STRUCTURE GEOTECHNICAL REPORT

PHASE 1 B GEOTECHNICAL REPORT
NEW I-74 BRIDGE OVER MISSISSIPPI RIVER
MOLINE, ILLINOIS TO BETTENDORF, IOWA
RAMP 6TH-C STRUCTURE
SECTION 81 – 1HVB
ROCK ISLAND COUNTY, ILLINOIS

PROPOSED STRUCTURE NO. 081-0186

PREPARED FOR

IOWA DEPARTMENT OF TRANSPORTATION
AND
ILLINOIS DEPARTMENT OF TRANSPORTATION

PREPARED BY

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JUNE 2008

Attachments: Figures 1 – 4
Boring Logs
Laboratory Test Results
Rock Core Photographs
Elastic Moduli and RMR Table
SGR Responsibility Checklist

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062-043400

1.0 PROJECT INFORMATION

Introduction

A study for a new Moline Viaduct, a section of the proposed I-74 crossing of the Mississippi River at the Quad Cities, was conducted by CH2M-HILL/JACOBS. The study results are presented in a Technical Memorandum titled "I-74 Iowa-Illinois Corridor Study-Moline Viaduct & Ramps, Proposed Span Arrangement, dated June 21, 2007. Figure 1 shows the structure location. Figure 2, Location Map, shows the overall Quad Cities area and Figure 3, Boring Location Plan, shows the alignment of both the existing and proposed I-74 Illinois Viaduct and Ramps. The ramp structure is located in Sections 32 and 33, Township 18N, Range 1 West.

Purpose

This Structural Geotechnical Report (SGR) presents the results of the Phase 1B geotechnical investigation performed for the proposed Ramp 6th-C structure (Structure No. 081-0186) in Moline, Illinois. This report deals only with the Ramp 6th-C substructure units that will be designed and constructed in Moline, Illinois. Five other reports will deal with the recommendations for the piers in the Mississippi River, the land based piers on the Bettendorf, Iowa side of the river, the Moline Main Line Viaduct, the 19th Street Bridge and Ramp 6th-D in Moline, Illinois. The purpose of this investigation was to determine the nature and condition of the subsurface materials, to describe the general site characteristics, and to formulate conclusions and recommendations for the preliminary design and construction of the Ramp 6th-C foundations and other subsurface related components of the proposed bridge structures.

Scope

The scope of this investigation includes reviewing available subsurface information for the project area, obtaining the required field and laboratory test data, performing the necessary engineering analyses, and formulating the conclusions and recommendations presented in this report. These conclusions and recommendations have been prepared considering the nature of the proposed project as presently planned and described in this report.

2.0 PROJECT DESCRIPTION

Site Description

The new Moline Viaduct and the associated Ramp 6th-C are located in Moline, Illinois, extending from River Drive (Third Avenue) southward to a proposed abutment location just south of 7th Avenue. The alignment continues southward and will encompass a new I-74 overpass of 19th Street. The proposed alignment is located just east (upstream) of the existing I-74 alignment through downtown Moline.

Proposed Ramp 6th-C

The proposed Ramp 6th-C will be an off-ramp from EB I-74 to 6th Avenue. Ramp 6th-C will consist of a 2-span structure extending from the north side of River Drive to an abutment in the Deere Co. parking lot between 4th and 5th Avenue. The structure will cross over several existing infrastructure features including 4th Avenue, an existing BNSF railroad track, and existing Ramps 3-N and N-3. Existing Ramps 3-N and N-3 will be removed after construction of the new I-74 Moline Viaduct.

The Ramp 6th-C structure has a total length of approximately 394.5 feet and has span lengths of 169 feet and 225.5 feet, respectively. Figure 3 shows a general plan view of the proposed ramp.

The abutment fill height at the Bridge Abutment is approximately 25 feet. The abutment with bridge seats will be set behind a typical IDOT MSE wrap-around wall sections. The MSE wall sections are addressed in another SGR (see Reference 14).

Preliminary AASHTO Group foundation loadings were not available for this ramp.

Potentially Contaminated Site

A Preliminary Environmental Site Assessment (PESA) was completed on the Illinois side of the new I-74 project corridor in August, 2002 by the Illinois State Geological Survey (ISGS). The Ramp 6th-C footprint will cross over a section of the Deere & Co. parking lot located between 4th and 5th Avenues and 21st Street to the existing I-74 viaduct. In the final Environmental Impact Statement (FEIS), the Deere & Co. parking lot property was identified as contaminated by VOC's and metals from machine shops and metals from the blacksmith and grinding facilities of a former industrial site and that any excavation or grading will require the management of special waste.

3.0 SUBSURFACE INVESTIGATION

Phase 1A

A subsurface investigation was conducted during Phase 1A of this project from October 2005 through December 2005 to assist in the conceptual study/selection of feasible foundation types. Two borings (PRMPC-01 and -02) were drilled near the proposed footprint for Ramp 6th-C. In addition, two borings (SB1030 and RB1031) were drilled for various retaining walls along the south half of the ramp which will include an embankment. PRMPC-01 and PRMPC-02 boring logs are attached as a part of this report.

Phase 1B

Two borings were drilled during the Phase 1B Geotechnical Investigation to determine the nature and condition of the subsurface materials along the proposed

Ramp 6th-C alignment. Boring VIAIL-108 was drilled near the location of the proposed gore with EB I-74. Boring PRMPC-03 was drilled in the Deere Co. parking lot near the proposed abutment location. The number of borings selected for this preliminary phase was based upon input and approvals from Iowa DOT and CH2M Hill. The locations of the borings are shown on the Boring Location Plan, Figure 3. The borings were located in the field by using a hand held GPS unit. Elevations were interpolated from project .tin files. Datum for the boring locations was the Iowa South State Plane Coordinate System 1402 and NAVD 88.

The borings were drilled between August 30, 2007 and September 4, 2007 by Terracon Consultants Inc. of Naperville, Illinois as part of the Phase 1B Geotechnical Investigation for the new I-74 Illinois Approach. The borings were drilled using a CME 55 truck rig and a CME 550 ATV rig owned and operated by Terracon. A Jacobs engineer provided on-site supervision throughout the boring operations, and prepared the boring logs found in the Appendix to this report.

The borings were typically advanced to a depth of 25 feet into bedrock. The total depth of the two borings ranged from 39 to 46 feet below ground surface in Borings VIAIL-108 and PRMPC-03, respectively. In both borings, the drilling method included advancing the borehole through the overburden soils to top of bedrock using 3-3/4 inch inner diameter hollow stem augers and then advancing the hole to the desired depth into bedrock using NQ-wireline rock coring methods. A table summarizing the Phase 1A and 1B boring programs is presented as Table 1

Standard Penetration Resistance Tests (ASTM D1586) were conducted in the overburden materials of each boring using standard split-spoon samplers and a CME automatic drive hammer. In general, SPT's were conducted at 2.5-foot intervals in the upper 30 feet of boring (or to refusal, whichever occurred first) and at 5-foot intervals thereafter to bedrock or bottom of boring. The samples obtained were placed in plastic bags and delivered to Terracon's laboratory. Core samples (NQ size) of the underlying bedrock were obtained and placed in wooden boxes for later laboratory testing. The core boxes were removed each day from the site and delivered to Terracon's office in Bettendorf, IA. All recovered rock core samples were photographed each day in order to provide a permanent record. Photographs of the rock cores collected are found in the Appendix.

Samples of cohesive soils encountered in the borings were typically tested for strength using both a pocket penetrometer and a Rimac Spring Tester. Test results are included in the boring logs. The boring logs are attached to this report. We have also included the log for boring VIAIL-107 which was drilled at Pier 1 of the main viaduct.

As part of the test drilling program, Jacobs provided field personnel to operate a photoionization detector (PID) to detect the presence of any volatile organic compounds (VOC's) in soil obtained from the geotechnical borings at levels requiring segregation and drummed storage of auger cuttings pending sampling and analysis or other method to determine appropriate disposition. To that end, a PID was used for headspace analysis of soil during drilling operations; scanning split spoon samples to identify any anomalous zones; sampling the borehole opening between split spoon sampling and coring runs as a general indication of the presence of VOC's; and measuring of VOC concentrations in the breathing zone during drilling/coring operations. In addition, a triple gas meter was used to scan for combustible gases at the top of the auger space during drilling operations.

Table 1 - SUMMARY OF RAMP 6TH-C PHASE 1A and 1B BORING PROGRAM

Boring No.	Date Drilled	Ground Elev.	Soil Thickness (ft)	Weathered/Soft Rock Thickness (ft)	Top of Rock Core Depth (ft)	Top of Rock Core Elev (ft)	Bottom of Hole Depth (ft)	Bottom of Hole Elev (ft)
Phase 1A								
PRMPC-01	10/31/2005	573.3	19.1	-	19.1	554.2	19.9	534.3
PRMPC-02	12/15/2005	576.0	19.5	0.5	20.0	556.0	20.0	536.0
Phase 1B								
VIAIL-108	8/30/2007	570.7	12.0	2.1	14.1	556.6	39.1	531.6
PRMPC-03	9/4/2007	575.8	16.0	2.4	18.4	557.4	46.0	529.8

Laboratory Testing

The laboratory testing program was directed toward establishing the classification and evaluating the general engineering properties of the subsurface materials. The testing was conducted by Terracon Consultants of Bettendorf, IA, and their subsidiary H.C. Nutting Company of Cincinnati, Ohio, in accordance with ASTM specifications. Laboratory tests were performed to determine the physical and engineering characteristics of selected split-spoon and NQ size rock core samples obtained during the subsurface investigation program. The testing program included moisture content determinations on soil samples, and uniaxial compression tests, dry density determinations, Moh's Hardness, and moisture content determinations on selected rock core samples.

The results of all laboratory tests have been summarized and are included in the Appendix to this report.

4.0 SUBSURFACE CONDITIONS

Subsurface Materials

A subsurface profile along the proposed Ramp 6th-C structure alignment is presented in Figure 4. In general, all three borings (VIAIL-108, PRMPC-01 and PRMPC-03) encountered Pennsylvanian-aged sandstone beneath varying types and thicknesses of soil overburden. The sandstone unit consisted of gray to brownish gray fine-grained sandstone which was typically of uniform size (well sorted), soft to very soft, and moderately to well cemented. Bedding spacing was generally non-distinct unless along occasional black bands, with horizontal sandy-rough fractures occurring at thin to thick bedded spacing. Rock quality designations (RQD's) of the sandstone ranged from 0 to 98 percent and averaged about 45 percent for the three borings. The uniaxial compressive strength of the lone sandstone sample tested from the applicable Phase 1B borings was 1,940 psi, although similar tests performed on sandstone core samples from other Phase 1B borings for the Moline Viaduct showed a ranged of 1,500 to 4,255 psi and averaged about 3,090 psi.

It should be noted that all three of the borings were drilled in sandstone and that the RQD values in Borings VIAIL-108 (typically 0 to 40 percent) are consistently lower than those from the other two borings, PRMPC-01 and PRMPC-03 (typically 60 to 100 percent). A note on the field boring log indicated that the relatively poor quality RQD designation of the sandstone in VIAIL-108 was due to very thin to thin spaced horizontal fractures and not to highly fractured rock. The horizontal fractures may have been mechanically induced.

The thickness and type of overburden materials varied slightly with location. At Boring VIAIL-108 (near Pier 2), the overburden consisted of an upper 1-foot thick layer of silt underlain by 6 feet of silty clay and 5 feet of sandy clay to clayey sand and gravel. Weathered sandstone was encountered between 12 and 14-foot depths prior to switching to rock coring in gray sandstone. At Borings PRMPC-01 (near Pier 3C), the overburden was approximately 19 feet thick and consisted of layers of silt, clayey silt to gravel, poorly graded gravel to clay, clay, and sandy clay. In Boring PRMPC-03 (Abutment), the overburden was about 16 feet thick and included an upper 8.5 feet of yellowish brown clayey silt, an intermediate 5-foot thick layer of soft sandy clay which was very moist to wet, and a lower 2.5-foot thick layer of saturated black fine to coarse-grained sand. A light gray sandy shale layer was encountered in PRMPC-03 at 16 to 18.4 feet depth prior to encountering the underlying sandstone.

It should be noted that there was a strong petroleum odor and free product in the soil sample collected from the saturated zone in Boring PRMPC-03 (located in the John Deere parking lot) at a depth of 13.5 to 15 feet below ground surface. Field PID readings of the soil sample were measured at 420 ppm. In addition, it was reported that Boring PRMPC-02, drilled just south of the abutment location during the Phase 1A geotechnical investigation, encountered asphalt concrete with petroleum odor at approximate 13 to 15 feet below ground surface. These findings correlate with notes in the FEIS identifying sections of the existing Deere Co. parking lot as a potentially contaminated site.

<u>Areas Requiring Additional Investigation</u>

For final design, it is recommended that a boring be drilled at Pier 3-C. A ramp closure and utility clearance will be required to access the boring location.

In addition, an Environmental Investigation needs to be performed to determine the extent of contamination at the Deere & Co. parking lot at 2000 4th Avenue. This investigation should address the quantity of contaminated material to be excavated; disposal methods and available landfills; special handling requirements, certifications and permits; water treatment method from water collected from excavations; site monitoring requirements during construction; and requirements for personnel protection and monitoring.

Groundwater Levels

Groundwater levels were noted from water on drill rods during the course of the Phase 1B drilling operations. In general, water levels noted during drilling in the borings along the proposed ramp alignment ranged from approximate El. 562 to El. 564 ft.

During the time of drilling, the Mississippi River level was at approximate El. 561.0 ft. The river levels are controlled by the downstream Mississippi River Lock and Dam No.15 at Rock Island, Illinois. The important water elevations for this project are presented in Table 2 below:

Table 2 - Important Mississippi River Water Elevations

Case	Elevation (NGVD 1912), ft.
Normal Pool	561.0
Cessation of Navigation	562.5
2% Flowline	563.5
100-Year Flood	569.6
500-Year Flood	572.2
High Water of Record	569.7

Note: The following conversions apply to the project location:
NGVD 1929 = NGVD 1912 - 0.510 ft
NAVD 88 = NGVD 1912 - 0.727 ft

Groundwater rises when the adjacent Mississippi River rises. Construction of Pier 2 can be influenced by river levels if spread footings are used to support this bent.

