STRUCTURE GEOTECHNICAL REPORT

IL-178 OVER ILLINOIS RIVER
STRUCTURE No. SN 050-0256
STATION 29+19.53
LASALLE COUNTY, ILLINOIS

Prepared For:
Illinois Department of Transportation, District 3

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1. INTRODUCTION

This report presents the results of the geotechnical evaluations and recommendations for the proposed reconstruction of the bridge carrying IL-178 over the Illinois River, North Utica, Illinois. A Site Location Map of the project area is presented in Exhibit 1.

The existing bridge (SN 05-0088) was originally constructed in 1962 as a seven span bridge that measure 1158’-0” back-to-back of abutments. The abutments are founded on 10-in H-piles driven to refusal in bedrock and the piers are founded on shallow foundations generally seated more than 1-ft into bedrock. The approach embankments were constructed with 2H:1V side slopes.

According to the Bridge Condition Report (BCR), dated November 2011, rehabilitation or replacement of the bridge is necessary to improve the deteriorating condition of the structural components and to provide an extended service life. The replacement bridge will be constructed directly to the east of the existing bridge and will consist of three spans crossing the Illinois River that measure 1158'-0" back-to-back from the abutments, and a superstructure with a wider deck cross section of 49'-10" to accommodate roadway traffic and a shared-use path. The vertical alignment would have to be raised approximately 5'-0" to maintain the existing vertical clearance. The proposed roadway horizontal alignment shift is 46 ft to the east, but within the existing right-of-way. The two proposed bridge piers are assumed to have a width of 10 ft, which corresponds to the pier width of the middle four piers of the existing bridge.

2. SUBSURFACE INVESTIGATION PROGRAM

2.1 Soil Boring Program

The geotechnical soil boring program consisted of drilling and testing of a total of ten (10) borings. Two borings, B-1 and B-2, were performed near the existing abutment locations by the Illinois Department of Transportation (IDOT) District 3 on April 2, 2012. The borings were advanced with using hollow stem augers and ranged in depth from 50 to 51-ft below the ground surface. Boring logs were provided by IDOT to Parsons Brinckerhoff for use in the geotechnical evaluations and foundation recommendations. It should be noted that neither Parsons Brinckerhoff, nor any person acting on their behalf, makes any warrant, express or implied, with the accuracy of any information included in the boring logs prepared by IDOT District 3.

The remainder of the borings, B-3 through B-10, was performed by Wang Engineering, Inc. of Lombard, IL under subcontract to Parsons Brinckerhoff. The north approach/abutment borings, B-3 through B-5, were performed on October 24, 2012; the south approach/abutment borings, B-8 through B-10, on October 29, 2012, and the river pier borings, B-6 and B-7, on December 3 and 4, 2012. Depths varied from 3.5 to 44 feet below the ground or mudline surface.

Land borings, Borings B-3 through B-5 and B-8 through B-10, were performed using a D-50 type truck-mounted drill rig utilizing 3-1/4 in. hollow stem auger to advance the borings and to provide wall stability. Marine borings, B-6 and B-7, were performed using a B-57 truck-mounted rig and advanced using rotary wash methods with 4-in casing and a 3.25-in roller bit.

Standard penetration testing (SPT) was performed in general accordance with the requirements of ASTM D-1586 using a 2-inch O.D. split spoon sampler driven by a 140-pound hammer falling 30 inches. In all borings, 18-inch SPT split spoon samples were obtained from 1-ft below the ground surface or at mudline and at a maximum of 2.5-foot intervals. For the marine borings, continuous SPT split spoon sampling was performed until refusal (defined as more than 50 blows per 6 inches) was encountered.

At borings B-5 through B-8, rock coring was performed to depths ranging from 11 to 40-ft below the top of rock. Mud or other drilling polymers were not added to the drilling water during coring. The retrieved cores were logged in the field, placed into standard, wooden core boxes, and photographed.

The subsurface investigation is summarized in Table 1. The as-drilled boring locations (stations and offsets) and elevations are shown on Exhibit 2 and on the attached boring logs (Appendix A). The elevations shown on the boring logs and referenced in this report are measured in respect to the project's elevation datum NAVD88.

Table 1 Subsurface Investigation Summary

Boring ID	Station	Offset (feet)	Ground Elevation (feet)	Boring Depth (feet)
B-1	35+37.1	41.8 Rt	499.0	51
B-2	23+03.1	41.8 Rt	498.3	50
B-3	20+05.8	65.3 Lt	457.2	3.5
B-4	22+02.5	70.7 Lt	456.4	4.5
B-5	23+54.8	74.4 Lt	556.0	18
B-6	27+27.9	4.3 Lt	432.9	42
B-7	31+49.0	35.3 Lt	436.9	44
B-8	35+02.4	87.9 Lt	452.4	15.5
B-9	36+98.5	134.1 Lt	449.4	7.5
B-10	39+03.2	133.7 Lt	448.9	10

2.2 Laboratory Testing Program

A series of laboratory tests, aimed at defining the geotechnical properties to be used for final design, were planned by Parsons Brinckerhoff and subsequently performed by Wang Engineering, Inc. The testing program included natural water content, unconfined compression (UC) tests on rock core samples, unit weight, porosity and specific gravity on rock core samples, Brazilian splitting tensile strength, and the point load test. Results from the testing are presented within Appendix B.

Due to the very limited depth of overburden material encountered at the project location, the majority of the laboratory testing was focused on defining the engineering properties of the bedrock encountered. Table 2 shows the number of laboratory tests performed and the applicable ASTM standards for the project.

Table 2 Summary of Laboratory Testing

Test	Number of Tests Performed	Applicable ASTM Standard
Natural Water Content	11	D2216
Unconfined Compression (Rock)	9	D7012
Unit Weight, Porosity, and Specific Gravity for Rock	12	ISRM
Splitting Tensile Strength – Brazilian	4	D3967-05
Point Load Test	5	D5731

3. GENERALIZED SUBSURFACE CONDITIONS

3.1 General

This section summarizes the general subsurface conditions for the project site, as determined from the geotechnical data obtained during the recent subsurface investigation program as well as selected existing geotechnical data. General brief descriptions of the strata and their respective design parameters are presented. A generalized subsurface profile from soil borings, shown in Exhibit 3 and 3A, was developed along the alignment of the bridge structure. The Exhibit 3 section was developed from the results of the borings drilled by Wang Engineering, and Exhibit 3A was developed from the borings conducted by IDOT. Due to scaling issues they could not be placed on the same figure.

Subsurface conditions regarding seismic conditions, liquefaction potential, and the presence of abandoned mines and utilities are also as discussed in the sections below.

3.2 Soil Profile

Subsurface conditions at the site consist generally of four soil strata overlying bedrock. The general descriptions are discussed below in descending order as they were encountered.

3.2.1 Existing Fill

The existing approach embankment ramps consist of over 50-ft of fill material. Generally, the fill on the south abutment, as observed in B-1, consists of granular materials although some layers of silty clay loam material was present in the upper 7.5 feet. Boring B-2, performed on the north abutment, encountered more fine grained material including silty loam and silty clay loam, although granular soils were also observed.

Although not indicated on the borings provided by IDOT, based on the record drawings for the original bridge structure, the existing subgrade material in the footprint of the embankments was excavated and removed to bedrock. Therefore, the fill materials extend the full depth to the top of bedrock.

SPT N-values in the fill materials generally increased with depth and ranged from 5 to 61 blows-per-foot, with an average of approximately 20. The average SPT N-value indicates a generally dense or very stiff consistency.

3.2.2 Topsoil

Surficial topsoil was encountered in Borings B-3 through B-5 and B-8 through B-10 and ranged in thickness from 8-in to 1.5-ft and, in some instances, extended to the top of bedrock (B-3, B-4, and B-8). During construction, topsoil will be stripped and stockpiled and therefore is not considered important for engineering consideration.

3.2.3 Loam Materials

Loam materials were encountered beneath the topsoil in Borings B-5, B-9 and B-10 and ranged in thickness from 1.5 to 5-ft in thickness and in all cases extended to bedrock. The materials were classified as silty loam, silty clay loam, and clay loam. In boring B-5, the materials were identified as “trench fill”. SPT N-values in the loam materials ranged from 6 to 9 blows-per-foot, with an average of approximately 8 indicating a generally medium stiff to stiff consistency.

3.2.4 Sand Materials

Sand materials were encountered at the mudline in Boring B-7 and extended to a depth of 2.6-ft to the top of bedrock. The material was classified as fine sand and had trace amount of gravel material. These sandy materials are subject to scour and are therefore not considered for engineering consideration.

3.2.5 Bedrock

Bedrock was encountered at relatively consistent depths across along the profile of the bridge alignment. Within the river, the borings encountered bedrock beginning at the mudline and at a depth of 2.6-ft below the river bottom. Onshore on the south side of the bridge, bedrock was located at a very shallow depth below the ground surface ranging from 8-in to just over 2-ft. At the north approach, bedrock was encountered at 1.5-ft below the ground surface at the abutment and greater than 6-ft in depth along the length of the approach embankment.

The bedrock in this area of Illinois is sedimentary in origin. Dolomitic rock was the most predominant rock type encountered at the project location although a small layer of sandstone was encountered at the abutments in Boring B-5 and B-8 and ranged in thickness from 1.5 to 3.5-ft. The dolomite was generally described as moderately strong to strong, poor to very good rock quality, slightly weathered joints to fresh, and was occasionally vuggy and cherty. The sandstone was generally described as moderately strong to weak, very poor to poor rock quality, and weathered to fresh.

Generally, the upper portions of the bedrock encountered were weathered and was able to be penetrated with the auger of the drill rig. Weathered bedrock ranged in thickness from 1.5 to 8 feet.

Recovery of all rock core samples, ranged between 58 percent and 100 percent, with an average of 91 percent. Rock quality designation (RQD) of the core samples ranged between 0 and 93 percent, with an average of 56 percent.

3.2.6 Groundwater

Groundwater was reported during drilling only within Borings B-8 and B-9 and ranged from 5 to 7.5 feet below the ground surface. Groundwater was not encountered within any of the other land borings. It is expected that the groundwater levels will vary from the observed values in the future on a seasonal basis, depending upon the precipitation, runoff, infiltration, land use, and the Illinois River levels.

3.3 Soil Properties

The following is a discussion of the geotechnical properties of the soil and rock strata recommended for use in design. Table 3 presents the geotechnical parameters considered for each stratum. The parameters were estimated from both the field investigation data and laboratory testing on rock cores, SPT results, and from published correlations and geotechnical analysis.

Table 3 Summary of Geotechnical Properties

Stratum	Geotechnical Parameters
Existing Fill – Cohesionless	Total Unit Weight, Υ_t = 110 pcf Angle of Internal Friction, φ' = 30 degrees
Existing Fill – Cohesive	Total Unit Weight, Υ_t = 110 pcf Angle of Internal Friction, φ' = 22 degrees Undrained Shear Strength, $s_{u,}$ = 750 psf
Loam Material	Total Unit Weight, Υ_t = 115 pcf Undrained Shear Strength, $s_{u,}$ = 750 psf

Bedrock	Total Unit Weight, Υ_t = 145 pcf Uniaxial Compressive Strength = 13,900 psi Angle of Internal Friction, φ' = 40 degrees

3.3.1 Existing Fill

The fill materials primarily consist of a wide range of materials including sand, silty loam and silty clay loam. The stratum is generally dense or very stiff in consistency based on SPT N values as discussed in Section 3.2.1.

The boring data indicates that the south approach embankment was primarily granular materials, i.e., cohesionless, and the north embankment primarily slightly plastic silt loam and silty clay loam materials. Based on this observation, properties for both cohesive and cohesionless fill materials are provided. Geotechnical design parameters developed for the cohesionless fill materials are based on SPT N values and empirical correlations. For the cohesive materials, correlations on SPT N values and Rimac tests were used to provide the geotechnical parameters.

3.3.2 Loam Material

The loam materials encountered were classified as silty loam, silty clay loam, and clay loam and SPT N-values in the loam materials indicated a medium stiff to stiff consistency. Pocket penetrometer testing on the loam materials ranged from 0.5 to 1.5 tsf, with an average of 1 tsf. Rimac testing was not performed on the loam materials.

3.3.3 Bedrock

Bedrock was encountered at relatively shallow depths across the profile of the bridge alignment. Within the river, the borings encountered bedrock beginning at the mudline and at a depth of 2.6-ft below the river bottom. Onshore on the north side of the bridge, bedrock was located at a very shallow depth below the ground surface ranging from 8-in to just over 2-ft. At the south approach, bedrock was encountered at 1.5-ft below the ground surface at the abutment and greater than 6-ft in depth along the length of the approach embankment.

Recovery of all rock core samples, ranged between 58 percent and 100 percent, with an average of 91 percent. Rock quality designation (RQD) of the core samples ranged between 0 and 93 percent, with an average of 56 percent. A total of 9 unconfined compressive strength tests were performed on the selected rock core samples with compressive strengths ranging from 5,727 to 22,057 psi. The mean strength of 13,902 psi is indicative of strong rock. Five point load tests were performed on select core samples. Correlations can be made to the compressive strength of the rock based on the point load test and estimated compressive strengths ranged from 1,900 to 19,300 psi with an average of 10,182 psi. Also, four Splitting Tensile Strength – Brazilian Tests were performed on select core samples and ranged from 365 to 2,410 psi with an average of 1,028 psi. Lab testing results of the rock cores are presented in Appendix B.

The properties of the rock mass were determined using the procedures for classifying the rock using the Rock Mass Rating system (RMR) as described in Section 10.4.6.4 of AASHTO LRFD Bridge Design Specifications (2010). The system uses a relative rating for five parameters of the rock mass, including strength, RQD, joint spacing, joint condition, and groundwater conditions, and the RMR is determined as the sum of all five relative ratings. The RMR is adjusted based on the joint orientation with respect to the type of structure. Based on these procedures, the RMR for the rock mass from the results of the core samples ranged from 12 to 55 which is indicative of very poor to fair rock.

3.4 Seismic Considerations

The design earthquake motions and forces specified by IDOT Bridge Manual (2012) and AASHTO LRFD Bridge Design Specifications (2010) are based on a low probability of their being exceeded during the normal life expectancy of the structure. Structures designed in accordance with AASHTO guidelines may suffer damage during the design earthquake, but should have low probability of catastrophic failure. For the purposes of this study, the bridge has been classified as “essential”. Essential structures are generally those that should, as a minimum, be open to emergency vehicles and for security/defense purposes immediately after the design earthquake.

According to the guidelines provided by the IDOT Bridge Manual (2012) and AASHTO LRFD Bridge Design Specifications (2010), the following seismic site-specific parameters can be used for TSL and plan development:

- Soil Site Class C
- Design Spectral Acceleration at 0 sec, A_s = 0.06
- Design Spectral Acceleration at 0.2 sec, S_{DS} = 0.12
- Design Spectral Acceleration at 1 sec, S_{D1} = 0.07
- Seismic Zone 1

The policies and details within the IDOT Bridge Manual (2012) meet the minimum AASHTO requirements for Seismic Performance Zone 1 (LRFD) with a low probability of being exceeded during the normal life expectancy of the bridge. Bridges and their components that are designed to resist Zone 1 forces and constructed in accordance with the design details contained in the IDOT Bridge Manual (2012) should not experience total collapse, but may sustain repairable damage due to seismically induced ground shaking.

3.5 Liquefaction Potential

Liquefaction is a soil behavior phenomenon in which a soil located below the groundwater surface loses a substantial amount of strength due to strong earthquake ground shaking. Some types of soil tend to compact during earthquake shaking, inducing excess pore water pressure in the saturated soil, which, in turn, causes a reduction in strength of the soil. Recently deposited (i.e., geologically young) and relatively loose natural soils, as well as uncompacted or poorly compacted fills, are potentially susceptible to liquefaction. Loose sands are particularly susceptible. Loose silts and gravel also have potential for liquefaction. Dense natural soils and well-compacted fills have low susceptibility to liquefaction. Clayey soils and bedrock generally are not susceptible to liquefaction.

Possible consequences of liquefaction include vertical settlement, lateral displacement, loss of bearing capacity for foundations supported by soil that liquefies, increased lateral loading on structures retaining soil that liquefies, and flotation of lightweight structures embedded in soil that liquefies.

Soil materials classified during the geotechnical investigation program included silty loam, silty clay loam, and clay loam materials with generally medium stiff to stiff consistencies. These materials are generally not considered to have a high potential for liquefaction and do not warrant additional analysis. Sandy materials identified below the mudline within the river would possibly be subject to liquefaction; however, these soils are also considered to be above the scour depth as described in Section 4.1.1 and cannot be relied upon for foundation resistance. For this reason, a detailed liquefaction analysis is not justified.

3.6 Abandoned Mines and Existing Utilities

The available ISGS records do not indicate any former mining activity near the project location. Also we are not aware of any records indicating the presence of utilities which could impact the construction of the proposed bridge.

4. DESIGN CONSIDERATIONS AND RECOMMENDATIONS

4.1 Foundations

Foundations for support of the new bridge piers and abutments were evaluated considering the subsurface conditions, proposed design loads, and constructability with respect to existing site conditions, particularly the presence of the existing bridge and abutments as well as the IDOT right-of-way. Selection of the foundation type, size, and bearing grade is based upon providing a foundation with sufficient resistance against a bearing capacity failure, negligible total and differential settlement, and acceptable lateral deflection. Cost is also a factor. During final design, loading from vessel impact shall be considered when sizing the foundations.

4.1.1 Scour Potential

Table 4 below provides the design scour elevations are presented in the format required by Section 2.3.6.3.2 of IDOT Bridge Manual (2012) taking into account soil conditions and any presence of stone rip rap protection at abutments. The elevations presented are revised from those reported in the Draft Hydraulic Report (2012) for the bridge. According to the IDOT Bridge Manual (2012), the design scour elevation at the abutment locations will be given by the elevation of the pile cap bottom. However, per the IDOT Bridge Manual reductions to the final design scour elevations can be made when cohesive soils or rock exist. Per the Manual, non-weathered limestone or dolomite is generally not considered susceptible to scour and, in most cases, should be assumed to arrest scour from extending below the non-weathered section. The elevations indicated for Pier 1 and 2 are revised based on the non-weathered rock elevation as encountered in SB-06 and SB-07, respectively.

Table 4 Design Scour Elevations

	North Abutment	Pier 1	Pier 2	South Abutment
Design Scour Elevation (ft)	490.44	430.9	432.9	490.44

4.1.2 Abutment Foundations

While spread footings would be the least expensive alternative, they are not suitable for support of the new bridge abutments based on anticipated loads and the subsurface conditions since the existing abutment fill consists of uncontrolled fill, which is generally considered unsuitable for foundation support. Accordingly, foundations for the new abutments will need to be supported on deep foundations. Considering that bedrock is within a reasonable depth to provide end bearing resistance and soils overlying the bedrock will not provide significant frictional resistance, the deep foundations will need to terminate on or in the bedrock. Two suitable foundation schemes recommended, considering the existing subsurface conditions, are steel H-piles driven to bedrock or drilled shafts socketed into bedrock.

The selection of the deep foundation type for support of the new bridge must consider the impacts of construction cost. Assuming that the length of the pile is sufficient enough such that pile fixity can be achieved, the relative cost to drive piles outweighs any added benefit from drilled shaft foundations. Therefore, H-piles driven to bedrock are recommended.

The H-piles for support of the bridge abutments will be driven through the overlying fill and natural soils to practical refusal end bearing on the dolostone bedrock. For properly driven piles bearing on the bedrock, a very high geotechnical resistance will be available. Therefore, assuming the geotechnical resistance is adequate, AASHTO LRFD Bridge Design Specifications (2010) recommends using the structural resistance of the pile section with an appropriate resistance factor.

Given that the piles are driven to refusal at bedrock, settlement of the pile group can be assumed to be less than ½-in which is acceptable for most bridge types. Further settlement analyses are not warranted.

Estimated minimum pile tip elevations based on the bedrock elevations as identified in the borings at the north abutment and south abutment are El. 447-ft and El. 444-ft, respectively. Tip reinforcement, i.e., pile shoes, should be used to prevent tip damage by driving the H-piles to bedrock. The requirement for test piles at each of the abutments is not warranted as the confidence level of the estimated pile lengths is relatively high since the bedrock surface is generally consistent. Furthermore, a conservative estimated pile length is provided which will provide cost savings in lieu of driving test piles.

H-piles bearing on rock should be driven to their Maximum Nominal Required Bearing and to the minimum tip elevations identified above. Maximum Nominal Required Bearing should be defined as 20 blows per 1-in of driving or where no measurable net penetration under hammer blows is recorded. Because the piles are being driven to bedrock, sustained driving resistance and practical refusal criteria should also be provided in the specifications.

Pile hammers should have a minimum energy to enable piles to be driven to a nominal resistance of at least 2.75 times the allowable pile resistance without damage to the pile. The location of the nominal resistance should be in the rising portion of the resistance versus blow count curve. The hammer should also not induce driving stresses in the pile in excess of 90 percent of the yield point of the pile material.

4.1.3 Pier Foundations

Given the very shallow depth to bedrock as indicated in Section 3, piles should not be considered a suitable foundation alternative for the bridge piers. Instead, the use of shallow foundations set on bedrock or drilled shafts socketted into bedrock may be considered. Although shallow foundations would provide for the most cost-effective foundation alternative, the size of the footing may be required to extend beyond the right-of-way limitations or may interfere with the existing bridge spread footing foundations in order to satisfy the loading conditions. Therefore, it is recommended during final design, when the final bridge pier loads are developed, that shallow footing be designed and sized. If geometry issues arise from the required size of the footings, drilled shaft foundations should then be considered. Recommendations for both foundation alternatives are provided below.

Shallow Foundations

The proposed bridge piers could be supported upon shallow foundations set into bedrock. It was estimated that spread foundations having an approximate dimension of 25 x 50-ft and a maximum nominal bearing resistance of 220 ksf, could anticipate estimated settlement of less than 1-in under the service limit state. Based on the relative uniformity of the rock conditions, differential settlements due to variations in the bedrock profile between the piers are not anticipated to be considerable. Under the strength limit state, a maximum factored resistance of 72 ksf was estimated based on the average rock strength and anticipated vertical joint spacing of the rock mass. It should be noted that the nominal bearing resistance of the foundations should not be greater than the compressive resistance of the footing concrete.

The foundations should be adequately designed to resist sliding and overturning caused by lateral and/or eccentric loading. Resistance to lateral loads can be developed by sliding friction between the bearing bedrock and the bottom of the footings. A nominal (unfactored) coefficient of sliding friction of 0.55 may be assumed between the bottom of the concrete footing and the bedrock and a resistance factor of 0.80 is recommended. Sliding resistance due to passive pressure in front of the footing can be applied given that the lower portion of the footing is keyed into bedrock. If sliding resistance is required from embedment in rock, the bottom of footing elevation should be adjusted to ensure the required minimum embedment. The rock excavation should 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. A nominal passive resistance equivalent fluid pressure of 380 pounds per cubic foot (pcf) acting against the embedded portion of the footing may be used with a resistance factor of 0.50.

