Structure Geotechnical Report

FAP631 (IL 102)
Section 110BR
Kankakee County
IL 102 over Rock Creek,
6.7 miles west of US 52
SN 046-0065(Existing)
SN 046-0149 (Proposed)
P93-048-04
Contract #66B55
PTB 133/10

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Date: August 16, 2011(Revised July 5, 2012)

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Attachments:
Aerial
Subsurface Profile
Boring Logs
Seismic Class Determination
Pictures
Current TSL

Contact the author if there are any questions regarding this report or if there are modifications to structure location, size, geometry or vertical alignment.

TLM/SGR for SN046-0065

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Project Description

The original structures, built in 1929, have a current sufficiency rating of 84.3, an inventory rating of 23.9 and an operating rating of 37.8. The existing three span, 99.9 ft long structure, has a bituminous wearing surface over a cast-in-place concrete deck resting on prestressed concrete box beams. The existing abutments and piers are founded on spread footings.

The project includes removing the existing three span structure (SN 046-0065) and replacing it with a two span structure (SN 046-0149). The new structure is to be 239 ft long. The new abutments will be placed just behind of the existing abutments and the new pier placed at the center of the structure. The structural engineer estimates the factored substructure loadings to be as high as 4500 kips at the pier and 2400 kips at the abutments. Pictures of the existing structure can be found as attached documents at the end of the report.

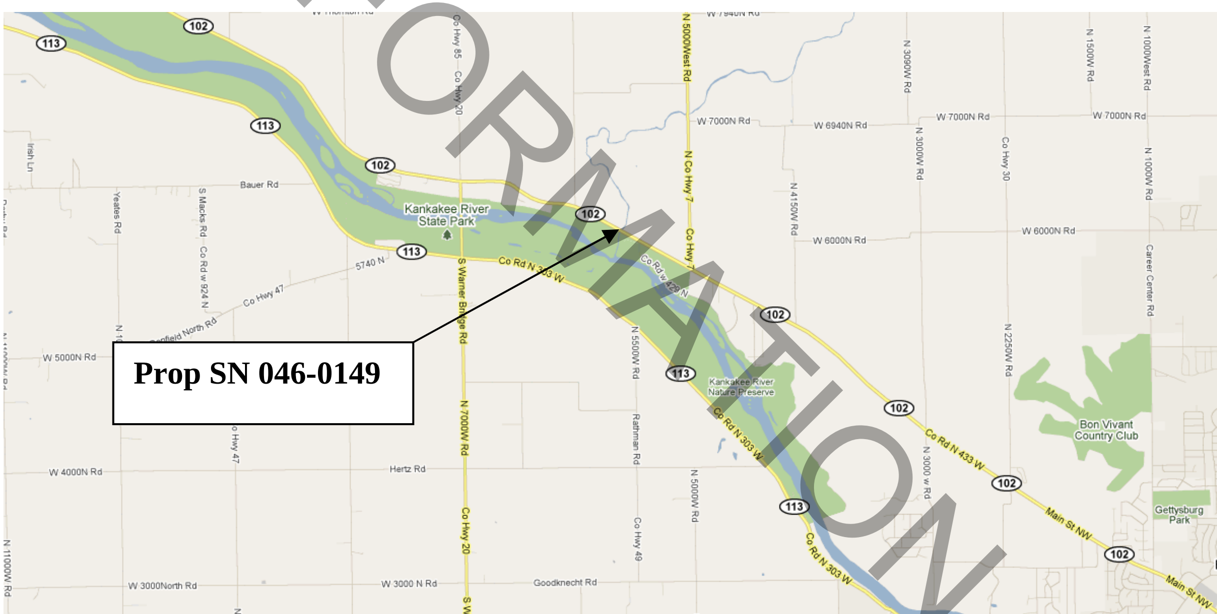


Figure 1: Map showing the location of the proposed bridge reconstruction

Subsurface Conditions

Geology

Most of the Northern Illinois subsoils were deposited during the glacial period known as "The Ice Age" or formally as the Pleistocene Epoch, ranging from approximately 2.5 million to 10,000 years ago. During this time, massive sheets of ice, hundreds of feet thick, called glaciers dominated the North American landscape. These massive ice sheets formed in Canada due to heavy snowfall. The snow compacted into ice and began flowing, some of which travelled through the Lake Michigan basin and as far south as Central Illinois and as far west as the Mississippi River.

The massive glaciers gouged and ground up any rock in their paths, erasing Northern Illinois of any existing topography. The ground up rock imbedded in the glaciers ranged in size from silt (rock flour) to boulders, with varying amounts of everything in between. This material, called till, was deposited at the margins of the ice in large curved landforms called moraines.

Once the glaciers began melting, the end moraines acted as dams holding back the water forming large glacial lakes. Eventually the moraines would fail, allowing the water to drain. These rushing waters would carry sediment and deposit it away from the original location of the ice. This deposit type called an outwash is unique from till because it is well sorted. The water will separate the particles of different sizes based on flow velocity. Typically, larger grain sizes are observed at the bottom of an outwash deposit, with decreasing grain size moving up in the stratigraphic column, indicating decreasing flow velocity as the glacial lakes drained.

Underlying the glacial deposits is bedrock. Bedrock is defined as the solid rock buried beneath any glacial or unlithofied material. Most of north-eastern Illinois bedrock is buried by glacial material, lessening to the west and south. The bedrock consists of limestone, dolomite, sandstone and shale. These were deposited by an ancient inland sea whose margins fluctuated across Illinois that existed approximately 510 million to 290 million years ago during the Cambrian Period. Underlying the bedrock is basement rock. This ancient rock consists mostly of the igneous rock granite.

This project site is located in an intermoranial area of the Wisconsin age glaciation. The upper most bedrock in this region is limestone, Silurian in age ranging from approximately 440 million to 415 million years ago. From previous cores, the bedrock is observed to be covered by approximately 3-5 ft of overlying sandy to silty clay loam.

Subsurface Profile

Three (3) borings were taken for this structure in February of 1980, but contained no rock cores. Borings 1 & 2 were advanced to accomplish the depth of one boring. A note, on boring 1 of the 1980 field logs, states that the auger was angling toward the creek due to the assumption that the crew had encountered a fault. The boring was moved and advanced to 6 ft. There was no mention as to the exact location of boring 2. Two borings were taken in August of 2010 and rock cores were taken April 6, 2011 and on July 13, 2011. Copies of the 2 most recent borings taken in August of 2010 and the subsequent rock cores taken in April and July of this year are included in this report. Please note the difference in datum from the 1980 borings and the more recent borings.

