Geotechnical Evaluation Report

Proposed Bridge Replacement
County Highway 63 over the James River
Ypsilanti Township
Stutsman County, North Dakota

Prepared for

Stutsman County

Professional Certification:
I hereby certify that this plan, specification, or report was prepared by me or under my direct supervision and that I am a duly Registered Professional Engineer under the laws of the State of North Dakota.

Sean S. Swartz, PE
Principal/Senior Engineer
Registration Number: PE-5131
May 3, 2010

SEAN STEVEN SWARTZ
REGISTERED
5-3-10
PROFESSIONAL
PE 5131
ENGINEER
NORTH DAKOTA

Project FA-10-00861

Braun Intertec Corporation

BRAUN
INTERTEC

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May 3, 2010 Project FA-10-00861

Stutsman County
c/o Mr. Steve Thompson
Interstate Engineering, Inc.
1903 12th Avenue Southwest
P.O. Box 2035
Jamestown, ND 58402-2035

Re: Geotechnical Evaluation
Proposed Bridge Replacement (Bridge No. 143-39.0)
County Highway 63 over the James River
Ypsilanti Township
Stutsman County, North Dakota

Dear Mr. Thompson:

Braun Intertec is pleased to present this Geotechnical Evaluation Report for the proposed Bridge replacement (Bridge No. 143-39.0) located on County Highway 63 over the James River, approximately 1-mile south of the town of Ypsilanti in Stutsman County, North Dakota. A summary of our results and a summary of our recommendations in light of the geotechnical issues influencing design and construction are presented below. More detailed information and recommendations follow.

Summary of Results

Two borings were performed near the proposed/existing bridge location. The borings encountered fill to a depth of about 12 feet that was underlain by 2 to 2 ½ feet of buried topsoil. Beneath the buried topsoil, Boring ST-1 encountered glacial outwash deposits consisting of silt and silty sands; and Boring ST-2 encountered glacial till deposits consisting of fat c lay with sand followed by glacial outwash deposits of poorly graded sand with silt and silty sand. Beneath the clay and sand layers, both borings encountered decomposed shale, which was texturally classified as fat clay to the termination depth of the borings.

Groundwater was observed at a depth of 30 feet in Boring ST-1 and at a depth of 16 feet in Boring ST-2 at the time of drilling. The water surface of the James River was observed at a depth of about 14 ½ feet below the top of the bridge. We anticipate that the groundwater level near the bridge will typically match and fluctuate in unison with the water level of the river.

Summary of Recommendations

We developed recommendations for driven pile foundations consisting of 12x53 H-piles. We estimate that full structural capacity of the piles can be achieved at an estimated pile tip depth of about 45 feet (+/- 5 feet) below existing roadway grades for both abutment and pier piles.

Remarks

Thank you for making Braun Intertec your geotechnical consultant for this project. If you have questions about this report, or if there are other services that we can provide in support of our work to date, please call Jennifer McKinnon or Sean Swartz at 701.232.8701.

Sincerely,

BRAUN INTERTEC CORPORATION

Jennifer McKinnon

Jennifer R. McKinnon, EI
Staff Engineer

Sean S. Swartz, PE
Principal/Senior Engineer

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A. Introduction

A.1. Project Description

This Geotechnical Evaluation Report addresses the proposed bridge replacement (Bridge No. 143-39.0) located on County Highway 63, approximately 1-mile south of the town of Ypsilanti in Stutsman County, North Dakota. The project details have not been specified, although it is our understanding the project will include the construction of either a new single- or multi-span prefabricated steel bridge that spans the James River.

A.2. Purpose

The purpose of this report is to provide Stutsman County and their design consultants with geotechnical information at the project location to aid them in preparing plans and specifications for the proposed bridge replacement.

A.3. Background Information and Reference Documents

To facilitate our evaluation, we were provided with or reviewed the following information or documents:

- A figure provided by Interstate Engineering showing project location, with no title or date;
- A topographic map provided by Interstate Engineering, with no title or date;
- Correspondence with Mr. Steve Thompson of Interstate Engineering; and
- A Geologic Map of North Dakota, by Lee Clayton and the North Dakota Geological Society, 1980.

A.4. Site Conditions

The project site is located on County Highway 63. The existing bridge spans the James River approximately 1 mile south of the town of Ypsilanti. The existing bridge is a multi-span structure supported over steel piles.

A.5. Scope of Services

Our scope of services for this project was originally submitted as a Proposal to Mr. Steve Thompson with Interstate Engineering, on March 5, 2010. We received authorization to proceed from Mr. Mark Close with Stutsman County, on April 6, 2010.

Tasks for the bridge replacement that were completed in accordance with our authorized scope of services included:

A.5.a. Staking and Surveying

We staked the exploration locations by measuring distances from the existing bridge abutments. Boring ST-1 was located approximately 20 feet south and 2 feet east of the southeast corner of the existing bridge; and Boring ST-2 was located approximately 18 feet north and 3 feet west of the northwest corner of the existing bridge.

Surface elevations were measured using a surveyor's level by referencing the pavement surface at the center of the existing bridge over the centerline of County Highway 63. For reporting purposes we used a reference elevation of 150.0 feet as shown on the Log of Boring Sheets in the Appendix. The ground surface elevations at Borings ST-1 and ST-2 were 149.3 and 149.4 feet, respectively.

A.5.b. Subsurface Exploration and Laboratory Testing

Prior to beginning subsurface exploration activities, we cleared the exploration locations of underground utilities through North Dakota One Call.

Our scope of services included the performance of two (2) standard penetration test borings to an estimated depth of 100 feet. We performed two (2) standard penetration test borings at the approximate locations shown on the sketch in the Appendix. Due to relatively shallow bedrock, both borings were terminated at a depth of 61 feet.

Moisture content (MC) and Atterberg limits tests were performed on selected samples.

