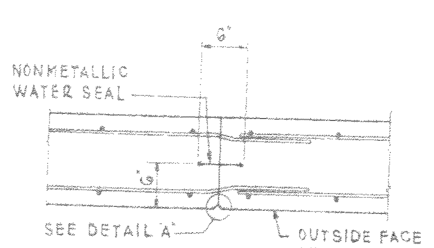
SECTION G-G

SCALE: 3/8"=1'-0"



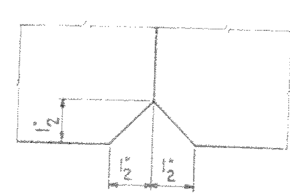
GUARD RAIL ANCHORAGE

NO SCALE



CONTRACTION JOINT

SCALE: 1"=1'-0"



DETAIL "A"

FULL SCALE.

DE LEUW, CATHER & COMPANY ENGINEERS
DESIGNED BY R. J. BLACK
DRAWN BY A. BUROKAS
CHECKED Robert F. Monker
IN CHARGE E. S. MARTINS
APPROVED W. G. HORN

ABUTMENT (M)

WINGWALLS & MISC. DETAILS

F.A.I. 74 - SECTION 81-IHB

F.A.I. 74 & RAMPS OVER RELOC. 19TH ST.

ROCK ISLAND COUNTY

STATION

SCALE: AS NOTED

DATE:

K&E CO. 10-P158

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 tsf $< Q_u <$ 2 tsf

$q_S = 0.47Q_u + 0.555$ for 2 tsf $\leq Q_u <$ 4.5 tsf

$q_S = 2.67$ ksf for 4.5 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.

<u>Maximum Nominal Required Bearing</u> ($R_{N\ MAX}$) 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:

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

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.

<u>Estimated Pile Lengths</u> 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.

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

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

$= \sqrt{1 - \xi^2}$ 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

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

= 1.45

$\lambda_{Q_{DD}}$ = bias for the median 50th 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========== 180 Pier 2 19BR-107

LRFD, ASD, or EXTREME EVENT ================== LRFD
PILE CUTOFF ELEV. ============================ 603.00 FT
GROUND SURFACE ELEV. AGAINST PILE ========== 601.00 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

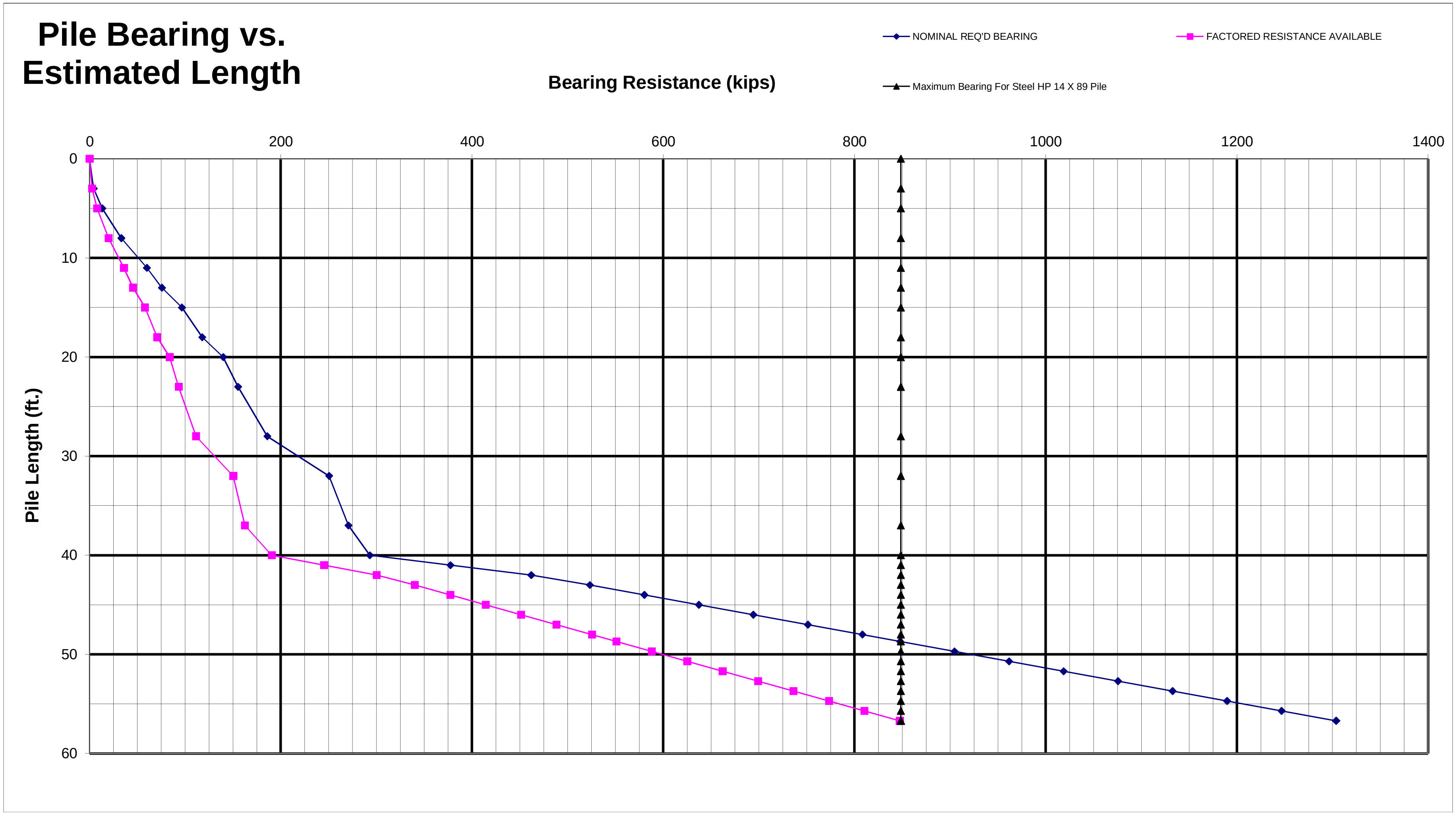
TOTAL FACTORED SUBSTRUCTURE LOAD ======== 4352 KIPS
TOTAL LENGTH OF SUBSTRUCTURE (along skew)==== 15.00 FT
NUMBER OF ROWS OF PILES PER SUBSTRUCTURE = 3
Approx. Factored Loading Applied per pile at 8 ft. Cts ============ 773.65 KIPS
Approx. Factored Loading Applied per pile at 3 ft. Cts ============ 290.12 KIPS

