MAGPIE AND WHITETAIL CROSSINGS
BILLINGS COUNTY, ND
ND FLAP 704(1) AND 795(1)

FINAL Geotechnical Report

Prepared by
Federal Highway Administration
Central Federal Lands Highway Division

Geotechnical Services Branch

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SECTION ONE - Introduction

1.1 BACKGROUND AND LOCATION

This report presents the results of the geotechnical engineering assessment and provides recommendations for culvert and bridge construction work to be performed on Whitetail Creek and Magpie Creek crossings in Billings County, North Dakota. The project is located near and within the Little Missouri National Grassland. The project proposes to improve two existing low-water crossings that provide access to the Little Missouri National Grassland with concrete box culverts at Whitetail Creek and a 3-span bridge at Magpie Creek. The project sites are in undeveloped grassland adjacent to the Whitetail Creek and Magpie Creek. A site map is presented on Plate 1 of this report.

The project roadways provide access to camping, fishing, hiking, and day use facilities. The existing stream crossings at Whitetail Creek and Magpie Creek consist of unimproved low water crossings. The crossings are inadequate and frequently overtop, resulting in significant maintenance costs, vehicle damage, and impassability.

1.2 SCOPE AND PURPOSE

The overall objective of this project is to improve the existing crossings to reduce maintenance and improve safety for the traveling public. A two-barrel concrete box culvert is proposed at Whitetail Creek and a 3-span bridge is proposed at Magpie Creek to improve the stream crossing conditions and improve safety for the traveling public. Work also consists of placing embankment fill to raise roadway grades and reconstruction of the gravel surface roadway approaches to the stream crossings. Outlet protection at the box culvert and abutment protection at the bridge are proposed. Ancillary construction including signage and revegetation will also be performed.

The scope of work included a geotechnical investigation, analysis, and recommendations for culvert foundations, bridge foundations, and cuts and fills within the project limits for development of design through construction. This involved several tasks including field reconnaissance, subsurface sampling, laboratory testing, interpretation and correlation of field measurements, and geotechnical engineering analysis. Specifically, this investigation was conducted to determine the soil profiles at the proposed culvert and bridge locations and develop recommendations concerning foundations, retaining walls, embankments, material shrink/swell, geologic hazards, and construction considerations for design and construction of structures and slopes within the alignment. The stationing in this report is based on the preliminary 70% project plans, dated October 2022.

Figure 1.2: Existing Conditions at Whitetail Creek Crossing (July 24, 2018)

Figure 1.1: Existing Conditions at Magpie Creek Crossing (July 24, 2018)

SECTION TWO - Geology and Seismicity

2.1 REGIONAL GEOLOGY

The project is located in the Unglaciated Missouri Plateau Section of the Great Plains physiographic province. The area is described by Carlson (1983) as part of the Little Missouri Badlands. The region is underlain by marine sedimentary deposits dating back to the Precambrian era. Within these sedimentary sections, periods of emergence and subsequent erosion are marked by regional unconformities. The surface and near subsurface is typically underlain by Tertiary sedimentary deposits that have been variably lithified. More resistant beds are typically composed of sandstone or limestone. Numerous lignite (low-grade coal) beds are present in the sedimentary deposits. “Clinker” or “scoria” is formed by baking of sediments where the lignite has burned.

During the most recent glacial period, the Little Missouri River was captured by the ice-marginal Missouri River. Subsequent warming and thawing lead to a lower regional base level, with streams and tributaries continuing to cut back into uplands. At this time, the Little Missouri River was incised approximately 200 to 500 feet into the relict upland surface. This created an area that was generally characterized by gently rolling uplands interrupted by buttes and ridges capped by more resistant rocks. Badlands areas continue to widen as the back-cutting erosion progresses northward. Landslides are common due to oversteepened slopes.

2.2 SITE GEOLOGY

Geologic mapping of the Whitetail Creek crossing is limited to 1:100,000 scale mapping (Carlson, 1983). This mapping depicts the Whitetail Creek crossing as underlain by the Tertiary Sentinel Butte Formation (Tsb), which is described as alternating beds of variably lithified sandstone, mudstone, claystone, clinker, and lignite. More recent 1:24,000 scale mapping was completed for the Magpie Creek Crossing (Gonzalez, 2004). This mapping depicts the Magpie Creek crossing as underlain by modern and older alluvial deposits (Qal & Qoal), consisting of sediment ranging in size from clay to gravel. These surficial deposits are underlain by the Tertiary Bullion Creek Formation (Tbc), which is described as alternating beds of variably lithified sandstone, mudstone, claystone, clinker, and lignite. Thickness of the Tertiary deposits at both sites is greater than 100 feet (Carlson, 1983).

It should be noted that although the project is underlain by formational units, these materials are not classified as rock for engineering purposes. These units are relatively young and have been subject to variable consolidation and cementation. These materials are typically considered stiff to very stiff soils in terms of strength and engineering behavior.

No seismic source and Quaternary faults are mapped within 40 miles of the project area (U.S. Geological Survey, 2021).

Refer to Plate 2 to view the geologic map and further unit descriptions that correspond to the project sites.

2.3 SEISMIC DESIGN PARAMETERS

Recommended seismic response parameters are based on the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications, 8th Edition, 2017, and represents horizontal peak ground acceleration (PGA) with 7 percent probability of exceedance in 75 years (approximate 1,000-year return period). The 1,000-year return period uniform hazard spectrum for the Whitetail Creek Crossing (47.19855º N latitude and 103.30238º W longitude) was obtained in accordance with the AASHTO ground motion maps.

The site was classified as "Class D" according to site class definitions specified in Table 3.10.3.1-1 of AASHTO based on a stiff to very stiff soil profile without bedrock within 100 feet of the surface. The recommended spectral acceleration coefficient values for probabilistic design are summarized in the following table.

Table 1:- Summary of Seismic Parameters Corrected for Site Class D

Peak Seismic Ground Acceleration Coefficient, (A_s)	0.031
Horizontal Response Spectral Acceleration at Period of 0.2 sec, (S_{DS})	0.074
Horizontal Response Spectral Acceleration at Period of 1.0 sec (S_{D1})	0.039
Site Factor at Zero-Period of Acceleration Spectrum, (F_{pga})	1.6
Site Factor at Short-Period of Acceleration Spectrum, (F_a)	1.6
Site Factor at Long-Period Range of Acceleration Spectrum, (F_v)	2.4

Based on the long acceleration coefficient S_{D1} value of 0.039, the site is assigned to seismic hazard Zone 1 in accordance with Table 3.10.6-1 of AASHTO. Seismic design parameters were also determined for Magpie Crossing. Due to similar geology and proximity of the two sites, the Magpie site displayed very similar seismic parameters and was also assigned to seismic hazard Zone 1. Seismic hazard zones reflect the variation in seismic risk in different regions needing different requirements for design as depicted in Table 4.7.4.3.1-1 in AASHTO. Based on a classification of Zone 1, seismic loading is not anticipated to impact the design of structures for the project sites.

2.4 GEOLOGIC HAZARDS

The project sites are located in a relatively flat lying area on a sequence of unconsolidated to lightly consolidated sediments. These materials can potentially have low bearing resistance and high settlement potentials. This hazard can be mitigated by appropriate foundation soil preparation and structural design. Coal and lignite beds are mapped in the area and were found during the subsurface investigation. These beds may be corrosive and contribute to increased acidity of surrounding soils. Landslides are mapped in the project area but are unlikely to affect the proposed culvert and bridge crossings due to the low topographic relief at the project sites. Flooding is also a concern in the project area and can be mitigated by appropriate hydraulic design of the project structures.

SECTION THREE - Subsurface Investigation

3.1 SUBSURFACE EXPLORATION PROGRAM

A subsurface investigation targeting the proposed stream crossings was performed by a Central Federal Lands Highway Division (CFLHD) of the Federal Highway Administration (FHWA) geotechnical engineer on October 24, 2018. The geotechnical subsurface exploration program consisted of drilling a total of three (3) borings to approximately 40 feet in depth. The original project scope proposed a series of box culverts at the Magpie Crossing, but the scope changed to a bridge structure in 2022 prior to the 70% design milestone. Thus, boring depths at Magpie are shallower than typical borings at bridge locations. One boring was drilled at the Whitetail Creek crossing and two borings at the Magpie Creek crossing. Standard penetration testing (SPT) and sample collection was performed at 5-foot intervals in each boring. Subsurface conditions were logged, and representative samples were collected and transported to the CFLHD Materials Laboratory in Lakewood, CO, for physical property testing. Logs of the explorations and boring locations are presented in Appendix A and Plate 3, respectively. Photographs related this exploration can be found in Appendix C. A summary of the field exploration is provided in Table 2.

Table 2:- Summary of the Field Exploration Program

EXPLORATION DESIGNATION	STATION	OFFSET	GROUND ELEVATION	TERMINATION ELEVATION	EXPLORATION DEPTH
BH18-01	204+09	34-ft LT	2,528 ft	2,488.5 ft	39.5 ft
BH18-02	106+24	8-ft LT	2,138 ft	2,098.5 ft	39.5 ft
BH18-03	107+71	12-ft RT	2,137 ft	2,097 ft	40 ft

Note: The exploration locations were estimated relative to existing features with stationing based on the preliminary 70% design plans. Ground elevations were estimated from CFLHD survey data.

3.2 LABORATORY TESTING PROGRAM

Soil samples recovered from the borings by SPT were tested in the laboratory to support the field classifications and to provide an estimate of the engineering characteristics and mechanical properties of the soil. Laboratory tests included moisture content (AASHTO T255), sieve analysis (AASHTO T11 & T27), classification (AASHTO M145 & ASTM D2487), Atterberg limits (AASHTO T89 & T90), and corrosivity of soils (AASHTO T288, T289, T290, T291). When the necessary tests were completed, samples were classified using the Unified Soil Classification

System (USCS) and AASHTO soil classification system. Results of the testing are shown below in Table 3 and 4, and are presented in Appendix B.

Table 3:- Summary of Laboratory Index Test Results

BORING NUMBER	SAMPLE DEPTH, ft	PERCENT GRAVEL	PERCENT SAND	PERCENT PASSING 200	LL	PI	USCS	AASHTO
BH18-01	3	0	69	31	21	5	SC-SM	A-2-4(0)
BH18-01	8	1	73	26	NV	NP	SM	A-2-4(0)
BH18-01	18	0	10	90	56	39	CH	A-7-6(37)
BH18-01	23	3	8	89	50	35	CH	A-7-6(32)
BH18-02	3	16	63	21	21	7	SC-SM	A-2-4(0)
BH18-02	13	0	8	92	38	19	CL	A-6(18)
BH18-02	23	0	11	89	50	29	CH	A-7-6(27)
BH18-03	9	0	2	98	30	12	CL	A-6(11)
BH18-03	19	0	46	54	NV	NP	ML	A-4(0)

Soil test results indicated a relatively shallow layer of silty to clayey sand (SC-SM and SM) overlying silts and clays with varying quantities of sand (CL, CH, and ML). The sands typically classified as A-2-4 soils by AASHTO, while the silts and clays classified as A-4, A-6, and A-7-6 soils by AASHTO.

Soil corrosivity testing of a bulk, streambed sample from the Magpie crossing site was sent to an outside laboratory for testing. Results are summarized below in Table 4.

Table 4:- Summary of Laboratory Corrosivity Results

SAMPLE LOCATION	SAMPLE DEPTH, ft	RESISTIVITY, ohm-cm	PH	SULFATE CONTENT, ppm / %	CHLORIDE CONTENT, ppm / %
Magpie Crossing Streambed	0-5	1,969	8.3	170 / 0.017	9 / 0.009

SECTION FOUR - Analysis & Recommendations

This section presents analysis and recommendations for bridge foundations, box culvert foundations, embankments, cut slopes, material shrink/swell, and construction considerations for the design and construction of the Magpie and Whitetail Crossings project. Based on discussions with the project team, proposed improvements as of the 70% design plan set includes new bridge construction for the Magpie Crossing and new concrete box culverts for the Whitetail Crossing. Generalized subsurface profiles were developed based on field reconnaissance, surficial visual evaluation, and subsurface investigations.

4.1 SUBSURFACE CONDITIONS

Bedrock was encountered during the subsurface exploration, but as discussed in subsection 2.2, it is considered soil for engineering characterization. The surficial deposits consist of relatively uniform moist silty, clayey sand. The SPT testing indicates that the subsurface materials are loose sands and very stiff to hard clays, signifying that the upper deposits are unconsolidated, and the deeper clays are overconsolidated. Potential for consolidation and settlement under loading is high due to these characteristics. Additionally, 1- to 2.5-foot-thick coal/lignite layers are found at shallow depths (less than 15 feet) capping the clay layer. The coal/lignite may be corrosive and contribute to increased acidity of surrounding soils. This will be further discussed in the foundation preparation subsections (4.3.3 and 4.4.3) and construction considerations (4.6).

Shallow groundwater was encountered at both project sites. Groundwater was 3 to 4 feet deep at Magpie Crossing and 9 feet deep at Whitetail Crossing. Surface and subsurface water is likely to be encountered as construction will take place in or near seasonally active stream channels. Considerations for implementation of dewatering and water diversion techniques are noted in the construction considerations subsection (4.6).

4.2 MAGPIE CROSSING BRIDGE FOUNDATIONS

A new bridge structure is proposed for the Magpie Crossing location. The bridge is anticipated to be a 114-foot long three-span structure. Based on the 70% design plans, the proposed abutment centerline station and cap elevation are shown in Table 5.

Table 5:- Proposed Bridge Foundation Locations

Foundation Location	Station	Centerline Elevation	Pile Cap Elevation
Abutment 1 (SE)	106+43.00	2,147.42	2,142.33
Pier 1	106+80.00	2,147.22	2,144.53
Pier 2	107+20.00	2,147.02	2,144.33
Abutment 2 (NW)	107+57.00	2,146.82	2,141.72

4.2.1 Bridge Foundation Selection

A driven pile foundation system is proposed for the new bridge at the Magpie Crossing location. A deep foundation was selected due to deep scour and settlement concerns associated with shallow footings at this site. Originally, a multi-celled box culvert was considered at this location, but due to large settlement and hydraulic inadequacy, a bridge structure with deep foundations was selected for design and construction prior to the 70% milestone. Drilled shafts were also considered for a deep foundation but were determined to have a higher cost and longer construction time compared to driven piles based on recent construction and bid history. Additionally, due to remote location of the bridge sites, transportation of concrete to the site would be challenging.

A driven pile foundation is feasible from a geotechnical perspective. Piles will be drivable through the moist, hard clay subsurface and develop required resistance at reasonable depth. Open-ended steel pipe piles are the recommended pile type due to the cohesive nature of the subsurface. Scour potential was determined to be present at the abutments and piers and is discussed in detail in the Draft Hydraulics report dated September 28th, 2022.

The following factored bridge loads were provided on January 25th, 2023, by the CFL bridge engineer, and are shown in Table 6.

Table 6:- Bridge Loads and Driven Pile Configuration

Foundation Location	Max Factored Axial Load per Pile (kip)	Piles per Bent	Pile Diameter (inch)
Abutment 1 (SE)	142	3	18
Pier 1	230	3	24
Pier 2	230	3	24
Abutment 2 (NW)	142	3	18

4.2.2 Driven Pile Axial Resistance

The driven pile axial resistance analysis was performed on 18-, 24-, and 30-inch diameter piles at the abutments and piers for 3, 4, and 5 pile configurations. Ultimately, three 18-inch piles at the abutments and three 24-inch piles at the piers were selected for final design based on pile drivability and minimization of estimated total pile length. Analysis was performed in Microsoft Excel using the API method (modified alpha-method) for cohesive soils outlined in the NHI driven pile design manual (FHWA, 2016). The API method was selected due to its applicability in design for highly overconsolidated clays with high undrained shear strengths, as recommended in the NHI drive pile design manual (FHWA, 2016). The abutment and pier foundations, characterized as friction piles, will obtain their resistance through both side friction and tip resistance developed in the clay subsurface, below the scour elevation.

Based on AASHTO guidelines, the nominal axial loads and resistance should be factored by the appropriate resistance factors for the Strength, Service, and Extreme Event limit states. The resistance factor selected depends on construction and field verification methods. It is recommended that at least 4 piles (one at each abutment and pier) be tested using the Pile Driving Analyzer (PDA) and signal matching at the end of driving. Inspector charts based on the wave equations will be developed by the contractor once the driving equipment and accessories are selected. In this case, a resistance factor of 0.65 should be applied to determine the factored resistance at the Strength Limit State (AASHTO Table 10.5.5.2.3-1). Based on this resistance factor, the design resistance lost to scour, and the design load at the Strength Limit State, the required nominal driving resistance was calculated. If PDA is not performed, a resistance factor of

0.40 should be used instead, and the minimum tip elevation and required nominal driving resistance should be reevaluated.

Results of the calculations are presented in Appendix D and calculated pile lengths, minimum tip elevation, and required nominal driving resistance are discussed in subsection 4.2.7.

4.2.3 Pile Setup

Pile setup is an increase in the nominal axial resistance that develops over time. When soils are compressed and disturbed due to pile driving, large excess pore pressures develop. These excess pore pressures are generated partly from the shearing and remolding of the soil and partly from radial compression as the pile displaces the soil. The excess pore pressures cause a reduction in the effective stresses acting on the pile, and thus a reduction in the soil shear strength. This results in a reduced pile resistance during driving, and for a period after driving. After driving is completed, the excess pore pressures will dissipate primarily through radial flow of the pore water away from the pile. As the pore pressures dissipate, the soil reconsolidates and shear strength increases. This increase in soil shear strength results in an increase in pile resistance referred to as soil setup.

Setup occurs more rapidly in cohesionless soils and more slowly in fine grained soils as pore water pressures dissipate. In some cases, setup in clays may continue to develop over a period of weeks and even months, and in large pile groups it can develop even more slowly. Although the on-site soils are fine grained and cohesive in nature, pile setup is not anticipated to result in significant additional capacity as the clays are hard, overconsolidated, and are not highly saturated.

4.2.4 Group Effects on Axial Resistance

The resistance of a pile group to the applied axial loads is not necessarily the sum of the axial resistance of individual piles within the group. The zone of influence from an individual pile in a pile group may overlap with other piles, depending on the pile spacing. Historically, the axial efficiency of groups of piles has not been a concern if the center-to-center spacing between piles is greater than 2.5 times the pile diameter (2.5B) or 30.0 inches, whichever is greater (AASHTO 10.7.1.2). However, pile groups in clay soils under certain pile cap ground conditions may warrant a reduction factor when spacings are greater than 2.5B. An efficiency factor (η) varying from 0.65 at spacing of 2.5B to 1.0 at 6.0B (found by interpolation) should be applied in instances where the pile cap is not in firm contact with the ground and if the soil at the surface is soft (AASHTO 10.7.3.9) If the cap is in firm contact with the ground, or if the cap is not in firm contact with the ground but the soil is stiff, no reduction in efficiency is required.

Pile groups at abutment locations will be in firm contact with the ground. Pile groups at the pier locations will not be in firm contact with the ground but the soil is stiff. Therefore, an efficiency factor (η) of 1.0 is applied for the pile group at all abutment and pier locations.

4.2.5 Lateral Loads

Lateral load analysis was performed by the CFLHD bridge engineer using the software program LPILE developed by Ensoft, Inc. This program analyzes a single pile or shaft considering deflection as a function of design loads, foundation construction, and subsurface conditions. A strength limit state resistance factor of 1.0 is specified in AASHTO 10.7.3.12 for lateral geotechnical resistance of a single pile or pile group. Factored strength limit state lateral loads should be used in the analysis.