A.5.c. Geotechnical Analyses and Recommendations

Our scope of services included the provision of the following within the geotechnical report:

- A CAD sketch showing project components, limits, and exploration locations;

- Logs of the borings describing the materials encountered and presenting the results of our groundwater measurements and laboratory tests;
- A summary of the subsurface profile and groundwater conditions;
- Endslope recommendations;
- Recommended pile types for the proposed bridge structure; and
- Graphical representation of the predicted nominal (ultimate) geotechnical pile resistances versus depth for the recommended pile type(s) in accordance with LRFD methodology, or ultimate working capacity versus depth in accordance with ASD methodology.

B. Results

B.1. Exploration Logs

B.1.a. Log of Boring Sheets

The Log of Boring sheets for our penetration test borings are included in the Appendix. The logs identify and describe the geologic materials that were penetrated, and present the results of penetration resistance tests performed within them, laboratory tests performed on penetration test samples retrieved from them, and groundwater measurements.

Strata boundaries were inferred from changes in the penetration test samples and the auger cuttings. Because sampling was not performed continuously, the strata boundary depths are only approximate. The boundary depths likely vary away from the boring locations, and the boundaries themselves may also occur as gradual rather than abrupt transitions.

B.1.b. Geologic Origins

Geologic origins assigned to the materials shown on the logs and referenced within this report were based on: (1) a review of the background information and reference documents cited above, (2) visual classification of the various geologic material samples retrieved during the course of our subsurface exploration, (3) penetration resistance and other in-situ testing performed for the project, (4) laboratory test results, and (5) available common knowledge of the geologic processes and environments that have impacted the site and surrounding area in the past.

B.2. Geologic Profile

B.2.a. Soils

The borings encountered fat clay fills to a depth of about 12 feet that were underlain by 2 to 2 ½ feet of buried topsoil. Beneath the buried topsoil, Boring ST-1 encountered glacial outwash deposits consisting of silt and silty sands; and Boring ST-2 encountered glacial till deposits consisting of fat clays with sand followed by glacial outwash deposits of poorly graded sands with silt and silty sands. Beneath the clay and sand layers, both borings encountered decomposed shale, which was texturally classified as fat clay to the termination depth of the borings.

Penetration resistances in the glacial till deposits ranged from 2 to 9 blows per foot (BPF), indicating they were soft to rather stiff; and in the glacial outwash deposits, penetration resistances ranged from 20 to 36 BPF, indicating they were medium dense to dense. Penetration resistances in the shale deposits ranged from 42 to 100+ BPF, indicating the soils were hard.

B.2.b. Groundwater

Groundwater was observed at a depth of 30 feet in Boring ST-1 and at a depth of 16 feet in Boring ST-2 at the time of drilling. The water surface of the James River was observed at a depth of about 14 ½ feet below the top of the bridge at the time of drilling. We anticipate that the groundwater level near the bridge will typically match and fluctuate in unison with the water level of the river. Seasonal and annual fluctuations of groundwater should be anticipated.

B.3. Laboratory Test Results

B.3.a. Moisture Content Tests

As part of our laboratory testing program, we performed a total of eight (8) moisture content (MC) tests that we used to aid in our classifications and estimations of the soils' engineering properties. The moisture contents of the materials tested ranged from 19 to 50 percent indicating they were near to above optimum, and were generally moderately to highly plastic. The results of the moisture content tests are listed in the "MC" column of the Log of Boring Sheets attached in the Appendix.

B.3.b. Atterberg Limits Tests

We performed two Atterberg limits tests on selected penetration test samples for classification and evaluation of the range of soil plasticity. The tests indicated the clays had liquid limits (LL) of 23 and 51 percent, plastic limits (PL) of 21 and 32 percent, and plasticity indices (PI) of 2 and 19 percent, indicating the soils were silts (classified under ASTM symbol "ML") and fat clays (CH), respectively. The results of the Atterberg limits tests are listed in the "Tests or Notes" column on the attached Log of Boring sheets.

C. Basis for Recommendations

C.1. Design Details

C.1.a. Foundation Type

The preferred deep foundation system for bridges around the site area are driven piles, typically consisting of H-pile or cast-in-place, closed-ended steel pipe piles. For this project, the soils become very hard rather quickly at a depth of about 30 to 35 feet, and there is not a development of significant skin friction within the upper soft clays. An H-pile would likely be the most suitable for this location. We developed recommendations for driven pile foundations consisting of 12x53 H-piles.

C.1.b. Foundation Loading

Interstate Engineering indicated that the proposed design factored loading per driven pile will be about 100 tons for the abutments and piers.

C.1.c. Anticipated Grade Changes

We have assumed that the top of the bridge will generally match the existing grade of County Highway 63.

C.1.d. Project Assumptions

We have assumed the bridge will be designed in accordance with the North Dakota Department of Transportation (NDDOT) 2007 Standard Specifications for Road and Bridge Construction. We have used LRFD methodology for our analyses and recommendations.

C.1.e. Precautions Regarding Changed Information

We have attempted to describe our understanding of the proposed construction to the extent it was reported to us by others. Depending on the extent of available information, assumptions may have been made based on our experience with similar projects. If we have not correctly recorded or interpreted the project details, we should be notified. New or changed information could require additional evaluation, analyses, and/or recommendations.

D. Recommendations

D.1. Bridge Endslope

A preliminary bridge survey has not yet been completed, thus a cross-section through the channel along the existing bridge alignment was not yet available. Assuming a flowline elevation of about 130 feet (on our assumed datum), it is our opinion that the endslopes may be designed for a gradient of 3H:1V or flatter.

We have assumed that there are no existing signs of slope movement near the existing bridge abutment walls. If there are any signs of movement, we should be contacted to reevaluate the endslope conditions. If existing movement has been/is occurring, the slopes may need to be flattened, or else overexcavation of the failed areas would likely be required.