The shallow foundation should be designed such that the eccentricity of loading at the strength limit state should not exceed 3/8 of the corresponding footing dimensions. AASHTO LRFD Bridge Design Specifications (2010) recommends sizing the footing based on eccentricity requirements at the strength limit state before checking the nominal bearing resistance at both the service and strength limit states.

Spread footings should be founded on unweathered bedrock, as described in Section 3. To ensure the entire footing will bear in competent, unweathered rock, it is recommended that the footing be placed at a minimum of 2-ft below the top of unweathered rock as identified in the boring logs. As such, the recommended elevations for the bottom of footings are 428.9 for the north pier (Pier 1) and 430.9 for the south pier (Pier 2). Per the recommendations of IDOT District 3 Project Implementation, a six inch thick "mud slab" should be installed below the bottom of the footings to order to maintain the freshly exposed bedrock's integrity and to maintain sliding resistance. Therefore, during construction, the excavations for the footings should be undercut by 6 in. and immediately filled with concrete.

It is recommended that a geotechnical engineer inspect the bearing rock exposed in footing excavations before the "mud slab" is placed. The geotechnical engineer should verify that the bearing stratum consists of unweathered rock. If weathered bedrock is exposed in any of the excavations, the footing should be deepened or the weathered bedrock should be excavated and replaced with concrete with a minimum compressive strength equivalent to the footing. The weathered bedrock should be completely removed in the area of the foundation.

A cofferdam will be required to keep the footing excavation dry. Based on the depth of water, the IDOT Bridge Manual requires that a Type 2 Cofferdam be installed. The Estimated Water Surface Elevation (EWSE) should be taken as El. 444.2 and, as per the requirements of the IDOT Bridge Manual (2012), the top of the Type 2 Cofferdam should be installed to minimum elevation of 3-ft above the (EWSE). Given the bedrock surface is very close to the mudline, sheet piles will not be able to penetrate deep enough below the mudline to provide adequate resistance. There are several alternative cofferdam solutions which could be utilized. It is recommended that the contractor be allowed to determine the most cost-effective solution for the cofferdam given his materials and resources provided that it meets the minimum requirements for the project including water flow and safety.

If water infiltration into the cofferdam is observed after excavation into the bedrock is complete, a concrete seal coat will be required to provide a watertight seal. The seal coat should be made of Class SC Concrete tremied underwater. It should be noted that the use of a seal coat will preclude the requirement of a mud slab, as indicated above. An initial seal coat thickness of 4.5-ft is estimated for the contract plans and quantities. The Contractor, however, will be required to provide a final estimated seal coat thickness once his means and methods for the cofferdam type are established.

Drilled Shafts

In lieu of shallow foundations, the bridge piers could also be founded on drilled shafts socketted into bedrock. Although shallow foundations are commonly more economical than drilled shafts, it is possible that drilled shafts could offer cost saving by eliminating the need for a cofferdam. Based on preliminary TS&L plans however, it appears the pile cap for drilled shaft foundations would extend to or near the mudline/top of rock elevation which would not eliminate the need for a cofferdam. If required based on the final design location of the drilled shaft supported footing, the drilled shafts should use temporary or permanent casing which is seated a minimum of 6-inches into competent bedrock. The rock socket should be 6-in smaller in diameter than the cased portion. Due to the potential for seepage from the weathered and fractured rock zones, construction of the rock sockets should be performed using wet excavation methods with a positive head to control potential seepage conditions.

The axial load resistance of the drilled shafts were developed using the methods and recommendations as presented in the FHWA publication “Drilled Shafts: Construction Procedures and LRFD Design Methods” (2010) and AASHTO LRFD Bridge Design Specifications (2010). The following recommendation regarding the axial resistance of the drilled shafts is provided:

- Temporary or permanent casing should be used and seated into competent rock. A minimum of 1% reinforcement should be included. The rock socket diameter should be constructed to 6-in smaller than the casing diameter in soil.

- Center to center shaft spacing for shaft group arrangements should be 3 times the shaft diameter in the longitudinal direction (parallel to the bridge centerline) and 2.5 times the diameter in the transverse direction (perpendicular to the bridge centerline).

- Side resistance from weathered rock and soil located above the competent rock is ignored. Based on the scour elevations as presented in Table 4, the material should not be relied upon for axial or lateral resistance.

- The maximum uniform nominal side friction for the rock socket was estimated to be 34.9 ksf. This value was based on a compressive strength of the drilled shaft concrete of 4000 psi. If the concrete compressive strength is increased to 5000 psi, the maximum uniform nominal side friction for the rock socket can be taken as 39 ksf.

- The nominal end bearing of the rock socket was estimated to be 995 ksf assuming a minimum rock socket penetration of 20 feet. It was determined that the value obtained by using the methods recommended in AASHTO LRFD Bridge Design Specifications (2007) was unnecessarily conservative. Therefore, the above value was based on the recommendations FHWA publication “Drilled Shafts: Construction Procedures and LRFD Design Methods” (2010). It should be noted that the nominal bearing resistance of the drilled shafts should not be greater than the compressive resistance of the shaft concrete. Also, note that a minimum penetration of 20-ft was estimated to achieve fixity of the shafts. Should during final design, the point of fixity be estimated less than 20-ft into bedrock, the end bearing recommendations for material above 20-ft will be provided.

- Based on the recommendations of the IDOT Bureau of Bridges and Structures Foundations Unit, the axial resistance of the shafts shall be estimated by only utilizing end bearing **OR** side friction, whichever is larger. That is, only one type of resistance may be used in the design and not a combination of both.

- A resistance factor of 0.55 and 0.50 are recommended for the side resistance and end bearing, respectively. The resistance factors are based on the recommendations as specified in AASHTO LRFD Bridge Design Specifications (2010) with no load testing performed.

- Uplift resistance of the shaft should only rely on socket side friction. A resistance factor of 0.40 is recommended based on as specified in AASHTO LRFD Bridge Design Specifications (2010).

- Per the recommendations of the IDOT Bridge Manual (2012), for drilled shafts in rock it can be assumed that shaft settlement will be minimal and a service vertical deflection check is not required.

The recommendations as presented above are only valid if the construction considerations as described in Section 4.4.8 are adhered to. It is imperative that all drilled shaft be observed and inspected full-time by an experienced Foundation/Geotechnical Engineer. Further discussion is provided in Section 4.4.8.

4.1.4 Lateral Deep Foundation Performance

As per the recommendations of the IDOT All Geotechnical Manual Users (AGMU) Memorandum 12.0 “New Structure Geotechnical Report Categories and Scope” deep foundations subject to lateral loads meet the criteria for a Geotechnical Design Memorandum. Therefore, during final design, when the lateral loads are more refined, a lateral load analysis of the foundations systems will be performed taking into account the effects of the pile group. Results of the analysis will indicate pile/shaft head deflections, maximum moments, and depth to fixity.

4.2 Slope Stability

Given the limited amount of overburden material located above bedrock, deep global instability conditions should not be a factor during the placement of additional fill adjacent to the existing abutments or during long-term service conditions of the bridge. New slopes should be graded no steeper than the slopes for the existing abutments. If surficial sloughing occurs during construction, the material should be re-graded to a flatter slope. If conditions at the site are different than anticipated, global stability analyses should be conducted.

4.3 Settlement

Construction of the replacement bridge will cause the proposed embankment centerline to be realigned up to 50 feet east of the existing centerline and the proposed profile will be approximately 7 feet higher than the existing embankment profile. The total embankment height will be up to 50 feet high in some areas and over 300 feet wide, although a considerable portion of it will be the existing embankment.

Given the limited amount of overburden material located above bedrock, additional fill placed adjacent to the existing abutments is not anticipated to cause any significant settlement. The only significant overburden material identified by the subsurface exploration were in Borings SB-9 and SB-10, which identified loam materials up to 5-ft in thickness.

These materials were classified as silty loam, silty clay loam, and clay loam and, based on SPT values, were generally medium stiff to stiff consistency. Groundwater was not observed in the loam materials and water contents ranged from 17 to 23%. As such, these materials are not considered to be subject to consolidation settlements.

During placement of fill, minor immediate, elastic settlements of the loam materials, as well as the existing fill material, may be observed. These settlements are estimated to range from 1 to 2 inches and are anticipated to occur immediately as the fill is being placed. In order to prevent downdrag forces from impacting the piles at the abutments, and to ensure the immediate settlements have completed, it is recommended that pile driving be prevented from commencing until at least 1-week after fill operations have been completed.

4.4 Construction Considerations

4.4.1 Site Preparation

All vegetation, surface topsoil, and debris should be cleared and stripped where foundations, embankments, structural fills, and pavements are to be placed. The exposed sub-grade should be proofrolled. To aid in locating unstable and unsuitable materials, the proofrolling should be observed by a qualified engineer.

If shallow foundations are determined suitable for the river piers, it is recommended that a geotechnical engineer inspect the bearing rock exposed in footing excavations before reinforcing steel and concrete are placed. The geotechnical engineer should verify that the bearing stratum consists of unweathered rock. If weathered bedrock is exposed in any of the excavations, the footing should be deepened or the weathered bedrock should be excavated and replaced with lean concrete. The weathered bedrock should be completely removed in the area of the foundation. The limits of the excavation can be defined by lines extending downward at an inclination of 1 horizontal to 1 vertical from the toe and heel of the footing to the bottom of the weathered bedrock.

4.4.2 Excavation

Foundation excavations should be performed in accordance with local, state, and federal regulations. The potential effect of ground movements upon nearby roadways and utilities should also be taken into consideration.

Design of excavation support or temporary open cut side slopes is the sole responsibility of the contractor. Piles of excavated soil and heavy construction equipment should not be permitted closer to the top of any excavation than a distance equal to two times the depth of the excavation.

4.4.3 Dewatering

Site grades should be sloped to prevent groundwater flow into open excavations and promote rapid precipitation runoff away from excavations. Water that does accumulate into the open excavations by seepage or runoff should be immediately removed. Groundwater appeared to be controlled by the river elevation. It is expected that the groundwater conditions will vary from the observed conditions in the future on a seasonal basis, depending upon the precipitation, runoff, infiltration, land use, and river levels. Depending on the actual groundwater conditions encountered during construction, it is anticipated that a conventional shallow sump/pump method would be appropriate. However, if significant water inflow occurs into an excavation, other methods such as deep sumps, well points or cutoff with tight sheeting or equivalent cofferdam methods may be considered by the contractor.

4.4.4 Drainage

To reduce the build-up of hydrostatic pressure behind the wing walls it is recommended that a free draining granular material be used as backfill. The drainage system may consist of geocomposite wall drain or weep holes. In the event it is decided to use weepholes to mitigate the hydrostatic pressure, the weepholes may be approximately 3 inch in diameter, spaced approximately 8 feet apart horizontally and 6 feet apart vertically. The weepholes should be protected on the soil side by using a properly designed granular filter, to avoid migration of fines, resulting in blockage of the weepholes.

4.4.5 Filling and Backfilling

Fill material used to construct the approach embankments should conform to and be placed in lifts and compacted per the requirements specified in Article 205, *Embankment*, of the IDOT *Standard Specifications* (2012). The fill material should be free of organic matter and debris.

All backfill materials must be pre-approved by the site engineer. To backfill the wing walls porous granular material that conforms to the requirements specified in Article 1003 or 1004 of the IDOT *Standard Specifications* (2012) is recommend. Backfill material should be placed and compacted in accordance with Article 205, *Embankment*, of the IDOT *Standard Specifications* (2012). Estimated design parameters for granular backfill materials assuming horizontal backfill surface are presented in Table 5.

Table 5 Estimated Granular Backfill Parameters for Horizontal Backfill Surface

Soil Description	Porous Granular Material Backfill
Unit Weight	125 lbs/ft^3
Angle of Effective Internal Friction	30 degrees
Active Earth Pressure Coefficient	0.33
Passive Earth Pressure Coefficient	3.0
At-Rest Earth Pressure Coefficient	0.5

4.4.6 Earthwork Operations

The required earthwork can be accomplished with conventional construction equipment. Moisture and traffic will cause deterioration of exposed sub-grade soils. Precautions should be taken by the contractor to prevent water erosion of the exposed sub-grade. A compacted sub-grade will minimize water runoff erosion.

Earth moving operations should be scheduled to not coincide with excessive cold or wet weather (early spring, late fall or winter). Soil exposed to cold weather should be protected from freezing. Any soil allowed to freeze or soften due to the standing water should be removed from the sub-grade. Wet weather can cause problems with sub-grade compaction when the water contents exceed optimum.

It is recommended that an experienced geotechnical engineer be retained to inspect the exposed sub-grade, monitor earthwork operations and provide material inspection services during the construction phase of this project.

4.4.7 Pile Driving

The piles should be installed according to the current IDOT Standard Specifications (2012), Section 512, "Piling."

4.4.8 Drilled Shaft Construction

If drilled shafts are chosen for the design of the bridge pier foundations they should be installed according to the current IDOT Standard Specifications (2012), Section 516, "Drilled Shafts" with the modifications as provided herein.

Prior to the start of construction, the contractor should be required to submit a Drilled Shaft Installation Plan. The plan should be specific to the project and should fully document the equipment and procedures to be used in the work. The plan should be used by the on-site personnel to confirm compliance of the contractor's proposed equipment and procedures with contract documents and with the planned procedures, and to identify proposed practices that may adversely influence the design characteristics of the structures. The installation plans should be a working document subject to revisions and IDOT review based on site conditions and lessons learned during construction.

The contractor should be required to install a full-sized technique shaft at the start of construction. The technique shaft should be installed using the equipment and procedures as identified in the Drilled Shaft Installation Plan. The demonstration shaft should be used to help identify difficulties to be encountered during installation of the demonstration shaft and to help develop modifications to the installation plan to avoid these problems at production shafts. The technique shaft can be installed on a production shaft, however, further installation of drilled shafts shall be halted until approval of the cross-hole sonic testing (CSL) as described below.

Since the performance of the drilled shafts depend largely on the care taken in their construction, it is particularly important to include a program of non-destructive testing (NDT) to assess the integrity of the completed shafts. It is recommended that NDT be performed on the technique shaftand a minimum of 1/3 of the remainder of production shafts. NDT should include cross-hole sonic logging (CSL) with steel access tubes installed in all shafts on the project. At each shaft, one CSL tube should be used for each foot of shaft diameter, and the tubes should be equally spaced around the inside of the steel reinforcement cage. In cross-hole sonic logging, vertical tubes are installed in the drilled shaft and are used to provide access for an ultrasonic transmitter probe and receiver probe for the full length of the drilled shaft. CSL tubes should be installed in every shaft with CSL testing performed at random drilled shaft locations during the progress of foundation construction to confirm the continued success of the contractor's drilled shaft installation procedures. The CSL methods provide data for assessing the quality of the shaft concrete, and the presence of voids or other discontinuities within the shaft. When the NDT testing identifies anomalies, coring of the shafts may be necessary, depending on the nature and location of the anomaly, to obtain samples of the concrete, or to identify the location and nature of defects in suspect shafts.

Given the rock sockets of the drilled shafts will be constructed with drilling fluids, inspection of the sockets after drilling should include the use of a borehole caliper used at all demonstration shafts, load test shafts and initial production shafts to measure the shape of the borehole as a function of depth. Also, a downhole shaft inspection device, or SID, should be used to provide a remote image of the socket to verify that the base of the shaft is properly cleaned.

It is imperative that all drilled shaft construction be observed and inspected full-time by an experienced Foundation/Geotechnical Engineer. The inspector should produce full documentation of the work being performed. Well prepared records of the contractor's equipment, materials and installation, along with post-construction integrity testing, are essential in order to help identify and assess problems or concerns should they occur, and to correct any problems during drilled shaft installation.

Prior to construction of the caissons and shafts, procedures should be established for the timely communication of information between the inspector, client and designer to quickly address any questions or problems that may arise. The inspector should not only be observing and recording but also should be proactive and must make judgments in order to identify any potential problems and to be able to communicate any identified issues to the construction manager and client in order to provide a timely resolution.

In order to identify differing subsurface conditions and to help avoid problems, all on-site inspection staff should have and become familiar with all appropriate records and documents including boring logs, the Geotechnical Report, plans and specifications, and the latest version of the Drilled Shaft Installation Plan. The inspector should also meet with the designer for a briefing on the key issues and criteria for the project prior to construction.

4.4.9 Construction Monitoring

With exception to the drilled shaft construction considerations mentioned above, there is no need for a special construction monitoring for the foundations except normally required by the IDOT Standard Specifications, Special Provisions and Contract Plans.

5. QUALIFICATIONS

The analysis and recommendations contained in this report are based on the soils and rock encountered at the boring locations shown in Exhibit 2. This report does not reflect any variations that may occur between the borings or elsewhere on the site, variations whose nature and extent may not become evident until a later stage of construction. Soil conditions may also change with the passage of time due to changes in the elevation of the groundwater table, changes in climatic conditions and other factors not evident at the time of the geotechnical subsurface investigation. Should conditions encountered during excavation and construction operations differ from those encountered in the borings, Parsons Brinckerhoff should be notified so that the foundation recommendations presented in this report can be reviewed, verified and revised in writing if necessary.

It has been a pleasure to assist IDOT on this phase of the project. Please contact us if there are any questions, or if we can be of further service.

Respectfully submitted,
PARSONS BRINCKERHOFF

Mark Stephani, PE
Supervising Geotechnical Engineer
stephani@pbworld.com

Sotirios Vardakos, PhD
Lead Geotechnical Engineer
vardakos@pbworld.com

6. REFERENCES

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications, 5th Ed. 2010.

2. Illinois Department of Transportation, Bureau of Bridges and Structures, Bridge Manual, January 2012.

3. Illinois Department of Transportation, All Geotechnical Manual Users Memorandum 12.0: New Structure Geotechnical Report Categories and Scope, June 2012.

4. Illinois Department of Transportation, Standard Specifications for Road and Bridge Construction, January 2012.

5. Illinois State Geological Survey, Directory of Coal Mined in Illinois, May 2000.

6. Parsons Brinckerhoff, Draft Hydraulic Report, FAS 1279 (IL 178) over Illinois River, S.S. 050-0088, May 2012.

7. U.S. Department of Transportation Federal Highway Administration, Drilled Shafts: Construction Procedures and LRFD Design Methods, Publication No. FHWA-NHI-10-016, May 2010.

EXHIBITS

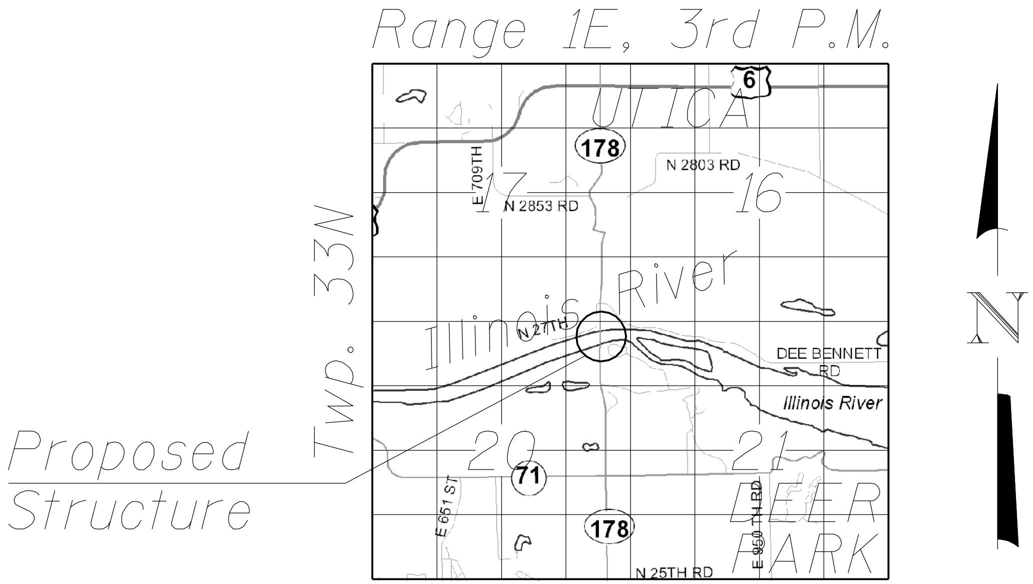

EXHIBIT 1

SITE LOCATION PLAN
IL 178 F.A.S. ROUTE 1279 OVER
ILLINOIS RIVER PUBLIC WATER
LASALLE COUNTY, SECTION (1)BR & 1
STATION 29+19.53
STRUCTURE NO. 050-0256

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11-FEB-2013 11:10

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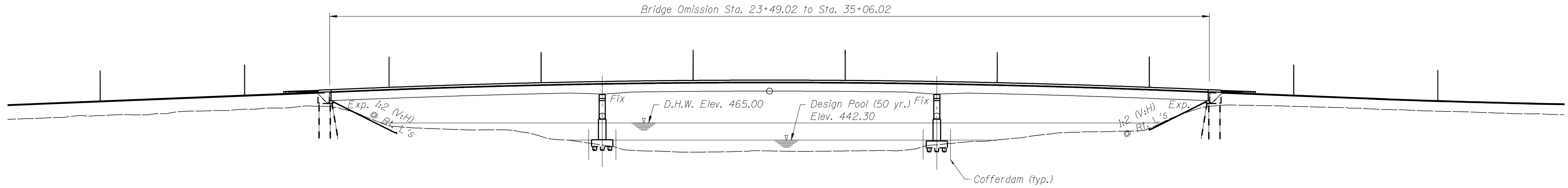
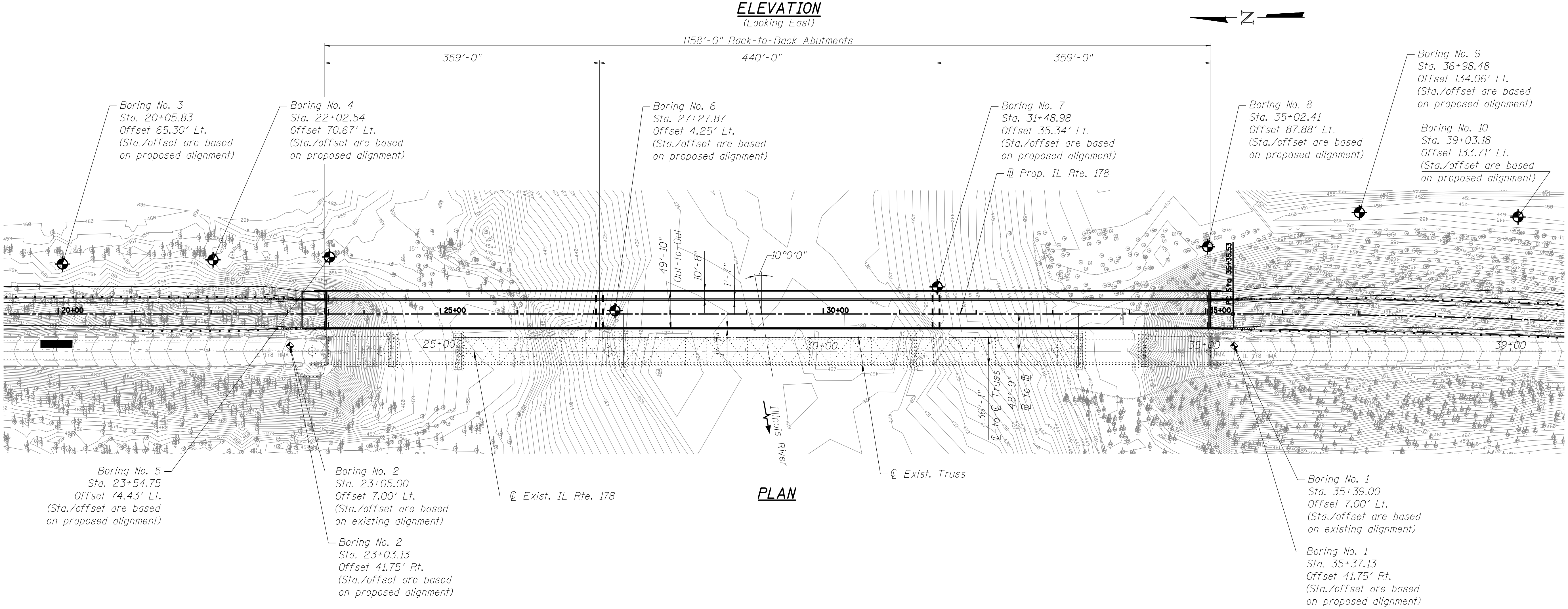
		CHECKED - A. Hammad	REVISED -	**DEPARTMENT OF TRANSPORTATION**	
	PLOT SCALE = NONE	DRAWN - D. C. Patel	REVISED -		
	PLOT DATE = 11-FEB-2013 11:10	CHECKED - Y. Ali	REVISED -		SHEET NO. OF SHEETS