Boring #1&2 (West Abutment 447+00 40 ft & 48 ft Rt) with a ground surface elevation of 97.6 ft, encountered 2.0 ft of dark brown sandy loam. At 95.6 ft a 2.0 ft layer of hard brown silty clay loam was encountered followed by 2.0 ft of very stiff, dark brown silty clay

Boring #3 (East Abutment 444+52 60 ft Lt of CL) with a ground elevation of 97.5 ft encountered 2.0 ft of dark brown silty clay loam. At 95.5 ft, a 1.0 ft thick layer of hard, dark brown silty clay loam was encountered followed by the top of the limestone bedrock.

Two 5 ft core runs were retrieved on April 6, 2011. The rock retrieved is tan to buff fractured dolomite. The fractures are both vertical and horizontal in first core run. These fractures in the first run are in a loose with slightly eroded faces to a tight condition.

The second core run shows no vertical fractures after 1 ft into the core. However in the bottom 12 in of the core two 1 in wide joints were encountered. The first joint was filled with a medium to dense tan silt and the second joint was filled with very stiff gray clay. It is suspected the coring operation may have softened the joint material with the added water.

The recovery of the first run was 63% while the second run was 100%. The rock quality designations, RQD's, were 22 and 17 for the first and second core runs respectively. Two unconfined compressive strength tests were performed, one from each core run on selected intact pieces.

On July 13, 2011 a second rock core with 1 - 2 ft run and 5 - 5 ft runs was retrieved from the southwest side of the structure with similar results as found in the core on the northeast side of the structure. This core went much deeper and showed 100% recovery below 7 ft into the rock formation. The RQD also improved with depth.

As can be seen in the boring logs and attached profile, the overburden thickness is relatively shallow, less than 6 feet. The overburden material consists of sandy loam and hard silty clay loam over the dense limestone. No borings were advanced for the pier due to the lack of access to get the drilling equipment into the canyon. No water was encountered during the subsurface exploration.

For a more detailed description of the soils encountered, please look at the attached profile and boring logs.

Groundwater

Due to such shallow borings (approximately 10 ft depths) groundwater was not encountered during any of the three (3) borings taken in 1980 or the recent 2011 exploration. The water surface of Rock Creek at the time of drilling in the 1980's was 42.6 ft below the surface of the boring. The configuration of this canyon is near vertical limestone walls

Scour Potential

This structure does cross a waterway (Rock Creek); therefore, scour was investigated briefly. The streambed and much of the stream walls are rock.

Table 1: Scour Elevations

SN 046-0149 Stream Bed Elevation of 558.70 ft			
	East Abutment	Piers	West Abutment
Design Scour Elevation	595.00 ft	555.00 ft	593.50 ft

The scour depth in the river bottom was determined by using the results of a field investigation where a dynamic cone penetrometer was used to determine the point of refusal beneath the streambed. A scour analysis was not included in the Districts Hydraulic Report due to the streambed being rock. The pier foundation design does not include scour as a geotechnical loss. The elevations in the above table are the bottom of footing elevations for the abutments and pier.

Abandoned Coal Mines

There are no records that indicate any mining activity at this specific project location. The map below (Figure 2) was taken from the website:

www.isgs.uiuc.edu/maps-data-pub/coal-maps/pdf-files/2008-coalindustry.pdf .

A more detailed map is not available for this exact location, alluding to the lack of mining activity at the bridge location.

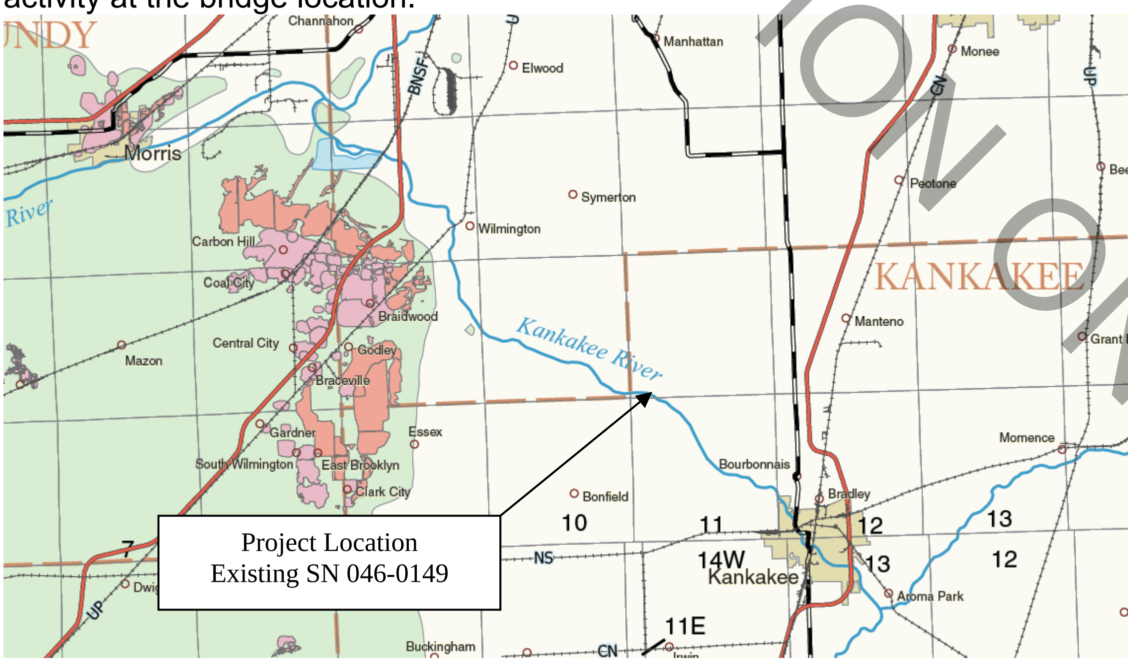


Figure 2: Map showing mining activity within the vicinity of the bridge

Geotechnical Evaluation

Slope Stability

Because the structure sets over a portion of the creek that has tall rock walls, traditional slope stability analyses were not performed. The thickness of the overburden material above the rock varies slightly from the east and west abutments. The top of weathered rock at the east abutment is 602.79 ft and 601.23 ft at the west abutment. These elevations are quite shallow considering the existing pavement elevation is 607.29 ft at the east abutment and 607.23 ft at the west abutment. The elevations indicate that the abutment cap will sit directly on rock. This and the strength of the rock make a spread footing abutment a good choice.