D.2. Bridge Foundation Subgrade Preparations

We have assumed the bridge abutment foundations (bottom of pile cap) will be set at a depth of approximately 7 to 8 feet below existing grades. At these depths, the anticipated soils encountered in the base of the excavation will likely consist of fat clay fills. These soils will be easily disturbed when wet. In order to maintain stability for workers during pile driving and the setting of formwork, we recommend overexcavating below the bottom of the pile cap to a depth of ½- to 1 foot and replacing the materials with a well-drained aggregate (such as a 1-inch or ¾-inch minus gravel).

D.3. Pile Recommendations

D.3.a. Pile Type

Interstate Engineering indicated the anticipated foundation system would be driven 12x53 H-piles. We performed our analyses and calculations based on driven 12x53 H-piles.

D.3.b. Calculation Method

We used the computer program, *DRIVEN®,* to estimate the nominal geotechnical vertical compressive resistance of the piles for the proposed bridge abutments and piers. The Federal Highway Administration developed *DRIVEN*®, which is a static pile analysis software program that uses a combination of the *Tomlinson alpha (α) method* for clays and the *Nordlund/Thurman method* for sands to estimate friction and end-bearing versus depth. We evaluated the "geotechnical static resistance", also referred to as the nominal pile bearing resistance (R_n) of 12x53 H-piles.

There are numerous methods of predicting the static capacities of piles based on the results of borings, and the results of the various methods often differ by a factor of two or more. Furthermore, measuring the ultimate capacity of a pile during or after installation is also subject to variability. The measured capacity depends on the method used (e.g., dynamic formula, wave equation, Pile Driving Analyzer (PDA) or static load test) and the criteria used with each method.

Our scope of services did not include drivability analyses using wave-equation-analysis-of-piles (WEAP) software. WEAP analyses should be completed by the pile driving contractor to analyze drivability of the proposed pile driving system.

D.3.c. Design Soil Parameters

The unit weights input into *DRIVEN*® were estimated based on the measured moisture contents, standard penetration test (SPT) resistances, visual classifications, and past experiences with other projects in the general area. When necessary, we use the Naval Facilities Engineering Command, Soil Mechanics Design Manual (pg. 7.1-149, Figure 7) to estimate friction angles of coarse-grained soils. We estimated the undrained shear strengths of fine-grained soils based on an average of the penetration resistances.

D.3.d. Assumptions

We assumed the bottom-of-pile-cap (BOPC) elevation to be 7 to 8 feet below existing ground surface for both the north and south abutments (the elevation for the BOPC for the abutments was assumed to be 142 feet, based on our assumed datum). We assumed the pile cut-off elevations would be approximately 1 foot above the BOPC elevations (143 feet at the abutments).

For calculation of the pile capacity at the pier location, we assumed the bottom of the channel at the pier location was approximately 6 feet below the observed water surface (bottom of channel elevation ≈ 130 feet based on our assumed datum). We also assumed 2 feet of contraction scour and 3 feet of local scour would be included in the design of the pier piles (bottom of local scour elevation ≈ 125 feet).

D.3.e. LRFD Geotechnical Resistances

As indicated in Section D.1.b above, we utilized *DRIVEN*® to estimate the nominal geotechnical vertical compressive resistance (R_n) for the different pile types. We have tabulated the output data from *DRIVEN*® in a graphical format and present the results in the Pile Capacity Chart, located in the Appendix. The calculated resistances indicated on the graph are an estimate of driving conditions (reductions for loss of skin friction due to scour are not included in the Chart).

D.3.f. Factored (LRFD) Geotechnical Pile Capacities

The nominal geotechnical resistance required during driving (R_n) is obtained by dividing the factored load per pile ($\Sigma\gamma Q_n$) by the appropriate pile driving resistance factor (ϕ) **($R_n = \Sigma\gamma Q_n / \phi$)**.

The American Association of State Highway and Transportation Officials (AASHTO) recommend relating ϕ to the degree of construction control. For situations where subsurface exploration and static calculations have been completed, AASHTO recommends the following ϕ factors.

Table 1. Recommended Pile Driving Resistance Factors (ϕ)[a]

Specified Construction Control	ϕ
Wave Equation	0.40
Wave Equation and Pile Dynamic Analyzer (PDA)	0.65
Driving criteria established by a static load test, quality control by wave equation and/or PDA[b].	0.55 to 0.90

[a] Based on Table 10.5.5.2.3-1 of AASHTO's LRFD Bridge Design Specifications, 2007
[b] Based on Table 10.5.5.2.3-2 of AASHTO's LRFD Bridge Design Specifications, 2007

The required nominal geotechnical pile resistance, R_n, to which the piles will need to be advanced, will be dependent upon the degree of construction control. If only the *Wave Equation* will be used the factored load should be divided by a ϕ of 0.4 to obtain R_n. If a combination of the *Wave Equation* and a *Pile Driving Analyzer (PDA)* will be used the factored load should be divided by a ϕ of 0.65 to obtain R_n.

D.3.g. Down Drag and Scour

Provided the grades surrounding the bridge will remain relatively unchanged (+/- 3 feet), we anticipate that down drag (DD) will not be a factor for this bridge design.

For report purposes, we have assumed a contraction scour depth of 2 feet and a local scour depth of 3 feet at the pier locations (bottom of total scour elevation of approximately 125 feet, on our assumed datum). Potential scour is accounted for by adding the side friction obtained within the scour depths to the required nominal pile bearing resistance, R_n. The scour is not factored. Based on our calculations, a total of 10 kips should be added to the R_n for the pier piles to account for the assumed scour. We have also assumed that scour will not be applicable at the abutment pile locations due to slope protection.