Table 7 below provides LPILE input parameters for the foundation soils based available subsurface information and presumptive engineering correlations. These values do not include load or resistance factors. Additionally, it is recommended that lateral support above the scour elevation be neglected due to the potential loss of material during the design flood event.

Table 7:- LPILE Input Parameters

APPROXIMATE ELEVATION[1]	DEPTH BELOW EXISTING GROUND SURFACE	LPILE P-Y MODEL	EFFECTIVE UNIT WEIGHT	STRENGTH PARAMETER[2]	FRICTION ANGLE	STRAIN PARAMETER
(FT)	*(FT)*	*Description*	γ' *(PCF)*	c or k *(PSI or PCI)*	Φ *(DEGREE)*	ε_{50}
Abutment 1 (SE, BH18-02)						
2,138 – 2,135	0 – 3	Sand	120	25	28	-
2,135 – 2,130.5	3 – 7.5	Sand	57.6	20	28	-
2,130.5 – 2,128[3]	7.5 – 10	Lignite (Stiff Clay Above the Water Table)	120	50	-	.005
2,128 – 2,098.5[4]	10 – 39.5	Stiff Clay Above the Water Table	120	50	-	.005
Piers 1 & 2 (No Borehole Data)						
2,136 – 2,117	0 – 19	Stiff Clay Above the Water Table	120	27.8	-	.005
2,117 – 2,061[4]	19 - 75	Stiff Clay Above the Water Table	120	50	-	.005
Abutment 2 (NW, BH18-03)						
2,137 – 2,133	0 – 4	Sand	120	25	28	-
2,133 – 2,132	4 – 5	Sand	57.6	20	28	-
2,132 – 2,130	5 – 7	Stiff Clay Above the Water Table	120	27.8	-	.005
2,130 – 2,129[3]	7 – 8	Lignite (Stiff Clay Above the Water Table)	120	50	-	.005
2,129 – 2,117	8 – 20	Stiff Clay Above the Water Table	120	27.8	-	.005
2,117 – 2,097[4]	20 – 40	Stiff Clay Above the Water Table	120	50	-	.005

[1]Neglect support above the scour elevation (2126.42 feet) at both abutments and both piers
[2]Undrained shear strength (c, psi) for use in clay soils and soil modulus (k, pci) for sands
[3]Neglect support in lignite layer
[4]Stiff clay layer may be assumed to extend to greater depth if needed for modeling. Borings did not advance past 40 feet in depth as a bridge structure was not anticipated at the time of the subsurface investigation

Material properties provided are for single piles and do not account for the reduced lateral resistance of piles in a group. P-multipliers are a function of the number of rows of piles and center-to-center pile spacing in the direction of loading. P-multipliers are required even for a single row of piles if the center-to-center spacing is less than 5 pile diameters. P-multipliers are specified in Table 10.7.2.4-1 in AASHTO. When this analysis method is used, the resistances at the strength limit state as represented by the P-y curves should not be factored since they already represent the nominal condition.

4.2.6 Settlement

Settlement of piles driven to anticipated tip elevation and nominal axial capacity were estimated in accordance with section 10.7.2.3.2 of AASHTO. Settlement of the pile group at each abutment and pier is expected to be approximately ½-inch or less.

4.2.7 Pile Lengths

The final lengths of the driven piles are based on both axial and lateral loading, and minimum embedment. Table 8 below shows the calculated pile lengths needed for both axial and lateral capacities. The information in Table 8 is based on the selected pile diameter of 18 inches at the abutments and 24 inches at the piers. The minimum tip elevations presented in Table 8 are based on the estimated elevations required for lateral resistance. Even though estimated lengths for axial compression resistance at the abutments are longer than estimated lengths for lateral resistance by 2 feet, axial compression will not control the minimum tip elevation, as this depth is solely based on bearing resistance, and not other concerns such as excessive settlement, uplift, downdrag, or liquefaction (AASHTO C10.7.6). The nominal bearing resistance will be verified by field testing at each structural element and may be achieved prior to the minimum tip elevation required for lateral deflection, fixity for resisting lateral loading, and structural requirements. In all cases, the piles should be driven to at least the minimum tip elevation shown on the project plans, even if required resistance is developed at a shallower depth. As discussed in subsection 4.2.2, if no PDA testing is performed, the resistance factor must be reduced to 0.40, and the minimum tip elevation and required nominal driving resistance per pile must be revisited.

Appendix D of this report presents a visual representation of axial, single pile capacity versus pile length.

Table 8:- Calculated Pile Lengths for Axial and Lateral Capacity

STRUCTURE ELEMENT	MAX FACTORED AXIAL LOAD PER PILE	REQUIRED PILE LENGTH FOR AXIAL CAPACITY	REQUIRED PILE LENGTH FOR LATERAL CAPACITY	MINIMUM TIP ELEVATION[1,2]	REQUIRED NOMINAL DRIVING RESISTANCE PER PILE[3]
	(KIP)	*(FT)*	*(FT)*	*(FT)*	*(KIP)*
Abutment 1 (SE)	142	29	27	2115	225
Pier 1	230	35	35	2110	282
Pier 2	230	35	35	2110	282
Abutment 2 (NW)	142	31	29	2113	194

[1]Estimated minimum tip elevation based on satisfying all strength, service, and extreme event limit state requirements
[2]Estimated minimum tip elevation controlled by lateral load requirements. Piles must be driven to this depth or greater.
[3]The required nominal driving resistance per pile must be achieved in addition to the minimum penetration depth

4.2.8 Pile Drivability and Testing

The contractor should conduct drivability analysis using the Wave Equation Analysis Program (WEAP) to select hammers that have sufficient energy to drive the piles to the required embedment without exceeding the allowable pile driving stresses and blows per foot presented in the project specifications. Test piles should be driven while instrumented with PDA to determine production pile driving criteria. One test pile per abutment and pier location, as indicated on the plans, should be tested and monitored by an engineer as outlined in the project specifications. The tests should be performed in accordance with ASTM D 4945. Signal matching program analyses should be performed to confirm the load resistance of the piles at the end of the test-driving program. The production piles should be driven with the same hammer as used for the test piles. If the contractor elects to bring another hammer on-site, the process of drivability analysis and test pile installation outlined above must be repeated for the new equipment.

4.2.9 Scour Potential and Erosion

Final long-term degradation, contraction scour, and local scour (abutment scour and pier scour) depths were determined by the hydraulics engineer. Total scour elevation was determined to be 2,126.42 for both abutments and piers. A detailed scour depth analysis is presented in Section 4.4 of the Draft Hydraulics Report.

4.3 WHITETAIL CROSSING BOX CULVERT DESIGN

A box culvert is proposed to replace the existing, unimproved low water crossing at Whitetail Creek, consisting of two 12-foot span by 10-foot height concrete box culverts. Based on the subsurface exploration, the culvert will be founded on silty, clayey sand overlying very stiff fat clay. It is recommended for the box culvert to be founded on geotextile wrapped granular fill to address anticipated settlement.

4.3.1 Settlement and Consolidation

Soils typically experience two types of volume change related to loading: short-term (immediate/elastic) settlement and long-term (primary consolidation) settlement. The classification and index testing of the on-site soils indicate that settlement due do structural loading will induce both elastic and consolidation settlement. It should be noted that no consolidation testing was performed on recovered soil samples. The soil properties for settlement analysis were selected based on correlations with index properties and used to estimate the total settlement potential for the box culverts. Due to the uncertainty of clay soil properties, a range of values (low, medium, high) were used in the analysis to better inform design. The range of values was determined by applying the minimum, average, and maximum index property values from lab testing on the on-site soils to the correlation equations.

Total settlement was evaluated using the Settle3 software from Rocscience. Analysis was first performed assuming native soil as the foundation material. Further analysis then included varying thicknesses of geotextile wrapped granular fill (12 to 36 inches) as options to reduce the magnitude of total settlement. In addition, differential settlement was calculated across the culverts both parallel and perpendicular to the travel way. Material properties and inputs used for analysis may be found in Table 9. Results of the settlement evaluation for the recommended granular fill thickness of 12-inches is presented in Table 10.

Table 9:- Material Properties and Inputs for Settlement Analysis

Whitetail Crossing									
Material Type	**Qualitative Designation**	**Settlement Type**	**γ *(kcf)***	**E_s *(ksf)***	**C_c**	**C_r**	**OCR**	**C_v *(ft^2/yr)***	**e_0**
Clay	Low	Consolidation	0.12	-	0.360	0.0436	5.94	109.5	0.567
	Middle		0.12	-	0.387	0.0475	3.39	83	0.618
	High		0.12	-	0.414	0.0514	1.7	65.7	0.67
Silty, Clayey Sand	-	Elastic	0.12	48	-	-	-	-	-
Lignite	-	Elastic	0.13	1,467	-	-	-	-	-
Granular Fill	-	Elastic	0.14	4,000	-	-	-	-	-

Table 10:- Box Culvert Settlement Analysis Results

Whitetail Crossing						
Scenario	**Qualitative Designation**	**Immediate Settlement *(in)***	**Primary Consolidation *(in)***	**Total Settlement *(in)***	**Differential Settlement Parallel to Travel Way *(in)***	**Differential Settlement Perpendicular to Travel Way *(in)***
No Improvements	Low	0.44	1.40	1.84	0.72	0.74
	Middle	0.44	1.47	1.91	0.74	0.76
	High	0.44	2.63	3.07	1.82	1.85
12-inch Granular Fill	Low	0.32	1.36	1.68	0.64	0.66
	Middle	0.32	1.44	1.76	0.67	0.70
	High	0.32	2.46	2.78	1.64	1.67

With no improvements, the results of the analysis indicate an estimated total settlement ranging from 1.84 to 3.07 inches with 0.74 to 1.85 inches of differential settlement. In addition, Table 10 above estimates settlement for the geotextile wrapped granular fill, but it should be noted that Settle3 software is unable to account for the addition of geotextile. Therefore, the actual magnitude of total and differential settlement is likely less than what was calculated as the geotextile wrapping will provide additional strength and stability to the foundation. Analysis results also indicate that approximately 50% to 60% of total settlement will occur within the first 3 months after installation of the box culverts.

4.3.2 Bearing Resistance

Ultimate bearing resistance of the culverts is dependent on the dimensions of the foundation elements and characteristics of the bearing material. For these calculations, a foundation length of 32 feet was used based on the proposed culvert length. Bearing resistance for the strength, extreme event, and service limit states are shown in Plate 4. This plate presents both the AASHTO presumptive service limit load and the strength limit load based on the calculations as described below. The fine-grained cohesive on-site soils provide insufficient bearing capacity and excessive settlement is anticipated. Total settlement is still estimated to be outside AASHTO LRFD Bridge Design Specifications (2017) standard serviceability limits (less than 1-inch) after following recommended foundation preparation that includes over-excavation and replacement with geotextile wrapped granular fill (1.68 to 2.78 inches estimated total settlement). Foundation preparation is detailed in the proceeding subsection.

For the Whitetail Crossing site, the presumptive bearing resistance of the silty, clayey sand is estimated to be between 2 to 4 ksf, with a recommended 3 ksf value of use. Provided values were obtained from Table C10.6.2.6.1-1 of AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017. The bearing resistance values presented in this table are based on a maximum settlement of one inch and only apply at the service limit state (resistance factor of 1.0). The base of the box culverts should extend below the depth of frost potential, which is 32-inches based on National Oceanic and Atmospheric Administration (NOAA) data and should be protected from scour as recommended by the hydraulic engineer.

Soil parameters recommended for culvert and retaining wall design are presented in Table 11. These values are unfactored loads and assume that the surface of the soil slope behind the wall is horizontal.

Table 11:- Design Parameters

MATERIAL TYPE	UNIT WEIGHT γ *(pcf)*	FRICTION ANGLE φ *(deg)*	COHESION INTERCEPT *c (psf)*	ACTIVE EQUIVELENT FLUID DENSITY *(pcf)*	AT-REST EQUIVALENT FLUID DENSITY *(pcf)*	NOMINAL FRICTION FACTOR
On-Site Soils	120	28	50[1]	44	64	0.53
Structural Backfill	130	34	0	37	58	0.67
Granular Fill (foundation material)	140	35	0	-	-	-

[1]Cohesion should not be relied upon when calculating active or at-rest pressures acting on structural elements

The AASHTO LRFD resistance factors for various limit states are presented in Table 12.

Table 12:- Resistance Factors (Φ) for each limit state

LIMIT STATE	BEARING	SHEAR RESISTANCE TO SLIDING	PASSIVE PRESSURE RESISTANCE TO SLIDING
Strength I	0.45	0.80	0.50
Service I, Extreme Event I & II	1.00	1.00	1.00

4.3.3 Foundation Preparation

Elastic settlement and potentially some primary settlement, are anticipated to be induced during construction due to equipment loads, compaction, and stages of box culvert and embankment placement. Additionally, the Settle3 model estimates that 50% to 60% of the total settlement will occur within the first 3 months after installation of the box culverts. If the construction schedule allows, and it is deemed feasible within the construction area, application of a pre-load is recommended to induce settlement prior to placement of the box culverts.

To minimize the potential for excessive total and differential settlements it is recommended to:

- Over-excavate by 12 inches to accommodate the granular fill section.
- Allow the excavated area to dry to the most reasonable extent possible. Dewatering should be performed if necessary.
- Compact the excavated foundation in accordance with FP-14 Section 209.10
- Perform construction monitoring of foundation stability as necessary
- Install 12-inch-thick geotextile wrapped granular fill section

The 12-inch geotextile wrapped granular fill section is recommended to prevent on-site soils from migrating into the fill, provide a stable foundation for construction, and distribute loads to help alleviate differential settlements. However, settlement is still anticipated after construction due to the weight of the box culverts, the majority of which (approximately 70%) will take place within the first year after construction.

Prepare the foundation subgrade according to FP-14 Section 209 and project SCRs. If coal/lignite is encountered during excavation, which is likely, it should be completely removed due to its inconsistency and potential for differential movement across its interface with other clay units. The foundation is anticipated to contain unsuitable materials and be wet and unstable. Unsuitable material and poor conditions are expected to continue with depth. It is recommended to allow the excavated area to dry to the most reasonable extent possible, with dewatering if necessary. It is then recommended to compact the excavated foundation and perform construction monitoring of foundation stability as necessary.

The granular fill section will consist of a minimum of 12 inches of granular backfill wrapped with separation geotextile once a compacted foundation is established. Granular backfill should meet the requirements of FP-14 Subsection 703.03(a) and be compacted in accordance with FP-14 Subsection 209.10. Fully encapsulate the granular fill with a separation geotextile, Class 1 (woven), Type C per FP-14 Subsection 714.01. If recommended by the culvert designer or manufacturer, place a layer of bedding material on the wrapped granular fill section to provide a level surface for placing the culverts.

The box culvert is in a seasonally active stream channel; therefore, surface and subsurface water are likely to be encountered during construction. Surface water should be diverted around the culvert construction area. If subsurface water is encountered, the foundation excavations should be provided with appropriate dewatering and/or water diversions.

4.3.4 Lateral Loads

The culverts should be designed to resist lateral loads based on the parameters reported in Table 11. The equivalent fluid densities do not include any surcharge for sloping backfill surfaces or other loads. These equivalent fluid densities do not include load factors or factors of safety; the designer should apply appropriate factors based on their design methodology. Below the mean water level, design the culvert wall for hydrostatic loading.

Lateral loads imposed on the structures are resisted by development of friction between the base of the structure footing and the supporting soils. The nominal friction factors presented in Table 12 above should be used for the design of the culverts.

4.4 ABUTMENT AND WINGWALL DESIGN

Cast-in-place concrete headwalls and wingwalls associated with the proposed culverts are anticipated, along with bridge abutment wingwalls. Abutments and wingwalls should be designed to resist lateral earth pressures and other applicable lateral loads in accordance with the AASHTO LRFD. Lateral earth pressures are influenced by the strength of the backfill, presence of water, and ability of the wall to deflect in response to loading. Other loads including live loads, construction loads, and soil compaction loads should also be considered in the design. Subsurface drainage should be incorporated into the wall to prevent the buildup of hydrostatic pressures behind the wall. Design of concrete structures should be based on the material parameters presented in Table 11 above.

4.4.1 Bearing Resistance

Bearing resistance was not considered for the Magpie bridge abutment wingwalls. The wingwalls are planned to be cantilevered off the end wall and will not require a footing. The following subsection applies only to the Whitetail box culvert wingwalls.

Ultimate bearing resistance of the box culvert wingwall foundations is dependent on the both the length and width of the foundation elements. For the purpose of these calculations, a foundation length of 25 feet was assumed based on the proposed culvert size (12-foot by 10-foot) and proposed embankment side slopes (1V:2.5H). The presumptive bearing resistance of the silty, clayey sand is estimated to be between 2 to 4 ksf, with a recommended 3 ksf value of use. Provided values were obtained from Table C10.6.2.6.1-1 of AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017. The bearing resistance values presented in this table are based on a maximum settlement of one inch and only apply at the service limit state (resistance factor of 1.0).

Plate 5 presents the ultimate bearing capacity of the foundation soils for various foundation widths. The appropriate resistance factor should be applied to the ultimate capacity to determine the factored capacity. Resistance factors for various limit states are presented in Table 12.

4.4.2 Settlement and Consolidation

Settlement of the culvert retaining walls is anticipated in the same nature as the box culvert. The classification and index testing of the on-site soils indicate that immediate settlement will occur due to the structural loading, but the primary form of settlement will be long-term (primary consolidation).

4.4.3 Foundation Preparation

Prepare the foundation soils for the culvert retaining walls in accordance with FP-14 Section 209 and as directed for the concrete box culverts (subsection 4.3.3), including use of a geotextile wrapped 12-inch granular fill section.

No foundation preparation is required for the Magpie bridge abutment wingwalls.

4.4.4 Lateral Loads

Retaining structures should be designed to resist lateral pressures depending on the restraint conditions. Retaining walls that can deflect (active condition) should be designed to resist loads based on the active equivalent fluid density presented in Table 11**Error! Reference source not found.**. Walls that are restrained (at-rest condition) should be designed based on the at-rest equivalent fluid density. If structural backfill is placed within the active or at-rest zones behind the retaining wall, reduced equivalent fluid densities may be used. The equivalent fluid densities do not include any surcharge for sloping backfill surfaces or other loads. These equivalent fluid densities do not include load factors or factors of safety; the designer should apply appropriate factors based on their design methodology. Below the mean water level, design the walls for the full hydrostatic condition.

Lateral loads imposed on the structures are resisted by development of friction between the base of the structure footing and the supporting soils. The nominal friction factors presented above for the culverts may be used for the design of cast-in-place footings established on similar soils to the culverts.

4.5 EARTHWORKS

4.5.1 Embankment and Fill Construction

Embankment construction is anticipated in relation to the construction of the proposed box culverts, bridge structure, wingwalls, and approach road reconstruction. Soils in the project area are typically silty, clayey sands, including existing embankment fills, and very stiff clay may be found at greater depth. The existing embankment fills were constructed at slopes of approximately 1(V):2(H) and are performing well. It is assumed that similar materials will be used for the proposed embankment expansions. Therefore, the expanded embankments should be constructed with maximum 1(V):2(H) side slopes to maintain slope stability and reduce potential for erosion. The expansions should be keyed into the existing ground and compacted in accordance with the requirements of Section 204 of the FP-14.