PILE TYPE AND SIZE ========== Steel HP 14 X 89
Plugged Pile Perimeter===================== 4.750 FT — Unplugged Pile Perimeter========= 7.033 FT
Plugged Pile End Bearing Area============== 1.409 SQFT — Unplugged Pile End Bearing Area==== 0.181 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
848 KIPS	**848** KIPS	**552** KIPS	**50** FT
	Avg. Est.'d Driving Stress		Soil Setup Pile Length
	36.6 KSI		**N/A - Rock** FT

BOT. OF LAYER ELEV. (FT)	LAYER THICK. (FT)	UNCONF. COMPR. STRENGTH (TSF)	S.P.T. N VALUE (BLOWS)	GRANULAR OR ROCK LAYER DESCRIPTION	NOMINAL PLUGGED SIDE RESIST. (KIPS)	NOMINAL PLUGGED END BRG. RESIST. (KIPS)	NOMINAL PLUGGED TOTAL RESIST. (KIPS)	NOMINAL UNPLUG'D SIDE RESIST. (KIPS)	NOMINAL UNPLUG'D END BRG. RESIST. (KIPS)	NOMINAL UNPLUG'D TOTAL RESIST. (KIPS)	NOMINAL REQ'D BEARING (KIPS)	FACTORED GEOTECH. LOSS FROM SCOUR or DD (KIPS)	FACTORED GEOTECH. LOSS LOAD FROM DD (KIPS)	FACTORED RESISTANCE AVAILABLE (KIPS)	ESTIMATED PILE LENGTH (FT)	SOIL SETUP PILE LENGTH (FT)	AVERAGE ESTIMATED DRIVING STRESS (KSI)
600.50	0.50	1.80	5		2.2		10.4	3.2		***4.3***	4	0	0	3	3	5	-
598.00	2.50	0.50	5		3.9	8.3	39.1	5.7	1.1	***13.2***	13	0	0	8	5	8	-
595.50	2.50	2.00	9		11.6	33.1	72.2	17.2	4.3	***33.1***	33	0	0	20	8	11	-
592.50	3.00	3.30	9		19.6	54.6	75.3	29.0	7.0	***60.0***	60	0	0	36	11	18	-
590.50	2.00	2.30	14		10.1	38.1	90.4	15.0	4.9	***75.6***	76	0	0	45	13	20	-
588.00	2.50	2.60	20		13.8	43.0	107.4	20.4	5.5	***96.5***	96	0	0	58	15	28	16.2
585.50	2.50	2.80	18		14.5	46.3	120.3	21.5	6.0	***117.7***	118	0	0	71	18	32	17.0
583.00	2.50	2.70	16		14.1	44.7	142.7	20.9	5.7	***139.7***	140	0	0	84	20	32	17.0
580.50	2.50	3.20	14		16.0	52.9	***155.3***	23.6	6.8	162.9	155	0	0	93	23	37	16.1
575.50	5.00	3.00	14		30.5	49.6	***185.8***	45.1	6.4	208.0	186	0	0	111	28	N/A - Rock	17.0
571.50	4.00	3.00	14		24.4	49.6	259.8	36.1	6.4	***250.5***	250	0	0	150	32	N/A - Rock	20.1
566.50	5.00		45	Hard Till	10.8	99.3	303.7	16.0	12.8	***270.7***	271	0	0	162	37	N/A - Rock	20.6
563.00	3.50		60	Hard Till	11.9	132.4	352.3	17.6	17.0	***293.1***	293	0	0	190	40	N/A - Rock	22.6