D.3.h. Estimated Pile Lengths

The pile lengths should be determined by calculating for R_n with the following equations:

- (Abutments) $R_n = (\sum \gamma Q_n) / \phi$
- (Piers) $R_n = 10 \text{ kips} + (\sum \gamma Q_n) / \phi$

The calculated values of R_n should then be plotted on the applicable Chart provided in the Appendix to find the estimated pile toe elevation/length.

For the abutments and pier piles, utilizing ϕ of 0.4 (Wave Equation only), we estimate a 12x53 H-pile depth of about 45 (+/- 5) feet; and utilizing ϕ of 0.65 (using the PDA), we estimate a pile depth of about 40 (+/- 5) feet. (Please note that these depths are noted as depths below the pavement surface at the boring locations).

We wish to note that the H-piles may not achieve their required capacities during driving. It will likely be necessary to drive the H-piles to their designated lengths and reevaluate their capacities upon restrike.

D.3.i. Pile Settlement

We anticipate total and differential deformation of the pile heads will be less than 1-inch and 1/2-inch, respectively, under the assumed loads. Piles driven with one of the previously referenced driving control methods are not designed to settle. The majority of deformation at the pile head is due to elastic shortening of the pile under the design loads.

D.3.j. Pile Specifications

We recommend that the piles conform to American Society of Testing and Materials (ASTM) A572 with minimum yield strength of 50 kips per square inch (ksi). We also recommend that pile construction be performed in general accordance with NDDOT Specification Section 622.

D.3.k. Pile Driving System

Using an under or oversized pile-driving hammer can be detrimental to the successful installation of piling. Prior to the driving system acceptance, we recommend performing a wave equation analysis modeling prospective contractors' pile installation systems. The wave equation analysis is used to estimate probable driving stresses and pile penetration resistance based on the type of hammer proposed, the specified pile type/size and the site-specific material conditions which, when combined, help evaluate system suitability. Our firm can discuss the requirements and limitations of wave equation analyses and, if needed, perform them.

D.3.l. Test Pile Program

We based the nominal resistance of the driven pile foundation system on our calculations using the soil conditions present at the boring locations. To more accurately predict actual pile lengths and capacities, we recommend performing a test pile program on at least one test pile for each substructure.

We recommend dynamically monitoring the test piles using the Case-Goble Pile-Driving Analyzer (ASTM Test Method D 4945). Data accumulated from the Pile-Driving Analyzer should be used to formulate driving/length criteria by which the remainder of the pile should be driven. We provide this service and will gladly discuss it with you further. Otherwise, the FHWA Modified Gates dynamic pile capacity formula or other dynamic formula may be used to predict pile capacity in the very stiff to hard glacial till during installation.

D.3.m. Production Pile Monitoring

A qualified bridge inspector should observe the installation of all piles. The inspector should document pertinent pile information such as lengths, elevations, and driving resistances, as well as note that the driving/length criteria has been achieved for each of the piles for satisfactory load-carrying capacities.

After the piles are driven to adequate bearing and cut off at design elevations, we recommend inspecting the piles for damage and plumbness/batter. The geotechnical and structural engineers should review the load-carrying capability of any pile that is damaged during driving, or at an angle outside the plumbness or batter specification. We recommend including contingencies in the project budget for additional piles and additional pile lengths below the predicted pile tip elevations.

E. Procedures

E.1. Penetration Test Borings

The penetration test borings were drilled with a truck-mounted auger drill equipped with hollow-stem auger. The borings were performed in accordance with ASTM D 1586. Penetration test samples were taken at 2 1/2- or 5-foot intervals. Actual sample intervals and corresponding depths are shown on the boring logs.

E.2. Material Classification and Testing

E.2.a. Visual and Manual Classification

The geologic materials encountered were visually and manually classified in accordance with ASTM Test Method D 2488. A chart explaining the classification system is attached. Samples were sealed in jars and returned to our facility for review and storage.

E.2.b. Laboratory Testing

The results of the laboratory tests performed on geologic material samples are noted on or follow the appropriate attached exploration logs. The tests were performed in accordance with ASTM or AASHTO procedures.

E.3. Groundwater Measurements

The drillers checked for groundwater as the penetration test borings were advanced. The boreholes were then backfilled or allowed to remain open for an extended period of observation as noted on the boring logs.

F. Qualifications

F.1. Variations in Subsurface Conditions

F.1.a. Material Strata

Our evaluation, analyses, and recommendations were developed from a limited amount of site and subsurface information. It is not standard engineering practice to retrieve material samples from exploration locations continuously with depth, and therefore strata boundaries and thicknesses must be inferred to some extent. Strata boundaries may also be gradual transitions, and can be expected to vary in depth, elevation, and thickness away from the exploration locations.

Variations in subsurface conditions present between exploration locations may not be revealed until additional exploration work is completed, or construction commences. If any such variations are revealed, our recommendations should be re-evaluated. Such variations could increase construction costs, and a contingency should be provided to accommodate them.

F.1.b. Groundwater Levels

Groundwater measurements were made under the conditions reported herein and shown on the exploration logs, and interpreted in the text of this report. It should be noted that the observation period was relatively short, and groundwater can be expected to fluctuate in response to rainfall, flooding, irrigation, seasonal freezing and thawing, surface drainage modifications and other seasonal and annual factors.

F.2. Continuity of Professional Responsibility

F.2.a. Plan Review

This report is based on a limited amount of information, and a number of assumptions were necessary to help us develop our recommendations. It is recommended that our firm review the geotechnical aspects of the designs and specifications, and evaluate whether the design is as expected, if any design changes have affected the validity of our recommendations, and if our recommendations have been correctly interpreted and implemented in the designs and specifications.

F.2.b. Construction Observations and Testing

It is recommended that we be retained to perform observations and testing during construction. This will allow correlation of the subsurface conditions encountered during construction with those encountered by the borings, and provide continuity of professional responsibility.