Seismicity

Seismic loads will not be considered in preliminary design due to the low seismicity of the project area. For final design, seismic forces will be computed and applied in accordance with AASHTO LRFD for Seismic Performance Zone 1 (per IDOT Seismic Design Guide p. 3.15-82).

The Ramp 6th-C profile is considered Site Class C per AASHTO (2008 Interim Revisions), Section 3.10.3.1, because of the shallow depth to bedrock. At Pier 2, Site Class B could be considered since the pier is founded directly on bedrock. The acceleration coefficient, A, to be used in the application of AASHTO LRFD criteria is 3.5 percent for a 1,000 year return period according to Figure 3.10.2.1-3 in the AASHTO LRFD (2008 Interim Revisions).

Scour

Scour is not applicable at these structures.

Mining Activity

A review of the Illinois State Geologic Survey (ISGS) maps indicates no past mining activities in the area of the proposed Ramp 6th-C footprint.

5.0 BRIDGE FOUNDATIONS

Limitations

These recommendations have been developed to aid in the preliminary design and construction of the bridge crossing foundations affected by the subsurface materials. These recommendations are limited to the scope of work and understanding of the proposed structures as detailed in this report. Significant changes in the anticipated project scope may invalidate these conclusions and recommendations. If, during construction, subsurface conditions different from those encountered in the borings are observed, or appear to be present beneath excavations, Jacobs should be advised at once so that Jacobs can review these conditions and reconsider these recommendations, when necessary.

Rock Mass Strength

The rock cores obtained from the exploration program were classified using the rock mass rating system (RMR). The RMR classification system is a widely used procedure for determining rock mass quality. This system considers the properties and conditions of the rock/rock mass. The RMR is calculated as the sum of the individual ratings for each of the five parameters minus an adjustment made for joint orientation. In general, the rock classified as Class III, Fair Rock to Class II, Good Rock per Table 10.4.6.4-3 of 2006 AASHTO LRFD.

The shear strength of the fractured rock masses was evaluated using the Hoek and Brown criteria as suggested by 2006 AASHTO LRFD. The estimated range of shear strength parameters for Piers 2 and 3C using borings VIAIL 108 and 110 are presented in Table 3.

Table 3 - Shear Strength Parameters

Material	Friction Angle (degs)	Cohesion (ksf)
Sandstone	40.8 - 43	1.3 – 1.9

Rock Mass Deformation

Elastic moduli were determined or estimated from intact modulus of rock core samples, and from the RMR rating per 2006 AASHTO LRFD. Engineering judgment was used to determine which moduli to use in settlement computations. In addition, elastic moduli estimated from the RMR system and unconfined compression tests for all test borings are included in the Appendix.

Abutment 6th-C

In CH2M-Hill's report titled "Structure Geotechnical Report Ramp 6th-C Retaining Wall, Structure No. 081-6019" dated January, 2008 (Reference 14), the results of global stability and settlement analyses are discussed for the 081-6019 wall alignment, which will be a wrap-around wall which will encompass the three sides of the Ramp 6th-C bridge abutment. The results of the analyses are presented below in the sections "Global and External Stability of MSE Wall" and "Settlement".

Global and External Stability of MSE Wall

Stability analyses were performed on models developed using available subsurface data and geometry from proposed cross sections. The analyses involved evaluation of the wall resistance against sliding (safety factor of 1.25), overturning (safety factor of 2.0), global failure (safety factor of 1.3 (1.5 for retaining walls beneath abutments)) and bearing failure (safety factor of 2.5) and were performed in accordance with the FHWA manual on MSE walls (Reference 15). Results of global stability analyses are presented in Table 4; the results of external stability analyses (sliding, overturning, bearing) are contained in Table 5.

According to FHWA guidelines the width of the reinforced zone for a MSE wall should be a minimum of 70% of the MSE height, or a length sufficient to satisfy external and global issues. At the "minimum 70%" width, the analyses indicate that the wall will have adequate mass to resist both sliding and overturning. However, global stability and/or bearing capacity issues still remained on one of the models analyzed. Subsequent analyses indicate that reinforced zones on the order of 1.0 to 1.14 times the retained height (lengths as great as 32 feet) for the side walls and 1.6 times the retained height for the abutment wall (length of 36 feet), as shown in Table 5 are necessary, with the required length varying along the alignment, dependant on subsurface conditions and retained height. Consequently, any reduction in reinforcement length will require soil strength improvement (staged construction, ground improvement, etc.) and/or a reduction in fill loading (lightweight fill, wall height reduction).

TABLE 4 - GLOBAL STABILITY ANALYSES RESULTS FOR MSE WALL SECTIONS

Location of Slope Analyzed	Loading Case	Failure Mode	FS with Recommended Shear Strength & Full MSE Section	B_{MSE} [C] (ft)	B_{MSE}/H_{MSE} [D] (%)
Station 331+50	Undrained	Circular	1.38	22[B]	73
		Block	1.34	22[B]	73
	Drained	Circular	1.38	22[B]	73
		Block	1.34	22[B]	73
Station 331+00	Undrained	Circular	[A]	32	114
		Block	[A]	32	114
	Drained	Circular	[A]	32	114
		Block	[A]	32	114
Abutment	Undrained	Circular	1.59	36[B]	157
		Block	1.46	36[B]	157
	Drained	Circular	1.67	28[B]	122
		Block	1.35	28[B]	122

[A] Minimum reinforced zone for MSE wall (70%) resulted in overlap of approximately 0.3 x H_{MSE} between the walls on either side of the ramp embankment. Given FHWA guidelines if overlap is greater than 0.3 x H_{MSE}, walls are not subject to lateral pressure from the other. Consequently, global stability analyses were not applicable.

[B] Length controlled by external bearing capacity analysis.

[C] B_{MSE} = Width of Reinforced Zone.

[D] H_{MSE} = Height of MSE Wall Section (including Embedment).

TABLE 5 - EXTERNAL STABILITY ANALYSES RESULTS FOR MSE WALL SECTIONS

Wall Station Analyzed	Height (ft)	Embed-ment (ft)	H_{MSE} (ft)	B_{MSE} (ft)	B_{MSE}/H_{MSE} (%)	Bearing F.S.	Sliding F.S.	Overturning F.S.
331+50	26	4	30	22	73	2.9	2.1	4.2
331+00	24	2	28	31	110	10.7	[C]	[C]
Abutment	19	4	23	36	157	2.5	2.6	9.6

[C] Minimum reinforced zone for MSE wall (70%) resulted in overlap of approximately 0.3 x H_{MSE} between the walls on either side of the ramp embankment. Given FHWA guidelines if overlap is greater than 0.3 x H_{MSE}, walls are not subject to lateral pressure from the other. Consequently, sliding and overturning analyses were not applicable.
[D] Length controlled by global stability analyses.

In addition to the above-described calculations, walls bearing on cohesive soils were also examined for local shear (lateral squeeze) failure. Cohesive soils encountered in the boring drilled on the northern portion of the wall alignment (PRMPC-03) were commonly weak, and often WOH. Given the amount of fill to be retained, these soils, in their current state, have inadequate resistance against local shear. Consequently, the soils in these areas will need to be improved by one of the construction alternatives presented in Section 5.1 of Reference 14. Conversely, weak cohesive soils were not encountered at the southern portion of the alignment, therefore similar local shear issues did not exist.

If staged construction, ground improvement, and/or lightweight fill are not suitable, and the wall height cannot be reduced, the MSE wall selection should be re-evaluated and compared with a CIP wall supported on a deep-foundation system. A deep-foundation-supported CIP wall may be a more suitable system at this location. However, settlement of the considerable fill behind the CIP wall footing/heel and that of the approach embankment will not be supported by the CIP deep foundations and, hence, staged construction, ground improvement, and/or lightweight fill of the embankment will still be required."

Settlement

According to Reference 14, "the most compressible soils appear to exist at the north end of the alignment, where coincidentally the highest proposed embankment/walls will be placed. Therefore, the greatest settlement is expected at the abutment. The analyses estimate settlements on the order of 3 inches at the face of the walls (east, west, abutment) at the northern end of the alignment and with settlements on the order of 5 inches occurring within 15 feet (behind the wall face). Settlement magnitudes are anticipated to decrease to the south, given the presence of less-compressible soils and lesser fill heights. Differential settlements (for both north and south) may approach total settlements.

> If these settlements are not acceptable, it is recommended that a multi-stage construction program be pursued, as discussed in Section 5.1 of Reference 14. Staged construction will result in considerably lower settlement magnitudes. The construction involves fill placement in several lifts. Extensive monitoring will be required during and after placement of each fill lift to ensure that the underlying soils do not become unstable and that settlement has been completed prior to placement of the next lift.
>
> While a majority of settlement will likely occur during construction, settlement may continue after fill placement, with almost all settlement occurring within 4 months construction. The magnitude and rate of settlement is a major factor in the selection, design, and construction of the retaining wall. Although the subsoils can be improved by a variety of methods, it is recommended that the selection of a MSE wall, accompanied by appropriate construction sequencing and methods, may provide adequate performance with a reasonable risk to the owner."

When settlement is greater than 0.4 inches, it must be accounted for as downdrag or negative skin friction for pile foundations. The downdrag geotechnical loss will account for the loss of maximum factored resistance available as well as the additional soil load.

Spread Footings

After a review of the boring logs, a target footing elevation of 556.0 was selected for Pier 2. The footing elevation is approximately 15 feet below grade.

The competency of the rock mass below Pier 2 was investigated during preliminary design and was based upon the procedures using the RMR rating system and applying the estimated shear strength parameters to the general bearing capacity formula. The nominal bearing resistance or ultimate bearing capacity for various footing widths was calculated by the methodology presented in the 2006 AASHTO LRFD (10.6.3.1.2a-1 to 10.6.3.1.2a-9).

The nominal bearing resistance of rock foundations are extremely high as would be expected for footings founded on bedrock. Depending on footing widths, the calculated bearing resistance ranged from 445 to 1,600 ksf. It should be noted that the effect of eccentricity was taken into account by using a reduced effective footing area. AASHTO requires that when factored loads are used that the eccentricity be less than 3/8 of the footing dimension in any direction for footings founded on cohesionless materials or rock.

The elastic settlement of spread footings founded on the underlying jointed/fractured bedrock formations was estimated with 2006 AASHTO LRFD Equation 10.6.2.4.4-1 using appropriate values of rock mass modulus, E_m. The elastic settlements are minimal and are in the range of 0.01 to 0.03 inches. It is estimated the elastic settlement of the rock mass beneath Pier 2 will be less than 0.25 inches for the range of bearing pressures that will be applied to the underlying rock mass.

To evaluate the ultimate sliding resistance of the footings cast on the underlying limestone and sandstone bedrock, a friction factor, tan δ, of 0.70 should be used because limestone typically breaks along bedding planes when excavated and can be quite smooth. Unless the footing is cast neat against the rock excavation sidewalls, it is recommended that passive resistance not be considered.

For preliminary design, it is recommended that an allowable net bearing pressure of 25 ksf be used to size the foundations. However, the structural designers indicate bearing pressures may not exceed 10 ksf due to a stability standpoint (stay within Kern area) according to their preliminary analysis.

Driven Piles

Pier 3-C and the abutment are recommended to be founded on driven H-piles bearing on the underlying bedrock. Driven piling (8BP36, 10BP42 and 10BP57) was used on several bents of the existing viaduct where the depth to bedrock was greater than 15 feet.

For preliminary design, the initial pile layout should be based upon using the IDOT Pile Data Guidelines for 2007 Standard Specifications dated November 17, 2006. Steel HP piles (AASHTO M270 Grade 50) driven to refusal should be used. Metal Shell Piles, Precast Concrete Piles and Timber Piles would not be considered viable options due to the damage potential during driving as bedrock approaches. Pile shoes should be used to protect the piles when driving into the weathered rock zone. Typical pile capacities for ASD and LRFD design are presented in Table 6:

Table 6 - Pile Capacities

Pile Section	Pile Area (sq. in.)	Maximum Nominal Required Bearing (NRB) (Kips)	Allowable Resistance Available (Kips)	Maximum Factored Resistance Available (Kips) Pier 5-C
HP 10X42	12.4	335	112	167
HP 10X57	16.8	454	151	227
HP 12X53	15.5	419	139	209
HP 12X63	18.4	497	165	248
HP 12X74	21.8	589	196	294
HP 12X84	24.6	664	221	332
HP 14X73	21.4	578	192	289
HP 14X89	26.1	705	235	352

For pile foundations which specify a Nominal Required Bearing above 600 kips, in lieu of hammer selection criteria and use of the FHWA Modified Gates formula specified in

Section 512 of the Standard Specifications, the contractor shall conduct a wave equation analysis to establish driving criteria. However, since the piles are so short and the driving time is minimal, the use of HP14X89 piles or larger is not cost effective to warrant a wave equation analysis.

The maximum nominal required bearing (NRB) and factored resistance available (FRA) are determined as per IDOT LRFD Pile Design Guides.

$$NRB = 0.54xF_YA_S$$

$$FRA = NRB\,(\phi_G) - (DD+Scour+Liq.)x(\phi_G)x(\lambda_G) - DDx(\gamma_p)$$

Maximum Factored Resistance Available (FRA) for abutment should be reduced for downdrag force. The downdrag force is determined by multiplying the values given in the table below by the perimeter of the corresponding pile. The Load factor γ_p applied to the downdrag force shall be as recommended by IDOT or as per AASHTO (Table 3.4.1-2).

Table 7 – Downdrag Force for Abutment

Depth El., ft	Downdrag Force, kips/ft
*590 to 576	7.9
576 to 562	11.8

* MSE selected fill material with $\phi = 34^\circ$, and unit weight of 125 pcf.

The downdrag force is significant and will reduce the maximum FRA. As discussed under the SGR for the MSE wrap around wall at the abutment, staged construction, ground improvement, and/or lightweight fill of the embankment will be required to minimize settlements and improve the stability of the abutment MSE wall. During final design it should be determined if there is sufficient FRA and the number of piles at the abutment are reasonable prior to determining if improvements in coordination with the design of the MSE wall needs to be made to the underlying soils to limit the settlement to less than 0.4 inches.