The vertical rock walls are made of horizontal bedding planes of varying thickness ranging from 3 inches to 12 inches. According to the rock cores taken April 6 and July 13, 2011, the material between the bedding planes appears to be silt and clay shale. The orientation of the joints and cracks are shown in the picture below.

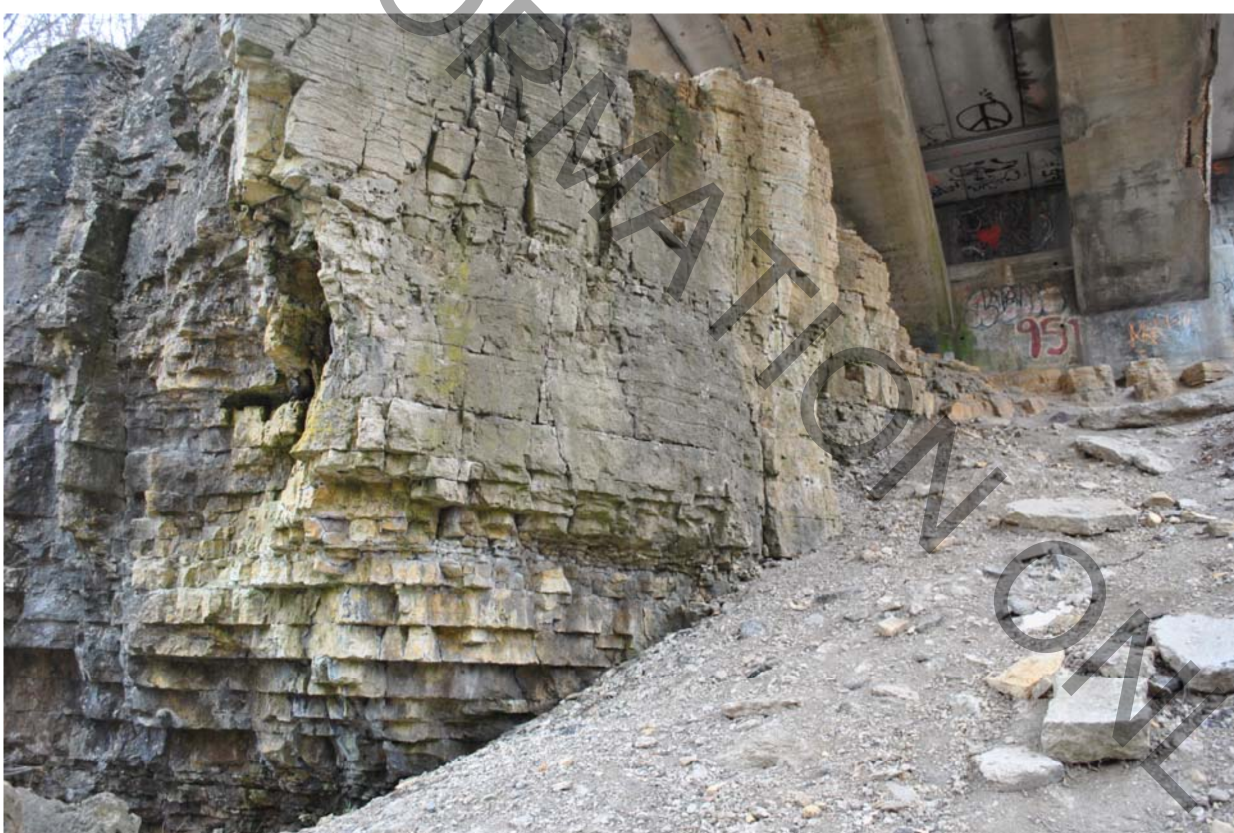


Figure 3: Picture of the rock formation in front of the east abutment of SN046-0065.

The picture above shows the bedding planes to have a horizontal physical expression in two directions. In this picture the rock face shown in the left ¼ of the picture is approximately 70° from the direction of the rock face toward the center of the picture. The vertical and near vertical jointing in the picture shows no relative movement and provides evidence of why this structure rock face has existed harmoniously for so many years.

With the orientation of the bedding planes and the joint sets as shown in the picture on the previous page, there is little concern for short term stability. Over time, many years, the rock has eroded near the stream surface. Care should be taken to properly drain the roadway storm water away from the face of the rock. Normal sheet flow does not appear to be greatly affecting the rock face. A concern that the roadway ditches are concentrating the flow of water for some distance may have been the reason for the retaining structure in the NW quadrant of the west abutment.

Figure 4: Picture of wooden retaining structure in the NW quadrant of the west abutment.

It is possible this wall is constructed in a crevice filled with sediment or the rock in this area had eroded. The appearance of stability would be improved if the new abutments were placed behind the existing abutments and the existing abutments staying in place up to a point where they would not interfere with the new structure.

Settlement

The new bridge is to be the same width as the existing and should not require any new embankment. The new profile grade line shows no appreciable increase in elevation from the existing profile grade. Because there is little if any increase in loading, high unconfined compressive strengths and relatively low moisture contents of the existing embankment soils there should be little additional consolidation of the existing overburden soils. The existing foundations are resting on a dense limestone formation with an estimated bearing capacity of 9.5 tsf, there is little concern for settlement at the abutments or the single pier in the middle of the structure.

For a load of 9.5 tsf; the author estimates the settlement to be less than 0.01 inches. The old repair plans from 1985 note the spread footings were designed for a maximum foundation pressure of 4.0 tsf.

Seismic Considerations

The probability of a seismic event, large enough to cause damage to the structure or embankment, is not high enough to warrant any undo concern. Therefore, seismic effects were not considered in the foundation design of this project. This bridge is considered to have an Importance Category of "Other Bridge". Please use the following seismic information for TSL and plan development.