F.3. Use of Report

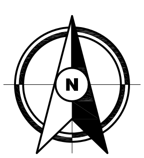
This report is for the exclusive use of Stutsman County and their design consultants. Without written approval, we assume no responsibility to other parties regarding this report. Our evaluation, analyses, and recommendations may not be appropriate for other parties or projects.

F.4. Standard of Care

In performing its services, Braun Intertec used that degree of care and skill ordinarily exercised under similar circumstances by reputable members of its profession currently practicing in the same locality. No warranty, express or implied, is made.

Appendix

F:\FA\2010\FA1000861.dwg,APORT,4/29/2010 3:15:37 PM



30'± 0 60'±

SCALE: 1" = 60'±

DENOTES APPROXIMATE LOCATION OF STANDARD PENETRATION TEST BORING

Sheet: of	
Fig:	
Project No:	FA1000861
Drawing No:	FA1000861
Scale:	1" = 60'±
Drawn By:	BJB
Date Drawn:	4/29/10
Checked By:	JM
Last Modified:	4/29/10

SOIL BORING LOCATION SKETCH
PROPOSED BRIDGE REPLACEMENT
STUTSMAN COUNTY
1 MILE SOUTH OF YPSILANTI, NORTH DAKOTA

BRAUN INTERTEC
11001 Hampshire Avenue So.
Minneapolis, MN 55438
PH. (952) 995-2000
FAX (952) 995-2020

BRAUN℠ INTERTEC

LOG OF BORING

Braun Project FA-10-00861
Geotechnical Evaluation
Bridge Replacement
County Highway 63 over the James River
Ypsilanti, North Dakota

BORING: **ST-1**

LOCATION: Southeast side of Bridge. See Sketch.

DRILLER: K. Miller | METHOD: 3 1/4" HSA, Autohammer | DATE: **4/19/10** | SCALE: **1" = 4.5'**

Elev. feet 149.3	Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	MC %	Tests or Notes
		FILL	FILL: Fat Clay with Sand, trace roots, dark brown and black, moist.	4			
				7			
				5			
				4		33	
				5			
137.3	12.0	CH	FAT CLAY, black, moist. (Buried Topsoil)	2			
134.8	14.5	ML	SILT, dark brown, moist to wet, rather stiff to soft. (Glacial Outwash)	9		19	LL=23, PL=21, PI=2
				2			
126.3	23.0	SM	SILTY SAND, fine- to coarse-grained, with GRAVEL, gray, waterbearing, medium dense to dense. (Glacial Outwash)	20			
			-with Clay lenses below 29 1/2 feet.	36	▽		
				25			

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING 00861.GPJ BRAUN.GDT 5/3/10 08:20

BRAUN INTERTEC

LOG OF BORING

Braun Project FA-10-00861 Geotechnical Evaluation Bridge Replacement County Highway 63 over the James River Ypsilanti, North Dakota	BORING: ST-1 (cont.) LOCATION: Southeast side of Bridge. See Sketch.

DRILLER: K. Miller	METHOD: 3 1/4" HSA, Autohammer	DATE: 4/19/10	SCALE: 1" = 4.5'

Elev. feet 113.3	Depth feet 36.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	MC %	Tests or Notes
109.3	40.0		SILTY SAND, fine- to coarse-grained, with GRAVEL, gray, waterbearing, medium dense to dense. (Glacial Outwash) *(continued)*				
		SH	PIERRE FORMATION, SHALE, gray, moist to wet, decomposed, very soft, hand deformed sample classified as FAT CLAY (CH).	120		33	
				*			*100/9"
				*		50	*100/12"
				58			
88.3	61.0			75			
			END OF BORING. Water observed at a depth of 30 feet with 59 1/2 feet of hollow-stem auger in the ground. Water observed at a depth of 16 feet with a cave-in depth of 20 feet immediately after withdrawal of auger. Boring then backfilled.				

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING 00861.GPJ BRAUN.GDT 5/3/10 08:20

LOG OF BORING

Braun Project FA-10-00861
Geotechnical Evaluation
Bridge Replacement
County Highway 63 over the James River
Ypsilanti, North Dakota

BORING: **ST-2**

LOCATION: Northwest side of Bridge. See Sketch.

DRILLER: K. Miller	METHOD: 3 1/4" HSA, Autohammer	DATE: 4/19/10	SCALE: 1" = 4.5'

Elev. feet 149.4	Depth feet 0.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	MC %	Tests or Notes
		FILL	FILL: Fat Clay with Sand, trace roots, dark brown, moist to wet.	4			
				6			
				6			
				5		32	LL=51, PL=32, PI=19
				6			
137.4	12.0	CH	FAT CLAY, dark brown and black, moist. (Buried Topsoil)	15			
135.4	14.0	CH	FAT CLAY with SAND, brown, moist, soft. (Glacial Till)	3	▽	27	
132.4	17.0	SP-SM	POORLY GRADED SAND with SILT, fine-grained, brown and gray, waterbearing, medium dense. (Glacial Outwash)	20			
126.9	22.5	SM	SILTY SAND, fine- to coarse-grained, with GRAVEL, brown, waterbearing, medium dense. (Glacial Outwash)	27		20	
120.4	29.0	SH	PIERRE FORMATION, SHALE, gray, moist to wet, decomposed, very soft, hand deformed sample classified as FAT CLAY (CH).	*			*100/10"
				68		29	

(See Descriptive Terminology sheet for explanation of abbreviations)

BRAUN℠ INTERTEC

LOG OF BORING

Braun Project FA-10-00861 Geotechnical Evaluation Bridge Replacement County Highway 63 over the James River Ypsilanti, North Dakota	BORING: ST-2 (cont.) LOCATION: Northwest side of Bridge. See Sketch.