Anticipated pile tip elevations are:

Table 8 - Pile Tip Elevations

Pier No.	Tip Elev. (ft)	Foundation Material
3-C	554.1	Sandstone
Abutment	557.5	Sandstone

For final design, point bearing piles on rock should be designed according to the 2006 LRFD Section 10.7.3.2. Also, a detailed lateral load analysis should be performed on these bents using GROUP 6.0/7.0 or FB MultiPier. The short piles at Pier 3-C may not have adequate embedment to develop fixity. The piles at Pier 3-C may need to be set in rock as specified in Bridge Manual Section 3.10.1.10 or driven on a batter.

Drilled Shafts

As an alternate to driven piles and spread footings, drilled shafts can be considered at Piers 2 and 3-C. AASHTO specifies that drilled shafts be designed to have adequate axial and structural resistances, tolerable settlements, and tolerable lateral displacements.

A single, two and four shaft layout under each column should be evaluated during final design. Where fixed piers are used resulting in high moments due to thermal movements, two to four shafts may be needed to resist the applied loadings. If a single shaft is used beneath the planned oblong pier column, a shaft diameter on the order of 9 feet may be required. For a two shaft supported column, drilled shafts on the order of 4 to 6 foot diameter are expected. A four shaft supported column would have shafts on the order of 3 to 4 foot diameter. Rock socket lengths would typically be on the order of 2 to 3 times the shaft diameter.

A mono column/drilled shaft substructure presents some benefits, namely:

a. Minimal contaminated soil and water disposal as compared to spread footings and driven pile groups.
b. No sheeting or shoring is required.
c. No pile caps or large footing is required.
d. Minimizes or eliminates conflicts with existing foundations.
e. Required limited space and provides maximum flexibility for construction staging.
f. No intensive handwork as required by spread footings.
g. Reduced uncertainty - final depth to quality rock determined during construction, quantity of manual preparation of rock surface, quantity of contaminated soil, groundwater level, dewatering, time for construction, etc.

Axial resistances of drilled shafts socketed into bedrock were evaluated using the methodology presented in 2006 AASHTO LRFD for determining side and tip resistance (Equations 10.8.3.5.4b-1, 10.8.3.5.4c-a, and 10.8.3.5.4c-2). The following ultimate side and tip resistances were calculated and are presented in Table 9 for several pier locations.

Table 9 - Drilled Shaft Unit Side and Unit Tip Resistance

Pier	Material Type	qs (psi)	qp (psi)
2 / 3C	Sandstone	150	350

Note: qs – ultimate skin resistance
qp – ultimate tip resistance

If drilled shafts are preferred, a cost analysis should be conducted for comparison with spread footings and driven piles.

Horizontal movements and stresses induced by lateral loads and applied moments should be evaluated using the methods in GROUP 6.0/7.0 or FB MultiPier software packages. Determination of whether a rock socket is necessary should be evaluated in final design. The effects of group interaction should be accounted for when analyzing the drilled shaft group horizontal response. Hyperbolic p-y curves can be developed for the rock formations using criterion proposed by Ke Yang (Reference 4) that uses theoretical derivations and numerical analysis results.

Abutment Earth Pressures

The proposed Abutment will be partially restrained at the top with MSE wall straps. However, the stub abutments will probably develop active pressure. The following parameters should be used to determine the static earth pressure on the abutment wall:

Table 10 - Abutment Earth Pressure Parameters

Parameter	Recommended Value
Unit Weight	125 pcf
Angle of Internal Friction, φ	34
Angle of Wall Friction, δ	17

Backfill behind the walls should be granular fill according to the latest Illinois DOT standard details.

Conclusions and Recommendations

Based on the analyses and subsurface conditions, conclusions and recommendations are summarized as follows:

- Parameters are provided for the analyses and design of spread footings and driven piles.
- Downdrag forces will develop on the abutment piles and will impact the maximum FRA.

6.0 CONSTRUCTION CONSIDERATIONS

Foundation Construction

The foundation types and bearing elevations closely match the foundations employed when constructing the existing viaduct. In general, the foundation construction and excavation and backfill should follow the plans and Illinois DOT Standard Specifications/Supplemental Specifications.

Spread Footing Construction

The foundations shall be excavated to the lines and approximate depths indicated on the Plans or to such depths determined in the field by the Engineer. It appears that the recommended embedded depths of the foundations for Pier 2 are on the order of 15 feet. Excavated material should be removed from the site and legally disposed of by the Contractor. Excavation should be performed according to the Section 502 of the Illinois DOT Standard Specifications for Road and Bridge Construction.

Special provisions will be required to specify that the final rock bearing surface shall be prepared by barring, picking or wedging, or similar hand methods to remove loose wedges and unsound rock so as to leave the foundation in an entirely sound and unshattered condition with a clean bonding surface. If seepage water is present in the foundation, it must be directed to a sump in one corner of the excavation and removed by pumping or air lift.

The following note should be added to the plans:

> *The bottom of footing elevation shall be adjusted to ensure a minimum embedment of 6 inches in non-weathered rock. The rock excavation shall be made with near-vertical sides at the plan dimensions to allow the sides and base of the embedded portion of the footing to be cast against undisturbed rock surfaces.*

It is anticipated that the soils at the site can be excavated using conventional excavation equipment. For all temporary excavations, space permitting, slopes in soil should be excavated to an inclination no steeper than 2 Horizontal : 1 Vertical. Temporary slopes may experience some sloughing and the Contractor should take caution and follow the appropriate OSHA regulations. Where space is limited, shoring will need to be installed. At Pier 2, River Drive could be impacted if an open cut excavation with side slopes is made.

Further environmental investigations should be conducted to determine whether the materials excavated in the areas identified in the FEIS will need to be disposed in special landfills.

Driven Pile Construction

As stated in 2006 AASHTO LRFD, care should be taken in driving piles to hard rock to avoid tip damage. The piles on this project will be relatively short. Piles should have minimum yield strength of 50 ksi. Pile tips should be protected using a cast steel tip.

Since the piles are so short, dynamic testing is thought not to be of much benefit. Piles should be driven in accordance with Illinois Department of Transportation Standard Specifications. The specifications specify the use of the FHWA Modified Gates formula.

Test piles should be driven at the abutment and bents where piles are specified.

Drilled Shaft Construction

The performance of drilled shafts is sensitive to the installation methods. Drilled shaft construction should follow the applicable sections of the Illinois DOT Standard Specifications for Concrete Drilled Shafts (SS-01032). The following are issues to be considered during final design in preparing the specifications and contract documents should drilled shafts be selected:

- Editing the Standard Specification for drilled shaft construction may be required.
- CSL tubes should be installed properly in each drilled shafts so that the Resident Engineer can select shafts to be integrity tested using Crosshole Sonic Logging (CSL) methodology. The number of tubes and locations should be incorporated into the contract drawings.
- All CSL tubes should be filled with water within two hours of concrete placement, in order to prevent debonding between the CSL tubes and the surrounding concrete. CSL tubes should be covered after being filled with water to keep debris from blocking the tubes.
- Either the State or Contractor should hire a qualified CSL testing company to perform and interpret the results of the CSL testing.
- It is anticipated that the shafts would be installed using soil augers and rock core barrels/rock augers. Temporary casing will need to be installed in the soil overburden. Water infiltration into the shaft excavation should be anticipated.

Drilled Shaft Testing

CSL testing is the preferred testing method during construction to ensure the shaft concrete is free of defects and the bottom of the shaft is sound.

7.0 Final Design Considerations

Final design will be performed using 2006 AASHTO LRFD specifications. The information presented in this report can easily be incorporated into LRFD for strength and service limits. Resistance factors for design of shallow and drilled shaft foundations should be selected from AASHTO LRFD Tables 10.5.5.2.2-1 and 10.5.5.2.4-1. For driven piles, References 10 and 11 provide guidance.

As recommended elsewhere in this report, an additional boring at Pier 3-C should be drilled.

Environmental investigations will be required at the contaminated areas (Deere & Co. parking lot) identified in this report and in other areas identified in the FEIS. Contaminated areas may have a major impact on project construction, cost and schedule. Disposal methods, material quantities, permitting, treatment and disposal of water from excavations, site monitoring activities and personnel protection will need to be evaluated during final design.

A detailed constructability comparison of the three foundation system alternatives should be conducted during final design to ensure the selected foundation system is compatible with the proposed staging phases. This comparison should include but not be limited to construction time, traffic impacts, safety, and risk/uncertainty.

8.0 REFERENCES

1. Technical Memorandum, I-74 Iowa-Illinois Corridor Study – Moline Viaduct & Ramps, Proposed Span Arrangements, dated June 21, 2007.

2. AASHTO LRFD Bridge Design Specifications, 2006 Interim Revisions, Third Edition.

3. AASHTO LRFD Bridge Design Specifications, 2008 Interim Revisions, 4th Edition, 2007.

4. AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002

5. Analysis of Laterally Loaded Drilled Shafts in Rock, A Dissertation Presented to The Graduate Faculty of The University of Akron, In Partial Fulfillment for the Degree Doctor of Philosophy, by Ke Yang, May 2006.

6. JACOBS Technical Memorandum, I-74 Iowa-Illinois Corridor Study, Bridge Design Criteria, dated November 14, 2005.

7. GROUP 6.0/7.0 for Windows, Analysis of a Group of Piles Subjected to Axial and Lateral Loading, Ensoft, Inc., February 2003/February 2006.

8. LPILE 5.0 for Windows, a Program for the Analysis of Piles and Drilled Shafts Under Lateral Loads, July 2004.

9. FB-MultiPier, Bridge Software Institute.

10. 2007 Illinois DOT Standard Specifications for Roadway and Bridge Construction.

11. IDOT Pile Data Guidelines for 2007 Standard Specifications, Bridge Memorandum 06.2, November 17, 2006.

12. IDOT Bridge Manual, May 2008.

13. Interstate 74 Quad Cities Corridor Study, Scott County, Iowa and Rock Island County, Illinois, Final Environmental Impact Statement and Section 4(f) Statement.

14. CH2M Hill, Structure Geotechnical Report, Ramp 6th-C Retaining Wall, Structure Number 081-6019, I-74 Iowa to Illinois Corridor Study, FAI Route 74, Section 81-1HVB, Ramp 6th-C Station 331+18.97 LT to 331+35.14 RT, Rock Island County, Illinois, P-92-032-01, January, 2008. Prepared for Illinois Department of Transportation.

15. “Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Design and Construction Guidelines,” FHWA-NH-00-043, March, 2001.

Appendix

Figures

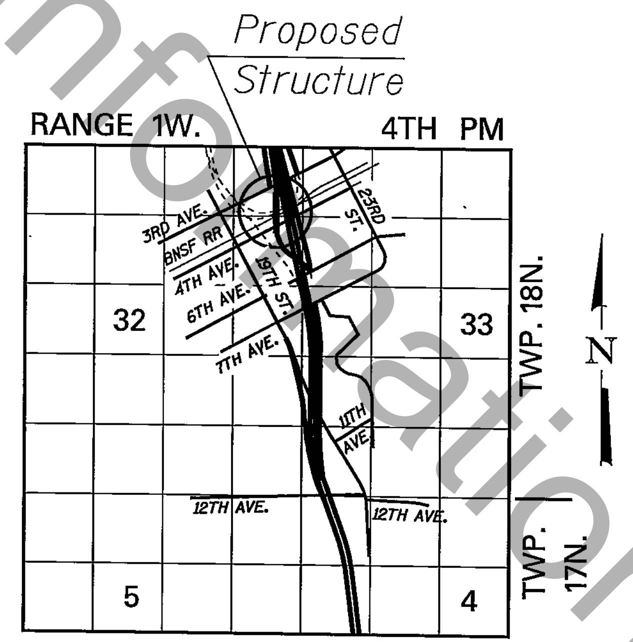


Figure 1
Ramp 6th C Structure Location Map
I-74 Iowa-Illinois Corridor Study
Map of Township 18N, Range 1W
Section 32 & 33