4.5.2 Cut Slopes & Temporary Shoring

Cut slopes are anticipated for construction of the box culverts, bridge foundations, and associated wingwalls. Shoring may be desirable depending on preferences to limit areas of disturbance. Groundwater is likely to be encountered during construction and dewatering could be necessary. Design and safety of this work is the responsibility of the contractor. The work shall be performed in a manner to minimize hazards and exposure to the public, construction personnel, and equipment.

4.5.3 Shrink/Swell Recommendations

For estimating shrink/swell values, silty, clayey sands are assumed to be the predominate material located in the cut areas. The recommended shrink/swell factors are based on a combination of standard tabled values for common materials in the FLH Technical Guidance Manual (2006) and experience with other CFLHD projects in similar materials.

The silty, clayey sand on-site materials are assumed to swell 15 percent (Swell Factor 1.15) if wasted without tight density control during placement. If the on-site soils are incorporated into embankments with tight density control during placement, a shrink of 10 percent (Swell Factor 0.90) should be used.

4.6 CONSTRUCTION CONSIDERATIONS

Excavation: Excavate using equipment capable of removing the material while preventing material from moving outside the construction limits.

The natural soils are relatively uniform both laterally and vertically. Few boulders were visible in stream banks adjacent to the project site; up to approximately 2 feet in diameter. Based on this, we do not anticipate difficult excavation conditions for conventional, heavy-duty construction equipment in good working order. Occasional boulders requiring special handling may be encountered in excavations for culverts or retaining walls. Additionally, the coal/lignite layers encountered during the subsurface investigation indicated very hard material (SPT refusal or blow counts > 50), but hollow stem augers were able to advance through the material and it is presumed rippable. The contractor should mobilize equipment capable of handling such materials (i.e. hydraulic rams and excavators).

Excavation will encounter wet, loose silty, clayey sands and possibly clay. These materials will likely be unstable and consideration for shoring may be necessary.

Culvert and Wingwall Foundation Preparation: The soils in the stream channel are likely to be soft and saturated in their natural condition. This report recommends replacing the on-site materials below the culverts and wingwalls with a wrapped granular fill section. Recommendations detailed in subsections 4.3.3 and 4.4.3 should be followed.

If issues related to foundation subgrade stability are encountered during construction, the CFLHD geotechnical engineer should be contacted to provide further guidance.

Dewatering: The box culverts and wingwalls are located in or near seasonally active stream channels; therefore, surface and subsurface water are likely to be encountered during construction. Surface water should be diverted around the culvert or wingwall construction area. If subsurface water is encountered, the foundation excavations should be provided with appropriate dewatering and/or water diversions.

Driven Piles: The contractor will be required to submit a driven pile construction plan according to project SCRs and Section 551.04 of the FP-14, which includes personnel qualifications, construction sequence and schedule, a wave equation analysis report, pile-driving equipment information, and details for splices and pile shoes. The pile hammer should be suitably sized to limit significant driving stresses and subsequent pile damage according to the wave equation analysis. The final blow count of production piles should be limited to 10 blows per inch.

Additionally, surface and subgrade materials, especially within the channel itself, are anticipated to be soft and saturated. Selected pile driving equipment and cranes should limit ground contact pressures or appropriate crane pads should be constructed.

Corrosive Soils: Lab testing on a streambed sample from the Magpie Crossing revealed a resistivity of 1,969 ohm-cm, 8.3 pH, and 0.017% sulfate content. Based on guidelines for concrete structures in AASHTO 5.14.2.2 & 5.14.2.4, and driven piles in AASHTO 10.7.5, this soil environment is not considered to be highly corrosive.

4.7 DISCLAIMER/LIMITATIONS CLAUSE

The recommendations in this report are based on the data obtained from exploratory borings, field review, and laboratory test results. The results of these explorations and tests represent conditions at the specific locations indicated. Subsurface variations across the site are likely and may not become evident until excavation is performed. The Analysis and Recommendations sections in this report include interpretations and recommendations developed by the Government in the process of preparing the design. These interpretations are not intended as a substitute for the personal investigation, independent interpretation, and judgment of the Contractor.

SECTION FIVE - References

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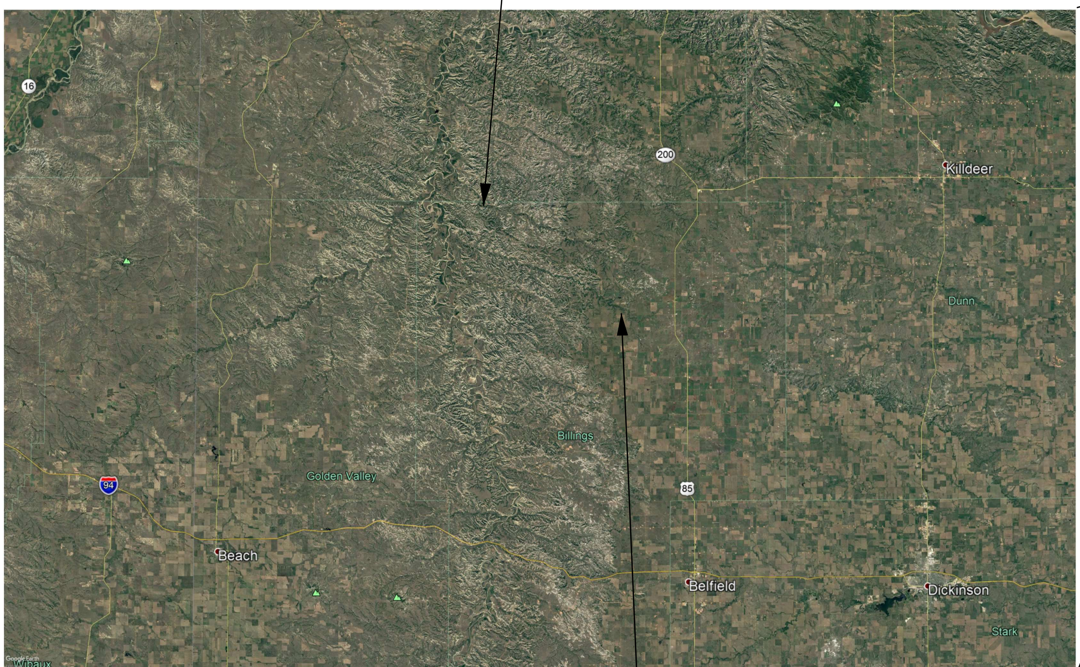
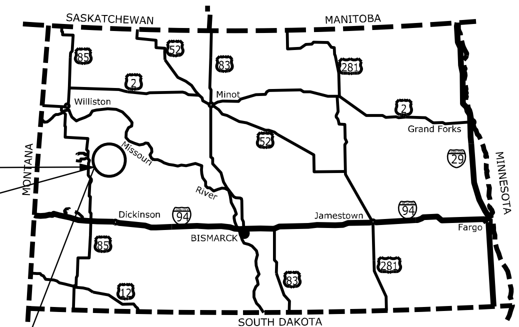
STATE	PROJECT	PLATE
ND	ND FLAP 704(1) & 795(1) Magpie and Whitetail Crossing	1

ND 704(1) Magpie Crossing



ND 795(1) Whitetail Crossing

Project Location
ND FLAP 704 (1) & 795(1)



KEY MAP OF NORTH DAKOTA

N

NO SCALE

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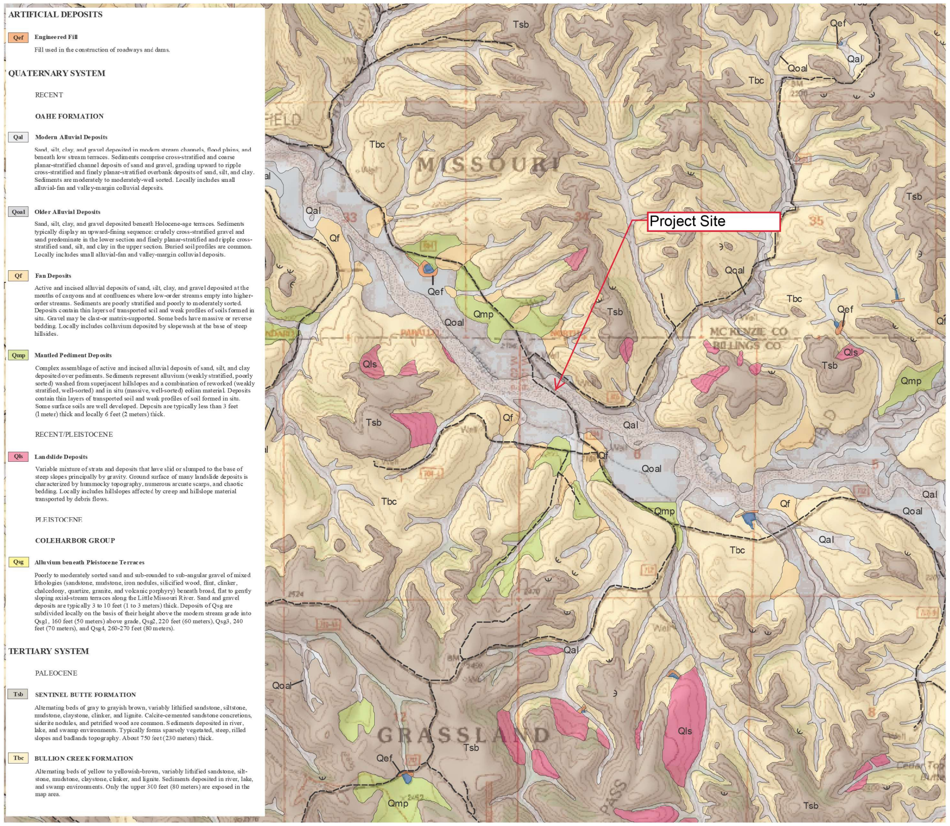
FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

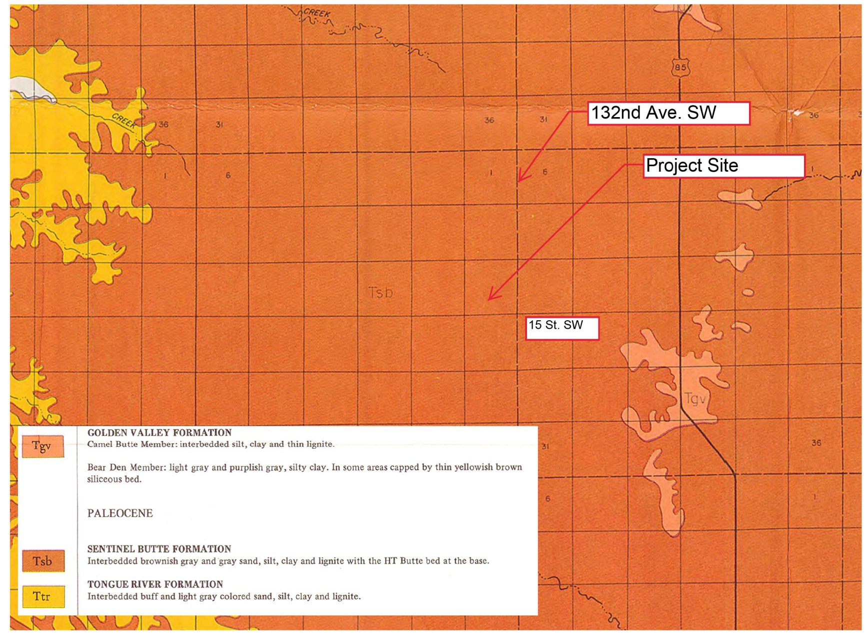
LOCATION MAP

STATE	PROJECT	PLATE
ND	ND FLAP 704(1) & 795(1) Magpie and Whitetail Crossing	2

ND 704(1) Magpie Crossing



ND 795(1) Whitetail Crossing



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FEDERAL HIGHWAY ADMINISTRATION
CENTRAL FEDERAL LANDS HIGHWAY DIVISION

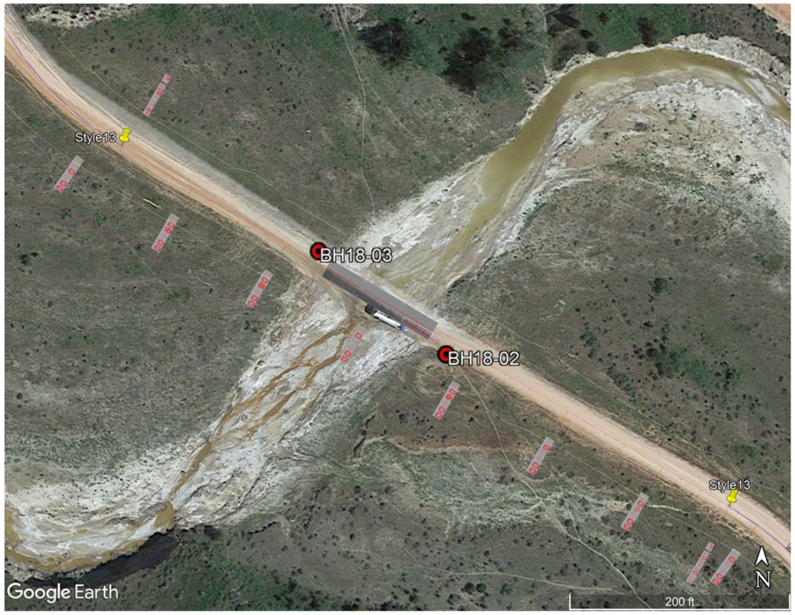
GEOLOGY MAP

STATE	PROJECT	PLATE
ND	ND FLAP 704(1) & 795(1) Magpie and Whitetail Crossing	3

795(1) Whitetail Crossing

Boring Locations ⊕			
Boring	Station	Latitude	Longitude
BH18-01	204+09 34 LT	47.198020°	-103.301940°

Boring locations are approximate. Station and offsets are based on the 70% plan set dated October 2022.



704(1) Magpie Crossing

Boring Locations ⊕			
Boring	Station	Latitude	Longitude
BH18-02	106+24 8 LT	47.325106°	-103.537450°
BH18-03	107+71 12 RT	47.325365°	-103.537922°

Boring locations are approximate. Station and offsets are based on the 70% plan set dated October 2022.

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GEOTECHNICAL BORING LOCATIONS

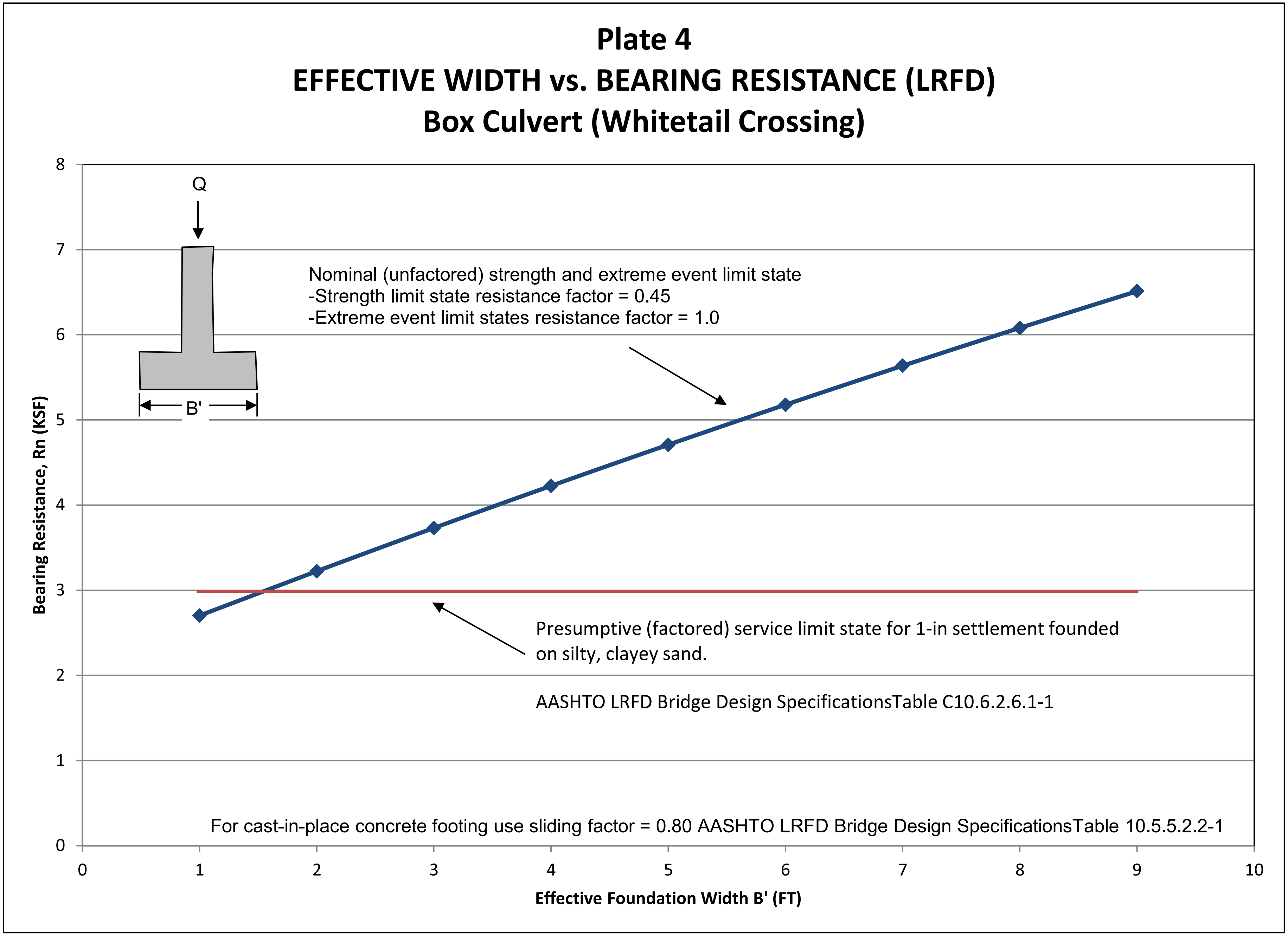


Plate 4
EFFECTIVE WIDTH vs. BEARING RESISTANCE (LRFD)
Box Culvert (Whitetail Crossing)
Q
B'
Nominal (unfactored) strength and extreme event limit state
-Strength limit state resistance factor = 0.45
-Extreme event limit states resistance factor = 1.0
Presumptive (factored) service limit state for 1-in settlement founded on silty, clayey sand.
AASHTO LRFD Bridge Design SpecificationsTable C10.6.2.6.1-1
For cast-in-place concrete footing use sliding factor = 0.80 AASHTO LRFD Bridge Design SpecificationsTable 10.5.5.2.2-1
Bearing Resistance, Rn (KSF)
Effective Foundation Width B' (FT)
0
1
2
3
4
5
6
7
8
9
10