Seismic Performance Zone (SPZ) = 1
Soil Site Class = C
Design Spectral Acceleration at 0.0 sec (As) = 0.057g
Design Spectral Acceleration at 0.2 sec (S_{DS}) = 0.126g
Design Spectral Acceleration at 1.0 sec (S_{D1}) = 0.072g

Because the Seismic Performance Zone equals 1 and because of the soils and rock encountered a liquefaction analysis was not performed. See the supporting documents portion of this report for the print out of the Department's Seismic Site Class Determination spreadsheet.

Foundation Recommendations

The proposed bridge is approximately 230 ft long from back to back of the abutments. Due to the high rock elevation only drilled shaft and spread footing foundation options were explored for this structure. The author presumes the use of a semi-integral or stub type abutment with these foundation types.

The use of a spread footing foundation is geotechnically possible at all substructure locations. The existing pier substructures are built on spread footings and have performed well, however this foundation type may require a coffer dam at the piers to construct.

The abutments of the original structure were constructed on spread footings due to the close proximity of bedrock. The length of the proposed structure is less than the 410 ft maximum for use with integral abutments for a concrete structure mentioned in the IDOT Bridge Manual. However, the proposed beam is a 72” bulb-T and the profile is to be raised only 2.25 inches above what is the current profile grade line, PGL. The depth below the current PGL is approximately 4.5 ft. This means the bottom of the abutment is going to be below the current top of weathered rock. The table below provides bearing resistance data for both the abutments and pier. Table 2, utilizes a resistance factor $\phi = 0.45$.

Table 2: Bearing Resistance for Spread Footing Option

Substructure Element	Nominal Bearing Resistance (KSF)	Footing Elevation (Ft)	Boring # (2011)
Pier	19.0	555.00	1a & 2a
East Abutment	19.0	595.00	1a & 2a
West Abutment	19.0	593.50	2a

The spread footing option precludes the use of integral abutments; however a semi-integral abutment may be used. The semi-integral abutment has the same, much appreciated, benefit as the integral abutment of no joint at the ends of a bridge. The loadings mentioned earlier in this document are relatively light therefore the widths of the footings may be quite reasonable, <10 ft wide. For the expected abutment loads of 2400 kips, the above bearing resistance and similar widths as the existing structure the widths may be reduced to as low as 4 ft. There is no eccentricity included in the bearing resistance calculations.

For the resistance to sliding calculations, use a friction angel, $\delta = 33^{\circ}$. This results in a coefficient of friction, $\tan \delta = 0.649$.

The pier foundations of the existing structures are spread footings and have performed well. The design scour depth at the piers of the proposed structures is relatively shallow at 1.77 ft. The pier footing elevations shown in Table 2 are approximately 2.3 ft below the design scour depth. As mentioned in the Settlement section of this report the spread footings should have minimal estimated elastic settlement at any of the substructure locations due to the relatively high strengths, joint spacing and limited gouge material.

The soil strengths and soil types would work well with drilled shafts. However, the location of the single pier in the middle of the stream may cause access problems for putting the large scale drilling equipment into position to excavate for the shafts. Tables 3, 4 and 5 show a sample of resistance values achievable with drilled shafts at this structure. Additional depths and diameters can be provided should the designer want to pursue this option. Like the spread footing, the drilled shaft option will require a semi-integral abutment to eliminate a joint at the ends of the structures.

Table 3: Drilled Shaft Allowable Resistance for East Abutment Foundation

Shaft Diameter (ft)	Approximate Tip Elevation (ft)	SN 046-0149		
		Factored Skin Friction (Kips)	Factored End Bearing (Kips)	Total Factored Resistance (Kips)
2.0	590	218	5.6	218
2.0	585	398	5.6	398
2.0	580	583	5.6	583
2.5	590	273	8.8	273
2.5	585	498	8.8	498
2.5	580	730	8.8	730
3.0	590	328	12.7	328
3.0	585	598	12.7	598
3.0	580	875	12.7	875
3.5	590	382	17.4	382
3.5	585	698	17.4	698
3.5	580	1021	17.4	1021
4.0	590	437	22.7	437
4.0	585	797	22.7	797
4.0	580	1167	22.7	1167
4.5	590	492	28.7	492
4.5	585	897	28.7	897
4.5	580	1313	28.7	1313

Table 4: Drilled Shaft Factored Resistance for West Abutment Foundation

Shaft Diameter (ft)	Approximate Tip Elevation (ft)	SN 046-0149		
		Factored Skin Friction (Kips)	Factored End Bearing (Kips)	Total Factored Resistance (Kips)
2.0	590	131	5.6	131
2.0	585	311	5.6	311
2.0	580	496	5.6	496
2.5	590	163	8.8	163
2.5	585	388	8.8	388
2.5	580	620	8.8	620
3.0	590	196	12.7	196
3.0	585	466	12.7	466
3.0	580	744	12.7	744
3.5	590	229	17.4	229
3.5	585	544	17.4	544
3.5	580	868	17.4	868
4.0	590	262	22.7	262
4.0	585	622	22.7	622
4.0	580	992	22.7	992
4.5	590	294	28.7	294
4.5	585	700	28.7	700
4.5	580	1116	28.7	1116

Table 5: Drilled Shaft Factored Resistance Table for Pier Foundation

Shaft Diameter (ft)	Approximate Tip Elevation (ft)	SN 046-0149		
		Factored Skin Friction (Kips)	Factored End Bearing (Kips)	Total Factored Resistance (Kips)
2.0	550	180	5.6	180
2.0	545	360	5.6	360
2.0	540	547	5.6	547
2.0	535	740	5.6	740
2.5	550	225	8.8	225
2.5	545	450	8.8	450
2.5	540	684	8.8	684
2.5	535	925	8.8	925
3.0	550	270	12.7	270
3.0	545	540	12.7	540
3.0	540	821	12.7	821
3.0	535	1110	12.7	1110
3.5	550	315	17.4	315
3.5	545	630	17.4	630
3.5	540	958	17.4	958
3.5	535	1295	17.4	1295
4.0	550	360	22.7	360
4.0	545	720	22.7	720
4.0	540	1095	22.7	1095
4.0	535	1480	22.7	1480
4.5	550	405	28.7	405
4.5	545	810	28.7	810
4.5	540	1232	28.7	1232
4.5	535	1665	28.7	1665

Driven piling is not a viable option for the piers or the abutments because of the close proximity of bedrock. Rock sockets were investigated but because of the closeness of the abutment elevation to the top of rock the allowable movement in the pile would not accommodate the expansion and contraction movement of an integral abutment which would be the benefit to a pile foundation. If an integral abutment cannot be used then drilled shaft or spread footing options are more desirable from a construction and performance stand point.