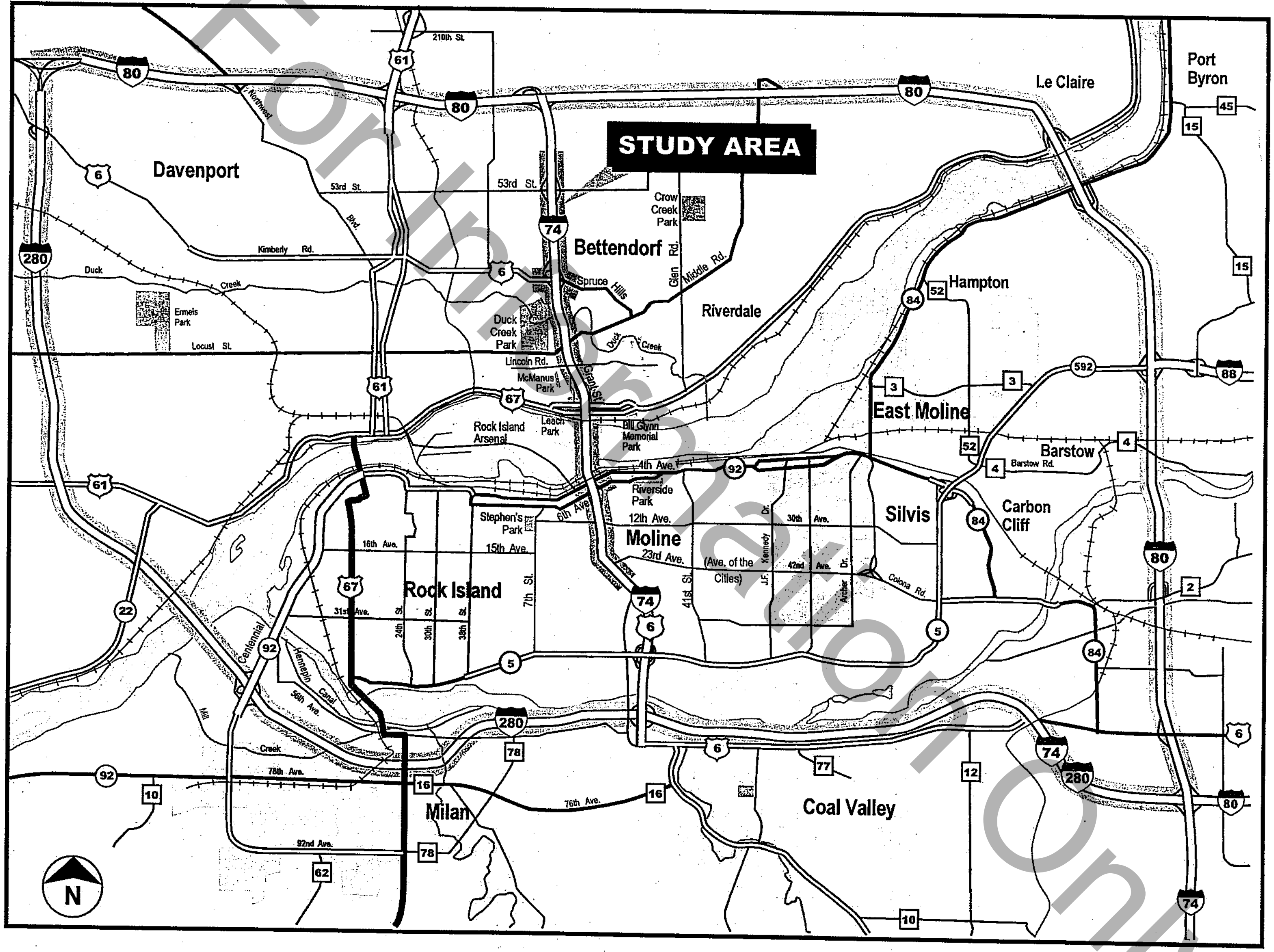


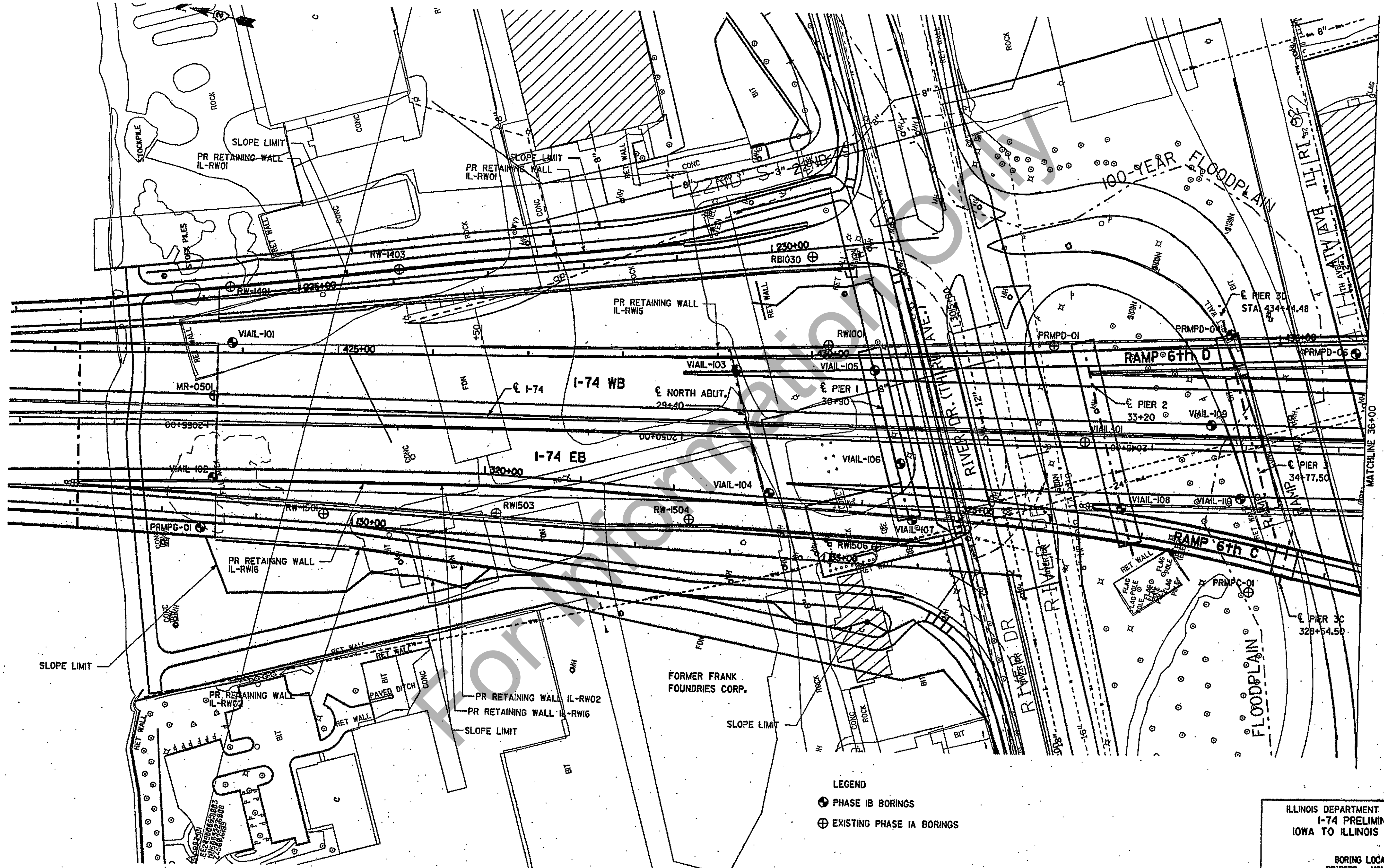
Figure 2
I-74 Iowa - Illinois Corridor Study Location Map

T158835.AA.ES.06 Fig 3-2(Loc_Map) 3-19-03 lg

F.A. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
74		ROCK ISLAND		

STA. TO STA.

FED. ROAD DIST. NO. 2 | ILLINOIS | FED. AID PROJECT



LEGEND

⊕ PHASE IB BORINGS

⊕ EXISTING PHASE IA BORINGS

ILLINOIS DEPARTMENT OF TRANSPORTATION
I-74 PRELIMINARY DESIGN
IOWA TO ILLINOIS CORRIDOR STUDY

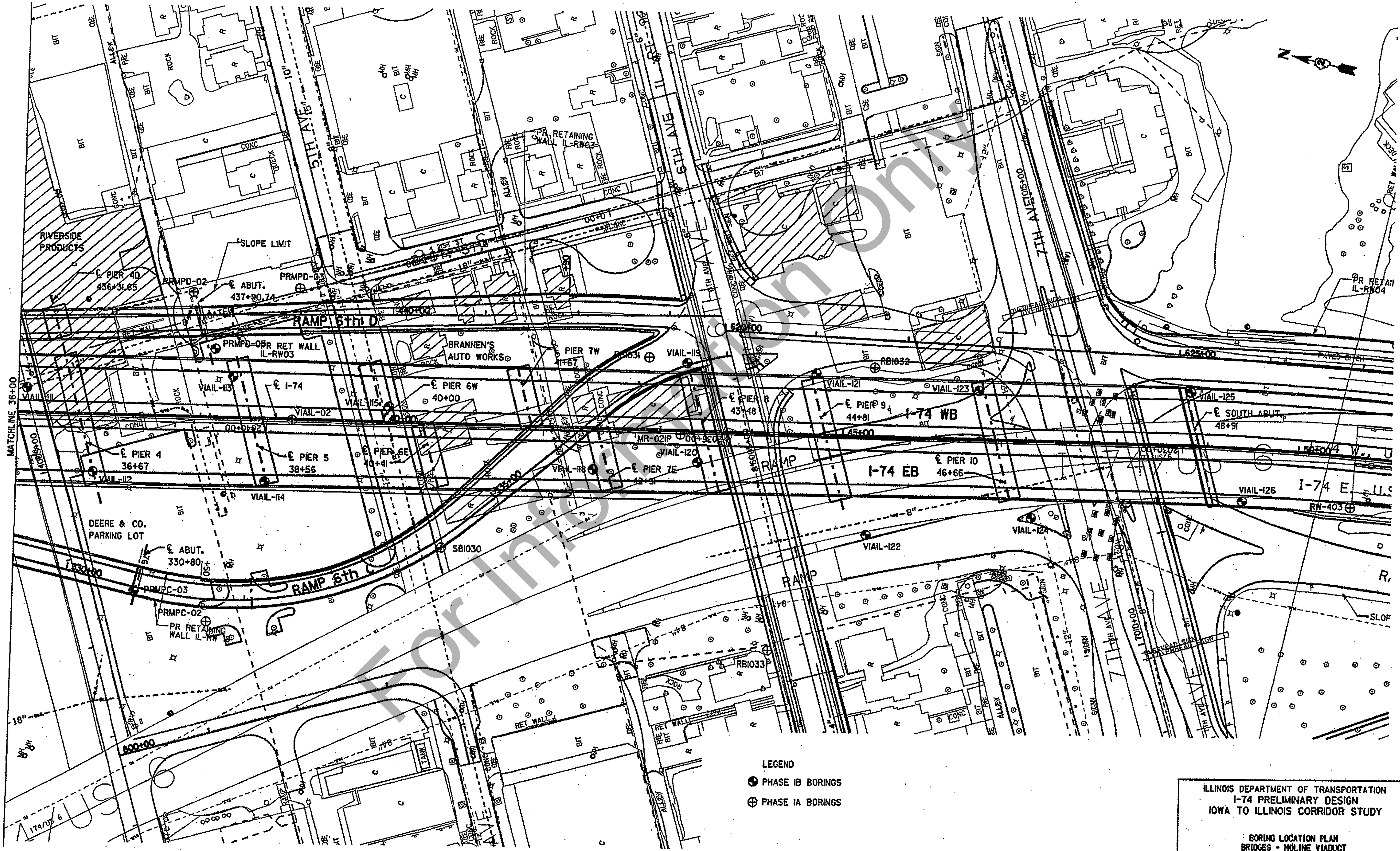
BORING LOCATION PLAN
BRIDGES - MOLINE VIADUCT
FIGURE 3

DRAWN BY

DATE 1 OF 2 CHECKED BY

PLOT DATE = 06-FEB-2008 PLOT SCALE = 100.000000:1.000000 FILE NAME = bor09.dgn

F.A. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
74		ROCK ISLAND		
STA.		TO STA.		
FED. ROAD DIST. NO. 2	ILLINOIS	FED. AID PROJECT		



LEGEND

● PHASE IB BORINGS

⊕ PHASE IA BORINGS

ILLINOIS DEPARTMENT OF TRANSPORTATION
I-74 PRELIMINARY DESIGN
IOWA TO ILLINOIS CORRIDOR STUDY

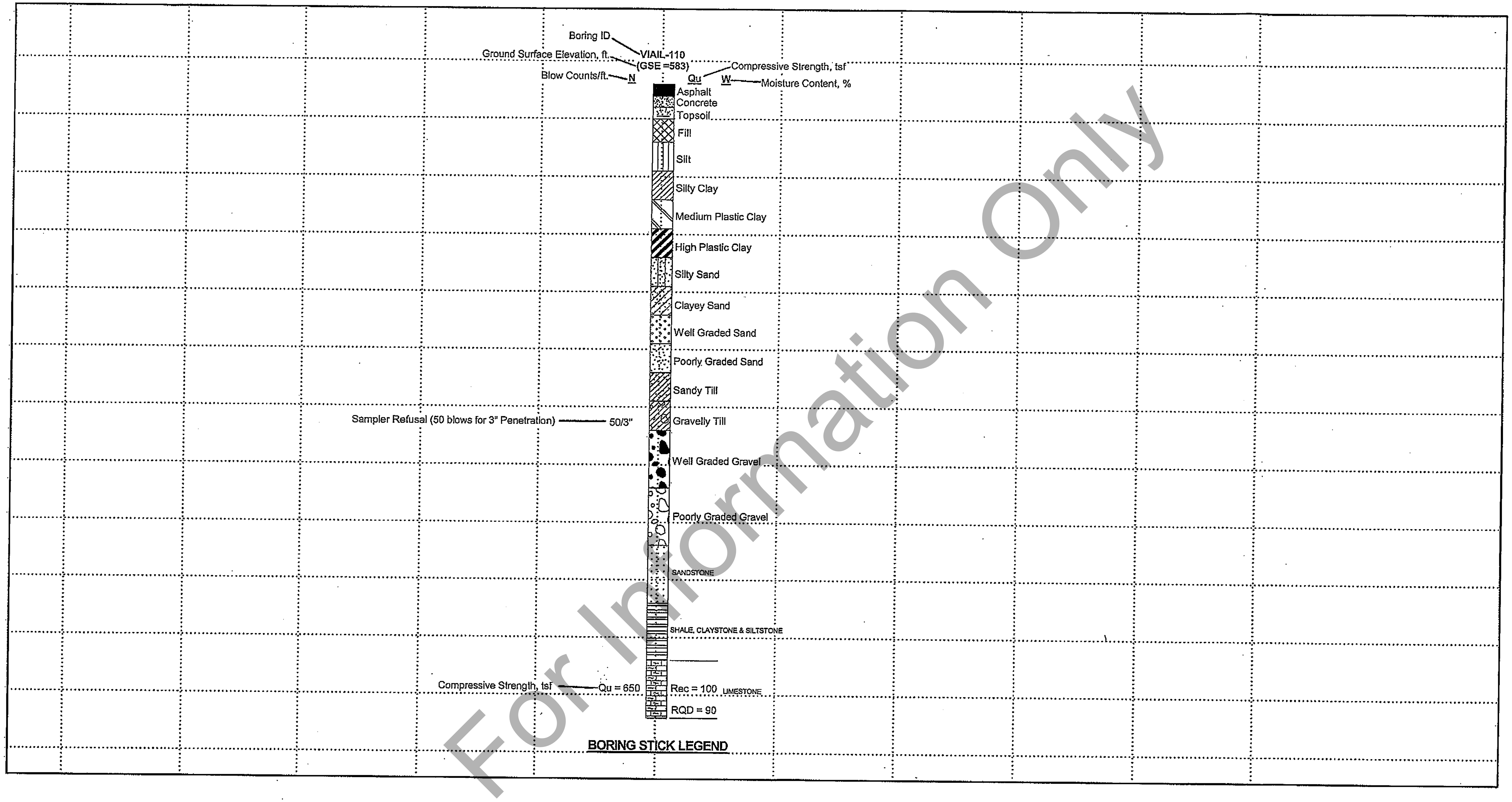
BORING LOCATION PLAN
BRIDGES - MOLINE VIADUCT
FIGURE 3