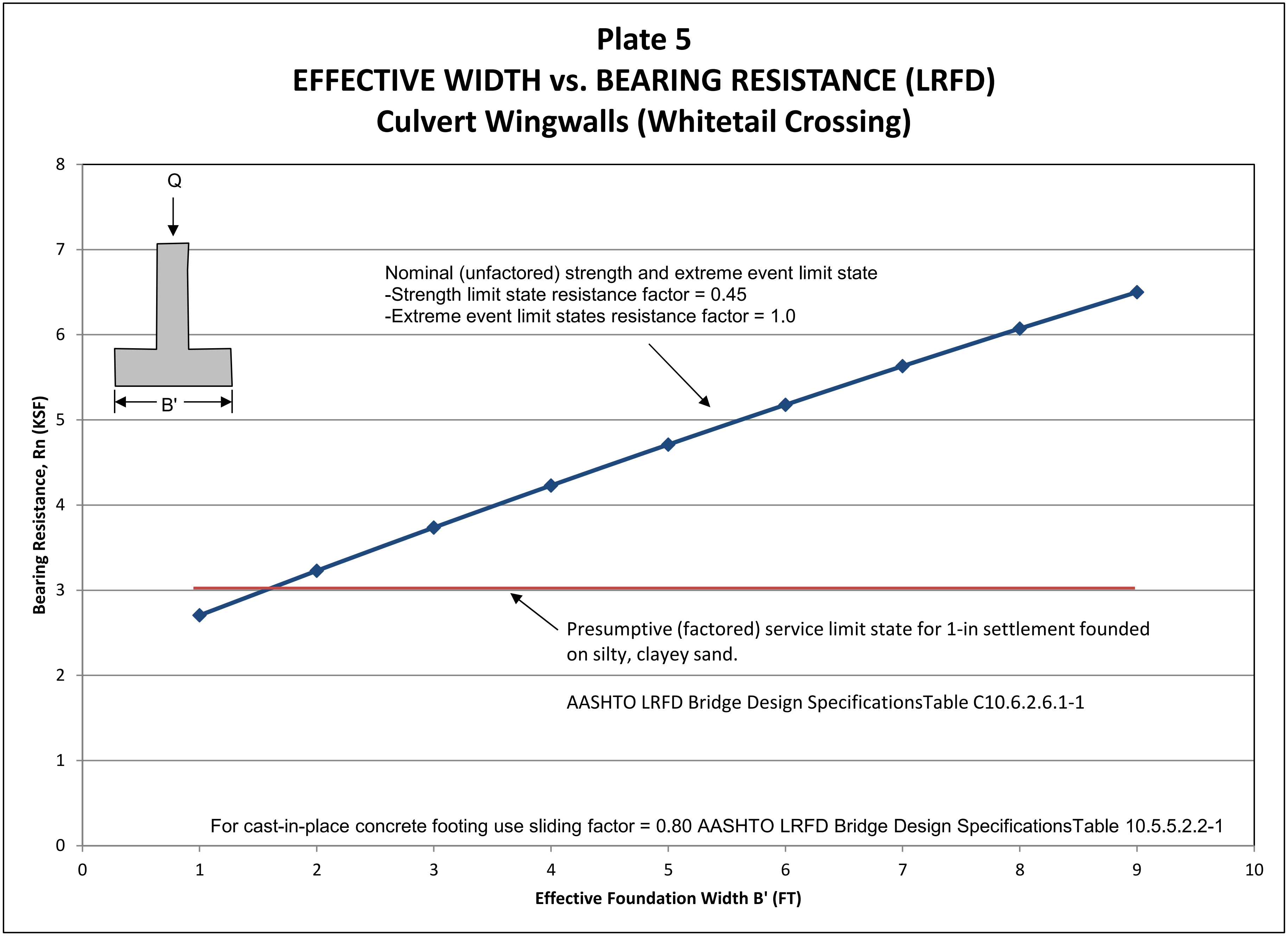


Plate 5
EFFECTIVE WIDTH vs. BEARING RESISTANCE (LRFD)
Culvert Wingwalls (Whitetail Crossing)
Q
B'
Nominal (unfactored) strength and extreme event limit state
-Strength limit state resistance factor = 0.45
-Extreme event limit states resistance factor = 1.0
Presumptive (factored) service limit state for 1-in settlement founded
on silty, clayey sand.
AASHTO LRFD Bridge Design SpecificationsTable C10.6.2.6.1-1
For cast-in-place concrete footing use sliding factor = 0.80 AASHTO LRFD Bridge Design SpecificationsTable 10.5.5.2.2-1
Bearing Resistance, Rn (KSF)
Effective Foundation Width B' (FT)
0
1
2
3
4
5
6
7
8
9
10

APPENDIX A

FIELD EXPLORATION PROGRAM

APPENDIX A

FIELD EXPLORATION PROGRAM

A.1 INTRODUCTION

The Central Federal Lands Highway Division (CFLHD) Geotechnical Section completed a field exploration program for the ND FLAP 704(1) & 795(1), Magpie and Whitetail Crossings project, on October 24, 2018. The scope of work for the geotechnical field exploration program included drilling two (2) borings at the proposed culvert location at Magpie Creek crossing, and one (1) boring at the proposed culvert location at Whitetail Creek crossing, totaling three (3) borings. The drilling component of the field exploration program was coordinated and observed by CFLHD Geotechnical personnel. Field exploration locations are illustrated on the "Geotechnical Boring Locations" sheet, Plate 3. Individual boring logs are attached. These logs represent a compilation of field and laboratory data and description of the soil and rock by CFLHD Geotechnical personnel. The methods used to conduct the field exploration program are described below. Photos of drilling equipment and field exploration activities are included in Appendix D. All soil samples collected during the field exploration program were transported to the CFLHD Materials Laboratory in Lakewood, Colorado for testing. A summary of the laboratory testing program is provided in Appendix B.

A.2 EXPLORATIONS

Materials Testing Services of Minot, North Dakota, provided drilling services for the soil borings. Boring were completed using a truck mounted CME-45 drill rig.

Borings were drilled on October 24, 2018. A total of three borings (BH18-01 to BH18-03) were completed to depths of approximately 40 feet below ground surface where culverts are proposed. Borings were drilled and sampled using 4-inch diameter solid-stem augurs and split barrel SPT sampling.

If water was encountered at the time of drilling, field personnel measured water levels in the borings. Fluctuations in the ground water level due to seasonal and climatic effects are expected. The location of individual borings were estimated relative to existing features shown on the pre-scoping project plans. Elevations were determined using CFLHD survey data. Boring locations are listed on individual boring logs and are shown on Plate 3.

A.3 SOIL AND ROCK SAMPLING

Disturbed samples were obtained from the borings in accordance with the Standard Penetration Test (SPT), the procedures of which are detailed in AASHTO T-206. The SPT involves driving a 2-inch outside diameter, 1.375-inch inside diameter split spoon sampler a depth of 18 inches with a 140-pound hammer falling a distance of 30 inches. The number of blows required to advance the split-spoon sampler through each of the 6-inch increments was recorded. The SPT resistance, or N-value, is defined as the number of blows required to drive the sampler over the second and third 6-inch increments. The N-value provides a means for evaluating the relative density or compactness of cohesionless (granular) soils and consistency or stiffness of cohesive (fine-grained) soils. An energy corrected N-value, N_{60}, is used to standardize the energy levels of the hammer system in the SPT to 60% efficiency. The automatic hammer system employed for the SPT's on this project is assumed to have an efficiency of 80%. Representative portions of the split-spoon samples obtained in conjunction with the SPT were placed in plastic baggies and transported to the CFLHD Materials Laboratory for testing.

A.4 SOIL AND ROCK CLASSIFICATION SYSTEMS

During the completion of borings, CFLHD Geotechnical personnel collected soil samples and prepared field logs of the borings. Soil identification and descriptions, as shown on the field logs, are based on ASTM D2488, a systematic process for identifying and describing individual soil samples by visual and manual means. When sufficient laboratory testing was completed, select samples from borings were classified using the Unified Soil Classification System (USCS) and American Association of State Highway and Transportation Officials (AASHTO) soil classification system. Both the visual soil identification system and the referenced soil classification systems are summarized in the attached Soil Classification Field Reference.

Central Federal Lands Soil Description Field Reference

Use the following descriptive sequence when classifying soils, both in the field and when entering data into gINT:

1.	Group Name (Pg. 3)	9.	Cementation (Pg. 2)
2.	Group Symbol (Pg. 3)	10.	Organics
3.	Consistency / Relative Density (Pg. 1)	11.	Dry Strength (Pg. 2)
4.	Color (Pg. 1)	12.	Dilatancy (Pg. 2)
5.	Moisture (Pg. 1)	13.	HCL Rxn (Pg. 2)
6.	Particle Size / Shape / Angularity (Pg. 1)	14.	Odor (Pg. 2)
7.	Plasticity (Pg. 2)	15.	Staining (Pg. 2)
8.	Structure (Pg. 2)		

Example, fine-grained soil:	**Lean CLAY with Sand (CL)-** stiff, brown, moist, medium plasticity, laminated
Example, coarse-grained soil:	**Silty SAND with Gravel (SM)-** medium dense, gray, wet, fine to coarse grained, angular to subangular gravel, weakly cemented
Example, fine-grained soil (Long Form):	**Clayey GRAVEL with SAND (CL-ML)-** loose to soft, dark brownish green to pale brownish gray, wet; fine to medium grained, angular, flat sand; coarse grained, rounded elongated gravel, some chert, trace coarse gravel, and cobbles, medium plasticity, dessicated, weak cementation, low dry strength, rapid dilatancy, moderate HCL reaction, hydrocarbon odor, iron oxide staining, alluvium fill, (Quaternary Alluvium), Additional Description.

3. Consistency and Density:

SAND & GRAVEL		SILT AND CLAY				
N	**Density**	**N**	**Consistency**	**Unconfined Compressive Strength q_u (tsf)**	**Undrained Compressive Strength s_u (tsf)**	**Behavior**
0-4	Very Loose	0-1	Very Soft	<0.25	<0.125	Extrudes between fingers when squeezed
5-10	Loose	2-4	Soft	0.25-0.50	0.125-0.25	Remolded by light finger pressure
11-30	Medium Dense	5-8	Firm	0.50-1.00	0.25-0.50	Imprinted easily with fingers, remolded by strong finger pressure
31-50	Dense	9-15	Stiff	1.00-2.00	0.50-1.00	Imprinted with considerable finger pressure, indented by finger nail
>50	Very Dense	16-30	Very Stiff	2.00-4.00	1.00-2.00	Barely imprinted by fingers or indented by finger nail
		>30	Hard	>4.00	>2.00	Not imprinted by fingers or difficult to indent with finger nail

4. Color

Use primary colors or hyphenated compound primary colors. Use "mottled" or "streaked" if necessary.

5. Moisture Content

Dry	Dry to touch, dusty
Moist	Damp but no visible water
Wet	Visible free water

6a. Particle Size

Material		Particle Size Sieve	Approximate Scale
Silt or Clay		< #200	Flour or smaller
Sand	Fine	> #200 to #40	Flour to sugar
	Medium	#40 to #10	Sugar to rock salt
	Course	#10 to #4	Rock salt to pea-sized
Gravel	Fine	#4 to 3/4 in.	Pea-sized to thumb
	Coarse	3/4 in. to 3 in.	Tumb to fist
	Cobble	3 in. to 12 in.	Fist to basketball
	Boulders	> 12 in.	Larger than Basketball

6b. Particle Angularity

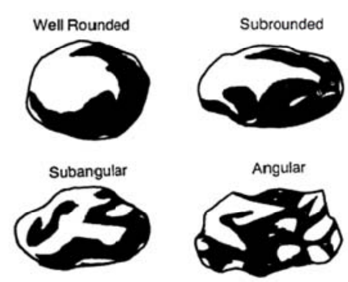
Applies to coarse sand, gravel, cobbles and boulders.

Angular	Sharp edges and relatively plane sides.
Subangular	Same as angular with rounded edges.
Subrounded	Nearly plane sides but well-rounded corners and edges.
Rounded	Smooth curved sides and no edges.
Well-Rounded	Very Smooth surfaces, spherical or ovular, no edges.

6c. Particle Shape

Applies to sand, gravel, cobbles and boulders. Length, width and thickness refer to the greatest, intermediate and least dimensions, respectively.

Flat	Width/Thickness >3
Elongated	Length/Width >3
Flat & Elongated	Meets both of the above



Central Federal Lands Soil Description Field Reference

7. Plasticity

On the basis of observations made during the toughness test, describe plasticity.

Toughness test: Shape the specimen into an elongated pat and rolled on a smooth surface or between the palms into a thread ~1/8". If the sample is too wet to roll, it should be allowed to dry. Fold the thread and reroll repeatedly until the thread crumbles at a diameter of ~1/8". This will be near the plastic limit. Note the pressure required to roll the thread near the plastic limit and the strength of the thread. After the thread crumbles, the pieces should be lumped together and kneaded until the lump crumbles. Note the toughness during kneading.

Nonplastic	Thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time kneading and rolling to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

8. Structure Terms

Stratified	Alternating layers > 1/4 inch, note thickness.
Laminated	Alternating layers < 1/4 inch, note thickness.
Fissured	Contains shears or separations along planes of weakness.
Slickensided	Shear planes appear polished or glossy, sometimes striated.
Blocky	Cohesive soil can be broken down into harder, angular lumps.
Lensed	Inclusion of small pockets of different soils, note thickenss.
Homogeneous	Same color and appearance throughout.
Mottled	Spots or patches possessing no apparent pattern
Varved	Distinct laminations of lacustrine sediments predominantly clayey
Desiccated	Shrinkage cracks due to dehydration of fine-clayey soil

9. Cementation

Intact coarse-grained soil

Weak	Crumbles with little finger pressure
Moderate	Crumbles with considerable finger pressure
Strong	Will not crumble with finger pressure

11. Dry Strength

1. Mold a ball ~1" diameter until it has the consistency of putty, adding water if necessary.
2. From the ball, make at least 3 ½" diameter balls. Allow to air dry.
3. If the specimen contains natural dry lumps, those that are ~1/2" diameter may be used in place of molded balls.
4. Test the strength of the dry balls or lumps by crushing between the fingers.

None	Crumbles into powder with mere pressure of handling.
Low	Crumbles into powder with some finger pressure.
Medium	Breaks into pieces with considerable finger pressure.
High	Cannot be broken with finger pressure, will break between hard surface and thumb.
Very High	Cannot be broken between hard surface and thumb.

12. Dilatancy

1. Mold soil, adding water if necessary, into ~1/2" diameter ball with soft but not sticky consistency.
2. Smooth in palm of one hand with knife blade. Shake horizontally, striking the side of the hand vigorously against the other hand several times. Note the reaction of water appearing on the surface.
3. Squeeze by closing the hand or pinching the soil between fingers. The reaction is the speed with which water appears while shaking and disappears while squeezing.
4. After Dilatancy has been determined perform the Toughness test (see explanation in #7).

None	No visible change
Slow	Water appears slowly during shaking and does not disappear or disappears slowly during squeezing.
Rapid	Water appears quickly during shaking and disappears quickly during squeezing.

13. HCL Reaction

None	No visible reaction
Weak	Some reaction, bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

14. Odor

None
Chemical
Hyrodcarbon
Organic

15. Staining

None
Hyrodcarbon
Iron Oxide

Central Federal Lands Soil Description Field Reference

1. & 2. Group Name & Group Symbol

Fine-Grained Soils (50% or more fines)

Group Symbol	Coarse Fraction	Coarse Fraction	Sand or Gravel	Group Name
CL	<30% plus No.200	<15% plus No.200		Lean CLAY
		15-25% plus No.200	% sand ≥ % gravel	Lean CLAY with SAND
			% sand < % gravel	Lean CLAY with GRAVEL
	≥30% plus No.200	% sand ≥ % gravel	< 15% gravel	SANDY lean CLAY
			≥ 15% gravel	SANDY lean CLAY with GRAVEL
		% sand < % gravel	< 15% sand	GRAVELLY lean CLAY
			≥ 15% sand	GRAVELLY lean CLAY with SAND
ML	<30% plus No.200	<15% plus No.200		SILT
		15-25% plus No.200	% sand ≥ % gravel	SILT with SAND
			% sand < % gravel	SILT with GRAVEL
	≥30% plus No.200	% sand ≥ % gravel	< 15% gravel	SANDY SILT
			≥ 15% gravel	SANDY SILT with GRAVEL
		% sand < % gravel	< 15% sand	GRAVELLY SILT
			≥ 15% sand	GRAVELLY SILT with SAND
CH	<30% plus No.200	<15% plus No.200		Fat CLAY
		15-25% plus No.200	% sand ≥ % gravel	Fat CLAY with SAND
			% sand < % gravel	Fat CLAY with GRAVEL
	≥30% plus No.200	% sand ≥ % gravel	< 15% gravel	SANDY fat CLAY
			≥ 15% gravel	SANDY fat CLAY with GRAVEL
		% sand < % gravel	< 15% sand	GRAVELLY fat CLAY
			≥ 15% sand	GRAVELLY fat CLAY with SAND
MH	<30% plus No.200	<15% plus No.200		Elastic SILT
		15-25% plus No.200	% sand ≥ % gravel	Elastic SILT with SAND
			% sand < % gravel	Elastic SILT with GRAVEL
	≥30% plus No.200	% sand ≥ % gravel	< 15% gravel	SANDY elastic SILT
			≥ 15% gravel	SANDY elastic SILT with GRAVEL
		% sand < % gravel	< 15% sand	GRAVELLY elastic SILT
			≥ 15% sand	GRAVELLY elastic SILT with SAND
OL/ OH	<30% plus No.200	<15% plus No.200		ORGANIC SOIL
		15-25% plus No.200	% sand ≥ % gravel	ORGANIC SOIL with SAND
			% sand < % gravel	ORGANIC SOIL with GRAVEL
	≥30% plus No.200	% sand ≥ % gravel	< 15% gravel	SANDY ORGANIC SOIL
			≥ 15% gravel	SANDY ORGANIC SOIL with GRAVEL
		% sand < % gravel	< 15% sand	GRAVELLY ORGANIC SOIL
			≥ 15% sand	GRAVELLY ORGANIC SOIL with SAND

Coarse-Grained Soils (less than 50% fines)

	Fines	Grade	Type of Fines	Group Symbol	Sand/Gravel	Group Name
Gravel	≤ 5%	Well		GW	< 15% sand	Well-graded GRAVEL
					≥ 15% sand	Well-graded GRAVEL with SAND
		Poorly		GP	< 15% sand	Poorly-graded GRAVEL
					≥ 15% sand	Poorly-graded GRAVEL with SAND
	10%	Well	ML or MH	GW-GM	< 15% sand	Well-graded GRAVEL with SILT
					≥ 15% sand	Well-graded GRAVEL with SILT and SAND
			CL or CH	GW-GC	< 15% sand	Well-graded GRAVEL with CLAY
					≥ 15% sand	Well-graded GRAVEL with CLAY and SAND
		Poorly	ML or MH	GP-GM	< 15% sand	Poorly-graded GRAVEL with SILT
					≥ 15% sand	Poorly-graded GRAVEL with SILT and SAND
			CL or CH	GP-GC	< 15% sand	Poorly-graded GRAVEL with CLAY
					≥ 15% sand	Poorly-graded GRAVEL with CLAY and SAND
	≥ 15%		ML or MH	GM	< 15% sand	SILTY GRAVEL
					≥ 15% sand	SILTY GRAVEL with SAND
			CL or CH	GC	< 15% sand	CLAYEY GRAVEL
					≥ 15% sand	CLAYEY GRAVEL with SAND
Sand	≤ 5%	Well		SW	< 15% gravel	Well-graded SAND
					≥ 15% gravel	Well-graded SAND with GRAVEL
		Poorly		SP	< 15% gravel	Poorly-graded SAND
					≥ 15% gravel	Poorly-graded SAND with GRAVEL
	10%	Well	ML or MH	SW-SM	< 15% gravel	Well-graded SAND with SILT
					≥ 15% gravel	Well-graded SAND with SILT and GRAVEL
			CL or CH	SW-SC	< 15% gravel	Well-graded SAND with CLAY
					≥ 15% gravel	Well-graded SAND with CLAY and GRAVEL
		Poorly	ML or MH	SP-SM	< 15% gravel	Poorly-graded SAND with SILT
					≥ 15% gravel	Poorly-graded SAND with SILT and GRAVEL
			CL or CH	SP-SC	< 15% gravel	Poorly-graded SAND with CLAY
					≥ 15% gravel	Poorly-graded SAND with CLAY and GRAVEL
	≥ 15%		ML or MH	SM	< 15% gravel	SILTY SAND
					≥ 15% gravel	SILTY SAND with GRAVEL
			CL or CH	SC	< 15% gravel	CLAYEY SAND
					≥ 15% gravel	CLAYEY SAND with GRAVEL