It is anticipated that the shafts if used will be entirely in the dolomite formation. This combined with the minimal lateral loads expected at this structure provided little reason to perform a lateral load analysis on the drilled shafts.

This structure is currently being designed as a reinforced concrete deck with 72 inch PPC Bulb T-beams on stub abutments. Because we are not expecting any settlement, the shafts were not designed to accommodate negative skin friction. Depending on the size of the shafts and how close the shafts are to each other will determine if the effects of group interaction should be taken into account. The lateral resistance of a single shaft may need to be reduced by P-multipliers provided in the AASHTO LRFD Bridge Design Specifications when determining the lateral resistance of a pile group. Please see Table 6, on page 14 for soil parameters used in the lateral load analysis.

The soil parameters used in the lateral load analysis are based on the rock cores taken in April and July of 2011.

Table 6: Soil Parameters at North Pier and Abutment, SN 046-0149

Soil Parameters For Static Lateral Load Analysis			
Soil Type From Borings 1a & 2a	Unconfined Compressive Strength (psi)	Effective Unit Wt. (pci)	Moist Unit Wt. (pci)
Jointed & Weathered Dolostone	6000.0	0.083	0.089

Construction Considerations

At this time, it is assumed this structure will be built under closed road conditions. Because driven pile are not practical for this structure there will be no need for test piles.

Load testing on the shafts may be performed however the cost of this testing for the benefit in size reduction may not warrant such an expense. A traditional cofferdam may not be necessary to keep the flow of Rock Creek out of the center pier excavation if the channel can be diverted to the east and west around the center pier excavation. To affectively divert the channel flow a temporary dike could be constructed using proprietary systems such as Aqua-Barrier. For pictures of this type of system go to www.hydrologicalsolutions.com. Due to the jointed condition of the streambed rock a seal coat is recommended.

Based on the EWSE of 560.00 ft. and ABD 11.2, a type I cofferdam is recommended. A seal coat should be expected for this application because of the fractures in the rock formation.

H-pile socketed into the rock was not fully examined because of the close proximity of the rock surface to the abutment structures could greatly reduce the desired flexibility of the H-pile themselves. Should this foundation type be desired the bearing capacities of the drilled shafts shown in Tables 3 through 5 may be used for the resistance in the rock.

Care should be taken with the excavations for the spread footings to ensure the desired bearing resistance is available. All loose material shall be removed from the bottom and sides of the excavation. The excavation of the fracture rock is possible with mechanical equipment. The elevation of the bottom of the excavation may vary slightly from the design elevations because the fracture pieces of rock will come loose at the fracture and not in the middle of an intact piece.

As mentioned above, the rock is quite shallow. The soils encountered are very stiff based on the Qu values. Depending on where the stage line hits there may be room to simply slope the soil back at a 1(H):0.5(V) from the rock surface up to the pavement. If there is not adequate space for this a temporary soil retention system may be required. Keep in mind all excavations should meet all applicable OSHA and IDOL standards.