F.A.S. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
178	(1)BR & 1	LASALLE		
		CONTRACT NO. 66992		
	ILLINOIS FED. AID PROJECT			



ELEVATION
(Looking East)



PLAN

LEGEND:

- Soil boring location (existing 1959)
- Soil boring location performed by IDOT (2012)
- Soil boring location performed by PB

Notes:

1. Boring Nos. 1 and 2 previously conducted by IDOT on 4/2/12.
2. Boring Nos. 3 thru 10 performed by PB.

EXHIBIT 2

BORING LOCATION PLAN
IL 178 F.A.S. ROUTE 1279 OVER ILLINOIS RIVER PUBLIC WATER
LASALLE COUNTY, SECTION (1)BR & 1
STATION 29+19.53
STRUCTURE NO. 050-0256

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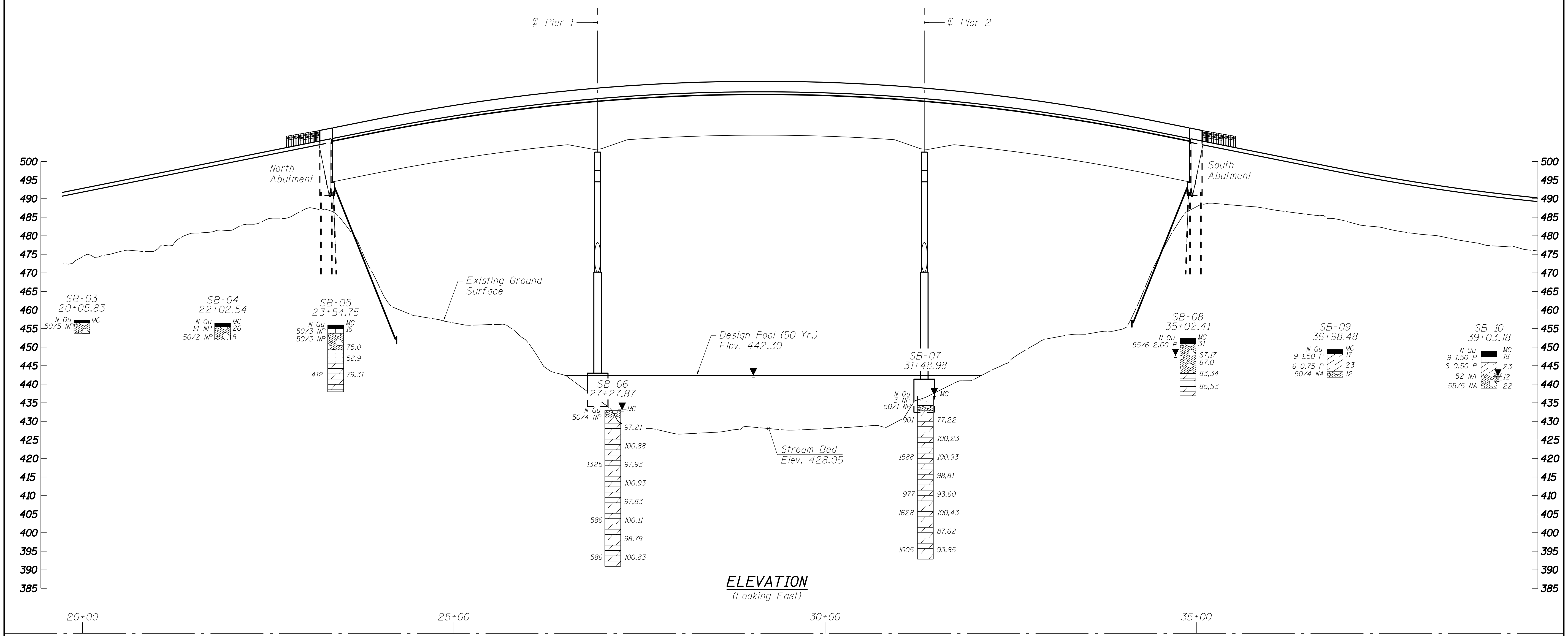
	PLOT DATE = 11-FEB-2013 11:10	CHECKED - Y. Ali	REVISED -		SHEET NO. OF SHEETS

F.A.S. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
178	(1)BR & 1	LASALLE		
		CONTRACT NO. 66992		
		ILLINOIS FED. AID PROJECT		



Explanation:

SB-03 — Borehole Number
22+02.54 — Station

Borehole Lithology

N--N-value, (blw/12 in)
Qu--UC Strength, (tsf)
MC--Moisture Content, (%)
100.93 - Recovery (%), RQD (%)

Water Level Reading at time of drilling.

Water Level Reading 24-hr after drilling or at end of drilling

Lithology Graphics

Topsoil	Weathered bedrock	Silt, Silty Loam	Sandstone	
Dolomite or Dolomitic Limestone	Sand, Sandy Loam	Clay Loam	Silty Clay, Silty Clay Loam	

EXHIBIT 3

SUBSURFACE PROFILE
IL 178 F.A.S. ROUTE 1279 OVER ILLINOIS RIVER PUBLIC WATER
LASALLE COUNTY, SECTION (1)BR & 1
STATION 29+19.53
STRUCTURE NO. 050-0256

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		CHECKED - A. Hammad	REVISED -		
	PLOT SCALE = NONE	DRAWN - D. C. Patel	REVISED -		
	PLOT DATE = 1/21/2014	CHECKED - Y. Ali	REVISED -		SHEET NO. OF SHEETS

F.A.S. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
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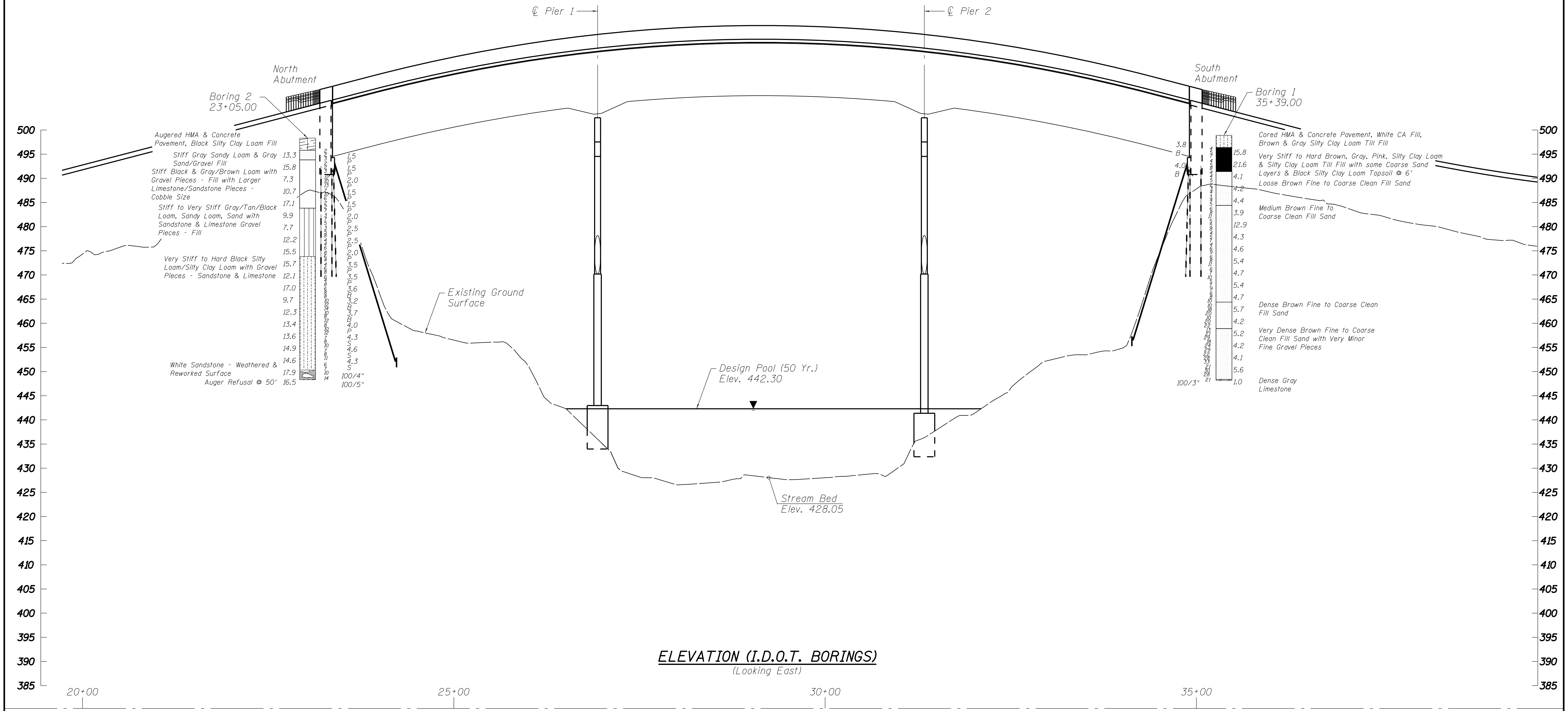
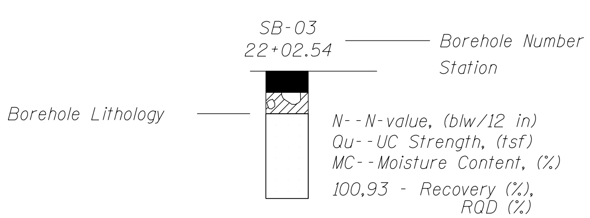
		CONTRACT NO. 66992		
		ILLINOIS FED. AID PROJECT		



ELEVATION (I.D.O.T. BORINGS)

(Looking East)

Explanation:



Water Level Reading at time of drilling.

Water Level Reading 24-hr after drilling or at end of drilling

Lithology Graphics

Topsoil	Weathered bedrock	Silt, Silty Loam	Sandstone	
Dolomite or Dolomitic Limestone	Sand, Sandy Loam	Clay Loam	Silty Clay, Silty Clay Loam	

EXHIBIT 3A

SUBSURFACE PROFILE
IL 178 F.A.S. ROUTE 1279 OVER ILLINOIS RIVER PUBLIC WATER
LASALLE COUNTY, SECTION (1)BR & 1
STATION 29+19.53
STRUCTURE NO. 050-0256

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	PLOT DATE = 1/21/2014	CHECKED - Y. Ali	REVISED -

STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

SHEET NO. OF SHEETS

F.A.S. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
178	(1)BR & 1	LASALLE		
		CONTRACT NO. 66992		
		ILLINOIS FED. AID PROJECT		

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APPENDIX A

BORING LOGS

Illinois Department of Transportation
Division of Highways
ILLINOIS DOT

SOIL BORING LOG

Date 4/2/12

ROUTE IL 178 (FAS 1279) **DESCRIPTION** Illinois River Bridge South of Utica **LOGGED BY** Larry Myers

SECTION (1)BR&I **LOCATION** SE 1/4, **SEC.** 17, **TWP.** 33N, **RNG.** 2E

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

STRUCT. NO. 050-0088 (Exist.)
Station 29+30.00

BORING NO. 1 (South Abut.)
Station 35+39.00
Offset 7.00ft Lt.
Ground Surface Elev. 498.99 ft

Surface Water Elev. ft
Stream Bed Elev. ft

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

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Cored HMA & Concrete Pavement, White CA Fill, Brown & Gray Silty Clay Loam Till Fill	496.49				
Very Stiff to Hard Brown, Gray, Pink, Silty Clay Loam & Silty Clay Loam Till Fill with some Coarse Sand Layers & Black Silty Clay Loam Topsoil @ 6'			4		
			5	3.8	15.8
			7	B	
		-5	4		
			5	4.0	21.6
			8	B	
	491.49				
Loose Brown Fine to Coarse Clean Fill Sand			4		
			4		4.1
			5		
		-10	2		
			3		4.2
			4		
			3		
			4		4.4
			5		
	484.49				
Medium Brown Fine to Coarse Clean Fill Sand		-15	6		
			8		3.9
			11		
			5		
			5		12.9
			8		
		-20			

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Medium Brown Fine to Coarse Clean Fill Sand *(continued)*			4		
			5		4.3
			7		
			4		
			6		4.6
			9		
		-25	6		
			8		5.4
			11		
			6		
			7		4.7
			10		
		-30	5		
			7		5.4
			9		
			6		
			8		4.7
			10		
	464.49				
Dense Brown Fine to Coarse Clean Fill Sand		-35	10		
			18		5.7
			20		
			10		
			20		4.2
			22		
	458.99	-40			

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

Illinois Department of Transportation
Division of Highways
ILLINOIS DOT

SOIL BORING LOG

Date 4/2/12

ROUTE IL 178 (FAS 1279) **DESCRIPTION** Illinois River Bridge South of Utica **LOGGED BY** Larry Myers

SECTION (1)BR&I **LOCATION** SE 1/4, **SEC.** 17, **TWP.** 33N, **RNG.** 2E

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

STRUCT. NO. 050-0088 (Exist.)
Station 29+30.00

BORING NO. 1 (South Abut.)
Station 35+39.00
Offset 7.00ft Lt.
Ground Surface Elev. 498.99 ft

Surface Water Elev. ft
Stream Bed Elev. ft

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

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Very Dense Brown Fine to Coarse Clean Fill Sand with Very Minor Fine Gravel Pieces		13		
		21		5.2
		29		
		18		
		24		4.2
		29		
	-45			
		22		
		28		4.1
		33		
		21		
		30		5.6
		28		
	-50			
448.49		21		
Dense Gray Limestone 448.24		100/3"		1.0
End of Boring				
	-55			
	-60			

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

Illinois Department of Transportation
Division of Highways
ILLINOIS DOT

SOIL BORING LOG

Date 4/2/12

ROUTE IL 178 (FAS 1279) **DESCRIPTION** Illinois River Bridge South of Utica **LOGGED BY** Larry Myers

SECTION (1)BR&I **LOCATION** SE 1/4, **SEC.** 17, **TWP.** 33N, **RNG.** 2E

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

STRUCT. NO. 050-0088 (Exist.)
Station 29+30.00

BORING NO. 2 (North Abut.)
Station 23+05.00
Offset 7.00ft Lt.
Ground Surface Elev. 498.32 ft

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

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

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Augered HMA & Concrete Pavement, Black Silty Clay Loam Fill				
495.82				
Stiff Gray Sandy Loam & Gray Sand/Gravel Fill		2 2 3	1.5 P	13.3
493.82				
Stiff Black & Gray/Brown Loam with Gravel Pieces - Fill with Larger Limestone/Sandstone Pieces - Cobble Size	-5	2 2 3	1.5 P	15.8
		12 10 13	2.0 P	7.3
	-10	7 10 6	1.5 P	10.7
		2 2 7	1.5 P	17.1
483.82				
Stiff to Very Stiff Gray/Tan/Black Loam, Sandy Loam, Sand with Sandstone & Limestone Gravel Pieces - Fill	-15	3 7 5	2.0 P	9.9
		3 5 8	2.5 P	7.7
	-20			

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Stiff to Very Stiff Gray/Tan/Black Loam, Sandy Loam, Sand with Sandstone & Limestone Gravel Pieces - Fill *(continued)*		4 4 6	2.5 P	12.2
		8 6 4	2.0 P	15.5
473.82				
Very Stiff to Hard Black Silty Loam/Silty Clay Loam with Gravel Pieces - Sandstone & Limestone	-25	4 6 8	3.5 P	15.7
		6 4 8	3.5 P	12.1
	-30	6 8 8	3.6 B	17.0
		10 12 14	3.2 B	9.7
	-35	10 8 12	3.7 B	12.3
		8 10 12	4.0 P	13.4
	-40			

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

Illinois Department of Transportation
Division of Highways
ILLINOIS DOT

SOIL BORING LOG

Date 4/2/12

ROUTE IL 178 (FAS 1279) **DESCRIPTION** Illinois River Bridge South of Utica **LOGGED BY** Larry Myers

SECTION (1)BR&I **LOCATION** SE 1/4, **SEC.** 17, **TWP.** 33N, **RNG.** 2E

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

STRUCT. NO. 050-0088 (Exist.)
Station 29+30.00

BORING NO. 2 (North Abut.)
Station 23+05.00
Offset 7.00ft Lt.
Ground Surface Elev. 498.32 ft

Surface Water Elev. ft
Stream Bed Elev. ft

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

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Very Stiff to Hard Black Silty Loam/Silty Clay Loam with Gravel Pieces - Sandstone & Limestone *(continued)*			7		
			8	4.3	13.6
			10	S	
			7		
			8	4.6	14.9
			11	S	
		-45	6		
			7	4.3	14.6
			10	S	
	450.32		14		
White Sandstone - Weathered & Reworked Surface			100/4"		17.9
	448.32	-50			
Auger Refusal @ 50'			100/5"		16.5
End of Boring					
		-55			
		-60			

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

BBS, form 137 (Rev. 8-99)

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BORING LOG SB-03

WEI Job No.: 107-08-01

Client: **Parsons Brinckerhoff**
Project: **IL 178 Over Illinois River**
Location: **SE 1/4 Section 17, T 33N, R 2E of 3rd PM**

Datum: NAVD88
Elevation: 457.20 ft
North: 1699208.33 ft
East: 798328.27 ft
Station: 20+05.83
Offset: 65.3 LT

Profile	Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample Type recovery	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
	456.5	8-inch thick, black SILTY CLAY, trace roots --TOPSOIL--						
		Very dense, brown and gray, weathered DOLOSTONE fragments --WEATHERED BEDROCK--			1	15 50/5	NP	
	453.7	--AUGER REFUSAL-- Boring terminated at 3.50 ft	5					
			10					
			15					
			20					
			25					

GENERAL NOTES

Begin Drilling **10-24-2012** Complete Drilling **10-24-2012**
Drilling Contractor **Wang** Drill Rig **D-50 TMR**
Driller **R&N** Logger **B. Wilson** Checked by **C. Marin**
Drilling Method **3.25-inch IDA HSA; boring backfilled upon completion**

WATER LEVEL DATA

While Drilling ▽ **DRY**
At Completion of Drilling ▼ **DRY**
Time After Drilling **NA**
Depth to Water ▼ **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

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BORING LOG SB-04

WEI Job No.: 107-08-01

Client: **Parsons Brinckerhoff**
Project: **IL 178 Over Illinois River**
Location: **SE 1/4 Section 17, T 33N, R 2E of 3rd PM**

Datum: NAVD88
Elevation: 456.40 ft
North: 1699011.55 ft
East: 798328.18 ft
Station: 22+02.54
Offset: 70.67 LT

Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
455.4	12-inch thick, dark brown SILTY CLAY --TOPSOIL--					
	Medium dense to very dense, brown and gray, weathered DOLOSTONE fragments --WEATHERED BEDROCK--		1	4 7 7	NP	26
451.9	--AUGER REFUSAL--		2	16 50/2	NP	8
	Boring terminated at 4.50 ft	5				
		10				
		15				
		20				
		25				

GENERAL NOTES

Begin Drilling **10-24-2012** Complete Drilling **10-24-2012**
Drilling Contractor **Wang** Drill Rig **D-50 TMR**
Driller **R&N** Logger **B. Wilson** Checked by **C. Marin**
Drilling Method **3.25-inch IDA HSA; boring backfilled upon completion**

WATER LEVEL DATA

While Drilling **DRY**
At Completion of Drilling **DRY**
Time After Drilling **NA**
Depth to Water **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

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BORING LOG SB-05

WEI Job No.: 107-08-01

Client: **Parsons Brinckerhoff**
Project: **IL 178 Over Illinois River**
Location: **SE 1/4 Section 17, T 33N, R 2E of 3rd PM**

Datum: NAVD88
Elevation: 456.00 ft
North: 1698859.29 ft
East: 798327.71 ft
Station: 23+54.75
Offset: 74.43 LT

Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
455.0	12-inch thick, brown SILTY CLAY, trace gravel and roots --TOPSOIL--					
453.8	Medium dense, brown SILTY LOAM, little gravel and cinders --TRENCH FILL--		1	4 16 50/3	NP	16
449.3	Very dense, brown and gray, weathered DOLOSTONE fragments --WEATHERED BEDROCK-- RECOVERY= 75% RQD= 0%	5	2	30 50/3	NP	
			1	CORE		
445.7	Moderately weak to weak, very poor rock quality, weathered to fresh, light gray to white, bedded SANDSTONE RECOVERY= 58% RQD= 9%	10	2	CORE		
438.0	Strong, poor rock quality, brown to gray, horizontally bedded, occasionally vuggy, DOLOSTONE with shale partings RECOVERY= 79% RQD= 31%	15	3	CORE	412.00	
	Boring terminated at 18.00 ft	20, 25				

GENERAL NOTES

Begin Drilling **10-24-2012** Complete Drilling **10-24-2012**
Drilling Contractor **Wang** Drill Rig **D-50 TMR**
Driller **R&N** Logger **B. Wilson** Checked by **C. Marin**
Drilling Method **3.25-inch IDA HSA; boring backfilled upon completion**