DRAWN BY

DATE 2 OF 2

CHECKED BY

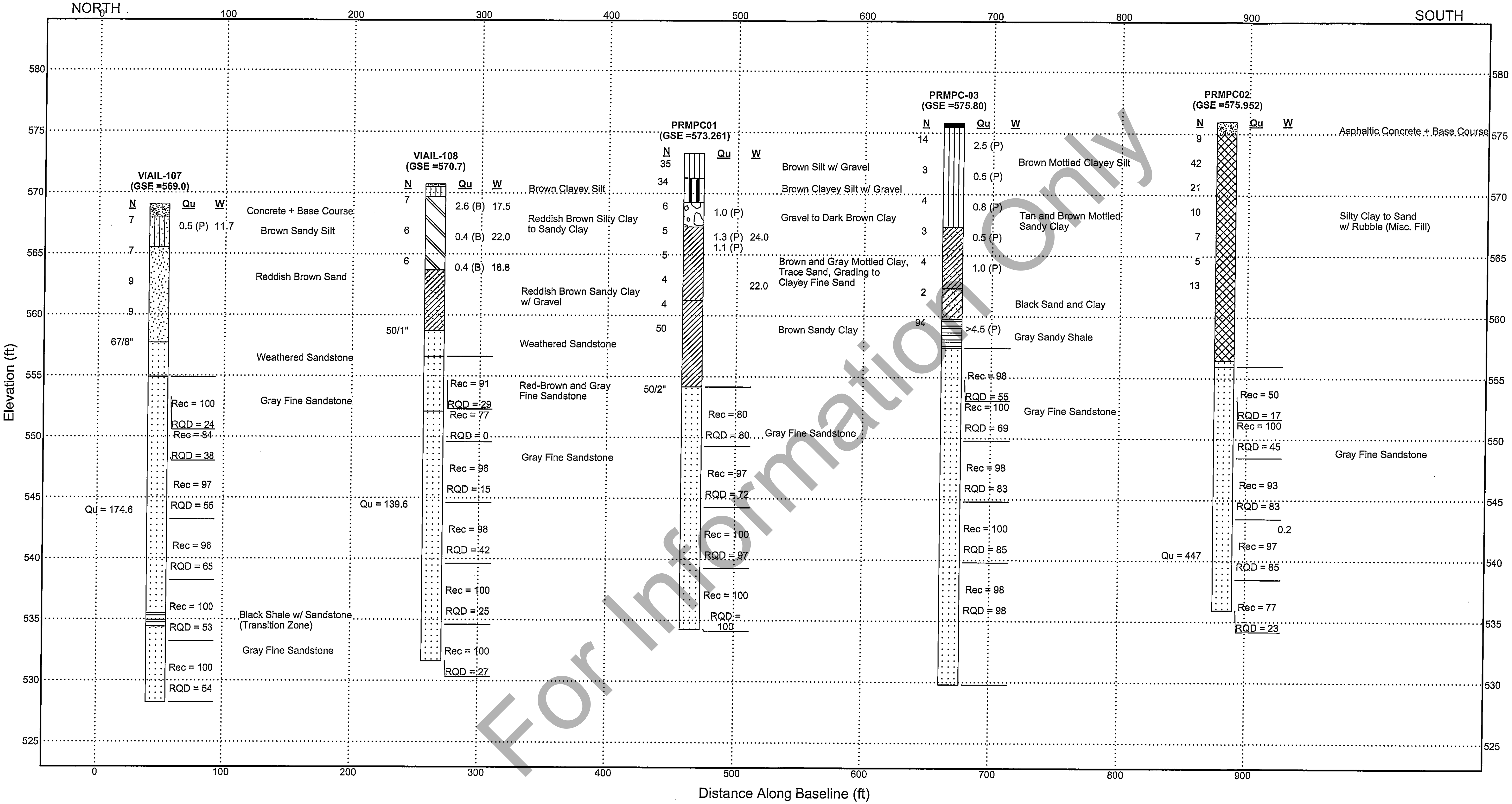
PLOT DATE = 06-FEB-2008 PLOT SCALE = 100.000000:1.000000 FILE NAME = bor10.dgn



BORING STICK LEGEND

SUBSURFACE PROFILE: BORING STICK LEGEND

FIGURE 4: Sheet 1 of 2



SUBSURFACE PROFILE: RAMP C

FIGURE 4: Sheet 2 of 2

BORING LOGS

GRAIN SIZE IDENTIFICATION

Name	Size Limits	U.S. Sieve Size
BOULDERS	12" or greater	
COBBLES	3" to 12"	
GRAVEL		
COARSE	3/4" to 3"	3/4" to 3"
FINE	3/16" to 3/4"	No. 4 to 3/4"
SAND		
COARSE	2.00 mm to 4.75 mm	No. 10 to No. 4
MEDIUM	0.42 mm to 2.00 mm	No. 40 to No. 10
FINE	0.07 mm to 0.42 mm	No. 200 to No. 40
SILT	0.002 mm to 0.07 mm	
CLAY	less than 0.002 mm	

RELATIVE PROPORTIONS OF SECONDARY COMPONENTS

Trace	0% to 10%
Little	10% to 20%
Some	20% to 35%
And	35% to 50%

PLASTICITY

Term	PI
Non-plastic	0-3
Slightly plastic	4-15
Medium plastic	16-30
Highly plastic	>30

RELATIVE DENSITY OF GRANULAR SOILS

SPT N-value (blows/ft)	Relative Density
0-4	Very loose
5-10	Loose
11-30	Medium dense
31-50	Dense
>50	Very dense

STRENGTH AND CONSISTENCY OF COHESIVE SOILS

SPT N-value (blows/ft)	Unconfined Compressive Strength (tons/ft^2)	Consistency
0-2	0.00-0.25	Very soft
3-4	0.25-0.50	Soft
5-8	0.50-1.00	Medium stiff
9-15	1.00-2.00	Stiff
16-30	2.00-4.00	Very stiff
>30	>4.00	Hard

Soil classifications shown on boring logs are determined by visual inspection of samples and from laboratory tests where available.

Split spoon samples are obtained by driving a 2" O.D. sampler 18" with a 140-pound hammer free-falling 30".
(Standard penetration test or "SPT", ASTM 1586)

Numbers shown next to split spoon symbol represent the number of hammer blows for the corresponding penetration (blows/inches).

LEGEND FOR BORING LOGS AND SOIL CLASSIFICATION SYSTEM

JACOBS

PHYSICAL PROPERTIES OF ROCK

Texture

- dense
- fine
- medium
- coarse
- crystalline

Bedding Characteristics

	Spacing
very thin	less than 2 in.
thin	2 in. to 1 ft.
medium	1 ft. to 3 ft.
thick	3 ft. to 10 ft.
massive	greater than 10 ft.

Hardness

	Compressive Strength (tsf)
very soft	10 – 250
soft	250 – 500
hard	500 – 1,000
very hard	1,000 – 2,000
extremely hard	> 2,000

Degree of Weathering

	Description
fresh	unweathered
very slight	rock fresh, joints stained
slight	rock fresh, discoloration may extend 1 in. into rock
moderate	significant portions show discoloration
moderately severe	all rock except quartz discolored
severe	rock fabric clear but reduced to soil strength
very severe	rock fabric discernible but mass reduced to soil
complete	rock reduced to soil, fabric not discernible

Lithologic Charactheristics

- clayey
- shaly
- calcareous
- siliceous
- sandy
- silty

Structure

Bedding Orientation
- gently dipping bedding
- steeply dipping bedding

Fractures
- scattered fractures
- closely spaced fractures
- cemented fractures
- tight fractures
- open fractures
- brecciated (fragmented)

Joints

	Spacing
very close	less than 2 in.
close	2 in. to 1 ft.
moderately close	1 ft. to 3 ft.
wide	3 ft. to 10 ft.
very wide	greater than 10 ft.

Miscellaneous
- slickensided

Solution and Void Conditions

- vuggy (pitted)
- vesicular (igneous)
- porous
- cavities
- cavernous

Miscellaneous

- swelling
- slaking

ROCK CORE PROPERTIES

Recovery (REC) is defined as the length of rock core recovered divided by the length of the core run (in percent).

Rock Quality Designator (RQD) is defined as the total length of rock core pieces greater than 4 in. long divided by the length of the core run (in percent).

RQD (%)	Diagnostic Description
90 – 100	Excellent
75 – 90	Good
50 – 75	Fair
25 – 50	Poor
0 – 25	Very Poor

LEGEND FOR BORING LOGS AND ROCK CLASSIFICATION SYSTEM

JACOBS

Boring Logs
Phase 1 B

Illinois Department of Transportation
Division of Highways
JCI

SOIL BORING LOG

Page 1 of 3

Date 9/4/07

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

SECTION ______ **LOCATION** (N=564052.458, E=2459235.291), **SEC.** 32, **TWP.** 18N, **RNG.** 1W, 4th **PM**

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

STRUCT. NO. ______
Station 330+80

BORING NO. PRMPC-03
Station ______
Offset ______
Ground Surface Elev. 575.80 ft

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

Groundwater Elev.:
First Encounter 562.3 ft ▼
Upon Completion ______ ft
After ______ **Hrs.** ______ ft

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
PAVEMENT - asphalt concrete (4" thick) 575.47				
SILT - yellowish brown to brown and orange-brown mottled to gray, little to some clay, powdery, slightly to medium plastic, medium stiff to stiff, moist		4 5 9	2.5 P	
- dark brown, little to some clay	-5	1 1 2	0.5 P	
		2 2 2	0.8 P	
- some clay, medium plastic 567.30				
CLAY - tan, brown and orange, little to some fine sand, soft to medium stiff, very moist to wet.	-10	2 1 2	0.5 P	
562.30 ▼		WOH 1 3	1.0 P	
SAND - black, fine to coarse, and dark gray medium to high plastic clay, very soft/loose, saturated. **[Note: strong petroleum odor and trace free product in saturated zone at 13.5'-15'; PID = 420 ppm]** 559.80	-15	WOH 2 0		
SHALE - light gray, sandy (hard clay), no laminations, dry. 557.40		20 34 60	>4.5 P	
Borehole continued with rock coring.	-20			

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

Page 2 of 3

Date 9/4/07

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

SECTION ______ **LOCATION** (N=564052.458, E=2459235.291), **SEC.** 32, **TWP.** 18N, **RNG.** 1W, 4th **PM**

COUNTY Rock Island **CORING METHOD** NQ Core

STRUCT. NO. ______
Station 330+80

CORING BARREL TYPE & SIZE NQ Wireline

Core Diameter 1.8 **in**
Top of Rock Elev. 559.80 **ft**
Begin Core Elev. 557.40 **ft**

BORING NO. PRMPC-03
Station ______
Offset ______
Ground Surface Elev. 575.80 **ft**

	DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
SANDSTONE - light brownish gray, fine grained, uniform grain size, well sorted, moderately well cemented, soft, localized black banding and light gray shale pod inclusions, primarily horizontal sandy rough fractures, non-distinct bedding with fractures at thin to thick bedded spacing, slightly weathered to fresh.	557.40 -20	Run 1	98	55	1.5	
-dark gray shale bed with numerous light gray sandstone partings and seams, soft, rock-like at 21' to 22.8'	-25	Run 2	100	69	0.8	
-4" thick dark gray to black sandy shale seam at 25.7' to 26.0' -brown spotted/speckled fine grained sandstone at 26' to 27.3'	-30	Run 3	98	83	0.6	
	-35	Run 4	100	85	0.6	
		Run 5	98	98	0.7	

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

ROCK CORE LOG

Page 3 of 3

Date 9/4/07

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

SECTION **LOCATION** (N=564052.458, E=2459235.291), **SEC.** 32, **TWP.** 18N, **RNG.** 1W, 4th **PM**

COUNTY Rock Island **CORING METHOD** NQ Core

STRUCT. NO.
Station 330+80

BORING NO. PRMPC-03
Station
Offset
Ground Surface Elev. 575.80 ft

CORING BARREL TYPE & SIZE NQ Wireline

Core Diameter 1.8 in
Top of Rock Elev. 559.80 ft
Begin Core Elev. 557.40 ft

	DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
SANDSTONE - light brownish gray, fine grained, uniform grain size, well sorted, moderately well cemented, soft, localized black banding and light gray shale pod inclusions, primarily horizontal sandy rough fractures, non-distinct bedding with fractures at thin to thick bedded spacing, slightly weathered to fresh. *(continued)*	-40 -45					
529.80						
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)

BBS, form 138 (Rev. 8-99)

Illinois Department of Transportation
Division of Highways
JCI

SOIL BORING LOG

Page 1 of 3

Date 8/28/07

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

SECTION ______ **LOCATION** (N=564672.846, E=2459200.272), **SEC.** 32, **TWP.** 18N, **RNG.** 1W, 4th **PM**

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

STRUCT. NO. ______
Station 30+90

BORING NO. VIAIL-107
Station ______
Offset ______
Ground Surface Elev. 569.00 **ft**

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

Groundwater Elev.:
First Encounter 563.0 **ft** ▼
Upon Completion ______ **ft**
After ____ **Hrs.** ______ **ft**

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
CONCRETE - 9" thick pavement + base course 568.00				
SILT - brown, little to some fine sand, trace clay, medium stiff, crumbles readily, moist 565.50		5 4 3	0.5 P	11.7
SAND - reddish brown to brown, fine to medium sand, trace coarse sand, trace silt, loose, moist to saturated below 6' depth - [sand blow-in occurred at 10'-11' depth]) 557.70	-5 ▼ -10	2 3 4 3 5 4 2 3 6		
WEATHERED SANDSTONE - augered through 554.90		3 17 50/2"		
Borehole continued with rock coring.	-15 -20			