Supporting Documents

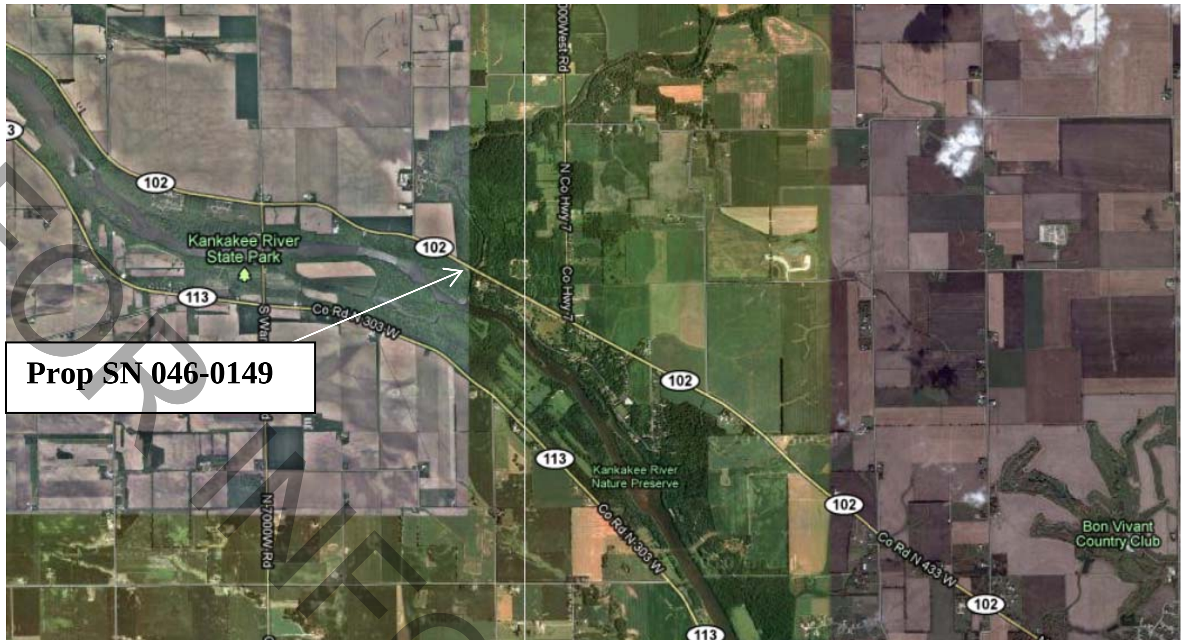


Figure 3: Aerial photo of structure location

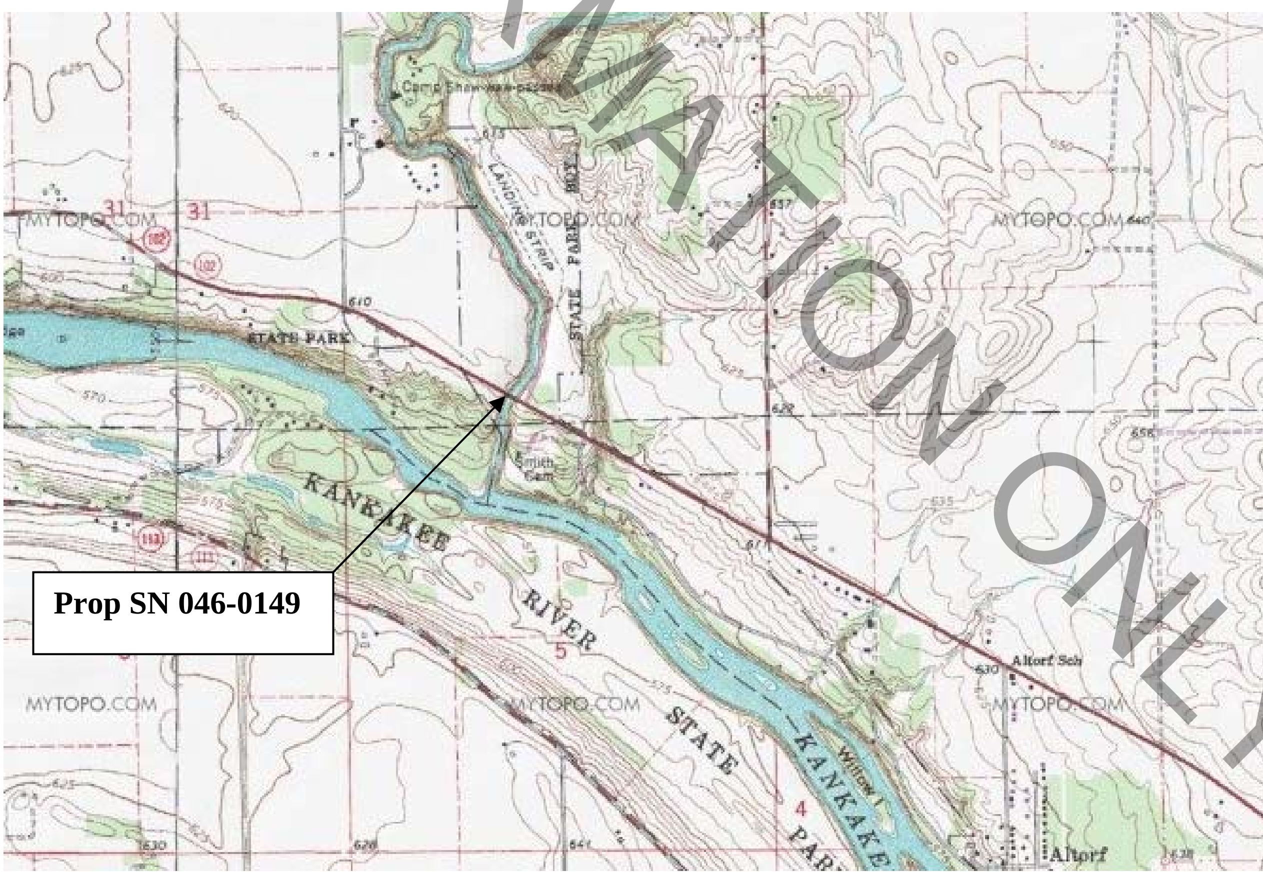
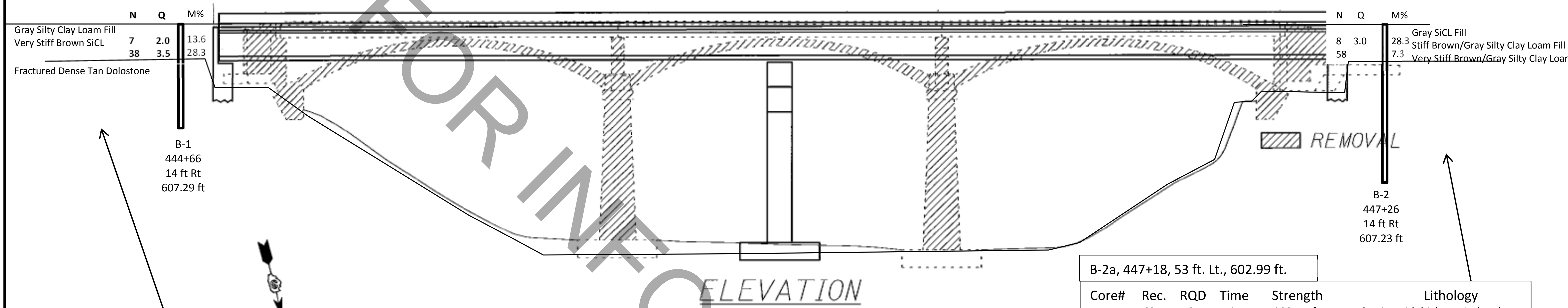


Figure 4: Image of topography in the area of SN 046-0065.



B-1a, 444+66, 13 ft. Rt., 607.29 ft.

Core#	Rec.	RQD	Time	Strength	Lithology
1	63	22	5.2 min.	650.2 tsf	Tan Dolomite – numerous vertical fractures. 50% tight and 50% open with slightly eroded surfaces. Some horizontal fractures at bedding planes.
2	73	7	11.2 min.	1167.8 tsf	Tan Dolomite – some vertical fractures in top 1 ft with no vertical fractures below 10.5 ft., numerous horizontal fractures with eroded faces – 1 inch layer of tan silt @ 14 ft. with grey clay fracture fill at 14.5 ft.

B-2a, 447+18, 53 ft. Lt., 602.99 ft.

Core#	Rec.	RQD	Time	Strength	Lithology
1	63	53	5 min.	1388.4 tsf	Tan Dolomite with high vertical and horizontal fracturing. Most seams filled with Shale.
2	73	7	11.2 min.		Same as above
3	100	58	5 min	441.2 tsf	Tan to White Dolomite – Vuggy with numerous horizontal fractures and minor vertical fracturing.
4	100	50	5.6 min	778.2 tsf	Same as above
5	100	73	7.2 min	467.1 tsf	Same as above

McCleary Engineering

Profile
Proposed SN 046-0149
Kankakee County
FAP Rte 631 (IL 102)
Over Rock Creek

Illinois Department of Transportation
Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Date 8/11/10

ROUTE IL 102 (FAP 631) **DESCRIPTION** IL 102 over Rock Creek, 6.5 miles Northwest of Bourbonnais **LOGGED BY** Larry Myers