WATER LEVEL DATA

While Drilling **DRY**
At Completion of Drilling **DRY**
Time After Drilling **NA**
Depth to Water **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

Boring SB-05:
Run #1, 5.0' to 7.0', RECOVERY = 75%, RQD = 0%
Run #2, 7.0' to 11.0', RECOVERY = 58%, RQD = 9%
Run #3, 11.0' to 18.0', RECOVERY = 79%, RQD = 31%

BEDROCK CORE: IL ROUTE 178 OVER ILLINOIS RIVER LaSALLE COUNTY, ILLINOIS		
SCALE: GRAPHICAL	SB-05	DRAWN BY: C. MARIN CHECKED BY: L.IORDACHE
Wang Engineering		1145 N. Main Street Lombard, IL 60148 www.wangeng.com
FOR PARSONS BRINCKERHOFF		107-08-01

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BORING LOG SB-06

WEI Job No.: 107-08-01

Client: **Parsons Brinckerhoff**
Project: **IL 178 Over Illinois River**
Location: **SE 1/4 Section 17, T 33N, R 2E of 3rd PM**

Datum: NAVD88
Elevation: 432.90 ft
North: 1698488.26 ft
East: 798247.20 ft
Station: 27+27.87
Offset: 4.25 LT

Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
	Very dense, gray, weathered DOLOSTONE fragments --WEATHERED BEDROCK--		1	50/4	NP	10
430.9	Strong, brownish gray to light gray, slightly weathered joints to fresh, poor to very good rock quality, horizontally bedded, horizontal greenish gray shale partings, occasionaly vuggy and cherty DOLOSTONE RECOVERY=97% RQD=21%	5	1	CORE		
	RECOVERY=100% RQD=88%	10	2	CORE		
	RECOVERY=97% RQD=93%	15	3	CORE	1325.00	
	RECOVERY=100% RQD=93%	20	4	CORE		
	RECOVERY=97% RQD=83%	25	5	CORE		
	RECOVERY=100% RQD=11%	30	6	CORE	586.00	
	RECOVERY=98% RQD=79%	35	7	CORE		
	RECOVERY=100% RQD=83%	40	8	CORE	586.00	
390.9	Boring terminated at 42.00 ft	45				
		50				

GENERAL NOTES

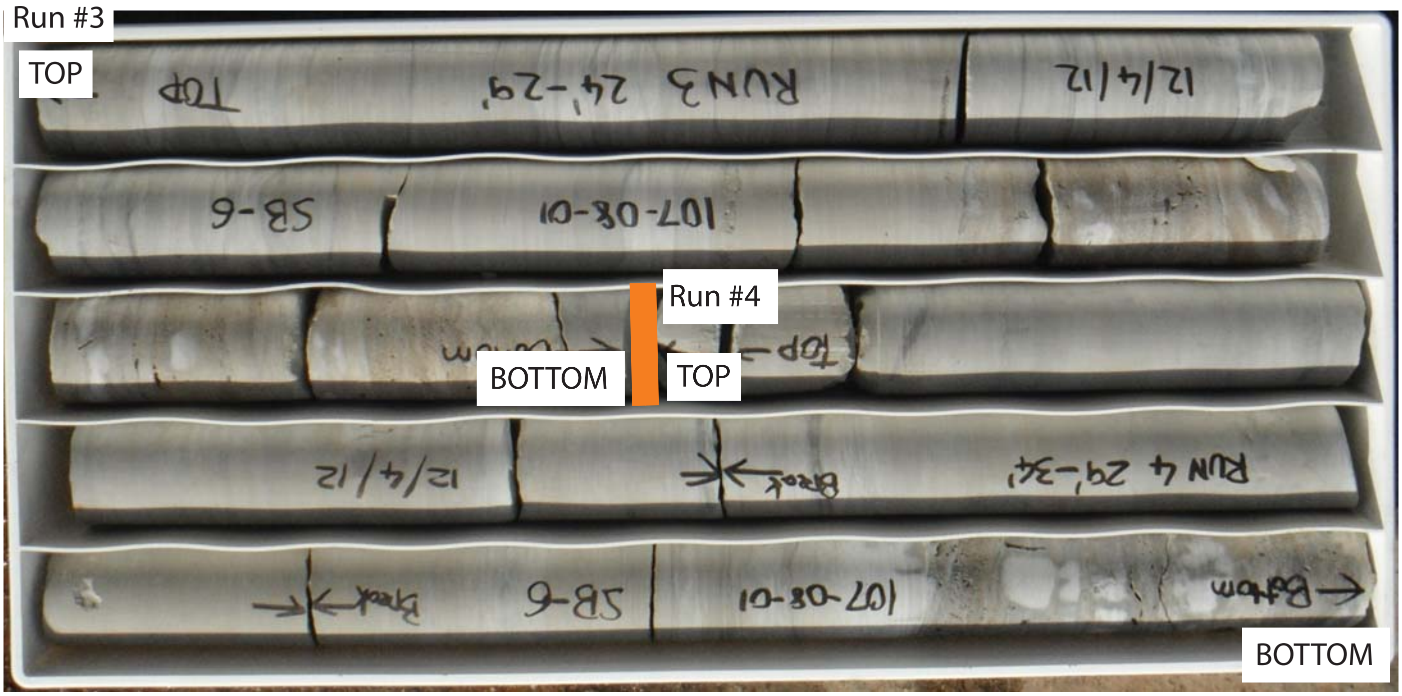
Begin Drilling **12-04-2012** Complete Drilling **12-04-2012**
Drilling Contractor **Wang** Drill Rig **B-57 TMR**
Driller **R&N** Logger **F. Bozga** Checked by **C. Marin**
Drilling Method **4-inch casing; 3.25-inch roller bit; boring backfilled upon completion**

WATER LEVEL DATA

While Drilling **0.00 ft**
At Completion of Drilling **0.00 ft**
Time After Drilling **NA**
Depth to Water **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

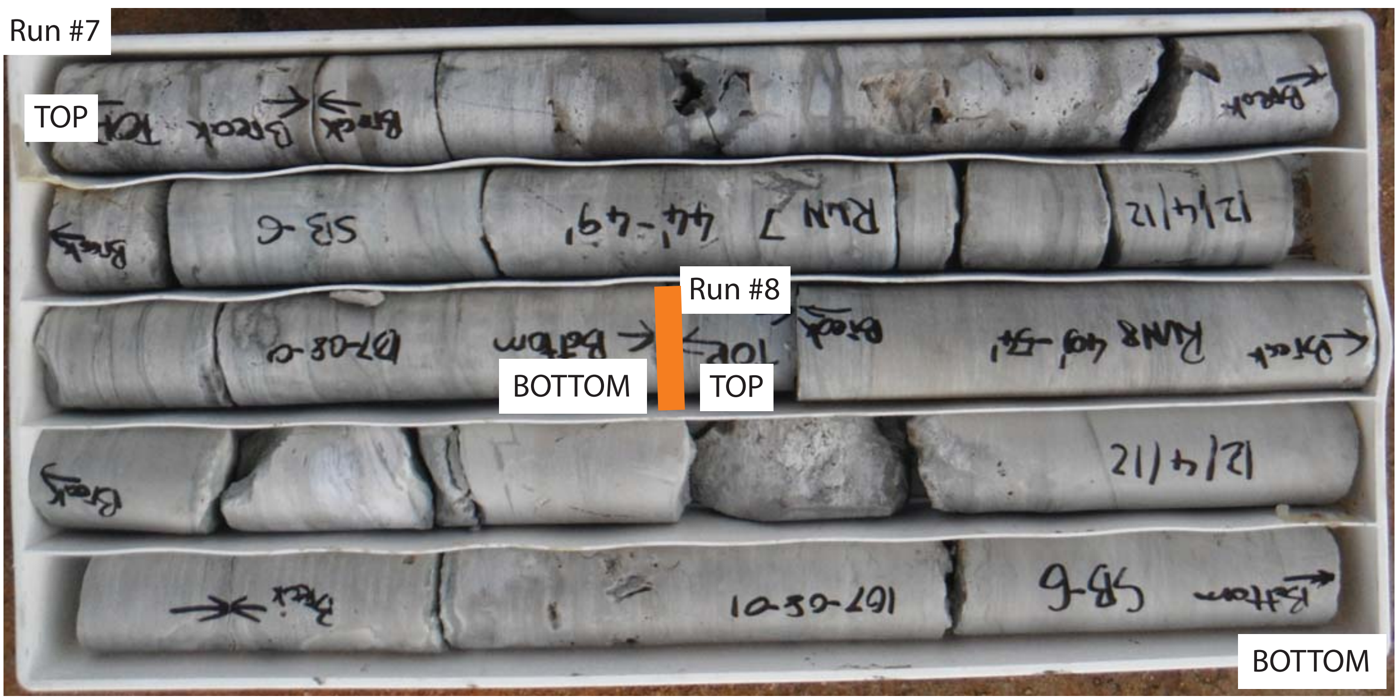
0 3 6 9 12 inches



Boring SB-06:
Run #1: 14.0' to 19.0'; RECOVERY = 97%, RQD = 21%
Run #2, 19.0' to 24.0'; RECOVERY = 100%, RQD = 88%
Run #3: 24.0' to 29.0'; RECOVERY = 97%, RQD = 93%
Run #4, 29.0' to 34.0'; RECOVERY = 100%, RQD = 93%

BEDROCK CORE: IL ROUTE 178 OVER THE ILLINOIS RIVER, LaSALLE COUNTY, ILLINOIS		
SCALE: GRAPHICAL	SB-06	DRAWN BY: C.Davis CHECKED BY:C. Marin
Wang Engineering		1145 N. Main Street Lombard, IL 60148 www.wangeng.com
FOR PARSONS BRINCKERHOFF		107-08-01

0 3 6 9 12 inches



Boring SB-06:
Run #5: 34.0' to 39.0'; RECOVERY = 97%, RQD = 83%
Run #6, 39.0' to 44.0'; RECOVERY = 100%, RQD = 11%
Run #7: 44.0' to 49.0'; RECOVERY = 98%, RQD = 78%
Run #8, 49.0' to 54.0'; RECOVERY = 100%, RQD = 83%

BEDROCK CORE: IL ROUTE 178 OVER THE ILLINOIS RIVER, LaSALLE COUNTY, ILLINOIS		
SCALE: GRAPHICAL	SB-06	DRAWN BY: C.Davis CHECKED BY: C. Marin
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FOR PARSONS BRINCKERHOFF		107-08-01

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BORING LOG SB-07

WEI Job No.: 107-08-01

Client: **Parsons Brinckerhoff**
Project: **IL 178 Over Illinois River**
Location: **SE 1/4 Section 17, T 33N, R 2E of 3rd PM**

Datum: NAVD88
Elevation: 436.90 ft
North: 1698066.45 ft
East: 798266.59 ft
Station: 31+48.98
Offset: 35.34 LT

Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
	Very loose, gray, fine SAND, trace gravel		1	0 0 3	NP	19
434.3			2	4 3 50/1	NP	20
432.9	Weathered DOLOSTONE fragments --WEATHERED BEDROCK--					
	Strong, brownish gray to light gray, slightly weathered joints to fresh, poor to very good rock quality, horizontally bedded, horizontal greenish gray shale partings, occasionaly vuggy and cherty DOLOSTONE RECOVERY=77% RQD=22%	5	1	CORE	901.00	
	RECOVERY=100% RQD=23%	10	2	CORE		
	RECOVERY=100% RQD=93%	15	3	CORE	1588.00	
	RECOVERY=98% RQD=81%	20	4	CORE		
	RECOVERY=93% RQD=60%	25	5	CORE	977.00	
	RECOVERY=100% RQD=43%	30	6	CORE	1628.00	
	RECOVERY=87% RQD=62%	35	7	CORE		
392.9	RECOVERY=93% RQD=85%	40	8	CORE	1005.00	
	Boring terminated at 44.00 ft	45				
		50				

GENERAL NOTES

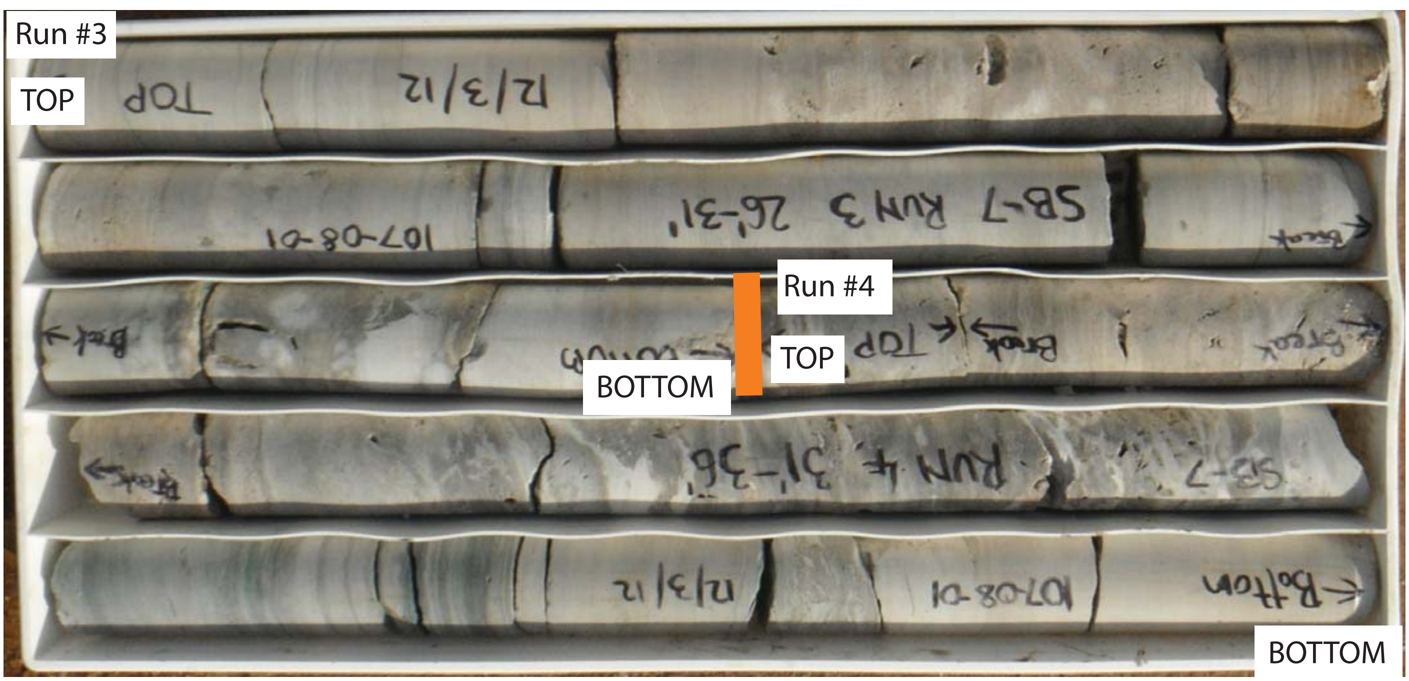
Begin Drilling: **12-03-2012** Complete Drilling: **12-03-2012**
Drilling Contractor: **Wang** Drill Rig: **B-57 TMR**
Driller: **R&N** Logger: **F. Bozga** Checked by: **C. Marin**
Drilling Method: **4-inch casing; 3.25-inch roller bit; boring backfilled upon completion**

WATER LEVEL DATA

While Drilling: **0.00 ft**
At Completion of Drilling: **0.00 ft**
Time After Drilling: **NA**
Depth to Water: **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

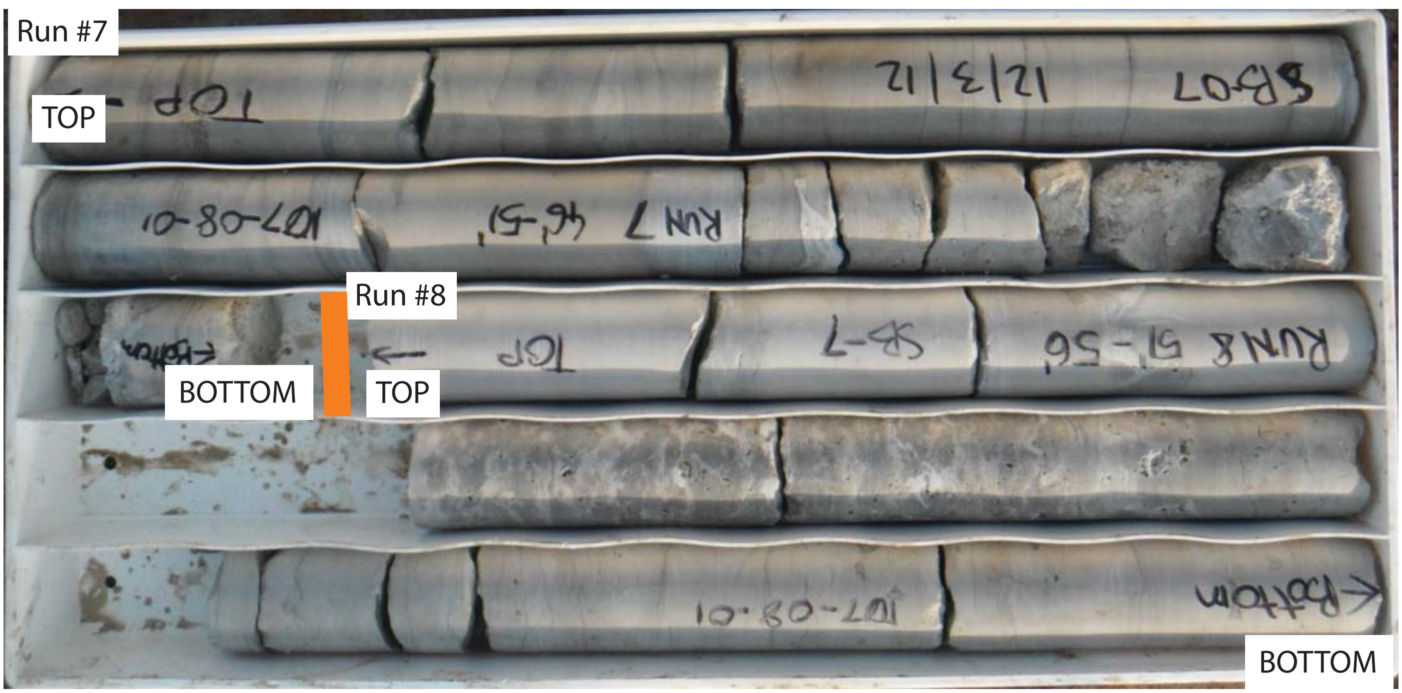
0 3 6 9 12 inches



Boring SB-07:
Run #1: 16.0' to 21.0'; RECOVERY = 77%, RQD = 22%
Run #2, 21.0' to 26.0'; RECOVERY = 100%, RQD = 23%
Run #3: 26.0' to 31.0'; RECOVERY = 100%, RQD = 93%
Run #4, 31.0' to 36.0'; RECOVERY = 98%, RQD = 81%

BEDROCK CORE: IL ROUTE 178 OVER THE ILLINOIS RIVER, LaSALLE COUNTY, ILLINOIS		
SCALE: GRAPHICAL	SB-07	DRAWN BY: C.Davis CHECKED BY: C. Marin
Wang Engineering		1145 N. Main Street Lombard, IL 60148 www.wangeng.com
FOR PARSONS BRINCKERHOFF		107-08-01

0 3 6 9 12 inches



Boring SB-07:
Run #5: 36.0’ to 41.0’; RECOVERY = 93%, RQD = 60%
Run #6, 41.0’ to 46.0’; RECOVERY = 100%, RQD = 43%
Run #7: 46.0’ to 51.0’; RECOVERY = 87%, RQD = 62%
Run #8, 51.0’ to 56.0’; RECOVERY = 93%, RQD = 85%

BEDROCK CORE: IL ROUTE 178 OVER THE ILLINOIS RIVER, LaSALLE COUNTY, ILLINOIS

SCALE: GRAPHICAL | SB-07 | DRAWN BY: C. Davis; CHECKED BY: C. Marin

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FOR PARSONS BRINCKERHOFF | 107-08-01

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BORING LOG SB-08

WEI Job No.: 107-08-01

Client **Parsons Brinckerhoff**
Project **IL 178 Over Illinois River**
Location **SE 1/4 Section 17, T 33N, R 2E of 3rd PM**

Datum: NAVD88
Elevation: 452.40 ft
North: 1697711.70 ft
East: 798309.30 ft
Station: 35+02.41
Offset: 87.88 LT

Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
	Very stiff, black SILTY CLAY --TOPSOIL--					
450.9	Very dense, brown, highly weathered DOLOSTONE, SANDSTONE and SHALE fragments --WEATHERED BEDROCK-- --AUGER REFUSAL--		1	12 55/6	2.00 P	31
	RECOVERY= 67% RQD= 17%	5 ▼	1	CORE		
	RECOVERY= 67% RQD= 0%		2	CORE		
442.9	RECOVERY= 83% RQD= 34%		3	CORE		
440.9	Moderately strong, poor rock quality, light gray, horizontally bedded DOLOSTONE with shale partings	10				
439.4	Moderately strong, fair rock quality, light gray SANDSTONE RECOVERY= 85% RQD= 53%		4	CORE		
436.9	Strong, fair rock quality, light gray, horizontally bedded, occasionally vuggy DOLOSTONE	15				
	Boring terminated at 15.50 ft					

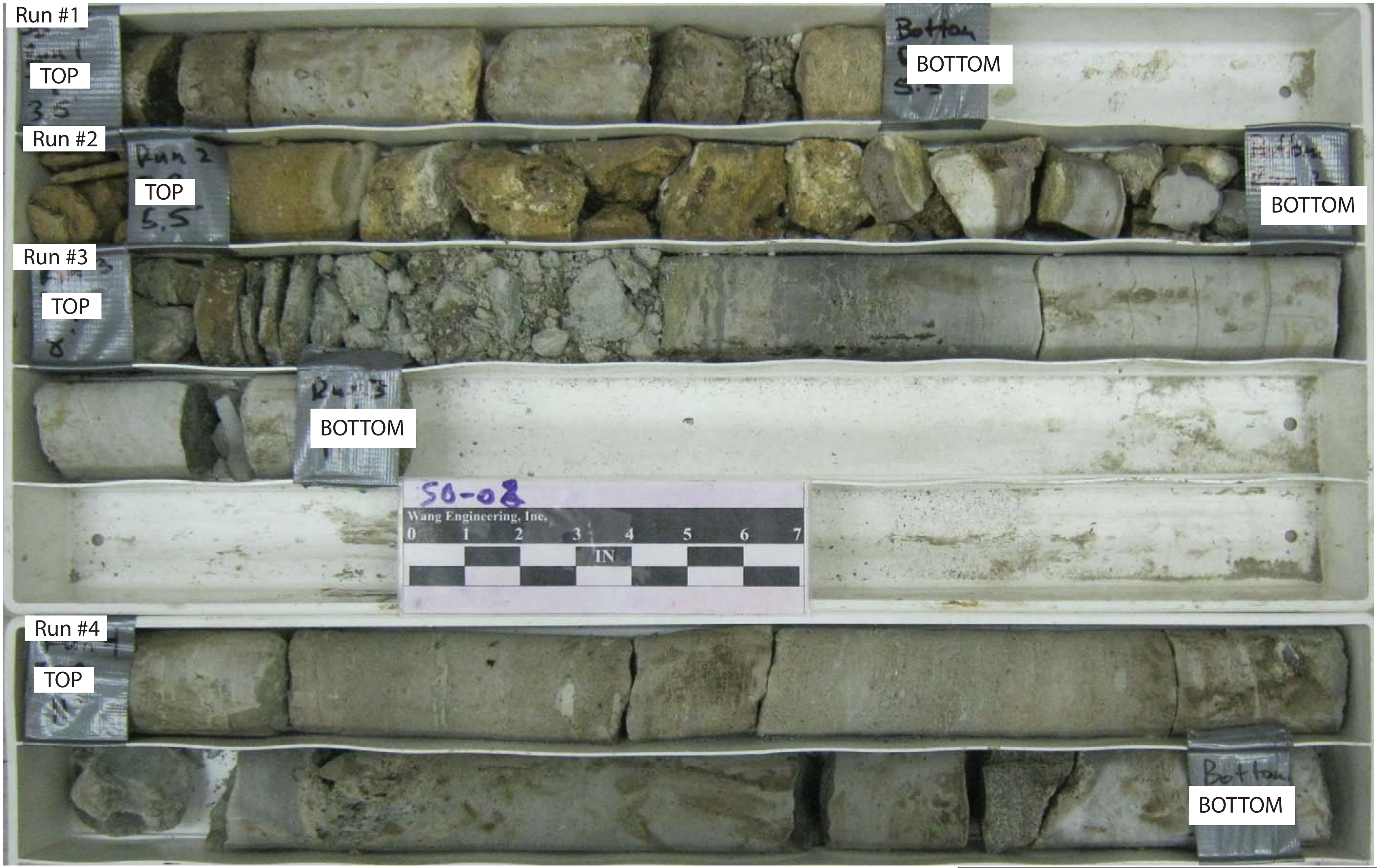
GENERAL NOTES

Begin Drilling **10-29-2012** Complete Drilling **10-29-2012**
Drilling Contractor **Wang** Drill Rig **D-50 TMR**
Driller **K&K** Logger **B. Wilson** Checked by **C. Marin**
Drilling Method **3.25-inch IDA HSA; boring backfilled upon completion**

WATER LEVEL DATA

While Drilling ▽ **DRY**
At Completion of Drilling ▼ **5.00 ft**
Time After Drilling **NA**
Depth to Water ▽ **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.