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

Page 2 of 3

Date 8/28/07

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

SECTION ______ **LOCATION** (N=564672.846, E=2459200.272), **SEC.** 32, **TWP.** 18N, **RNG.** 1W, 4th **PM**

COUNTY Rock Island **CORING METHOD** NQ Core

STRUCT. NO. ______
Station 30+90

BORING NO. VIAIL-107
Station ______
Offset ______
Ground Surface Elev. 569.00 **ft**

CORING BARREL TYPE & SIZE NQ Wireline

Core Diameter 1.8 **in**
Top of Rock Elev. 557.70 **ft**
Begin Core Elev. 554.90 **ft**

		DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
SANDSTONE - brownish gray to gray, fine grained, with minor thin black banding, porous, moderately to well cemented, soft, non-distinct horizontal planar fractures at thin to medium bedding spacing, occasional shale seams, slightly weathered to fresh	554.90	-15	Run 1	100	24		
- possible 9" core loss at 15.8' to 16.6'. Driller reported black water return (shale?) at top of run [Driller reported no voids/seams in Run 2. Loss could be due to wash out of poorly cemented material]		-20	Run 2	84	38		
- shale partings at 18.3' (1/3"), 22.9' (1/4"), 24.0' (1/3") - iron-stained layer at 25.8'-25.9'		-25	Run 3	97	55	0.6	174.6
- iron-stained gray fine sandstone with black seams and limestone clasts at 29.0'-29.3' - numerous black shale partings at 29.3'-30.1'		-30	Run 4	96	65	1.4	
	535.50		Run 5	100	53	1.6	
SHALE - dark gray to black shale with light gray sandstone partings (transtional zone)							

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

ROCK CORE LOG

Page 3 of 3

Date 8/28/07

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

SECTION ______ **LOCATION** (N=564672.846, E=2459200.272), **SEC.** 32, **TWP.** 18N, **RNG.** 1W, 4th **PM**

COUNTY Rock Island **CORING METHOD** NQ Core

STRUCT. NO. ______
Station 30+90

BORING NO. VIAIL-107
Station ______
Offset ______
Ground Surface Elev. 569.00 **ft**

CORING BARREL TYPE & SIZE NQ Wireline

Core Diameter 1.8 **in**
Top of Rock Elev. 557.70 **ft**
Begin Core Elev. 554.90 **ft**

	DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
534.40						
SANDSTONE - light brownish gray, fine grained with black "needle" inclusions and occasional gray shale pods, soft, well cemented with some healed vertical joints, fresh	-35					
		Run 6	100	54	1	
	-40					
528.20						
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
JCI

SOIL BORING LOG

Page 1 of 3

Date 8/30/07

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

SECTION ____ **LOCATION** (N=564459.202, E=2459256.895, **SEC.** 32, **TWP.** 18N, **RNG.** 1W, 4th **PM**

COUNTY Rock Island **DRILLING METHOD** HSA, CME 550X **HAMMER TYPE** CME AUTOMATIC

STRUCT. NO. ____
Station 33+20

BORING NO. VIAIL-108
Station ____
Offset ____
Ground Surface Elev. 570.70 **ft**

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

Groundwater Elev.:
First Encounter 563.7 **ft** ▼
Upon Completion ____ **ft**
After ____ **Hrs.** ____ **ft**

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
TOPSOIL - (grass roots, silt) 2" thick	570.50				
SILT - brown to dark brown, little to some clay, moist	569.70		2		
CLAY - reddish brown, little grading to and silt, trace fine sand grading to sandy clay, medium to high plastic, very stiff to soft, moist			4	2.6	17.5
			3	B	
			2		
			3	0.4	22.0
		-5	3	B	
[Upon completion, offset 7' and drilled to 4' depth for Shelby tube sample.]					
			2		
	563.70 ▼		2	0.4	18.8
CLAY - reddish brown, sandy, saturated, grading downward to clayey sand with gravel			4	B	
[shelby tube recovery unsuccesful at 8.5'-10']					
		-10			
[driller reported sand blow-in after pulling out the shelby tube]	558.70		25		
WEATHERED SANDSTONE - augered through			50/1"		
	556.60				
Borehole continued with rock coring.					
		-15			
		-20			

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

Page 2 of 3

Date 8/30/07

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

SECTION ______ **LOCATION** (N=564459.202, E=2459256.895, **SEC.** 32, **TWP.** 18N, **RNG.** 1W, 4th **PM**

COUNTY Rock Island **CORING METHOD** NQ Core

STRUCT. NO. ______
Station 33+20

CORING BARREL TYPE & SIZE NQ Wireline

Core Diameter 1.8 **in**
Top of Rock Elev. 558.70 **ft**
Begin Core Elev. 556.60 **ft**

BORING NO. VIAIL-108
Station ______
Offset ______
Ground Surface Elev. 570.70 **ft**

		DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
SANDSTONE - brownish gray to mostly red brown, fine grained, iron-pigmented, uniform, well sorted, soft, moderately well cemented, non-distinct bedding at very thin to thin bedded spacing, horizontal fractures, slightly to moderately weathered	556.60		Run 1	91	29	1	
		-15					
			Run 2	77	0	1.2	
SANDSTONE - gray, fine grained, with occasional light gray shale pods and localized bandings, uniform, well sorted, porous, moderately well to well cemented, soft, non-distinct bedding with primarily horizontal sandy planar to slightly undulating fractures ranging from very thin to thin bedded spacing, fresh	552.10						
		-20					
			Run 3	96	15	0.8	
							139.6
		-25					
			Run 4	98	42	1.2	
		-30					
			Run 5	100	25	1.2	

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

ROCK CORE LOG

Page 3 of 3

Date 8/30/07

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

SECTION ______ **LOCATION** (N=564459.202, E=2459256.895, **SEC.** 32, **TWP.** 18N, **RNG.** 1W, 4th **PM**

COUNTY Rock Island **CORING METHOD** NQ Core

STRUCT. NO. ______
Station 33+20

CORING BARREL TYPE & SIZE NQ Wireline

Core Diameter 1.8 **in**
Top of Rock Elev. 558.70 **ft**
Begin Core Elev. 556.60 **ft**

BORING NO. VIAIL-108
Station ______
Offset ______
Ground Surface Elev. 570.70 **ft**

	DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
SANDSTONE - gray, fine grained, with occasional light gray shale pods and localized bandings, uniform, well sorted, porous, moderately well to well cemented, soft, non-distinct bedding with primarily horizontal sandy planar to slightly undulating fractures ranging from very thin to thin bedded spacing, fresh *(continued)*	-35					
		Run 6	100	27	1	
531.60						
End of Boring	-40 -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)

Boring Logs
Phase 1A

Illinois Department of Transportation
Division of Highways
JCI

SOIL BORING LOG

Page 1 **of** 1

Date 10/31/05

ROUTE I-74 **DESCRIPTION** **LOGGED BY** L. Hunt

SECTION **LOCATION** VIADUCT, RAMP 6TH-C, **SEC., TWP., RNG.**

COUNTY Rock Island **DRILLING METHOD** CME-550 Hollow Stem Auger **HAMMER TYPE**

STRUCT. NO.
Station

BORING NO. PRMPC01
Station
Offset
Ground Surface Elev. 573.26 **ft**

Surface Water Elev. ft
Stream Bed Elev. ft

Groundwater Elev.:
First Encounter 562.3 ft ▼
Upon Completion ft
After **Hrs.** ft

	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Silt (ML) Silt, little gravel, brown to light brown to black, dry to moist.		5		
		13		
		22		
571.26		12		
Clayey Silt to Gravel(MH-GM) Clayey Silt to Gravel, little brick, dark brown to white, dry to moist, stratified.		12		
		16		
		18		
569.26		12		
Poorly Graded Gravel to Clay (GP-CL) Poorly Graded Gravel to Clay, trace sand, brown to dark brown, dry to moist, stratified.		7		
	-5	4	1.0	
		2	P	
567.26		2		
Clay (CL) Clay, trace sand, gray brown, mottled orange brown and brown, dry to moist, homogeneous. Shelby sample from 6ft-8ft obtained from adjacent hole on 11/10/2005		2		
		2	1.3	24.0
		3	P	
		3	1.1	
		Push	P	
Clay, trace sand and gravel, gray brown, mottled orange brown and brown, dry to moist, homogeneous.		2		
		2		
	-10	3		
		3		
(CL-SC) Clay to Clayey Fine Sand, trace gravel, gray brown, mottled orange brown and brown, wet, stratified. ▼		3		22.0
		2		
561.26		2		
Water at 11ft while drilling		3		
Sandy Clay(CL) Sandy Clay, trace gravel, brown, wet, homogeneous.		2		
		2		
		2		
		6		
Sandy Clay, trace gravel, brown, wet, homogeneous.	-15	7		
		16		
		34		
		50/5		
554.16				
Auger refusal at 19ft at 13:39, start coring Horizontal fractures,		50/2		
	-20			

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

Page 1 of 1

Date 10/31/05

ROUTE I-74 **DESCRIPTION** ______ **LOGGED BY** L. Hunt

SECTION ______ **LOCATION** VIADUCT, RAMP 6TH-C, **SEC.**, **TWP.**, **RNG.**

COUNTY Rock Island **CORING METHOD** NQ DOUBLE BARREL DIAMOND TIP

STRUCT. NO. ______
Station ______

BORING NO. PRMPC01
Station ______
Offset ______
Ground Surface Elev. 573.26 ft

CORING BARREL TYPE & SIZE ______

Core Diameter ______ in
Top of Rock Elev. 554.16 ft
Begin Core Elev. 554.16 ft

		DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
Sandstone Sandstone, gray, fine to medium grained, slightly weathered, weak rock, laminated to medium bedding, poorly to well sorted, well rounded.	554.16	-20	R1	80	80		
Sandstone, gray, fine to medium grained, slightly to moderately weathered, very weak to weak rock, laminated to thick beds, well sorted, well rounded. Horizontal fractures, extremely fractured to sound, extremely close to wide discontinuity, smooth to rough (planar) joints, slightly altered to stiff clay mineral coatings with >1/4" thick rock wall separation; alternates rock/clay for about 10" at 24.83'.		-25	R2	97	72		
Sandstone, gray, fine to medium grained, unweathered to slightly weathered, very weak rock, thick to massive beds, well sorted, well rounded. Horizontal fractures, moderately fractured to sound, very close to wide discontinuity, smooth to rough (undulating) joints, unaltered to slightly altered joints with no clay mineral coatings.		-30	R3	100	97		
Sandstone, gray, fine to medium grained, unweathered, very weak rock, thick to massive beds, well sorted, well rounded. Horizontal fractures at ends, sound, wide to very wide discontinuity, no joints, unbroken rock core.	534.26	-35	R4	100	100		

End of Boring

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)

BBS, form 138 (Rev. 8-99)

Illinois Department of Transportation
Division of Highways
JCI

SOIL BORING LOG

Page 1 of 1

Date 12/15/05

ROUTE I-74 **DESCRIPTION** ______ **LOGGED BY** B. Karnik

SECTION ______ **LOCATION** VIADUCT, RAMP 6TH-C, **SEC.**, **TWP.**, **RNG.**

COUNTY Rock Island **DRILLING METHOD** CME-550 6" power auger, HSA **HAMMER TYPE** Automatic CME-50

STRUCT. NO. ______
Station ______

BORING NO. PRMPC02
Station ______
Offset ______
Ground Surface Elev. 575.95 **ft**

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

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

		DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
3" asphalt concrete, underlain by 9" crushed gravel	574.95				
Miscellaneous Fill Poorly graded sand, brown, moist, fine to coarse, fill, underlain by 3" thick brick, clay, gravel mix			5		
			4		
			5		
			14		
Sand, gravel, silty clay mix			28		
			24		
			18		
		-5	8		
Concrete pieces, gravel, sand			3		
			11		
			10		
			5		
Bricks, concrete rubble, gravel, silty clay, gray, brown, moist, soft, low plasticity			6		
			6		
			4		
			3		
Reddish brown silty sandy clay, moist, soft/loose, fine sand seams with alternating silty clay seams			4		
		-10	4		
			3		
			3		
Gray sandy clay, moist/wet, soft, fine sand and fines with iron oxide streaks with poorly graded fine to medium sand seams			3		
			3		
			2		
			3		
Gray/black sandy clay, moist/wet, asphalt concrete with petroleum odor			3		
			7		
			6		
		-15	8		
	556.45				
Sandstone Auger refusal and	555.95	-20	50/2		

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

Page 1 of 1

Date 12/15/05

ROUTE I-74 **DESCRIPTION** **LOGGED BY** B. Karnik

SECTION **LOCATION** VIADUCT, RAMP 6TH-C, **SEC.**, **TWP.**, **RNG.**

COUNTY Rock Island **CORING METHOD** NQ DOUBLE BARREL DIAMOND TIP

STRUCT. NO.
Station

BORING NO. PRMPC02
Station
Offset
Ground Surface Elev. 575.95 ft

CORING BARREL TYPE & SIZE
Core Diameter in
Top of Rock Elev. 555.95 ft
Begin Core Elev. 555.95 ft

		DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
Sandstone Light gray, fine grained, slightly weathered, weak to moderately strong, extremely to moderately fractured Horizontal fractures, no staining, extremely close to close spacing, vertical fracture at bottom 3", black sandstone striations throughout, smooth undulating joints, thin silty infilling at 9" from the top, no infilling elsewhere	555.95		R-1	50	17		
Light gray, fine grained, slightly weathered, weak to moderately strong, extremely fractured to sound, with shale seams throughout, Coring rate: 4 minutes for 2.5' Fractures are mostly horizontal, extremely close to moderate spacing, no staining, smooth undulating joint surfaces, highly fractured zones at 2' 3" and 4' 6" from the top, zones have silty infilling coating with fractured pieces		-25	R-2	100	45		
Light gray, fine grained, extremely fractured to sound, unweathered, moderately strong, shale seams scattered throughout Coring rate: 14 minutes for 5' Horizontal fractures, no staining, smooth undulating surfaces, discontinuities are extremely close to moderately spaced, shaley infilling (very thin) and coating at some joint surfaces, tightly healed joints		-30	R-3	93	83		
moderately fractured to sound, unweathered Coring rate: 6 minutes for 5' Horizontal joints, no staining, smooth undulating joints, some joints are at 20 degrees, no infilling except at 37' where 2" thick soft silty infilling is present preventing rock wall contact other joints are tightly healed, close to moderately spaced discontinuities		-35	R-4	97	85		447.0
Light gray, fine grained, no shale seams, extremely fractured to slightly fractured, moderately strong, slightly weathered Horizontal joints, no staining, no infilling, very close to close spacing, rough irregular surfaces, tightly healed joints	535.95	-40	R-5	77	23		