562.00	1.00			Shale	57.0	169.1	409.3	84.4	21.8	***377.5***	377	0	0	245	41	N/A - Rock	23.0
561.00	1.00			Shale	57.0	169.1	466.3	84.4	21.8	***461.9***	462	0	0	300	42	N/A - Rock	24.3
560.00	1.00			Shale	57.0	169.1	***523.3***	84.4	21.8	546.3	523	0	0	340	43	N/A - Rock	27.3
559.00	1.00			Shale	57.0	169.1	***580.3***	84.4	21.8	630.7	580	0	0	377	44	N/A - Rock	29.0
558.00	1.00			Shale	57.0	169.1	***637.3***	84.4	21.8	715.1	637	0	0	414	45	N/A - Rock	30.7
557.00	1.00			Shale	57.0	169.1	***694.3***	84.4	21.8	799.5	694	0	0	451	46	N/A - Rock	32.2
556.00	1.00			Shale	57.0	169.1	***751.3***	84.4	21.8	883.9	751	0	0	488	47	N/A - Rock	33.7
555.00	1.00			Shale	57.0	169.1	***808.3***	84.4	21.8	968.3	808	0	0	525	48	N/A - Rock	35.5
554.31	0.69			Shale	39.3	169.1	***847.6***	58.2	21.8	1026.5	848	0	0	551	48.7	N/A - Rock	36.6
553.31	1.00			Shale	57.0	169.1	***904.6***	84.4	21.8	1110.9	~~905~~	~~0~~	~~0~~	~~588~~	~~49.7~~	~~N/A - Rock~~	~~38.1~~
552.31	1.00			Shale	57.0	169.1	***961.6***	84.4	21.8	1195.3	~~962~~	~~0~~	~~0~~	~~625~~	~~50.7~~	~~N/A - Rock~~	~~39.8~~
551.31	1.00			Shale	57.0	169.1	***1018.6***	84.4	21.8	1279.7	~~1019~~	~~0~~	~~0~~	~~662~~	~~51.7~~	~~N/A - Rock~~	~~41.5~~
550.31	1.00			Shale	57.0	169.1	***1075.6***	84.4	21.8	1364.1	~~1076~~	~~0~~	~~0~~	~~699~~	~~52.7~~	~~N/A - Rock~~	~~42.9~~
549.31	1.00			Shale	57.0	169.1	***1132.6***	84.4	21.8	1448.5	~~1133~~	~~0~~	~~0~~	~~736~~	~~53.7~~	~~N/A - Rock~~	~~44.9~~
548.31	1.00			Shale	57.0	169.1	***1189.6***	84.4	21.8	1532.9	~~1190~~	~~0~~	~~0~~	~~773~~	~~54.7~~	~~N/A - Rock~~	~~46.7~~
547.31	1.00			Shale	57.0	169.1	***1246.6***	84.4	21.8	1617.3	~~1247~~	~~0~~	~~0~~	~~810~~	~~55.7~~	~~N/A - Rock~~	~~47.9~~
546.31	1.00			Shale	57.0	169.1	***1303.6***	84.4	21.8	1701.7	~~1304~~	~~0~~	~~0~~	~~847~~	~~56.7~~	~~N/A - Rock~~	~~49.8~~
545.31	1.00			Shale		169.1			21.8								



Pile Bearing vs.
Estimated Length
Bearing Resistance (kips)
NOMINAL REQ'D BEARING
FACTORED RESISTANCE AVAILABLE
Maximum Bearing For Steel HP 14 X 89 Pile
0
200
400
600
800
1000
1200
1400
0
10
20
30
40
50
60
Pile Length (ft.)

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).