Boring SB-08:
Run #1, 3.5' to 5.5', RECOVERY = 67%, RQD = 17%
Run #2, 5.5' to 8.0', RECOVERY = 67%, RQD = 0%
Run #3, 8.0' to 11.0', RECOVERY = 83%, RQD = 34%
Run #4, 11.0' to 15.5', RECOVERY = 85%, RQD = 53%

BEDROCK CORE: IL ROUTE 178 OVER ILLINOIS RIVER LaSALLE COUNTY, ILLINOIS		
SCALE: GRAPHICAL	SB-08	DRAWN BY: C. MARIN CHECKED BY: L.IORDACHE
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FOR PARSONS BRINCKERHOFF		107-08-01

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BORING LOG SB-09

WEI Job No.: 107-08-01

Client **Parsons Brinckerhoff**
Project **IL 178 Over Illinois River**
Location **SE 1/4 Section 17, T 33N, R 2E of 3rd PM**

Datum: NAVD88
Elevation: 449.40 ft
North: 1697512.28 ft
East: 798348.61 ft
Station: 36+98.48
Offset: 134.06 LT

Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
	15-inch thick, dark brown SILTY CLAY --TOPSOIL--					
448.2	Medium stiff to stiff, brown and gray CLAY LOAM, trace gravel		1	4 5 4	1.50 P	17
		5	2	3 3 3	0.75 P	23
443.4	Very dense, brown and gray, weathered DOLOSTONE fragments --WEATHERED BEDROCK-- --AUGER REFUSAL--		3	18 50/4		12
441.9	Boring terminated at 7.50 ft					
		10				
		15				
		20				
		25				

GENERAL NOTES

Begin Drilling **10-29-2012** Complete Drilling **10-29-2012**
Drilling Contractor **Wang** Drill Rig **D-50 TMR**
Driller **K&K** Logger **B. Wilson** Checked by **C. Marin**
Drilling Method **3.25-inch IDA HSA; boring backfilled upon completion**

WATER LEVEL DATA

While Drilling **DRY**
At Completion of Drilling **DRY**
Time After Drilling **NA**
Depth to Water **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

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BORING LOG SB-10

WEI Job No.: 107-08-01

Client **Parsons Brinckerhoff**
Project **IL 178 Over Illinois River**
Location **SE 1/4 Section 17, T 33N, R 2E of 3rd PM**

Datum: NAVD88
Elevation: 448.90 ft
North: 1697305.17 ft
East: 798337.01 ft
Station: 39+03.18
Offset: 133.71 LT

Profile	Elevation (ft)	SOIL AND ROCK DESCRIPTION	Depth (ft)	Sample Type recovery	Sample No.	SPT Values (blw/6 in)	Qu (tsf)	Moisture Content (%)
		15-inch thick, dark brown SILTY CLAY --TOPSOIL--						
	447.4	Stiff, brown and gray SILTY CLAY, trace sand seams			1	3 3 6	1.50 P	18
	445.9	Medium stiff, dark brown CLAY LOAM	5		2	2 3 3	0.50 P	23
	442.7	Very dense, brown SANDY GRAVEL, some weathered DOLOSTONE fragments --WEATHERED BEDROCK--			3	2 3 49		12
					4	8 55/5		22
	438.9	--AUGER REFUSAL--	10					
		Boring terminated at 10.00 ft						
			15					
			20					
			25					

GENERAL NOTES

Begin Drilling **10-29-2012** Complete Drilling **10-29-2012**
Drilling Contractor **Wang** Drill Rig **D-50 TMR**
Driller **K&K** Logger **B. Wilson** Checked by **C. Marin**
Drilling Method **3.25-inch IDA HSA; boring backfilled upon completion**

WATER LEVEL DATA

While Drilling ▽ **7.50 ft**
At Completion of Drilling ▼ **7.00 ft**
Time After Drilling **NA**
Depth to Water ▼ **NA**

The stratification lines represent the approximate boundary between soil types; the actual transition may be gradual.

APPENDIX B

LABORATORY TEST DATA

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/21/13-1/28/13
Tested By:	jsc
Checked By:	mpd

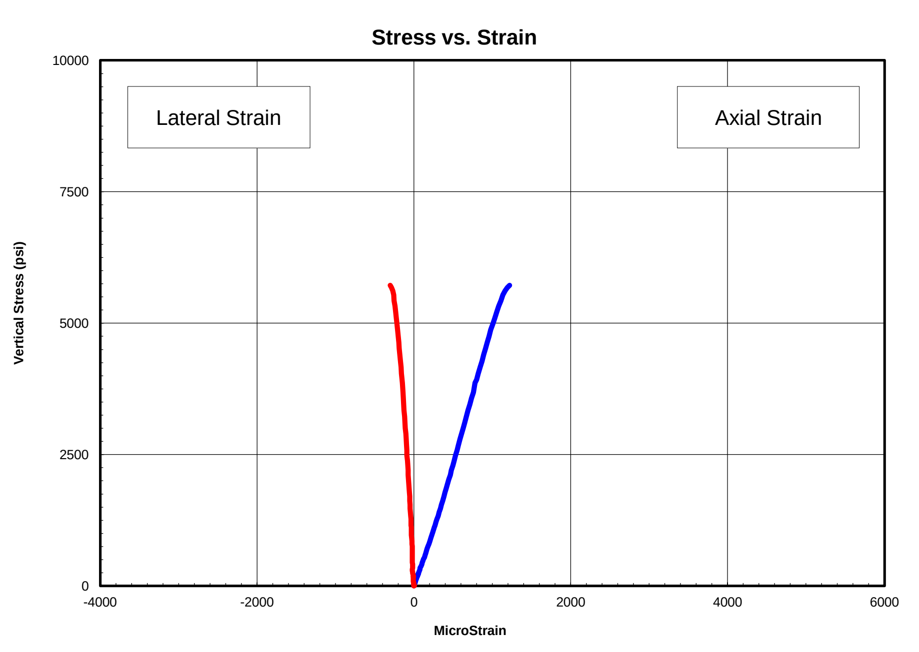
Unit Weight, Porosity & Specific Gravity of Rock - ISRM Method 2 & 3

Boring ID	Sample ID	Depth, ft.	Average Moisture Content, %	Average Dry Unit Weight, pcf	Average Bulk Specific Gravity	Average Porosity
SB-05	Run 2	7.0-11.0	2.82	141	2.32	0.12
SB-05	Run 3	11.0-18.0	0.63	156	2.52	0.09
SB-06	Run 1	14.0-19.0	0.12	164	2.62	0.06
SB-06	Run 4	29.0-34.0	0.29	151	2.43	0.11
SB-06	Run 5	34.0-39.0	0.24	167	2.73	0.03
SB-06	Run 8	49.0-54.0	0.25	144	2.41	0.11
SB-07	Run 1	16.0-21.0	3.37	153	2.45	0.11
SB-07	Run 4	31.0-36.0	1.00	148	2.39	0.13
SB-07	Run 6	41.0-46.0	1.26	153	2.47	0.11
SB-07	Run 8	51.0-56.0	6.50	142	2.32	0.15
SB-08	Run 1	3.5-5.5	0.18	168	2.71	0.03
SB-08	Run 4	11.0-15.5	0.38	140	2.29	0.12

Notes: Results are based on the average of three saw-cut test specimens except as noted.
Unit weight obtained by digital caliper measurement for all samples tested by ISRM Method 2.
Pore Volume obtained by water saturation.
Bulk Specific Gravity obtained by buoyancy technique.
SB-06 Run 5 and SB-07 Run 8 based on average of two specimens.
SB-06 Run 1 tested by ISRM Method 3 based on an average of six irregular pieces.
SB-07 Run 1 tested by ISRM Method 3 based on an average of five irregular pieces.

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/24/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	SB-05
Sample ID:	Run 3
Depth, ft:	11.0-18.0
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 5,727 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
600-2100	4,720,000	0.16
2100-3600	5,380,000	0.23
3600-5200	5,130,000	0.31

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

GeoTesting EXPRESS

Client:	Wang Engineering	Test Date:	1/23/2013
Project Name:	IL 178 over the Illinois River	Tested By:	daa
Project Location:	IL	Checked By:	mpd
GTX #:	300122		
Boring ID:	SB-05		
Sample ID:	Run 3		
Depth:	11.0-18.0 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.02	4.02	4.02
Specimen Diameter, in:	2.05	2.05	2.05
Specimen Mass, g:	541.86		
Bulk Density, lb/ft^3	155		
Length to Diameter Ratio:	2.0	**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? NO

Maximum difference must be < 0.020 in.
Straightness Tolerance Met? **NO**

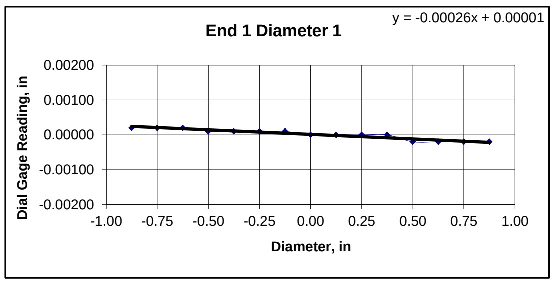
END FLATNESS AND PARALLELISM (Procedure FP1)

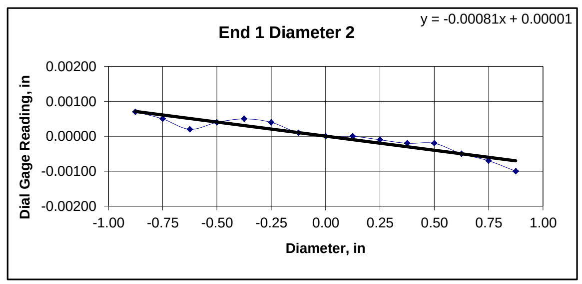
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00020	0.00020	0.00020	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00020	-0.00020	-0.00020	-0.00020
Diameter 2, in (rotated 90°)	0.00070	0.00050	0.00020	0.00040	0.00050	0.00040	0.00010	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00050	-0.00070	-0.00100

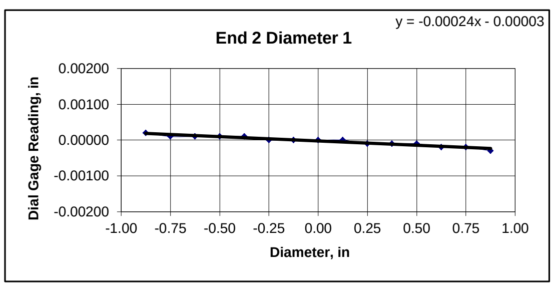
Difference between max and min readings, in:
0° = 0.00040 90° = 0.00170

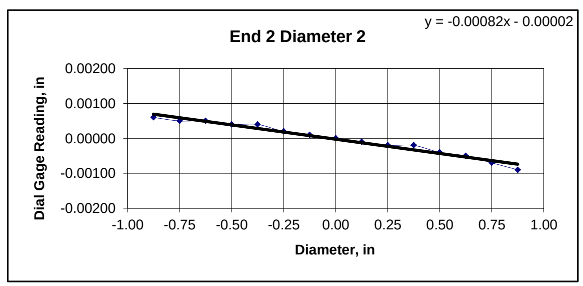
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00020	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00020	-0.00020	-0.00030
Diameter 2, in (rotated 90°)	0.00060	0.00050	0.00050	0.00040	0.00040	0.00020	0.00010	0.00000	-0.00010	-0.00020	-0.00020	-0.00040	-0.00050	-0.00070	-0.00090

Difference between max and min readings, in:
0° = 0.0005 90° = 0.0015
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00085
Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:		
	Slope of Best Fit Line	-0.00026
	Angle of Best Fit Line:	-0.01490
End 2:		
	Slope of Best Fit Line	-0.00024
	Angle of Best Fit Line:	-0.01375
Maximum Angular Difference:		0.00115

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:		
	Slope of Best Fit Line	-0.00081
	Angle of Best Fit Line:	-0.04641
End 2:		
	Slope of Best Fit Line	-0.00082
	Angle of Best Fit Line:	-0.04698
Maximum Angular Difference:		0.00057

Parallelism Tolerance Met? **YES**
Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00040	2.050	0.00020	0.011	YES
Diameter 2, in (rotated 90°)	0.00170	2.050	0.00083	0.048	YES
END 2					
Diameter 1, in	0.00050	2.050	0.00024	0.014	YES
Diameter 2, in (rotated 90°)	0.00150	2.050	0.00073	0.042	YES

Maximum angle of departure must be $\leq$ 0.25°
Perpendicularity Tolerance Met? **YES**

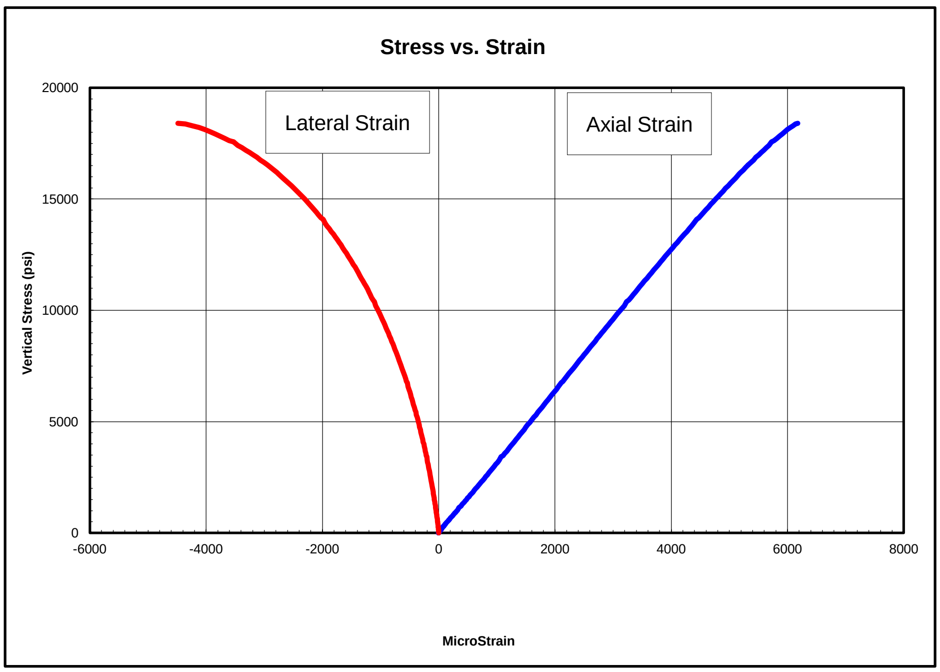
Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/24/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	SB-05
Sample ID:	Run 3
Depth, ft:	11.0-18.0

After cutting and grinding

After break

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/18/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	SB-06
Sample ID:	Run 3
Depth, ft:	24.0-29.0
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 18,407 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1800-6700	3,220,000	0.29
6700-11700	3,200,000	---
11700-16600	2,910,000	---

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Wang Engineering	Test Date:	1/17/2013
Project Name:	IL 178 over the Illinois River	Tested By:	daa
Project Location:	IL	Checked By:	mpd
GTX #:	300122		
Boring ID:	SB-06		
Sample ID:	Run 3		
Depth:	24.0-29.0 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.03	4.03	4.03
Specimen Diameter, in:	2.05	2.05	2.05
Specimen Mass, g:	535.16		
Bulk Density, lb/ft^3	153		
Length to Diameter Ratio:	2.0		

Length to Diameter Ratio Tolerance Met? YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? YES

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00090	0.00090	0.00070	0.00050	0.00030	0.00020	0.00020	0.00000	-0.00010	-0.00020	-0.00040	-0.00050	-0.00070	-0.00070	-0.00080
Diameter 2, in (rotated 90°)	-0.00020	-0.00020	-0.00020	-0.00020	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010

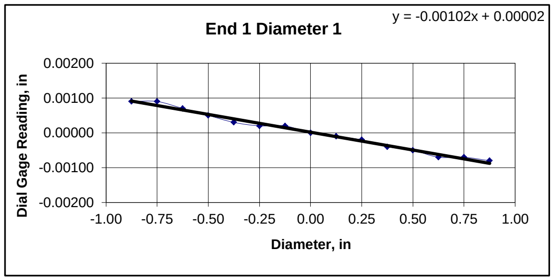
Difference between max and min readings, in:
0° = 0.00170 90° = 0.00030

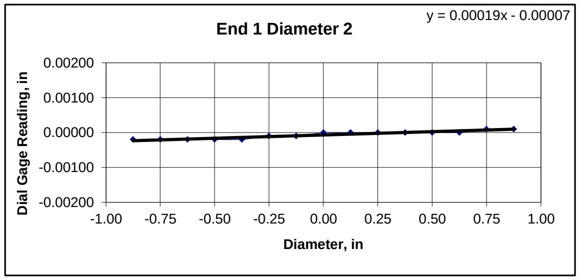
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00090	0.00080	0.00070	0.00060	0.00040	0.00030	0.00020	0.00000	0.00000	-0.00010	-0.00030	-0.00050	-0.00060	-0.00070	-0.00090
Diameter 2, in (rotated 90°)	-0.00020	-0.00020	-0.00020	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010

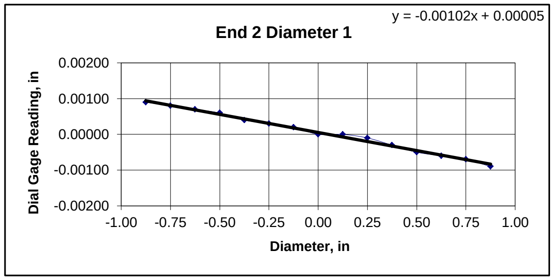
Difference between max and min readings, in:
0° = 0.0018 90° = 0.0003

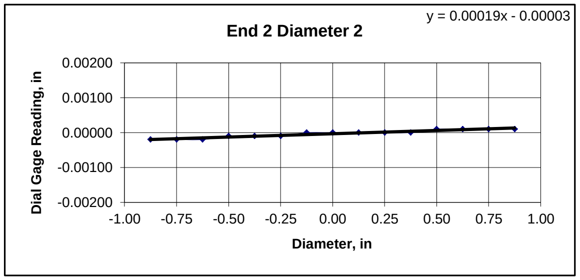
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00090

Flatness Tolerance Met? YES









DIAMETER 1

End 1:
Slope of Best Fit Line -0.00102
Angle of Best Fit Line: -0.05844

End 2:
Slope of Best Fit Line -0.00102
Angle of Best Fit Line: -0.05844

Maximum Angular Difference: 0.00000

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00019
Angle of Best Fit Line: 0.01089

End 2:
Slope of Best Fit Line 0.00019
Angle of Best Fit Line: 0.01089

Maximum Angular Difference: 0.00000

Parallelism Tolerance Met? YES
Spherically Seated

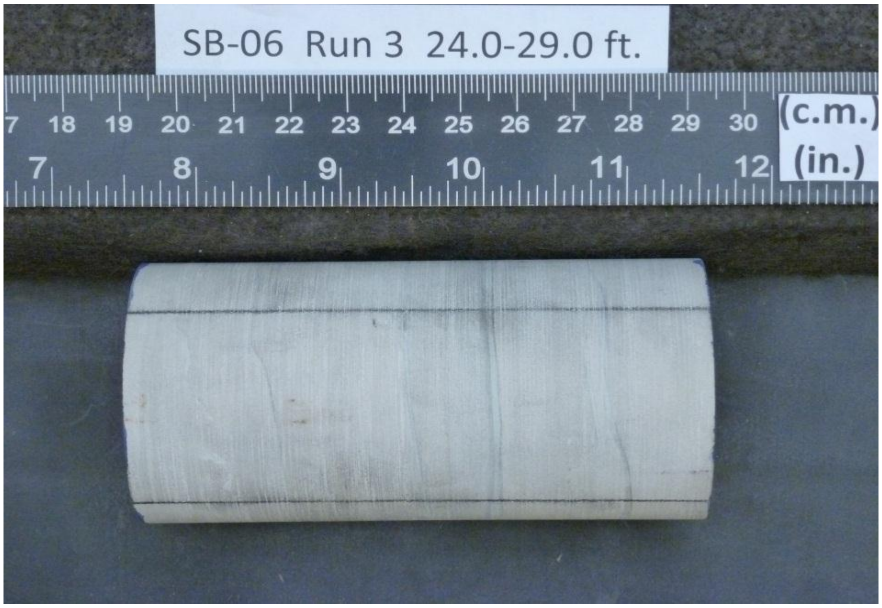
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00170	2.050	0.00083	0.048	YES
Diameter 2, in (rotated 90°)	0.00030	2.050	0.00015	0.008	YES
END 2					
Diameter 1, in	0.00180	2.050	0.00088	0.050	YES
Diameter 2, in (rotated 90°)	0.00030	2.050	0.00015	0.008	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/18/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	SB-06
Sample ID:	Run 3
Depth, ft:	24.0-29.0