DRILLER: K. Miller	METHOD: 3 1/4" HSA, Autohammer	DATE: 4/19/10	SCALE: 1" = 4.5'

Elev. feet 113.4	Depth feet 36.0	Symbol	Description of Materials (Soil- ASTM D2488 or D2487, Rock-USACE EM1110-1-2908)	BPF	WL	MC %	Tests or Notes
			PIERRE FORMATION, SHALE, gray, moist to wet, decomposed, very soft, hand deformed sample classified as FAT CLAY (CH). *(continued)*				
				64			
				42			
				47			
				85			
				*			*100/11 1/2"
88.4	61.0		END OF BORING. Water observed at a depth of 16 1/2 feet with 19 1/2 feet of hollow-stem auger in the ground. Water observed at a depth of 38 feet with 59 1/2 feet of hollow-stem auger in the ground. Water not observed to cave-in depth of 12 1/2 feet immediately after withdrawal of auger. Boring then backfilled.				

(See Descriptive Terminology sheet for explanation of abbreviations)

LOG OF BORING 00861.GPJ BRAUN.GDT 5/3/10 08:20

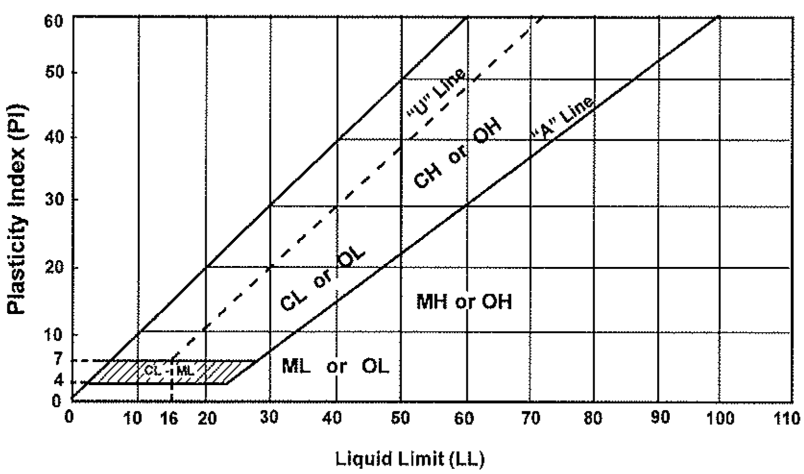
BRAUN INTERTEC

Descriptive Terminology of Soil

Standard D 2487 - 00
Classification of Soils for Engineering Purposes
(Unified Soil Classification System)

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [a]				Soils Classification: Group Symbol	Soils Classification: Group Name [b]
Coarse-grained Soils more than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels 5% or less fines [e]	$C_u \geq 4$ and $1 \leq C_c \leq 3$ [c]	GW	Well-graded gravel [d]
			$C_u < 4$ and/or $1 > C_c > 3$ [c]	GP	Poorly graded gravel [d]
		Gravels with Fines More than 12% fines [e]	Fines classify as ML or MH	GM	Silty gravel [d f g]
			Fines classify as CL or CH	GC	Clayey gravel [d f g]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands 5% or less fines [i]	$C_u \geq 6$ and $1 \leq C_c \leq 3$ [c]	SW	Well-graded sand [h]
			$C_u < 6$ and/or $1 > C_c > 3$ [c]	SP	Poorly graded sand [h]
		Sands with Fines More than 12% [i]	Fines classify as ML or MH	SM	Silty sand [f g h]
			Fines classify as CL or CH	SC	Clayey sand [f g h]
Fine-grained Soils 50% or more passed the No. 200 sieve	Silts and Clays Liquid limit less than 50	Inorganic	PI > 7 and plots on or above "A" line [j]	CL	Lean clay [k l m]
			PI < 4 or plots below "A" line [j]	ML	Silt [k l m]
		Organic	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OL	Organic clay [k l m n]
				OL	Organic silt [k l m o]
	Silts and clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line	CH	Fat clay [k l m]
			PI plots below "A" line	MH	Elastic silt [k l m]
		Organic	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OH	Organic clay [k l m p]
				OH	Organic silt [k l m q]
Highly Organic Soils	Primarily organic matter, dark in color and organic odor			PT	Peat

a. Based on the material passing the 3-in (75mm) sieve.
b. If field sample contained cobbles or boulders, or both, add "with cobbles or boulders or both" to group name.
c. $C_u = D_{60} / D_{10}$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
d. If soil contains ≥15% sand, add "with sand" to group name.
e. Gravels with 5 to 12% fines require dual symbols:
   GW-GM well-graded gravel with silt
   GW-GC well-graded gravel with clay
   GP-GM poorly graded gravel with silt
   GP-GC poorly graded gravel with clay
f. If fines classify as CL-ML, use dual symbol GC-GM or SC-SM.
g. If fines are organic, add "with organic fines" to group name.
h. If soil contains ≥ 15% gravel, add "with gravel" to group name.
i. Sands with 5 to 12% fines require dual symbols:
   SW-SM well-graded sand with silt
   SW-SC well-graded sand with clay
   SP-SM poorly graded sand with silt
   SP-SC poorly graded sand with clay
j. If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.
k. If soil contains 10 to 29% plus No. 200, add "with sand" or "with gravel" whichever is predominant.
l. If soil contains ≥ 30% plus No. 200, predominantly sand, add "sandy" to group name.
m. If soil contains ≥ 30% plus No. 200 predominantly gravel, add "gravelly" to group name.
n. PI ≥ 4 and plots on or above "A" line.
o. PI < 4 or plots below "A" line.
p. PI plots on or above "A" line.
q. PI plots below "A" line.