SECTION (110)BR **LOCATION** SW 1/4, **SEC.** 32, **TWP.** 32N, **RNG.** 11E

COUNTY Kankakee **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** CME Automatic

STRUCT. NO. 046-0065 (Exist.)
Station 446+00

BORING NO. 1 (N.E. Quad.)
Station 444+66
Offset 14.00ft Rt.
Ground Surface Elev. 607.29 ft

Surface Water Elev. 562.14 ft
Stream Bed Elev. 561.64 ft

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

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Augered Bituminous Shoulder, Gray Silty Clay Loam Fill				
604.79				
Very Stiff Reddish Brown Silty Clay Loam/Clay Loam		3		
		4	3.0	28.3
		4	P	
602.79				
Dense Tan Dolomite with Sandy Loam Matrix Crack/Crevice Fill Weathered/Fractured/Reworked Rock Surface	-5	13		
		26		7.3
		32		
600.79				
End of Boring				
	-10			
	-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)

BBS, from 137 (Rev. 8-99)

Illinois Department of Transportation
Division of Highways
Illinois Department of Transportation

SOIL BORING LOG

Date 8/11/10

ROUTE IL 102 (FAP 631) **DESCRIPTION** IL 102 over Rock Creek, 6.5 miles Northwest of Bourbonnais **LOGGED BY** Larry Myers

SECTION (110)BR **LOCATION** SW 1/4, **SEC.** 32, **TWP.** 32N, **RNG.** 11E

COUNTY Kankakee **DRILLING METHOD** Hollow Stem Auger **HAMMER TYPE** CME Automatic

STRUCT. NO. 046-0065 (Exist.)
Station 446+00

BORING NO. 2 (N.W. Quad.)
Station 447+26
Offset 14.00ft Rt.
Ground Surface Elev. 607.23 **ft**

Surface Water Elev. 562.14 **ft**
Stream Bed Elev. 561.64 **ft**

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

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Augered Bituminous Shoulder, Brown Fill Sand, Gray Silty Clay Loam Fill	604.73				
Stiff to Very Stiff Brown & Gray Silty Clay Loam Fill			5		
			4	2.0	13.6
	602.73		3	P	
Very Stiff Brown/Gray Silty Clay Loam/Clay Loam		-5			
			4		
	601.23		6	3.5	28.3
Dense Tan Dolomite with Sandy Loam Matrix Crack/Crevice Weathered/Fractured/Reworked Rock Surface			32	P	
	599.65		57		
End of Boring			100/1"		6.6
		-10			
		-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
ILLINOIS DOT

ROCK CORE LOG

Date 4/6/11

ROUTE IL 102 (FAP 631) **DESCRIPTION** IL 102 over Rock Creek, 6.5 miles Northwest of Bourbonnais **LOGGED BY** Larry Myers

SECTION (110)BR **LOCATION** SW 1/4, **SEC.** 32, **TWP.** 32N, **RNG.** 11E

COUNTY Kankakee **CORING METHOD** Split Barrel Wire Line

STRUCT. NO. 046-0065 (Exist.) **CORING BARREL TYPE & SIZE** N W/L 2
Station 446+00

Core Diameter 1.9 in
Top of Rock Elev. 602.79 ft
Begin Core Elev. 602.79 ft

BORING NO. 1a (N.E. Quad.)
Station 444+66
Offset 13.00ft Rt.
Ground Surface Elev. 607.29 ft

Description	Elev.	DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
Dolomite, Tan to Buff - Numerous Vertical Fractures. 50% tight, 50% loose with slightly eroded faces. Some Horizontal Fractures at Bedding Planes	597.79	-5	1	63	22	5.2	650.2
Tan to Buff Dolomite - some Vertical Fractures in top 1' (9.5' - 10.5') No Vertical Fractures after 10.5' - Numerous Horizontal Fractures with eroded faces - 1" Layers of Tan Silt @ 14' with Grey Clay Fracture Fill at 14.5'	592.79	-10	2	100	17	8	1167.8
Locked Rock Core Barrel in Stone - End of Boring		-15					
		-20					

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

Illinois Department of Transportation
Division of Highways
ILLINOIS DOT

ROCK CORE LOG

Date 7/13/11

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SECTION (110)BR **LOCATION** SW 1/4, **SEC.** 32, **TWP.** 32N, **RNG.** 11E

COUNTY Kankakee **CORING METHOD** Solid Barrel Wire Line

STRUCT. NO. 046-0065 (Exist.) **CORING BARREL TYPE & SIZE** N W/L 2
Station 446+00

BORING NO. 2a (S.W. Quad.)
Station 447+18
Offset 53.00ft Lt.
Ground Surface Elev. 602.99 ft

Core Diameter 1.9 in
Top of Rock Elev. 598.99 ft
Begin Core Elev. 598.99 ft

Description	DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
Tan Dolomite - High Vertical & Horizontal Fracturing most Seams filled with Weathered Gray Shale	-5	1	63	53	5	1388.4
6" of Weathered & Reworked Gray Shale Layer at 8.5'	-10	2	73	7	11.2	
591.99						
Tan to Off-White Dolomite - Vuggy with Numerous Horizontal Fractures with Weathered Fracture Surfaces - Minor Vertical Fracturing. Horizontal Fracturing Decreases with Depth	-15	3	100	58	5	441.2
	-20	4	100	50	5.6	778.5
		5	100	73	7.2	467.1

Color pictures of the cores Yes
Cores will be stored for examination until Construction Complete
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SECTION (110)BR **LOCATION** SW 1/4, **SEC.** 32, **TWP.** 32N, **RNG.** 11E

COUNTY Kankakee **CORING METHOD** Solid Barrel Wire Line

STRUCT. NO. 046-0065 (Exist.)
Station 446+00

CORING BARREL TYPE & SIZE N W/L 2

Core Diameter 1.9 in
Top of Rock Elev. 598.99 ft
Begin Core Elev. 598.99 ft

BORING NO. 2a (S.W. Quad.)
Station 447+18
Offset 53.00ft Lt.
Ground Surface Elev. 602.99 ft

Description	Elev.	DEPTH (ft)	CORE (#)	RECOVERY (%)	R.Q.D. (%)	CORE TIME (min/ft)	STRENGTH (tsf)
Tan to Off-White Dolomite - Vuggy with Numerous Horizontal Fractures with Weathered Fracture Surfaces - Minor Vertical Fracturing. Horizontal Fracturing Decreases with Depth *(continued)*		-25					389.3 752.6
			6	100	83	5	480.1 298.4
	571.99	-30					
End of Boring		-35 -40					

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

SEISMIC SITE CLASS DETERMINATION

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

Modified on 12/10/10

PROJECT TITLE=====Exist SN 046-0065 , Proposed SN 046-0149

Substructure 1

Base of Substruct. Elev. (or ground surf for bents)	605	ft.
Pile or Shaft Dia.		inches
Boring Number	B-1	
Top of Boring Elev.	605	ft.
Approximate Fixity Elev.	605	ft.