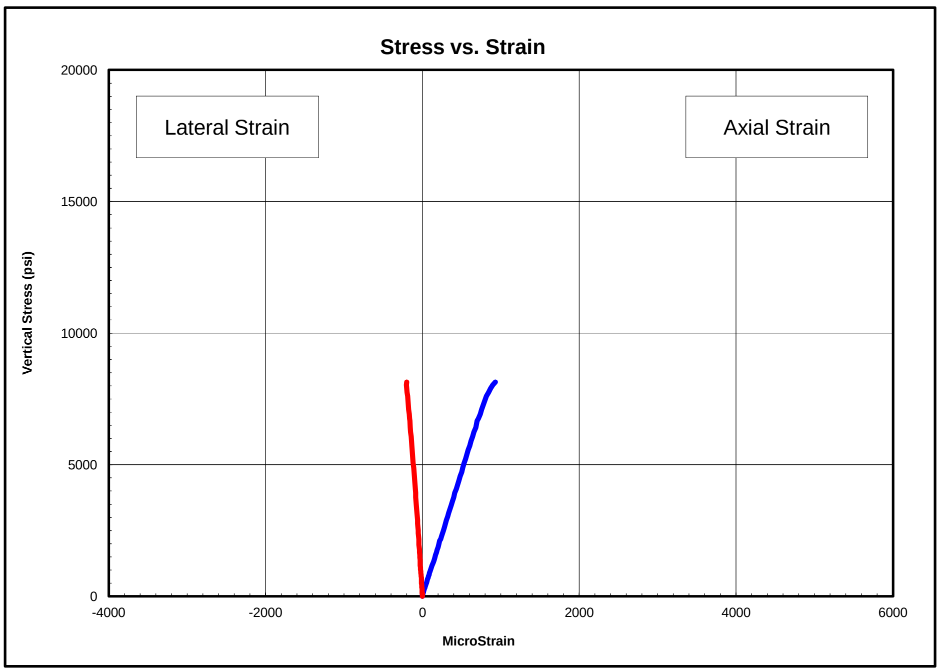


After cutting and grinding

After break

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/24/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	SB-06
Sample ID:	Run 6
Depth, ft:	39.0-44.0
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 8,141 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
800-3000	9,260,000	0.20
3000-5200	9,580,000	0.24
5200-7300	9,160,000	0.26

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Wang Engineering	Test Date:	1/23/2013
Project Name:	IL 178 over the Illinois River	Tested By:	daa
Project Location:	IL	Checked By:	mpd
GTX #:	300122		
Boring ID:	SB-06		
Sample ID:	Run 6		
Depth:	39.0-44.0 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.08	4.08	4.08
Specimen Diameter, in:	2.05	2.06	2.06
Specimen Mass, g:	574.25		
Bulk Density, lb/ft^3	161		
Length to Diameter Ratio:	2.0	**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? NO

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? NO

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00040	0.00030	0.00030	0.00020	0.00020	0.00010	0.00010	0.00000	-0.00010	-0.00010	-0.00030	-0.00030	-0.00040	-0.00050	-0.00060
Diameter 2, in (rotated 90°)	-0.00060	-0.00050	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00020	0.00020	0.00030	0.00030	0.00040

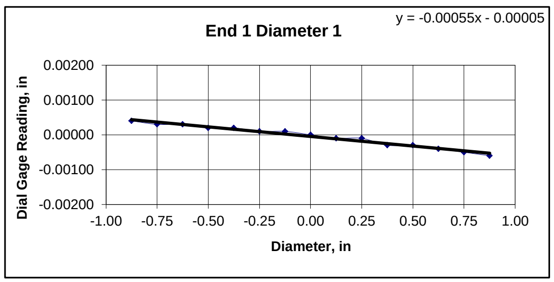
Difference between max and min readings, in:
0° = 0.00100 90° = 0.00100

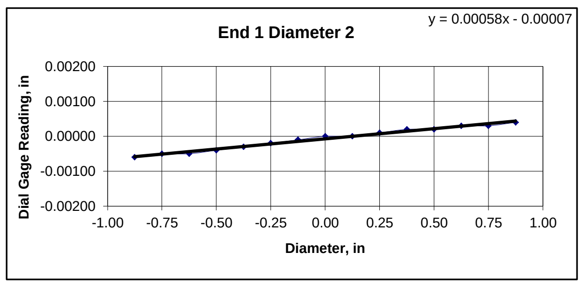
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00040	0.00040	0.00030	0.00030	0.00020	0.00010	0.00010	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00040	-0.00050	-0.00060
Diameter 2, in (rotated 90°)	-0.00050	-0.00050	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00020	0.00030	0.00040	0.00040	0.00050

Difference between max and min readings, in:
0° = 0.001 90° = 0.001

Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00050

Flatness Tolerance Met? YES





DIAMETER 1

End 1:

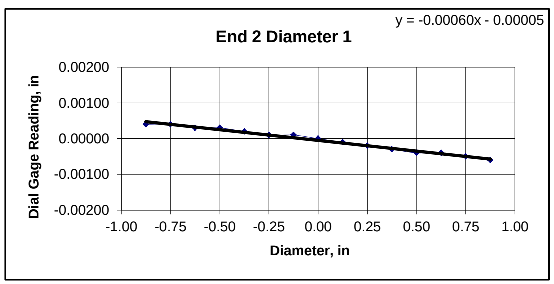
Slope of Best Fit Line	-0.00055
Angle of Best Fit Line:	-0.03151

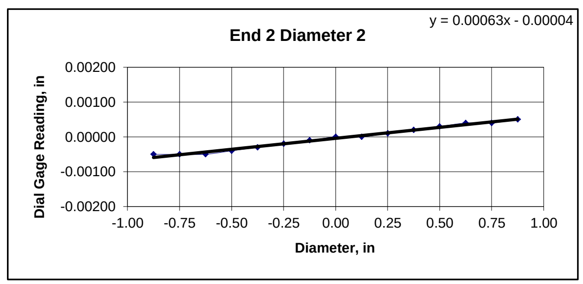
End 2:

Slope of Best Fit Line	-0.00060
Angle of Best Fit Line:	-0.03438

Maximum Angular Difference: 0.00286

Parallelism Tolerance Met? YES
Spherically Seated





DIAMETER 2

End 1:

Slope of Best Fit Line	0.00058
Angle of Best Fit Line:	0.03323

End 2:

Slope of Best Fit Line	0.00063
Angle of Best Fit Line:	0.03610

Maximum Angular Difference: 0.00286

Parallelism Tolerance Met? YES
Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

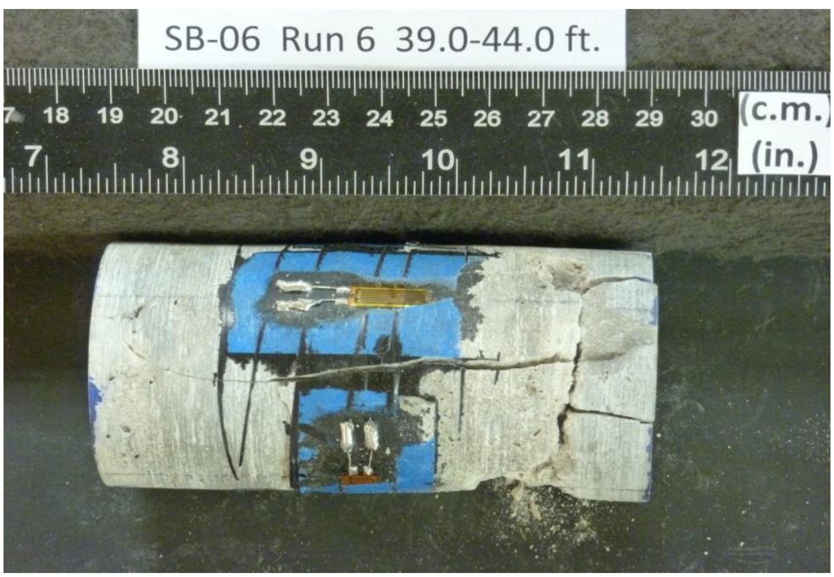
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00100	2.055	0.00049	0.028	YES
Diameter 2, in (rotated 90°)	0.00100	2.055	0.00049	0.028	YES
END 2					
Diameter 1, in	0.00100	2.055	0.00049	0.028	YES
Diameter 2, in (rotated 90°)	0.00100	2.055	0.00049	0.028	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/24/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	SB-06
Sample ID:	Run 6
Depth, ft:	39.0-44.0

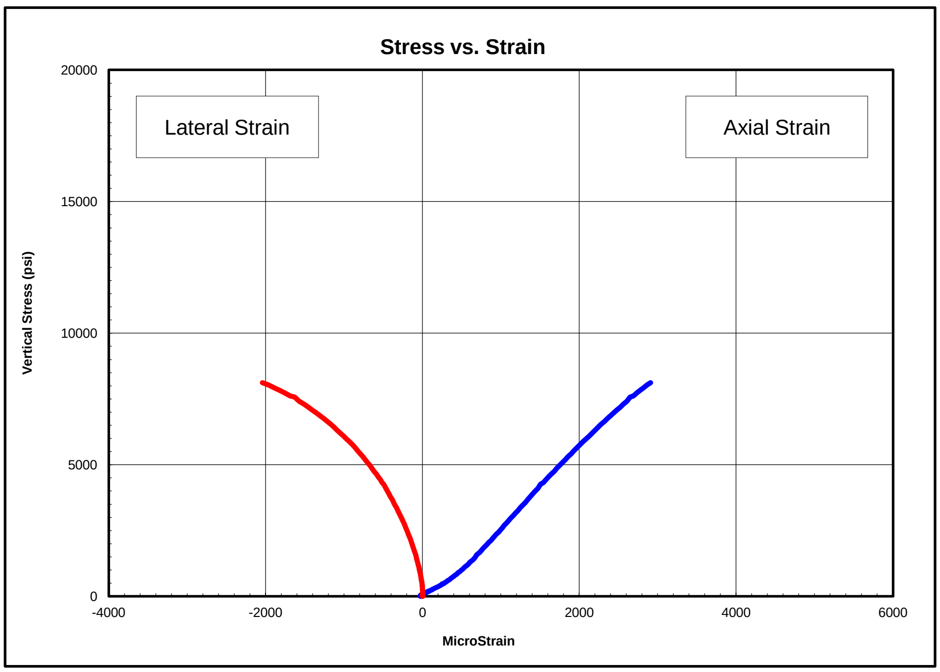
After cutting and grinding



After break

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/18/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	SB-06
Sample ID:	Run 8
Depth, ft:	49.0-54.0
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 8,144 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
800-3000	3,080,000	0.34
3000-5100	3,220,000	---
5100-7300	2,840,000	---

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Wang Engineering		Test Date:	1/17/2013
Project Name:	IL 178 over the Illinois River		Tested By:	daa
Project Location:	IL		Checked By:	mpd
GTX #:	300122			
Boring ID:	SB-06			
Sample ID:	Run 8			
Depth:	49.0-54.0	ft		
Visual Description:	See photographs			

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.18	4.18	4.18
Specimen Diameter, in:	2.05	2.05	2.05
Specimen Mass, g:	537.26		
Bulk Density, lb/ft^3	148		
Length to Diameter Ratio:	2.0	**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap $\leq$ 0.02 in.? NO

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? NO

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00080	0.00060	0.00060	0.00040	0.00030	0.00020	0.00010	0.00000	0.00000	-0.00020	-0.00040	-0.00050	-0.00060	-0.00080	-0.00090
Diameter 2, in (rotated 90°)	-0.00060	-0.00060	-0.00050	-0.00050	-0.00040	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00020	0.00040	0.00050	0.00050	0.00060

Difference between max and min readings, in:

0° = 0.00170 90° = 0.00120

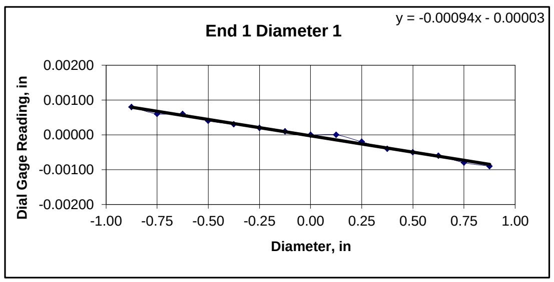
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00080	0.00080	0.00070	0.00060	0.00040	0.00020	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00050	-0.00060	-0.00080	-0.00090
Diameter 2, in (rotated 90°)	-0.00060	-0.00060	-0.00050	-0.00040	-0.00020	-0.00010	-0.00010	0.00000	0.00010	0.00020	0.00030	0.00040	0.00050	0.00060	0.00060

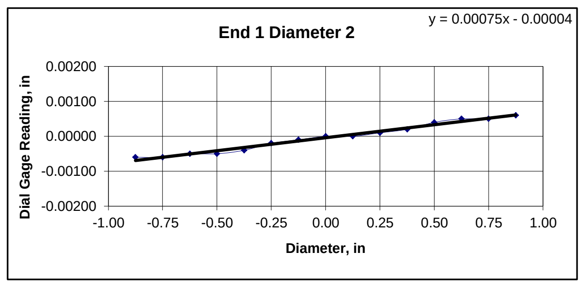
Difference between max and min readings, in:

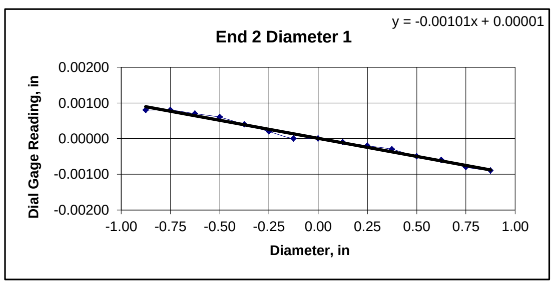
0° = 0.0017 90° = 0.0012

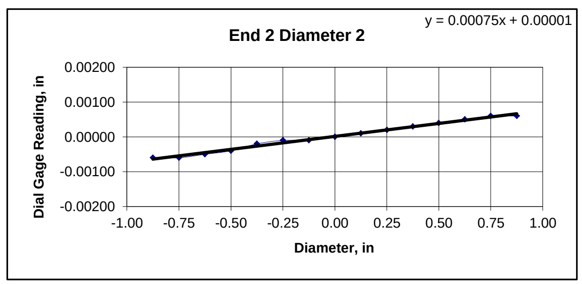
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00085

Flatness Tolerance Met? YES









DIAMETER 1

End 1:		
	Slope of Best Fit Line	-0.00094
	Angle of Best Fit Line:	-0.05386
End 2:		
	Slope of Best Fit Line	-0.00101
	Angle of Best Fit Line:	-0.05787
Maximum Angular Difference:		0.00401

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:		
	Slope of Best Fit Line	0.00075
	Angle of Best Fit Line:	0.04297
End 2:		
	Slope of Best Fit Line	0.00075
	Angle of Best Fit Line:	0.04297
Maximum Angular Difference:		0.00000

Parallelism Tolerance Met? YES
Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00170	2.050	0.00083	0.048	YES
Diameter 2, in (rotated 90°)	0.00120	2.050	0.00059	0.034	YES
END 2					
Diameter 1, in	0.00170	2.050	0.00083	0.048	YES
Diameter 2, in (rotated 90°)	0.00120	2.050	0.00059	0.034	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? YES

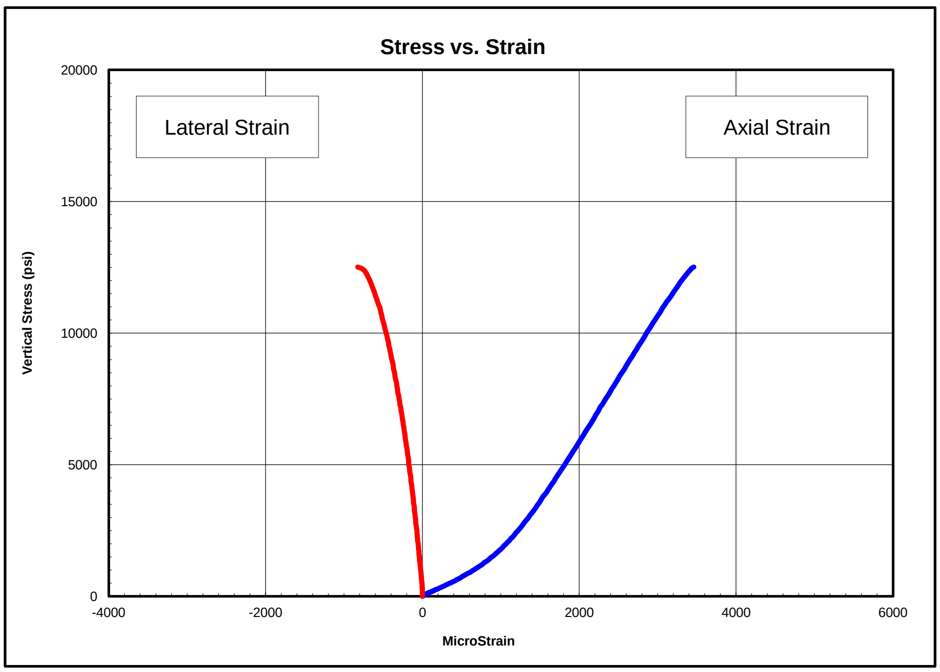
Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/18/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	SB-06
Sample ID:	Run 8
Depth, ft:	49.0-54.0

After cutting and grinding

After break

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/21/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	SB-07
Sample ID:	Run 1
Depth, ft:	16.0-21.0
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 12,510 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1300-4600	3,480,000	0.13
4600-7900	4,770,000	0.24
7900-11300	4,730,000	0.36

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Wang Engineering	Test Date:	1/17/2013
Project Name:	IL 178 over the Illinois River	Tested By:	daa
Project Location:	IL	Checked By:	mpd
GTX #:	300122		
Boring ID:	SB-07		
Sample ID:	Run 1		
Depth:	16.0-21.0 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543

BULK DENSITY

	1	2	Average
Specimen Length, in:	3.88	3.88	3.88
Specimen Diameter, in:	2.02	2.03	2.03
Specimen Mass, g:	444.95		
Bulk Density, lb/ft^3	135		
Length to Diameter Ratio:	1.9	**Length to Diameter Ratio Tolerance Met?**	NO

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? YES

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00050	0.00050	0.00040	0.00040	0.00030	0.00020	0.00010	0.00000	0.00000	-0.00010	-0.00030	-0.00040	-0.00060	-0.00070	-0.00080
Diameter 2, in (rotated 90°)	-0.00050	-0.00050	-0.00040	-0.00040	-0.00030	-0.00020	-0.00020	0.00000	0.00000	0.00000	0.00010	0.00010	0.00020	0.00030	0.00040

Difference between max and min readings, in:

0° = 0.00130 90° = 0.00090

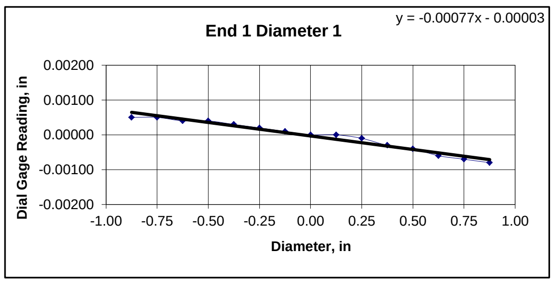
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00050	0.00050	0.00040	0.00040	0.00030	0.00030	0.00010	0.00000	0.00000	-0.00010	-0.00030	-0.00040	-0.00050	-0.00070	-0.00090
Diameter 2, in (rotated 90°)	-0.00060	-0.00060	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00010	0.00020	0.00020	0.00030	0.00040

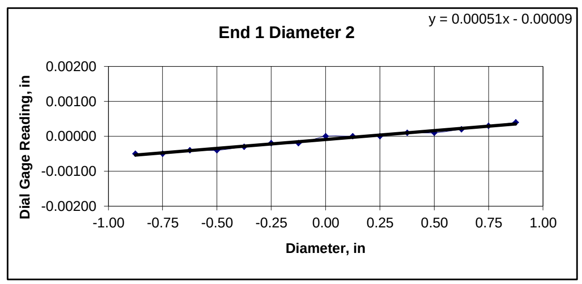
Difference between max and min readings, in:

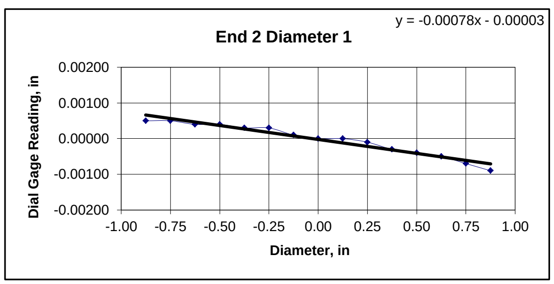
0° = 0.0014 90° = 0.001

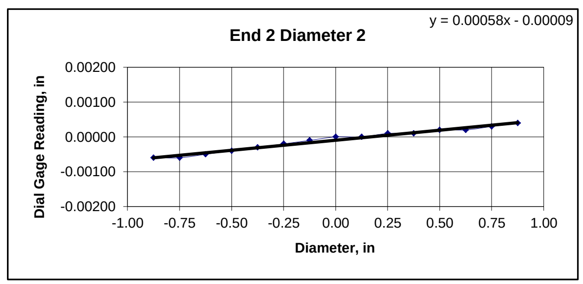
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00070

Flatness Tolerance Met? YES









DIAMETER 1

End 1:

Slope of Best Fit Line	-0.00077
Angle of Best Fit Line:	-0.04412

End 2:

Slope of Best Fit Line	-0.00078
Angle of Best Fit Line:	-0.04469

Maximum Angular Difference: 0.00057

Parallelism Tolerance Met? YES

Spherically Seated

DIAMETER 2

End 1:

Slope of Best Fit Line	0.00051
Angle of Best Fit Line:	0.02922

End 2:

Slope of Best Fit Line	0.00058
Angle of Best Fit Line:	0.03323

Maximum Angular Difference: 0.00401

Parallelism Tolerance Met? YES

Spherically Seated

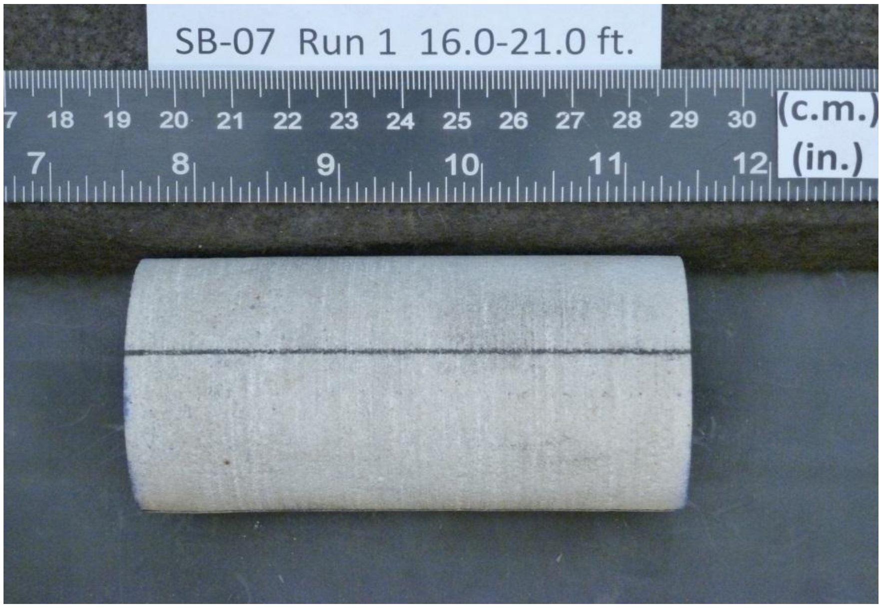
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00130	2.025	0.00064	0.037	YES
Diameter 2, in (rotated 90°)	0.00090	2.025	0.00044	0.025	YES
END 2					
Diameter 1, in	0.00140	2.025	0.00069	0.040	YES
Diameter 2, in (rotated 90°)	0.00100	2.025	0.00049	0.028	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/21/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	SB-07
Sample ID:	Run 1
Depth, ft:	16.0-21.0

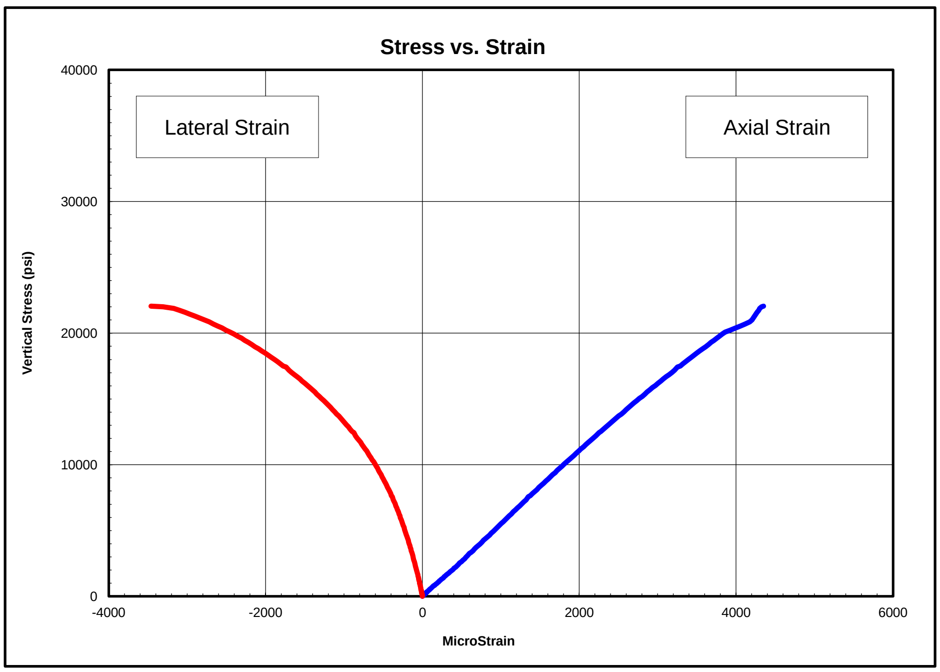


After cutting and grinding

After break

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/18/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	SB-07
Sample ID:	Run 3
Depth, ft:	26.0-31.0
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 22,057 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
2200-8100	5,700,000	0.33
8100-14000	5,350,000	---
14000-19900	4,690,000	---

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

GeoTesting EXPRESS

Client:	Wang Engineering	Test Date:	1/17/2013
Project Name:	IL 178 over the Illinois River	Tested By:	daa
Project Location:	IL	Checked By:	mpd
GTX #:	300122		
Boring ID:	SB-07		
Sample ID:	Run 3		
Depth:	26.0-31.0 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.28	4.29	4.29
Specimen Diameter, in:	2.05	2.04	2.05
Specimen Mass, g:	574.72		
Bulk Density, lb/ft^3	155		
Length to Diameter Ratio:	2.1		

Length to Diameter Ratio Tolerance Met? YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? YES

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00040	-0.00040	-0.00040	-0.00030	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00010	0.00020	0.00020	0.00030	0.00040	0.00050
Diameter 2, in (rotated 90°)	0.00080	0.00070	0.00060	0.00050	0.00040	0.00030	0.00010	0.00000	-0.00020	-0.00040	-0.00050	-0.00060	-0.00070	-0.00080	-0.00100

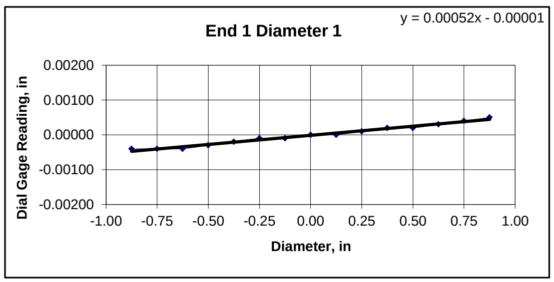
Difference between max and min readings, in:
0° = 0.00090 90° = 0.00180

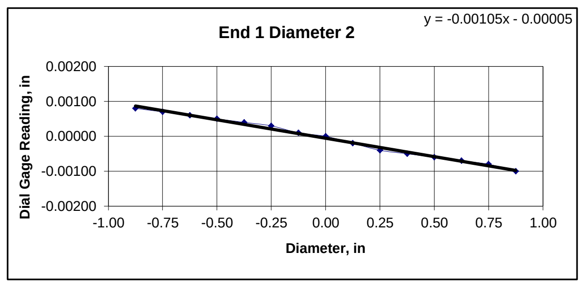
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00060	-0.00060	-0.00050	-0.00050	-0.00040	-0.00030	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	0.00070	0.00070	0.00050	0.00050	0.00030	0.00030	0.00010	0.00000	-0.00020	-0.00040	-0.00050	-0.00070	-0.00090	-0.00100	-0.00110

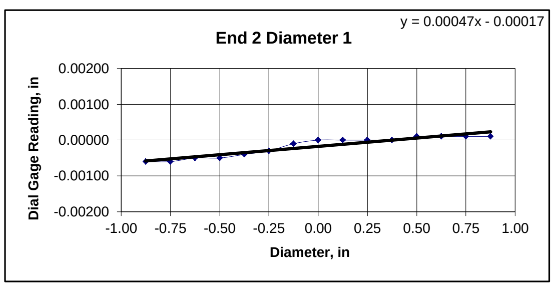
Difference between max and min readings, in:
0° = 0.0007 90° = 0.0018

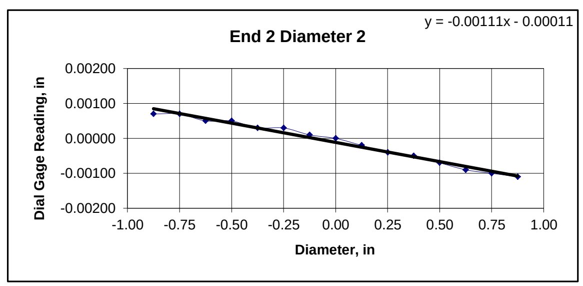
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00090

Flatness Tolerance Met? YES









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00052
Angle of Best Fit Line: 0.02979

End 2:
Slope of Best Fit Line 0.00047
Angle of Best Fit Line: 0.02693

Maximum Angular Difference: 0.00286

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line -0.00105
Angle of Best Fit Line: -0.06016

End 2:
Slope of Best Fit Line -0.00111
Angle of Best Fit Line: -0.06360

Maximum Angular Difference: 0.00344

Parallelism Tolerance Met? YES
Spherically Seated

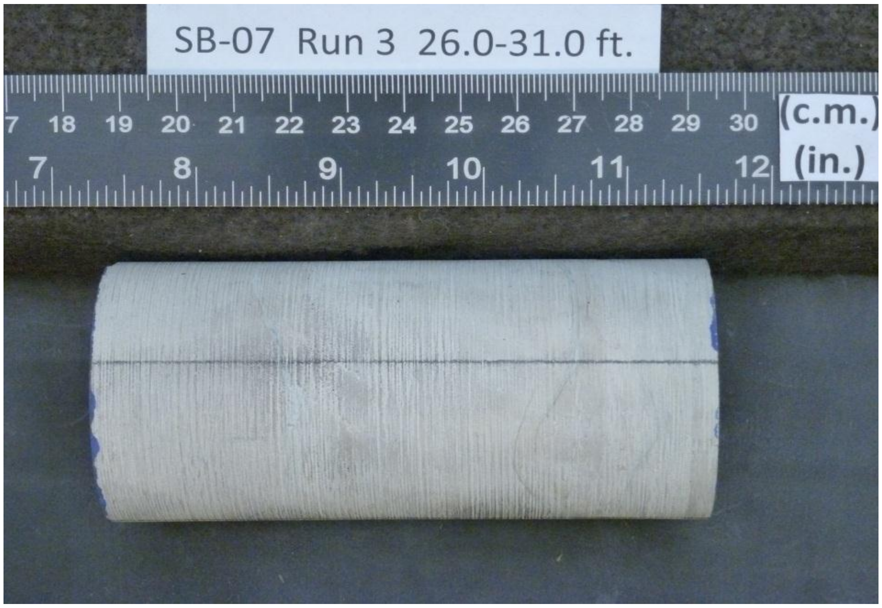
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00090	2.045	0.00044	0.025	YES
Diameter 2, in (rotated 90°)	0.00180	2.045	0.00088	0.050	YES
END 2					
Diameter 1, in	0.00070	2.045	0.00034	0.020	YES
Diameter 2, in (rotated 90°)	0.00180	2.045	0.00088	0.050	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/18/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	SB-07
Sample ID:	Run 3
Depth, ft:	26.0-31.0

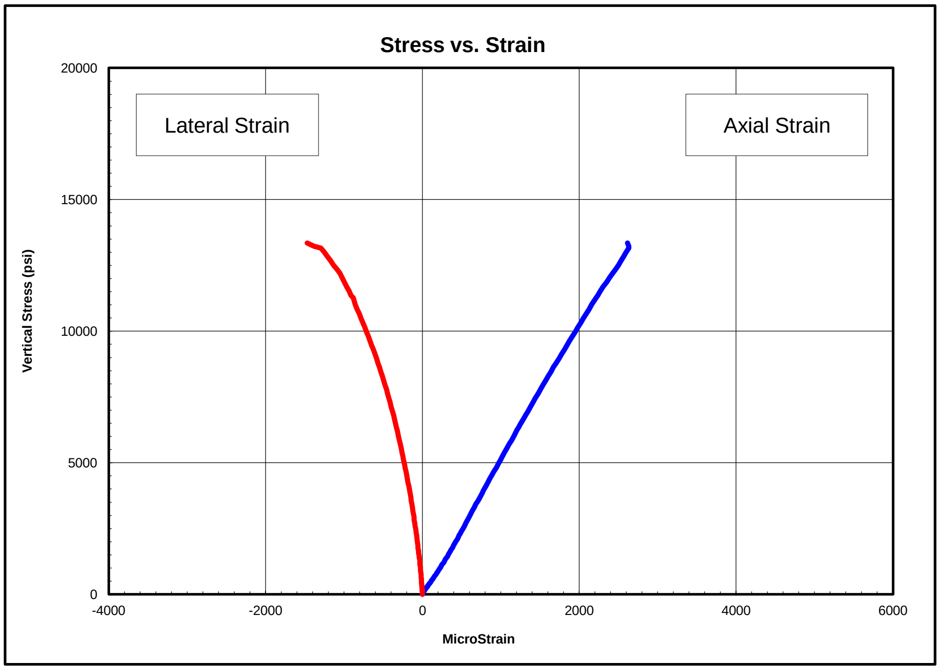


After cutting and grinding

After break

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/18/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	SB-07
Sample ID:	Run 5
Depth, ft:	36.0-41.0
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 13,573 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1400-5000	5,450,000	0.28
5000-8600	5,210,000	0.45
8600-12200	4,780,000	---

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

GeoTesting EXPRESS

Client:	Wang Engineering	Test Date:	1/17/2013
Project Name:	IL 178 over the Illinois River	Tested By:	daa
Project Location:	IL	Checked By:	mpd
GTX #:	300122		
Boring ID:	SB-07		
Sample ID:	Run 5		
Depth:	36.0-41.0 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.38	4.39	4.39
Specimen Diameter, in:	2.05	2.05	2.05
Specimen Mass, g:	585.3		
Bulk Density, lb/ft^3	154		
Length to Diameter Ratio:	2.1		

Length to Diameter Ratio Tolerance Met? YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? YES

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00100	-0.00090	-0.00080	-0.00070	-0.00050	-0.00040	-0.00020	0.00000	0.00000	0.00020	0.00030	0.00040	0.00060	0.00070	0.00070
Diameter 2, in (rotated 90°)	0.00040	0.00040	0.00040	0.00040	0.00040	0.00030	0.00020	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00050	-0.00050

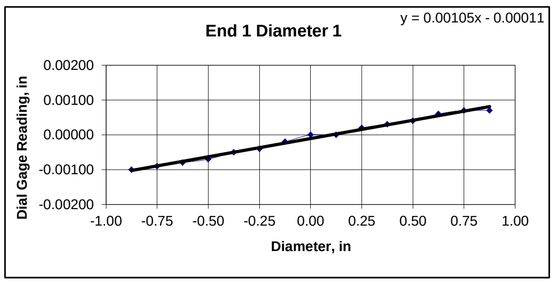
Difference between max and min readings, in:
0° = 0.00170 90° = 0.00090

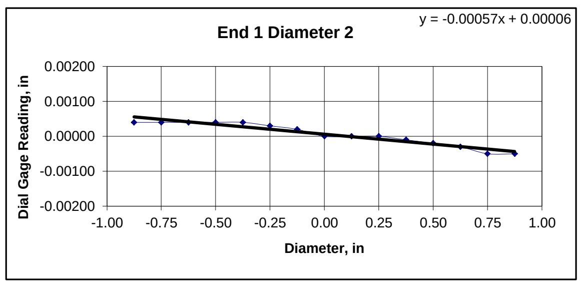
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00080	-0.00080	-0.00080	-0.00060	-0.00050	-0.00030	-0.00010	0.00000	0.00010	0.00020	0.00030	0.00040	0.00060	0.00070	0.00080
Diameter 2, in (rotated 90°)	0.00040	0.00030	0.00020	0.00020	0.00010	0.00010	0.00000	0.00000	0.00000	-0.00010	-0.00030	-0.00030	-0.00040	-0.00050	-0.00060

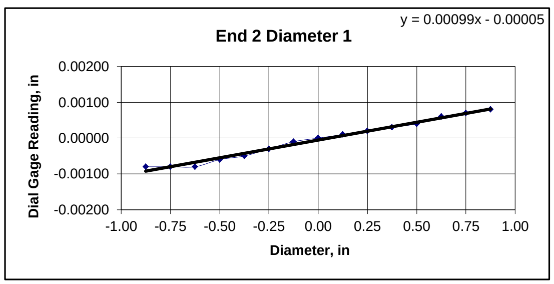
Difference between max and min readings, in:
0° = 0.0016 90° = 0.001

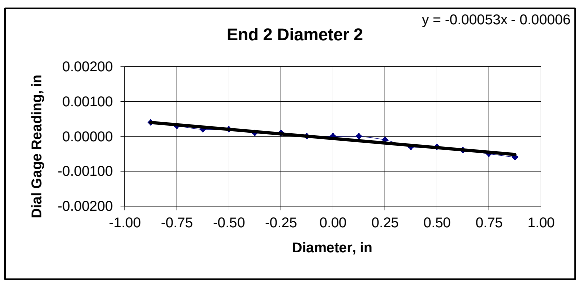
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00085

Flatness Tolerance Met? YES









DIAMETER 1

End 1:		
	Slope of Best Fit Line	0.00105
	Angle of Best Fit Line:	0.06016
End 2:		
	Slope of Best Fit Line	0.00099
	Angle of Best Fit Line:	0.05672
Maximum Angular Difference:		0.00344

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:		
	Slope of Best Fit Line	-0.00057
	Angle of Best Fit Line:	-0.03266
End 2:		
	Slope of Best Fit Line	-0.00053
	Angle of Best Fit Line:	-0.03037
Maximum Angular Difference:		0.00229

Parallelism Tolerance Met? YES
Spherically Seated

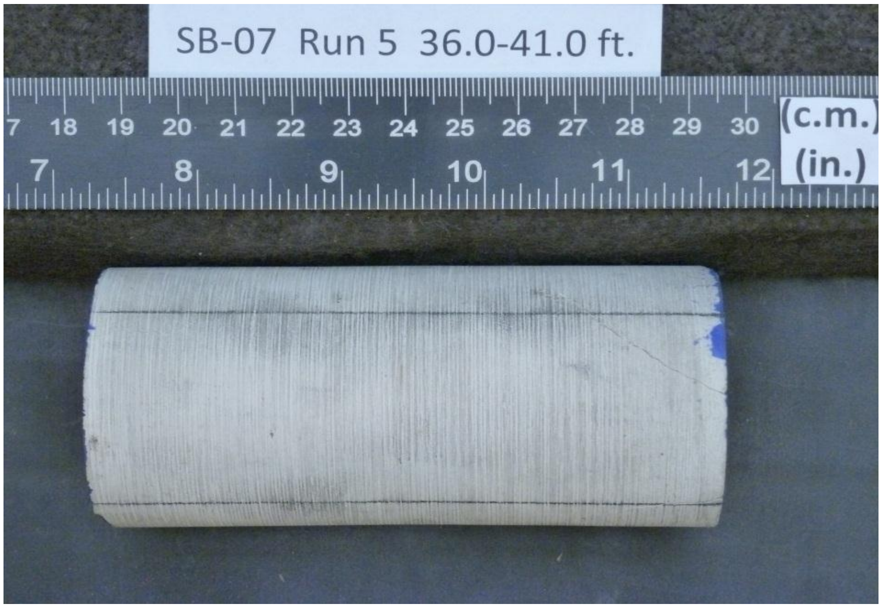
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00170	2.050	0.00083	0.048	YES
Diameter 2, in (rotated 90°)	0.00090	2.050	0.00044	0.025	YES
END 2					
Diameter 1, in	0.00160	2.050	0.00078	0.045	YES
Diameter 2, in (rotated 90°)	0.00100	2.050	0.00049	0.028	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/18/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	SB-07
Sample ID:	Run 5
Depth, ft:	36.0-41.0

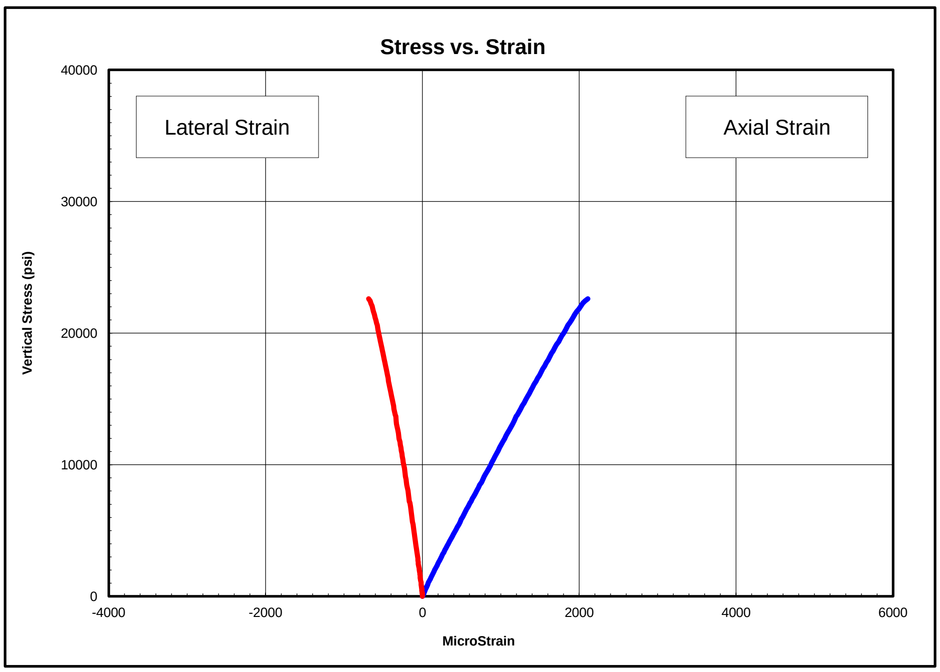


After cutting and grinding

After break

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/24/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	SB-07
Sample ID:	Run 6
Depth, ft:	41.0-46.0
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 22,606 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
2300-8300	11,200,000	0.27
8300-14300	11,100,000	0.32
14300-20300	10,500,000	0.36

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

GeoTesting EXPRESS

Client:	Wang Engineering	Test Date:	1/23/2013
Project Name:	IL 178 over the Illinois River	Tested By:	daa
Project Location:	IL	Checked By:	mpd
GTX #:	300122		
Boring ID:	SB-07		
Sample ID:	Run 6		
Depth:	41.0-46.0 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.11	4.11	4.11
Specimen Diameter, in:	2.05	2.05	2.05
Specimen Mass, g:	607.54		
Bulk Density, lb/ft^3	170		
Length to Diameter Ratio:	2.0	**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.
Straightness Tolerance Met? YES

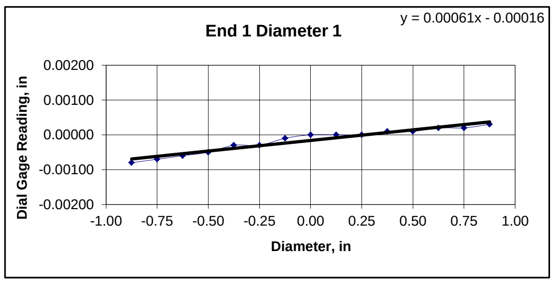
END FLATNESS AND PARALLELISM (Procedure FP1)