Central Federal Lands Soil Description Field Reference

SUBSURFACE INVESTIGATION CHECKLIST

PRE-DRILLING	1	has the work order been prepared?
	2	Is the project # or identifier provided?
	3	Has contact info been provided?
	4	Have directions to the site been provided?
	5	Is a description of the anticipated subsurface strata available?
	6	Is there drilling water available?
	7	Is the boring on/near water?
	8	Have utilities been cleared?
	9	Is traffic control necessary? Who is providing TC?
	10	Have any required permits been obtained?
	11	Have site access instructions been provided?
	12	Has a boring layout been provided?
	13	Is info for each boring provided that includes:
		Boring # of identifier
		Total depth
		Sample intervals
		Sample types
		Boring termination criteria
	15	Are there any special instruction for sampling bore sealing, instrumentation, monitoring wells, ground water level readings, field testing, etc?
EQUIPMENT	16	Does the rig have the proper capabilities?
	17	If rock coring; is the rig equipped with gauges that display the drilling pressure applied?
	18	Are the casing and rod the correct sizes and types?
	19	Is there significant length of casing and rod? Are they Straight?
	20	Are the proper samplers provided?
		Are the samplers and thin walls complete and in proper working condition?
		Are there liners and catchers for the split-spoons?
	21	Are the correct bits provided and in good condition?
	22	Do you have the proper sampler containers and transport equipment?
	23	Do you have the proper borehole sealing materials?
	24	Do you have the proper forms? Logging & reference sheets, and this checklist.
DRILLING	25	Is the boring in the correct location?
	26	Was the boring moved from the planned location? (Who authorized)?
	27	Has the boring location been measured off of known landmarks?
	28	Will the drill mast be cleared from overhead utilities?
	29	Are the underground utility markings at a safe distance from the boring?
	30	Is the rig level and plumb?
	31	Is drilling fluid being used? Is it mixed according the mfg recommendations?
	32	If a portable sump is to be used:
		Does it have baffels?
		Is it sealed at the bottom of the tub and ground surface interface or casing?
SOIL SAMPLING	33	Are the requested samplers being used?
	34	Does the borehole appear to be clean prior to sampling?
		Is H.S.A canter plug being used?
	35	Are the samples taken at the correct depths?
	36	Is the requested sample interval being adhered to?
	37	For split spoon sampling:
		Are the tests conducted properly? (ASTM D 1586, AASHTO T206)
		Is sample recovery measured?
		Are samples placed in moisture proof containers, label & stored properly? (ASTM 4220)
	38	For thin walled tube samples:
		Are the tests conducted properly? (ASTM D 1587, AASHTO T207, ASTM D 6519)
		Is sample recovery measured?
		Are the tubes handled with minimal disturbance?
		Are the tubes sealed properly? (ASTM D 1587)
		Are the tubes labeled and stored properly? (ASTM D 1587)
	39	For other soil sample types:
		Are the samples obtained properly (ASTM D 1452)
		Are the samples placed in appropriate moisture proof containers?
		Are the samples labeled & stored properly?

ROCK SAMPLING	40	Is the hole clean and the test performed properly? (ASTM D 2113)
	41	Is the core barrel lowered to the proper depth?
	42	Is the requested length of run being performed?
	43	Is the core barrel; single, double, triple, carbide, diamond, in good condition?
	44	Coring time per foot or per run recorded?
	45	Is down pressure recorded?
	46	Is the removed core specimen handled, boxed and labeled properly? (ASTM M 5079)
	47	Were breaks in the core properly labeled? (ASTM M 5079)
	48	Was the percent recovery (REC) calculated? (ASTM D 2113)
	49	Was the Rock Quality Designation (RQD) calculated? (ASTM D 6032)
WATER LEVEL READINGS – ON LAND	50	Was the ground water level encountered drilling and recorded?
	51	Was the ground water level recorded at hole completion? (ASTM D 4750)
	52	Was GW recorded after stabilizing 24 hrs? (ASTM D 4750)
WATER LEVEL READINGS – ON WATER	53	Was there a change of water level during drilling?
	54	Were changes in water level documented (e.g. tides)?
	55	Was the estimated ground elevation noted?
SEALING	56	Was the borehole sealed in a proper manner?
	57	Was the sealing material type and amount recorded?
PIEZOMETERS/ WELLS	58	Was the instrumentation installed to the correct depth?
	59	Was the instrumentation installed in accordance with the provided instructions?
	60	Was well development performed?
GENERAL SITE OBSERVATIONS	61	Is there any change is surface elevation over the site?
	62	Are there any surface anomalies or irregularities? (i.e. rock outcrops, springs, slope distress, excavations)
	63	Are there any ponds, ditches, or standing water on-site or immediately adjacent?
	64	Are there any wells on-site?
	65	Does the site appear to receive run-off from adjacent properties?
	66	is there any evidence of past fill placement, debris or dumping on-site?
	67	Is there any significant vegetation change over part of the site?
	68	Is there any evidence of surface depressions?
	69	Is there any evidence of distress in adjacent structures?
FIELD LOG	70	Project name and Number
	71	Boring # or identifier
	72	Start date and Completion date including time of day
	73	Boring location (offset/ direction and distance)
	74	Ground water surface elevation. (depth, date, time)
	75	Rig type and Drilling method
	76	Rod and casing sizes OD & ID
	77	Hammer type (Auto/Manuel)
	78	Sample numbers and depths
	79	Sample descriptions
	80	Blows per 6" increments for split spoon
	81	Thin-wall tube/piston sampler type
		Recovery lengths
	82	Rock coring data:
		Run length and time of run
		Drilling pressure/ comments
		%REC and RQD
		Core description
	83	Stratum breaks/ mat'l changes
	84	Bottom of boring depth
	85	Notes/ Comments on:
		Weather (Cloudy, clear, rainy, temp.)
		Losses of circulation (depth; est. amount)
		Rod drops (depth form-depth to) for voids
		Obvious changes in drilling (hard/soft)
PHOTOS	86	Drill rig up and down station for each hole.
	87	Samples prior to shipment. (Core samples should be wet down prior to photo for consistency in color).
	88	General site photographs
	89	Any noted anomalies seen on-site.

Central Federal Lands Rock Core Description Field Reference

Core should be placed in core boxes from left to right, top to bottom. The rock description for each core run should include, in this order:

1	Rock Type (CAPITAL LETTERS) (Pg.1)	6	Discontinuities (Pg.2)
2	Color (Pg.1)		a. Type e. Separation
3	Grain Size or Bedding (Pg.1)		b. Stratification f. Infilling & Weathering
4	Weathering (Pg.1)		c. Spacing g. Roughness
5	Strength (Pg.1)		d. Orientation
		7	Miscellaneous (Pg.2)
		8	Formation or Unit Name (CAPITAL LETTERS)

EXAMPLES

GNEISS- Dark gray, moderately weathered, strong. Biotite foliation, low angle, close. Quartz veins, close, low angle, stepped. Primary joint set, close, low angle, tight, moderately weathered, very narrow with rust surface staining and spotty clay infilling, rough planar. (SLIVER PLUME GRANITE)
GRANODIORITE- Grey to white, medium grained, slightly weathered, strong, joints are moderate to high angle, very close, rough, open to closed, FE stained joints. Poor Circulation.
SANDSTONE- Tan to reddish brown, fine to medium grained, sub rounded, thinly bedded, moderately weathered, strong, joints are low angle, very close, closed, rough. FE surface staining throughout sample, some organics seen in joint sets.

1. Rock Type

Common classifications; gneiss, granite, shale, etc. A modifier may be necessary to describe a sedimentary rock formed from a combination of soil types, i.e., Silty SANDSTONE.

2. Color

For consistency, describe when wet. Use primary or hyphenated compound primary colors.

3a. Grain Size

V. Coarse Grained	> 1/4 in.	
Coarse Grained	3/16-1/4 in.	Easily distinguished by naked eye
Medium Grained	1/16-3/16 in.	Can be distinguished by naked eye
Fine Grained	Up to 1/16 in.	Barley distinguished by naked eye
V. Fine Grained		Cannot distinguished by naked eye

3d. Structure

For Sedimentary Rock

Banded
Bedded
Cross Bedded
Flow Banded
Foliated
Interbedded
Laminated
Massive

3c. Bedding

For Sedimentary Rock

V. thickly bedded	> 3ft
Thickly bedded	18 in. - 3 ft
Thinly bedded	2-18 in.
V. thinly bedded	3/8- 2 in.
Laminated	3/16- 3/8 in.
Thinly laminated	< 3/16 in.

3b. Grain Shape

For Sedimentary Rock

Angular	Show very little wear, grain edges are sharp
Subangular	Show definite effects of wear, grain edges slightly rounded
Subrounded	Shows considerable wear , grain edges rounded smooth
Rounded	Shows extreme wear, grain edges smoothed to broad curves
Well-Rounded	Very Smooth surfaces, spherical or ovular, no edges.

4. Weathering

Fresh	No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces.
Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker externally than in its fresh condition.
Moderately weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework as corestones.
Highly weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework as corestones.
Completely weathered (Decomposed)	All rock material is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.
Residual Soil	All rock mass is converted to a soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the volume has not been significantly transported.

5. Description of Relative Strength / Hardness

Grade	Description	Field Identification	psi
R0	Extremely weak rock	Indented by thumbnail.	50-150
R1	Very weak rock	Crumbles under firm blows with point of geological hammer, can peeled with pocket knife.	150-750
R2	Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentation made by firm blow with point of geologic al hammer.	750- 3,500
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife, specimen c an be fractured with single firm blow of geological hammer.	3,500-7,500
R4	Strong rock	Specimen requires more than one blow of geological hammer to fracture it.	7,500-15,000
R5	Very strong rock	Spec imen requires many blows of geologic al hammer to fracture it.	15,000- 35,000
R6	Extremely strong rock	Specimen can only be chipped with geological hammer.	> 35,000

Central Federal Lands Rock Core Description Field Reference

6a. Discontinuities

Foliation	Planar arrangement of textural features, usually applied to schistocity or cleavage
Vein	A body of minerals intruded into a joint or fault
Joint	A break of structural origin with no visible displacement
Shear	A discontinuity along which sufficient differential displacement has occurred to produce slickensides
Fault	Major discontinuity with significant displacement, with gouge or adjacent zone of severely fractured rock
Shear or Fault Zone	Band of closely spaced discontinuities along which differential movement has occurred
Bedding	A layered arrangement within the rockmass predominatly sedimentary rock.

6b. Stratification Discontinuities

Lamination	Thin beds (<3/8 in.)
Fissile	Tendency to break along laminations
Parting	Tendency to break parallel to bedding, any scale
Foliation	Segregation and layering of minerals in metamorphic rocks

6c. Spacing Discontinuities

Perpendicular distance between the planes of the discontinuities.

Very Wide	Greater than 10 ft
Wide	3 – 10 ft.
Moderately Close	1 – 3 ft.
Close	2 in. – 1 ft.
Very Close	Less than 2 in.

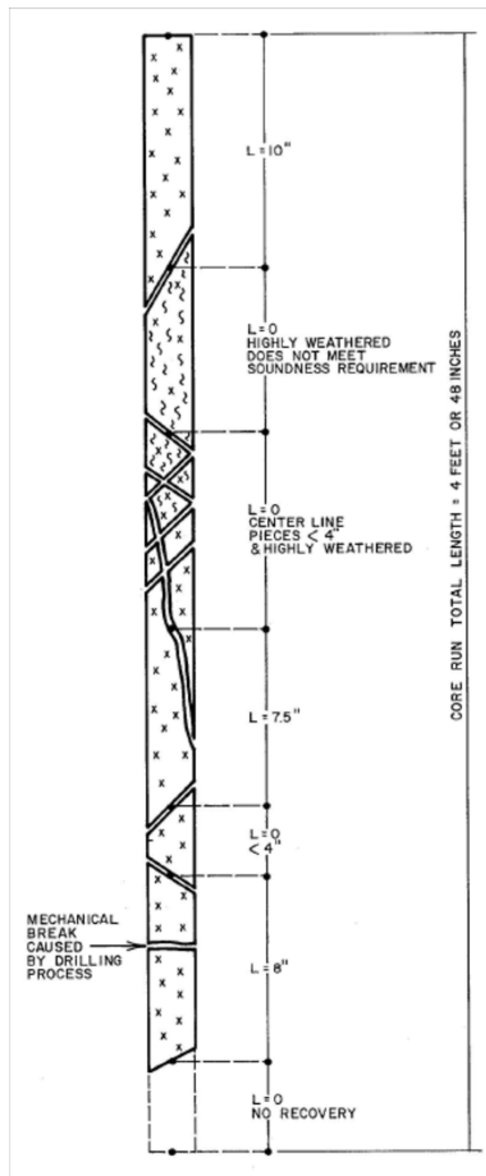
6d. Orientation Discontinuities

Dip angle of discontinuity should be measured with protractor to perpendicular from core axis (0° is perpendicular, 90° is parallel). To describe range of orientations, use the following terms:

Horizontal (for vertical boreholes)	0° - 5°
Low Angle	5°- 35°
Moderate Angle	35° - 55°
High Angle	55° - 85°
Vertical (for vertical boreholes)	85° - 90°

6f. Infilling Discontinuities

Types of common infilling materials include: clay, calcite, chlorite, iron oxide, gypsum/talc., pyrite, quartz, and sand.



6e. Separation Discontinuities

Note: These terms are for core logging, others that describe opening width should be used for outcrop mapping.

Healed	Breaks easily or with difficulty, hairline or seam, usually with infilling.
Closed	Seen as a hairline trace, no infilling.
Open	Core pieces separated or easily separated, may have staining or mineralization on joint surfaces.

6g. Roughness Discontinuities

Large scale – planar, stepped, or undulating. Small scale – use the following terms:

Slickensided	Smooth, glassy surface sometime with striations.
Smooth	Looks and feels smooth.
Slightly Rough	Asperities are distinguishable and can be felt.
Rough	Some ridges and steps are evident, asperities are clearly visible, surface feels very abrasive.
Very Rough	Near-vertical steps and ridges.

$$RQD = \frac{\sum \text{LENGTH OF SOUND CORE PIECES} > 4 \text{ INCHES (100mm)}}{\text{TOTAL CORE RUN LENGTH}}$$

$$RQD = \frac{10 + 7.5 + 8}{48} \times 100\%$$

RQD = 53% (FAIR)

RELATION OF RQD & ROCK QUALITY	
RQD, Rock Quality Designation %	Description of Rock Quality
0-25	Very Poor
25-50	Poor
50-75	Fair
75-90	Good
90-100	Excellent

Core Measurements

Recovery = Total length of recovered core / Total length of run
RQD = Total length of core pieces > 4 in. / Total length of run
(RQD may also be calculated separately for different rock types in one run – be consistent by project.)

7a. Vesicularity

For volcanics only

Term	% by Volume
Some Vesicles	5-25
Highly Vesicular	15-50
Scoriaceous	Greater than 50

7b. Moisture

Damp
Dripping
Dry
Flowing
Wet

7c. Staining

Iron Oxide
Hydrocarbon
None

7d. Odor

None
Slight
Moderate
Strong

7e. HCL Reaction

None	No visible reaction
Weak	Some reaction, bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

U. S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
FEDERAL LANDS HIGHWAY DIVISION

BORING LOG LEGEND

Project Name: Magpie and Whitetail Crossings
Project Location: Billings County, ND

SAMPLE TYPE SYMBOLS

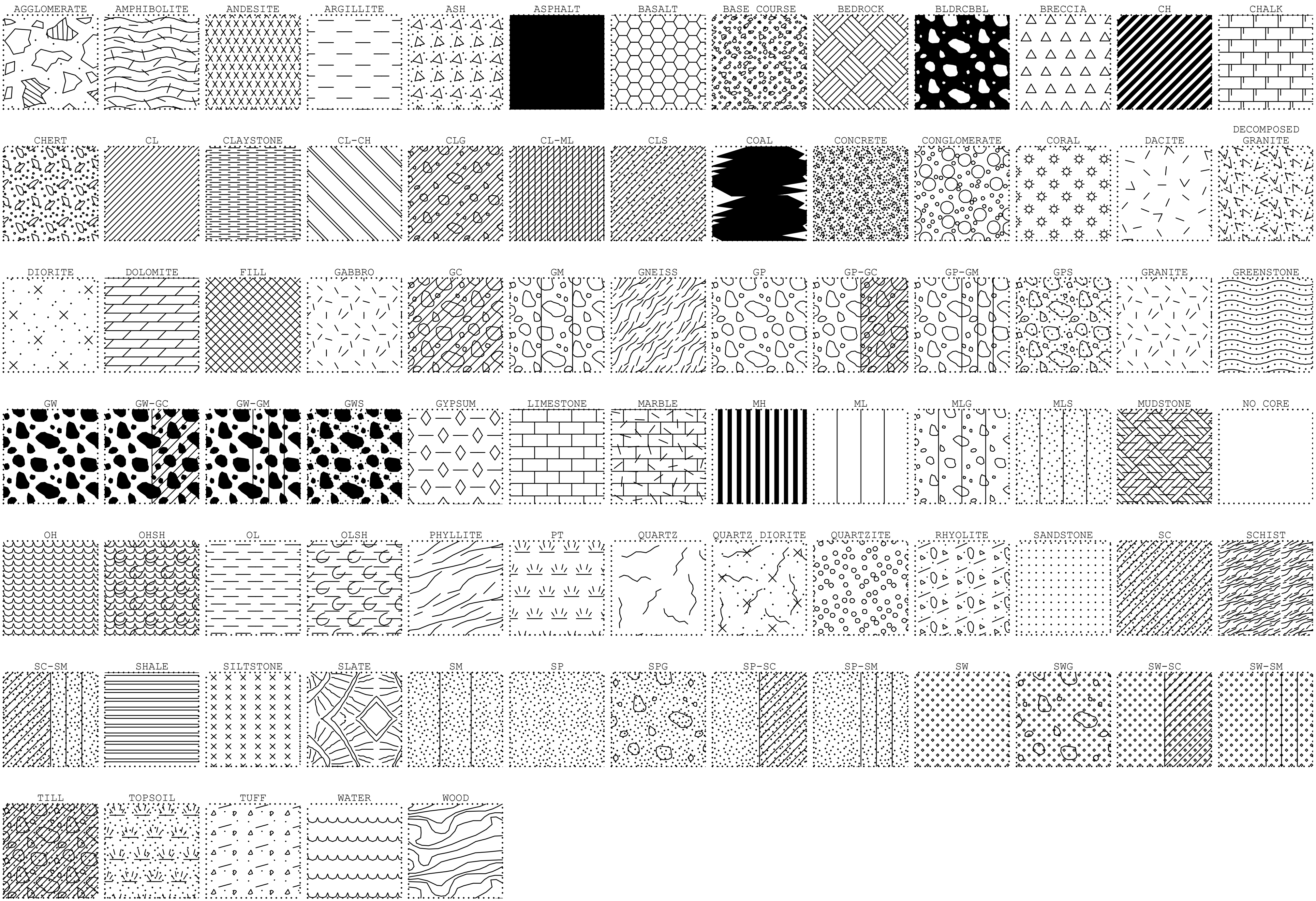
Standard Penetration Test (2" OD)