Laboratory Tests

DD	Dry density, pcf	OC	Organic content, %
WD	Wet density, pcf	S	Percent of saturation, %
MC	Natural moisture content, %	SG	Specific gravity
LL	Liquid limit, %	C	Cohesion, psf
PL	Plastic limit, %	∅	Angle of internal friction
PI	Plasticity index, %	qu	Unconfined compressive strength, psf
P200	% passing 200 sieve	qp	Pocket penetrometer strength, tsf

Particle Size Identification

Boulders .... over 12"
Cobbles .... 3" to 12"
Gravel
  Coarse .... 3/4" to 3"
  Fine .... No. 4 to 3/4"
Sand
  Coarse .... No. 4 to No. 10
  Medium .... No. 10 to No. 40
  Fine .... No. 40 to No. 200
Silt .... < No. 200, PI < 4 or below "A" line
Clay .... < No. 200, PI ≥ 4 and on or above "A" line

Relative Density of Cohesionless Soils

Very loose .... 0 to 4 BPF
Loose .... 5 to 10 BPF
Medium dense .... 11 to 30 BPF
Dense .... 31 to 50 BPF
Very dense .... over 50 BPF

Consistency of Cohesive Soils

Very soft .... 0 to 1 BPF
Soft .... 2 to 3 BPF
Rather soft .... 4 to 5 BPF
Medium .... 6 to 8 BPF
Rather stiff .... 9 to 12 BPF
Stiff .... 13 to 16 BPF
Very stiff .... 17 to 30 BPF
Hard .... over 30 BPF

Drilling Notes

Standard penetration test borings were advanced by 3 1/4" or 6 1/4" ID hollow-stem augers unless noted otherwise. Jetting water was used to clean out auger prior to sampling only where indicated on logs. Standard penetration test borings are designated by the prefix "ST" (Split Tube). All samples were taken with the standard 2" OD split-tube sampler, except where noted.

Power auger borings were advanced by 4" or 6" diameter continuous-flight, solid-stem augers. Soil classifications and strata depths were inferred from disturbed samples augered to the surface and are, therefore, somewhat approximate. Power auger borings are designated by the prefix "B."

Hand auger borings were advanced manually with a 1 1/2" or 3 1/4" diameter auger and were limited to the depth from which the auger could be manually withdrawn. Hand auger borings are indicated by the prefix "H."

BPF: Numbers indicate blows per foot recorded in standard penetration test, also known as "N" value. The sampler was set 6" into undisturbed soil below the hollow-stem auger. Driving resistances were then counted for second and third 6" increments and added to get BPF. Where they differed significantly, they are reported in the following form: 2/12 for the second and third 6" increments, respectively.

WH: WH indicates the sampler penetrated soil under weight of hammer and rods alone; driving not required.

WR: WR indicates the sampler penetrated soil under weight of rods alone; hammer weight and driving not required.

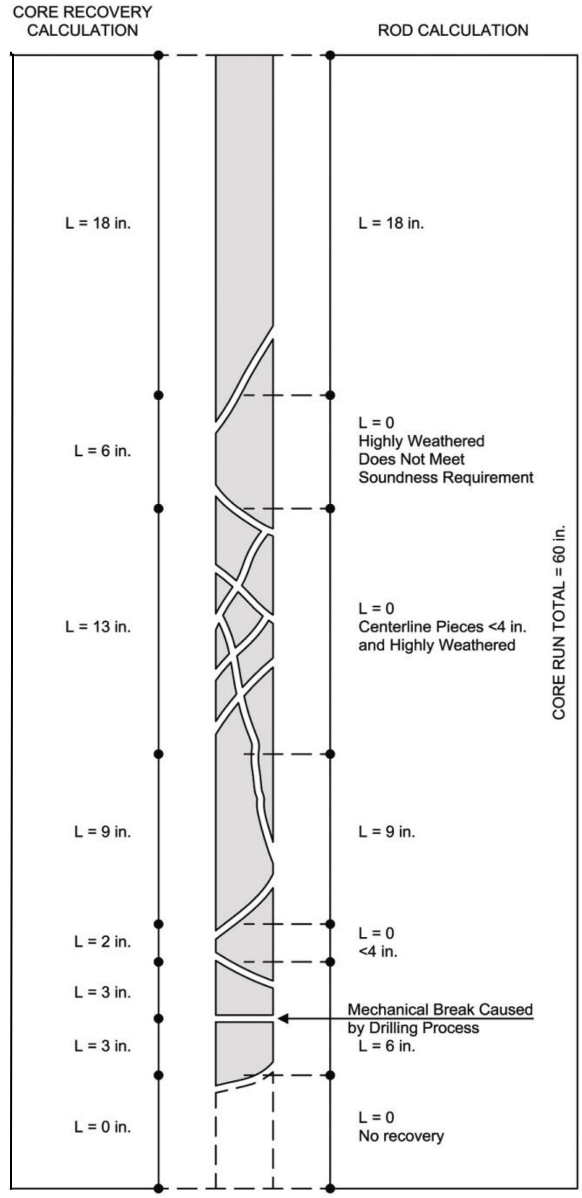
TW indicates thin-walled (undisturbed) tube sample.

Note: All tests were run in general accordance with applicable ASTM standards.

BRAUN INTERTEC

Descriptive Terminology of Rock

Based on U.S. Army Corps of Engineers EM 1110-1-2908



Example Calculations

Core Recovery, CR = $\frac{\text{Total length of rock recovered}}{\text{Total core run length}}$

Example: $CR = \frac{(18 + 6 + 13 + 9 + 2 + 3 + 3)}{(60)}$

CR = 90%

RQD = $\frac{\text{Sum of sound pieces longer than 4 inches}}{\text{Total core run length}}$

RQD Percent	Rock Quality
<25	very poor
25 < 50	poor
50 < 75	fair
75 < 90	good
90 < 100	excellent

Example: $RQD = \frac{(18 + 9 + 4 + 6)}{(60)}$

RQD = 62%

Weathering

Unweathered: No evidence of chemical or mechanical alteration.