Individual Site Class Definition:

N (bar):	66	(Blows/ft.)	Soil Site Class C
N_{ch} (bar):	74	(Blows/ft.)	Soil Site Class C <----Controls
s_u (bar):	4.07	(ksf)	Soil Site Class C

Seismic Soil Column Depth (ft)	Bot. Of Sample Elevation	Sample Thick. (ft.)	N	Qu (tsf)	Layer Description Boundary
2.0	603.0	2.00	10	4.13	
4.0	601.0	2.00	25	10.30	B
6.0	599.0	2.00	48	3.10	
100.0	505.0	94.00	150	55.00	R

Substructure 2

Base of Substruct. Elev. (or ground surf for bents)	558	ft.
Pile or Shaft Dia.		inches
Boring Number	B-1	
Top of Boring Elev.	605	ft.
Approximate Fixity Elev.	558	ft.

Individual Site Class Definition:

N (bar):	100	(Blows/ft.)	Soil Site Class C <----Controls
N_{ch} (bar):	100	(Blows/ft.)	Soil Site Class C
s_u (bar):	5	(ksf)	Soil Site Class C

Seismic Soil Column Depth (ft)	Bot. Of Sample Elevation	Sample Thick. (ft.)	N	Qu (tsf)	Layer Description Boundary
	603.0	2.00	10	4.13	
	601.0	2.00	25	10.30	B
	599.0	2.00	48	3.10	
100.0	458.0	141.00	150	55.00	R

Substructure 3

Base of Substruct. Elev. (or ground surf for bents)	605	ft.
Pile or Shaft Dia.		inches
Boring Number	B-2	
Top of Boring Elev.	605	ft.
Approximate Fixity Elev.	605	ft.

Individual Site Class Definition:

N (bar):	96	(Blows/ft.)	Soil Site Class C
N_{ch} (bar):	NA	(Blows/ft.)	NA
s_u (bar):	4.97	(ksf)	Soil Site Class C <----Controls

Seismic Soil Column Depth (ft)	Bot. Of Sample Elevation	Sample Thick. (ft.)	N	Qu (tsf)	Layer Description Boundary
3.0	602.0	3.00	45	4.13	B
100.0	505.0	97.00	150	55.00	R

Substructure 4

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

Individual Site Class Definition:

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

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

Global Site Class Definition: Substructures 1 through 3

N (bar):	87	(Blows/ft.)	Soil Site Class C
N_{ch} (bar):	91	(Blows/ft.)	Soil Site Class C <----Controls
s_u (bar):	4.68	(ksf)	Soil Site Class C

Pictures

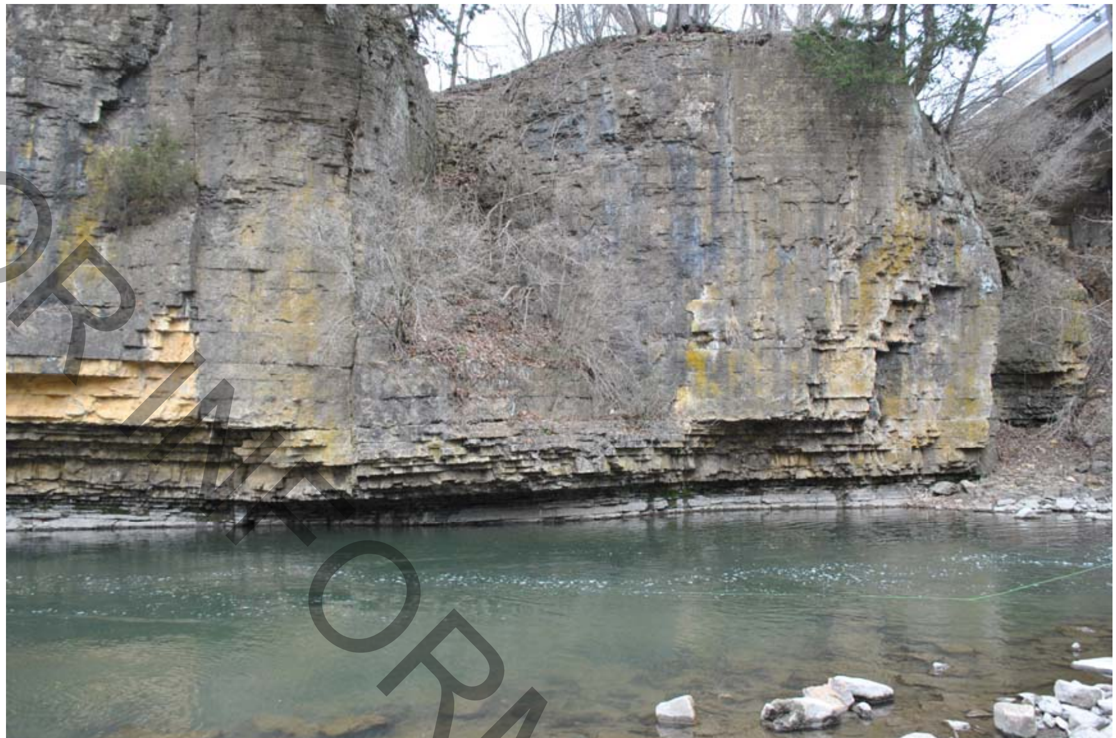

Figure 4: Image of layers and jointing of the rock formation along the west side of Rock Creek.

Figure 5: Image of Rock Creek downstream from atop the IL 102 Bridge SN 046-0065.

Figure 6: Image of creek bottom at IL 102 Bridge SN 046-0065

Figure 7: Image of existing structure SN 046-0065 from foot bridge downstream.

Figure 8: Existing east abutment.

Figure 9: Failing wood retaining wall immediately north of west abutment.