DRILLING METHOD SYMBOLS

Solid-Stem Auger

ABBREVIATIONS

FF	-	Fracture Frequency (fractures per foot)
Fines	-	Percent Passing No. 200 Sieve
LL	-	Liquid Limit (%)
NP	-	Non-Plastic
PL	-	Plastic Limit (%)
PP	-	Pocket Penetrometer Reading
Rec	-	Rock Core Recovery
RQD	-	Rock Quality Designation
SG	-	Specific Gravity
UC	-	Unconfined Compressive Strength
VWP	-	Vibrating Wire Piezometer
WC	-	Water Content (%)

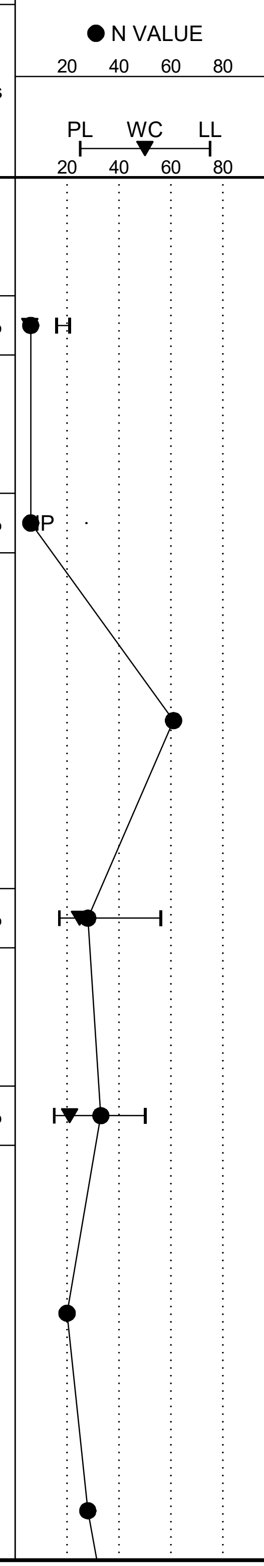

AGGLOMERATE
AMPHIBOLITE
ANDESITE
ARGILLITE
ASH
ASPHALT
BASALT
BASE COURSE
BEDROCK
BLDRCBBL
BRECCIA
CH
CHALK
CHERT
CL
CLAYSTONE
CL-CH
CLG
CL-ML
CLS
COAL
CONCRETE
CONGLOMERATE
CORAL
DACITE
DECOMPOSED GRANITE
DIORITE
DOLOMITE
FILL
GABBRO
GC
GM
GNEISS
GP
GP-GC
GP-GM
GPS
GRANITE
GREENSTONE
GW
GW-GC
GW-GM
GWS
GYPSUM
LIMESTONE
MARBLE
MH
ML
MLG
MLS
MUDSTONE
NO CORE
OH
OHSH
OL
OLSH
PHYLLITE
PT
QUARTZ
QUARTZ DIORITE
QUARTZITE
RHYOLITE
SANDSTONE
SC
SCHIST
SC-SM
SHALE
SILTSTONE
SLATE
SM
SP
SPG
SP-SC
SP-SM
SW
SWG
SW-SC
SW-SM
TILL
TOPSOIL
TUFF
WATER
WOOD

U. S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
FEDERAL LANDS HIGHWAY DIVISION

BORING LOG BH18-01

Project Name:	Magpie and Whitetail Crossings		
Project Location:	Billings County, ND	Surface Elevation:	2528 ft
Groundwater Depth:		Station and Offset:	204+09 34 ft Lt.
⊻ While Drilling:	9 ft / Elev 2519 ft	Latitude:	47.19802° Longitude: -103.30194°
At Completion:	---	State Plane Coors:	N 5228568.3 ft E 628616.83 ft
After Drilling:	---	Date Started:	10/24/18 Date Completed: 10/24/18
Notes:		Driller/Company:	Craig/Materials Testing Services, LLC Drill: CME-45
Whitetail Crossing, East Abutment Area		Hammer Type:	140 lbs Automatic
		Logger/Company:	JMA/FHWA
		Weather:	Sunny, 40's

Elevation (ft)	Depth (ft)	Material Description	Drilling Method	Sample No.	Field Blow Count (Recovery)	Test Results
2528	0	Silty SAND (SC-SM), loose, light brown, medium sand, non plastic to low plasticity. Dry to wet.	4" Solid Auger			
2525	3			1	2-3-3 (9" = 50%)	Fines = 31%
2520	8	Coarse sand and gravel.		2	2-2-4 (16" = 89%)	Fines = 26%
	9	Caving soils encountered from 9 to 12 feet.				
2514.5	13.5	El. 2514.5 ft 13.5 ft LIGNITE, very stiff, black, moist, blocky.		3	6-34-27 (12" = 67%)	
2513	15	El. 2513 ft 15 ft Fat CLAY with sand (CH), very stiff, light to dark gray, moist, fine sand, medium to high plasticity, laminated.				
2510	18			4	7-12-16 (18" = 100%)	Fines = 90%
2505	23	Increased sand content		5	8-13-20 (14" = 78%)	Fines = 89%
2500	28			6	5-9-11 (18" = 100%)	
	30	Minor calcareous inclusions between 30' and 35'				
	32	Coal inclusions and minor beds (<2" thick) between 30' and 40'				
2495	33			7	9-12-16 (18" = 100%)	



U. S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
FEDERAL LANDS HIGHWAY DIVISION

BORING LOG BH18-01

Project Name: Magpie and Whitetail Crossings

Project Location: Billings County, ND

Elevation (ft)	Depth (ft)	Graphic Log	MATERIAL DESCRIPTION	Drilling Method	SAMPLE Type	No.	Field Blow Count (Recovery)	Test Results	N VALUE / PL WC LL
2490			Fat CLAY with sand (CH), very stiff, light to dark gray, moist, fine sand, medium to high plasticity, laminated. *(continued)* El. 2488.5 ft 39.5 ft	4" Solid Auger		8	12-16-26 (18" = 100%)		

Bottom of borehole at 39.5 ft.

U. S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
FEDERAL LANDS HIGHWAY DIVISION

BORING LOG BH18-02

Project Name: Magpie and Whitetail Crossings	
Project Location: Billings County, ND	Surface Elevation: 2138 ft
Groundwater Depth:	Station and Offset: 106+24 8 ft Lt.
⊻ While Drilling: 3 ft / Elev 2135 ft	Latitude: 47.325106° Longitude: -103.53745°
At Completion: ---	State Plane Coors: N 5242330.12 ft E 610513.99 ft
After Drilling: ---	Date Started: 10/24/18 Date Completed: 10/24/18
Notes:	Driller/Company: Craig/Materials Testing Services, LLC Drill: CME-45
Magpie Crossing, Southeast Abutment Corner.	Hammer Type: 140 lbs Automatic
	Logger/Company: JMA/FHWA
	Weather: Sunny, 50's

Elevation (ft)	Depth (ft)	MATERIAL DESCRIPTION	Drilling Method	Sample Type	No.	Field Blow Count (Recovery)	Test Results	N VALUE (20 40 60 80) / PL WC LL (20 40 60 80)
2138–2130.5	0–7.5	Silty SAND (SC-SM), very loose, brown to dark gray, wet, non plastic to low plasticity, with gravel. El. 2130.5 ft 7.5 ft	4" Solid Auger		9	3-1-2 (6" = 33%)	Fines = 21%	
2130.5–2128	7.5–10	LIGNITE, very stiff, black, fine sand, non plastic, laminated, trace fine sand El. 2128 ft 10 ft			10	0-0-100 (5" = 28%)		
2128–	10–	Fat CLAY (CH), very stiff, light to dark gray, moist, medium to high plasticity, with silt, trace fine sand, Coal inclusions and lenses, 1/4" to 2" thick.. SHALE, light to dark grey, fine grained, slightly weathered, very thinly bedded, interbedded, extremely weak rock (R0). iron oxide staining, Bullion Creek Formation.			11	12-20-34 (18" = 100%)	Fines = 92%	
					12	25-35-65 (18" = 100%)		
					13	20-32-68 (16" = 89%)	Fines = 89%	
					14	20-32-68 (13" = 72%)		
					15	32-34-36 (18" = 100%)		

Elevation scale: 2135, 2130, 2125, 2120, 2115, 2110, 2105. Depth scale: 5, 10, 15, 20, 25, 30.

U. S. DEPARTMENT OF TRANSPORTATION
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FEDERAL LANDS HIGHWAY DIVISION

BORING LOG BH18-02

Project Name: Magpie and Whitetail Crossings

Project Location: Billings County, ND

Elevation (ft)	Depth (ft)	Graphic Log	MATERIAL DESCRIPTION	Drilling Method	SAMPLE Type	No.	Field Blow Count (Recovery)	Test Results	N VALUE (20 40 60 80) / PL WC LL (20 40 60 80)
2100			Fat CLAY (CH), very stiff, light to dark gray, moist, medium to high plasticity, with silt, trace fine sand, Coal inclusions and lenses, 1/4" to 2" thick.. SHALE, light to dark grey, fine grained, slightly weathered, very thinly bedded, interbedded, extremely weak rock (R0). iron oxide staining, Bullion Creek Formation. *(continued)*	4" Solid Auger					
			El. 2098.5 ft 39.5 ft			16	0-28-100 (10" = 56%)		>>

Bottom of borehole at 39.5 ft.

U. S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
FEDERAL LANDS HIGHWAY DIVISION

BORING LOG BH18-03

Sheet: 1 of 2

Project Name: Magpie and Whitetail Crossings
Project Location: Billings County, ND
Groundwater Depth:
While Drilling: 4 ft / Elev 2133 ft
At Completion: ---
After Drilling: ---
Notes:
Magpie Crossing, Northwest Abutment Corner

Surface Elevation: 2137 ft
Station and Offset: 107+71 12 ft Rt.
Latitude: 47.325365° Longitude: -103.537922°
State Plane Coors: N 5242358.23 ft E 610477.78 ft
Date Started: 10/24/18 Date Completed: 10/24/18
Driller/Company: Craig/Materials Testing Services, LLC Drill: CME-45
Hammer Type: 140 lbs Automatic
Logger/Company: JMA/FHWA
Weather: Sunny, 60's

Elevation (ft)	Depth (ft)	MATERIAL DESCRIPTION	Drilling Method	SAMPLE No.	Field Blow Count (Recovery)	Test Results	N VALUE (20 40 60 80) / PL WC LL (20 40 60 80)
2137–2132	0–5	Silty SAND (SC-SM), very loose, brown to dark gray, non plastic to low plasticity, with gravel, dry to wet, fine to coarse sand.. El. 2132 ft 5 ft	4" Solid Auger	17	3-2-1 (12" = 67%)	Fines = 6%	NP
2132–2130	5–7	Lean CLAY (CL), very stiff, light gray to gray, moist, medium plasticity, with fine sand. El. 2130 ft 7 ft					
2130–2129	7–8	LIGNITE, very stiff, black, moist, non plastic, blocky, trace fine sand. El. 2129 ft 8 ft					
2129–	8–	Lean CLAY (CL), very stiff, light gray to gray, moist, medium plasticity, lensed, with coal, trace fine sand, Coal inclusions and lenses present throughout, 1/4" to 2" thick.. SHALE, light to dark grey, fine grained, slightly weathered, very thinly bedded, interbedded, extremely weak rock (R0). iron oxide staining, Bullion Creek Formation.		18	12-17-20 (14" = 78%)	Fines = 98%	
		More sandy with 1/4" coal inclusions.		19	0-22-100/0" (12" = 100%)		>>
		Hard layer					
				20	17-17-18 (15" = 83%)	Fines = 54%	NP
				21	21-19-25 (14" = 78%)		
		1" Thick Coal Bed					
				22	22-26-29 (18" = 100%)		
				23	0-36-60/0"		>>

U. S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
FEDERAL LANDS HIGHWAY DIVISION

BORING LOG BH18-03

Project Name: Magpie and Whitetail Crossings

Project Location: Billings County, ND

Elevation (ft)	Depth (ft)	Graphic Log	MATERIAL DESCRIPTION	Drilling Method	SAMPLE Type	No.	Field Blow Count (Recovery)	Test Results	N VALUE (20 40 60 80) / PL WC LL (20 40 60 80)
2100			Lean CLAY (CL), very stiff, light gray to gray, moist, medium plasticity, lensed, with coal, trace fine sand, Coal inclusions and lenses present throughout, 1/4" to 2" thick.. SHALE, light to dark grey, fine grained, slightly weathered, very thinly bedded, interbedded, extremely weak rock (R0). iron oxide staining, Bullion Creek Formation. *(continued)*	4" Solid Auger			(12" = 100%)		
			1" Thick Coal Bed			24	0-34-60/0" (10" = 83%)		>>
	40		El. 2097 ft — 40 ft						

Bottom of borehole at 40 ft.

FHWA BORING LOG - FHWA_DATATEMPLATE.GDT - 12/8/22 16:03 - N:\ND\704(1)795(1)\GEOTECH\3_DATA\BORING LOCATION & SITE MAP\ND FLAP 704(1)795(1) MAGPIE-WHITAIL.GPJ

APPENDIX B

LABORATORY TEST RESULTS

APPENDIX B

LABORATORY TEST RESULTS

B.1 INTRODUCTION

Laboratory tests for the Magpie and Whitetail Crossings project were completed on select soil samples recovered from the field exploration program in general accordance with the American Association of State Highway and Transportation Officials (AASHTO) and ASTM testing methods. The laboratory testing program was completed to provide data for engineering studies and to classify the materials into similar geologic groups. The testing program included index tests and geotechnical engineering property tests. The following sections describe the laboratory testing procedures.

B.2 INDEX TESTS

Classification and index laboratory testing included identification by visual and manual means, and tests to determine natural water content, unit weight, grain size distribution, fines content, and Atterberg limits. When sufficient laboratory testing was completed, select samples from borings were classified using the Unified Soil Classification System (USCS) and American Association of State Highway and Transportation Officials (AASHTO) soil classification system. Both the visual soil identification system and the referenced soil classification systems are summarized in the Soil Classification Field Reference in Appendix A. Index test results are presented in the attached laboratory reports. Index tests are generally conducted on disturbed or remolded soil samples. The following sections describe individual index test procedures.

Moisture Content

Water content was determined for select samples retrieved from the exploration in general accordance with AASHTO T 265 (ASTM D 2937). To perform this test method, the sample was weighed before and after oven drying, and the water content was calculated. The moisture content of soils, when combined with data obtained from other tests, produces significant information about the characteristics of the soil, including general correlations with strength, settlement, and workability.

Gradation

The grain size distribution of selected samples was determined on thirteen samples obtained from borings BH18-01 through BH18-04 in general accordance with the AASHTO T 311 and ASTM D 1140. These tests aid in the classification of soils and provide correlating data with engineering properties of soils, such as permeability, strength, swelling potential, and susceptibility to frost action.

Atterberg Limits

Liquid and plastic limit tests were performed on selected fine-grained portions from ten samples. The tests were completed in general accordance with AASHTO T 89 and T 90 (ASTM D 4318). The Atterberg limits include liquid limit (LL), plastic limit (PL), and plasticity index (PI), which is the plastic limit subtracted from the liquid limit. These limits are generally used to assist in classification of soils, to indicate soil consistency, and to provide correlation to engineering properties.

B.3 GEOTECHNICAL ENGINEERING PROPERTY TESTS FOR SOIL

Geotechnical engineering property testing for soil included corrosivity of soils. Geotechnical engineering property test results are presented in the attached laboratory reports. The following section describes the test procedures for the soil.

Corrosivity of Soils

Tests to determine the corrosivity (resistivity, pH, sulfate content, chloride content) of soils along the alignment were performed in general accordance with AASHTO T 288 (ASTM G 187), T 289, T 290, and T 291. These test results are used to determine the corrosion resistance of steel elements in contact with soil or the durability of concrete elements and geosynthetics in contact with soil. Tests for sulfate and chloride content are not required when the resistivity of selected samples is greater than 5000 ohm-centimeters.

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Federal Highway Administration

Central Federal Lands Highway Division Laboratory

An AASHTO and ISO Accredited Laboratory

Report of Soil or Aggregate Tests

AASHTO ACCREDITED

Project: North Dakota FLAP 704(1) & 795(1) Magpie and Whitetail Crossings

Submitted By: James Arthurs

Date Reported: 1/23/2019

Sample Number	Lab Number		18-2217-SB	18-2218-SB	18-2219-SB	18-2220-SB	18-2221-SB
	Hole Number		BH18-01	BH18-01	BH18-01	BH18-01	BH18-02
	Field Number		SPT-1	SPT-2	SPT-4	SPT-5	SPT-9 and SPT-17B
Sample Location	Station or Location		Not Furnished	Not Furnished	Not Furnished	Not Furnished	Not Furnished
	Offset		Not Furnished	Not Furnished	Not Furnished	Not Furnished	Not Furnished
	Depth		3 Feet	8 Feet	18 Feet	23 Feet	3 Feet
AASHTO T 11 & T 27 Washed Sieve Analysis % Passing	3"	75.0 mm					
	1 1/2"	37.5 mm					
	1"	25.0 mm					
	3/4"	19.0 mm				100	100
	1/2"	12.5 mm				98	96
	3/8"	9.5 mm		100		98	93
	#4	4.75 mm		99		97	84
	#8	2.36 mm					
	#10	2.00 mm		98		96	65
	#16	1.18 mm	100	97	100	96	58
	#30	600 µm					
	#40	425 µm	99	93	99	96	49
	#50	300 µm					
	#100	150 µm	52	50	95	92	27
	#200	75 µm	31	26	90	89	21
AASHTO T 255	Moisture, %		5.7	27.5	24.8	21.0	18.0
AASHTO T 89 & T 90	Liquid Limit		21	NV	56	50	21
	Plasticity Index		5	NP	39	35	7
Soil Classification	AASHTO M 145		A-2-4(0)	A-2-4(0)	A-7-6(37)	A-7-6(32)	A-2-4(0)
	ASTM D 2487		SC-SM	SM	CH	CH	SC-SM
AASHTO T 190	R - Value						
AASHTO T 288	Min. Resistivity, ohm x cm						
AASHTO T 289	pH						
AASHTO Method	Optimum Moisture, %						
	Maximum Dry Density, pcf						

Distribution:	Num. / Project File
Laboratory	Patrick Kowing
Geotechnical	James Arthurs
Technical Services	Gary Strike

Remarks:

Reported By:

Patrick K

Patrick Kowing
Laboratory Team Leader

Central Federal Lands Highway Division Laboratory

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Report of Soil or Aggregate Tests

AASHTO ACCREDITED

Project: North Dakota FLAP 704(1) & 795(1) Magpie and Whitetail Crossings
Submitted By: James Arthurs
Date Reported: 1/23/2019