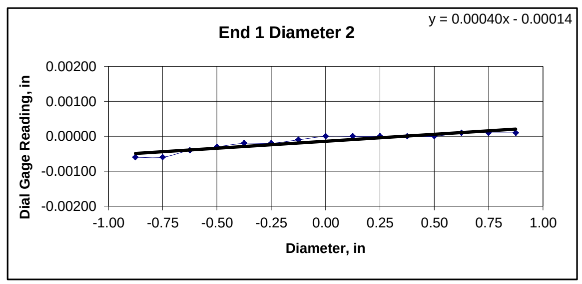
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00080	-0.00070	-0.00060	-0.00050	-0.00030	-0.00030	-0.00010	0.00000	0.00000	0.00000	0.00010	0.00010	0.00020	0.00020	0.00030
Diameter 2, in (rotated 90°)	-0.00060	-0.00060	-0.00040	-0.00030	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010

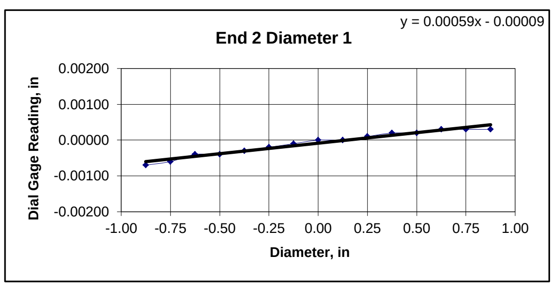
Difference between max and min readings, in:
0° = 0.00110 90° = 0.00070

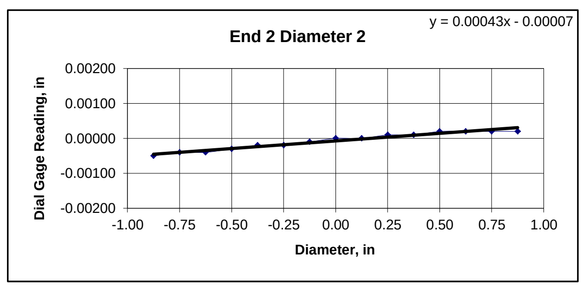
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00070	-0.00060	-0.00040	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00020	0.00020	0.00030	0.00030	0.00030
Diameter 2, in (rotated 90°)	-0.00050	-0.00040	-0.00040	-0.00030	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00010	0.00020	0.00020	0.00020	0.00020

Difference between max and min readings, in:
0° = 0.001 90° = 0.0007
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00055
Flatness Tolerance Met? YES









DIAMETER 1

End 1:	
Slope of Best Fit Line	0.00061
Angle of Best Fit Line:	0.03495
End 2:	
Slope of Best Fit Line	0.00059
Angle of Best Fit Line:	0.03380
Maximum Angular Difference:	0.00115

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:	
Slope of Best Fit Line	0.00040
Angle of Best Fit Line:	0.02292
End 2:	
Slope of Best Fit Line	0.00043
Angle of Best Fit Line:	0.02464
Maximum Angular Difference:	0.00172

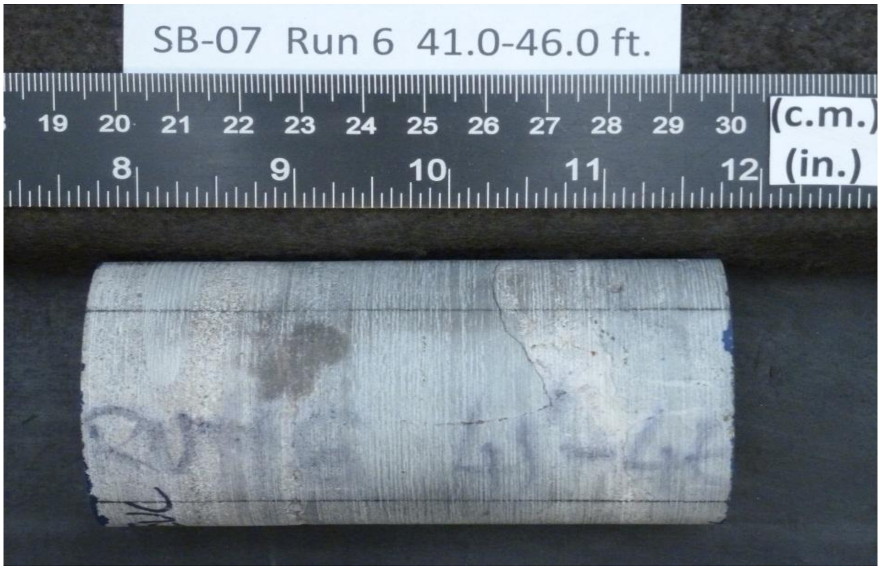
Parallelism Tolerance Met? YES
Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00110	2.050	0.00054	0.031	YES
Diameter 2, in (rotated 90°)	0.00070	2.050	0.00034	0.020	YES
END 2					
Diameter 1, in	0.00100	2.050	0.00049	0.028	YES
Diameter 2, in (rotated 90°)	0.00070	2.050	0.00034	0.020	YES

Maximum angle of departure must be $\leq$ 0.25°
Perpendicularity Tolerance Met? YES

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/24/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	SB-07
Sample ID:	Run 6
Depth, ft:	41.0-46.0

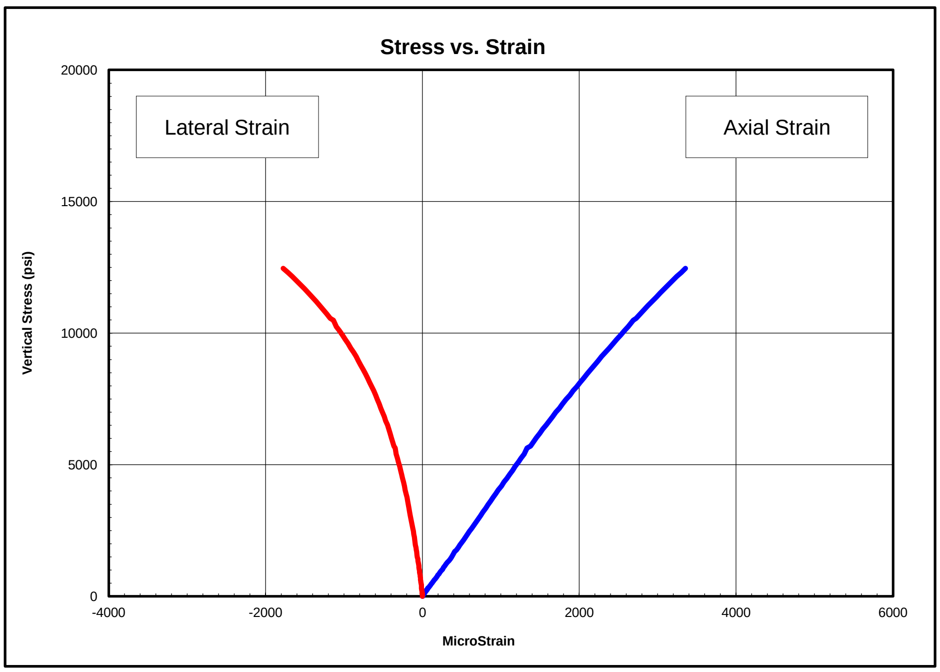


After cutting and grinding

After break

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/24/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	SB-07
Sample ID:	Run 8
Depth, ft:	51.0-56.0
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 13,956 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1400-5100	4,240,000	0.27
5100-8800	3,780,000	---
8800-12500	3,150,000	---

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

GeoTesting EXPRESS

Client:	Wang Engineering	Test Date:	1/23/2013
Project Name:	IL 178 over the Illinois River	Tested By:	daa
Project Location:	IL	Checked By:	mpd
GTX #:	300122		
Boring ID:	SB-07		
Sample ID:	Run 8		
Depth:	51.0-56.0 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.29	4.29	4.29
Specimen Diameter, in:	2.05	2.05	2.05
Specimen Mass, g:	553.97		
Bulk Density, lb/ft^3	149		
Length to Diameter Ratio:	2.1		

Length to Diameter Ratio Tolerance Met? YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? YES

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00050	-0.00050	-0.00040	-0.00030	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	-0.00050	-0.00050	-0.00050	-0.00030	-0.00030	-0.00020	-0.00020	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010

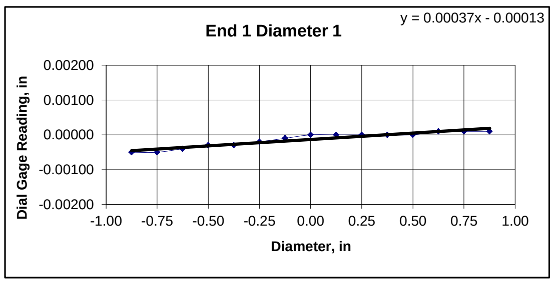
Difference between max and min readings, in:
0° = 0.00060 90° = 0.00060

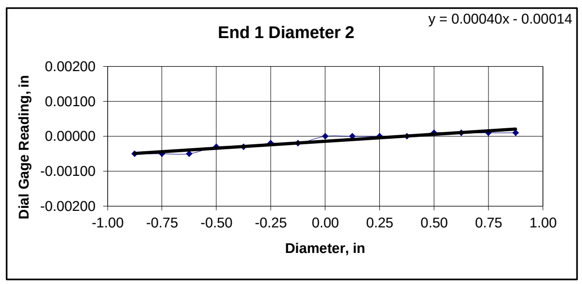
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00040	-0.00040	-0.00040	-0.00030	-0.00030	-0.00020	-0.00020	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010
Diameter 2, in (rotated 90°)	-0.00050	-0.00040	-0.00030	-0.00030	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010	0.00020

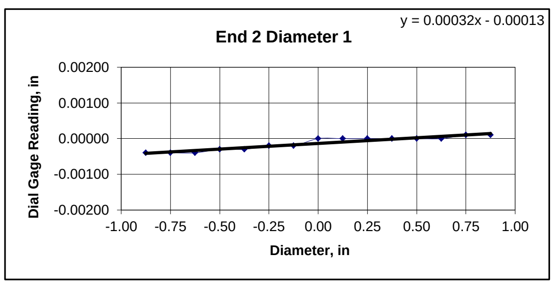
Difference between max and min readings, in:
0° = 0.0005 90° = 0.0007

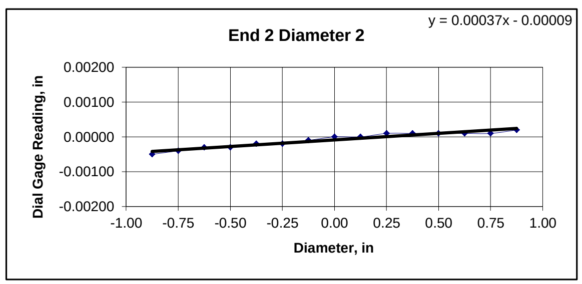
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00035

Flatness Tolerance Met? YES









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00037
Angle of Best Fit Line: 0.02120

End 2:
Slope of Best Fit Line 0.00032
Angle of Best Fit Line: 0.01833

Maximum Angular Difference: 0.00286

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00040
Angle of Best Fit Line: 0.02292

End 2:
Slope of Best Fit Line 0.00037
Angle of Best Fit Line: 0.02120

Maximum Angular Difference: 0.00172

Parallelism Tolerance Met? YES
Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00060	2.050	0.00029	0.017	YES
Diameter 2, in (rotated 90°)	0.00060	2.050	0.00029	0.017	YES
END 2					
Diameter 1, in	0.00050	2.050	0.00024	0.014	YES
Diameter 2, in (rotated 90°)	0.00070	2.050	0.00034	0.020	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	1/24/2013
Tested By:	daa
Checked By:	mpd
Boring ID:	SB-07
Sample ID:	Run 8
Depth, ft:	51.0-56.0

After cutting and grinding

After break

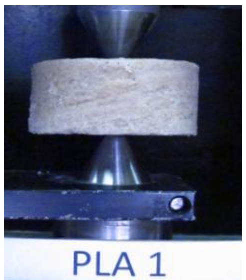
GeoTesting EXPRESS

Client:	Wang Engineering	Test Date:	01/18/13
Project Name:	IL 178 over the Illinois River	Tested By:	jsc
Project Location	IL	Checked By:	mpd
GTX #:	300122	Sample Type:	rock core

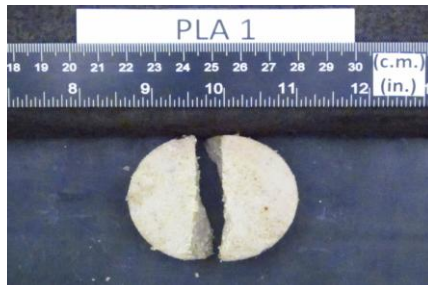
Point Load Strength Index of Rock by ASTM D 5731

Boring No.	Sample No.	Depth, ft.	Test No.	Test Type	Width (W), in.	Depth (D), in.	Failure Load (P), lb	D_e^{2}, in^2	D_e, in.	I_s, psi	F	$I_{s(50)}$, psi	Generalized Correction Factor, K	Estimated Compressive Strength, psi
SB-05	Run 2	7.0-11.0	PLA - 1	Axial	2.05	0.97	153	2.52	1.59	61	0.908	55	18	1,090

PLA - 1
before



PLA - 1
after



Intact Material Failure

Notes:

Generalized correction factor, K, used to estimate the compressive strength based on the specimen depth and ASTM D 5731 Table 1.

D_e = the equivalent core diameter

I_s = the uncorrected point load strength

F = the size correction factor

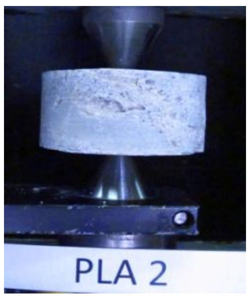
$I_{s(50)}$ = the size corrected point load strength index

Client:	Wang Engineering	Test Date:	01/18/13
Project Name:	IL 178 over the Illinois River	Tested By:	jsc
Project Location	IL	Checked By:	mpd
GTX #:	300122	Sample Type:	rock core

Point Load Strength Index of Rock by ASTM D 5731

Boring No.	Sample No.	Depth, ft.	Test No.	Test Type	Width (W), in.	Depth (D), in.	Failure Load (P), lb	D_e^2, in^2	D_e, in.	I_s, psi	F	$I_{s(50)}$, psi	Generalized Correction Factor, K	Estimated Compressive Strength, psi
SB-06	Run 1	14.0-19.0	PLA - 2	Axial	2.04	1.04	1009	2.70	1.64	374	0.922	344	19	7,100

PLA - 2
before



PLA - 2
after

Intact Material Failure

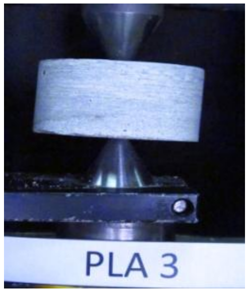
Notes: Generalized correction factor, K, used to estimate the compressive strength based on the specimen depth and ASTM D 5731 Table 1.
D_e = the equivalent core diameter
I_s = the uncorrected point load strength index
F = the size correction factor
$I_{s(50)}$ = the size corrected point load strength index

Client:	Wang Engineering	Test Date:	01/18/13
Project Name:	IL 178 over the Illinois River	Tested By:	jsc
Project Location	IL	Checked By:	mpd
GTX #:	300122	Sample Type:	rock core

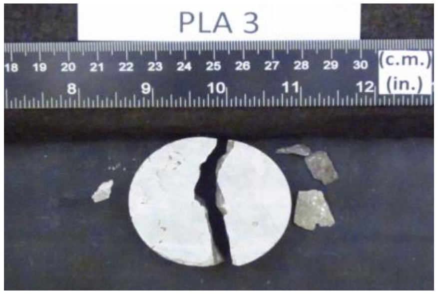
Point Load Strength Index of Rock by ASTM D 5731

Boring No.	Sample No.	Depth, ft.	Test No.	Test Type	Width (W), in.	Depth (D), in.	Failure Load (P), lb	D_e^2, in^2	D_e, in.	I_s, psi	F	$I_{s(50)}$, psi	Generalized Correction Factor, K	Estimated Compressive Strength, psi
SB-06	Run 5	34.0-39.0	PLA - 3	Axial	2.05	0.97	2729	2.54	1.59	1074	0.909	977	18	19,300

PLA - 3
before



PLA - 3
after



Intact Material Failure

Notes:

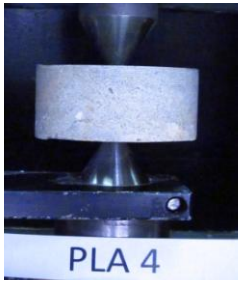
Generalized correction factor, K, used to estimate the compressive strength based on the specimen depth and ASTM D 5731 Table 1.
D_e = the equivalent core diameter
I_s = the uncorrected point load strength index
F = the size correction factor
$I_{s(50)}$ = the size corrected point load strength index

Client:	Wang Engineering	Test Date:	01/18/13
Project Name:	IL 178 over the Illinois River	Tested By:	jsc
Project Location	IL	Checked By:	mpd
GTX #:	300122	Sample Type:	rock core

Point Load Strength Index of Rock by ASTM D 5731

Boring No.	Sample No.	Depth, ft.	Test No.	Test Type	Width (W), in.	Depth (D), in.	Failure Load (P), lb	D_e^2, in^2	D_e, in.	I_s, psi	F	$I_{s(50)}$, psi	Generalized Correction Factor, K	Estimated Compressive Strength, psi
SB-08	Run 1	3.5-5.5	PLA - 4	Axial	2.22	0.99	2518	2.80	1.67	900	0.929	837	18	16,200

PLA - 4
before



PLA - 4
after

Intact Material Failure

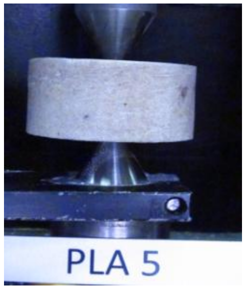
Notes: Generalized correction factor, K, used to estimate the compressive strength based on the specimen depth and ASTM D 5731 Table 1.
D_e = the equivalent core diameter
I_s = the uncorrected point load strength index
F = the size correction factor
$I_{s(50)}$ = the size corrected point load strength index

Client:	Wang Engineering	Test Date:	01/18/13
Project Name:	IL 178 over the Illinois River	Tested By:	jsc
Project Location	IL	Checked By:	mpd
GTX #:	300122	Sample Type:	rock core

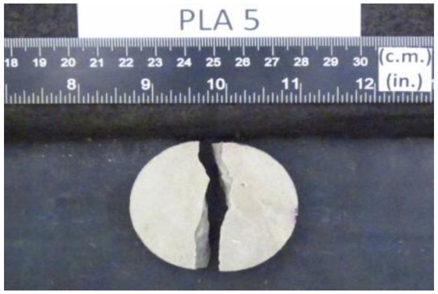
Point Load Strength Index of Rock by ASTM D 5731

Boring No.	Sample No.	Depth, ft.	Test No.	Test Type	Width (W), in.	Depth (D), in.	Failure Load (P), lb	D_e^2, in^2	D_e, in.	I_s, psi	F	$I_{s(50)}$, psi	Generalized Correction Factor, K	Estimated Compressive Strength, psi
SB-08	Run 4	11.0-15.5	PLA - 5	Axial	2.05	0.99	1036	2.58	1.61	401	0.913	366	18	7,220

PLA - 5
before



PLA - 5
after



Intact Material Failure

Notes: Generalized correction factor, K, used to estimate the compressive strength based on the specimen depth and ASTM D 5731 Table 1.

D_e = the equivalent core diameter

I_s = the uncorrected point load strength index

F = the size correction factor

$I_{s(50)}$ = the size corrected point load strength index

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122
Test Date:	01/18/13
Tested By:	jsc
Checked By:	mpd
Sample Type:	Core
Sample Description:	---
Strain Rate:	2.5%/min.

Splitting Tensile Strength of Intact Rock Core Specimens by ASTM D 3967

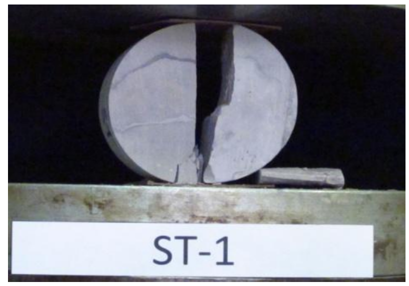
Boring ID	Sample ID	Depth, ft.	Test No.	Thickness (L), in.			Diameter (D), in.	Failure Load (P), lb.	Splitting Tensile Strength, psi
SB-06	Run 2	19.0-24.0	ST-1	0.98	0.99	0.98	2.05	1,611	508
SB-06	Run 7	44.0-49.0	ST-2	1.03	1.02	1.03	2.05	7,985	2,410
SB-07	Run 2	21.0-26.0	ST-3	1.04	1.03	1.05	2.05	2,126	635
SB-07	Run 7	46.0-51.0	ST-4	1.02	1.04	1.02	2.05	2,741	832

Client:	Wang Engineering
Project Name:	IL 178 over the Illinois River
Project Location:	IL
GTX #:	300122

Splitting Tensile Strength of Intact Rock Core Specimens by ASTM D 3967

SB-06
Run 2
19.0-24.0 ft

ST-1

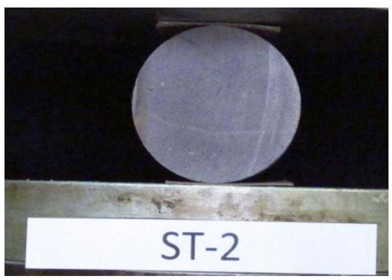


Intact material failure

L/D: 0.5

SB-06
Run 7
44.0-49.0 ft

ST-2

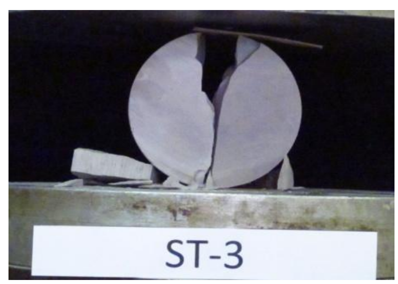


Intact material failure

L/D: 0.5

SB-07
Run 2
21.0-26.0 ft

ST-3

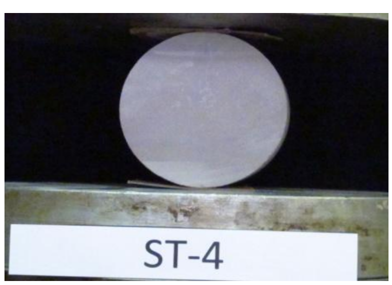


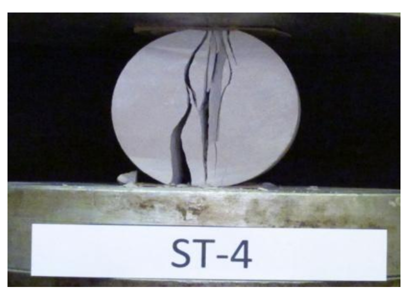
Intact material failure

L/D: 0.5

SB-07
Run 7
46.0-51.0 ft

ST-4





Intact material failure

L/D: 0.5