End of Boring
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)

Laboratory Test Results

SUMMARY OF LABORATORY TEST RESULTS FOR SOIL

PROJECT NO: C1X13500
PROJECT: I-74 River Crossing, Bettendorf-Moline
Illinois Land Based Borings

Boring	Sample No.	Depth			Moisture Content %	Dry Unit Weight pcf	Atterberg Limits			Grain Size Passing				Compressive Strength tsf
		From	To				LL %	PL %	PI %	4 %	10 %	40 %	200 %	
PRMPC-03	SS-2	3.5	5.0		16.1									
	SS-4	8.5	10.0		21.3									
VIAIL-108	SS-1	1.0	2.5		17.5									
	SS-2	3.5	5.0		22.0		24	16	8					
	SS-3	6.0	7.5		18.8									

H.C. Nutting Company
611 Lunken Park Dr.
Cincinnati, Ohio 45226

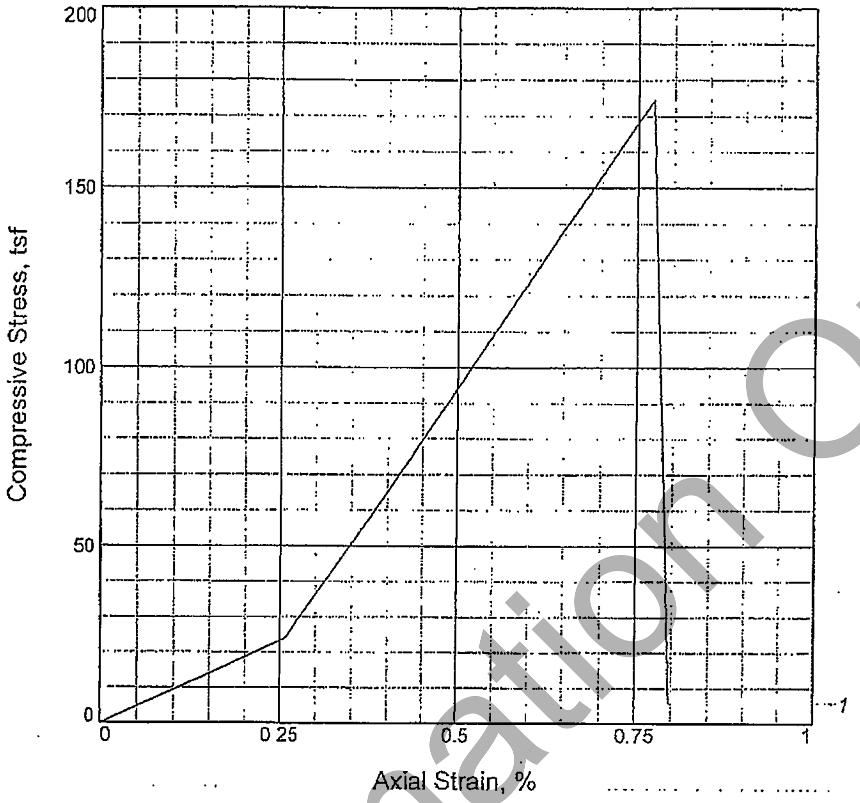
Terracon
I-74 Crossing-Bettendorf-Moline (Job #07045052)
Baettendorf, Iowa
HCN W.O. #19636

TABLE: TABULATION OF UNDISTURBED DATA

Lab No.	Boring No.	Sample No.	Depth (ft.)	Unconfined Strength (tsf)	Material Description	Moh's Hardness	Failure Strain (%)	Dry Density (pcf)	Water Content (%)
~~9976~~	~~VIAIL-101~~	~~RUN 3~~	~~22.2-22.5~~	~~141.1~~		~~3~~	~~2.1~~	~~137.8~~	~~0.4~~
~~9977~~	~~VIAIL-103~~	~~RUN 2~~	~~22.2-22.7~~	~~170.6~~	~~Shaly Sandstone~~	~~6.5~~	~~6.9~~	~~152.5~~	~~0.8~~
~~9978~~	~~VIAIL-104~~	~~RUN 2~~	~~16.7-17.2~~	~~305.0~~	~~Shaly Sandstone~~	~~3~~	~~1.3~~	~~152.7~~	~~0.8~~
~~9979~~	~~VIAIL-105~~	~~RUN 2~~	~~18-18.4~~	~~306.4~~	~~Sandstone~~	~~1~~	~~1.3~~	~~144.9~~	~~5.2~~
~~9980~~	~~VIAIL-105~~	~~RUN 4~~	~~30.2-30.8~~	~~179.5~~	~~Sandstone~~	~~1~~	~~1.0~~	~~147.0~~	~~3.3~~
~~9981~~	~~VIAIL-106~~	~~RUN 2~~	~~17.7-18.8~~	~~226.2~~	~~Sandstone~~	~~4~~	~~4.2~~	~~122.5~~	~~0.6~~
9982	VIAIL-107	RUN 3	22-22.8	174.6	Sandstone	4	0.8	123.0	1.2
9983	VIAIL-108	RUN 3	23.4-23.8	139.6	Sandstone	3	0.9	128.7	1.0
~~9984~~	~~VIAIL-109~~	~~RUN 2~~	~~29.8-30.2~~	~~114.6~~	~~Sandstone~~	~~3~~	~~3.4~~	~~122.3~~	~~4.5~~
~~9985~~	~~VIAIL-110~~	~~RUN 3~~	~~32-32.8~~	~~239.9~~	~~Sandstone~~	~~1~~	~~0.2~~	~~118.6~~	~~0.2~~
~~9986~~	~~PRMPG-01~~	~~RUN 4~~	~~27.4-28~~	~~190.8~~	~~Sandy Shale~~	~~4~~	~~2.1~~	~~141.0~~	~~2.4~~

Un table 9-13-07

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	174.5702			
Undrained shear strength, tsf	87.2851			
Failure strain,	0.8			
Strain rate, in./min.	0.500			
Water content, %	1.2			
Wet density, pcf	124.5			
Dry density, pcf	123.0			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.850			
Specimen height, in.	3.890			
Height/diameter ratio	2.10			

Description: SANDSTONE (MOH'S - 4)

LL = **PL =** **PI =** **GS=** **Type:** Sandstone

Project No.: 19636.040

Date: 9-14-07

Remarks:
Lab No. 9982

Figure ________

Client: TERRACON (#07045052)

Project: I-74 CROSSING-BETTENDORF-MOLINE

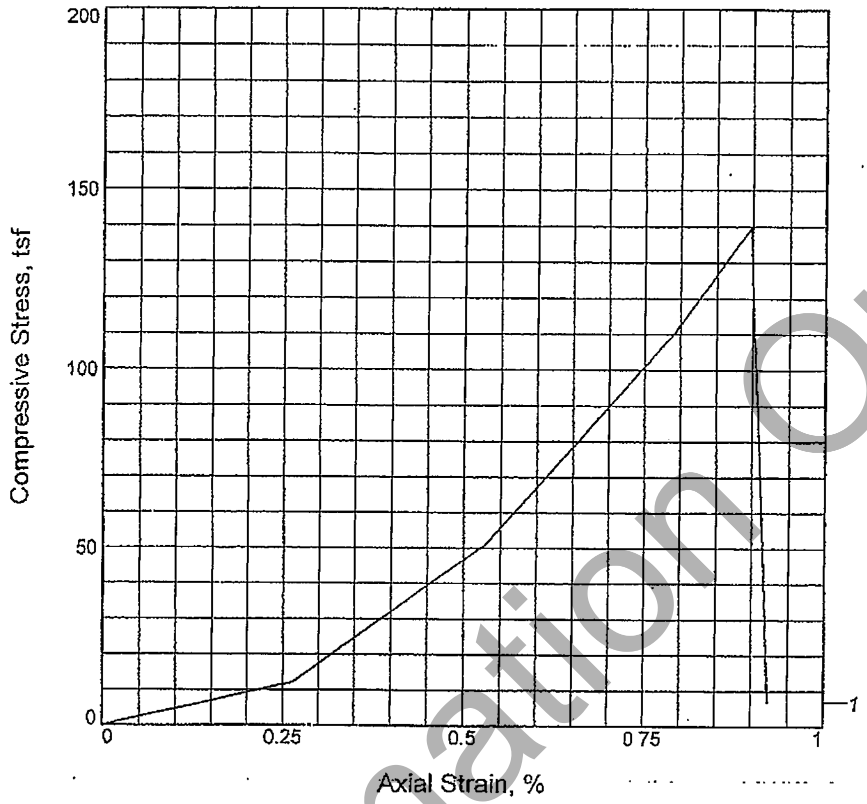
Source of Sample: VIAIL-107 **Depth:** 22-22.8'
Sample Number: RUN-3

UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

Tested By: JB **Checked By:** GS

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	139.5745			
Undrained shear strength, tsf	69.7873			
Failure strain,	0.9			
Strain rate, in./min.	0.500			
Water content, %	1.0			
Wet density, pcf	129.9			
Dry density, pcf	128.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.850			
Specimen height, in.	3.790			
Height/diameter ratio	2.05			

Description: SANDSTONE (MOH'S - 3)

LL = **PL =** **PI =** **GS=** **Type:**

Project No.: 19636.040

Date: 9-14-07

Remarks:
Lab No. 9983

Figure ______

Client: TERRACON (#07045052)

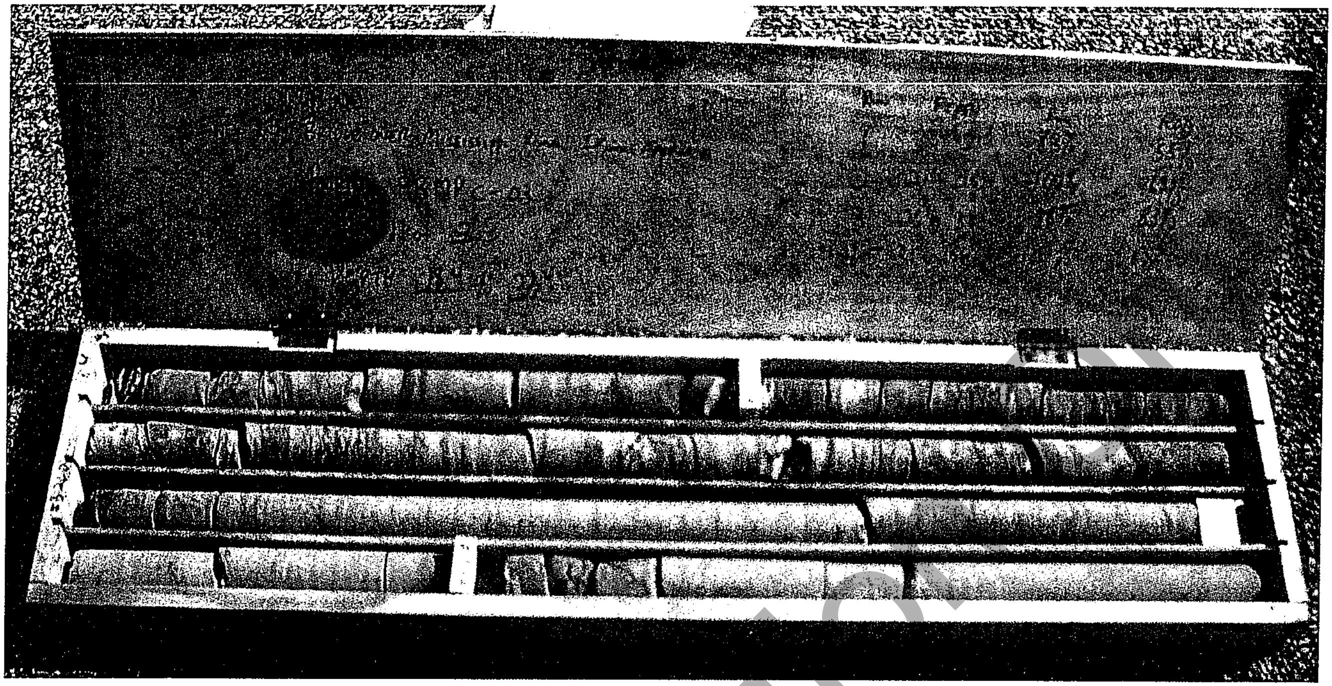
Project: I-74 CROSSING-BETTENDORF-MOLINE