Sample Number	Lab Number		18-2222-SB	18-2223-SB	18-2224-SB	18-2225-SB	
	Hole Number		BH18-02	BH18-02	BH18-03	BH18-03	
	Field Number		SPT-11	SPT-13	SPT-18	SPT-20	
Sample Location	Station or Location		Not Furnished	Not Furnished	Not Furnished	Not Furnished	
	Offset		Not Furnished	Not Furnished	Not Furnished	Not Furnished	
	Depth		13 Feet	23 Feet	9 Feet	19 Feet	
AASHTO T 11 & T 27 Washed Sieve Analysis % Passing	3"	75.0 mm					
	1 1/2"	37.5 mm					
	1"	25.0 mm					
	3/4"	19.0 mm					
	1/2"	12.5 mm					
	3/8"	9.5 mm					
	#4	4.75 mm		100			
	#8	2.36 mm					
	#10	2.00 mm	100	99			
	#16	1.18 mm	99	97			
	#30	600 µm					
	#40	425 µm	97	94	100	100	
	#50	300 µm					
	#100	150 µm	94	91	99	99	
	#200	75 µm	92	89	98	54	
AASHTO T 255	Moisture, %		21.4	25.9	19.5	23.9	
AASHTO T 89 & T 90	Liquid Limit		38	50	30	NV	
	Plasticity Index		19	29	12	NP	
Soil Classification	AASHTO M 145		A-6(18)	A-7-6(27)	A-6(11)	A-4(0)	
	ASTM D 2487		CL	CH	CL	ML	
AASHTO T 190	R - Value						
AASHTO T 288	Min. Resistivity, ohm x cm						
AASHTO T 289	pH						
AASHTO Method	Optimum Moisture, %						
	Maximum Dry Density, pcf						

Distribution:	Num. / Project File
Laboratory	Patrick Kowing
Geotechnical	James Arthurs
Technical Services	Gary Strike

Remarks:

Reported By:

Patrick Kowing
Patrick Kowing
Laboratory Team Leader

U.S. Department of Transportation
Federal Highway Administration

Central Federal Lands Highway Division Laboratory

An AASHTO and ISO Accredited Laboratory

Report of Soil or Aggregate Tests

AASHTO ACCREDITED

Project: North Dakota FLAP 704(1) & 795(1) Magpie and Whitetail Crossings

Submitted By: James Arthurs

Revised 2/4/2019

Date Reported: 1/28/2019

Sample Number	Lab Number		18-2216-S				
	Hole Number						
	Field Number		Bulk-1				
Sample Location	Station or Location		Magpie Streambed				
	Depth						
AASHTO T 11 & T 27 Washed Sieve Analysis % Passing	3"	75.0 mm					
	2 1/2"	63.0 mm	100				
	2"	50.0 mm	99				
	1 1/2"	37.5 mm	95				
	1"	25.0 mm	90				
	3/4"	19.0 mm	86				
	1/2"	12.5 mm	81				
	3/8"	9.5 mm	74				
	#4	4.75 mm	54				
	#8	2.36 mm					
	#10	2.00 mm	37				
	#16	1.18 mm	30				
	#40	425 µm	20				
	#100	150 µm	8				
	#200	75 µm	6.1				
AASHTO T 255	Moisture, %						
AASHTO T 89 & T 90	Liquid Limit		NV				
	Plasticity Index		NP				
Soil Classification	AASHTO M 145		A-1-a(0)				
	ASTM D 2487		SW-SM				
AASHTO T 190	R - Value						
AASHTO T 288	Min. Resistivity, ohm x cm		1,989				
AASHTO T 289	pH		8.3				
AASHTO Method	Optimum Moisture, %						
	Maximum Dry Density, pcf						
AASHTO T 290	Sulfate, ppm / %		170 / 0.017				
AASHTO T 291	Chloride, ppm / %		9 / 0.0009				

Distribution:	Num. / Project File
Laboratory	Patrick Kowing
Geotechnical	James Arthurs
Technical Services	Gary Strike

Remarks:
This report was revised to include Sulfate and Chloride content testing which was performed by a FHWA consultant, Colorado Analytical Laboratories

Reported By:

Patrick K

Patrick Kowing
Laboratory Team Leader

U.S. Department of Transportation
Federal Highway Administration

Federal Highway Administration - Central Federal Lands
12300 W. Dakota Ave
Lakewood, CO 80228

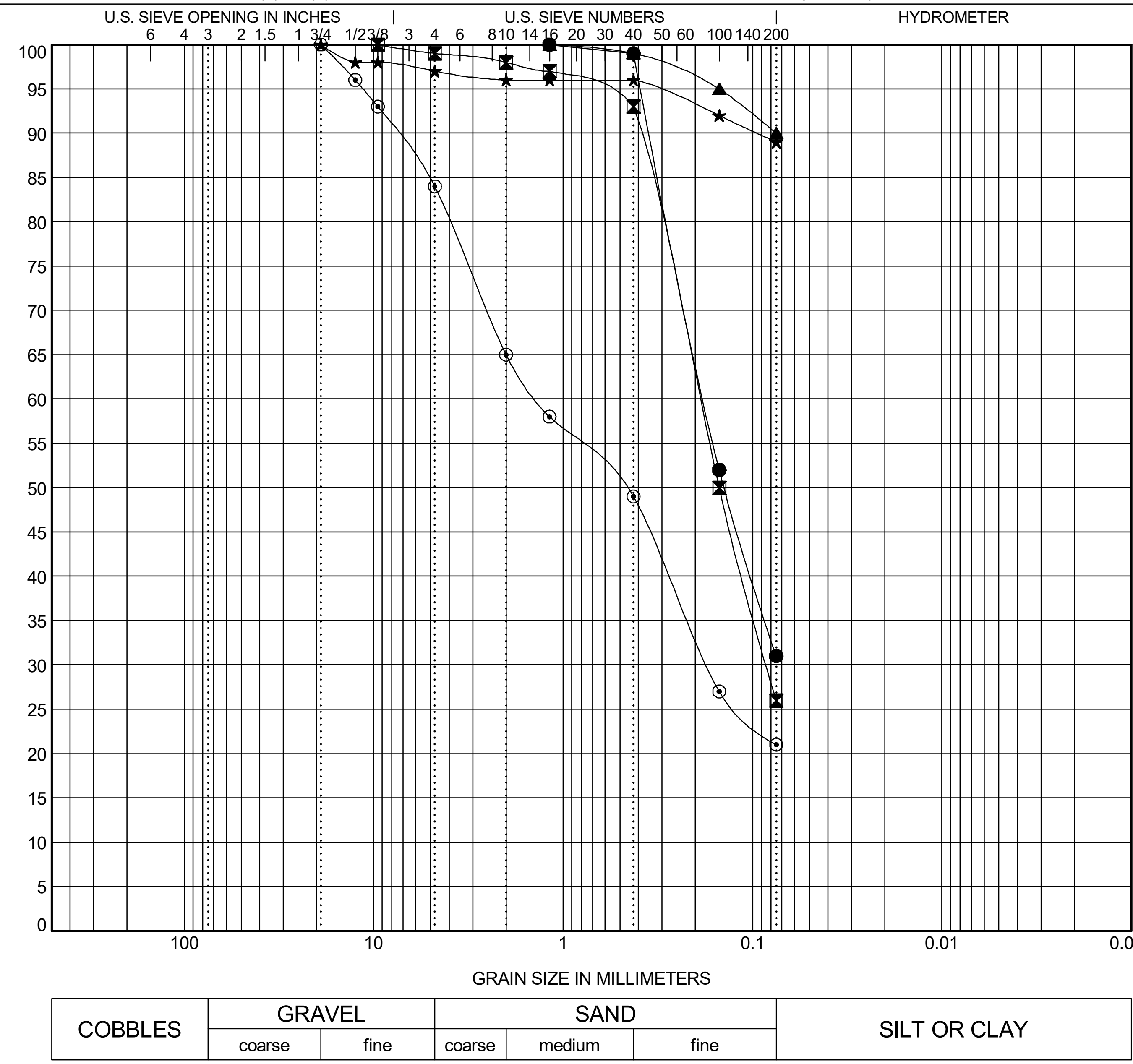
GRAIN SIZE DISTRIBUTION

CLIENT ______

PROJECT NAME Magpie and Whitetail Crossings

PROJECT NUMBER ND FLAP 704(1)795(1)

PROJECT LOCATION Billings County, ND



	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**BH18-1**	**3.0**	**SILTY, CLAYEY SAND(SC-SM)**	**21**	**16**	**5**		
⧗	**BH18-1**	**8.0**	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		
▲	**BH18-1**	**18.0**	**FAT CLAY(CH)**	**56**	**17**	**39**		
★	**BH18-1**	**23.0**	**FAT CLAY(CH)**	**50**	**15**	**35**		
⊙	**BH18-2**	**3.0**	**SILTY, CLAYEY SAND with GRAVEL(SC-SM)**	**21**	**14**	**7**		

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH18-1**	**3.0**	**1.18**	**0.179**			**0.0**	**69.0**	**31.0**	
⧗	**BH18-1**	**8.0**	**9.5**	**0.191**	**0.084**		**1.0**	**73.0**	**26.0**	
▲	**BH18-1**	**18.0**	**1.18**				**0.0**	**10.0**	**90.0**	
★	**BH18-1**	**23.0**	**19**				**3.0**	**8.0**	**89.0**	
⊙	**BH18-2**	**3.0**	**19**	**1.372**	**0.173**		**16.0**	**63.0**	**21.0**	

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Lakewood, CO 80228

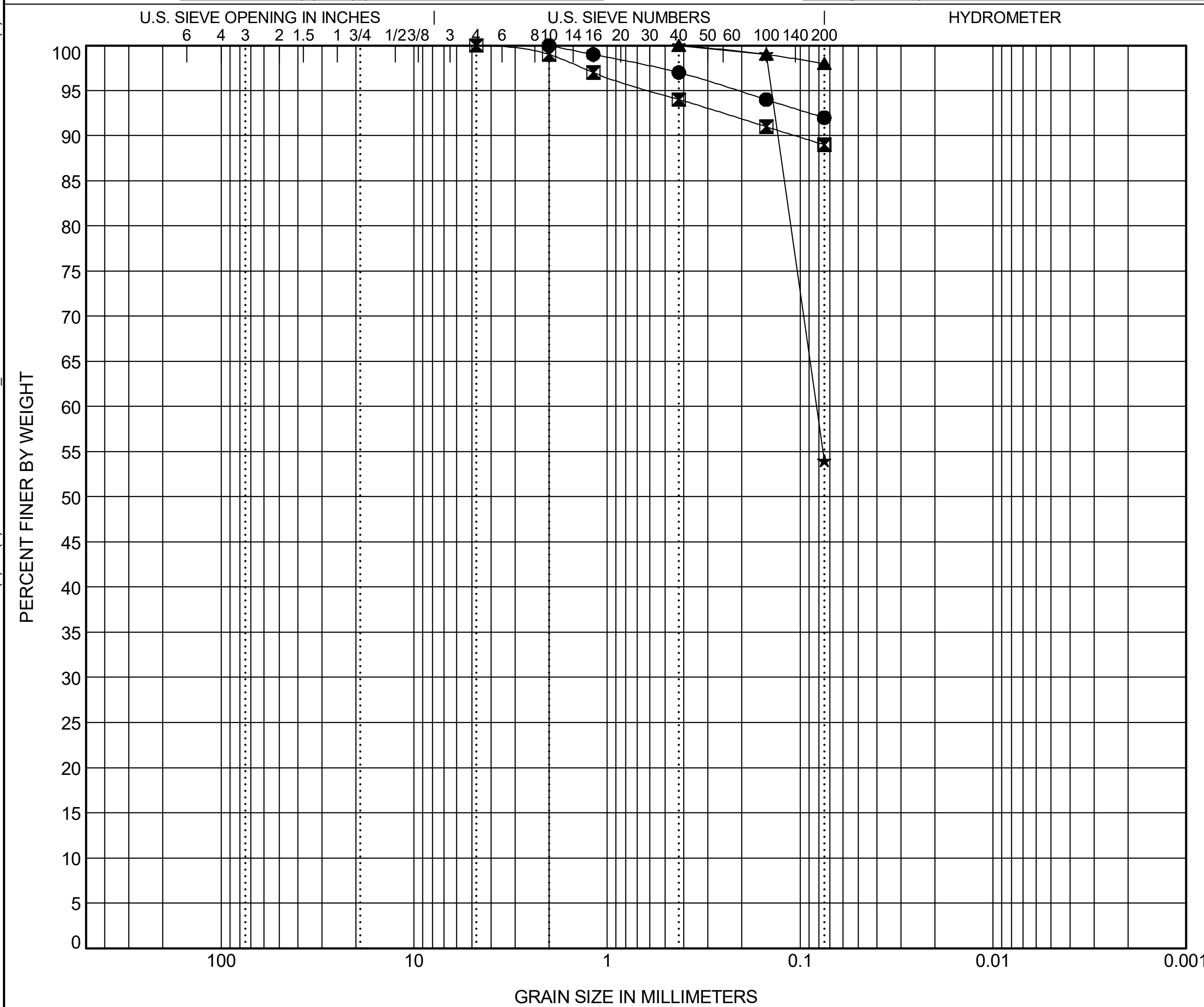
GRAIN SIZE DISTRIBUTION

CLIENT ______

PROJECT NAME Magpie and Whitetail Crossings

PROJECT NUMBER ND FLAP 704(1)795(1)

PROJECT LOCATION Billings County, ND



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**BH18-2**	**13.0**	**LEAN CLAY(CL)**	**38**	**19**	**19**		
⊠	**BH18-2**	**23.0**	**FAT CLAY(CH)**	**50**	**21**	**29**		
▲	**BH18-3**	**9.0**	**LEAN CLAY(CL)**	**30**	**18**	**12**		
★	**BH18-3**	**19.0**	**SANDY SILT(ML)**	**NP**	**NP**	**NP**		

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH18-2**	**13.0**	**2**				**0.0**	**8.0**	**92.0**	
⊠	**BH18-2**	**23.0**	**4.75**				**0.0**	**11.0**	**89.0**	
▲	**BH18-3**	**9.0**	**0.425**				**0.0**	**2.0**	**98.0**	
★	**BH18-3**	**19.0**	**0.425**	**0.082**			**0.0**	**46.0**	**54.0**	

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Federal Highway Administration

Federal Highway Administration - Central Federal Lands
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Lakewood, CO 80228

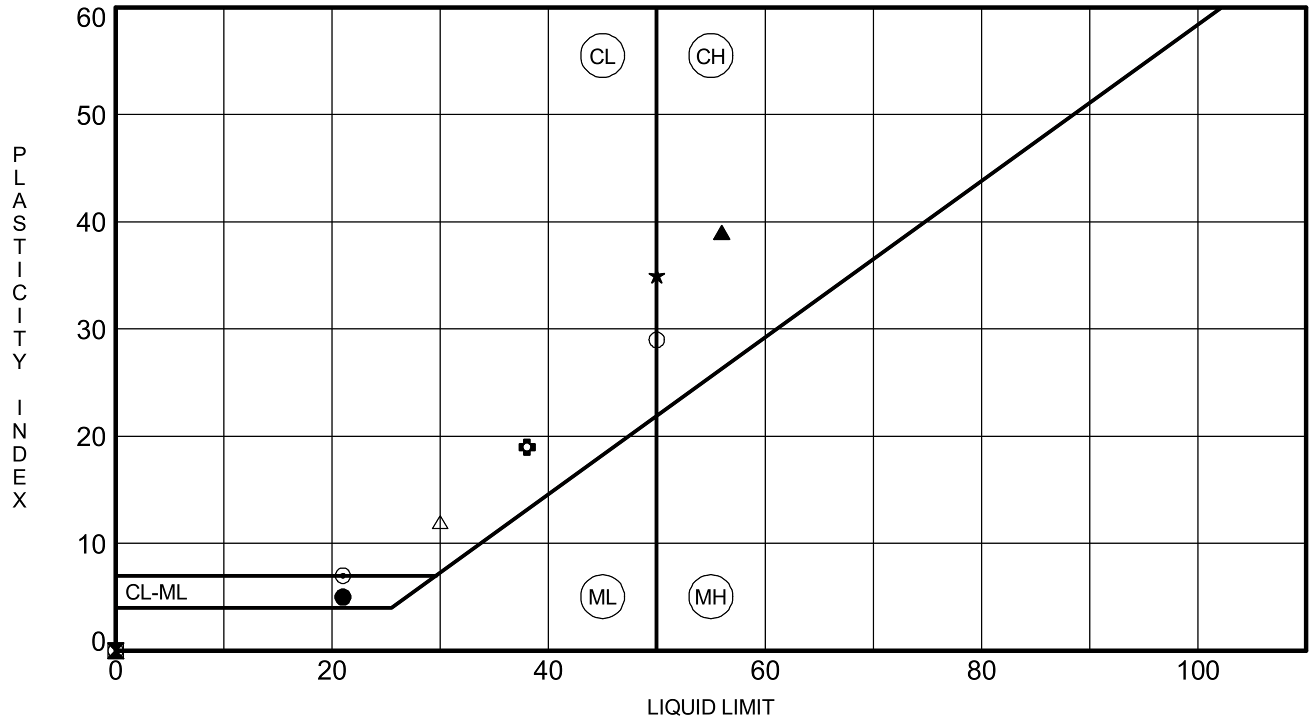
ATTERBERG LIMITS' RESULTS

CLIENT ______

PROJECT NAME Magpie and Whitetail Crossings

PROJECT NUMBER ND FLAP 704(1)795(1)

PROJECT LOCATION Billings County, ND



	BOREHOLE	DEPTH	LL	PL	PI	Fines	Classification
●	**BH18-1**	**3.0**	**21**	**16**	**5**	**31**	**SILTY, CLAYEY SAND(SC-SM)**
⧗	**BH18-1**	**8.0**	**NP**	**NP**	**NP**	**26**	**SILTY SAND(SM)**
▲	**BH18-1**	**18.0**	**56**	**17**	**39**	**90**	**FAT CLAY(CH)**
★	**BH18-1**	**23.0**	**50**	**15**	**35**	**89**	**FAT CLAY(CH)**
⊙	**BH18-2**	**3.0**	**21**	**14**	**7**	**21**	**SILTY, CLAYEY SAND with GRAVEL(SC-SM)**
✿	**BH18-2**	**13.0**	**38**	**19**	**19**	**92**	**LEAN CLAY(CL)**
○	**BH18-2**	**23.0**	**50**	**21**	**29**	**89**	**FAT CLAY(CH)**
△	**BH18-3**	**9.0**	**30**	**18**	**12**	**98**	**LEAN CLAY(CL)**
⊗	**BH18-3**	**19.0**	**NP**	**NP**	**NP**	**54**	**SANDY SILT(ML)**

APPENDIX C

PHOTOGRAPHS

DRILLING PHOTOS

Photo 1:- Borehole BH18-01

Photo 2:- Borehole BH18-01, SPT-1

Photo 3:- Borehole BH18-01, SPT-2.