Slightly weathered: Slight discoloration on surface, slight alteration along discontinuities, less than 10% of rock volume altered.

Moderately Weathered: Discoloration evident, surface pitted and altered with alteration penetrating well below rock surfaces, weathering halos evident, 10% to 50% of the rock altered.

Highly Weathered: Entire mass discolored, alteration pervading nearly all of the rock, with some pockets of slightly weathered rock noticeable, some mineral leached away.

Decomposed: Rock reduced to a soil consistency with relict rock texture, generally molded and crumbled by hand.

Hardness

Very soft:	Can be deformed by hand
Soft:	Can be scratched with a fingernail
Moderately hard:	Can be scratched easily with a knife
Hard:	Can be scratched with difficulty with a knife
Very hard:	Cannot be scratched with a knife

Texture

Sedimentary Rocks:	Grain Size
Coarse grained	2 – 5 mm
Medium grained	0.4 – 2 mm
Fine grained	0.1 – 0.4 mm
Very fine grained	< 0.1 mm

Igneous and Metamorphic Rocks:

Coarse grained	5 mm
Medium grained	1 – 5 mm
Fine grained	0.1 – 1 mm
Aphanitic	< 0.1 mm

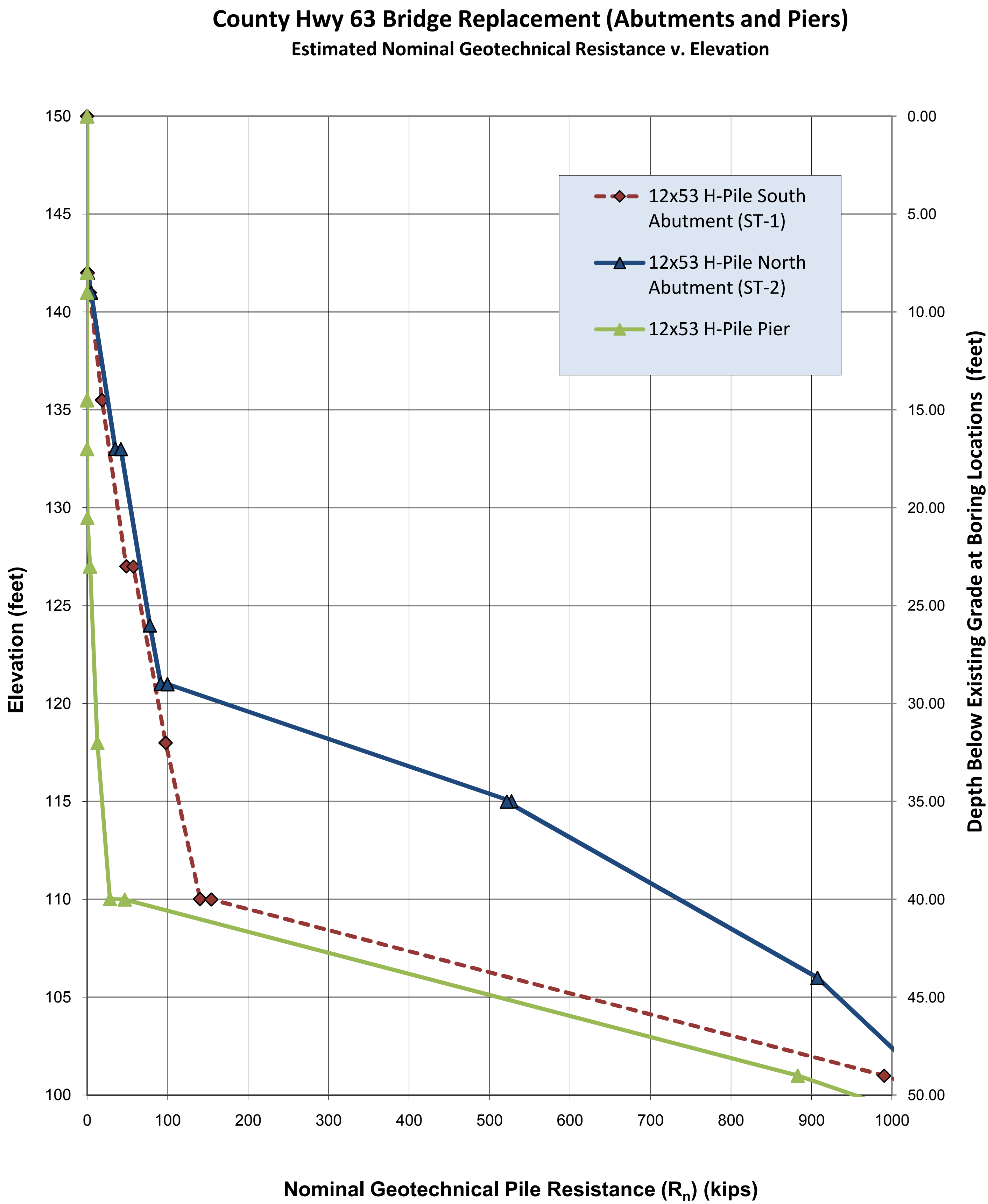
Thickness of Bedding

Massive:	3 ft. thick or greater
Thick bedded:	1 to 3 ft. thick
Medium bedded:	4 in. to 1 ft. thick
Thin bedded:	4 in. thick or less

Degree of Fracturing (Jointing)

Unfractured:	Fracture spacing 6 ft. or more
Slightly fractured:	Fracture spacing 2 to 6 ft.
Moderately fractured:	Fracture spacing 8 in. to 2 ft.
Highly fractured:	Fracture spacing 2 in. to 8 in.
Intensely fractured:	Fracture spacing 2 in. or less

2/2010



- The nominal geotechnical resistance indicated on this chart assumes infinite pile structural capacity.