Source of Sample: VIAIL-108 **Depth:** 23.4-23.8'
Sample Number: RUN-3

UNCONFINED COMPRESSION TEST
H. C. NUTTING COMPANY

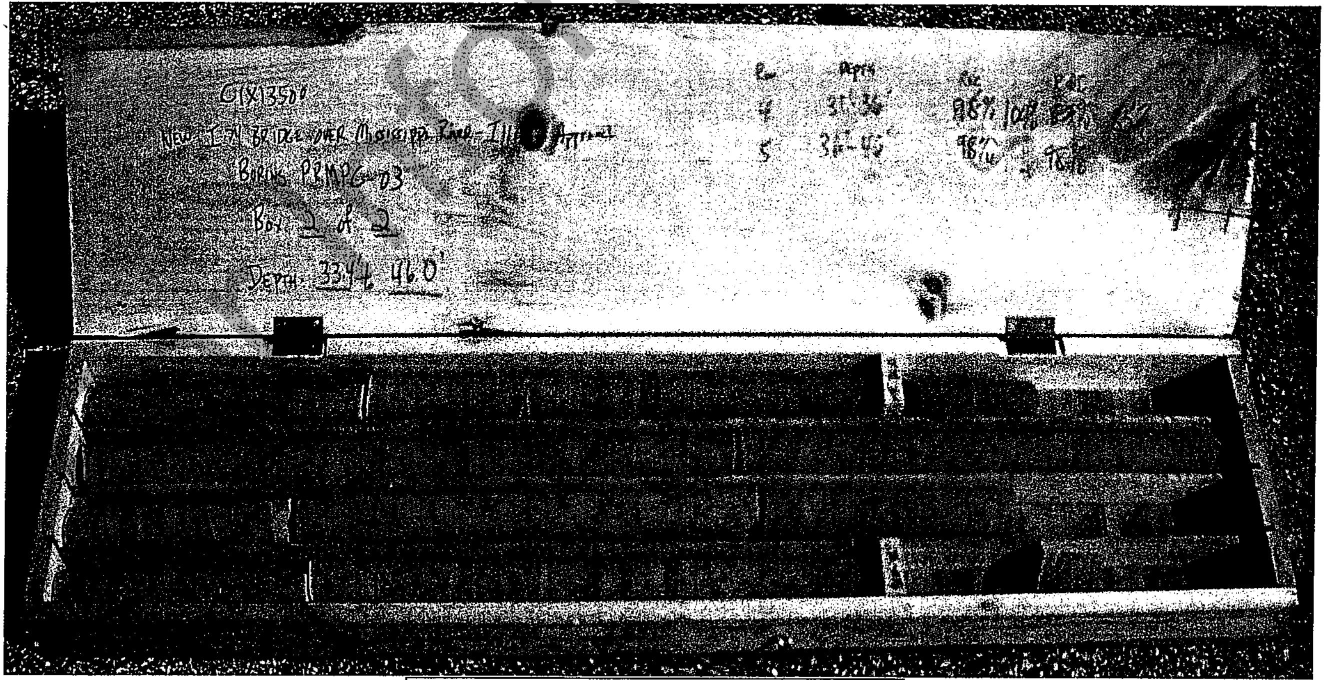
Tested By: JB **Checked By:** GS

ROCK CORE PHOTOGRAPHS



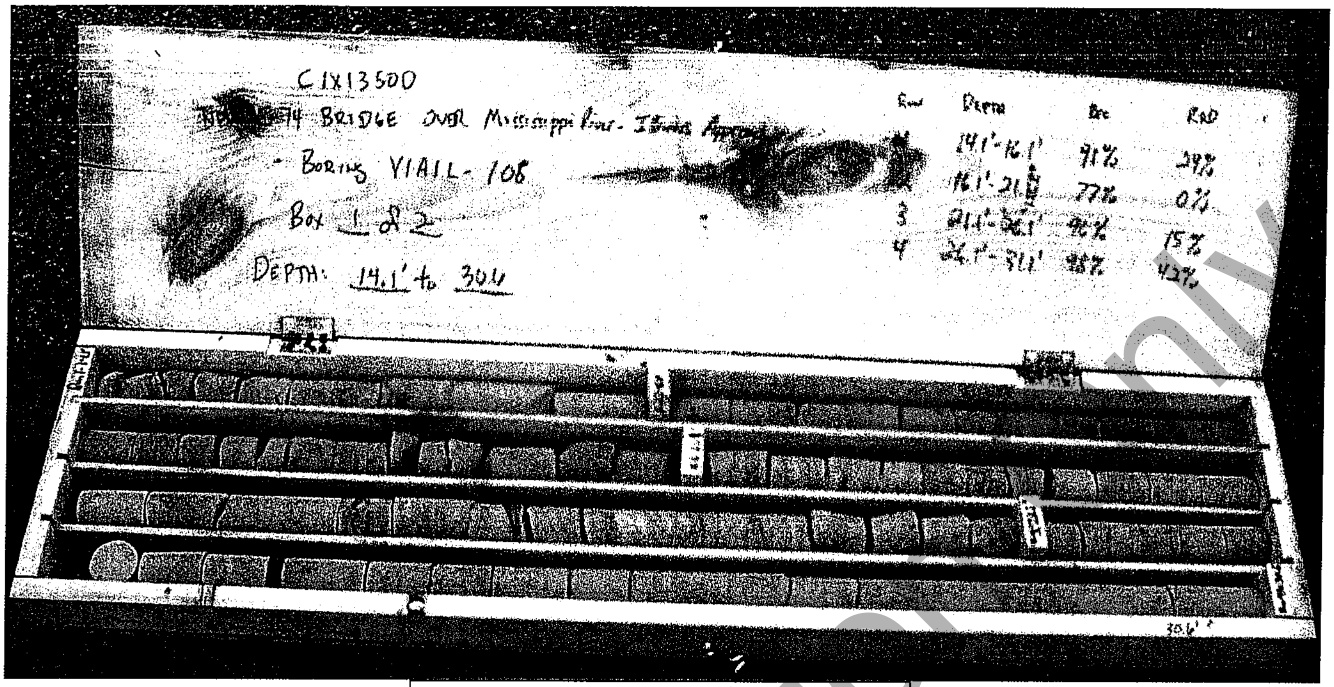
Boring PRMPC-03

Run	Depth (ft)	REC (%)	RQD (%)
1	18.4 – 21.0	98	55
2	21.0 – 26.0	100	69
3	26.0 – 31.0	98	83
4	31.0 – 36.0	100	85



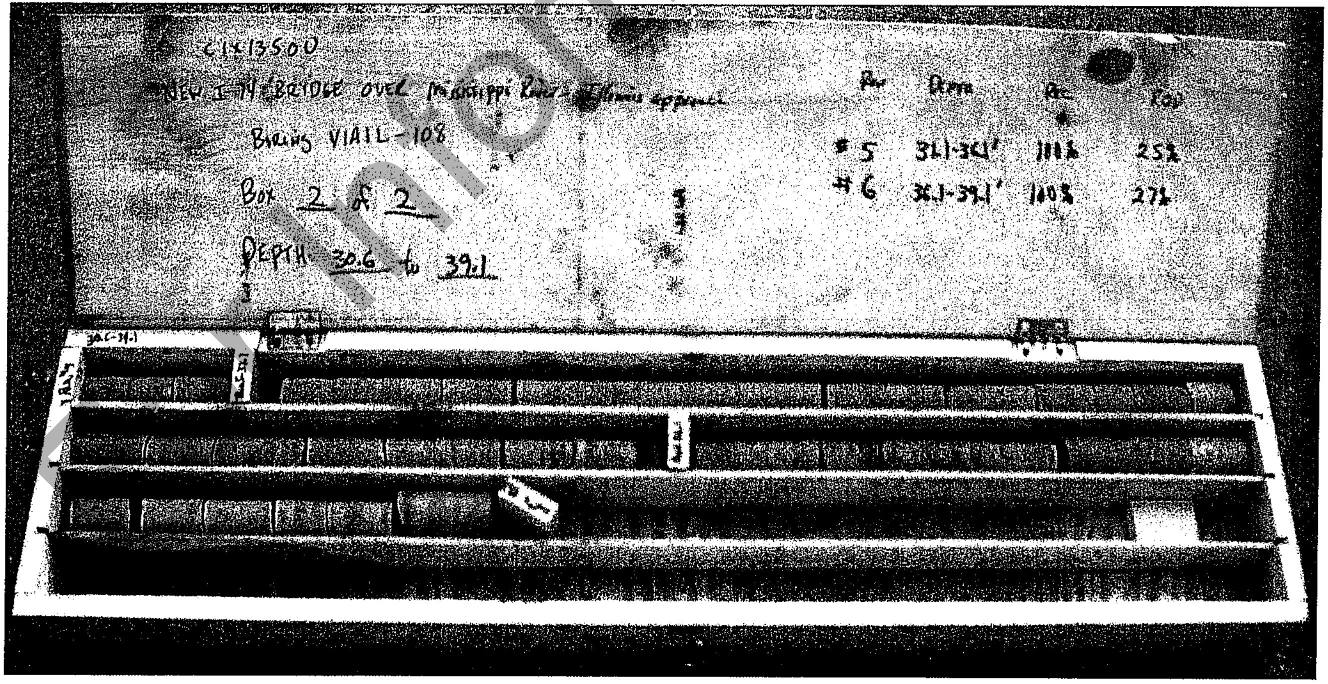
Boring PRMPC-03

Run	Depth (ft)	REC (%)	RQD (%)
5	36.0 – 46.0	98	98



Boring VIAIL-108

Run	Depth (ft)	REC (%)	RQD (%)
1	14.1 – 16.1	91	29
2	16.1 – 21.1	77	0
3	21.1 – 26.1	96	15
4	26.1 – 31.1	98	42



Boring VIAIL-108

Run	Depth (ft)	REC (%)	RQD (%)
5	31.1 – 36.1	100	25
2	36.1 – 39.1	100	29

Summary of RMR and Elastic Moduli

SUMMARY OF ROCK MASS RATING (RMR) AND ELASTIC MODULI

Pier	Boring No.	Run No.	REC (%)	RQD (%)	RMR (Lower)	RMR (Upper)	RMR (Ave.)	Em (ksi)	Ei (ksi)
33+20 Pier 2	VIAIL-108	1	91	29	43	45	44	1026.5	
		2	77	0	35	42	39	747.9	
		3	96	15	41	44	43	941.6	269
		4	98	42	45	51	48	1292.3	
		5	100	25	43	48	46	1119.1	
		6	100	27	43	47	45	1087.3	

Em: Elastic Modulus of Rock Mass
Ei: Elastic Modulus of Intact Rock from Test

Illinois Department of Transportation

Structure Geotechnical Report Responsibility Checklist

Structure Number: 81-1HVB (prop.) 081-0186 (exist.) Contract Number: ______ Date: 6/26/2008
Route: I-74 Section: Ramp 6th - C County: Rock Island
TSL plans by: Jacobs
Structure Geotechnical Report and Checklist by: Jacobs
IDOT Structure Geotechnical Report Approval Responsibility : ☒ Qualified District Geotechnical Personnel
☐ BBS Central Geotechnical Unit

Geotechnical Data, Subsurface Exploration and Testing

	Yes	No	N/A
All pertinent existing boring data, pile driving data, site inspection information included in the report?	☒	☐	☐
Are the preliminary substructure locations, foundation needs, and project scope discussions between Geotechnical Engineer and Structure Planner included in the report?	☒	☐	☐
All ground and surface water elevations shown on all soil borings and discussed in the report?	☒	☐	☐
Has all existing and new exploration and test data been presented on a subsurface data profile?	☒	☐	☐
Is the exploration and testing in accordance with the IDOT Geotechnical Manual policy?	☐	☒	☐
Are the number, locations, depths, sampling, testing, and subsurface data adequate for design?	☒	☐	☐

Geotechnical Evaluations

Have structure or embankment settlement amounts and times been discussed in report?	☒	☐	☐
Does the report provide recommendations/treatments to address settlement concerns?	☒	☐	☐
Has the critical factor of safety against slope instability been identified and discussed in the report?	☒	☐	☐
Does the report provide recommendations/treatments to address stability concerns?	☒	☐	☐
Is the seismic design data (PGA, amplification, category, etc.) noted in the report?	☒	☐	☐
Have the vertical and horizontal limits of any liquefiable layers been identified and discussed?	☐	☐	☒
Has seismic stability been discussed and have any slope deformation estimates been provided?	☐	☒	☐
Has the report discussed the proximity of ISGS mapped mines or known subsidence events?	☒	☐	☐
Has scour been discussed, any Hydraulics Report depths reported & soil type reductions made?	☒	☐	☐
Do the Factors of Safety meet AASHTO and IDOT policy requirements?	☒	☐	☐

Geotechnical Analyses and Design Recommendations

When spread footings are recommended, has a bearing capacity and footing elevation been provided for each substructure or footing region?	☒	☐	☐
Has footing sliding capacity been discussed?	☒	☐	☐
When piles are recommended, does the report include a table indicating estimated pile lengths vs. a range of feasible required bearings and design capacities for each pile type recommended?	☒	☐	☐
Have any downdrag, scour, and liquefaction reductions in pile capacity been addressed?	☒	☐	☐
Will piles have sufficient embedment to achieve fixity and lateral capacity?	☒	☐	☐
Have the diameters & elevations of any pile pre-coring been specified (when recommended)?	☐	☐	☒
Has the need for test piles been discussed and the locations specified (when recommended)?	☒	☐	☐
Has the need for metal shoes been discussed and specified (when recommended)?	☒	☐	☐
When drilled shafts are recommended, have side friction and/or end-bearing values been provided?	☒	☐	☐
Has the feasibility of using belled shafts been discussed when terminating above rock, or have estimated top of rock elevations been provided when extending into rock?	☐	☒	☐
Have shaft fixity, lateral capacity, and min. embedment been discussed?	☐	☒	☐
When retaining walls are required, has feasibility and relative costs for various wall types been discussed?	☐	☐	☒
Have lateral earth pressures and backfill drainage recommendations been discussed?	☒	☐	☐
Has ground modification been discussed as a way to use a less expensive foundation or address feasibility concerns?	☒	☐	☐
Have any deviations from IDOT Geotechnical Manual or Bridge Manual policy been recommended?	☐	☒	☐

Construction Considerations

Has the need for cofferdams, seal coat, or underwater structure excavation protection been discussed?	☐	☐	☒
Has stability of temporary construction slopes vs. the need for temporary walls been discussed?	☒	☐	☐
Has the feasibility of cantilevered sheeting vs. a temporary soil retention system been discussed?	☐	☐	☒
Has the feasibility of using a geotextile wall vs. a temp. MSE for any temp fill retention been noted?	☐	☐	☒

"In order to aid in determining the level of departmental review, please attach additional documentation or reference specific portions of the SGR to clarify any checklist responses that reflect deviation from IDOT policy/practice."

I-74 Ramp 6th-C Structure Geotechnical Report Responsibility Checklist Notes:

1. Soil classification based upon Jacobs Soil and Rock classification System per previous agreement with Iowa DOT and CH2M Hill.

2. Lateral capacities using GROUP 7.0 or Florida Multi Pier should be performed during final design once the pile/drilled shaft layouts are made and group reduction factors can be applied.