Photo 4:- Borehole BH18-01, SPT-3

Photo 5:- Boring BH18-01, SPT-4

Photo 6:- Boring BH18-01, SPT-5

Photo 7:- Boring BH18-01, SPT-6

Photo 8:- Boring BH18-01, SPT-7

Photo 9:- Boring BH18-01, SPT-8

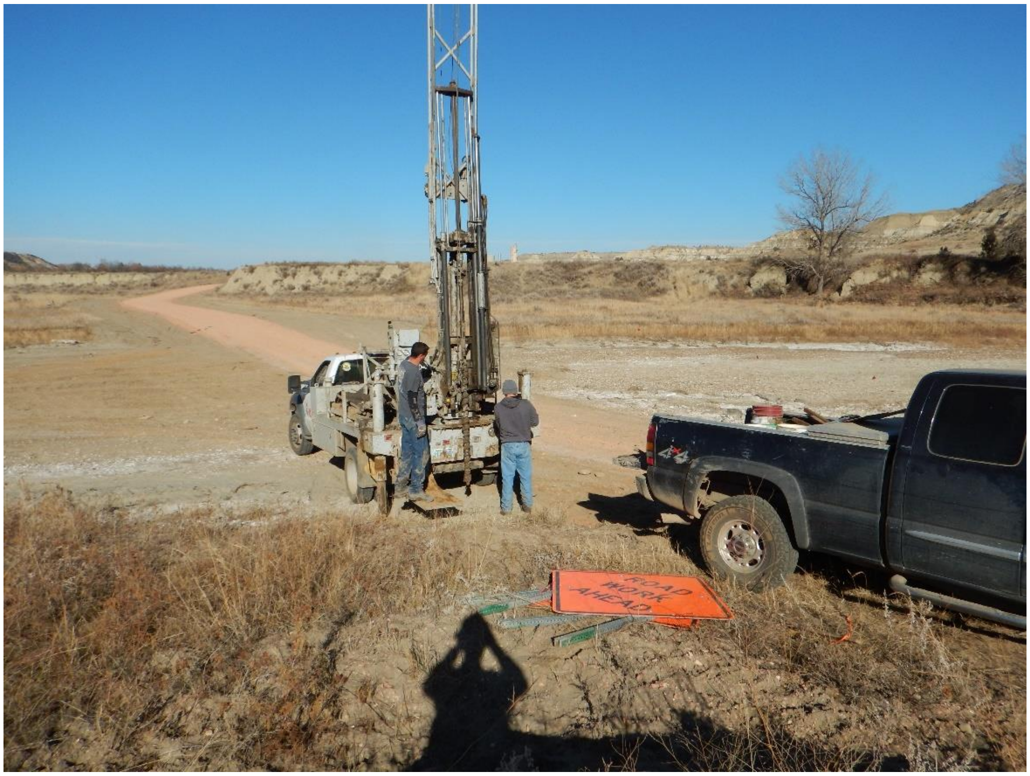


Photo 10:- Boring BH18-02

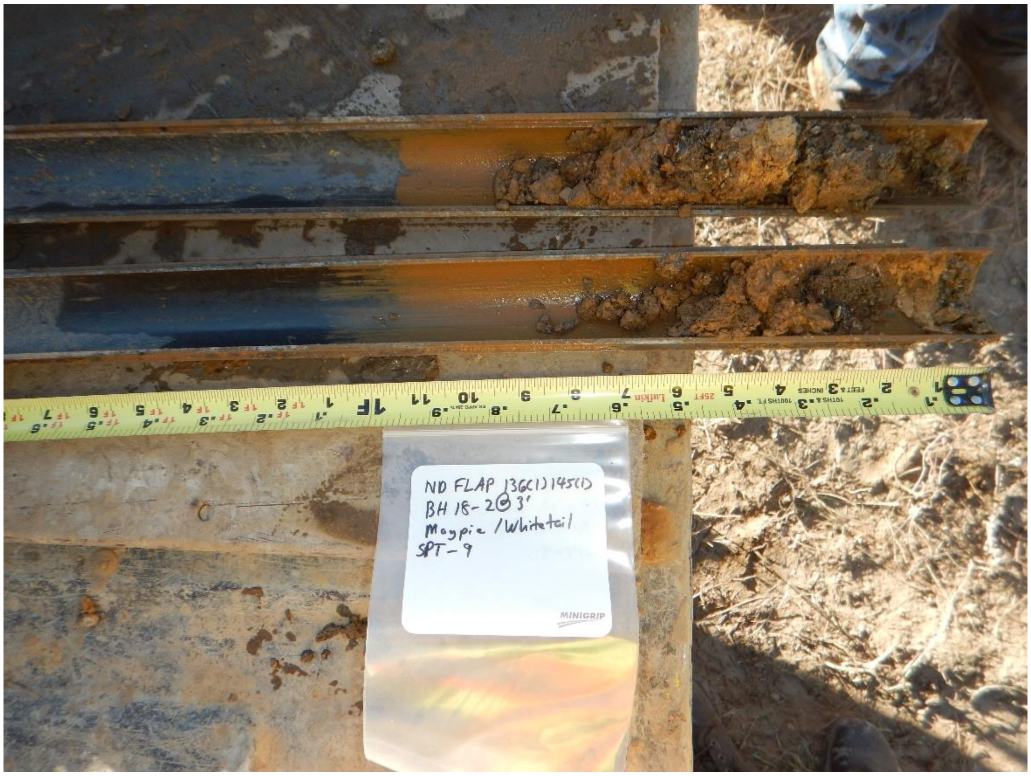


Photo 11:- Boring BH18-02, SPT-9

Photo 12:- Boring BH18-02, SPT-10

Photo 13:- Boring BH18-02, SPT-11

Photo 14:- Boring BH18-02, SPT-12

Photo 15:- Boring BH18-02, SPT-13

Photo 16:- Boring BH18-02, SPT-14

Photo 17:- Boring BH18-02, SPT-15

Photo 18:- Boring BH18-02, SPT-16

Photo 19:- Boring BH18-03

Photo 20:- Boring BH18-03, SPT-17

Photo 21:- Borehole BH18-03, SPT-18

Photo 22:- Borehole BH18-03, SPT-19

5. 

Photo 23:- Borehole BH18-03, SPT-20

Photo 24:- Borehole BH18-03, SPT-21

Photo 25:- Borehole BH18-03, SPT-22

Photo 26:- Borehole BH18-03, SPT-23

Photo 27:- Borehole BH18-03, SPT-24

EXISTING SITE CONDITIONS

Photo 28:- Existing low water crossing at Magpie Creek

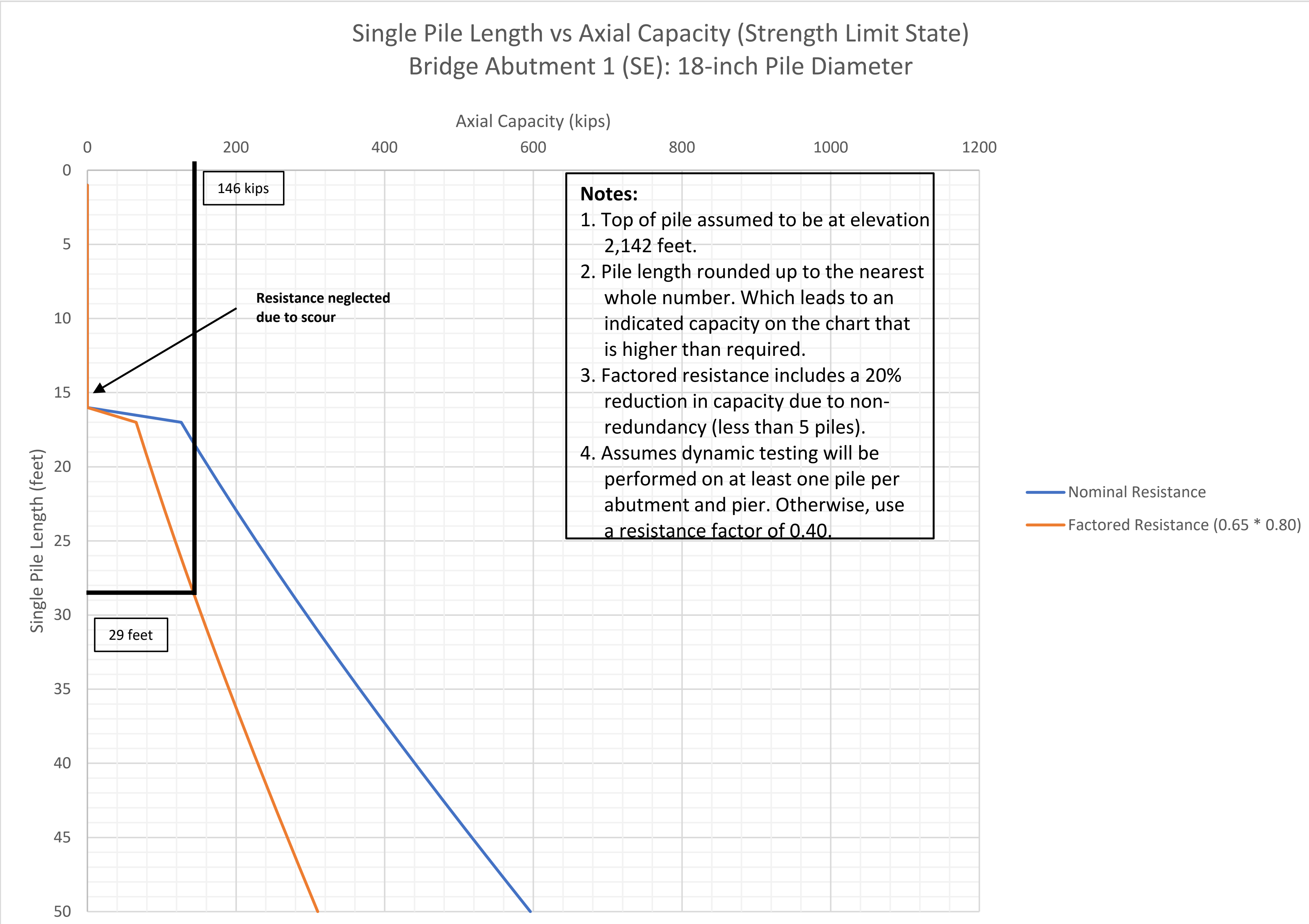
Photo 29:- Magpie Creek looking north

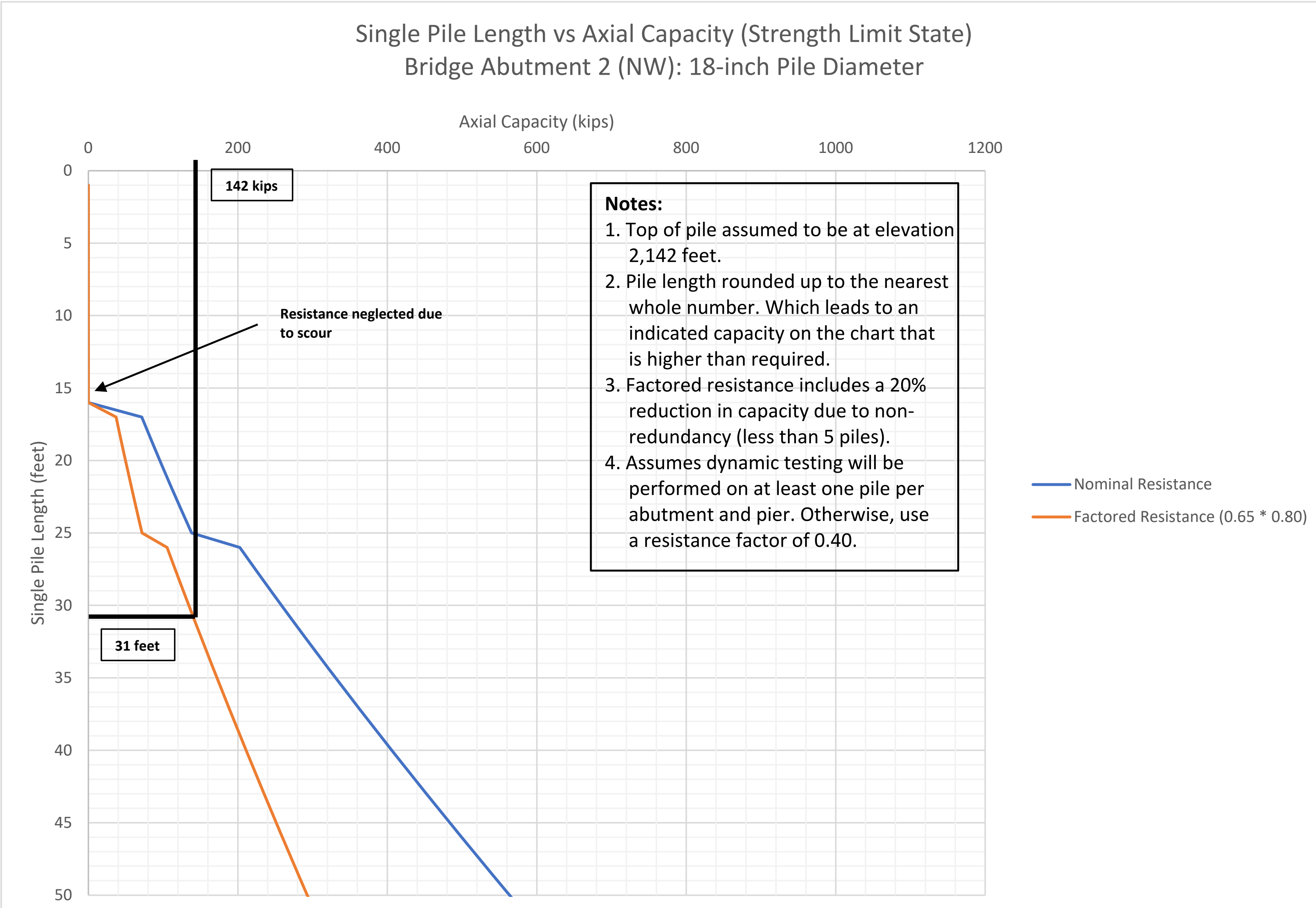
Photo 30:- Existing low water crossing at Whitetail Creek

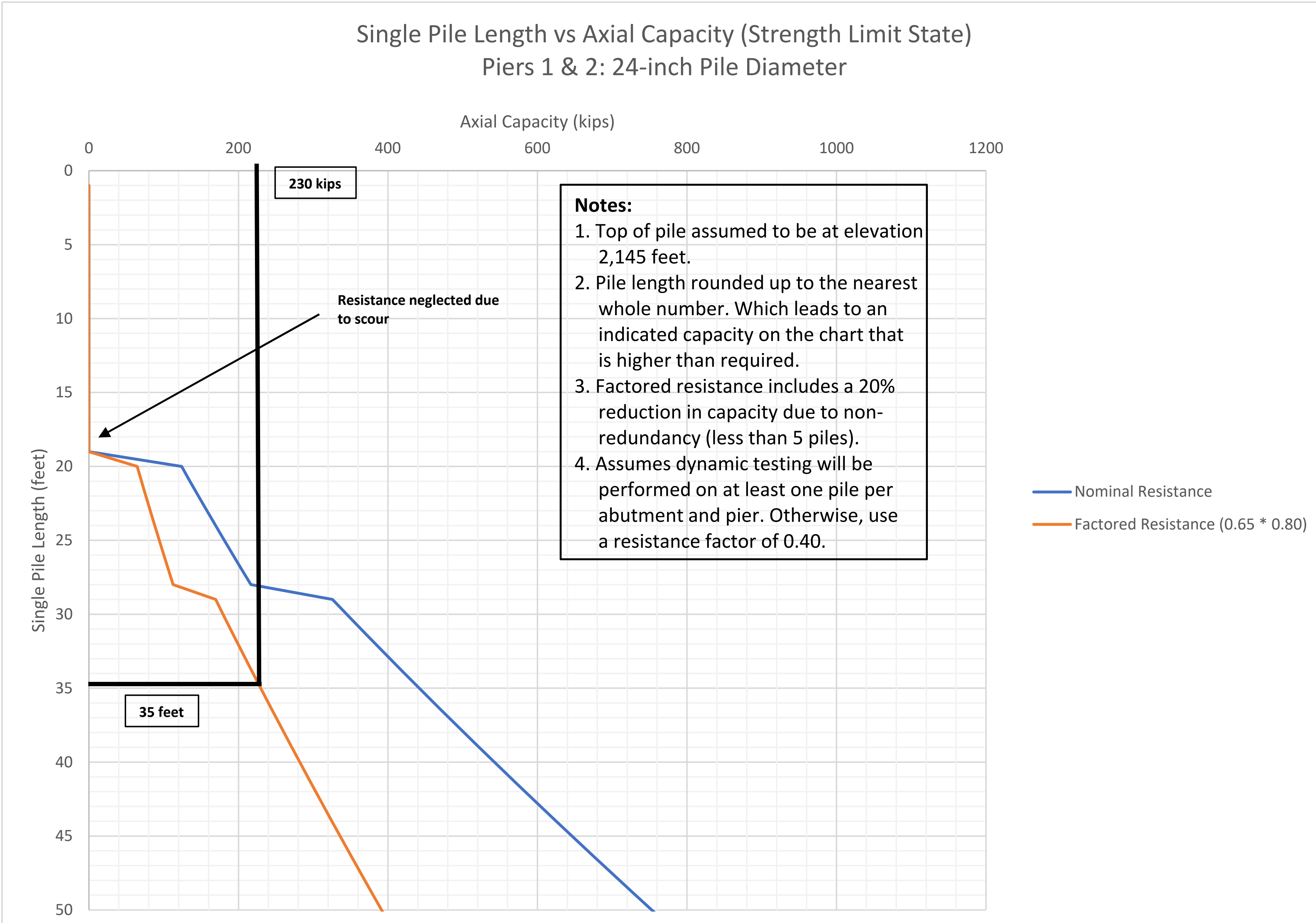
Photo 31: Whitetail Creek facing south

APPENDIX D

PILE STATIC LOAD ANALYSIS


Single Pile Length vs Axial Capacity (Strength Limit State)
Bridge Abutment 1 (SE): 18-inch Pile Diameter
Axial Capacity (kips)
0
200
400
600
800
1000
1200
Single Pile Length (feet)
0
5
10
15
20
25
30
35
40
45
50
146 kips
Resistance neglected due to scour
29 feet
Notes:
1. Top of pile assumed to be at elevation 2,142 feet.
2. Pile length rounded up to the nearest whole number. Which leads to an indicated capacity on the chart that is higher than required.
3. Factored resistance includes a 20% reduction in capacity due to non-redundancy (less than 5 piles).
4. Assumes dynamic testing will be performed on at least one pile per abutment and pier. Otherwise, use a resistance factor of 0.40.
Nominal Resistance
Factored Resistance (0.65 * 0.80)


Single Pile Length vs Axial Capacity (Strength Limit State)
Bridge Abutment 2 (NW): 18-inch Pile Diameter
Axial Capacity (kips)
0
200
400
600
800
1000
1200
Single Pile Length (feet)
0
5
10
15
20
25
30
35
40
45
50
142 kips
Resistance neglected due to scour
31 feet
Notes:
1. Top of pile assumed to be at elevation 2,142 feet.
2. Pile length rounded up to the nearest whole number. Which leads to an indicated capacity on the chart that is higher than required.
3. Factored resistance includes a 20% reduction in capacity due to non-redundancy (less than 5 piles).
4. Assumes dynamic testing will be performed on at least one pile per abutment and pier. Otherwise, use a resistance factor of 0.40.
Nominal Resistance
Factored Resistance (0.65 * 0.80)


Single Pile Length vs Axial Capacity (Strength Limit State)
Piers 1 & 2: 24-inch Pile Diameter
Axial Capacity (kips)
0
200
400
600
800
1000
1200
Single Pile Length (feet)
0
5
10
15
20
25
30
35
40
45
50
230 kips
35 feet
Resistance neglected due to scour
Notes:
1. Top of pile assumed to be at elevation 2,145 feet.
2. Pile length rounded up to the nearest whole number. Which leads to an indicated capacity on the chart that is higher than required.
3. Factored resistance includes a 20% reduction in capacity due to non-redundancy (less than 5 piles).
4. Assumes dynamic testing will be performed on at least one pile per abutment and pier. Otherwise, use a resistance factor of 0.40.
Nominal Resistance
Factored Resistance (0.65 * 0.80)